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Title: A MODEL FOR SIMULATING RIVER AND RESERVOIR TEMPERATURES WITH APPLICATIONS FOR ANADROMOUS FISH MANAGEMENT

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Abstract approved: ______________________
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A problem confronting the Corps of Engineers and other water resources planners in the Pacific Northwest is the prediction of anadromous fish enhancement benefits that would result from a proposed high dam project. These benefits are expected from augmenting natural streamflows with relatively cold reservoir withdrawals. The resulting increased streamflows and decreased river temperatures downstream are expected to sustain larger salmon populations than would occur without an impoundment.

The objectives of this study are to develop a computer simulation model for continuously predicting reservoir and downstream temperatures and mass flows for a single high dam system; and to demonstrate how the model could be used in an actual planning situation to provide information on anadromous fish production.

The reservoir submodel constructed for this study is based upon the stratified reservoir model developed at M. I. T. A downstream bulk flow river temperature and routing algorithm, solar flux submodel, and several decision submodels are constructed and incorporated into this system. The model permits determination of all values of mass flows, depths, and water temperatures at any designated point in time for specified points along a river, and for specified reservoir elevations. The model structure allows decision routines to be called at each time interval update for determining reservoir withdrawal, the amount of withdrawal from each of three specified reservoir outlets, and the channel withdrawal for irrigation.

This model is applied to the proposed 145,000 acre-feet reservoir on the Calapooia River at Holley, Oregon, to investigate the expected trade-offs and product mix between anadromous fish (a function of water temperature and streamflow) and reservoir-based recreation (a function of reservoir elevation).

It was found that the river temperatures downstream from the proposed reservoir could not be maintained within the optimum range for Pacific salmon. Further, it was found that the temperatures of the reservoir withdrawals would have a negligible effect upon river temperatures beyond 24 miles downstream from the dam site.

An economic analysis suggests that the level of anadromous fish enhancement and recreation benefits predicted by the Corps of

Engineers could not be achieved by constructing the proposed Holley project.

The model developed in this study is structured so that any water quality parameter, not just temperature, could be simulated with the downstream routing algorithm. The model can also be used to evaluate the operational efficiency of existing multiple objective water projects. In addition, the model structure would make it relatively easy to build additional submodels into the algorithm. For example, the inclusion of a flood control or dynamic fish population submodel would depict a more realistic situation as well as broaden the economic analysis.

The computer model structure requires the solution sequence to pass through all relevant subprograms at each time interval update. Thus, calculations covering only one internal time period are made on each pass of a subprogram. This model structure allows for considerable operating flexibility. For example, because all hydrometeorological data are read from subprograms, the model can be operated using either historical or stochastically generated data.

A Model for Simulating River and Reservoir Temperatures with Applications for Anadromous Fish Management

by

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A MODEL FOR SIMULATING RIVER AND RESERVOIR TEMPERATURES WITH APPLICATIONS FOR ANADROMOUS FISH MANAGEMENT

I. INTRODUCTION

Problem

For years, considerable controversy has surrounded the accounting procedure for determining the benefits and costs that might accrue to the nation from any specific water resources project. Although the general principles and standards for water resources planning have recently been altered (Water Resources Council, 1973), the means and techniques for adhering to the general procedure are left to the discretion of the planning agencies. In many instances, it has been necessary for these agencies to rely on simplified assumptions and "guesstimates" because analytical models which might provide more rigorous engineering and economic analyses were non-existent. The advent of the high speed digital computer has provided the means for analyzing a complex system such as a river basin. However, the impact of computer modeling has been felt mainly in the research area rather than at the operational level of policy and decision making, and the debate over the accurate accounting for benefits and costs by planning agencies remains.

An evaluation of a specific water resources project should, ideally, account for all associated benefits and costs. The planning agencies attempt to do this by analyzing specifically designated project purposes. These purposes may fall into such categories as flood control, hydropower, irrigation, municipal and industrial water supply, recreation, navigation, and fish and wildlife enhancement. It is this last project purpose, fish enhancement--specifically, anadromous fish enhancement--that inspired this research.

Fish enhancement benefits are dependent upon the expected annual increase in fish production and the associated prices for fish. The fish benefit function may be stated as

$$B_{tj} = f(\Delta POP_{tj}) = P_j \times \Delta POP_{tj} \qquad (1\text{-}1)$$

where

B = gross fish enhancement benefits

P = total value per fish

j = particular species of fish

t = time period

ΔPOP = change in fish production due to project.

The present value of gross fish enhancement benefits (PVGB) [1] is

[1] The present value for any other project benefit function may be found using the same procedure.

$$PVGB = \sum_{t=1}^{n} \frac{B_{tj}}{(1+i)^t} = \sum_{t=1}^{n} \frac{(P_j \times \Delta POP_{tj})}{(1+i)^t} \qquad (1\text{-}2)$$

where

i = discount rate

n = project life.

It should be clear from (1-2) that the fish population may vary by species and through time. If the total value (price) per fish in (1-2) is assumed constant and known, the problem facing the planning agency is reduced to that of providing continuous predictions of fish populations over time.

An increase in fish production may arise due to increased water quality and quantity during low flow periods in the river below a dam. An impoundment may damage an upstream salmonid fishery by eliminating spawning within the impounded area and blocking upstream migration. However, for the benefits to be positive, the downstream increase in fish benefits would have to more than compensate for the upstream loss.

The improvement in water quality during low flow periods is due primarily to a decrease in water temperature and/or an increase in the dissolved oxygen content of the water. Dissolved oxygen saturation, or the ability of a body of water to hold oxygen, is a direct function of the water temperature. Thus, for a river with a relatively low biochemical oxygen demand (BOD) load, the dissolved oxygen (DO) may approach saturation. In such a case, water temperature would become the critical water quality determinant in the production of anadromous fish.[2]

In general, the determination of expected fish benefits from a proposed project is multi-disciplinary, involving the fishery biologist, the engineer, and the economist. In the Pacific Northwest, fishery biologists are conducting research related to the effects of various water temperatures on Pacific salmon. Their work has centered primarily on determining optimum and "lethal" temperatures for the species under consideration.

Within broad ranges, fisheries agencies in the Pacific Northwest have recommended optimum temperatures conducive to the production of Pacific salmon in the Columbia River Basin. As outlined in the Columbia River Thermal Effects Study (Environmental Protection Agency (EPA), 1971), these ranges are:

Migration routes:	45 to 60°F (7.2 to 15.6°C);
Spawning areas:	45 to 55°F (7.2 to 12.8°C);
Rearing areas:	50 to 60°F (10.0 to 15.6°C).

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[2] This assumes that nitrogen super-saturation, which usually is not a problem in a single high dam system without hydropower, is not a factor.

Laboratory experiments have demonstrated that water temperatures at 69.8°F (21°C) and above are directly lethal to more than one-half the adult salmon and steelhead exposed to that level; that is, temperature alone would kill the fish at that level (EPA, 1971).

Experimentation from which the temperature-survival relationships are being derived requires considerable time and other resources, because the biological phenomena involved necessitate a costly experimental approach to determination of the temperature effects on each species at a given stage of development and under numerous water quality conditions.

In order to utilize the temperature-survival relationships made available by the biologist, the water temperature in the reservoir and at selected sites downstream must be available, preferably at the same point in time and on a continuous basis. If a dam has been constructed and is in operation, these downstream temperatures could be physically monitored, although to do so might be relatively costly. If the project of interest is yet to be constructed, physically monitoring the temperatures becomes an impossibility. Therefore, there must be a way to continuously predict water temperatures in the proposed reservoir and downstream. This becomes the responsibility of the engineer. He may utilize hydraulic and heat transfer fundamentals in a computerized model to determine the reservoir release temperature and flows, and to determine the water temperature at selected sites along the river while routing the flows downstream.

Information on reservoir and downstream water temperatures, in addition to data on temperature-survival and temperature-growth relationships, may be used as inputs in a fish production model. The economist may participate in this phase of the planning process by helping determine the most efficient means of fish production while considering utilization of this water for other purposes. He would also be expected to estimate future demand for the fish and the prices to be used in determining the fish enhancement benefits.

These benefits are potentially critical factors in determining not only the scale of a proposed project, but whether or not the impoundment is constructed at all. The proposed Holley Dam on the Calapooia River in Oregon provides a case study for this research.

Holley Project

Project History

In 1950, Congress authorized the construction of a 97,000 acre-feet (AF) reservoir on the Calapooia River at Holley, Oregon (Figure 1). The planning for dam and channel work was discontinued in 1958 due to lack of the required local participation. The following year, a review study of the Calapooia was initiated by the U.S. Army Corps of Engineers (Corps) under authorization of the Senate

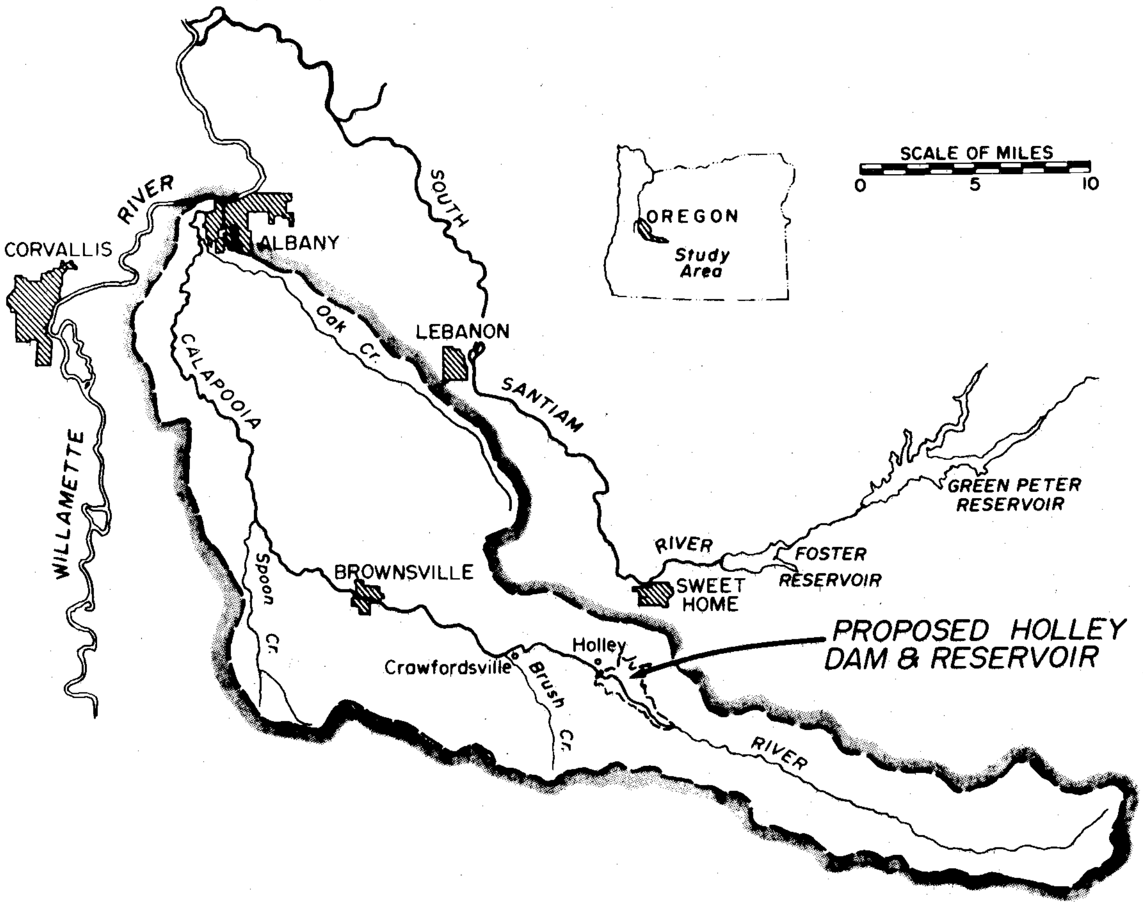

Figure 1. Calapooia River basin.

Committee on Public Works. That study provided additional information on the modified plan for a larger reservoir of 145,000 AF. The most recent formal proposal on the 145,000 AF reservoir was in 1970 (U.S. Army Corps of Engineers, 1970). The project status is uncertain at this writing, due to recent increases in the discount rate and revisions in the general planning procedure (Water Resources Council, 1973). The potential flood benefits are currently undergoing reevaluation and no new additional information on the proposed project is expected until Fiscal Year 1975.[3]

Benefits and Costs

The most recent comparison of separable costs to benefits for the Holley project is shown in Table 1. Figures shown in the table are based on two assumptions: (1) a project life of 100 years, and (2) a discount rate of 4 and 7/8 percent.

The separable benefit-cost ratios for fish enhancement and recreation are both 1.1. A slight decrease in expected benefits, or a small increase in costs, would make these two purposes appear unfavorable.

[3] Information received in personal correspondence with Ken Johnson, Planner, Portland District, U.S. Army Corps of Engineers, December 1973.

Table 1. Separable cost-benefit comparison in thousands of dollars--145,000 AF Holley project.

Purpose	Annual Separable Cost	Expected Annual Benefit	Separable Benefit-Cost Ratio
Flood control	232	1,606	6.9
Recreation	370	419	1.1
Fish enhancement	236	268	1.1
Irrigation	28	60	2.1
M & I water	0	43	+
Navigation	0	11	+
Total	866	2,407	2.8

Source: U.S. Army Corps of Engineers (1970), p. 45.

Figure 2 graphically compares the total annual benefit and cost curves associated with different-sized structures proposed for the Holley site. The unusual shape of the benefit curve is due to negative fish benefits expected to occur with a project size of less than 138,000 AF.

Table 2 compares the 145,000 AF project, which, according to the Corps, would maximize expected net benefits, with the originally proposed 97,000 AF project.

Table 2. Benefit-cost comparisons of 97,000 AF and 145,000 AF projects at Holley dam site.

Storage Acre-Feet (AF)	Expected Net Annual Benefits	Benefit-Cost Ratio
97,000	$506,000	1.365
145,000	$510,000	1.269

Source: U.S. Army Corps of Engineers (1970).

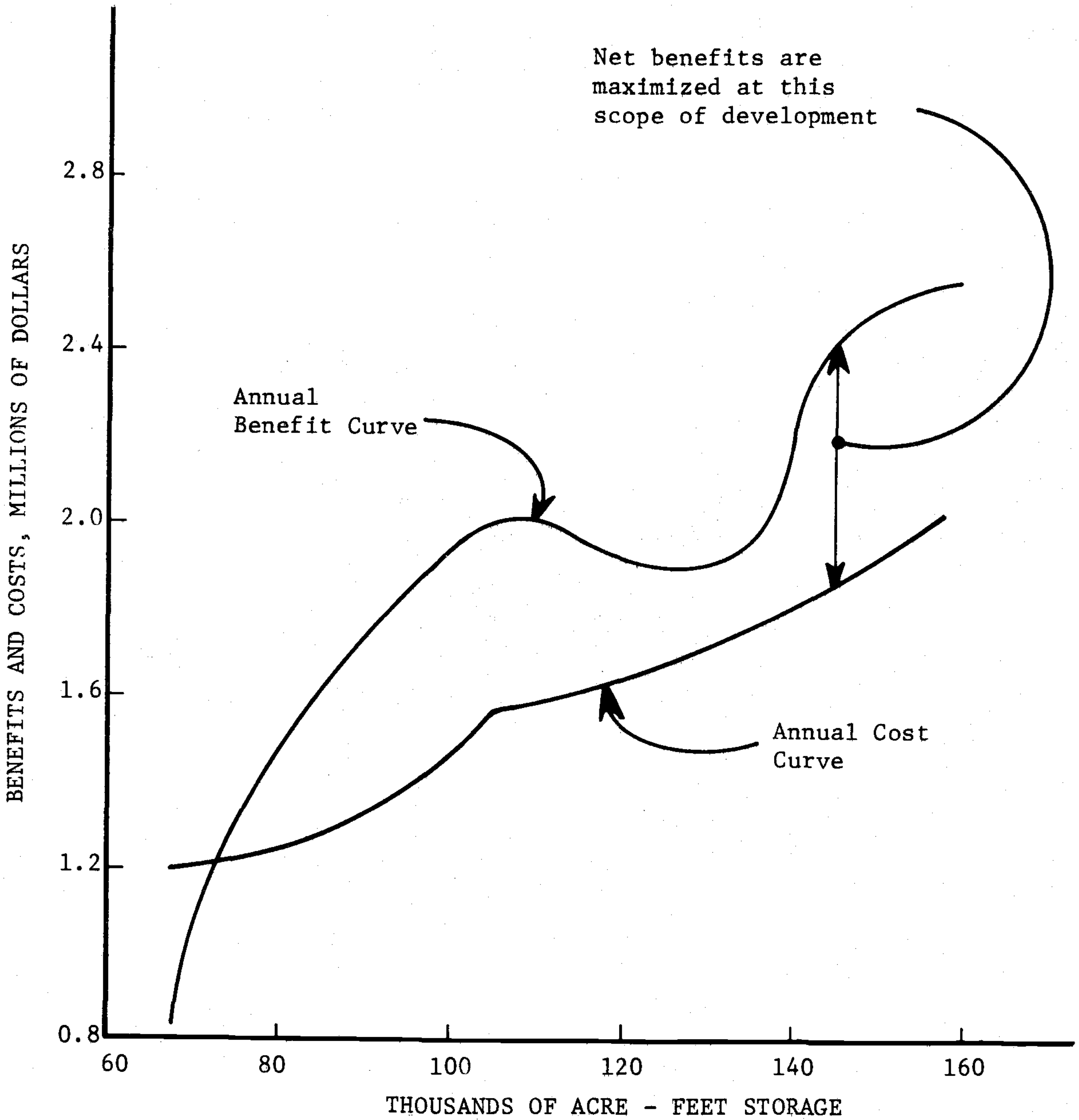

Figure 2. Holley Project Net Benefit Maximization

Source: U. S. Army Corps of Engineers (1970), p.44.

The difference in annual net benefits for the proposed project with fish enhancement, as opposed to that with no fish enhancement, is predicted to be only $4,000. Because this difference is so small, the scale of the project is particularly sensitive to the procedure used in evaluating project benefits.

Procedure Used in Estimating Fish Benefits

The estimated annual salmon production and resulting commercial and sport harvests were provided to the Corps by the Bureau of Sport Fisheries and Wildlife,[4] in cooperation with the Oregon State Game Commission,[5] the Fish Commission of Oregon (FCO), and the National Marine Fisheries Service (NMFS). Prices used in the Corps' evaluation of the sport fisheries were based on criteria contained in Supplement No. 1 to Senate Document No. 97. The National Marine Fisheries Service provided the prices used in evaluating the commercial harvest.

The annual value of expected fish benefits for the Holley project is based on the assumption of ideal, or optimum, water quantity and

[4] U.S. Bureau of Sport Fisheries and Wildlife, 1971. A letter to the District Engineer, Portland District, Corps of Engineers. Preliminary draft of proposed report.

[5] Oregon State Game Commission is now called Oregon State Wildlife Commission.

quality conditions existing within, and downstream from, the proposed reservoir over the life of the project.[6] If less than optimum conditions were to prevail at different times throughout the life of the project, the annual expected fish benefits would subsequently be reduced.

Optimum Flows and Temperatures

The optimum levels of channel flows in the Calapooia range from a maximum of bankfull conditions of 3500 cubic feet per second (cfs) (Avey, 1972) to a minimum of 100 cfs to 160 cfs, depending on the river reach.[7] These minimums were recommended to the Corps by the fisheries agencies mentioned above.

However, the optimum temperatures recommended were in terms of the release temperatures from the proposed reservoir, not the optimum river temperatures. The release temperatures initially recommended ranged from 55 to 60° F. As noted earlier, these temperatures are generally considered to fall within the optimum river temperature range for Pacific salmon. The Corps initially determined that the release water temperature criteria could always be met.

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[6] U.S. Bureau of Sport Fisheries and Wildlife, 1971. A letter to the District Engineer, Portland District, Corps of Engineers. Preliminary draft of proposed report, p. 16.

[7] Ibid., p. 14.

The responsibility for providing information on water quality aspects of proposed projects, however, is that of the Environmental Protection Agency (EPA). The forerunner of EPA, the Federal Water Quality Administration (FWQA), issued a report and partial analysis of water quality conditions that may result from the proposed Holley project (1969). The FWQA analysis suggests that neither the agencies providing initial information on release temperatures nor the Corps of Engineers analytically addressed the problem of the potential increase in river water temperature after release from the proposed Holley impoundment. If the recommendations made to the Corps were to be used operationally, river temperatures downstream from the proposed project might, at times, approach the lethal temperature limit. The result could be mortalities and losses in fish enhancement benefits.

Research Direction and Objectives

If the water temperature could be predicted continuously at different elevations in the proposed reservoir, and at selected sites downstream, these predictions could be used in conjunction with temperature-survival relationships as one set of inputs used to model the salmon population. Avey (1972) has modeled the salmon population associated with the proposed Holley project. However, the survival rates generated by Avey's model are functions only of the amount of streamflow, the reservoir stocking rates for salmon, and the stage of development of the specific species. If water temperatures appeared to vary sufficiently to be a critical factor in the production of salmon in the river basin, then a continuous water temperature model could be considered a valid addition to the basic framework of Avey's model. The salmon population over the life of the proposed project could then be modeled and an estimate of fish enhancement benefits would be available. Unfortunately, there cannot be found in the literature a general simulation model for reservoir and streamflow thermal conditions providing information on a continuous basis. However, this type of model is necessary to proceed in the direction outlined above. Therefore, the objectives of this research are:

1. To develop a general model that will operate on a continuous basis while solving for water temperatures at specified elevations in a stratified reservoir and at specified sites downstream in any given time period.
2. To demonstrate how the model could be used to (a) predict water temperatures in the Calapooia River basin, and (b) provide a framework for analyzing the economic feasibility of a portion of the Holley project.

Research Outline

Chapter II describes the rationale behind the water temperature simulation model and presents a description of the basic computer algorithm. Chapter III presents the general equations of the model and then reduces them to the simple situation of an isothermal reservoir. This is done primarily for clarity and to introduce the meteorological parameters used in the prediction of water temperatures. Chapter IV expands the isothermal model to one in which vertical temperature variations are present in the reservoir. The specific operational procedure used to predict reservoir temperature profiles is also presented.

Chapter V describes alterations to the basic heat transfer equations by considering lateral temperature variations in a river. The basic routing algorithm, which allows for advected flows, is examined in detail.

Chapter VI demonstrates the interactive roles of the model components by displaying the results of simulating reservoir and river water temperatures under different management strategies for given sets of meteorological and hydrologic conditions for the proposed Holley project.

An economic framework is presented in Chapter VII for investigating the economic feasibility of low flow project purposes.

Chapter VIII includes the summary, conclusions, and general relevance of this research effort.

II. DESCRIPTION OF THE WATER TEMPERATURE MODEL

Introduction

The general framework of the water temperature model will be developed while attempting to describe a realistic process of surface water allocation in a river basin having one reservoir. The process is one which requires the reservoir operator to make decisions over time regarding the withdrawal rates from the reservoir, and, hence, the allocation of the resource to different project purposes. Similar decisions must also be made on the allocation of water from the river below the dam. These allocation decisions may have an effect on the water quality which, in turn, may affect the time stream of benefits from the different project purposes. Thus, at any point in time, the reservoir and downstream water quality could be an important factor in the decision calculus.

Although there are always existing rules and institutional constraints for the allocation of surface water, it is reasoned that timely water quality forecasts to the decision makers could provide one method for achieving a more efficient allocation of the resource. No attempt will be made to analyze the political and social structure responsible for any set of allocation rules; rather, the model developed in this study will provide a base for testing withdrawal and allocation strategies which might lead to a new set of operating rules within a river basin.

Model Conception

In general terms, the conceptual model appears quite simplistic, resembling a standard inventory model in many respects. The objective is to account for all mass (water) and associated water temperatures moving through the system. This accounting would provide decision makers with information at any point in time for all points in space within the system.

This is depicted in Figure 3, which shows the reservoir, parcels of water moving downstream (flags attached), river stations (circled numbers), and river reaches between the stations. The model continuously monitors the temperatures at different elevations in the reservoir, as well as the mass flows (Q), average water speeds (U), average reach depths (D), and water temperatures (T) at the river stations. This information would be fed into a central unit where it is to be used by a decision maker in choosing reservoir withdrawal rates and river water allocations.

Referencing Time Units

In Figure 3, the times recorded at the central unit may not have the same frame of reference, and thus require an explanation. The

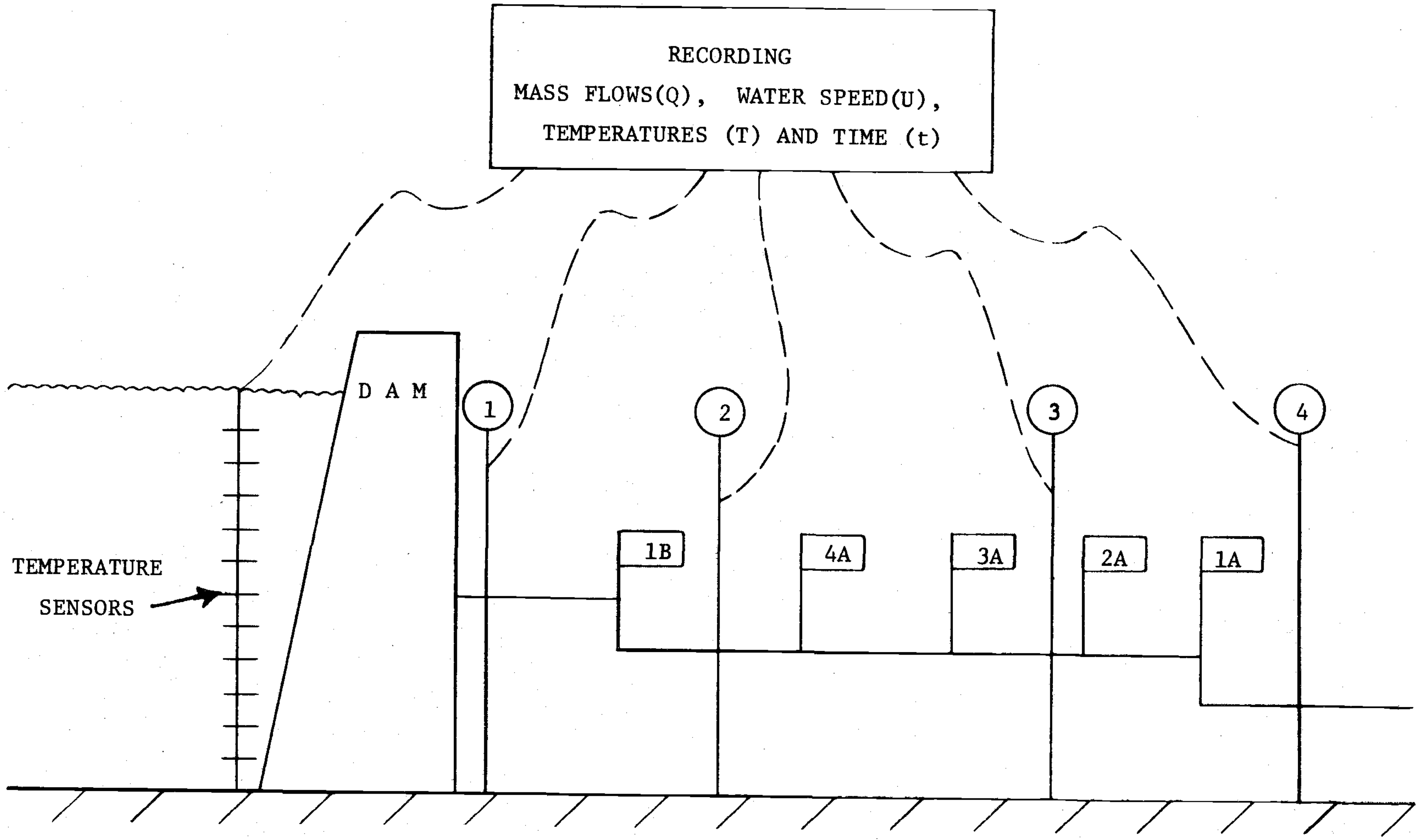

Figure 3. Continuous Monitoring of Physical Parameters

three different times, or time increments, internal to the model computations are presented below.

Δt_r = the time length between reservoir temperature incrementation and, likewise, changes in reservoir release rates. For this study, Δt_r is always equal to one day.

t_v = travel time; the amount of time for a flagged parcel to traverse a given river reach = f(water speed, distance).

Δt = time length between (1) river water temperature calculations at the river stations and (2) when the water parcels are flagged at the base of the dam. The model structure prohibits Δt_r from being less than Δt, and Δt must divide evenly into Δt_r.

The following is an example of the time counting system with reference to Figure 3. Assume that

Δt = 6 hours

Δt_r = 1 day, with updating at 2400 hours.

Given these assumptions, flag 1A was at station one at midnight (2400 hours). Also at station one, in succession, were flag 2A at 0600 hours, flag 3A at 1200 hours, and flag 4A at 1800 hours. Twenty-four hours after flag 1A appeared at the base of the dam, the reservoir temperatures were updated and a decision was made on the rates and elevations of reservoir withdrawal. In this case, the total withdrawal was increased, as indicated, by the increase in depth of the parcel of water associated with flag 1B.

The flows and temperatures associated with each flag are, of course, known at station one, and can be followed downstream while updating these values every six hours (Δt). However, the travel time (t_v) for a flag to move from one station to the next, would only by pure coincidence, be equal to Δt. Therefore, the problem of being able to read (by computer) the water temperatures and mass flows at each station every six hours (Δt) becomes more complex than it may have first appeared. It is imperative, however, that the model demonstrate this ability in order to utilize the information for making decisions affecting resource allocations in the immediate future time period.

Description of the Basic Computer Model

The main computer program, ROUTR, which embodies the elements of the conceptual model, is shown as a flow chart in Figure 4. Flow charts of the subprograms comprising ROUTR are presented in subsequent chapters. The program begins at statement one, where the constants are read and initialized. The value for the first year of the computer run is set equal to one, and, at statement two, the value for the first day of the first year is set equal to the number of that

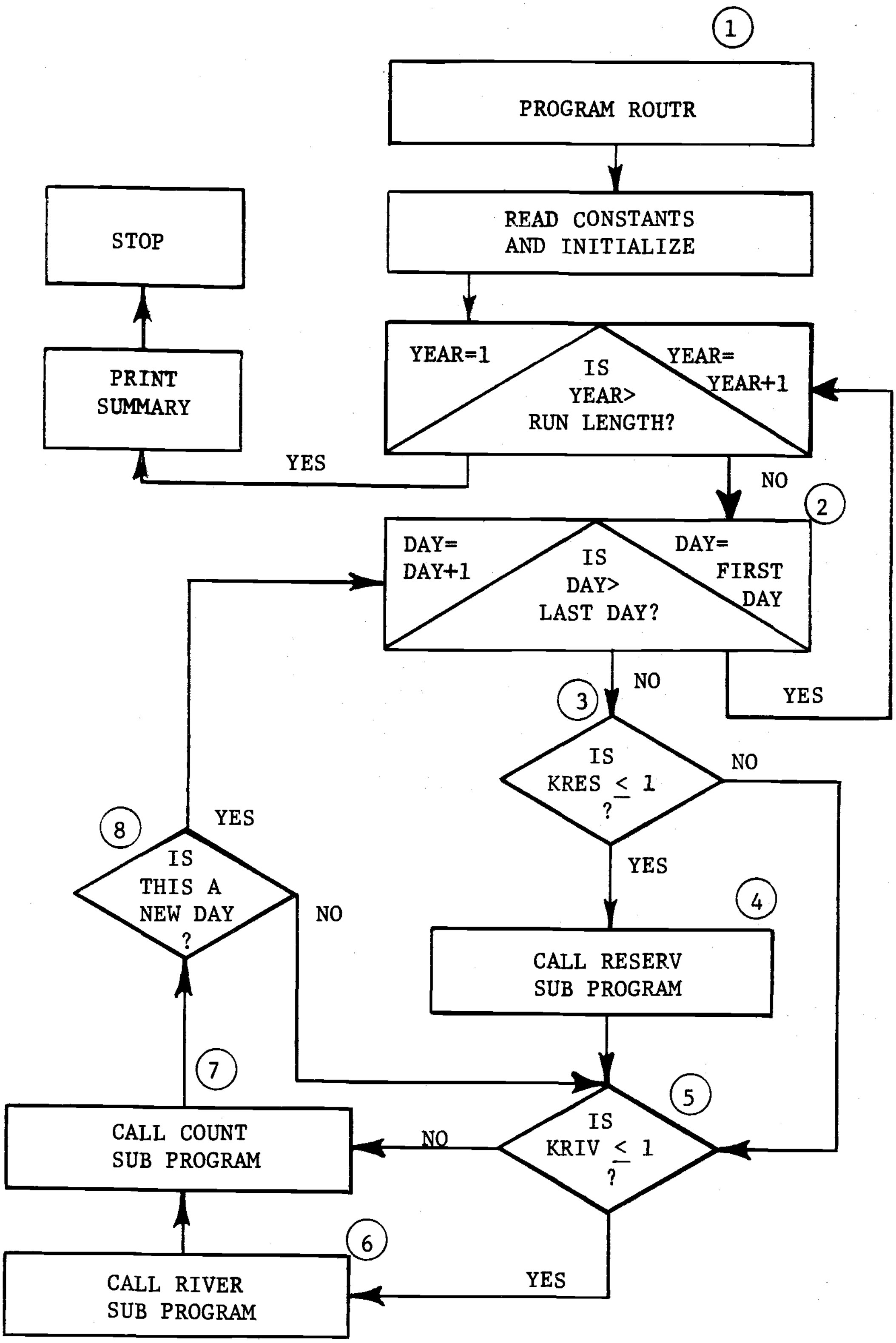

Figure 4. Flow Diagram of Main Computer Program, ROUTR

day within the year. For example, April 15 was chosen as the starting day on all computer runs in this study, and was subsequently assigned a value of 105, as April 15 is the 105th day of the year. Statement three is an integer switch which allows the program to bypass the reservoir submodel at statement four if the value of the integer identifier, KRES, is greater than one. Statement five is another switch which determines if the downstream submodel, RIVER (statement six), should be bypassed. The values of KRES and another identifier, KRIV, are specified initially; only the value of KRIV may be altered during a computer run. The subprogram COUNT (statement seven) is used for the internal counting of time and space, and for dumping extraneous data to avoid computer storage problems. This routine accounts for the intra-day time incrementation, the Δt discussed earlier, as well as for the day-to-day counting.

Statement eight is another switch which uses information from statement seven to determine if the day has changed. If the day has changed, statement two is again encountered, with the possibility of going back through statement three to statement four where a new day's reservoir surface level and water temperature profile will be calculated. If the day has not changed, control shifts from statement eight to the switch at statement five, and at statement six the river water temperature and flow rates are updated at each river station, for this particular time period. Desired information may be printed at the end of each time period and each day, at specified time periods and daily intervals, or a shortened summary may be printed at the end of the computer run.

The model structure allows the experimenter to insert an assortment of decision rules and functions into separate submodels which could use prior, present, and future information for allocating the water. For example, a submodel internal to statement four is used to choose the elevations and amounts of reservoir withdrawals. Another submodel, internal to statement six, is used to determine river water withdrawals at any river station for a particular time period, using up-to-the-minute information. Thus, as suggested above, the model structure will allow representation of authentic water resource allocation problems by not requiring the specification of actual allocation levels in the data input to the computer model.

The basic equations used in this computer model to solve for the water temperatures in a well-mixed body of water are discussed in the next chapter.

III. WATER TEMPERATURE PREDICTION

Introduction

This chapter describes the isothermal water temperature model. The governing differential equations are presented, along with the meteorological parameters necessary for solving these equations. The chapter is concluded by presenting the governing equations in a simple difference notation which lends itself to a computerized solution.

Governing Equation

The fundamental differential equation governing the distribution of heat (temperature) in a fluid is the conservation of heat equation. In vector notation (Bird, et al., 1965), this equation is

$$\frac{\partial T}{\partial t} + \bar{V} \cdot \nabla T = \nabla \cdot E \nabla T + D_m \nabla^2 T + \frac{H}{\rho c_p} \qquad (3\text{-}1)$$

where

T = temperature = f(space, time)

t = time

$\bar{V}$ = velocity vector = f(space, time)

E = turbulent diffusivity of heat = f(space, time)

D_m = molecular diffusivity of heat

H = rate of heat generation or dissipation per unit volume (energy/time-volume) = f(space, time)

ρ = fluid density = f(temperature)

c_p = fluid specific heat = f(temperature)

∇ = "del", the vector operator.

Expanding (3-1) for a two-dimensional situation yields

$$\frac{\partial T}{\partial t} + u\frac{\partial T}{\partial x} + v\frac{\partial T}{\partial y} = \frac{\partial}{\partial x}\left[(E+D_m)\frac{\partial T}{\partial x}\right] + \frac{\partial}{\partial y}\left[(E+D_m)\frac{\partial T}{\partial y}\right] + \frac{H}{\rho c_p} \quad (3\text{-}2)$$

where x and y refer to the longitudinal and vertical directions, respectively.

The model for the prediction of reservoir and downstream temperatures assumes that the water body is completely mixed and isothermal at any specified instant in time. This implies that the water temperature is not a function of the space vectors, x and y, but of time only. Under this assumption, Equation (3-2) reduces to

$$\frac{\partial T}{\partial t} = \frac{H}{\rho c_p} \quad (3\text{-}3)$$

By introducing the volume $(\forall)$ of the water body and rearranging Equation (3-3), the time rate of change of heat in the reservoir becomes

$$H = \rho c_p \forall \frac{\partial T}{\partial t} \quad (3\text{-}4)$$

where H is energy per unit time.

Heat or Energy Balance

It is necessary to examine the parameters which comprise H in Equation (3-4) in order to eventually solve the equation for the temperature change over time, $\partial T/\partial t$.

If the reservoir in Figure 5 were isothermal and well insulated, then the rate of change of heat, H, may be expressed as

$$H = f(\phi, \phi_{in}, \phi_{out}) \tag{3-5}$$

In this equation,

ϕ = net heat flux across water surface

ϕ_{in} = net heat flux from tributary inflows

ϕ_{out} = net heat flux from withdrawals.

The three right-hand terms in (3-5) will now be examined in turn.

Surface Flux

The total heat, or energy, flux across the water surface may be expressed as

$$\phi = \phi_o - \phi_L = [\phi_s + \phi_a] - [\phi_b + \phi_e \pm \phi_c] \tag{3-6}$$

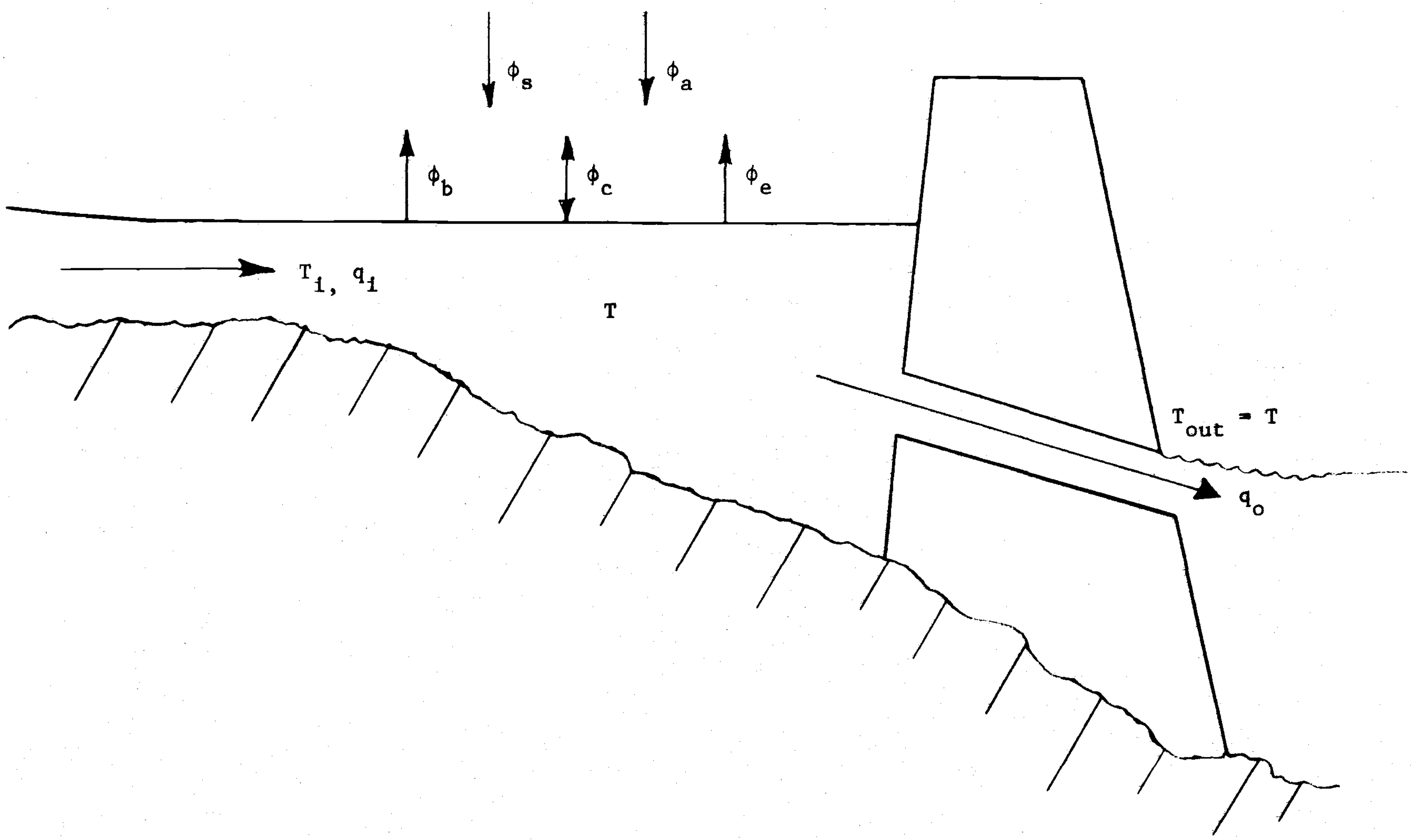

Figure 5. The Isothermal Reservoir

where

ϕ = net surface heat flux (energy/area-time)

ϕ_s = net shortwave solar radiation entering the water surface

ϕ_a = longwave atmospheric radiation absorbed by the water

ϕ_b = longwave back radiation emitted at the water surface

ϕ_e = evaporation losses at the water surface

ϕ_c = heat transfer due to conduction.

The following five meteorological parameters are used to solve a series of equations for the energy balance, ϕ, in Equation (3-6):

1. Solar radiation striking the earth's surface.
2. Atmospheric radiation.
3. Dry-bulb air temperature.
4. Relative humidity.
5. Wind speed.

It has been suggested that these parameters be measured in the field (Huber and Harleman, 1968). Historical data on solar and atmospheric radiation may not be available for the geographical area of interest, or these parameters may be relatively costly to monitor. If this is the case, estimates of these parameters may be made by solving a series of equations utilizing another meteorological parameter, cloud cover.

A short discussion on each of these energy budget parameters follows in order to clarify the particular equations used in this study. For further details, the reader may consult Huber and Harleman (1968).

Solar Radiation (ϕ_s). In many locations and at many weather stations, solar radiation is either not measured or the data are of short duration. For project planning, it is desirable to have a continuous time series of considerable duration for all the meteorological parameters used in order to investigate different situations that may arise, and for estimating probabilities derived from historical frequencies. Therefore, a subprogram, RADR, has been written, which calculates the net solar radiation and may be used at the discretion of the investigator. RADR was, in fact, used in this study because measured values of solar radiation were unavailable.

Subroutine RADR calculates hourly solar radiation, regardless of the time increment between successive observations of the input parameter, cloud cover. The calculations are for an area that is roughly halfway between the site of the proposed Holley reservoir and the last point of interest downstream, a distance of approximately 24 miles. RADR is called at the beginning of each day from either of the two submodels, RESERV or RIVER.

The following equation for generating net solar radiation incident on a horizontal surface was validated by Wunderlich as discussed by Huber and Harleman (1968).

$$\phi_s = \frac{\phi_{sc} \sin \alpha}{r^2} (a_t^m)(1-R)(1-0.65C^2) \tag{3-7}$$

where

ϕ_s = direct and diffuse solar radiation penetrating the water surface (energy/area-time)

ϕ_{sc} = solar constant (1.94 cal/cm^2-min)

α = solar altitude

r = normalized radius of the earth's orbit

a_t = atmospheric transmission coefficient

m = optical air mass

R = albedo or reflection coefficient

C = cloudiness as a fraction of sky covered.

The equations and step functions used to determine r, α, m, and R are detailed in Huber and Harleman (1968).

Longwave Atmospheric Radiation (ϕ_a). If measured values of ϕ_a are not available, this parameter may be estimated, as was done in this study, by the following expression:

$$\phi_a = 9.37(10^{-6})(\sigma)(E_p)(T_a^6)(1.0 + 0.17C^2) \tag{3-8}$$

where

ϕ_a = longwave radiation absorbed at the water surface (energy/area-time)

σ = Stephan-Boltzman Constant (1.17×10^{-6} KCAL/m^2-day-°K^4)

E_p = percent radiation absorbed by water surface (0.97)

T_a = absolute dry-bulb temperature measured two meters above the surface in °K

C = cloudiness as a fraction of the sky covered.

Longwave Back Radiation (ϕ_b). Longwave radiation emitted by the water surface is calculated by:

$$\phi_b = \varepsilon \sigma T_s^4 \tag{3-9}$$

in which

ϕ_b = longwave back radiation (energy/area-time)

ε = emissivity of water (0.97)

σ = Stephan-Boltzman Constant

T_s = absolute water surface temperature in °K.

Evaporation and Conduction Flux (ϕ_e, ϕ_c). The reservoir component of the model contains the option of using either the Rohwer or the Kohler formula, whereas only the Rohwer formula is presently available for use in the downstream component.

Field evaporation and conduction losses are difficult to predict, or even to measure. Most formulae are based on Equation (3-10), Dalton's general law of mass transfer, modified to allow for the effect of wind.

$$E_m = K(a+bw)(e_s - \psi e_a) \tag{3-10}$$

where

E_m = mass flux (mass/area-time)

a, b = empirical constants

w = wind speed

e_s = saturation vapor pressure of the water surface

e_a = saturation vapor pressure of the air at dry-bulb

ψ = relative humidity

K = exchange coefficient.

The evaporation losses are given by

$$\phi_e = E_m(L_v + c_p T_s) \tag{3-11}$$

where

ϕ_e = evaporation flux (energy/area-time)

E_m = mass flux (mass/area-time)

L_v = latent heat of vaporization (energy/mass)

c_p = specific heat (energy/mass-°C)

T_s = water surface temperature - °C.

Conduction losses may be related to the mass flux (E_m) by the Bowen Ratio, R.

$$R = N \frac{T_s - T_a}{(e_s - \psi e_a)} \tag{3-12}$$

where

N = constant

T_a = dry-bulb temperature in °C

T_s, e_s, e_a, ψ = as described above.

Finally, from the preceding,

$$\phi_c = E_m R \tag{3-13}$$

where

ϕ_c = conduction flux (energy/area-time)

E_m = mass flux (mass/area-time)

R = Bowen Ratio (energy/mass).

The Rohwer and Kohler formulae, along with equations for estimating e_s, e_a, and L_v may be found in Huber and Harleman (1968).

The vapor pressure gradient $(e_s - e_a)$ could be negative, which would lead to a negative evaporation loss. While it is known that this phenomenon occurs in nature in the formation of dew, little is known about the process as it occurs over a water surface. For this reason, and the fact that the constants appearing in Equation (3-11) are for positive evaporation rates only, it is suggested by Ryan and Harleman (1971) that all calculated negative evaporation rates be set equal to zero. However, all values of negative conduction rates, which are functions of evaporation, are retained. Therefore, the accuracy of this procedure is open to question.

It should be emphasized that the particular form of the equations and values for the empirically determined coefficients should be investigated for each physical locale where the energy budget is used to predict heat transfer.

Advected Flux

The heat flux associated with streamflows entering the reservoir may be expressed as

$$\phi_{in} = \rho c_p \sum_{j=1}^{n} q_j T_j \tag{3-14}$$

where

ϕ_{in} = total advected flux into reservoir (energy/time)

q_j = flow rate of the jth tributary to the reservoir

T_j = temperature of the jth tributary to the reservoir.

Likewise, the advected heat loss from the reservoir is

$$\phi_{out} = \rho c_p \sum_{k=1}^{m} q_k T_k \tag{3-15}$$

where

k = outlet number

m = total number of outlets

ϕ_{out} = energy/time.

For clarity, the total reservoir flux (energy/time) is restated as

$$H = A\phi + \phi_{in} - \phi_{out} \tag{3-16}$$

where

A = surface area

ϕ = surface flux (energy/area-time)

ϕ_{in} = inflow flux (energy/time)

ϕ_{out} = withdrawal flux (energy/time).

Solution of Equations

A scheme for continuously predicting water temperatures from solutions of the conservation of heat equation over time will now be presented.

Equation (3-4) may be rewritten in difference notation as

$$\frac{\Delta T}{\Delta t} = \frac{H}{\rho c_p \forall} \tag{3-17}$$

Rearranging and expanding Equation (3-17) yields

$$T_{t+1} = T_t + [\frac{H}{\rho c_p \cancel{V}}] \Delta t \tag{3-18}$$

By substituting the average reservoir depth, D, for $\cancel{V}/A$ (volume/area), and further expanding the bracketed term in Equation (3-18), the equation now reads

$$T_{t+1} = T_t + \left[\frac{\phi_o}{\rho c_p D} - \frac{\phi_L}{\rho c_p D} + \phi_{in} - \sum_{k=1}^{m} q_k T_t \right] \Delta t \tag{3-19}$$

While T_t must be available to determine T_{t+1}, T_t must also be known to determine the rate of temperature change $(T_{t+1}-T_t)/\Delta t$. This is because the surface losses (ϕ_L) and outflow losses (ϕ_{out}) are functions of T_t, as was shown above. Thus, for successively iterating (3-19) in order to determine sequential reservoir temperatures through time, the initial reservoir temperature must be specified.

Equation (3-19) indicates that, for a thermally well-mixed reservoir, the outflow temperature is the same as the reservoir temperature, T_t. If the reservoir were density-stratified, thus exhibiting vertical temperature differentials, the outflow temperature would not necessarily be the average reservoir temperature, and the simple form of the conservation of heat Equation (3-19) should not be used to predict the reservoir temperature. The reasons for this will become apparent in the following chapter, which details the reservoir temperature prediction component that was used in this study.

IV. TEMPERATURE PREDICTION IN STRATIFIED RESERVOIRS

Introduction

The equations relevant to the isothermal model are expanded in this chapter to include vertical temperature variations within a reservoir. The theoretical structure of a reservoir temperature model developed at the Massachusetts Institute of Technology (MIT) and used in this study is also discussed. Finally, a detailed flow chart of the reservoir computer submodel actually used is presented, with references to the specific modifications of the MIT model that were necessary for incorporation into the general water temperature model.

Stratification in a Reservoir

The temperature distribution within an impoundment is a function of the hydrologic and climatic conditions, the physical characteristics of the reservoir, and the management of withdrawals. During the winter months, relatively deep reservoirs with a large ratio of storage to stream inflow will be nearly isothermal. During the summer, however, these reservoirs tend to become density stratified. Density stratification is characterized by a decrease in temperature and an increase in density with depth.

As this stratification process continues, the water body may become defined by three major strata: the epilimnion, the metalimnion or thermocline, and the hypolimnion. The epilimnion is the region nearest the surface. It is characterized by nearly uniform temperatures and is constantly influenced by wind mixing and advection of solar radiation. Below the epilimnion is the metalimnion, the region of greatest temperature gradient, and, hence, greatest stability. The lower stratum, the hypolimnion, contains the coldest and often most turbid water.

Reservoir Temperature Prediction Submodel

Considerable progress has been made in predicting the temperatures and resulting thermal stratification of deep reservoirs by computer simulation (Beard and Willey, 1970; Huber, et al., 1972; Orlob, et al., 1969). Although the models developed by Beard and Willey (1970) and Orlob (1969) describe continuous temperature distributions, the algorithms depend upon assumed values of vertical turbulent diffusivities which are time and depth dependent while varying from one location to another. The other model (Huber, et al., 1972) developed at MIT determines the temperature distribution in stratified reservoirs while including the effects of heat sources and sinks at the boundaries, internal absorption of solar radiation, and heat transport by advection and diffusion.

The method is of a predictive nature, in that it is not necessary for certain parameters to be determined by making prior temperature measurements in the reservoir of interest. Rather, the required parameters are those that could be determined even if the reservoir did not exist, thus making the model a useful tool for prediction. Because of its predictive nature, the MIT model was chosen as the basis of the reservoir component in this study.

Deep Reservoir Criteria

The reservoir models referenced above are only applicable to so-called "deep" reservoirs. The deep reservoirs generally exhibit certain unique characteristics, notably the presence of horizontal isotherms and, hence, temperature and density stratification.

If a hypothetical reservoir were still in the planning stage, it could be difficult to determine if the deep reservoir model would apply to this particular impoundment. Fortunately, researchers have developed certain criteria for classifying reservoirs as "deep", "weakly stratified", or "completely well mixed". These criteria are given by the densimetric Froude number[8] (Orlob, et al., 1969) and

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[8] The densimetric Froude number may be defined as $F = 320 \frac{LQ}{DV}$

where L = reservoir length in meters
D = mean reservoir depth in meters
Q = average annual discharge in cubic meters per second
V = reservoir volume in cubic meters.

the average discharge to reservoir volume ratio (Huber and Harleman, 1968).

Table 3 indicates that the proposed Holley project on the Calapooia River is classified as a deep reservoir according to both criteria.

Table 3. Comparison of densimetric Froude number and discharge-volume ratio for proposed Holley project with suggested deep reservoir criteria.

	Holley Project	Deep Reservoir
Densimetric Froude number	0.004	<0.3183
Discharge-volume ratio	2.34 years^{-1}	<8.0 years^{-1}

Theoretical Basis and Assumptions

Equation (4-1), which is derived from (3-2), gives the general form of the conservation of heat equation to be used in solving for reservoir temperatures when only vertical spatial variations occur.

$$\frac{\partial T}{\partial t} + v\frac{\partial T}{\partial y} = \frac{\partial}{\partial y}\left[(E+D_m)\frac{\partial T}{\partial y}\right] + \frac{H}{\rho c_p} \qquad (4\text{-}1)$$

The reservoir is assumed to contain many horizontal isothermal strata of thickness ΔY. The average vertical velocity across a horizontal section is v, and H in (4-1) includes all heat sources and sinks along vertical boundaries, such as those due to inflows and outflows.

Internal Solar Radiation Absorption

It is assumed that net incoming solar radiation, ϕ_s, may be separated into a fraction, β, absorbed at the surface, and absorbed internally $(1-\beta)$. The solar radiation, ϕ_w, at any depth y is thus taken as

$$\phi_w = (1-\beta)\phi_s e^{-\eta y} \tag{4-3}$$

in which η = the extinction coefficient for solar radiation in water.

Inflow Velocity Distributions

When an inflowing river enters a reservoir, mixing occurs as the river engages the reservoir surface water. The temperature of the incoming water will then be an average of the river inflow temperature and the temperature of the water mixed with the inflow. This is simulated in the computer model by uniformly withdrawing water from over a specified depth at the surface and mixing this with the inflow.

This mixed inflow is assumed to enter the reservoir at the elevation where the density of the reservoir water is equal to that of the inflow. The inflow velocity distribution is then centered about this elevation.

The fact that warm inflows flow directly over the surface, and dense inflows (whether cold or sediment-laden) flow along the bottom is accepted in the literature (Ryan and Harleman, 1971). However, information pertaining to the behavior of flows at intermediate depths is sparse. Dye tests performed in Fontana Reservoir, North Carolina, indicated that the inflow velocity profiles could be approximated by a Gaussian distribution (Ryan and Harleman, 1971).

Little is known about the variance of the inflow velocity distribution. Ryan and Harleman (1971) used a constant value of 16.0. The same value is used in this study.

Outflow Velocity Distributions

The outflow velocity distributions are assumed to be Gaussian. However, there is considerably more analytical and experimental evidence behind this assumption than for the inflow distributions (Huber, et al., 1972).

When a reservoir has multiple outlets in which water is withdrawn from different elevations, the velocity distributions are found for each outlet and the results are superimposed to obtain the overall velocity distribution used internally to the reservoir energy transfer process. Although no field verification is available for this procedure, laboratory results presented by Ryan and Harleman (1971) appear consistent with assumptions used in the computer model.

Turbulent Diffusivity

Turbulence in a water body may arise from a dynamic instability associated with the fluid motion, and it may also result from a gravitational instability associated with an unstable temperature gradient. In this model, diffusion[9] by the former has been neglected while diffusion by the latter has been included by an approximating technique based on an instantaneous energy balance (Huber, *et al.*, 1972).

Governing Equations and Boundary Conditions

The governing differential equation of heat transfer is a variation of (4-11)[10] and is applied to each horizontal stratum.

The boundary condition at the water surface specified that the heat diffused away from the surface equals the solar radiation absorbed at the surface $(\beta\phi_s)$ plus atmospheric (longwave) radiation absorbed at the surface (ϕ_a), minus surface losses (ϕ_L) due

[9] The word "diffusion" in this study is used to describe the process of heat transport often termed "conduction".

[10] For a complete theoretical description of this component, the reader is referred to Huber, *et al.* (1972), and Ryan and Harleman (1971).

to evaporation, conduction, and radiation. This condition may be stated as

$$\rho c_p(D_m+E)\left.\frac{\partial T}{\partial y}\right|_{y=y_s} = \beta\phi_s + \phi_a - \phi_L \tag{4-4}$$

where y_s = reservoir surface elevation.

The bottom boundary condition is one of zero flux across the boundary, i.e.,

$$\left.\frac{\partial T}{\partial y}\right|_{y=y_b} = 0 \tag{4-5}$$

where y_b = reservoir bottom elevation.

Although any prescribed initial temperature distribution could be used, this study assumes an initial isothermal condition,

$$T(y, 0) = T_o \tag{4-6}$$

Reservoir Submodel Validation

The reservoir submodel has been validated by solving for historical reservoir temperature profiles in Fontana Reservoir, North Carolina, and in a laboratory model (Huber et al., 1972). There have been later successful field applications of the submodel in both the United States and Australia.[11]

Reservoir Submodel Algorithm

The basic computer algorithm of Ryan and Harleman (1971) has been altered to increase its flexibility and for incorporation into the general water temperature model. Figure 6 is a flow chart of the reservoir subroutine, RESERV, used in this study.

The algorithm begins with the name of the submodel at statement one. Certain data and initial conditions must be read into the program during the first pass through RESERV. This occurs at statement two, and includes the specification of the thickness, ΔY, of each horizontal stratum, the time of release (the time period internal to RESERV), Δt_r, and all data necessary to describe the volumetric shape of the reservoir.

WREAD. At statement three a decision is made to either read the weather at statement four, or to skip to statement five. Statement four (WREAD) is for reading one card containing eight values for each particular meteorological parameter. The number of meteorological observations per day (NWD) must divide evenly into the number of values per card (NVC) so that the last value read on a card is also the

[11] Information received in personal communication with Patrick Ryan, Engineer. (Oakridge National Radiation Lab., Environmental Science Division, Oakridge, Tennessee). April 1973.

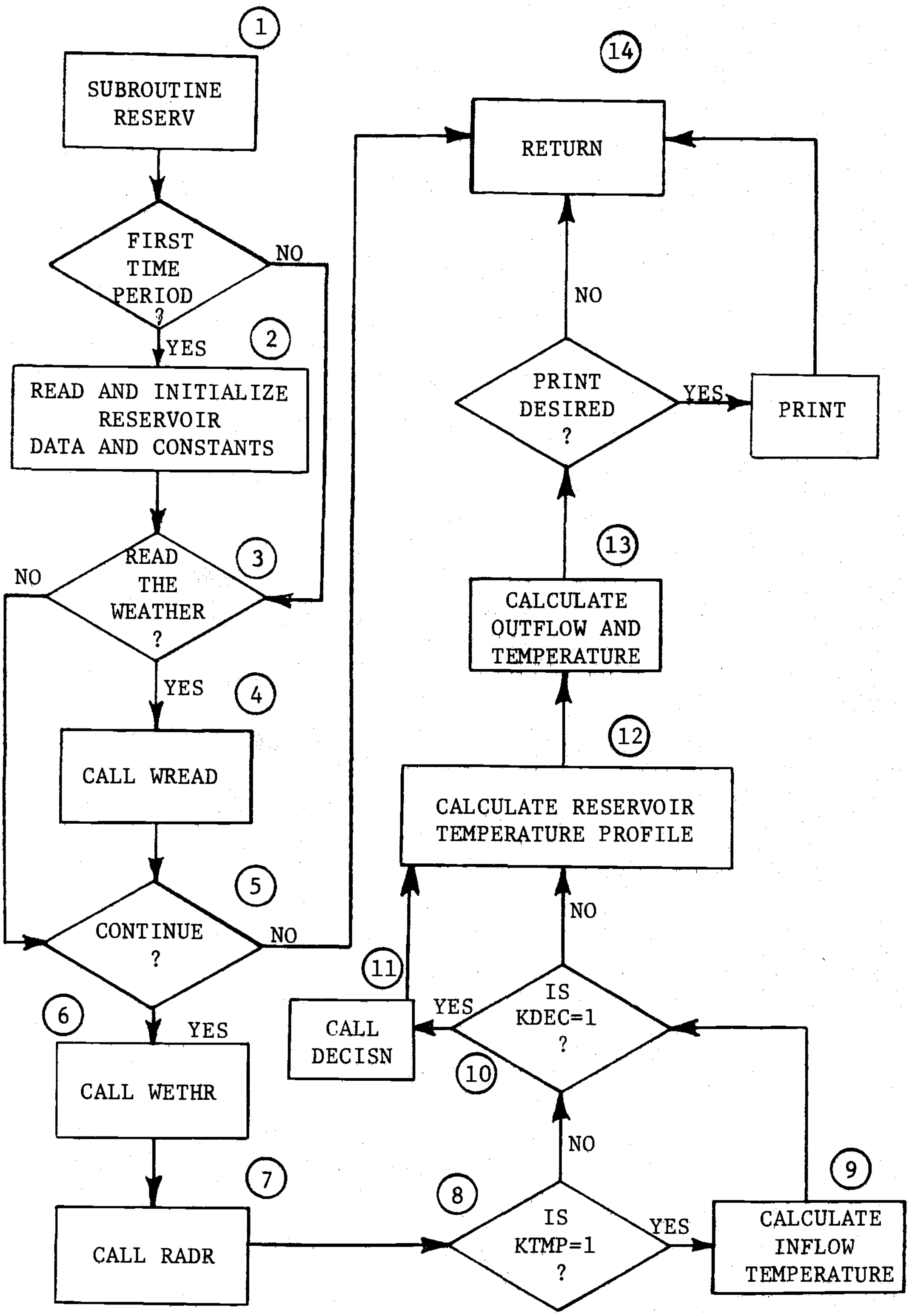

Figure 6. Flow Diagram of Reservoir Computer Submodel, RESERV

last observation for a particular day. Therefore, the maximum number of meteorological observations allowed per day is eight, while the minimum is one.[12] The reservoir inflows are also read from WREAD; these are daily values only.

Continuation. At statement five, a decision is made to continue or not. This depends on the relative values of the time increments internal to RESERV and RIVER. If the time increment internal to RESERV, Δt_r, is greater than the time increment internal to RIVER, Δt, then RESERV returns to the main calling program, ROUTR, via statement 14, unless the sum of the present series of Δt's is equal to Δt_r. Recall that, for this study, Δt_r was always equal to one day, while Δt varied from one day to three hours. If, for example, Δt were equal to three hours (1/8 day), then on every eighth pass (8 x 1/8 = 1 day) through statement five, the routine would proceed to statement six.

WETHR. The subprogram at statement six, WETHR, computes the daily average of each parameter called from WREAD. It is used only if the data are less than daily observations.

RADR. Statement seven, RADR, was discussed in Chapter II. This routine calculates hourly solar radiation striking the earth's

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[12] If the downstream algorithm, RIVER, is not being used, then the minimum number of weather observations per day is equal to $1/\Delta t_r$ days, where Δt_r is an even multiple of one.

surface for one day, and averages the radiation to coincide with the internal time increments of RESERV and RIVER.

Reservoir Inflow Temperature. KTMP, statement eight, is a switch which allows the estimation of the temperature of the reservoir inflow at statement nine, rather than relying on historical data. There are at least three reasons why this is desirable. First, historical streamflow records tend to be of much greater duration than are historical water temperature records. For example, the historical streamflow record for the Calapooia River at Holley, Oregon, spans a period of over 30 years, while the historical stream temperature record is barely ten years in length. If the experimenter wished to simulate an historical sequence of events, his effort could be limited by the length of the stream temperature record.

Second, the dependence upon historical stream temperature data precludes the use of stochastically-generated streamflows, because streamflow and its temperature are highly correlated.

Finally, there remains the convenience of not having to handle data for this particular parameter once a means of estimation is available. Therefore, the following linear model, which estimates the daily temperature of the river water flowing passed the dam site, has been developed.

$$T_{in_t} = \underset{(2.682\times10^{-1})}{0.9598} - \underset{(3.973\times10^{-4})}{1.271\times10^{-3}}Q_t + \underset{(2.983\times10^{-4})}{2.862\times10^{-3}}\phi_{s_t} + \underset{(1.569\times10^{-2})}{0.85696}T_{in_{t-1}} \tag{4-7}$$

where

T_{in_t} = water temperature of reservoir inflow for time, t, in °C

Q_t = streamflow for time, t, in cubic feet per second

ϕ_{s_t} = net solar radiation for time, t, in langleys per day

$T_{in_{t-1}}$ = water temperature of reservoir inflow for time, t-1, in °C.

The standard error of the estimates are shown in parentheses below the coefficients in (4-7). The equation has an R^2 of 0.930 with 522 degrees of freedom; all the coefficients are significant at the 0.005 level. The data used in estimating the coefficients in (4-7) were from April to November for the years 1969 through 1971 at the Holley dam site.

In the simulation process, $T_{in_{t-1}}$ would be specified in the initial conditions. The three independent variables in (4-7) may be obtained either from measured values or stochastic determinations.

<u>DECISN</u>. Statement ten is a switch which governs the use of DECISN, statement 11. DECISN may be any type of decision function in the form of a subprogram. The routine may utilize past, present, and/or future information (probability statements) to specify the

amounts of withdrawal from each outlet in the reservoir. DECISN plays a major role in demonstrating the decision techniques available to the user and will be discussed in more detail in Chapter VI.

If DECISN is not used (i.e., if KDEC is not equal to one), the withdrawal rate per Δt_r from each reservoir outlet must be specified in advance for the entire length of the computer run.

Reservoir Temperature Profile. Statement 12 is essentially the MIT algorithm for computing reservoir temperature profiles.

Total Outflow Temperature. The MIT model will print the outflow and water temperature from each outlet. However, as input to the downstream algorithm, RIVER, it is necessary to combine these outflows and temperatures in computing one mixed outflow and its associated temperature. The conservation of mass and energy equations are used to accomplish this at statement 13. The procedure is

$$Q_o = \sum_{i=1}^{n} Q_i \qquad (4\text{-}8)$$

where

Q_o = total outflow

Q_i = outflow from each outlet

n = number of outlets

and

$$T_o = \frac{\sum_{i=1}^{n} T_i Q_i}{Q_o} \tag{4-9}$$

where

T_o = temperature associated with Q_o

T_i = temperature associated with Q_i .

PRINT. The reservoir temperature profile, outlet flows, and temperatures and relevant coefficients and constants may be printed at any multiple of the time increment internal to RESERV. A sample of the output is contained in Appendix A.

Summary

It has been shown that the deep stratified models are applicable to the proposed Holley Reservoir. One deep reservoir model in particular, the MIT model, has been discussed in detail and forms the basis of the reservoir submodel used in this study. It was explained that by employing the physical characteristics of the proposed reservoir, the inflow and outflow hydrology and temperatures, and the relevant meteorological parameters, this submodel can provide sequential predictions of the reservoir temperature profile, and, thus, the temperature of the reservoir outflow. The prediction of the changes in water temperature as the mass of water moves downstream from the dam is the subject of Chapter V.

V. RIVER TEMPERATURE AND MASS FLOW SUBMODEL

Introduction

Public concern about water quality during the past decade has prompted several studies on the prediction of river water temperatures. To date, most of this work has been conducted in the Pacific Northwest, where the concern is focused on attempting to decrease fish mortalities by lowering river temperatures. Raphael (1962), and Delay and Seaders (1966) conducted studies of the Columbia and Umpqua rivers, respectively. Although these studies presented basic heat-transfer schemes, hand calculations were used in the solutions for both models. Recently published river temperature models making extensive use of the computer (Yearsley, 1969; Morse, 1970, 1972; Norton, 1971) provide the groundwork for a river temperature prediction scheme which could also be incorporated as a component of a general reservoir-river temperature model.

The desired river temperature prediction scheme is one that would be capable of spatially predicting all temperatures and mass flows of interest while holding time constant within the algorithm. It was shown in Chapter IV that the reservoir submodel has the capability to accomplish this by predicting water temperatures along a vertical grid within one time period. However, the river temperature models developed by Yearsley, Morse, and Norton do not exhibit the desired time-related characteristics; hence, flexibility in the operation of these models is lacking. It would be difficult, if not impossible, to link them to a reservoir model for operation as a continuous system.

This chapter details the theoretical and time-dependent structure of a river temperature submodel that is capable of sequentially predicting river temperatures along a longitudinal grid while holding time constant; and may be linked to the reservoir submodel of the previous chapter for operation as a continuous system.

Basic Equations and Review

The two-dimensional conservation of heat equation (3-2) forms the mathematical basis of the river temperature predicting submodel. It may be altered to represent a one-dimensional situation. However, unlike the reservoir submodel described in Chapter IV, the spatial variation occurs longitudinally. This situation is represented by Equation (5-1) as:

$$\frac{\partial T}{\partial t} + U \frac{\partial T}{\partial x} = \frac{\partial}{\partial x} \left[(E+D_m) \frac{\partial T}{\partial x} \right] + \frac{H}{\rho c_p} \qquad (5\text{-}1)$$

where U is the average longitudinal velocity through a vertical section, and H includes <u>all</u> heat sources and sinks.

In the computer models of Yearsley, Morse, and Norton, the diffusion terms in (5-1) were assumed to have a negligible effect on the longitudinal temperature distribution. For this reason, they were not considered by these authors.

Although the reasons for excluding the diffusion terms were not made explicit by the authors, their assumption appears justifiable. For example, it has been shown that in natural streams where turbulence is prevalent (Reynolds Number ≥ 2300),[13] molecular diffusivity, D_m, is several orders of magnitude smaller than turbulent diffusivity, E, and can therefore be ignored (Wunderlich, 1969). In fact, Fischer (1973) stated that longitudinal diffusion, either molecular or turbulent, is relatively unimportant compared to the effect of velocity upon the longitudinal temperature distribution and is, therefore, usually ignored. From a theoretical viewpoint, it would appear that for a first approximation, longitudinal diffusion may be excluded from (5-1).

There is evidence to suggest the exclusion of diffusion from a practical standpoint as well. First, it would appear to be relatively costly to obtain realistic estimates of turbulent diffusivity from many points along a natural river. Second, it adds more data and

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[13] The Reynolds Number is the ratio of internal forces to viscous forces, and is used to distinguish flow regimes.

complexity to the programming problem. Finally, and perhaps most significantly, excellent results in predicting river temperatures have been obtained without the diffusion terms (Yearsley, 1969; Morse, 1970, 1972).

By ignoring diffusion, and assuming a mean river depth, D, Equation (5-1) may be reduced to

$$\frac{dT}{dt} = \frac{\partial T}{\partial t} + U\frac{\partial T}{\partial x} = \frac{H}{\rho c_p D} \tag{5-2}$$

where H is the amount of energy per unit time.

The solution of Equation (5-2) is straightforward when considering the total differential which resembles Equation (3-17), the isothermal model. As a result, in difference notation, Equation (5-2) becomes

$$T_{t+1} = T_t + \frac{H}{\rho c_p D} \cdot \Delta t \tag{5-3}$$

The boundary conditions for (5-2) are identical to those applied to the isothermal model of Chapter III. Heat exchange occurs only at the air-water interface, and the initial water temperature, T_t, must be known or specified.

In order to solve Equation (5-3), the correct time increment, a function of the longitudinal distance (X) and the water speed (U), must be determined. A simplified exemplary solution of Equation (5-3) is outlined below, with reference to Figure 7.

Assume that the river reach of length, ΔX, has an average flow rate (Q) and that the water temperature (T_t) at the base of the dam is noted and recorded. The depths, D_1 and D_2, are recorded, and the average reach depth (D) is designated as the mean of D_1 and D_2. Assume that cross-sections of the channel are available at points 1 and 2, and because the depths at these points are also known, the water cross-sectional areas may be calculated. These two areas may then be averaged to provide a mean area for the reach. Because the average flow rate (Q) and cross-sectional area (A) are known, the average water speed for the reach (U) can be calculated as $U = Q/A$. This allows the reach travel time (t_v) to be found as $t_v = \Delta X/U$. The heat flux (H), which is a function of T_t, may be found as described in Chapter III. Thus, Equation (5-3) may be solved for T_{t+1}. The solution process is relative to the position of the leading edge of the water parcel in time t. In essence, the water is flagged at the dam, and this flag is followed downstream.

This procedure has, in general, been followed by Yearsley (1969), Morse (1970, 1972), and Norton (1971). However, there have been variations in the methods used to evaluate the average water speed (U), the net heat flux per time period (H), and the process

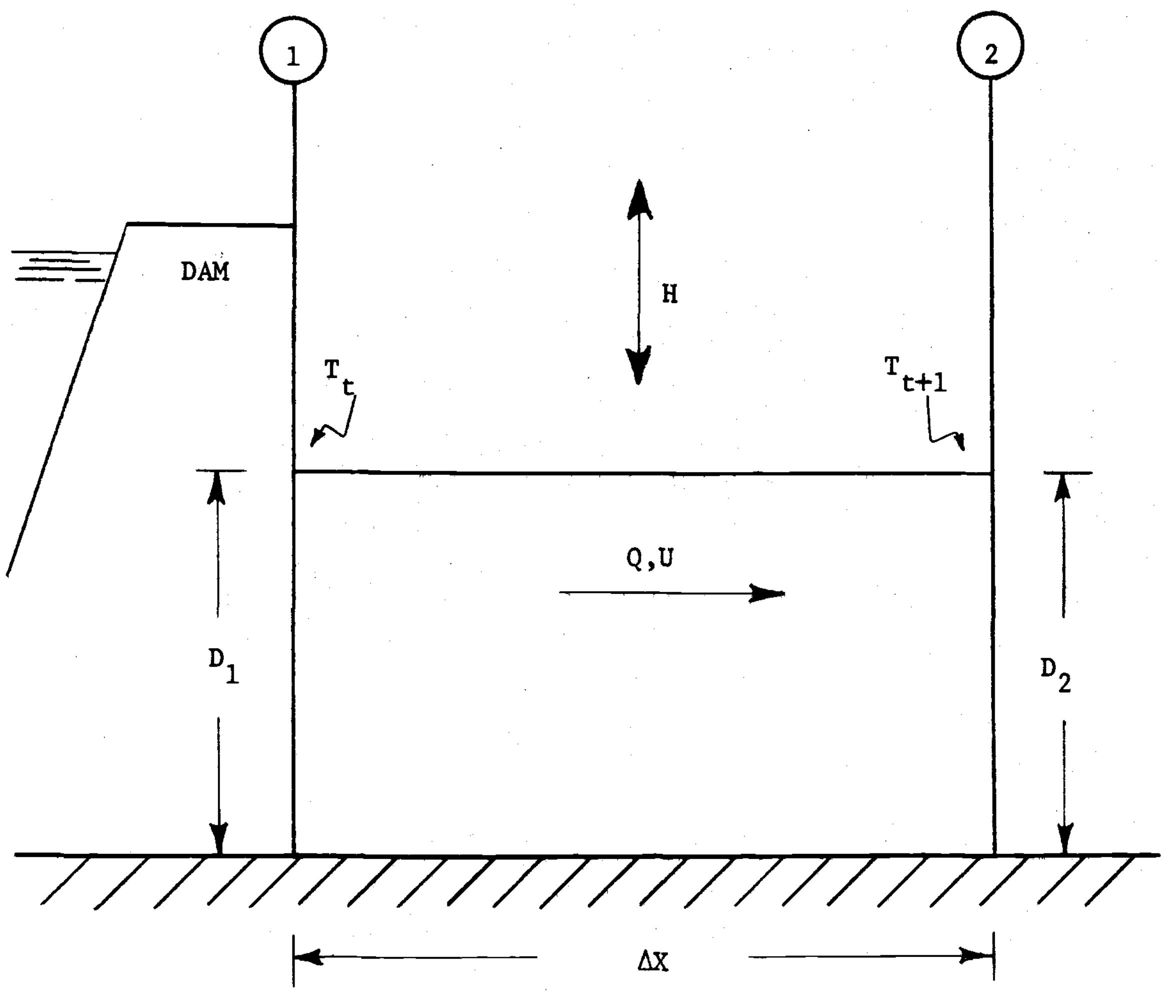

Figure 7. Representation of Parameters Used in Solution of River Temperatures

by which the algorithm solves for the water temperature.

Average Water Speed

Delay and Seaders (1966) determined the average water speed (U) from travel time studies using Rhodamine-B dye. Norton (1971) employed the gradually varied flow equation to determine backwater profiles, and, ultimately, the average water speed. Yearsley (1969) and Morse (1970, 1972) used a length-averaging technique for determining average flows for each reach. They also delineated a representative water profile which was eventually used to determine the water speed. In fact, both Yearsley and Morse used the identical model which shall be referred to as the EPA model.[14]

Net Heat Flux

Raphael (1962) and Norton (1971) determined the net heat flux (H) by the same general procedure as was described above, and detailed in Chapter III, using time steps of three hours and one hour, respectively.

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[14] Bruce Tichenor is responsible for the initial work and conceptualization of the hydraulic components of the EPA model. Morse and Yearsley also contributed to the creation and validation of this model. (Personal communication, William Morse, Mathematician, Bonneville Power Administration, 1973; and personal communidation, Bruce Tichenor, Sanitary Engineer, Thermal Pollution Branch, Pacific Northwest Environmental Research Laboratory, Environmental Protection Agency, Corvallis, Oregon.)

Delay and Seaders (1966) averaged each meteorological parameter over a five- to seven-hour period for ten days. The mean of these ten values was assumed to be a representative value of the parameter for that particular five- to seven-hour period over the ten days. A range of water temperatures was selected and a plot of net heat flux (H) versus water temperature was calculated for each representative period.

The EPA model outlines the identical procedure used by Delay and Seaders (1966) with two exceptions. First, the plotted functions were fitted to quadratic equations which were incorporated into the computer model. Second, the representative periods of time were of three and six hours in length, rather than five and seven hours.

EPA Algorithm

It appears from the published work cited above that the EPA model is the best river temperature predicting model of those reviewed. Also, the computer program is a relatively efficient algorithm, requiring a minimum of hydraulic data.

This algorithm typifies the other available models in that all data are read into the program at the beginning of the computer run. The average water speed (U) and depth (D) are solved for each reach over all time periods, proceeding downstream one reach at a time. As a result, the solution for the hydraulic parameters occurs while varying time and holding space constant. However, for programming ease, the most recent observations are considered first.[15] Hence, the incrementation proceeds backwards in time when solving for water speed and depth.

A characteristic shared by all the models reviewed is the process involving time and space incrementation of Equation (5-3) for solution of the water temperatures. Recall from the discussion of Figure 6 that the travel time (t_v) over the reach length (ΔX) is unique for any given average water speed (U). Thus, if a parcel of water were flagged at the dam in time t_1, the time of arrival at station two would be unique for any U, and equal to $t_1 + t_v$. In fact, only by pure coincidence would the travel time (t_v) be equal to Δt, the designated rate at which the algorithm is to calculate the water temperature at each station.

Thus, as noted in Chapter II, the discreteness of the problem makes it difficult to determine temperatures at all stations at designated points in time. The remainder of this chapter is devoted to the formulation of an algorithm which addresses this particular problem.

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[15] Personal communication, Bruce Tichenor, Sanitary Engineer, Thermal Pollution Branch, Pacific Northwest Environmental Research Laboratory, Environmental Protection Agency, Corvallis, Oregon 1973.

Hydraulic Component

The hydraulic component of the river temperature submodel is an adaptation of the EPA model with exceptions where noted. This model requires a minimum of hydraulic data, and, as mentioned previously, excellent results in predicting water temperatures have been obtained with its use.[16] A critical assumption governing the EPA model is that the river may be divided into many free-flowing, or independent, reaches. The hydraulic parameters over the time period between temperature calculations (Δt) may be averaged for the river reach in question. Thus, for any time period, each reach has the shape of a rectangular parallelopiped.

Estimation of Surface Profiles

Tichenor found that, on the lower Columbia River, a specific change in depth (ΔY) at any station was reflected in an approximate

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[16] Accurate prediction of water temperatures does not, of course, imply accurate prediction of hydraulic parameters. However, it does suggest one of two things: (1) any biases present in the EPA hydraulic model were consistent between river reaches in the studies in which the model was used; or (2) the river temperatures in the studies where the EPA model was used are not particularly sensitive to relatively small random changes in exposure time to heat sources.

change of ΔY at all stations.[17] He was then able to define a representative stream profile which was incorporated into the EPA model. Although this procedure appeared to suffice for the Columbia and Little Deschutes Rivers over a narrow range of flows (Morse, 1970, 1972) it is not appropriate for a cascading stream full of pools and riffles such as the upper Calapooia River.[18]

The rationale for the preceding statement can best be shown with reference to Figure 8. As detailed previously, station locations are restricted to points along the river where area cross-sectional data of the channel are available. Assume that the stream profile is initially along the line marked A in Figure 8. Now, let the river stage at station one drop by an amount ΔY^*. The EPA assumption would call for subsequent drops of the river stage by ΔY^* at stations two, three, and four, resulting in a new stream profile shown by the dotted line marked C. This would be impossible; there would be no water along a significant portion of the channel. Consequently, the assumption that was apparently permissible in the two applications of the EPA model discussed above does not appear to be valid for the

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[17] Personal communication with Bruce Tichenor, Sanitary Engineer, Thermal Pollution Branch, Pacific Northwest Environmental Laboratory, Environmental Protection Agency, Corvallis, Oregon, 1973.

[18] The assumption of free-flowing and independent reaches may not be justified either for a cascading stream, but is made for want of a better approach.

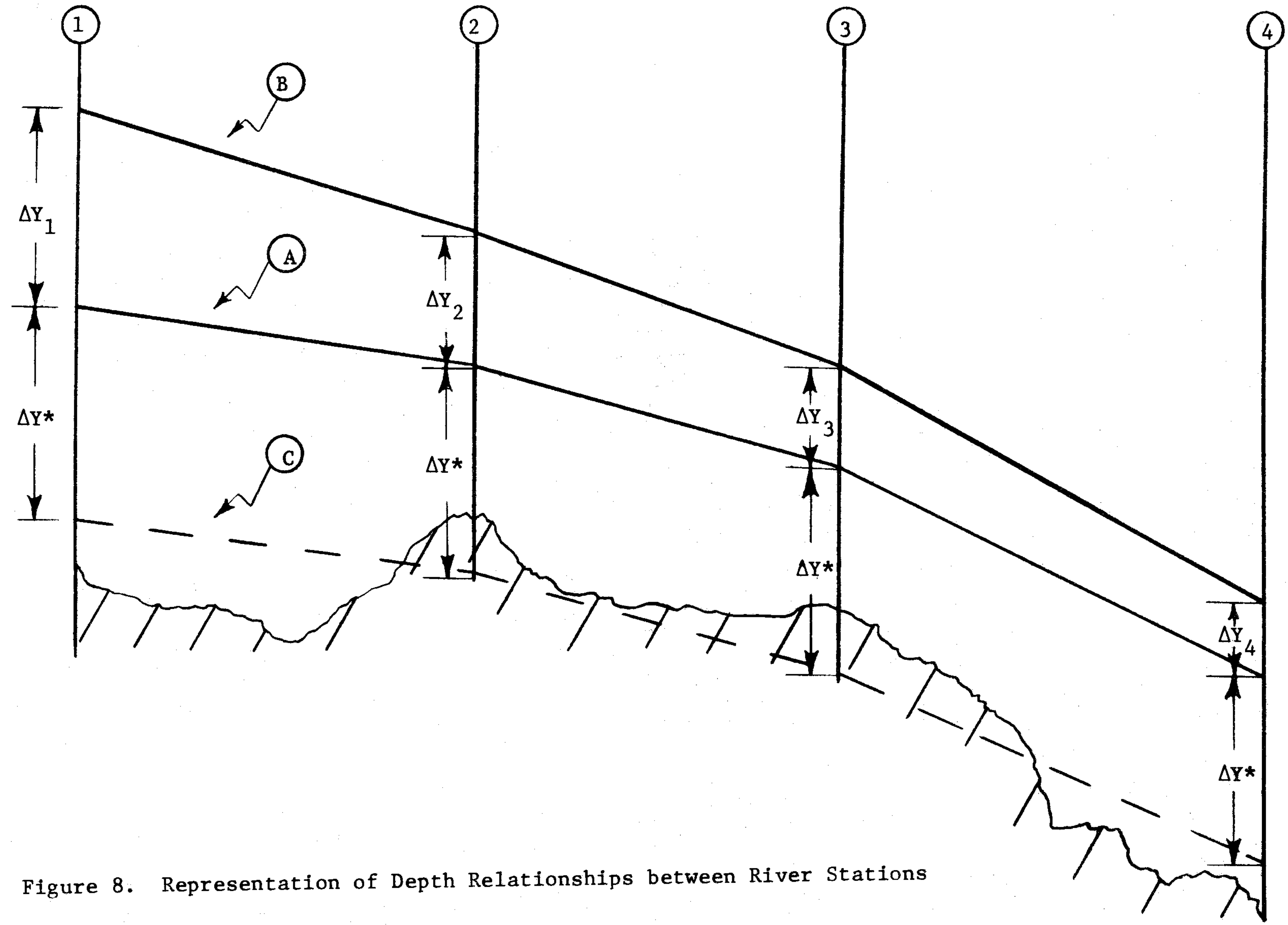

Figure 8. Representation of Depth Relationships between River Stations

Calapooia River during low flow periods.

Traditional hydraulic approaches for determining surface profiles, such as the standard step method, would be difficult to use because within any designated reach (500 feet to 1-1/2 miles) there could be a great many profiles over a wide range of frictional coefficients. The use of dye studies for determining water speeds in each reach would be a possibility, but limited resources for this study prohibited this approach.

The procedure used to determine surface profiles for each reach and resulting depths is strictly empirical and was based on the available hydrologic data. A permanent recording staff gauge is located at the site of the proposed Holley Dam (station one in Figure 8). This gauge provides for a functional relationship between stage (depth) and discharge, and has been approximated by three least-squares equations over three ranges of flow. Unfortunately, this type of recording gauge is not available in the downstream reaches used in this study. However, a limited amount of data was available from ten crest gauges located downstream for the proposed dam. Although most of the data associated with these gauges were recorded at relatively high flows, there was enough information available to formulate the following relationship:

$$\left(\frac{\partial Y_j}{\partial Y_1}\right)_Q = K_j \tag{5-4}$$

for $j = 1$, JMAX subject to $90 \leq Q \leq 1000$

where

j = station number

1 = station 1

Q = streamflow in cfs

K = constant

Y = river depth.

Equation (5-4) states that, for a given streamflow at station one, as well as at any downstream station (j), a constant relationship exists between the depths at station one and the jth station within the range of the stream flow constraint. For example, in Figure 8 the water profile is assumed to be represented by the line marked B. If ΔY_1 were equal to 1.0 feet, then ΔY_2 might be equal to 0.5 feet and the new stream profile would be the line marked A. The ratio, $\Delta Y_2/\Delta Y_1$, would equal 0.5. For this study, this relationship would be expected to hold as long as the stream flow at stations one and two were identical and within the constraint.

Estimation of Reach Averages

The hydraulic data available at the river stations are the widths and areas of the channel cross-sections, and the longitudinal distances between these sections. If the depths and flows at each station are added to this list, and a Δt of interest is specified, all required information is available to determine the reach averages of areas, widths, depths, water speeds, and streamflows for the specified time period.

The procedure used can best be demonstrated by referring to Figure 9. Assume that time is t and is recorded. Assume also that a new flow period is just beginning. The model is structured so that there are $24/\Delta t$ flow periods in a day, and at the beginning of each flow period, the flow at the base of the dam is flagged.

A decision to change the withdrawal rate from the reservoir can be made only every Δt_r, i.e., once a day. The discrete step in the water surface at the flag in Figure 9 indicates the beginning of a new Δt_r as well as the beginning of a new flow period. At time (t), the depth (D_{ji}), cross-sectional area (A_{ji}), and channel width (W_{ji}) are known at station J for the streamflow (Q_i) in period i. . Because the depth (D_{ji}) is known, the depth $(D_{j+1\,i})$ that would occur at station J+1 for the flow, Q_i, is also known. Consequently, the area $(A_{j+1\,i})$ and width $(A_{j+1\,i})$ are known. The

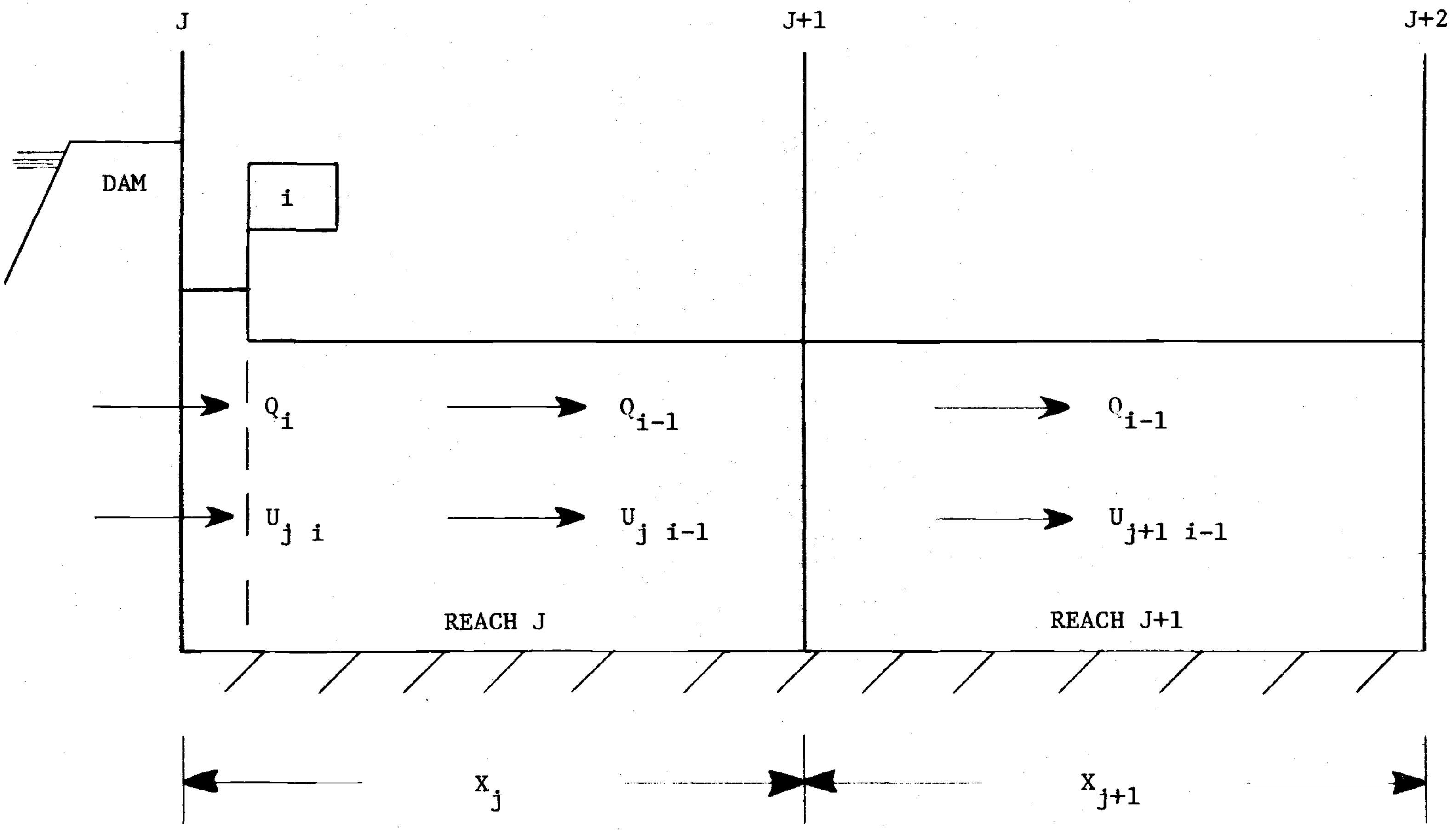

Figure 9. Stream Flows, Water Speeds, and Reach Lengths for Flow Periods i and i-1 at Time t.

reach averages associated with flow period i may then be calculated as

$$AREA_{ji} = AR_{ji} = \frac{A_{ji} + A_{j+1\,i}}{2} \tag{5-5}$$

$$WIDTH_{ji} = WR_{ji} = \frac{W_{ji} + W_{j+1\,i}}{2} \tag{5-6}$$

$$DEPTH_{ji} = \frac{AR_{ji}}{WR_{ji}} \tag{5-7}$$

The average water speed in reach J associated with the flow period i is

$$U_{ji} = \frac{Q_i}{AR_{ji}} \tag{5-8}$$

The system operates for a time increment, Δt, which is the time between river temperature calculations (three hours or six hours for this study). The flow volume that occurs in reach J over Δt for flow period i is then compared to the available reach volume:

$$VOL_{ji} = (X_j) \cdot (AR_{ji}) \tag{5-9}$$

$$FLOVOL_{ji} = (Q_i) \cdot \Delta t \tag{5-10}$$

Assume in this case that the flow volume $(FLOVOL_{ji})$ was greater than the available reach volume (VOL_{ji}) which resulted in the leading edge (flag) of the streamflow in flow period i to penetrate into the next reach, as shown in Figure 10. Because there is only one flow period present in reach J at time $t+1$, the reach averages in $t+1$ are equivalent to the reach averages for the flow period i. Thus:

$$\eta_{j\ t+1} = \eta_{ji} \tag{5-11}$$

where

j = reach number

i = flow period

t = time period

η = any relevant hydraulic reach parameter.

The general case of (5-11) is valid only if all observations of a particular parameter on i in the relevant time period are identical. From Figure 10, it is seen that this is not the case for reach $J+1$, as both flow periods i and $i-1$ are present at time $t+1$. Therefore,

$$\eta_{j+1\ t+1} \neq \eta_{j+1\ i} \neq \eta_{j+1\ i-1} \tag{5-12}$$

The method for determining $\eta_{j+1\ t+1}$ proceeds in a similar manner as before. Thus, for flow period i,

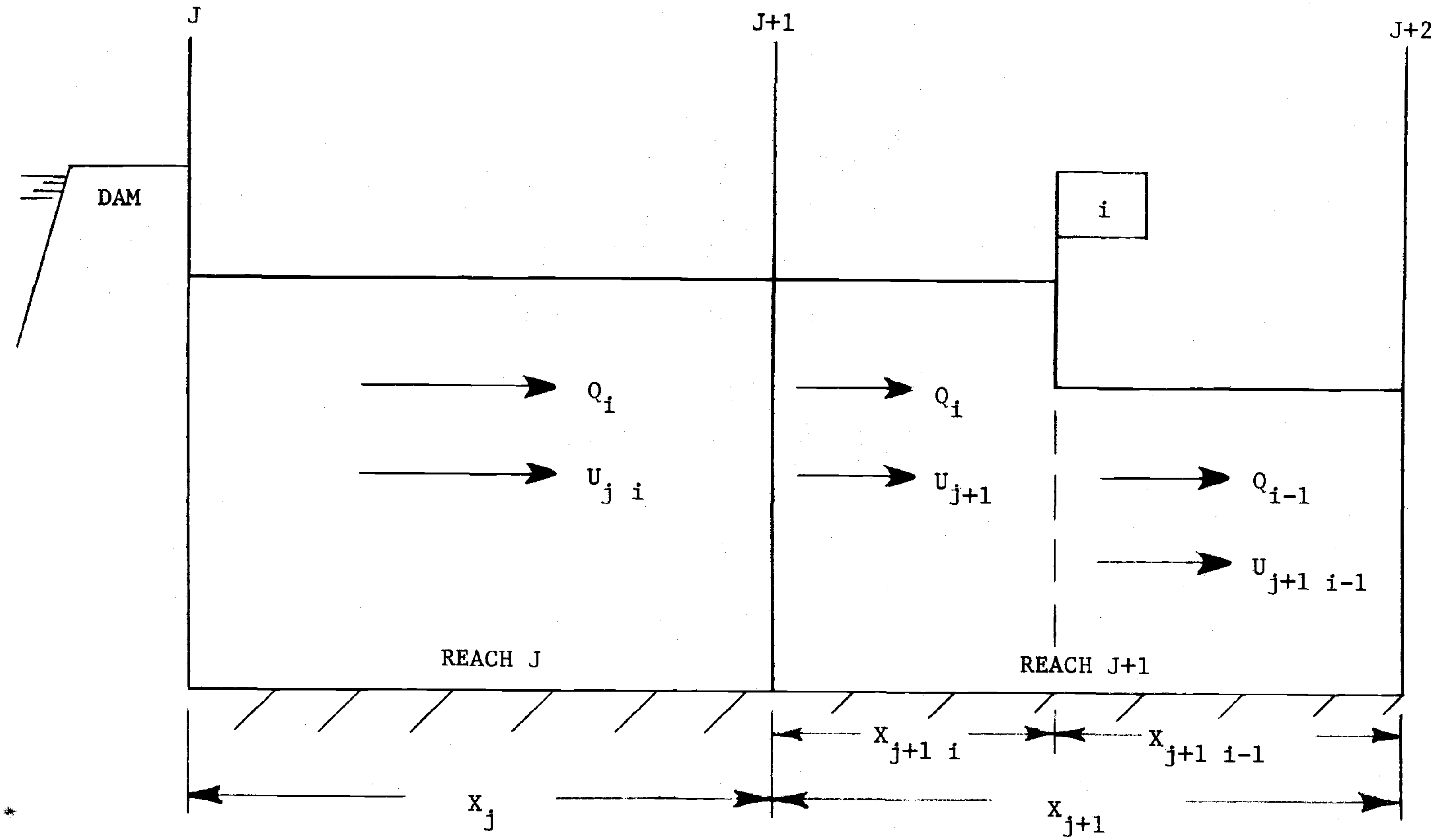

Figure 10. Stream Flows, Water Speeds, and Reach Lengths for Flow Periods i and i-1 at Time t+1.

$$\eta_{j+1\ i} = \frac{\eta^*_{j+1\ i} + \eta^*_{j+2\ i}}{2} \tag{5-13}$$

where

η^* = parameter value at station

η = average value of parameter in reach

i = flow period

j = station and reach.

The available reach volume is found as before.

$$VOL_{j+1\ i} = (X_{j+1}) \cdot (AR_{j+1\ i}) \tag{5-14}$$

and the flow volume for $t+1$ is

$$FLOVOL_{j+1\ i} = (Q_i) \cdot (\Delta t_{j+1\ i}) \tag{5-15}$$

The parcel of water released from the dam in time (t) exceeded reach J over an interval Δt. The time for the flagged parcel to traverse reach J was

$$\Delta t_{ji} = X_{ji}/U_{ji} \tag{5-16}$$

and

$$\Delta t_{j+1\ i} = \Delta t - \Delta t_{ji} \tag{5-17}$$

where $\Delta T_{j+1\ i}$ is the time of penetration into reach J+1 by the parcel of water released in flow period i during the interval Δt.

The volume and flow volume are compared, and, as was assumed, the reach volume available was greater than the flow volume. Consequently, the hydraulic parameters associated with the previous flow period are present in reach J+1 at time t+1.

The mean value for each hydraulic parameter is calculated as before.

$$\eta_{j+1\ i-1} = \frac{\eta^*_{j+1\ i-1} + \eta^*_{j+2\ i-1}}{2} \tag{5-18}$$

From information of the state of reach J+1 in the previous time period (t) it is ascertained that there are no more flow periods associated with J+1 in t+1.

The average values of the reach parameters in time t+1 may be found by distance averaging. The distance that flow period i has penetrated reach J+1 at t+1 is

$$X_{j+1\ i} = (U_{j+1\ i}) \cdot (\Delta t_{j+1\ i}) \tag{5-19}$$

The distance allocated to flow period i-1 in reach J+1 at t+1 is

$$X_{j+1\ i-1} = X_{j+1} - X_{j+1\ i} \tag{5-20}$$

The average values of depth, water speed, and streamflow in reach J+1 at time t+1 are

$$\eta_{j+1\ t+1} = \frac{(\eta_{j+1\ i}) \cdot (X_{j+1\ i}) + (\eta_{j+1\ i-1}) \cdot (X_{j+1\ i-1})}{X_{j+1}} \qquad (5\text{-}21)$$

where

η = the relevant hydraulic parameter

X = the longitudinal distance.

The general form of Equation (5-21) for determining the average value of η in any reach j at any time t is

$$\eta_{jt} = \frac{\sum\limits_{i=i} \eta_{ji} X_{ji}}{X_j} \qquad (5\text{-}22)$$

for $j = 1$, JMAX and where

$$X_j = \sum_{i=i} X_{ji} \qquad (5\text{-}23)$$

There appears to be no theoretical justification for Equation (5-22), although it does have certain intuitive appeal. It is a method used in the EPA model for deriving average reach values in an attempt to render the continuous river system discrete.

The similarities between the EPA model and the model developed in this study end here. The primary difference in the two modeling approaches, as emphasized earlier, is the time-space solution sequence. The EPA model exhibits the following solution sequence for η_{jt} :

$$\eta_{jt} \quad \text{where } [t=1, \text{TMAX}]_{j=1,\ \text{JMAX}} \tag{5-24}$$

while the hydraulic component developed above is solved as

$$\eta_{jt} \quad \text{where } [j=1, \text{JMAX}]_{t=1,\ \text{TMAX}} \tag{5-25}$$

Equation (5-24) indicates that values of η are found over all time periods while holding j constant. The value of j is then incremented and the process proceeds through $j = \text{JMAX}$.

Equation (5-25) denotes a solution sequence where η is found over all j's (the river stations) while holding time constant. After each solution by (5-25) of the reach parameters, the river temperature $(T_{j+1\ t})$ at the downstream station within this reach is found as detailed in the next section.

Solution of River Temperatures

A general procedure for solving a specific form of the energy equation (5-3) for river temperatures is presented in this section. Although a form of Equation (5-3) was used in the models reviewed, the solution sequence for river water temperatures common to these models is:

$$T_{jt} \text{ where } \left[\begin{array}{c} j=1, JMAX \\ \\ t=1, TMAX \end{array} \right] \tag{5-26}$$

The notation in (5-26) is to indicate that time and space vary together. This would be the case if a parcel of water were released from the dam and then followed downstream to the last point of interest while incrementing times, space, and river temperatures simultaneously.

By comparison, the solution sequence for river water temperatures in this study is identical to the solution sequence for η in Equation (5-25), the relevant hydraulic parameter. This is given by Equation (5-27) where time is again held constant over the river stations.

$$T_{jt} \text{ where } [j=1, JMAX]_{t=1, TMAX} \tag{5-27}$$

As shown in Figure 10, the solution sequence begins at a river station (J+1) at some time (t+1). The temperature of the water at this point in space and time is unknown. The temperature of the water at some time in the past must be known before the new temperature can be calculated. However, the water now at station J+1 was somewhere upstream in the previous time period. Thus, the procedure "moves backward" in space (upstream) and backward in time until an upstream reference point (river station) is encountered and the temperature of the river at that point in time and space is calculated. The procedure then moves ahead (downstream) in space and ahead in time, updating the river temperature every Δt until the original station (J+1) and original time period (t+1) are encountered. The algorithm then treats the next downstream station (J+2) for the same time period (t+1). The following mathematical description with reference to Figure 10 explicitly details this procedure.

The task is to demonstrate the solution for river water temperatures in time (t+1) at stations J+1 and J+2. The temperature at station J (the base of the dam) is known and constant over the time period Δt_r and the flow period i.

Equation (5-3) is restated with reference to Figure 10 as

$$T_{j+1\ t+1} = T^*_{jt} + \frac{H_{t+1}}{\rho c_p D_{j\ t+1}} \cdot \frac{\Delta t^*_{j\ t+1}}{\Delta t} \qquad (5\text{-}28)$$

where

$D_{j\ t+1} = \eta_{jt}$ from Equation (5-22)

$\Delta t^*_{j\ t+1} = \Delta t$ or the travel time, $TRAV_{j\ t+1}$ from the most recent upstream station (J in this case)--the smaller of the two

H_{t+1} = total accumulated flux in Δt (from t to t+1)

T^* = temperature that the water at J+1 in time t+1 was $\Delta t^*_{j\ t+1}$ time ago.

The ratio $\Delta t^*/\Delta t$ is necessary because the algorithm accumulates the flux over a period of Δt --usually three or six hours. If Δt were three hours and Δt^* were two hours, then the use of the ratio allows the flux to act over the leading water edge for only that amount of time for which the water is exposed to the prevailing meteorological conditions.

The water speed in reach J for time $t+1$ is calculated from Equation (5-22) as $U_{j\ t+1}$. Thus, the potential distance traveled by the water at station $J+1$ over the time period t to $t+1$ (i.e., Δt) is

$$X_{j\ t+1} = U_{j\ t+1} \cdot \Delta t \tag{5-29}$$

But, from Figure 10 and preceding calculations, it is apparent that this potential distance traveled, $X_{j\ t+1}$, is greater than the reach length, X_j. Thus, the travel time for the water now at station $J+1$ becomes the relevant time increment for Δt^* in Equation (5-28) and is

$$t^*_{j\ t+1} = TRAV_{j\ t+1} = X_j/U_{j\ t+1} \tag{5-30}$$

Note that the water now at $J+1$ had a temperature of T^* and was at the dam Δt^* time ago. The resulting solution of Equation (5-28) for $T_{j+1\ t+1}$ now becomes routine and the temperatures at two stations, J and $J+1$, are known at $t+1$.

Following is the procedure for finding the temperature at station J+2 in time t+1. The procedure is similar to that used at station J+1, but is somewhat more involved. Equation (5-28) is rewritten for station J+2 as

$$T_{j+2\ t+1} = T^*_{j+1\ t} + \frac{H_{t+1}}{\rho c_p D_{j+1\ t+1}} \cdot \frac{\Delta t^*_{j+1\ t+1}}{\Delta t} \tag{5-31}$$

The water at station J+2 in time t+1 (flag W_{t+1} in Figure 11) must be monitored backwards in time to determine when the flag (W) was at the last upstream temperature reference point (station).[19] The initial step, as before, is to calculate the potential distance traveled by the flag over the time period t to t+1. This is

$$X_{j+1\ t+1} = U_{j+1\ t+1} \cdot \Delta t \tag{5-32}$$

Assume that $X_{j+1\ t+1}$ is of the magnitude depicted in Figure 11 and, thus, the flag was at W_t, Δt hours ago. The temperature reference point, station J+1, has not been reached or passed ($X_{j+1} > X_{j+1\ t+1}$), which implies this procedure must be repeated for the time period t-1 to t as follows.

$$X_{j+1\ t} = U_{j+1\ t} \cdot \Delta t \tag{5-33}$$

[19] The position of the water surface in Figure 11 is for the time t+1. The water surface profiles for times t and t-1 are not shown.

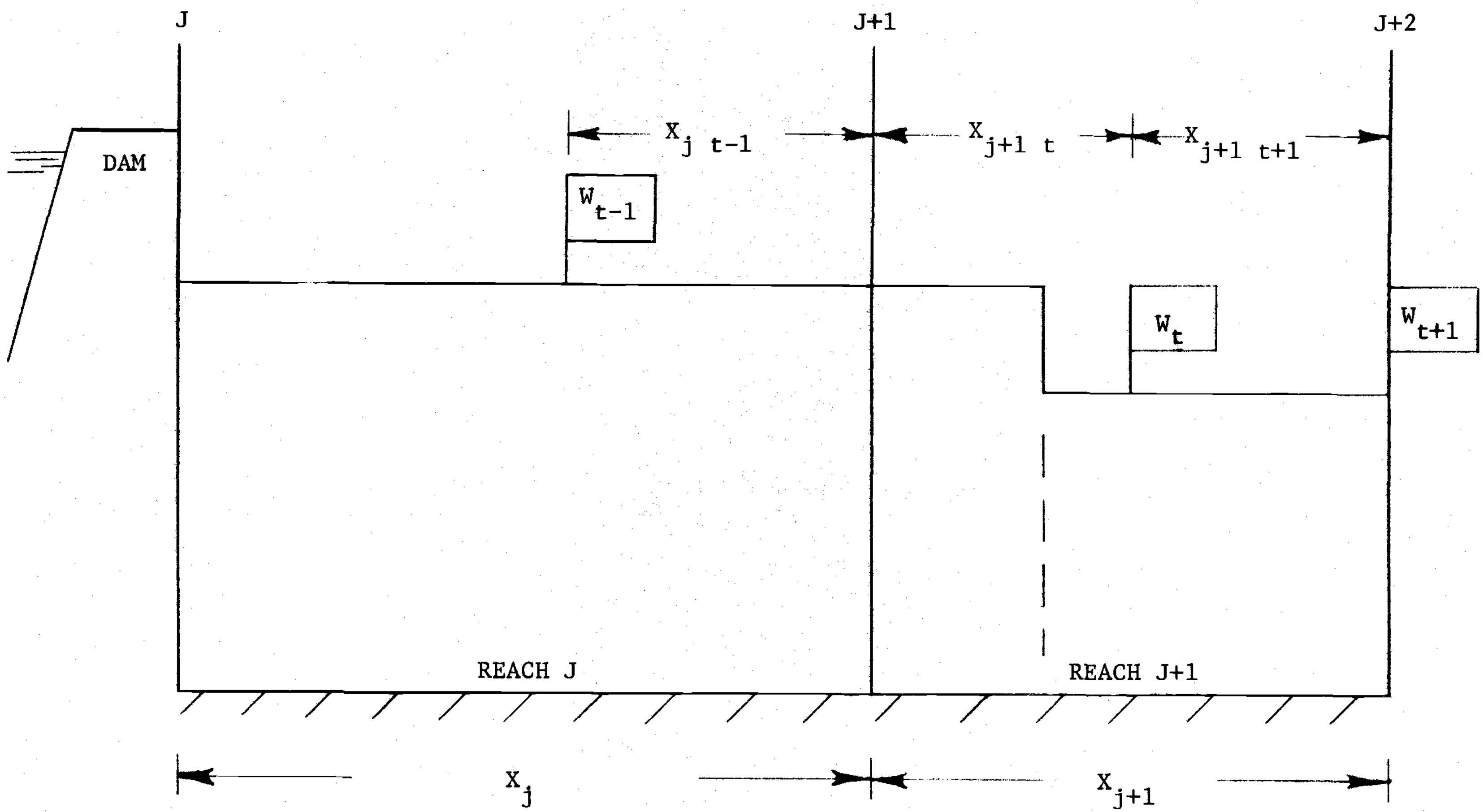

Figure 11. Positions of the Flag, W, in Times t+1, t, and t-1.

The potential distance traveled, $X_{j+1\ t}$, can be no greater than the available distance in reach J+1. Hence, another value of $X_{j+1\ t}$ is calculated which becomes the limiting value of distance traveled in reach J+1 for time t-1 to t. This limiting value is the difference between the reach length and the distance traveled in all time periods from the period now under consideration to the period t+1 --i.e., when the flag is at station J+2. This value is

$$X_{j+1\ t} = X_{j+1} - X_{j+1\ t+1} \tag{5-34}$$

Assume that Equation (5-34) yields a smaller value of $X_{j+1\ t}$ than does Equation (5-33), and that the value associated with (5-34) thus becomes the relevant value. This implies that at the beginning of the time period under consideration, the flag was upstream of station J+1 and, therefore, a temperature reference point was passed. However, it remains to be shown at what time the flag W_{t-1} passed station J+1 while moving upstream in the solution sequence. This time, which must be somewhere between the times t-1 and t, is found as follows.

First, the travel time over the distance $X_{j+1\ t}$ from J+1 to the flag W_t is calculated as

$$\Delta t^*_{j+1\ t} = TRAV_{j+1\ t} = \frac{X_{j+1\ t}}{U_{j+1\ t}} \tag{5-35}$$

Then, the following subtraction is made.

$$\text{Point in time that flag } (W_t) \text{ was at } J+1 = t - \Delta t^*_{j+1\ t} \tag{5-36}$$

An example may be helpful in clarifying the above with reference to Figure 12. Assume that $t+1$ equals 1800 hours and Δt = three hours. Thus, t equals 1500 hours and $t-1$ equals 1200 hours. If $\Delta t^*_{j+1\ t}$ were equal to two hours, then the flag would have been at station $J+1$ at 1300 hours with a temperature denoted as $Tw_{j+1\ t-1}$. This temperature is found by linearly interpolating between the reference temperatures $T_{j+1\ t}$ and $T_{j+1\ t-1}$ The equation is:

$$Tw_{j+1\ t=1} = T_{j+1\ t} + \frac{\Delta t^*_{j+1\ t}}{\Delta t} [T_{j+1\ t-1} - T_{j+1\ t}] \tag{5-37}$$

The rounding error in this approach will increase as the value of Δt increases. For a Δt of three hours, the river temperatures calculated by this method differed by a maximum 0.2° F from temperatures predicted using a routing scheme similar to the EPA model.

All water temperatures in Figure 12 are now known with the exception of $T_{j+2\ t+1}$. However, this temperature may be obtained by routing the water back downstream in time by first solving for the temperature at the flag W_t in Figure 11, as follows:

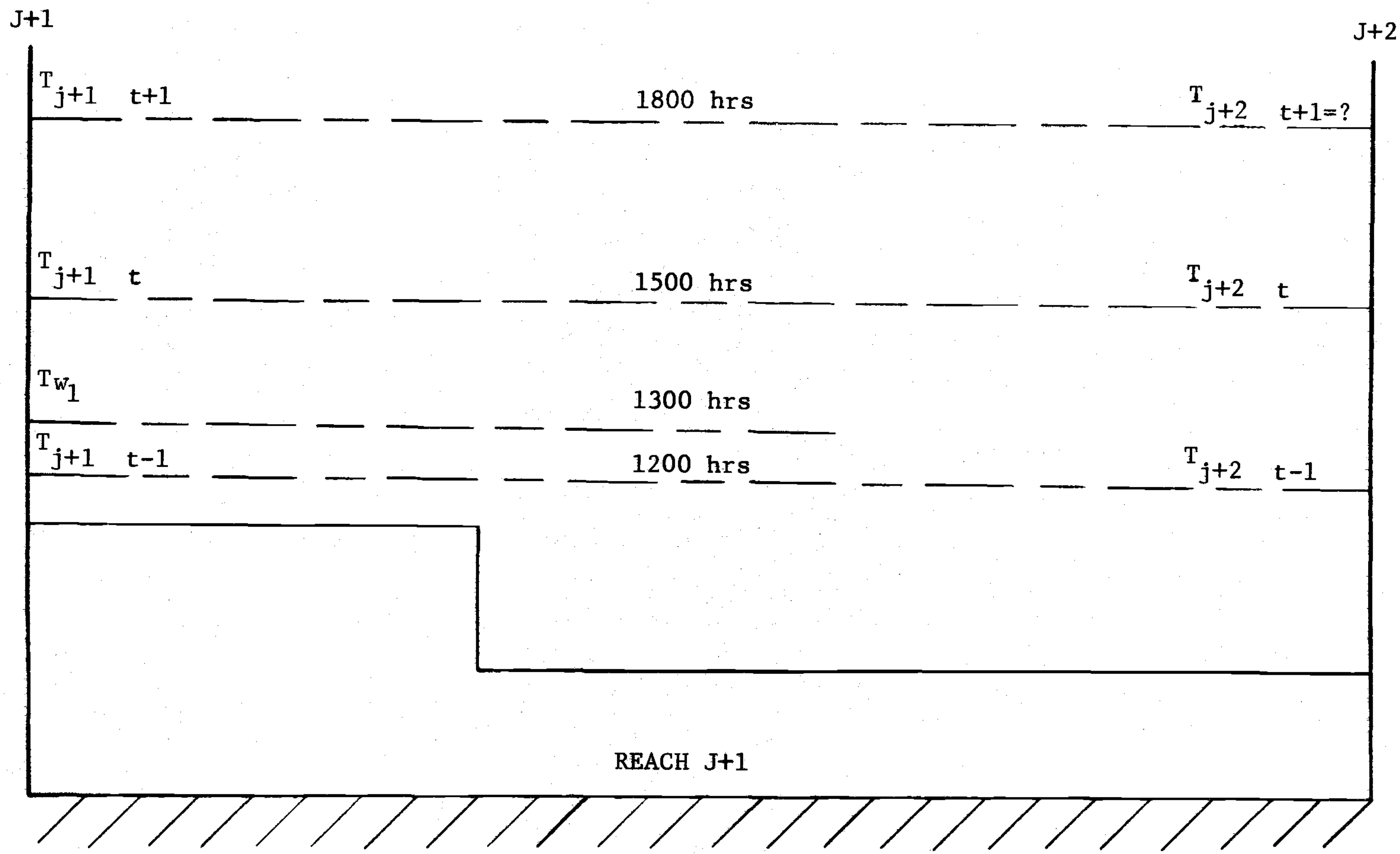

Figure 12. River Temperature References at Reach J+1 for Solution at Time t+1

$$Tw_t = Tw_{j+1\ t-1} + \frac{H_t}{\rho c_p D_{j+1\ t}} \cdot \Delta t^* \tag{5-38}$$

and, finally, the solution for the river temperature at $J+2$ is

$$T_{j+2\ t+1} = Tw_{j+2\ t+1} = Tw_t + \frac{H_{t+1}}{\rho c_p D_{j+1\ t+1}} \cdot \frac{\Delta t^*_{j+1\ t+1}}{\Delta t} \tag{5-39}$$

where $t^*_{j+1\ t+1}$ is equal to Δt in this particular case.

Thus, the temperatures at stations J, $J+1$, and $J+2$ are known at time $t+1$ and the algorithm would then proceed to the next reach and station where the identical process would be repeated.

Treatment of Advected Flows

The downstream temperature submodel described in the preceding section appears structurally adequate for rivers or river reaches with no major tributaries, or from which there are no sizable withdrawals. However, a river that does not exhibit tributary inflows or withdrawals is rare. If the objective of constructing a realistic model is to be realized, provisions must be made for coping with advection to and from the parcels of water released from the dam.

Three approaches for incorporating advection into the river model have been considered. Each will be discussed in turn.

Flow Dependent Approach

The simplest approach to modeling advected flows so that the algorithm will demonstrate at least a minimum of flexibility is presented in Figure 13. The flags, as before, represent flow periods Δt hours apart. Assume that advection occurs at station J+6 in the form of withdrawals for irrigation. One approach would be to activate a decision submodel when any flag reaches station J+6. The decision routine could then fix the amount of withdrawal for that particular flow period. Thus, in Figure 13, the amount of water being withdrawn from flow period i+1 is represented by the cross-hatched area between station J+6 and the flag i+1. Likewise, the withdrawal from flow period i is represented by the lined area between flags i and i+1, and that from flow period i-1 by the shaded portion to the right of flag i. The withdrawals from each flow period would not necessarily be identical and would be equal to zero at certain times during the year.

While this approach is appealing because of the relative ease in which the computer programming may be carried out, there is an inherent drawback: the lack of time dependency, or, a realistic time reference from which to make withdrawal decisions. A decision maker, be he engineer, watermaster, or local farmer, would not know when a flag was at station J+6, the point of withdrawal. Therefore, this flow dependent approach was discarded in favor of a

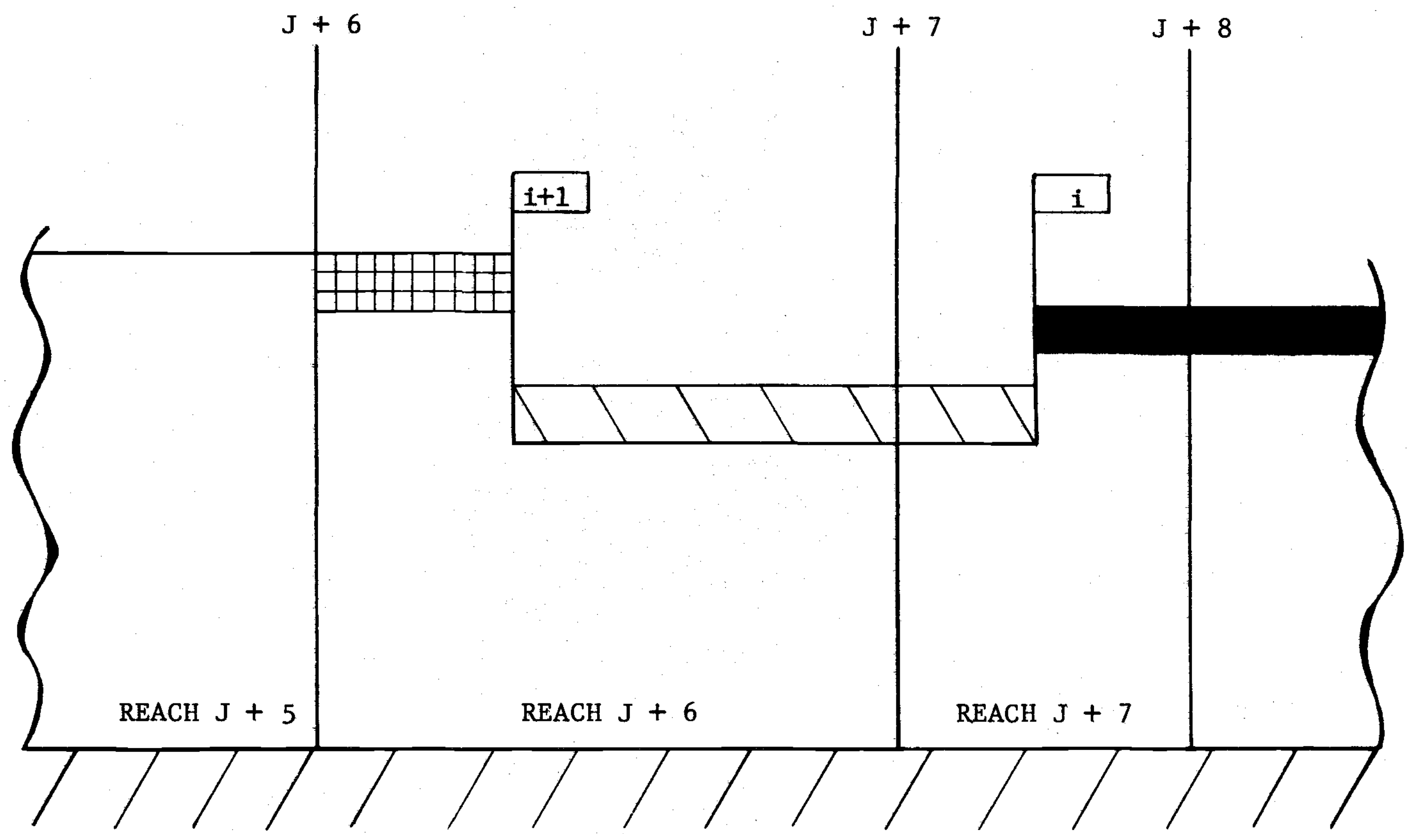

Figure 13. Flow Dependent Approach to Channel Advection

method that would allow decisions on withdrawals to be made very Δt --the time period for updating the mass flows and river temperatures.[20]

Time Dependent Approach

The time dependent approach to treating advected flows is presented in Figure 14. Assume again that the only place along the river where advection is of concern is at station J+6. In this particular scheme, the flows at J+6 become redefined whenever either of the following conditions prevail:

1. The leading edge of a flow period parcel, i.e., a flag, is at station J+6; or,
2. A new time period (Δt) begins.

Figure 14 shows that the flow parcels are double-dimensioned; that is, a numeric prefix is used along with the flow period identifier, i.

All parcels released from the dam have the prefix 1 and, thus, the entire parcel retains its identity throughout the total length of the river under consideration.

When the leading edge of parcel 1i in Figure 14 reached station J+6, it was flagged with the identifier, 2i. When a new

[20] It is not being argued that the first approach is necessarily inferior to the second approach. Rather, the first approach is not appropriate for the type of system under consideration.

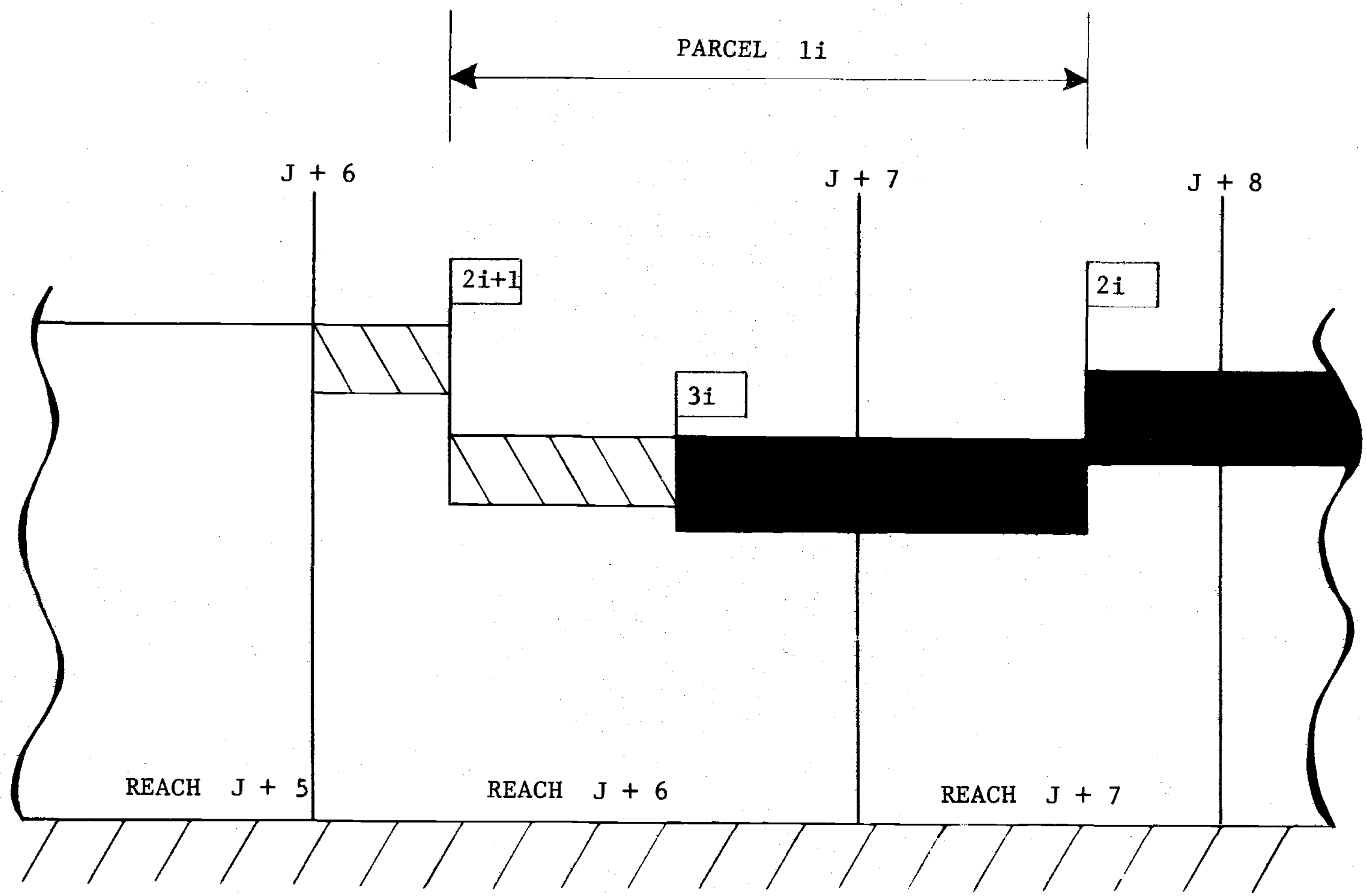

Figure 14. Time Dependent Approach To Channel Advection

time period was encountered at J+6, the flag marked 3i appeared. Thus, the flow period 1i has been broken into subperiods 2i and 3i. The shaded area to the right of flag 3i represents the withdrawal for time period t. When flag 3i was at station J+6, a decision was made to alter the withdrawal rate for time t+1, as represented by the lined area to the left of flag 3i.

The solution process for the hydraulic parameters remains unchanged, with one exception. Downstream from the point of advection, the subperiods are analyzed independently. Although each flow period 1i is flagged at the dam every Δt hours, this is not the case at the point of advection. Thus, the time available for any subperiod to penetrate into a reach may range from zero to Δt, necessitating an additional time dimension in this particular approach.

A third possibility for treating advection would be the use of lateral flow equations which are continuous functions of space (longitudinal distance) and time. It was felt that this approach was not appropriate for two reasons. First, the flows consist of discrete parcels. Second, insufficient information and data exist to warrant refinements based on continuous functions at this stage of model development.

River Temperature Submodel Algorithm

The entire theoretical procedure as it appears in the computer submodel, RIVER, may be reviewed with the aid of the flow charts in Figures 15 and 16.

In Figure 15, the algorithm begins with the identification of subroutine RIVER at statement one. The process moves to statement two, where it is determined if advection occurs in the upcoming reach. If this is a reach where advection may occur, the process moves to a decision function, QADV, at statement three. Because time is held constant in the solution sequence, the magnitude of a withdrawal does not have to be specified prior to the computer run. The decision to make a withdrawal and the magnitude of the withdrawal are determined within QADV. These decisions may be functions of the water temperature and streamflow at any station, or functions of any other time dependent activity.

Once these decisions are made, the process moves to statement four, where the reach averages for the specific hydraulic parameters are made for flow period mi, where m is the subperiod numerical identifier, and i is the flow period originating at the dam every day (Δt). The reach averages for the present time period are then found as noted in statement five.

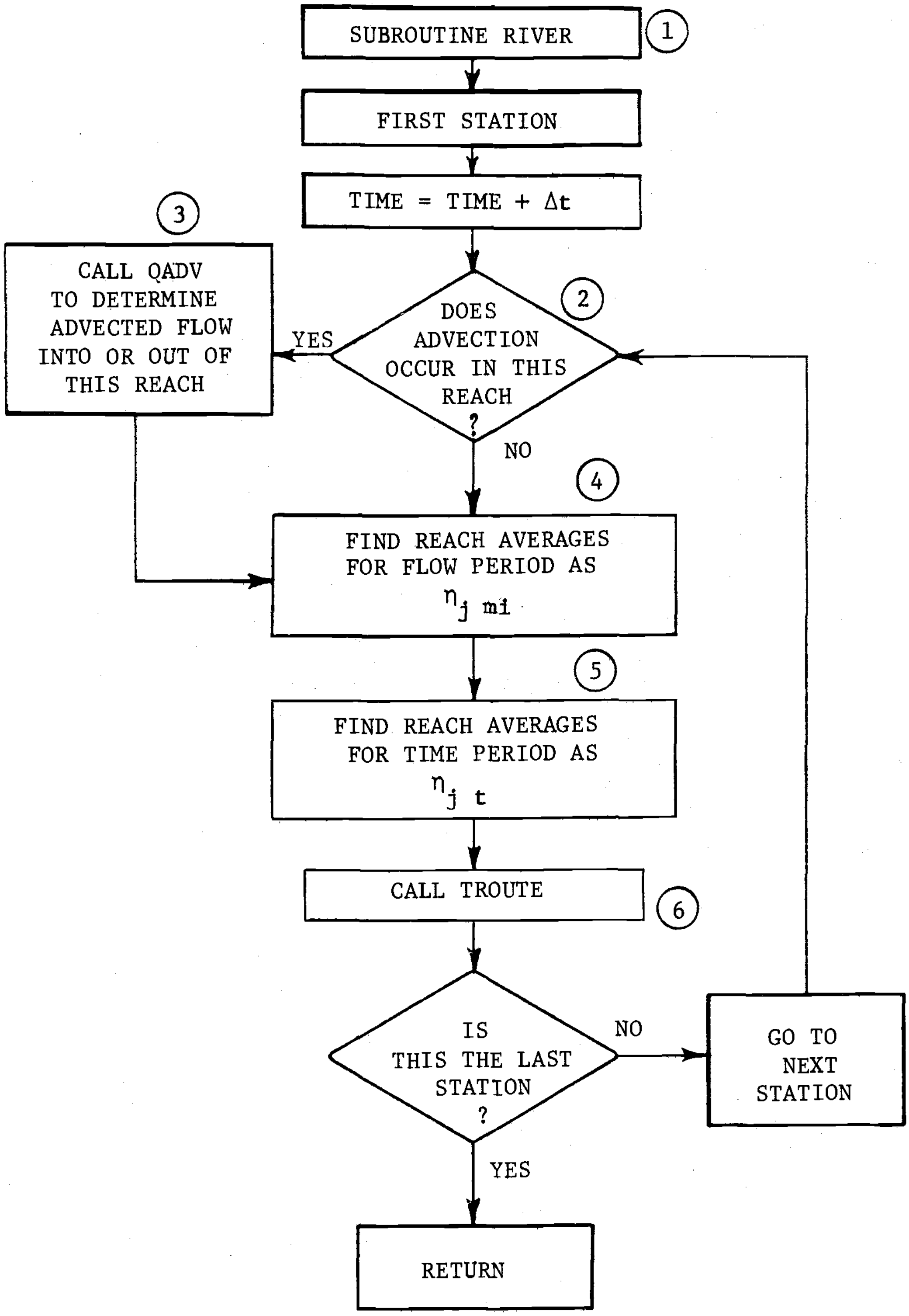

Figure 15. Flow Diagram of River Computer Submodel, RIVER

TROUTE, a submodel at statement six, is called for updating the river water temperatures. The algorithm then moves to the next river station or returns to the main computer program, ROUTR.

Figure 16 is a flow chart of the submodel TROUTE. This component routes the flagged parcel at station J to the previous upstream station, counting backward in space and time (statement one). Submodel TEMPDS, statement two, solves for the water temperature at the upstream station for that particular point in time. If the answer to the question at statment three is "no", the parcel is routed downstream (forward counting) until the next time period is encountered (statement four) and the water temperature is again found at this point in space and time by statement two. The process continues until the parcel (flag) is back at the same station, and likewise, the same time period in which TROUTE was called from RIVER.

Validation of RIVER Submodel

Validation of this component was not a particularly simple task, and the analysis remains incomplete. However, the following discussion demonstrates that the component is capable of predicting maximum daily river temperatures which exhibit the same magnitudes and trends as do historically recorded river temperatures at the Holley dam site.

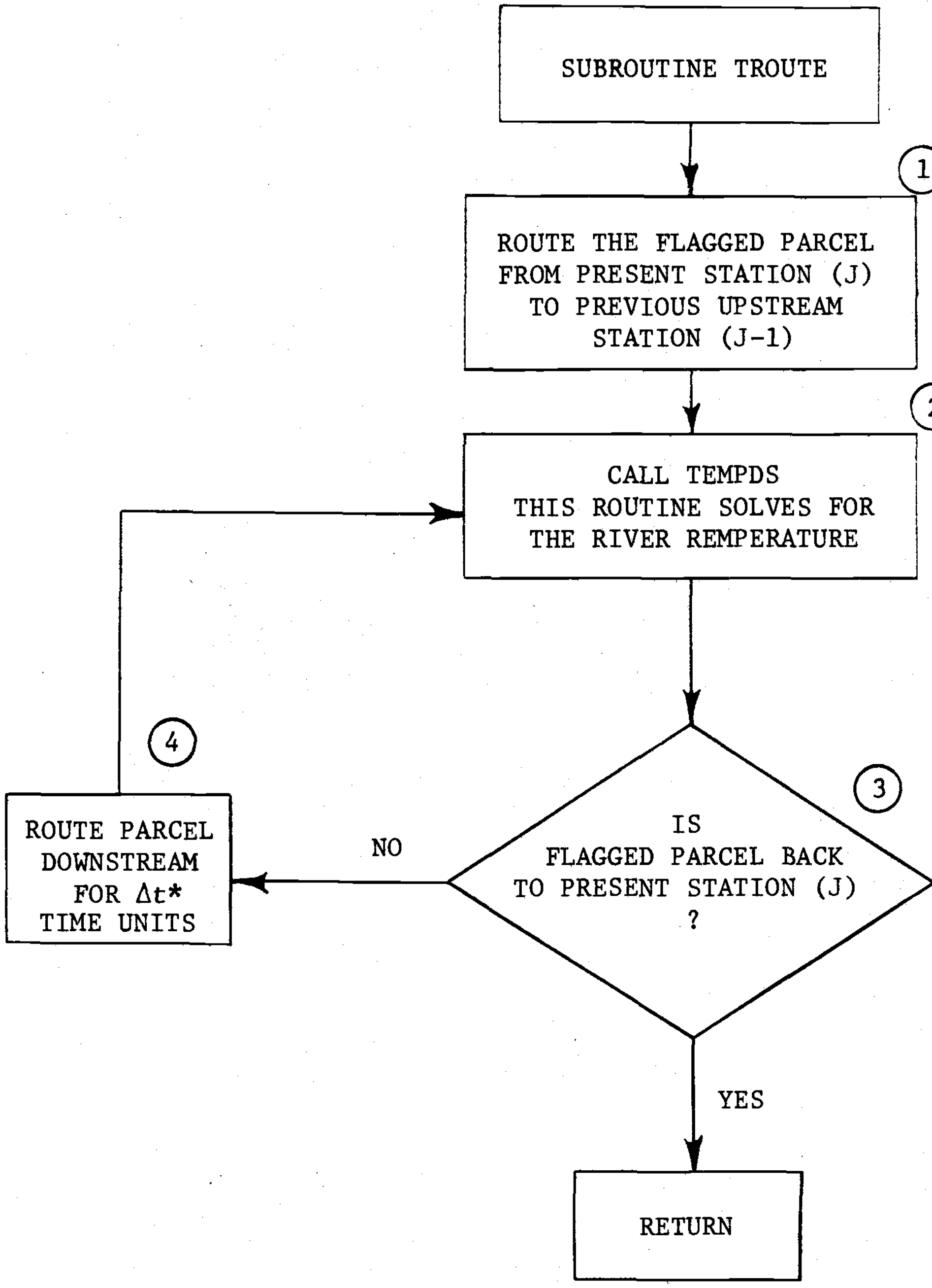

Figure 16. Flow Diagram of River Temperature Computer Submodel, TROUT

There are two continuous temperature recording gauges located on the Calapooia River. One is at the Holley dam site; the other is located approximately 42 river miles below Holley near the confluence of the Willamette and Calapooia Rivers at Albany, Oregon. Channel cross-sections and hydraulic characteristics are available for only approximately 24 river miles immediately below the gauge at Holley. Thus, without additional investment to acquire information on river temperatures and channel characteristics immediately upstream from either Holley or Albany, validation of the ability of the downstream component to predict river temperatures can, at best, be a gross approximation.[21]

Validation Procedure

The validation procedure employed compared the maximum daily three-hour temperatures predicted in the 24-mile reach below Holley with the historically recorded daily maximum temperatures at Holley for the month of June, 1967. A number of trial runs were made in which different coefficients for each run were varied to test the sensitivity of the submodel. At the beginning of each day (mid-night) the streamflow at Holley was set equal to the identical

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[21] Recall that the river temperature in the present time period is a function of the temperature in the previous time period.

historical streamflow for that particular day. During this validation analysis, the river temperatures at Holley were always set between 56°F and 60°F. The algorithm was operated for one month, and the maximum daily three-hour river temperature predicted in the 24-mile reach below Holley was then recorded. It was found that the component was quite sensitive to relatively small changes in the shading factor and the evaporation coefficient.

The evaporation coefficient has historically been a troublesome addend to the heat flux equation. Reviews of the literature indicate that this coefficient may have been used implicitly in the past as a catch-all error term in estimating the total heat flux. If, for instance, predicted water temperatures are not as desired, this coefficient is usually changed to bring the predictions into line. Although different values of the evaporation coefficient were tried in this study, the final value chosen for this component was identical to the value used in the reservoir component.

The procedure for selecting a realistic shading factor was similar to that used in selecting the evaporation coefficient. The value of the shading factor eventually chosen for the downstream component was ten percent. That is, only 90 percent of the hourly solar radiation over the entire 24-mile stretch of river was assumed to penetrate the stream-side shading and actually strike the water surface.[22]

Validation Results

The results of this validation are presented in Figure 17. The two curves in Figure 17 follow the same trends; the peaks and valleys are nearly congruent, except for results of June 4 and June 26. The mean of the historical daily maximum water temperatures at Holley for June 1967 was 68°F. This value is identical to the mean of the daily three-hour maximum temperatures predicted in the 24-mile stretch below Holley.

Variations between the curves in Figure 17 were expected. The values of the meteorological parameters used in this study (cloud cover, wind speed, relative humidity, air temperature) were obtained from historical data recorded at the Salem, Oregon, airport. Salem, located in the Central Willamette Valley, is approximately 38 miles north-northwest of Holley, at an elevation of 160 feet mean sea level (msl). Holley, also located in the Willamette Valley, lies on the western slope of the Cascade Foothills at an elevation of 527 feet msl. Because of the distance between the two locales, as well as the

[22] The value of the reservoir shading factor in this study was zero.

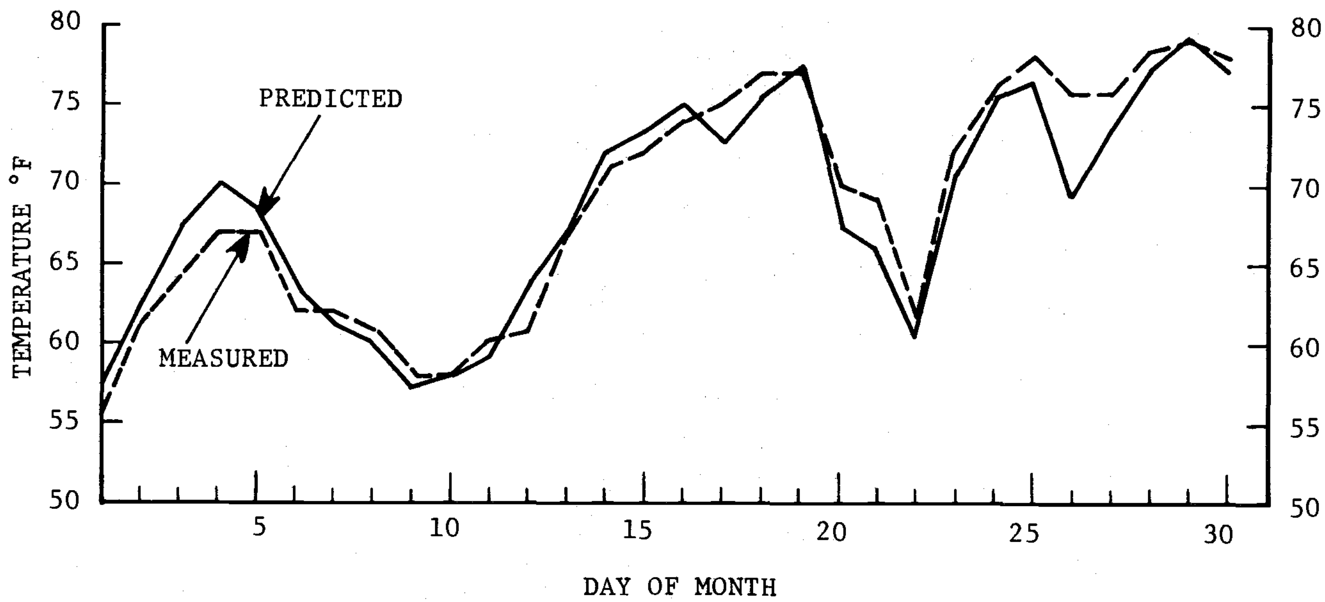

Figure 17. Calapooia River Temperatures Measured At Holley and Predicted Below Holley, June, 1967.

difference in elevations, it is to be expected that the values of the meteorological parameters recorded at Salem would not exactly match those which simultaneously occurred at Holley but were not measured. Measurements of wind speed appear to be the most difficult of all the meteorological parameters to be transferred. It has been shown that among these parameters, wind speed exhibits the most spatial variability, vertically as well as radially (Sellers, 1965; Brooks and Carruthers, 1953).

Another possible reason, and, in this case, a source of error, accounting for variation between the two curves in Figure 17 is related to the depth factors at each river station. Data limitations constrained the definition of the depth factor to streamflows between 90 cfs and 1000 cfs. Approximately one-third of the streamflows at Holley in June 1967 were below 90 cfs with the lowest flow being 63 cfs on June 30. However, the water speeds, depths, and water temperatures associated with the flows below 90 cfs behaved as would be expected.

A final reason for the expected variability between the measured and predicted water temperatures in Figure 17 involves the absolute depths at the downstream stations. This may be explained by referring to the water temperature prediction equation:

$$T_{t+1} = T_t + \frac{H}{\rho c_p D} \cdot \Delta T \tag{5-40}$$

Assume that all the variables on the right side of Equation (5-40) are known for two successive river stations. If the values of T_t, H, ρ, c_p, and Δt relative to one station are identical to those characterizing the other station, then the water temperature (T_{t+1}) at each site becomes a linear function of the value of the reach depth (D) used for that site. If the value of the reach depth relative to the two stations differed, then the water temperatures would be expected to differ in proportion to the respective depths. None of the stations downstream from Holley has the same depth factor as the Holley station. Therefore, water temperatures at these downstream stations would be expected to differ from the water temperatures at Holley for identical streamflows.

It has been shown that some of the variations between the measured and predicted water temperature curves in Figure 17 can be both expected and explained. For purposes of this study, the downstream temperature component is thus assumed to be validated.

This chapter has presented the structure of the computer submodel, RIVER, and its various components. In Chapter VI, a demonstration of applications of the general water temperature model which combines the reservoir and river temperature models to the proposed Holley project will be presented.

VI. ANALYSIS AND RESULTS

Introduction

This chapter presents the analysis and results of applying the general water temperature model to the proposed Holley Reservoir on the Calapooia River in Oregon. In the first section of this chapter, the general reservoir withdrawal requirements are presented along with a discussion of the computer algorithm used to simulate reservoir operations.

The second section investigates the flexibility of the river temperature submodel and how sensitive the predicted downstream temperatures are to changes in the reservoir withdrawal strategies. The objective was to reduce the infinite number of possible reservoir withdrawal strategies to only a few to be used in the final intensive analysis.

The final section of this chapter consists of a demonstration of how the general water temperature model could be used to predict the reservoir and downstream river water temperature effects of the proposed Holley Dam.

Reservoir Operation

Diversions and Withdrawal Requirements

The reservoir operation procedure as planned by the Corps of Engineers is designed for the reservoir to be at a low level prior to and during the months of high precipitation and runoff. The regulation rule curve for the proposed Holley reservoir is shown by the solid line in Figure 18. The conservation storage season is between February 1 and April 30 of each year. During this period, water would be stored at a fairly rapid rate, as indicated by the slope of the rule curve. Ideally, the pool would remain at a constant elevation until September 1, then lowered throughout a three-month period until November 30. Lowering of the pool is in anticipation of the major flood season, which, in the Pacific Northwest, occurs from December 1 to January 31.

Multiple Outlets. The arrows along the right-hand vertical axis in Figure 18 indicate the elevations of the three outlets used in this study. Although the Corps of Engineers has included cost estimates for a multiple outlet facility, the selection of the elevations and plans for the design of the outlet facility have yet to be finalized.[23] For this reason, the outlet elevations represented by the two bottom

[23] Personal communication, Ken Johnson, Planner, Portland District, U.S. Army Corps of Engineers, 1973.

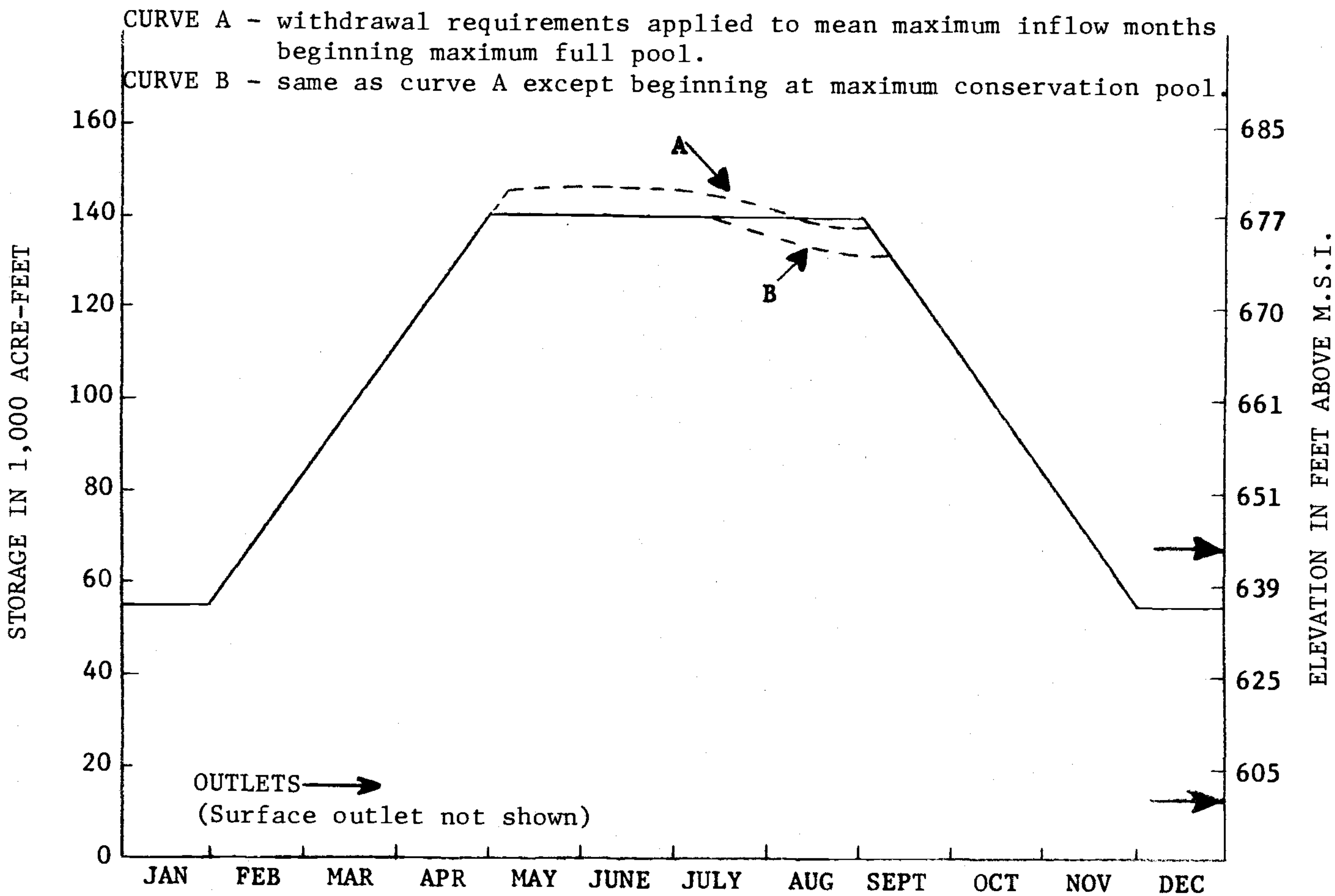

Figure 18. Rule Curve For Proposed Holley Reservoir

Source: U. S. Army Corps of Engineers (1970).

arrows (solid lines) in Figure 18 were arbitrarily chosen as 550 feet and 643 feet above mean sea level (msl) respectively. It was reasoned that the bottom most outlet would be located in the hypolimnion region of the reservoir. It was likewise reasoned that the middle outlet (643 msl) would be located along the thermocline. The third outlet used in this study has a variable elevation and is represented by the broken lined arrow in Figure 18. The centerline for this outlet was assumed to always be located between five and nine feet below the reservoir surface elevation and in the region of transition between the epilimnion and thermocline. Although the outlet works might never be constructed in this fashion, it was felt that this specification, especially that of a variable surface discharge elevation, would allow for maximum flexibility in developing reservoir withdrawal strategies.

Reservoir Withdrawal Requirements. The Bureau of Sport Fisheries and Wildlife has determined the reservoir outflow and water temperature requirements which are necessary for downstream anadromous fish enhancement. These minimum releases and associated maximum temperatures are presented in Table 4.

Table 4. Required minimum flows and maximum allowable temperatures of releases from Holley Reservoir.

Month	Release Flow (cfs)	Release Temperature °F
January	130	55
February	130	55
March	130	55
April	130	55
May	130	55
June	160	60
July	160	60
August	160	60
September	130	60
October	130	55
November	130	55
December	130	55

Source: U.S. Army Corps of Engineers (1970).

Irrigation Requirements. Decisions would be made at the reservoir whether or not to add downstream irrigation requirements to the required minimum flows. The total seasonal irrigation requirements are estimated by the Corps of Engineers to be 21,400 AF. This total requirement would be diverted within the first ten miles below the dam with the majority of it, approximately 80 percent of the total, being diverted at 9.8 miles below the dam. For this reason, in the analysis the total irrigation diversion is allowed to occur at this point on the river.

The Corps of Engineers estimates that, on the average, 20 percent of the irrigation diversions will be return flows that can be used to augment the streamflows downstream from the points of diversion. For simplicity, this study assumes that all return flows occur at the point of diversion and have the same temperature as the streamflows in the channel. Again for simplicity, this reduces the irrigation requirements by 20 percent. It is realized that this assumption could bias the results; however, the bias cannot be predicted at this time. For instance, it is not known if the regulatory agencies will consider the return flows to be part of the minimum requirement or in addition to the minimum. It was expected, however, that the 20 percent reduction in irrigation requirements would have a negligible effect on the reservoir temperature profile. The results, discussed later, do, in fact, bear this out.

The total monthly irrigation requirement as given by the Corps of Engineers in acre-feet is shown in Table 5. Also shown is 80 percent of this amount (the requirement used in this study) in acre-feet and cubic feet per second.

Table 5. Monthly irrigation requirements, Calapooia River project.

Month	Acre-Feet	Acre-Feet x 0.8	Cubic Feet Per Second
May	2000	1600	26
June	5650	4520	77
July	6090	4872	80
August	5050	4040	62
September	2610	2088	35

Source: U.S. Army Corps of Engineers (1970).

Other Diversions. Municipal and industrial (M & I) diversions are expected to occur above and below the major point of irrigation diversion. The Corps of Engineers estimates that the maximum daily M & I diversion upstream from the major irrigation diversion would be 0.6 million gallons per day (mgd) by the year 2010. This is equivalent to less than one cubic foot per second per day (sfd) and, therefore, will not be considered in this study.

The maximum daily M & I diversion below the point of major irrigation diversion expected by the year 2010 is 5.4 mgd or about eight sfd. The expected monthly maximum is about six sfd. Because of their relatively small magnitudes, these M & I diversions will not be considered, either.

Adherence to Rule Curve

The rule curve (solid line, Figure 18) implies that from May 1 to September 1, the reservoir level would ideally remain unchanged. One reason for desiring a full reservoir with constant elevation during the summer months involves the recreational potential of the reservoir. Empirical evidence indicates a significant inverse relationship between reservoir elevation (a proxy for beach length) and recreation user days (Gibbs, 1973; Johnston and Pankey, 1968).

One of the project purposes associated with the Holley development relates to recreation. If the reservoir is lowered in the summer months in order to increase downstream water quality for anadromous fish benefits, the recreational attendance and subsequent benefits at the reservoir would be reduced. Therefore, it is important to investigate the possibility of releasing the minimum flow requirements and the resulting probabilities of maintaining a constant surface elevation.

While it might be desirable to maintain a constant reservoir elevation during the summer months, this can be accomplished only if the reservoir withdrawals are equal to the reservoir inflows.[24] However, there has not been one year since continuous streamflows were recorded (1936) at Holley that the rule curve could have been adhered to, assuming the minimum required withdrawals shown in Table 4.

In fact, even by assuming the reservoir was at maximum full pool (145,000 AF) on June 1, and that the streamflows for the maximum mean flow months for June, July, and August were to occur in succession as illustrated in Table 6, the rule curve could not be adhered to, and a deficit would occur in August (curve A, Figure 18).

[24] Evaporation and seepage losses and local inflows are not considered in this study.

Table 6. Streamflows of maximum mean flow months of June, July, and August at Holley with minimum required withdrawals.

Month and Year of Maximum Mean Flow	Maximum Mean Flow - cfs	Minimum Withdrawal Requirements - cfs
June 1937	534	160
July 1969	153	160
August 1968	106	160

Source: U.S. Army Corps of Engineers (1970).

The chances of equaling or exceeding these monthly means of June, July, and August in any one year are approximately one in 50,000.[25]

Curve B in Figure 18 shows the deficit (difference between dotted line and solid line) that would occur by using the mean maximum months inflow hydrology but preventing the reservoir to fill to 145,000 AF, that is, maintaining the rule curve at 139,000 AF, the maximum conservation pool.

Thus, while adherence to the rule curve might appear desirable, it probably is not possible, given the downstream requirements. The submodel (DECISN) discussed below will detail how the decisions relating to rule curve adherence are arrived at and implemented within the general model framework.

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[25] Assuming independent events, 35 years of record, and using the widely accepted Weibull method for estimating probabilities for recurring hydrologic events, the results are

$$\left(\frac{1}{35+1}\right)^3 = 2.14 \times 10^{-5} = \frac{1}{46,729}.$$

DECISN Submodel

Information on required withdrawals for anadromous fish and irrigation is used with daily reservoir inflows and the rule curve in a decision routine diagrammed in Figure 19, to select the magnitude of daily reservoir withdrawal and the fraction of this withdrawal to be released from each outlet. This routine, DECISN, is called at the beginning of each day (midnight) by the submodel RESERV.

Statement one, Figure 19, determines if this is the first time DECISN has been called in this particular computer run. If the answer to statement one is "yes", the reservoir level at the end of the previous day is determined by the rule curve at statement two. The rule curve is described mathematically in the computer program as a function of the day of the year. Thus, the storage in acre-feet is determined by the day of the year. A quadratic equation, in which the independent variable is storage, is then used to determine the elevation in feet of the reservoir surface. This equation was estimated by applying a least-squares fit to data provided by the Corps of Engineers.

At statement three, the total reservoir inflow for the upcoming day is added to the existing reservoir volume. This implies that at midnight the reservoir inflow for the upcoming day is known with certainty. This is not an unreasonable assumption for the summer

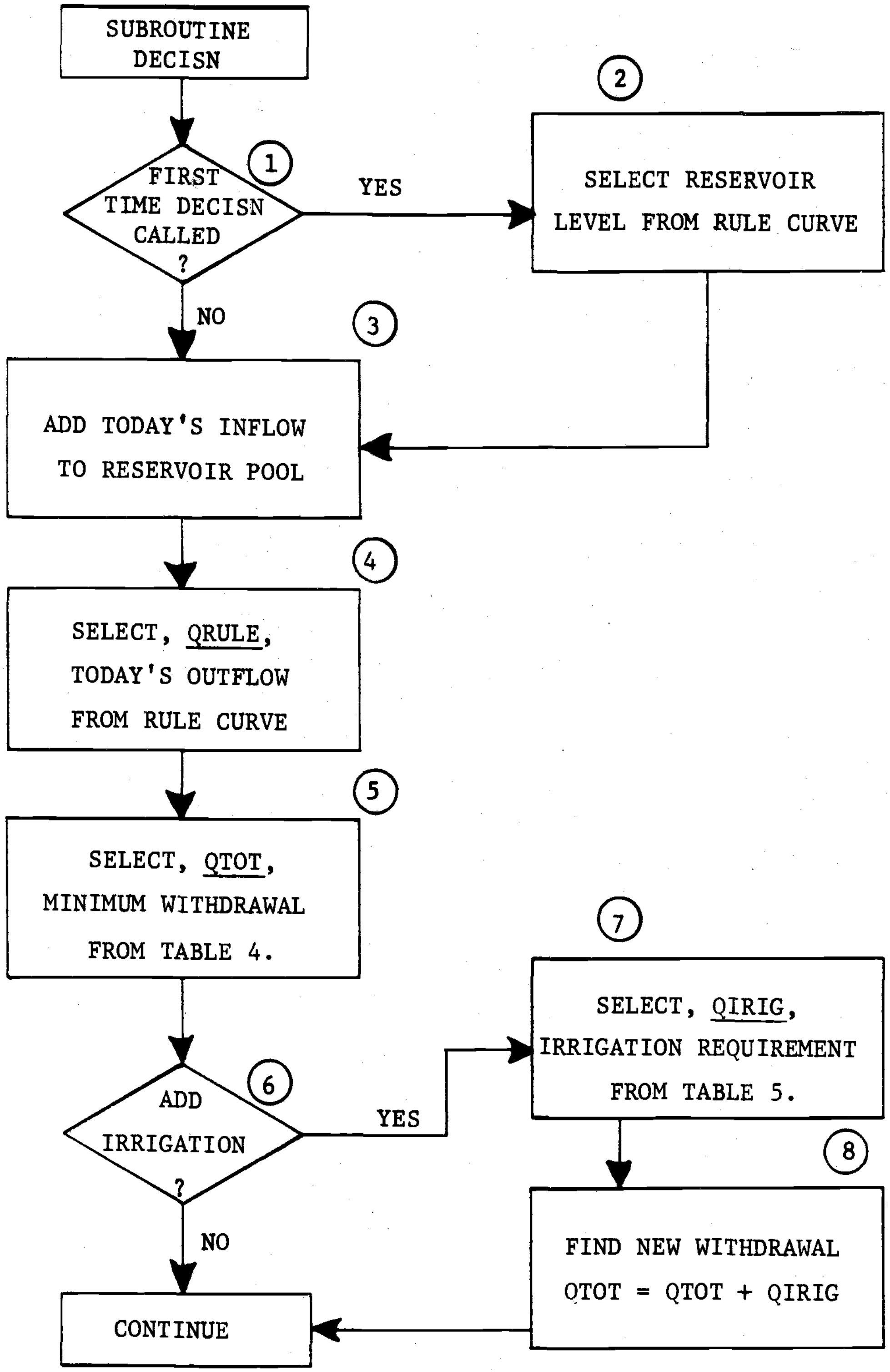

Figure 19. Flow Diagram of Reservoir Decision Computer Submodel, DECISN.

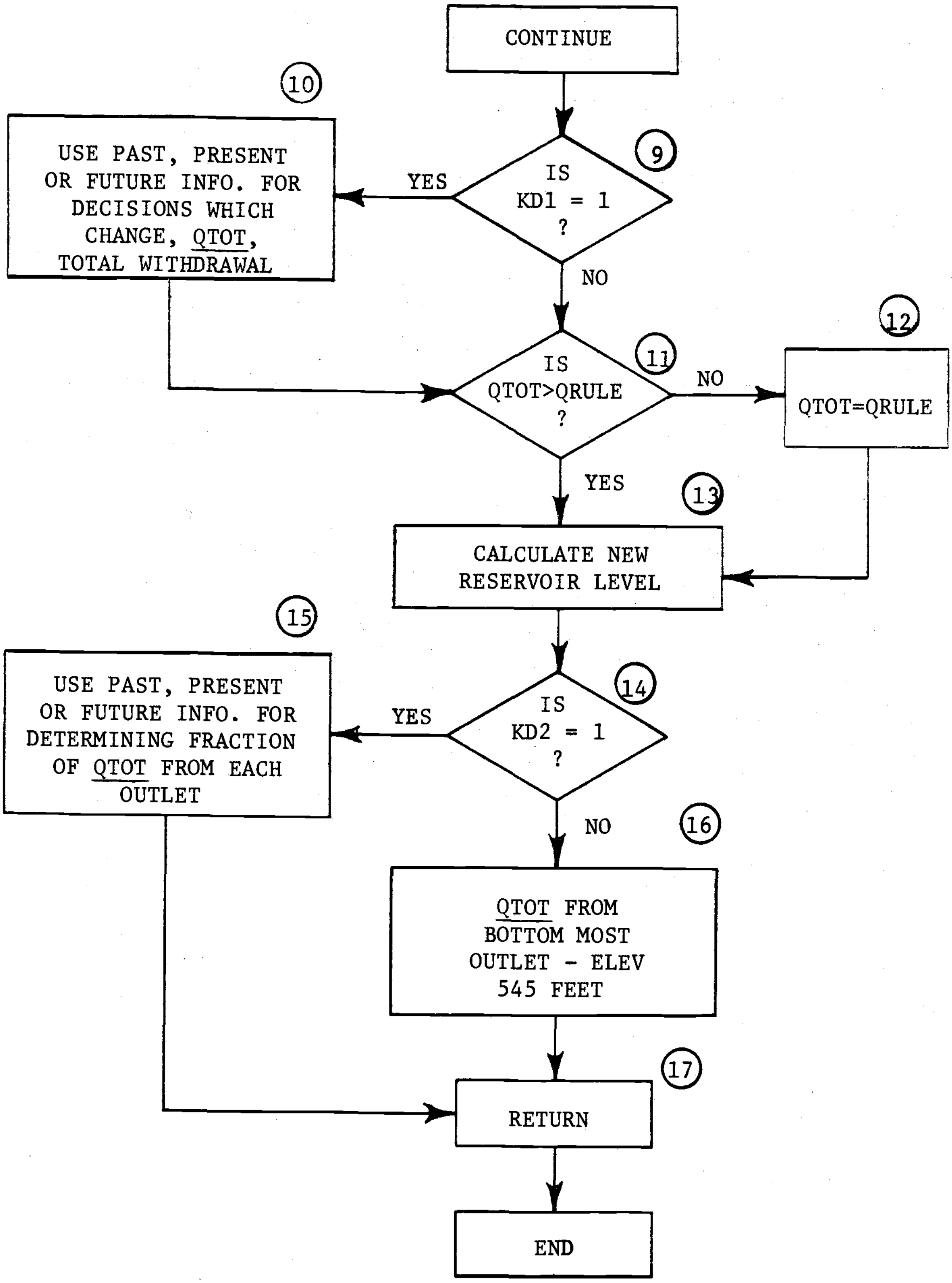

Figure 19 (Cont). Flow Diagram of Reservoir Decision Computer Submodel, DECISN.

months in the Pacific Northwest.

Statement four determines QRULE, the reservoir outflow as governed by the rule curve. This is accomplished by subtracting the desired reservoir volume for this particular day from the reservoir volume determined at statement three. This is given by Equation (6-1):

$$\text{QRULE} = \text{Today's desired volume} - \left(\begin{array}{l}\text{yesterday's volume} \\ + \text{ today's inflow}\end{array}\right) \tag{6-1}$$

The minimum required withdrawal, QTOT, is then determined from Table 4 at statement five.

Statement six is a switch which determines if irrigation is being considered in the present computer run. If irrigation is a consideration, statement seven selects the required amount from Table 5.

The value of the required total outflow, QTOT, is updated at statement eight to include the minimum withdrawal requirement determined at statement five plus the irrigation requirement of statement seven.

Statement nine is another switch which allows for additional changes in the total required outflows, QTOT, if the value of the integer constant, KD1, is equal to one. The routine would then move to statement ten, which allows the user to make additions to or deletions from the required outflow, QTOT, via an assortment of decision rules. It was hypothesized that the use of various forecasts and decision rules could possibly alter the shape of the reservoir temperature profile and/or change the resulting downstream water temperature distributions. The results of these simulations will be presented in a later section.

The magnitudes of QTOT and QRULE are compared at statement 11. QRULE is the desired outflow as determined by the rule curve, whereas QTOT is the sum of minimum requirements, irrigation requirements, and the amount, if any, added to or deleted from statement ten. If QRULE is greater than QTOT, then QRULE is released (statement 12) and adherence to the rule curve is maintained. If QRULE is less than QTOT, then QTOT is released and adherence to the rule curve is sacrificed in favor of downstream water use. The reservoir outflow is never allowed to exceed 3500 cfs, the estimated channel capacity, unless the reservoir elevation exceeds the maximum rule curve elevation.

Statement 13 calculates the new reservoir volume by subtracting the present day's withdrawal (QTOT) from the present reservoir volume.

The switch at statement 14 sends the routine to statement 15 if the integer constant KD2 is equal to one. Statement 15, similar in structure to statement ten, determines the fraction of total withdrawal (QTOT) to be withdrawn from each of the three outlets.

If the integer KD2 is not equal to one, the routine goes to statement 16 which requires that the total outflow be withdrawn from the bottom most outlet (elevation 550 msl).

The submodel DECISN then returns to the calling program, RESERV.

Reservoir and Channel Withdrawal Strategies

The discussion of Figure 18 indicated a high probability of the proposed reservoir showing a withdrawal deficit every summer for the life of the project. Because of this and the suggested inverse relationship between reservoir elevation and recreation user days, it would appear reasonable to first investigate the expected river water temperatures associated with the minimum temperature and withdrawal requirements (Table 4) from the proposed Holley Dam. This release strategy would require the least drawdown of the reservoir and would provide the expected maximum amount of recreational user days resulting from the project. If the minimum release requirements appear to provide optimum downstream temperatures, then these requirements could possibly be reduced in order to increase the recreational user days. If the proposed minimum releases do not appear to provide optimum downstream conditions, then a search would be in order to determine which release strategy, if any, would provide optimum downstream conditions.

Channel Requirements

The downstream flow and temperature requirements are not necessarily the minimum release requirements from Table 4. The flow requirements in the downstream channel as recommended by the Corps of Engineers are shown in Table 7.

Table 7. Minimum recommended flows--Calapooia River.

	Minimum Flows in cfs		
Location	1 Sept. - 31 May	1 June - 15 June	16 June - 31 August
Holley Dam site	130	160	160
Brownsville	130	110	90
Sodom Ditch	75	75	50
Parallel to Sodom Ditch	55	35	40
Below Sodom Ditch	100	100	100

Source: U.S. Army Corps of Engineers (1970).

A better perspective of the streamflow and temperature requirements may be possible by referring to Figure 20, a map of the Calapooia River Basin.

Note that station one is at the proposed Holley Dam site. Station eight is at the point of irrigation diversion, approximately ten miles from station one. Station 12 is just upstream from Sodom Ditch and represents the downstream limits of the spawning area which begins at the dam site. The recommended maximum temperature is 55°F for spawning and hatching of fall and spring Chinook,

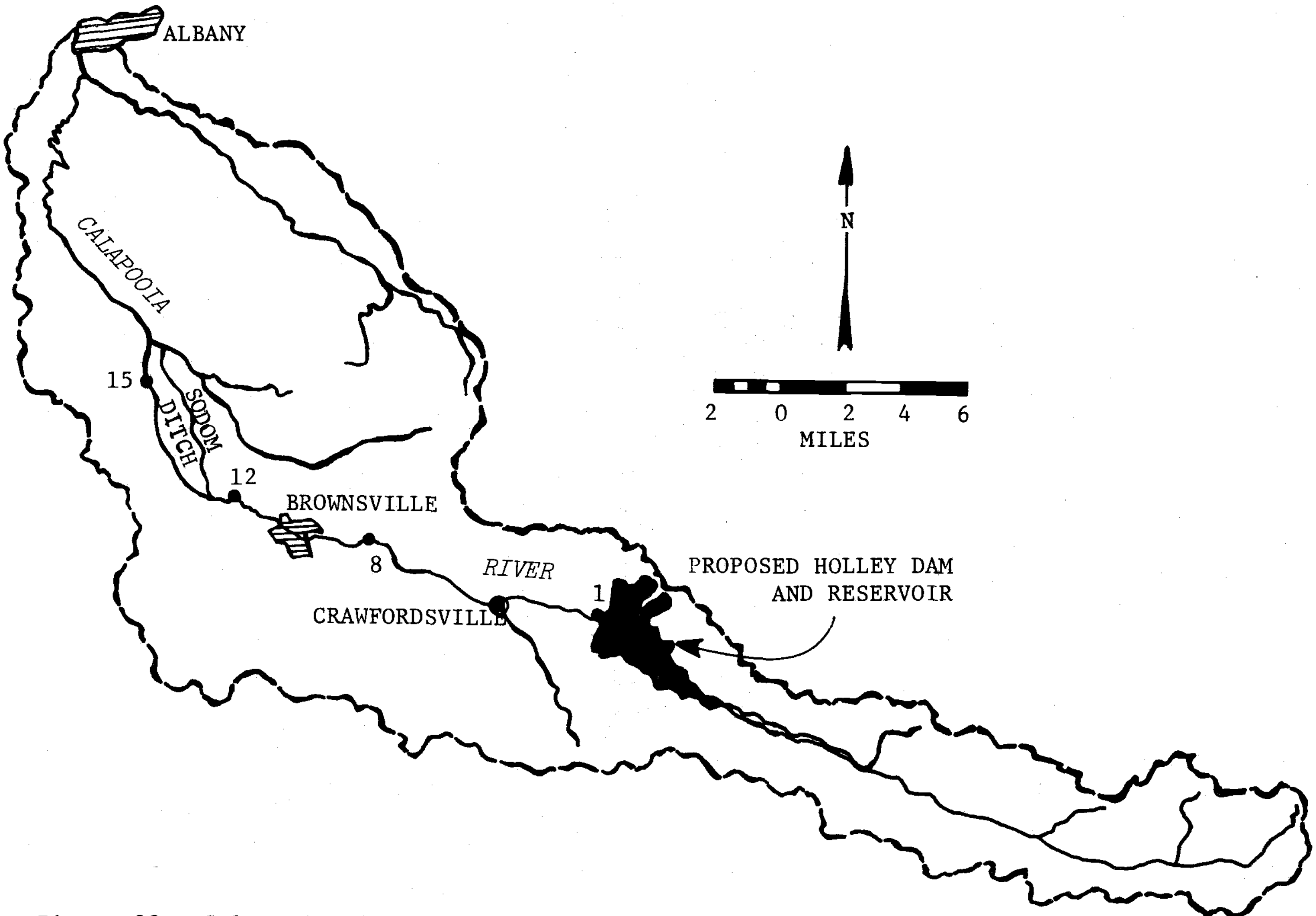

Figure 20. Calapooia River Basin With Numbered Downstream Stations.

silver salmon, and winter steelhead which would occur in every month of the year in this reach of the river (Avey, 1972).

Downstream from station 12, the river splits into two channels: the western or main channel and Sodom Ditch. The minimum required streamflows in Sodom Ditch and the main channel parallel to it (Table 7) approximate the historical averages for the Calapooia River at Holley and Albany. For example, the mean streamflow for the month of July at Holley and Albany in 1967 and 1968 was 54.2 cfs, whereas the required flows in Sodom Ditch and the paralleling channel would be 50 cfs and 40 cfs respectively for the month of July.

The daily maximum river temperatures at Holley and Albany during July for 1967 and 1968 rarely differed by more than 2°F. Therefore, one estimate of the expected daily maximum river temperatures at station 15, shown by Figure 20 as 24 miles below the dam site, would be the mean of the historical daily maximums at Holley and Albany. This mean for July 1967 and 1968 was 75.3°F; temperatures as high as 83°F were recorded. These figures should be quite representative of the expected maximum river temperatures in both Sodom Ditch and the approximately eight miles of channel parallel to it under the minimum flow conditions specified in Table 7.

The recommended maximum river temperature for salmonid migration, as noted in Chapter I, is 60°F. Therefore, in lieu of any computer analysis, the above information suggests problems for anadromous fish attempting to penetrate this stretch of the river during periods of relatively low streamflow and high river temperatures.

Except for the validation in Chapter V, streamflows within this range (55 cfs) were not treated in this model because the depth factors were not defined at these low flows. In the computer analysis, it was assumed that there was no diversion of flow into Sodom Ditch. Thus, the maximum river temperatures predicted at station 15 are representative of approximately twice the flows that would be present at that station, and may appear to be low estimates.

Simulated River Temperatures

The river temperatures simulated for all stations except 15 are believed to be realistic estimates of what would occur if the proposed project were constructed.

River Temperatures and Minimum Requirements. Figure 21 shows the predicted river temperatures at various mileage stations below the proposed dam for different release flows at 3:00 P.M. on June 18, 1967. The two curves emanating from the vertical axis imply that there are two different reservoir release strategies presented in this graph. Because both curves begin at the same point (about 46° F) the outflow temperatures associated with the two release strategies are identical, creating an equal basis for comparison.

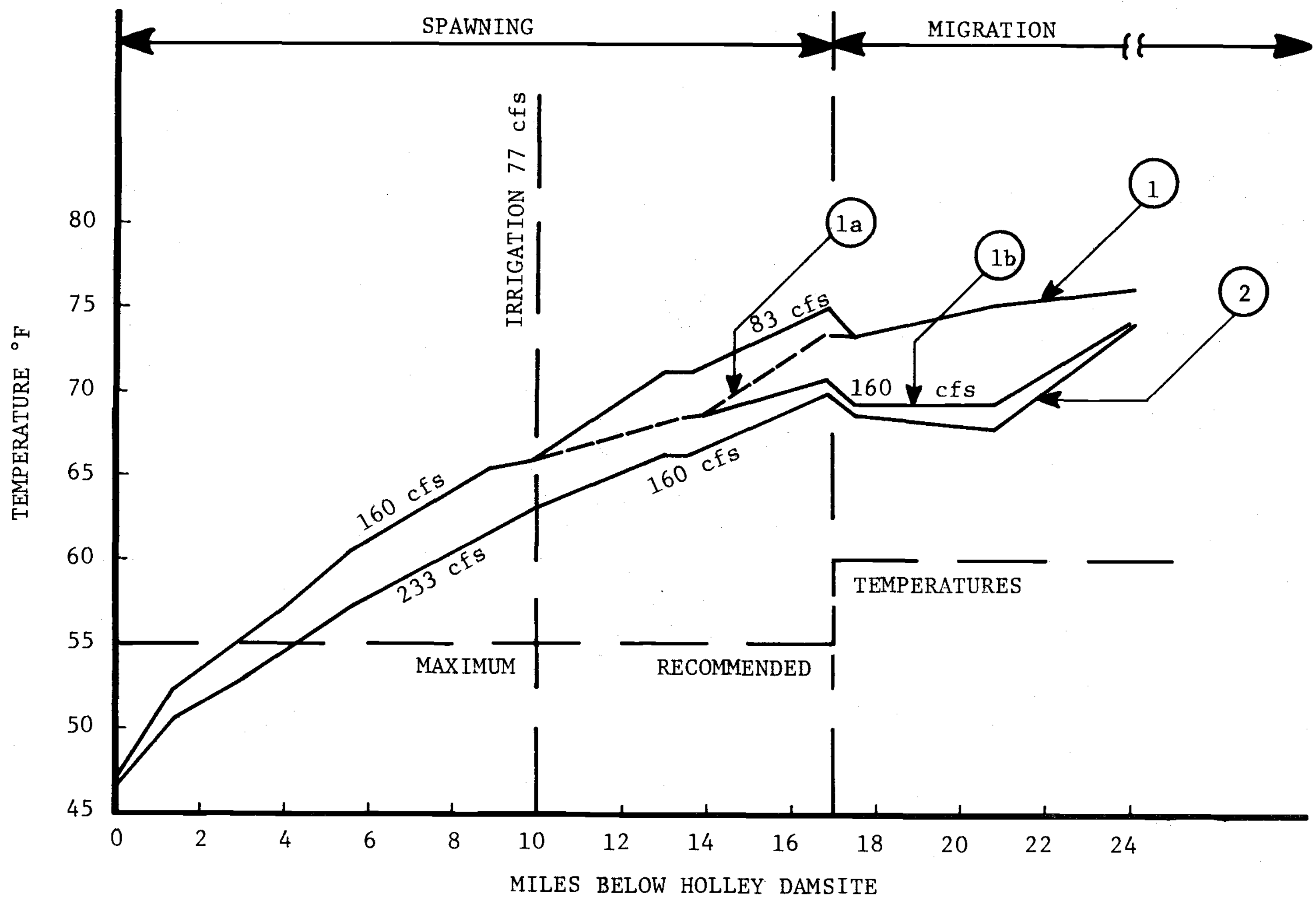

Figure 21. Predicted Calapooia River Temperatures for Various Release Strategies at 3:00 P.M. on June 18, 1967.

Curve one, as shown in Figure 21, represents the predicted river temperatures resulting from the minimum withdrawal requirements (160 cfs) from the proposed dam with the irrigation requirement (77 cfs) being withdrawn from the channel. The reservoir withdrawal associated with curve two is the minimum requirement plus the irrigation requirement which is withdrawn from the channel 9.8 miles below the dam site. Curve 1B is curve one without the allowance for irrigation withdrawals.

Within the spawning area, the release strategy associated with curve two predicts river temperatures up to 5°F below those predicted by curve one. However, the river temperatures associated with both curves one and two for this particular day are above the recommended maximums for both the spawning and migration.

Curve 1A is the result of a decision strategy applied to the channel at the point of irrigation withdrawal. This strategy involved the restriction of irrigation withdrawals during critical periods of low streamflows and high river temperatures.

The strategy allowed the total daily irrigation requirement to be withdrawn from the channel but at different rates throughout the day. The common and justifiable assumption associated with an irrigation project is that the user of irrigation water must virtually be guaranteed a certain daily withdrawal, subject to his water right and a safe minimum standard required in the stream. If the user does not have this near guarantee of water, he may not be willing to risk as much, or any, of his capital in an irrigable farming venture. Here there is a similarity to the production of salmon in the channel, that is, a sequential dependence on the water requirement. The requirement in time period t may be a function of the water used for production in time period $t-1$, as well as the stage of crop development.

If the water were not withdrawn for irrigation, it might decrease downstream temperatures, and, by the increase in channel flow volume, decrease the density of fish. The decrease in fish density apparently may be quite important in reducing fish mortalities. For example, <u>columnaris</u> is a disease which becomes well-established in young salmon as water temperatures reach 63°F-64°F where the salmonids are crowded together. It becomes extremely virulent at temperatures greater than 70°F (Lantz, 1970). Thus, the river channel would have a greater mass of water in lieu of irrigation withdrawals which implies a decrease in fish density (<u>cet</u>. <u>par</u>.). This, coupled with the possibility of lower water temperatures from the greater mass of water, would be expected to decrease downstream mortalities.[26]

The particular strategy resulting in curve 1A of Figure 21 is as follows. Computations of predicted river temperatures were made,

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[26]Personal communication, James Lichatowich, Project Leader, Oregon Wildlife Commission, Corvallis, Oregon, 1974.

using three-hour intervals. Also, at these three-hour intervals, a decision was made whether or not to withdraw irrigation water. If the river temperature at the point of withdrawal was predicted to be greater than 62°F and the time of day was less than 6:00 P.M., then no irrigation withdrawal was allowed. However, when the withdrawal was allowed to resume, a greater quantity of water was removed to ensure the daily irrigation allotment. This strategy resulted in dropping the predicted river temperatures by as much as 3°F within the spawning area of the river.

While this approach would allow withdrawal of the total irrigation requirement in any one day, it might also decrease the streamflow below the recommended minimum. This, in turn, could result in an increase in river temperature the next day due to the "low flow" block of water moving downstream into the migration area of the river. This was observed to be the case. However, these downstream increases in river temperatures were predicted to be less than 1°F. The reasons for this will be discussed in detail later.

An alternate approach would be either to allow irrigation withdrawals at the required rate during any three-hour (Δt) interval, or not to allow the withdrawals. The deficit in withdrawals that may occur in one day could be compensated for the next day by increasing reservoir withdrawals.[27]

Other Release Strategies. Figure 22 presents the predicted river temperature below the proposed dam for five different release flows that would have been expected on June 18, 1967, at 3:00 P.M. The required irrigation withdrawal rate from the channel was allowed to occur continuously throughout the day.

There are several important points here that should be emphasized. First, the water temperatures associated with the different release strategies are again identical, creating an equal basis for comparison.

The second point of interest involves the predicted river temperatures associated with different magnitudes of streamflow. It was suggested earlier that one means of reducing downstream temperatures might be to increase the magnitude of reservoir withdrawals, filling the channel with a greater mass of water which would decrease the rate of temperature rise (cet. par.). Indeed, decisions based on this premise may have been made in the past in the operations of present Oregon reservoirs. For instance, the fishery resource agencies may suggest release strategies beneficial to

[27] It is realized that there would be costs imposed on the irrigators, and that these are not addressed here. The intention is to demonstrate how this type of model could be used in the planning as well as operational stage of such a project.

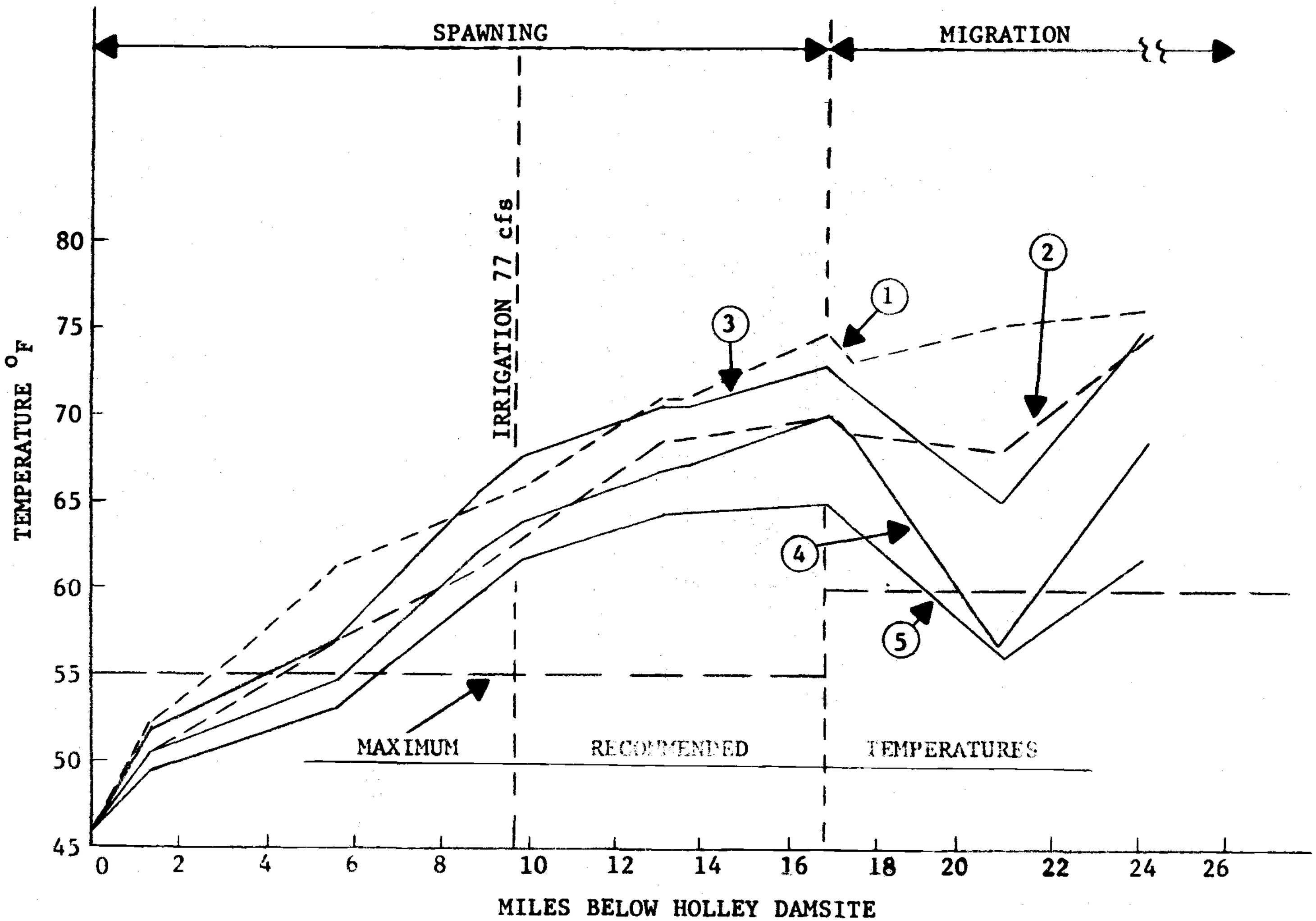

Figure 22. Predicted Calapooia River Temperatures for Five Release Strategies at 3:00 P.M. on June 18, 1967.

anadromous fish for the Corps of Engineers to use as guidelines in Oregon reservoir operations. During extremely warm periods these agencies may request the Corps to deliver as much cold water as can be spared.[28] In light of the limited hydraulic information usually available to these agencies, such a request would appear reasonable to help alleviate high stream temperatures below a dam. However, the evidence displayed in Figure 22 and discussed below suggests that a decision of this type may not always yield lower downstream river temperatures.

All curves in Figure 22 represent an irrigation withdrawal of 77 cfs at 9.8 miles below the Holley Dam site. Curve one results from the minimum reservoir withdrawal (160 cfs) while the reservoir withdrawal associated with curve two (237 cfs) represents the minimum plus irrigation requirements. Curve two always lies below curve one as hypothesized and as was shown earlier in Figure 21.

Curve three represents the predicted river temperatures resulting from a withdrawal rate of 300 cfs. However, within the spawning area, curve three (300 cfs) does not fall below curve two (237 cfs). In fact, in the stretch of river where the irrigation withdrawal occurs, curve three (300 cfs) even rises above curve one (160 cfs). The result of nearly doubling the reservoir withdrawal rate

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[28]Personal communication, James Lichatowich, Project Leader, Oregon Wildlife Commission, Corvallis, Oregon, 1974.

while maintaining the same outflow temperature would be to increase the downstream river temperatures. Also, within the spawing area, the river temperatures predicted with strategy four (400 cfs withdrawal) are essentially identical to those predicted by strategy two (237 cfs).

This apparent anomaly can be explained, in part, by reference to Equation (6-2) and Figure 23.

$$T_{t+1} = T_t + \frac{H}{\rho c_p D} \cdot \Delta t \tag{6-2}$$

The average reach depth (D) is the key parameter in Equation (6-2). The average depth in some of the river reaches at streamflows of 300 cfs and 400 cfs is less than the average depth at lower streamflows. This is because the river channel in certain areas, especially in the upper river within the spawning area, has a configuration comparable to that profiled in Figure 23.

The "summer" or low flow channel has a relatively steep side slope resulting in a relatively large average depth (cross-sectional area divided by the water surface width). The portion of the "winter" or high flow channel outside the summer channel has a side slope not nearly so steep as that of the summer channel. Therefore, the average depth of the winter channel is less than that of the summer channel. If the only variable that changes in Equation (6-2) between

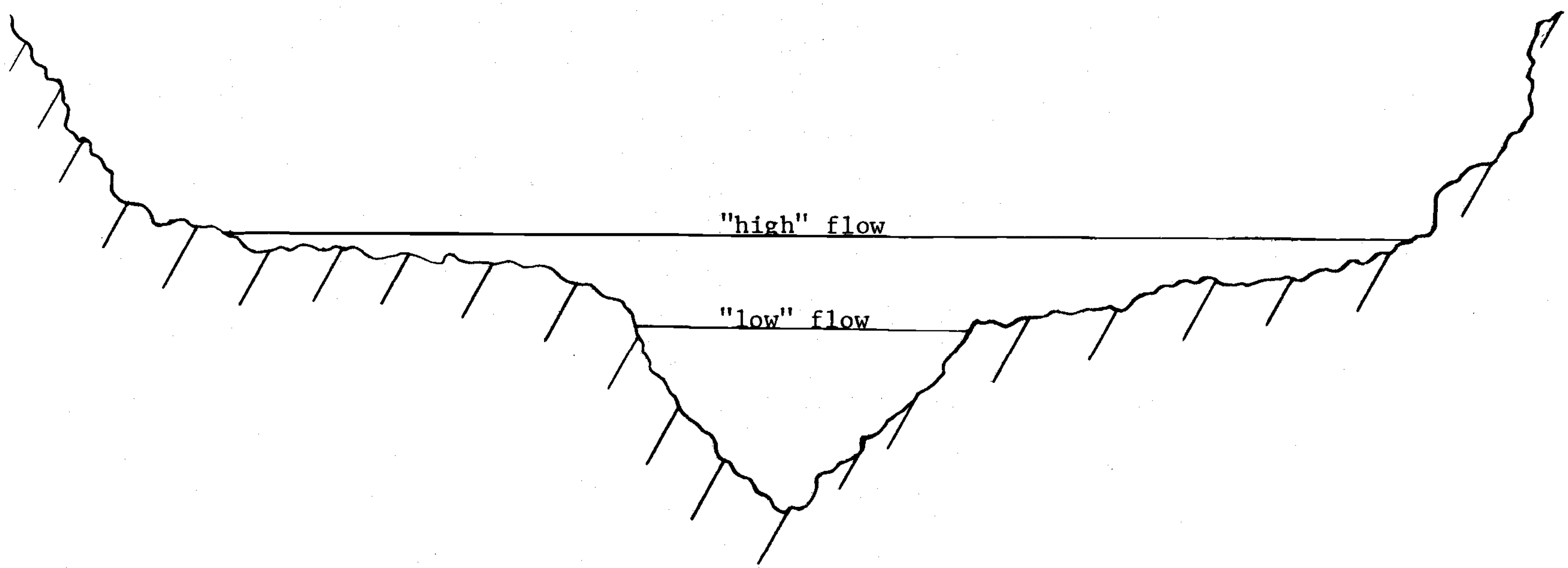

Figure 23. Typical Channel of Upper Calapooia River

two different streamflows is the average depth (D), then the amount of change in river temperature between these two streamflows for a specific river reach is merely a representation of the ratio of the average depths.

In reference to the predicted river temperatures associated with the curves of Figure 22, reliance on Equation (6-2) as stated does not provide a truly accurate picture. The total amount of travel time between river stations would be expected to differ for different flow rates. This travel time is a function of the average water speed which is defined as the average flow rate divided by the average cross-sectional area. By increasing the flow rate from 237 cfs to 300 cfs, not only did the average depth decrease, but the average cross-sectional area increased at a rate greater than the flow rate increased, which caused the average water speed to decrease. This is consistent with conventional hydraulic theory as the high flow in Figure 23 might be expected to have a speed less than the low flow speed, due to the sizable increase in the wetted perimeter indicating a decreasing hydraulic radius[29] and increasing bottom friction.

Reference again to Figure 22 shows that little, if any, improvement in lowering the downstream temperatures occurs from

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[29] Hydraulic radius is defined as cross-sectional area divided by the wetted perimeter.

increasing the reserovir withdrawal until 500 cfs (curve five) is released. Within the spawning section of the river, this release strategy is predicted to provide up to 5°F lower temperatures over the minimum plus irrigation releases (237 cfs, curve two) and up to 10°F lower temperatures than predicted by curve one representing the minimum release requirements (160 cfs). Subsequently, it is suggested that the release strategies associated with curve one (160 cfs), curve two (237 cfs), and curve five (500 cfs) are the three release strategies, in terms of total withdrawal, that require further investigation over a wider range of hydrometeorological conditions.

The final point of interest in Figure 22 is that there are predicted dips in the curves at about 21 miles below the proposed dam. This is because downstream from the spawning area, the bottom gradient lessens, causing a decrease in water speed and an increase in depth. In addition, the water that would be released from the dam in the evening during periods of low solar radiation would have made its way into the deeper portions of the river before becoming exposed to periods of intense solar radiation the following day. Therefore, it is because of the greater river depths and the fact that the flows present just below the spawning area were not subjected to high solar radiation while in the shallow upstream portions of the river that these relatively low temperatures are predicted for this section of the river.

Demonstration of the General Water Temperature Model

One way to demonstrate the usefulness of this water temperature model is to compare its capability and predictions with those provided by the Corps of Engineers and the FWQA. The Corps of Engineers designated 1958 as representative of a warm year for the reservoir temperature analysis. As mentioned in Chapter I, however, the Corps performed no analysis on the downstream river temperatures. The FWQA also used 1958 data for both reservoir and downstream analysis.

Data Base

Data from 1958 were also selected for the reservoir temperature analysis in this particular study, although data for 1958 were not used in the downstream analysis. This was because the FWQA and Corps of Engineers studies utilized 1958 average weekly weather data from Albany, Oregon, whereas the meteorological data base for this study consisted of hourly observations from Salem, Oregon.

Because a relatively long series of reliable weather records (over 30 years) is available from Salem, Oregon, it first appeared desirable to utilize the entire time series through some averaging technique. However, searching for representative averages is quite futile if a realistic data set is the objective. An example should lend clarity to this point. The occurrence of a three- or four-day sequence of clear afternoon skies and high dry-bulb temperatures is not uncommon in the Willamette Valley during the summer months. Because these sequences appear every summer and represent extreme short-term conditions, they become important data sets in assessing the effects of the proposed Holley reservoir on water temperatures. Any averaging technique would smooth the data and essentially eliminate the extremes. The average daily or hourly meteorological effects would tend to be investigated, rather than the effects of, say, the series of daily or hourly data in an average meteorological year.

This suggests that a stochastic model of the meteorological parameters could be useful for generating subsequent short-term representative extremes. However, it has been shown that at least some of these parameters are jointly determined, and stochastic simulation of jointly determined meteorological parameters, not a simple task, requires considerable time and resources (Hogan, et al., 1973). Therefore, it was decided that the primary thrust of this research was to develop, and demonstrate the usefulness of, a general reservoir-river temperature model using representative years of historical meteorological and hydrologic data.

Data for the years 1967 and 1968 were used in this analysis for detailed investigation of both the reservoir and downstream temperatures. It would appear that 1967 is more representative of an extremely warm year than is 1958, the year used in the Corps and FWQA analyses, because of sustained periods of higher air temperatures and incident solar radiation in each of the three months of June, July, and August. Conversely, the summer months of 1968 are characterized by relatively cool air temperatures and high historical streamflows for the Calapooia River.

Reservoir Temperatures

Analysis for 1958. All computer runs in this study used April 15 as the first day of the run. While the maximum run length varied, no run was longer than 200 days, ending on October 31. Two 200-day computer runs were made using 1958 data. Both runs exhibited the following similarities: (1) adherence to the rule curve where possible; and (2) all withdrawals were from the bottom outlet (550 feet msl).

Representative temperature profiles for these two runs are shown in Figures 24 and 25, along with profiles determined by the Corps of Engineers and the FWQA. In both figures, curve A, determined by the model used in this study, is the result of daily withdrawals equal to the sum of the minimum outflow and irrigation requirements shown in Tables 4 and 5. The withdrawal strategies used in the Corps of Engineers and FWQA analyses were essentially the same as for the curve A analysis. Curve B in both figures is the

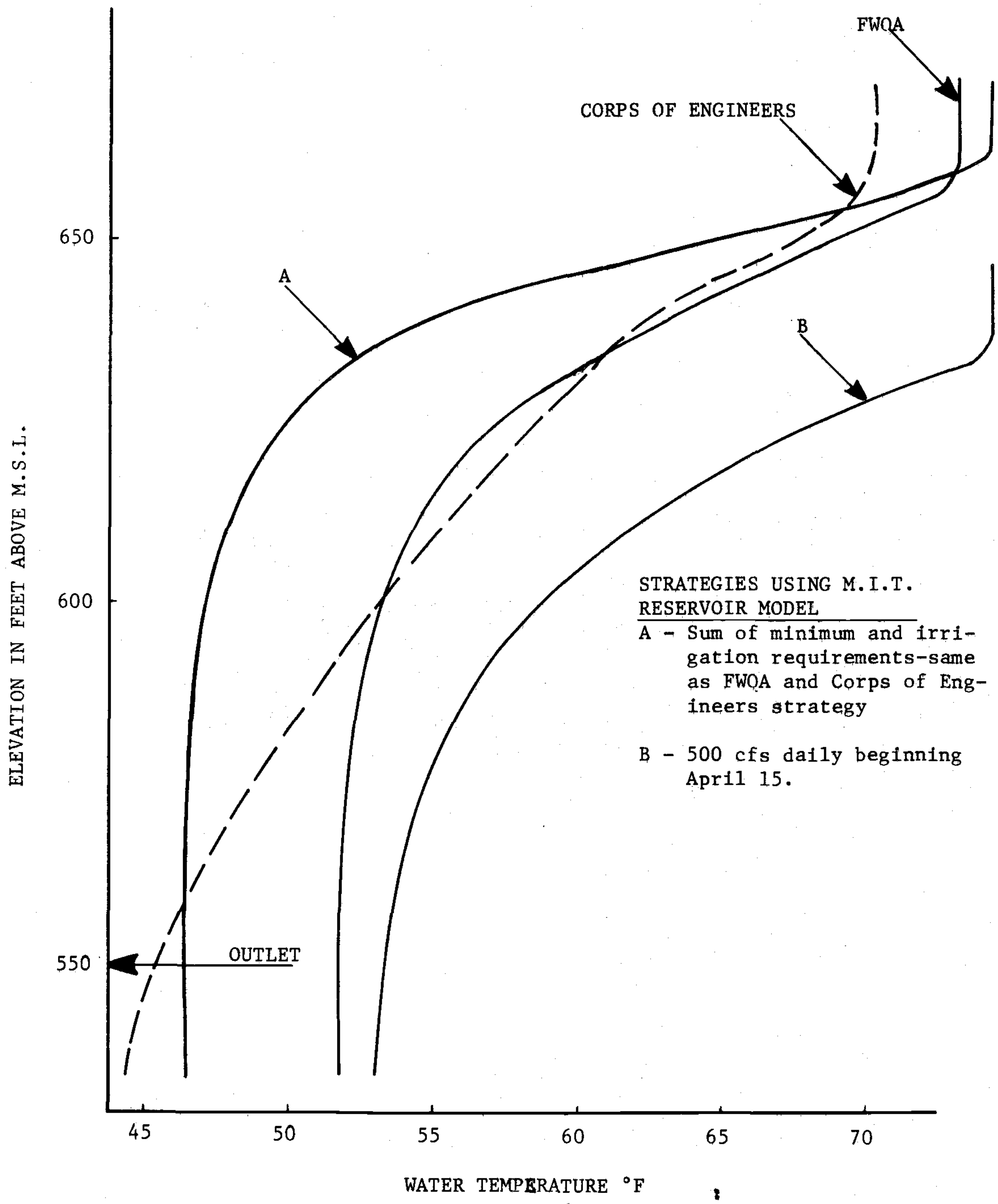

Figure 24. Predicted Temperature Profiles for Proposed Holley Reservoir on August 1, 1958.

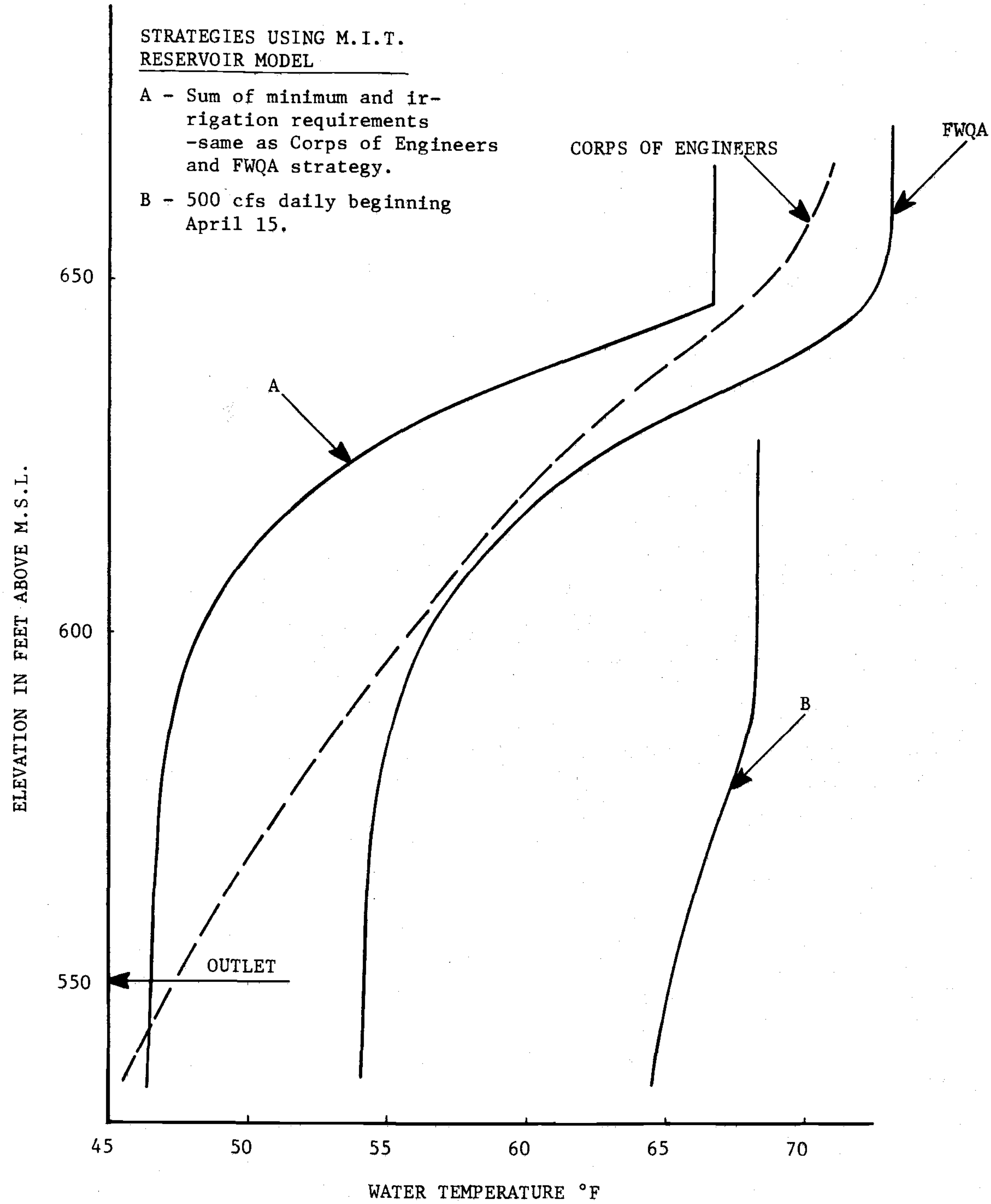

Figure 25. Predicted Temperature Profiles for Proposed Holley Reservoir on September 1, 1958.

result of using a daily withdrawal rate of 500 cfs.

The most noteworthy aspect of Figures 24 and 25 is that the strategy that resulted in curve A indicates that a greater mass of relatively cold water (45-50° F) would have been available on August 1 and September 1, 1958, than was predicted by either the Corps of Engineers or the FWQA analyses. The temperature gradient in curve A is also more pronounced than indicated by either the Corps of the FWQA analysis. The FWQA used the model developed by Orlob and Selna (1969), which uses a different mechanism than does the MIT model for exchanging heat between reservoir strata. This would, in part, explain the differences in the thermoclines.

The temperature gradient of the curve predicted by the Corps of Engineers analysis is less developed than the gradients of either curve A or the FWQA curve. It is not entirely clear how the Corps analysis was conducted, although it was based upon observed temperature profiles from existing reservoirs in the Willamette Valley.[30]

Curve B, Figures 24 and 25, indicates that a daily withdrawal strategy of 500 cfs could not be maintained under 1958 hydrometeorological conditions while adhering to the maximum allowable reservoir release temperature of Table 4. Between August 1 and September 1, curve B loses almost all semblance of representing a

[30] Personal communication, Ken Johnson, Planner, Portland District, U.S. Army Corps of Engineers, 1973.

strongly stratified reservoir with the outflow temperature on September 1 being greater than 67° F, whereas the recommended maximum from Table 4 is 60° F.

Analysis for 1967 and 1968. The temperature profiles generated by the reservoir submodel using 1967 and 1968 hydro-meteorological data were similar to those predicted by this submodel for 1958 data.

Figures 26 and 27 show reservoir temperature profiles for August 1 and September 1, 1967, under different withdrawal strategies. The profiles for 1968 are not shown because they closely resemble those predicted for 1967. The arrows along the vertical axis again indicate outlet elevations.

The results presented in Figures 26 and 27 indicate that the multiple outlet system would have virtually no effect on the temperature profiles for identical magnitudes of daily withdrawals. This essentially negates the previously stated hypothesis about significantly altering the thermocline and, hence, the amount of water available at any temperature for the proposed project.

The curve A-B-C in Figure 26 (curves A, B, and C are superimposed) represents different outlet strategies using the identical magnitudes of withdrawal--the sum of the minimum release requirements and irrigation requirements from Tables 4 and 5. The strategy for curve A was to release one-half the required outflow from

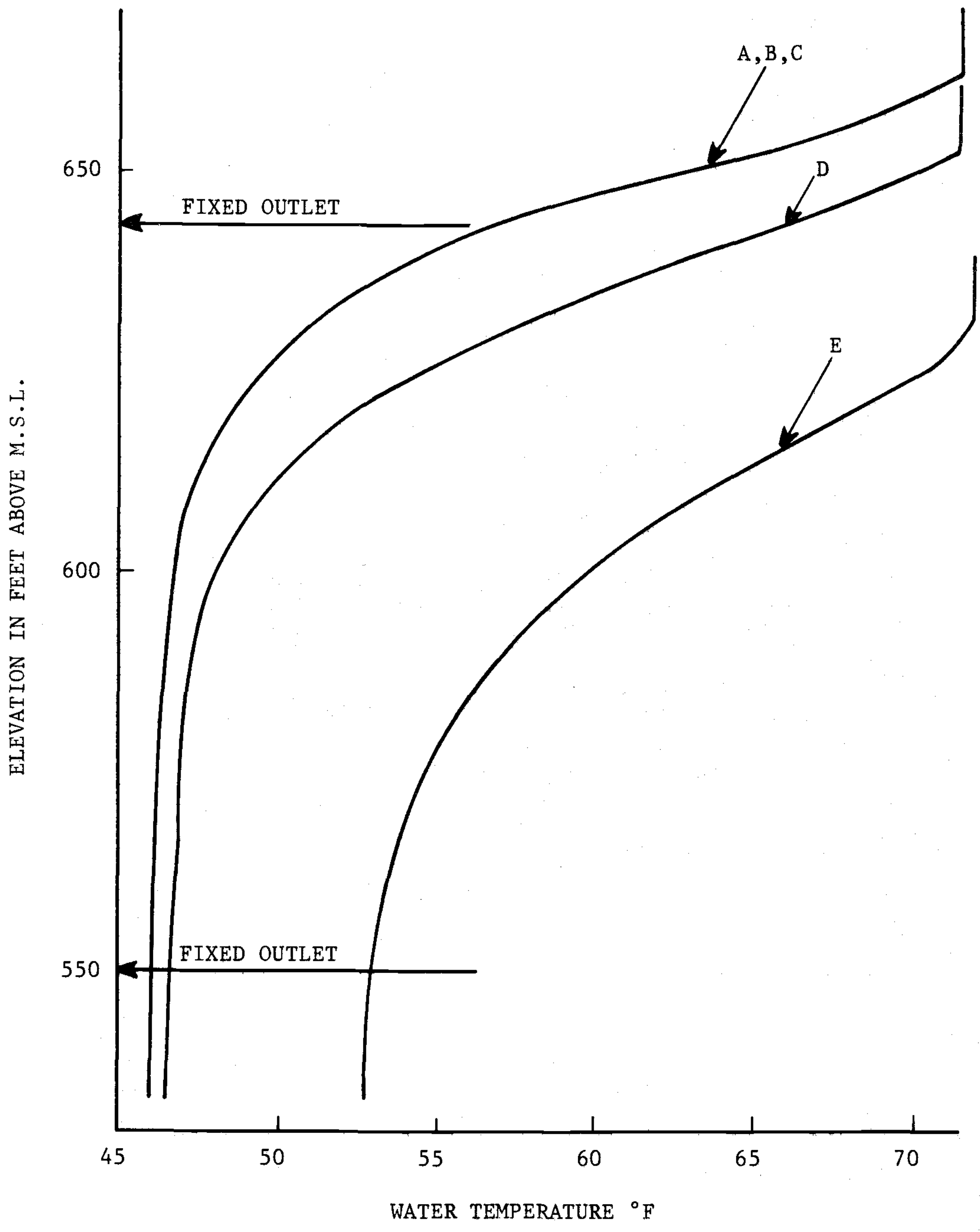

Figure 26. Predicted Temperature Profiles For Proposed Holley Reservoir on August 1, 1967 Under Various Release Strategies.

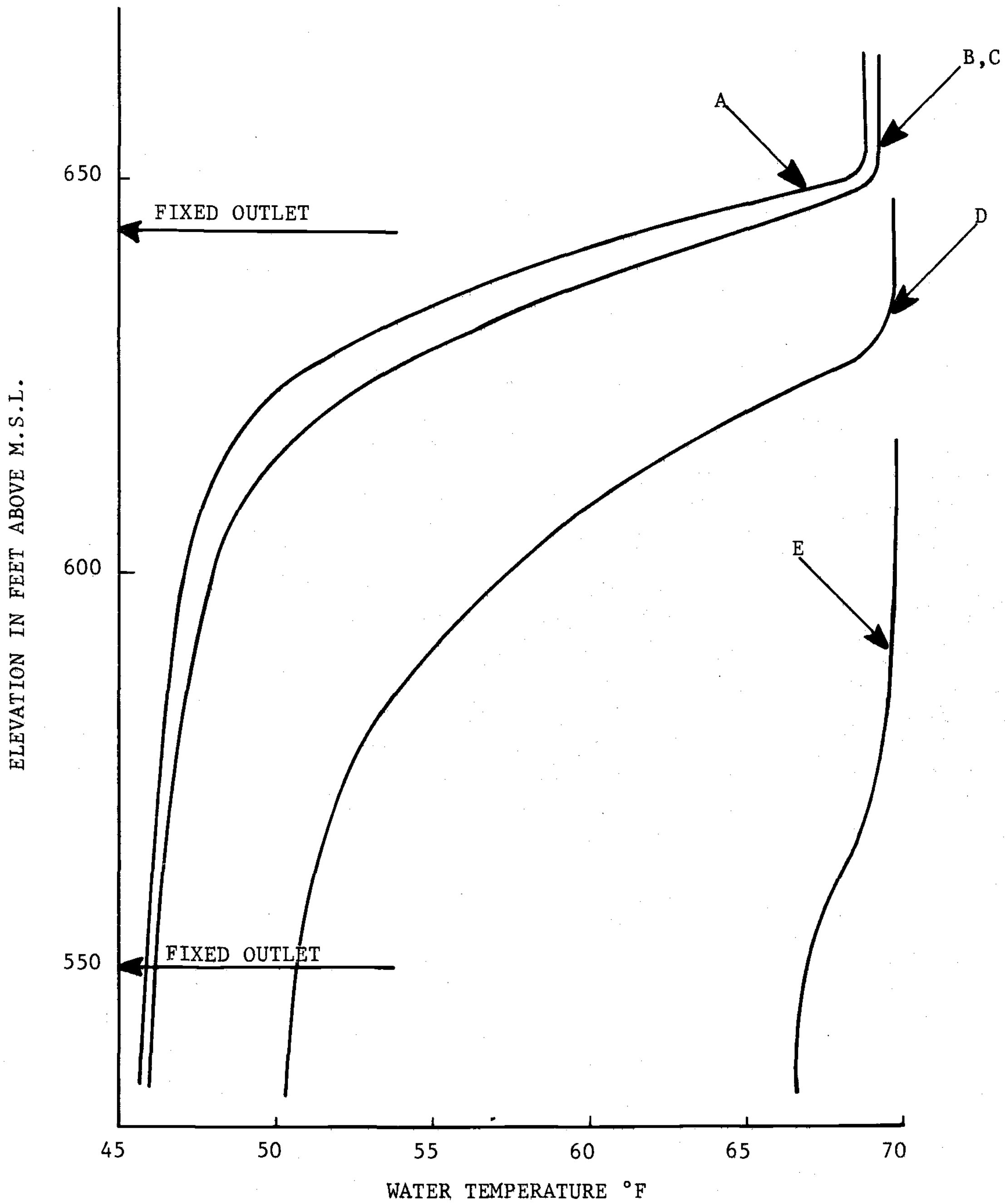

Figure 27. Predicted Temperature Profiles for Proposed Holley Reservoir On September 1, 1967 Under Various Release Strategies.

the surface (variable outlet not shown) and the other half from the outlet at elevation 643 feet msl. The curve B strategy consisted of releasing one-half the withdrawal from the middle outlet (643 feet msl) and the other one-half from the bottom outlet. The strategy for curve C was to release the entire withdrawal from the bottom outlet, elevation 550 msl. The model predicts that there would have been no discernible difference among these three profiles on August 1, 1967 (Figure 26), and little difference on September 1, 1967 (curves A and B-C, Figure 27).

Analyses presented earlier in this chapter suggested that a daily reservoir withdrawal rate of 500 cfs from the bottom outlet might significantly lower the downstream river temperatures, and should be given consideration. Curve E in Figures 26 and 27 is the result of this particular strategy. The predictions are similar to those derived from the 1958 data shown in Figures 24 and 25, that is, extreme reservoir drawdown with release temperatures on September 1 greater than 67° F, thus violating the recommended minimum of 60° F.

If the concern were only with attempting to provide optimum reservoir and downstream temperatures while adhering to the minimum release constraints, then a release strategy which would be a compromise of strategies C and E could be used.

The conditions for this strategy which resulted in curve D are the release of the minimum withdrawal requirements (Table 4) plus the irrigation requirements (Table 5), or 500 sfd if either of the following occurs:

1. Predicted solar flux striking the river surface is greater than 6800 ly per day.
2. Maximum daily three-hour dry-bulb temperature exceeds 85° F.

Thus, if neither of the above two conditions were met, the withdrawal strategy was that which resulted in curve C, a daily release of 165 cfs to 240 cfs from the bottom outlet (550 msl) depending on the month.

If either of the two conditions were predicted, the daily reservoir withdrawal was increased to 500 cfs from the bottom outlet --more than double the flow relative to curve C. This increase in flow would result in depth increases of from two to eight inches within the spawning area of the river.

Although reductions in downstream temperatures would be expected, these fluctuating discharges could create at least two additional problems within the spawning area. The velocity changes associated with the fluctuating discharges could cause local scouring and subsequent sediment deposition along the streambed. This action, if significant, would be expected to have an adverse effect on the redds (nests of eggs). The other problem relates to the change in wetted perimeter (lineal cross-sectional distance of the river bed) with fluctuating discharges. If the discharge were 500 cfs, salmon might migrate into the spawning area due to the lowering of downstream temperatures. These salmon might then deposit their eggs in gravel that would be exposed when the discharge is reduced. While it is realized these problems may exist, they are not a consideration in this study.

Curve D, Figure 27, does indicate that on September 1, 1967, the low withdrawal temperatures could still be maintained with the outflow temperature at 51° F.[31]

Reservoir Elevations and Storage

The reservoir analysis discussed above can be made more complete by relating specific temperature profiles and withdrawal strategies to the rule curve for comparison of expected deficits. Curve A, Figure 28, shows that the least deficit of all the curves presented would result from the release of the minimum reservoir withdrawals (Table 4) under 1968 hydrometeorological conditions. Curve B was predicted from a strategy identical to curve A using 1967 data.

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[31] During the first week of October (not shown here), the release temperatures reached 56° F--one degree above the suggested minimum for that month.

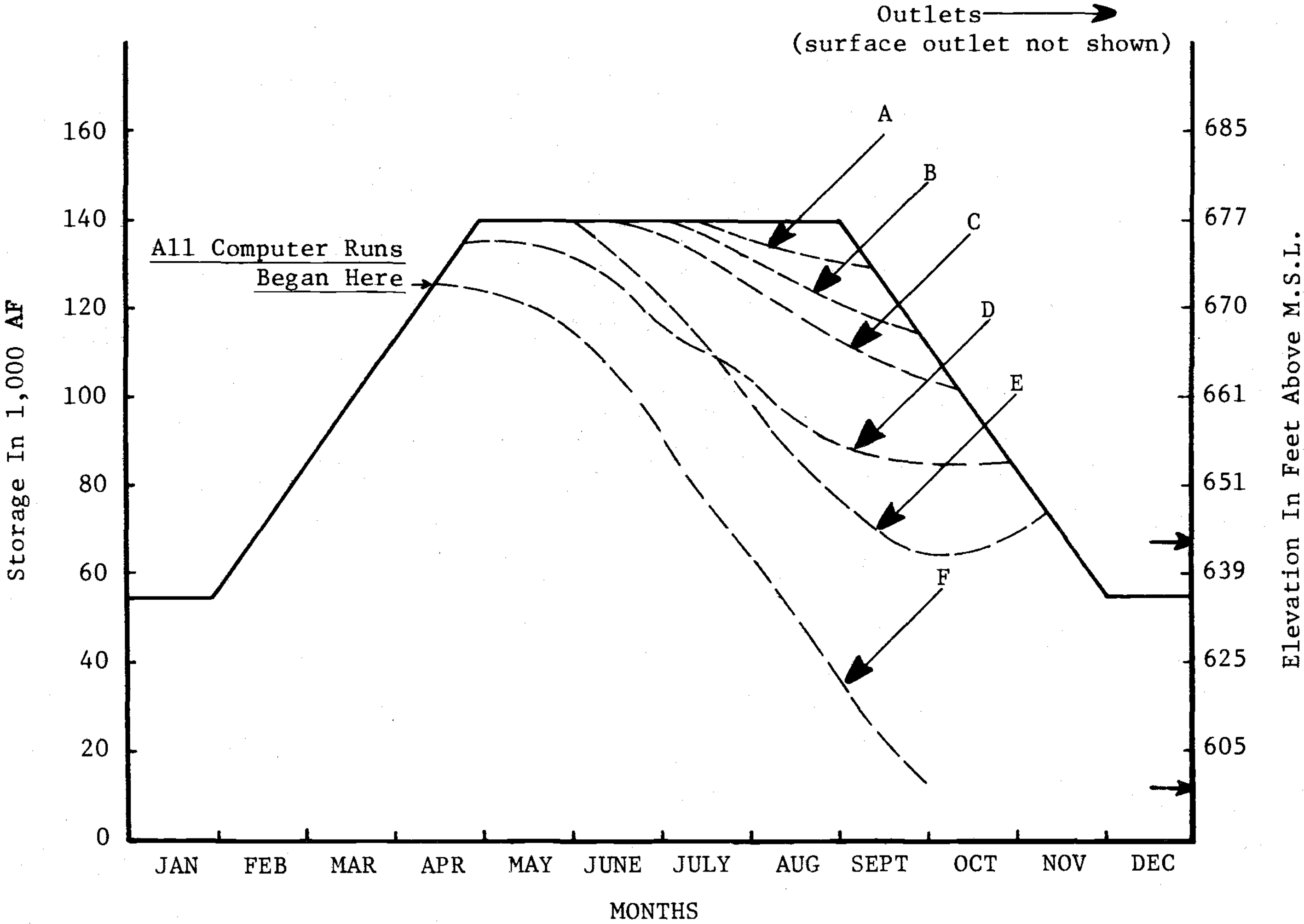

Figure 28. Effects of Various Withdrawal Strategies on Reservoir Drawdown, Proposed Holley Reservoir.

Curve C is associated with curve A in Figures 24 and 25. It is the predicted result of withdrawing the sum of the minimum withdrawal and irrigation requirements for 1958 data. The curves predicted for 1967 and 1968 under this strategy are not shown because they are similar to curve C.

Curves D and E result from an identical strategy being applied to 1968 and 1967 data, respectively. This strategy, stated in Figure 28, is a function of the time of year, the predicted flux striking the river surface, and the predicted maximum three-hour dry-bulb temperature. Taken together, these three variables determine the deficits for curves D and E. Curve E in Figure 28 is associated with curve D in Figures 26 and 27.

Curve F, Figure 28, is associated with curve E in Figures 26 and 27, and is the result of a daily withdrawal rate of 500 cfs beginning on the first day of the run, April 15, 1967. The strategy associated with curve F predicted that the reservoir surface would have been at an elevation of 550 feet (bottom outlet) on or about October 1, leaving the impoundment essentially empty. Even if the reservoir were allowed to completely fill before applying the 500 sfd withdrawal rate, the reservoir would have been emptied by October 12.

River Temperature Analysis

The analysis to this point has suggested that there are certain reservoir withdrawal strategies that deserve further investigation over a wide range of hydrometeorological conditions. Although the release strategy of 500 sfd appeared desirable for downstream conditions, it simply cannot be maintained throughout a low flow season. The strategies that have survived the reservoir analysis are the three from which curves A through E in Figure 28 were predicted. The predicted downstream temperatures associated with these strategies will now be examined in turn. These three strategies are:

1. Minimum reservoir release requirements (Table 4) from the bottom outlet--a function of the month of the year.
2. Minimum requirements (Table 4) plus irrigation requirements (Table 5) from the bottom outlet--a function of the month of the year.
3. Strategy two, or 500 sfd from the bottom outlet, if either of the following occurs:
   a. Predicted flux striking the river surface is greater than 6800 ly per day; or,
   b. Predicted maximum three-hour dry-bulb temperature is greater than 85° F.

River temperatures using strategies one, two, and three were predicted for river stations eight, 12, and 15 (see Figure 20) under 1967 and 1968 conditions. The predictions using 1968 data exhibited the same maximum temperatures as for 1967 data, but the mean maximum temperatures were predicted to be lower with the 1968 data.

Figures 29, 30, and 31 depict the predicted maximum and minimum three-hour river temperatures at river station eight under strategies one, two, and three, respectively, using 1967 data.[32] Station eight about 10 miles below the proposed reservoir, is the site of the irrigation withdrawals. Thus, the flow in the river at station eight represents the reservoir withdrawal as noted in the figures.

Recall from Chapter I that the generally recommended optimum temperature ranges for salmonids are:

Migration: 45° -60° F

Spawning: 45° -55° F

Rearing: 50° -60° F.

At station eight, each strategy violates both the recommended maximum and minimum temperatures, and it is not uncommon for the diel (24 hour range) fluctuations to exceed 15° F.

The maximum temperatures in Figures 29 and 30 begin to recede around the first of September, while those in Figure 31 increase

[32] The 1968 predictions are not shown because of the similarities to the 1967 predictions.

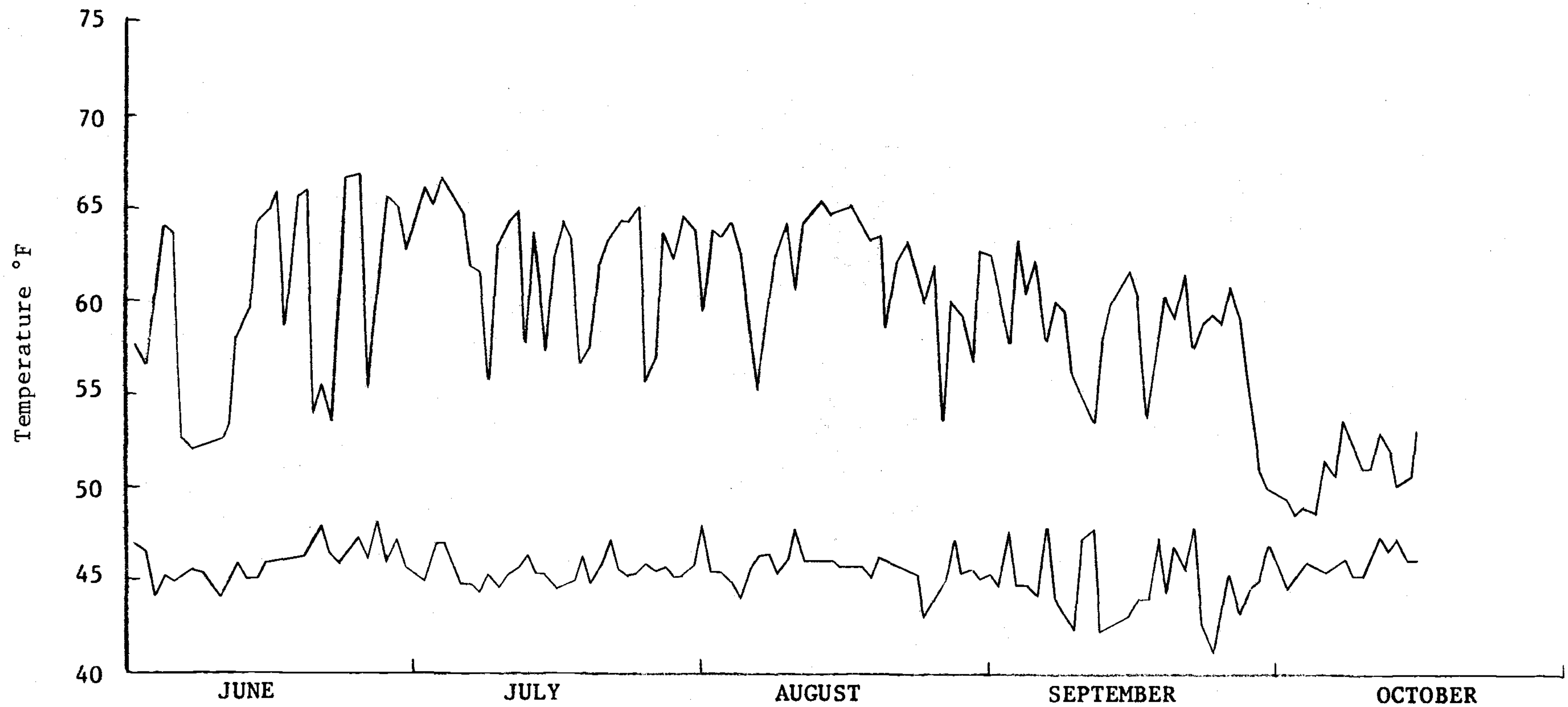

Figure 29. Predicted Maximum and Minimum Calapooia River Temperatures, Strategy 1, Station 7, 1967.

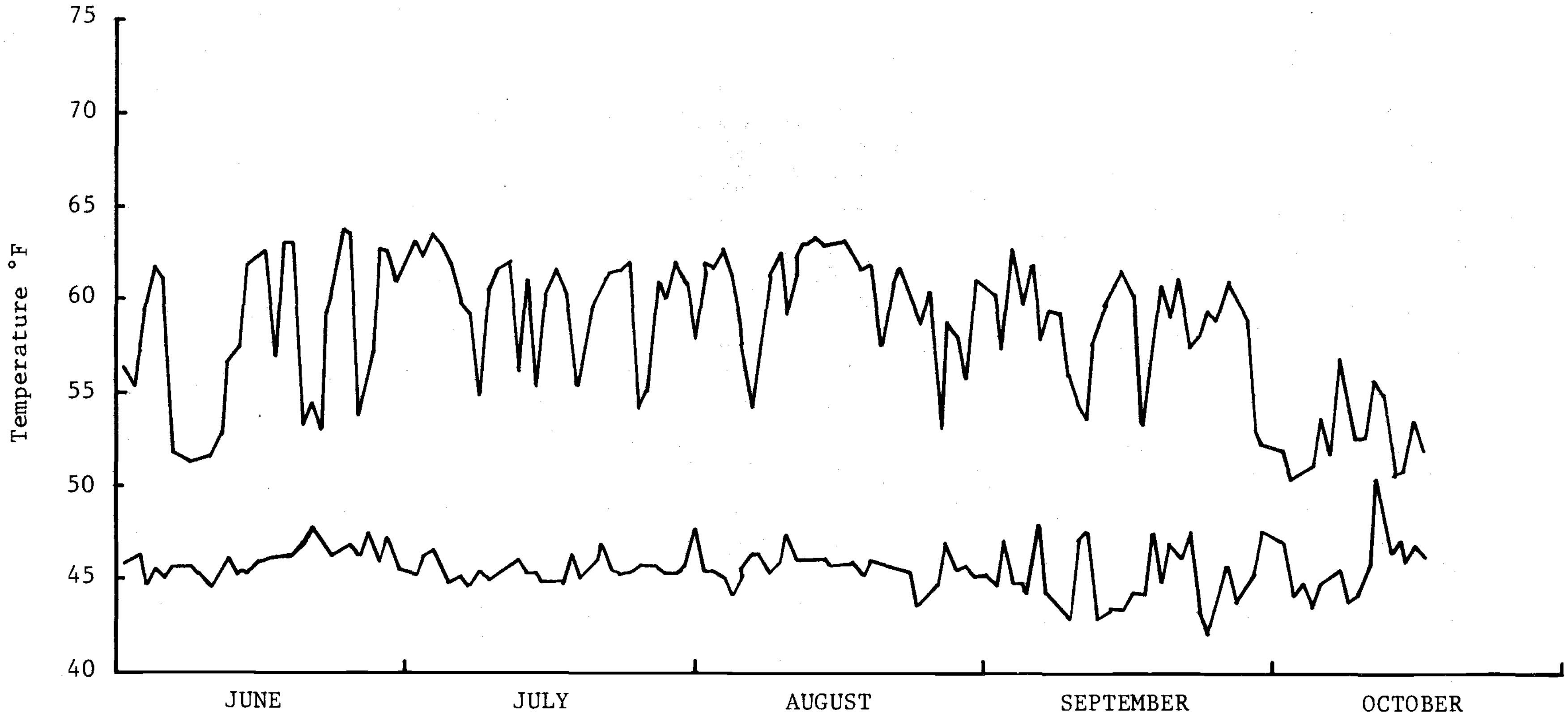

Figure 30. Predicted Maximum and Minimum Calapooia River Temperatures, Strategy 2, Station 7, 1967.

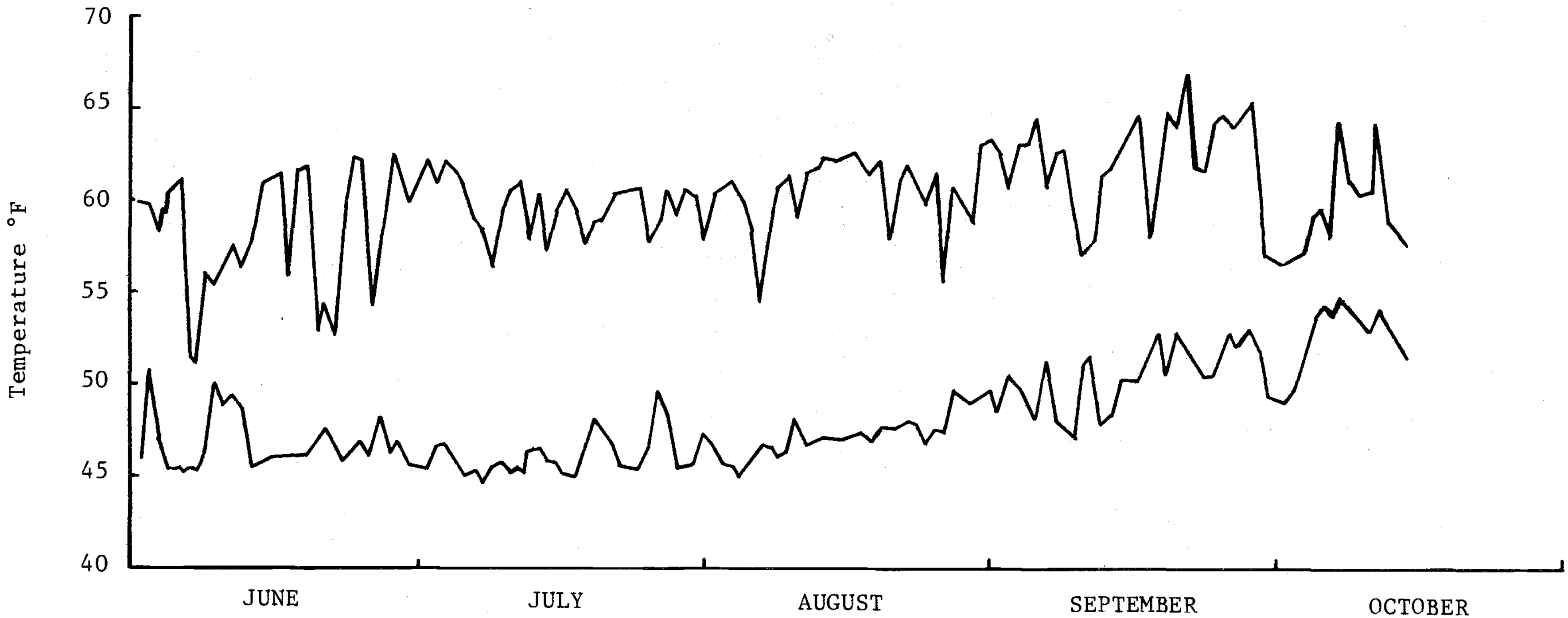

Figure 31. Predicted Maximum and Minimum Calapooia River Temperatures, Strategy 3, Station 7, 1967.

steadily. Also, the minimum temperatures shown in Figures 29 and 30 change little, while those in Figure 31 increase steadily until the first week in October. This is an indication of the increasing temperature and depletion of the hypolimnion under strategy three.

Attention is now turned to Figures 32 through 36, which illustrate the predicted river temperatures at station 12. This station is 16.8 miles below the dam site, and is considered to be the farthest downstream point for spawning. Figure 32 was predicted from strategy one, using 1967 data. The three-hour maximum temperatures exceed 75°F, while the minimums approach 41°F. The maximum diel fluctuation is in excess of 27°F with 20°F fluctuations occurring over half the time. The relatively stable and narrow range of minimum temperatures suggests that station 12 is well within the downstream range under the influence of the cold reservoir releases.

Figure 33 resulted from strategy two being applied to 1967 data. The peak maximums and diel fluctuations are about 5°F below those in Figure 32.

Figure 34 also resulted from strategy two, but represents 1968 conditions. This was included as a comparison to 1967 data in Figure 33. Note that there are no differences in the peak temperatures. However, the daily sequences of high maximum temperatures are less in number and shorter in duration than in Figure 33, the 1967 conditions. Except for a two-week period around the first of September,

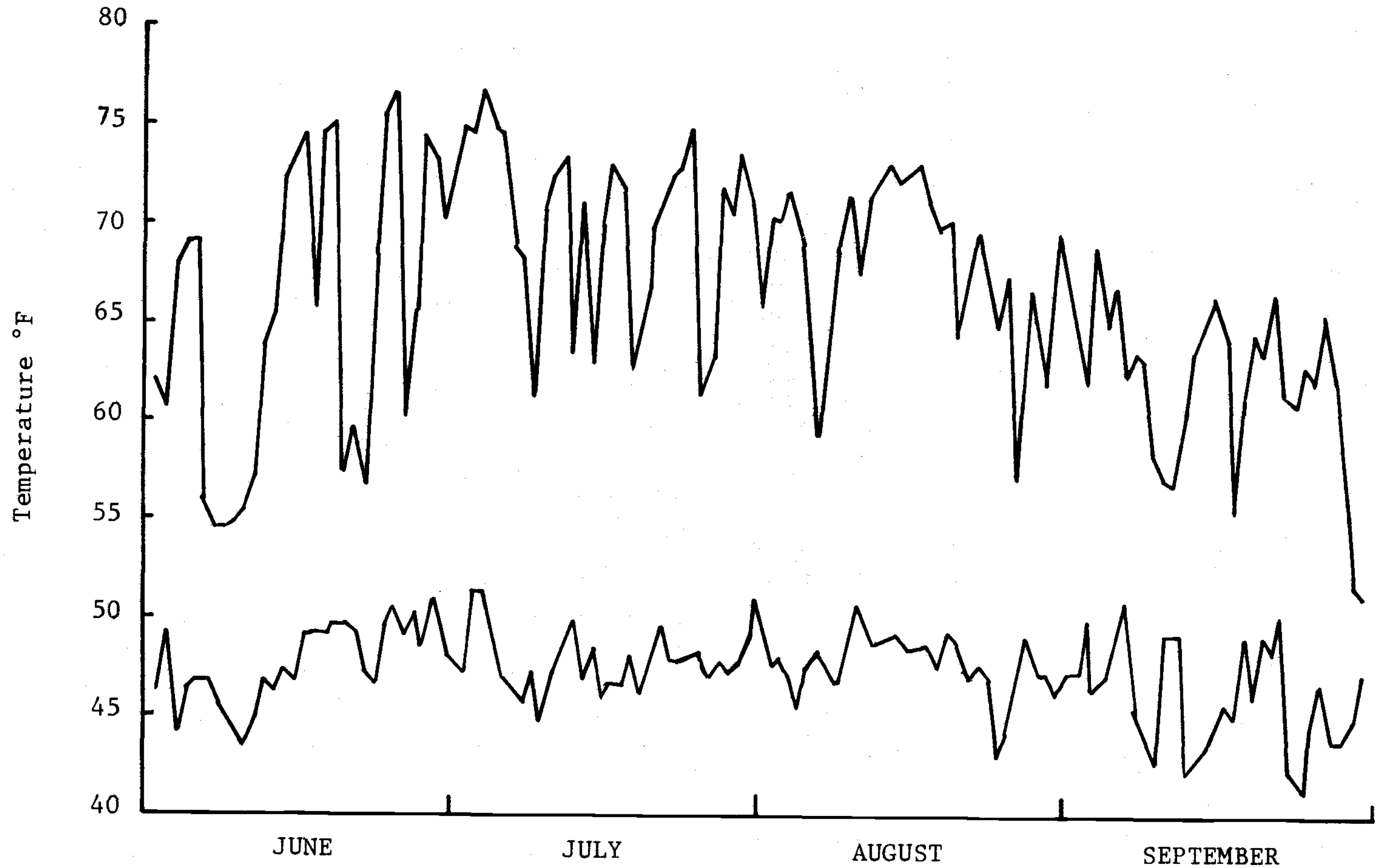

Figure 32. Predicted Maximum and Minimum Calapooia River Temperatures, Strategy 1, Station 12, 1967.

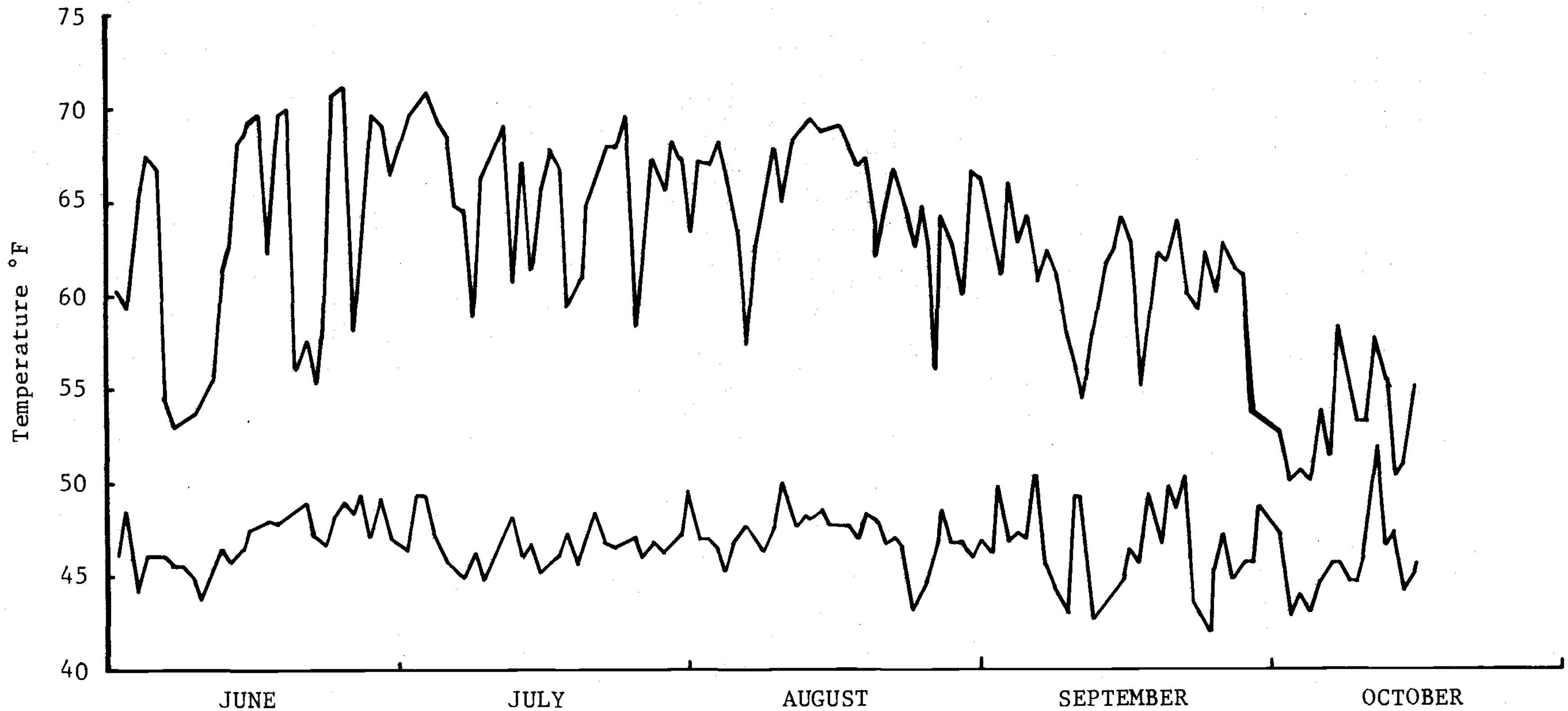

Figure 33. Predicted Maximum and Minimum Calapooia River Temperatures, Strategy 2, Station 12, 1967.

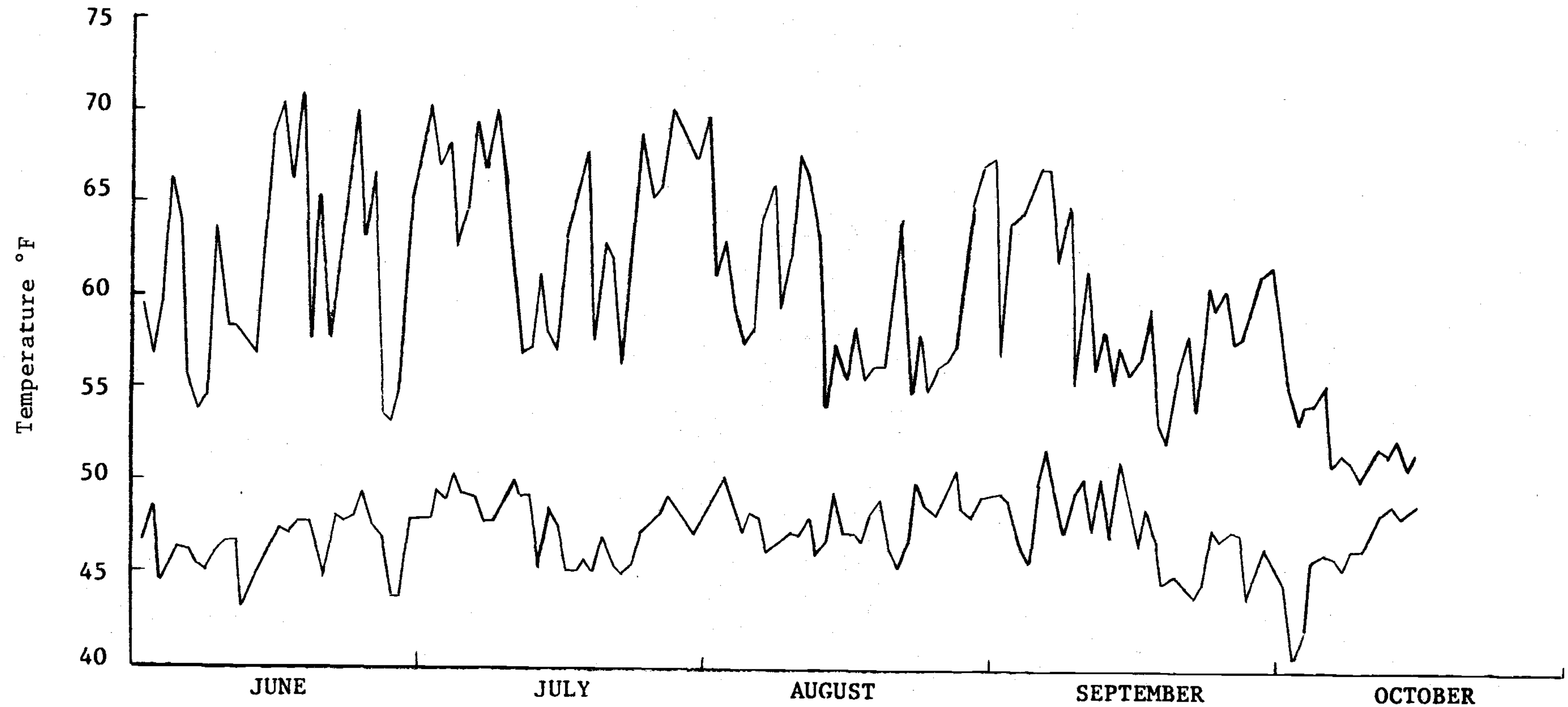

Figure 34. Predicted Maximum and Minimum Calapooia River Temperatures, Stategy 2, Station 12, 1968.

the maximums exhibit a general downward trend beginning August 1. This trend is similar to the one for 1967 found in Figure 33 but with generally lower temperatures.

Figure 35 resulted from applying strategy three to 1967 data. As at station eight, the minimum temperatures steadily increase after the first of August. However, with the exception of September 21, the daily three-hour maximums are not greater than 67° F. The early summer "hump" of maximum temperatures associated with strategies one and two (1967 data) has been smoothed out, while the maximums during the month of September have been increased by two to three degrees Fahrenheit.

The maximum temperatures associated with strategy three are about 8° F below those of strategy one, and 4° F below those of strategy two. The minimum temperatures shown in Figure 34 are maintained above 45° F (except for 44.7° F on July 7), whereas the minimums associated with strategies one and two occasionally fall below 43° F.

Figure 36 was the result of applying strategy three to 1968 data. The peak maximum temperatures are of the same order of magnitude of those found from 1967 data (67° F). However, unlike the 1967 predictions shown in Figure 35, the 1968 analysis indicates a general downward trend in maximum temperatures beginning August 1, similar to the trend found using strategy two and 1968 conditions (Figure 34).

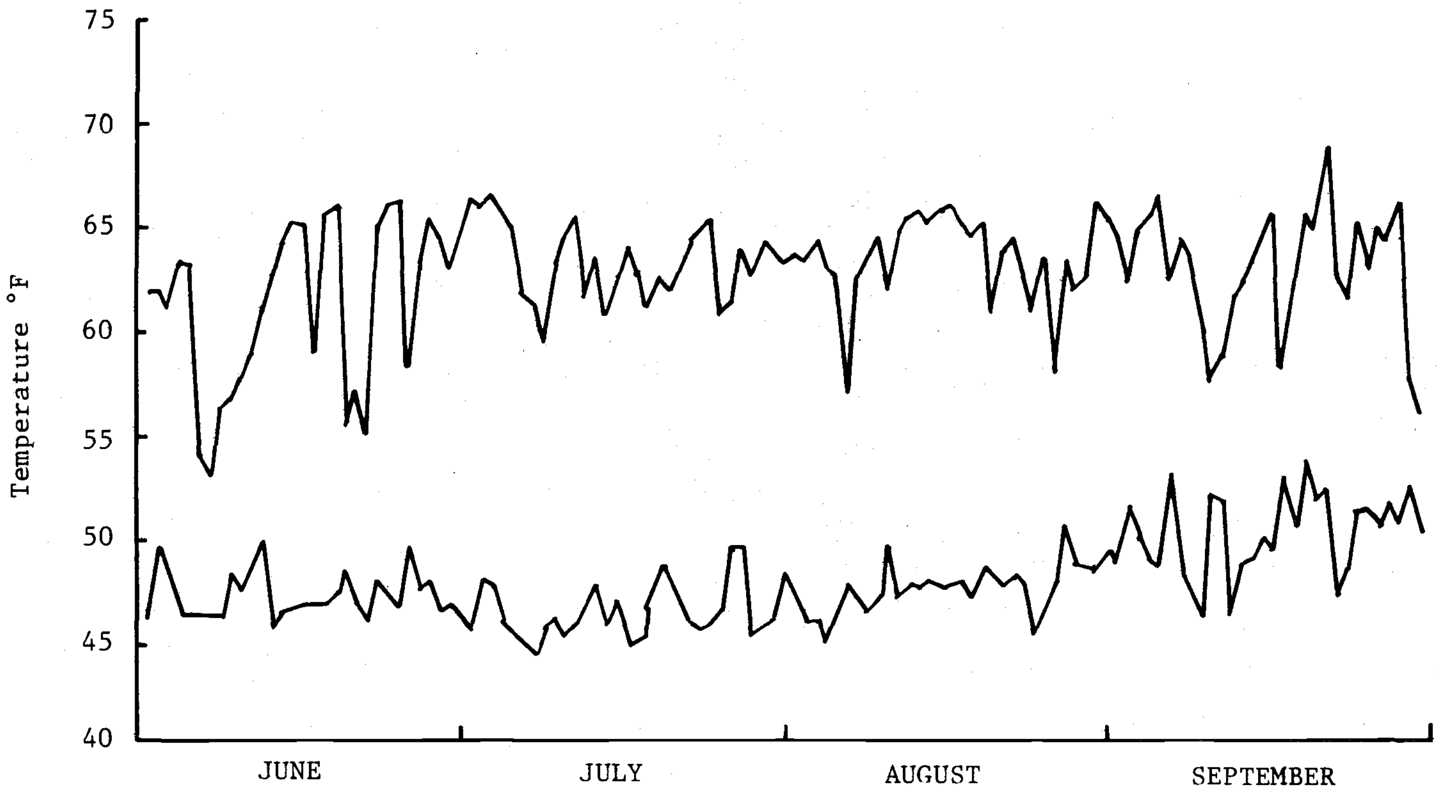

Figure 35. Predicted Maximum and Minimum Calapooia River Temperatures, Strategy 3, Station 12, 1967.

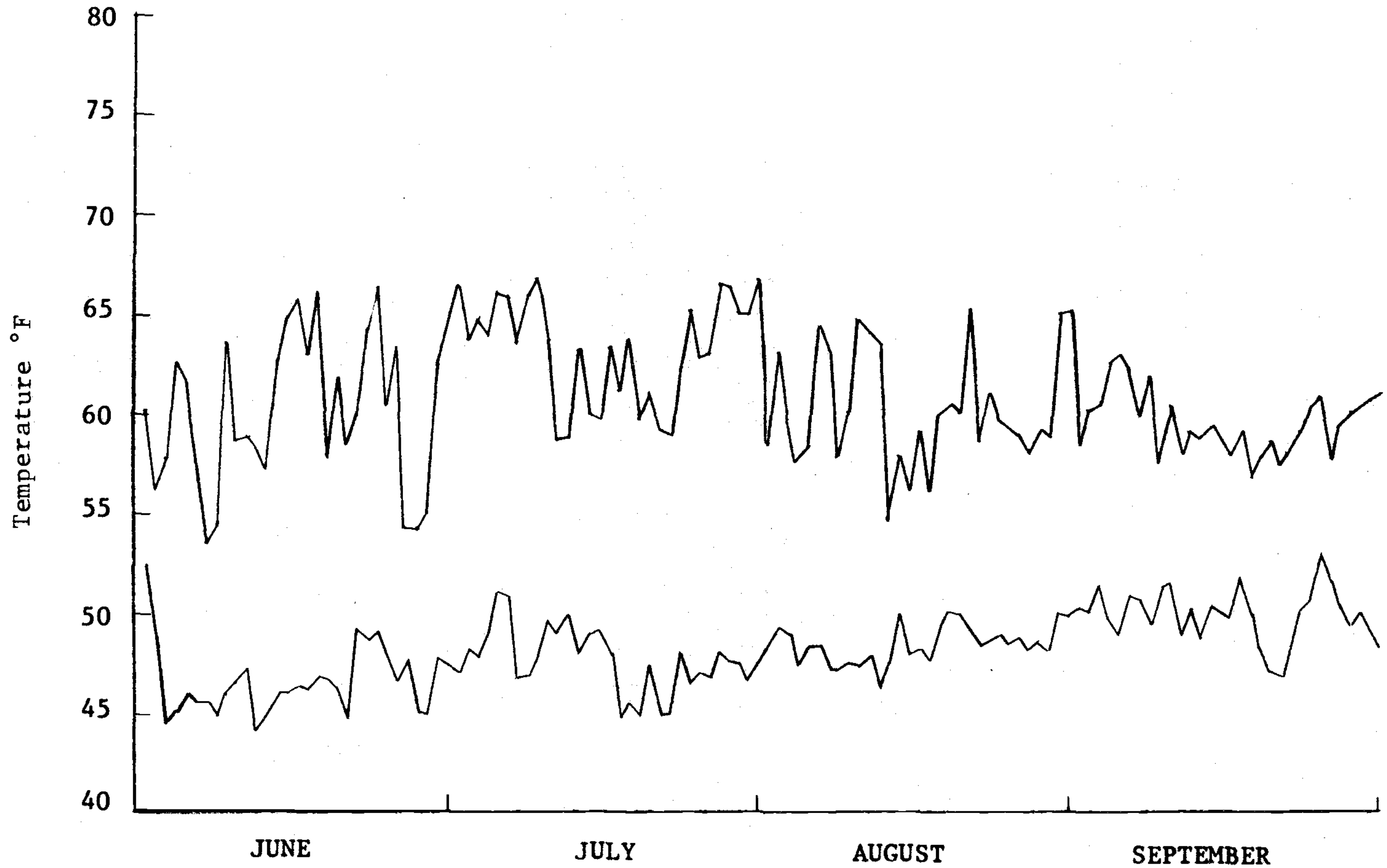

Figure 36. Predicted Maximum and Minimum Calapooia River Temperatures, Strategy 3, Station 12, 1968.

Figures 37 through 40 show the predicted maximum and minimum temperatures at station 15, located 24 miles below the dam site. As mentioned earlier, temperatures predicted at station 15 are considered to be low estimates, as the magnitude of flow in this portion of the channel was always approximately twice that which would be expected to occur under actual conditions.

One important aspect to note in Figures 37 and 38 is the relatively high minimum temperatures that appear to move with the maximums. This situation yields diel fluctuations of 10° F or less, and indicates that the daily extreme river temperatures at this station would be influenced little by the presence of Holley Reservoir if operated under strategies one through three.

Figures 39 and 40 present predictions for 1967 and 1968, using strategy three. They are similar except that the characteristic trends of each year's meteorological data are present.

Summary

This chapter investigated the reservoir temperature profiles using hydrometeorological data from 1958, 1967, and 1968. It was predicted that there would be a greater mass of cold water available for release from the proposed Holley Reservoir than predicted by either the Corps of Engineers or the FWQA.

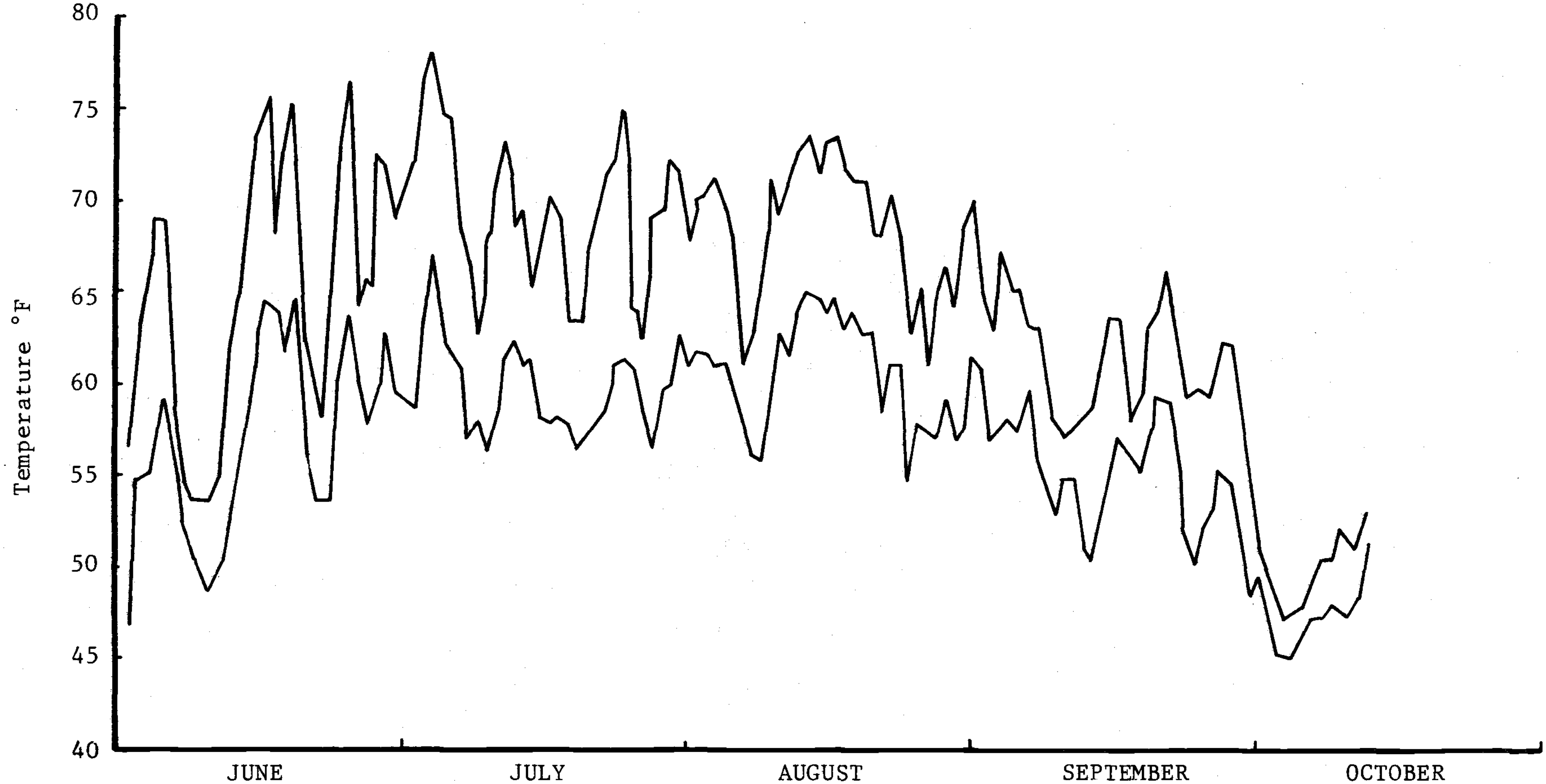

Figure 37. Predicted Maximum and Minimum Calapooia River Temperatures, Strategy 1, Station 15, 1967.

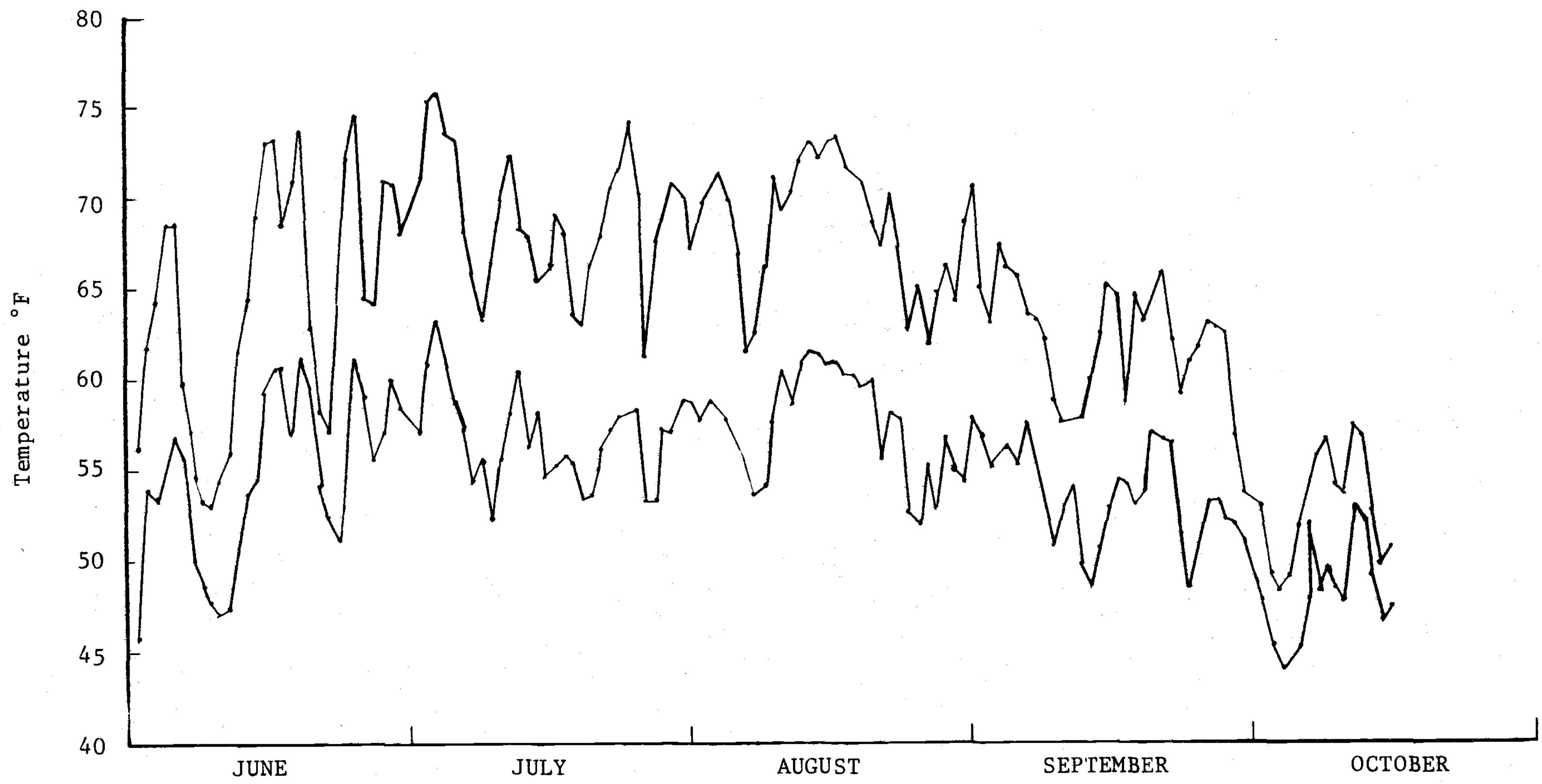

Figure 38. Predicted Maximum and Minimum Calapooia River Temperatures, Strategy 2, Station 15, 1967.

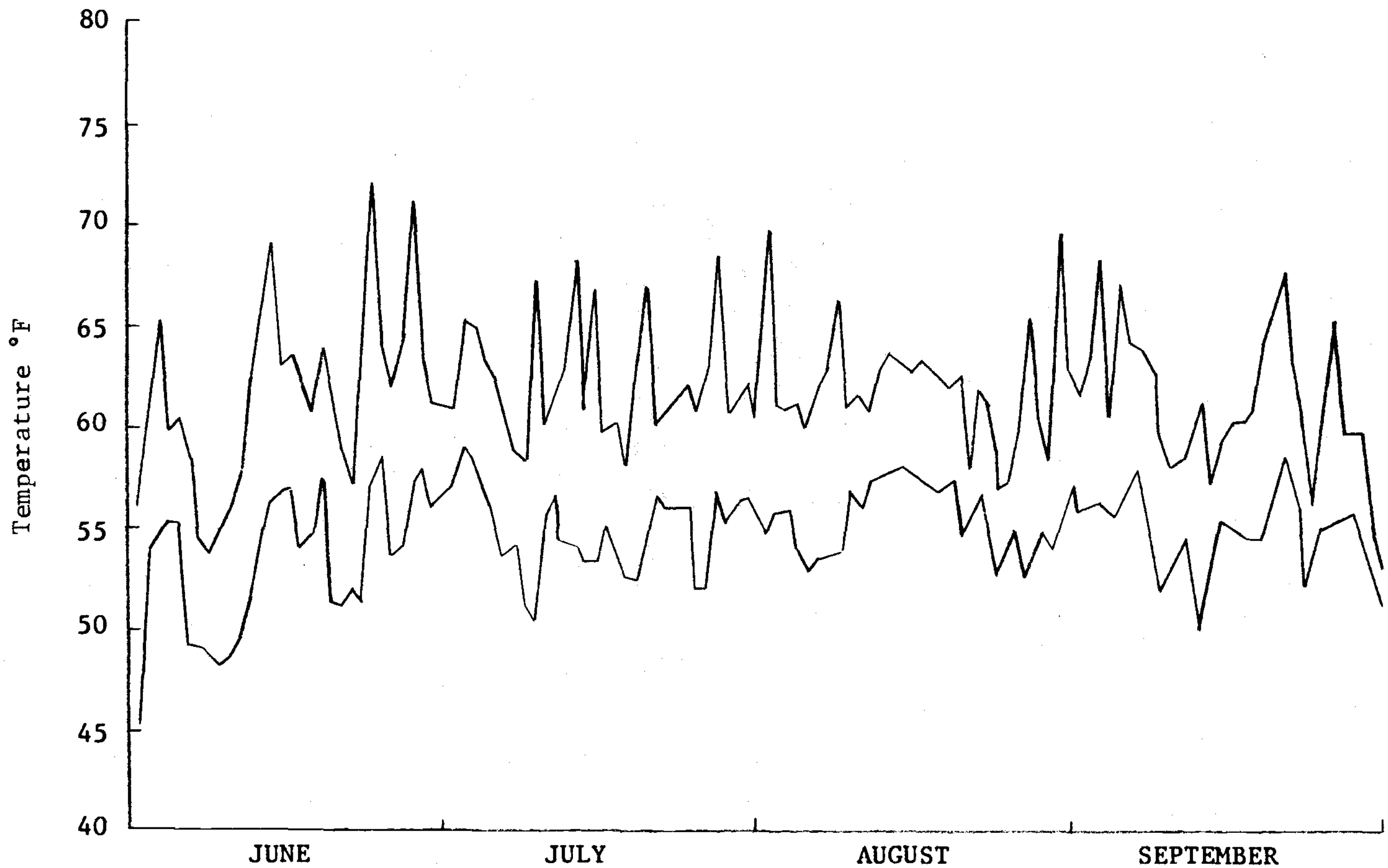

Figure 39. Predicted Maximum and Minimum Calapooia River Temperatures, Strategy 3, Station 15, 1967.

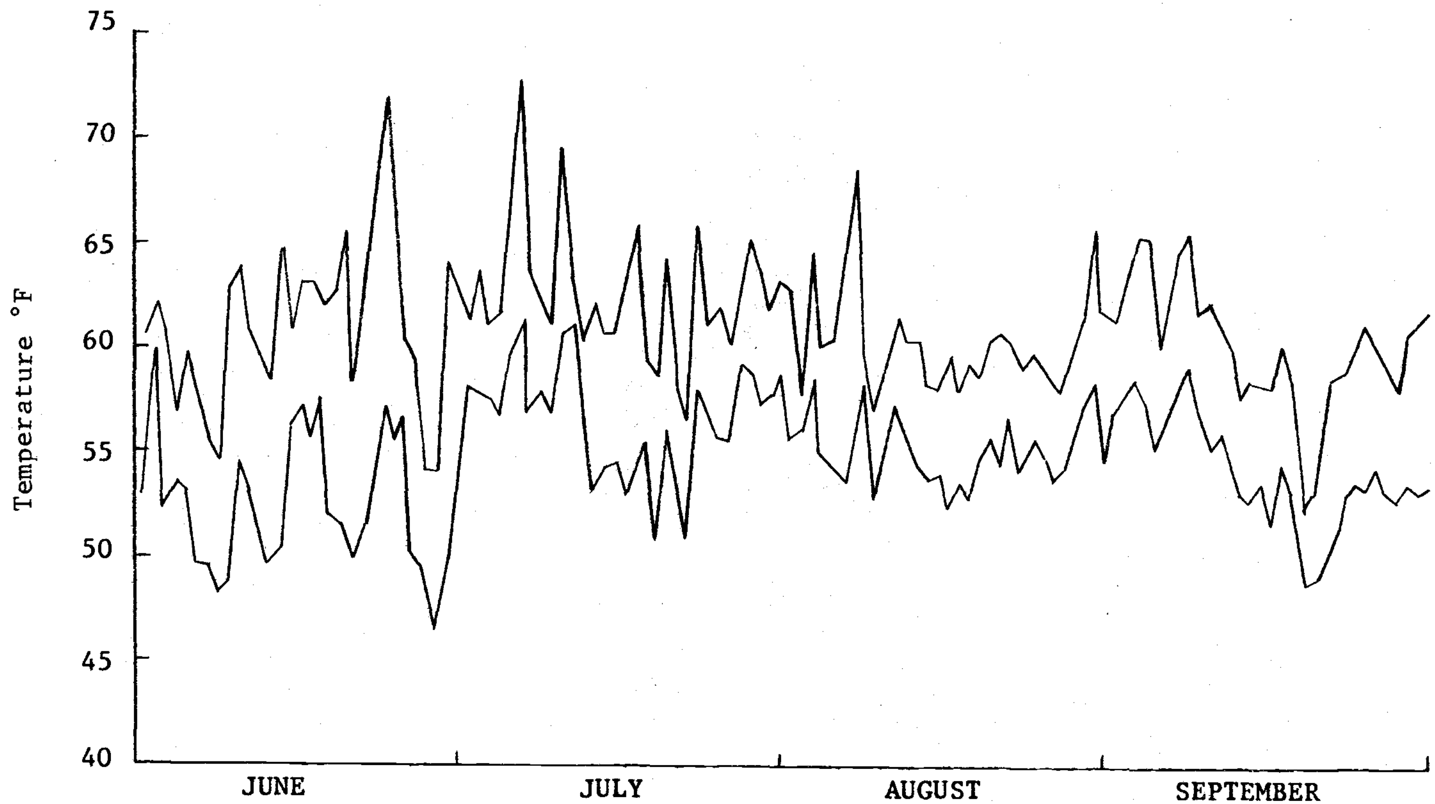

Figure 40. Predicted Maximum and Minimum Calapooia River Temperatures, Strategy 3, Station 15, 1968.

Three specific release strategies were delineated for further analysis of downstream temperatures using 1967 and 1968 data.

It was predicted that no strategy was available that could maintain the river temperatures within the optimum range for salmonids.

If the objective were to maintain the downstream maximum temperatures within the spawning area below some specified minimum, say 68°F, then strategy three appeared to be the most promising.

It was also found that the temperature of the reservoir withdrawals would have little or no effect upon either the maximum or minimum river temperatures 24 miles below the dam site.

This concludes the physical analysis of river and reservoir temperatures, mass flows, and reservoir drawdowns, of this study.

It remains to be shown in Chapter VII how these physical results are used as inputs to the production of anadromous fish and recreation days, as well as to the potential resulting benefits from these project purposes.

VII. PROJECT FEASIBILITY

Introduction

The physical information describing streamflows and river temperatures must be transformed into an economic framework before decisions regarding project feasibility may be made. The present chapter demonstrates how this model could be used to evaluate benefits and opportunity costs for a proposed water resources project. Where possible, quantities, production relationships, and prices are those provided by the Corps of Engineers.

Benefits and Costs

This study dealt only with the project purposes of anadromous fish enhancement, recreation, and irrigation. A necessary condition used by the Corps of Engineers for project feasibility is that each project purpose must have a favorable (≥ 1.0) benefit-cost (B-C) ratio. Benefits and costs as given by the Corps for the three low flow purposes under consideration are stated in Table 8.

The annual fish benefits were originally stated as $268,000 in the review report referenced in this study. The most recent information, not completed until after the first review report, indicates that the expected annual fish benefits are approximately $531,511. The assumption of optimum water quality conditions prevailed in this most recent analysis.[33]

Table 8. Separate benefit-cost comparisons for low flow purposes, Holley Dam.[a]

Purpose	Cost	Benefit	Δ	B/C
Recreation	$370,000	$419,000	+$ 49,000	1.13
Fish enhancement	236,000	531,511	+ 315,511	2.34
Irrigation	28,000	60,000	+ 32,000	2.14

Source: U.S. Army Corps of Engineers (1970).

[a]100-year life at 4 and 7/8 percent.

Recreation Benefits

The expected recreation user days associated with the proposed project may be determined from Figure 41, provided by the Corps of Engineers. The three functions of Figure 41 reflect the estimated increase in recreation and fishing over that which would occur without the project. The estimates were based on projections of per capita recreation use that take into account population concentrations at various commuting distances from the dam site. The user days were valued in accordance with Senate Document No. 97 as $1.00 for general reservoir recreation, $2.00 for reservoir fishing, and $3.00 for fishing upstream from the reservoir.

[33]Personal communication, Ken Boire, Chief of Economic Studies, Corps of Engineers, Porland, Oregon, May 30, 1974.

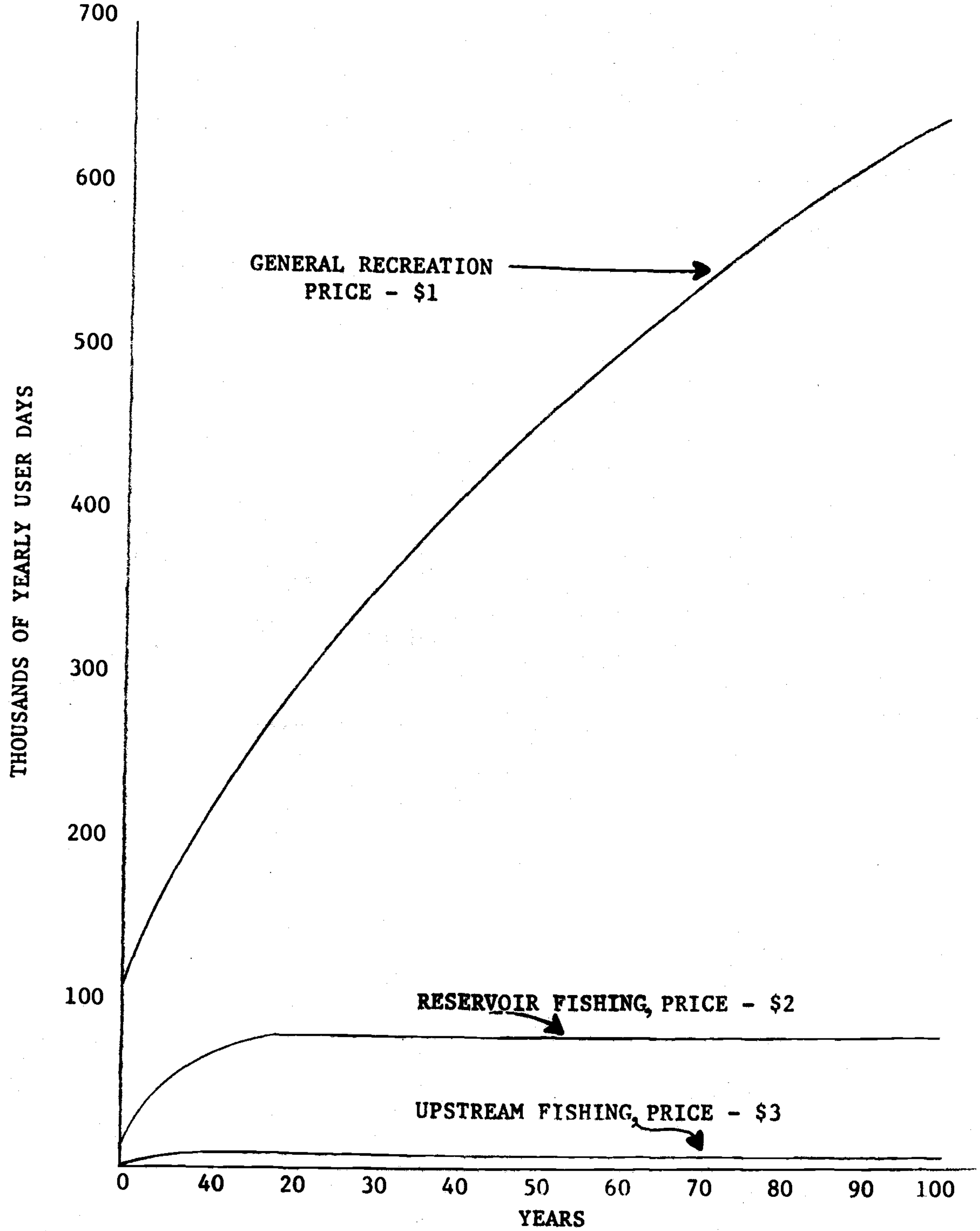

Figure 41. Projected User Days For Proposed Holley Project

Source: U. S. Army Corps of Engineers (1970).

The estimated reservoir user days for general recreation and fishing are assumed to reflect a near optimum reservoir level, i.e., adherence to the rule curve in reservoir operation. In accounting for the effect of reservoir drawdown on reservoir recreation, Avey (1972) used functional relationships provided by the Corps of Engineers to estimate the expected reservoir use as an explicit function of beach length and maximum reservoir use. The beach length, defined as the linear ground distance measured between the rule curve elevation and actual elevation, was found by using the average reservoir side slope.

The same functional relationships used by Avey were used in this study. The average side slope was designated as 176.0; the beach length in feet was found by multiplying the side slope by the difference between the rule curve elevation and the actual elevation. A subroutine inserted in the general computer model computed the values for beach length and stored them on a computer file for future access.

The recreation benefits were then calculated as follows, by a separate computer program, BENIES. The maximum expected general recreation and reservoir fishing use was determined from least-squares estimates of the growth functions of Figure 41. Resulting values of the maximum expected use were then used to determine the actual daily general recreation and reservoir fishing use from Equations (7-1) and (7-2).

$$GR(D) = GRMAX(D) - (GRMAX(D)/G1) \times BLENGTH(D) \quad (7\text{-}1)$$

where

D = day of year

GR(D) = number of general reservoir recreation users on day D

GRMAX(D) = number of maximum (rule curve) general reservoir recreation users on day D

BLENGTH(D) = length of beach in feet on day D

G1 = 15,000.0, the functional intercept in feet (Avey, 1972).

$$RF(D) = RFMAX(D) - (RFMAX(D)/R1) \times BLENDTH(D) \quad (7\text{-}2)$$

where

RF(D) = number of reservoir anglers on day D

RFMAX(D) = number of maximum (rule curve) reservoir anglers on day D

R1 = 8,000.0, the functional intercept in feet (Avey, 1972).

Upstream fishing enhancement benefits were assumed to occur from increased fishing pressure due to anglers' expectations of catching trout moving upstream from the reservoir. These benefits, however, were not considered to be a function of beach length.

All recreation was assumed to occur with an equal daily likelihood within the four summer months of June, July, August, and

September. For example, if the maximum expected yearly users were estimated to be U_o, then the daily expectation for that particular year would be $U_o/122$.[34] This would be the value used in the appropriate equation previously mentioned, to calculate the actual number of expected daily users. Prices were then multiplied by the total user days and these values were summed to determine yearly values. The yearly values were then summed over the three recreational functions for each year, and discounted at 4 and 7/8 percent over the expected 100-year project life.

Anadromous Fish Benefits

Anadromous fish benefits occur from the increase in fish production expected with the project over the possible fish production without the project. The expected increase in fish production without the project is due in part to a recently completed fish passage facility at Willamette Falls in Oregon City, and to a greater investment in hatcheries and stocking programs. The potential production resulting from this intensive management cannot be counted as accruing to a particular project now under consideration. Thus, it is necessary to ascertain the potential salmonid production in the Calapooia River with and without the project. These figures are given in Table 9.

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[34] There are 122 days in the four summer months.

Table 9. Species and numbers of spawning anadromous fish without the project and with project fish mitigation and enhancement.

Reach or Section of Project Area	Anadromous Species	Estimated Spawners Without Project Fish Mitigation or Enhancement	Estimated Spawners With the Project and Recommendations	Increase in Spawners With Project and Recommendations
Downstream reach below Holley Dam	Fall Chinook Salmon	1,000	1,900	900[a]
	Spring Chinook Salmon	0	450	450[a]
	Winter Steelhead Trout	0	700	700[a]
Dam site or reservoir	Spring Chinook	0	6,250	6,250[a]
Upstream reach above Holley Reservoir	Fall Chinook Salmon	1,200	1,200[b]	0
	Spring Chinook Salmon	300	300[b]	0
	Winter Steelhead Trout	1,400	1,400[c]	0
	Silver (Coho) Salmon	600	600[b]	0

[a] Enhancement means increases in spawners or introduction of species.

[b] Mitigation means maintenance of spawners.

[c] Hatched and reared in the hatchery.

Source: U.S. Department of Interior, Bureau of Sport Fisheries and Wildlife, 1971.

The production figures associated with fish enhancement are assumed to increase linearly from zero during the third year of the project to the maximum values shown at the fifth year and every year thereafter through the project life.

The combined commercial and sport prices used to value the fishery are presented in Table 10.

Table 10. Multiplier price for each species of fish.

Species of Fish	Multiplier Price
Coho Salmon	$10.152
Winter Steelhead	8.247
Fall Chinook	87.840
Spring Chinook	89.385
Summer Steelhead	65.795

Source: Clifford E. Soderstrom, Fishery Biologist, National Marine Fishery Service, 1969.

The above prices are multiplied times the expected returning spawners[35] of each species to determine the annual gross values. The annual gross values are then summed over all species, amortized to a present value, and then converted to an annual basis for project life.

[35] Returning spawners are usually called "escapees", or, in total, the "escapement". Thus, these are the fish which have escaped the commercial catch, the angler, and all natural mortalities.

The procedure used in this study to determine the amount of fish production expected to result from a given reservoir withdrawal strategy does not follow the traditional methods of production economics. Such methods usually involve estimation of a production function and resulting outputs by one or more statistical techniques. This approach was not possible in this study due to the scarcity of available data describing the production of salmonids in a near natural habitat. What data are available pertain to hatchery conditions where one or more variables are under constant control and most relevant variables are monitored.

The approach used in this study was to confer with fishery biologists with recognized expertise in the effects of Pacific salmon production from (1) stream temperature and water velocities; and (2) the construction of high dams. All experts consulted were familiar with the Calapooia River system.[36]

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[36]Harry Wagner, Chief, Research, Oregon Wildlife Commission; Homer Campbell, Supervisor of Fishery Research, Oregon Wildlife Commission; James Lichatowich, Project Leader, Oregon Wildlife Commission; Richard Lantz, Assistant Regional Supervisor, Oregon Wildlife Commission; and Richard Giger, Project Leader, Oregon Wildlife Commission. All the above consultants are presently with the Corvallis office of the Oregon Wildlife Commission.

Irrigation Benefits

The irrigation benefits were assumed to be directly proportional to the daily withdrawals from the Calapooia River. Due to limited resources, this study initially considered only two alternatives: (1) no water was withdrawn from the Calapooia River, resulting in zero irrigation benefits; and, (2) the maximum irrigation requirement was withdrawn from the channel, resulting in the maximum annual irrigation benefits of $60,000.

Trade-Offs

The problem of determining trade-offs between reservoir and downstream use has been alluded to throughout this study, and will now be described in further detail.

A mass of water is available over the summer season for producing products at the reservoir and downstream. The product mix resulting from the use of the water is determined by the reservoir and channel withdrawal strategies.

For simplicity, and because it was not thoroughly investigated, irrigation will be treated as a given. This reduces the problem to the case of the production of two products, anadromous fish and recreation days; and one factor, namely water.

The analysis proceeds by estimating the annual benefits accruing to fish enhancement and recreation from each of the three strategies for 1967 and 1968 conditions. It was assumed for purposes of this demonstration, that a 100-year sequence of 1967 data constituted one set of conditions, and a 100-year sequence of 1968 data constituted the other set of conditions.

The year 1967 was characteristically warm and dry, while 1968 was a cool, wet year. Thus, the annual benefit streams predicted from the two 100-year sequences should approximate the bounds of an expected average annual benefit stream. These estimations for the three strategies and the three project purposes under 1967 and 1968 conditions are presented in Table 11.

The annual recreation benefits were calculated from the computer program BENIES by the procedure described earlier. The predicted recreation benefits as a percent of the optimum recreation benefits range from about 74 percent for strategy three to over 90 percent for strategy one. The annual irrigation benefits were taken as given by the Corps at $60,000.

Although the water quality associated with strategy three approached the optimum conditions for fish enhancement, the predicted maximum stream temperatures exceeded the recommended limits. However, for demonstration purposes and because the fishery experts consulted agreed that the stream temperatures predicted by strategy three were close to the optimal conditions, the fish production expected to result from the use of strategy three was taken as the maximum expected production.

Table 11. Comparison of predicted annual benefits for 1967 and 1968 conditions, proposed Holley Reservoir.[a]

Purpose	1967	% of Optimum	1968	% of Optimum
Strategy 1				
Recreation	$381,132	91.0	$390,182	93.1
Irrigation	60,000	100.0	60,000	100.0
Fish enhancement	457,955	86.2	448,905	84.4
Total	$899,087		$903,147	
Strategy 2				
Recreation	$359,684	85.8	$366,359	87.4
Irrigation	60,000	100.0	60,000	100.0
Fish enhancement	479,403	90.2	472,720	88.9
Total	$899,087		$903,147	
Strategy 3				
Recreation	$307,576	73.4	$311,636	74.3
Irrigation	60,000	100.0	60,000	100.0
Fish enhancement	531,511	100.0	531,511	100.0
Total	$899,087		$903,147	

[a]Assumes a project life of 100 years at 4 and 7/8 percent.

The values for the annual fish benefits in Table 11 under strategies one and two were obtained by subtraction. These values represent the level of annual fish benefits that would allow the total annual benefits for low flow purposes to be identical to the total estimated under strategy three. These total annual values were $899,087 for 1967 conditions and $903,147 for 1968 conditions. Thus, for strategy one, 1967 conditions, the fish enhancement benefits of Table 11 are $899,087-$381,132 (recreation benefits) - $60,000 (irrigation benefit) = $457,955. The other values are calculated similarly.

Although the fishery biologists agreed that fish production associated with the temperatures predicted from strategies one and two would be less than the production from strategy three, they were unable to quantify this relative to the maximum production. However, the numbers presented in Table 11 and graphed in Figure 42 do provide one means for evaluating the trade-offs between anadromous fish enhancement and recreation benefits.

The solid line in Figure 42 is an iso-revenue line plotted from the 1967 conditions of Table 11. Three of the four points on this line represent the three different withdrawal strategies. The fourth point, zero (0), represents adherence to the rule curve during the summer season. The recreation benefits have been estimated for each of the four points delineated along this line. The anadromous fish benefits have been estimated for point three only, representing strategy three. The fish benefits corresponding to the other three points (2, 1, 0) on the line were found as stated earlier by subtracting the estimated recreation benefits at each point from the sum of the recreation and

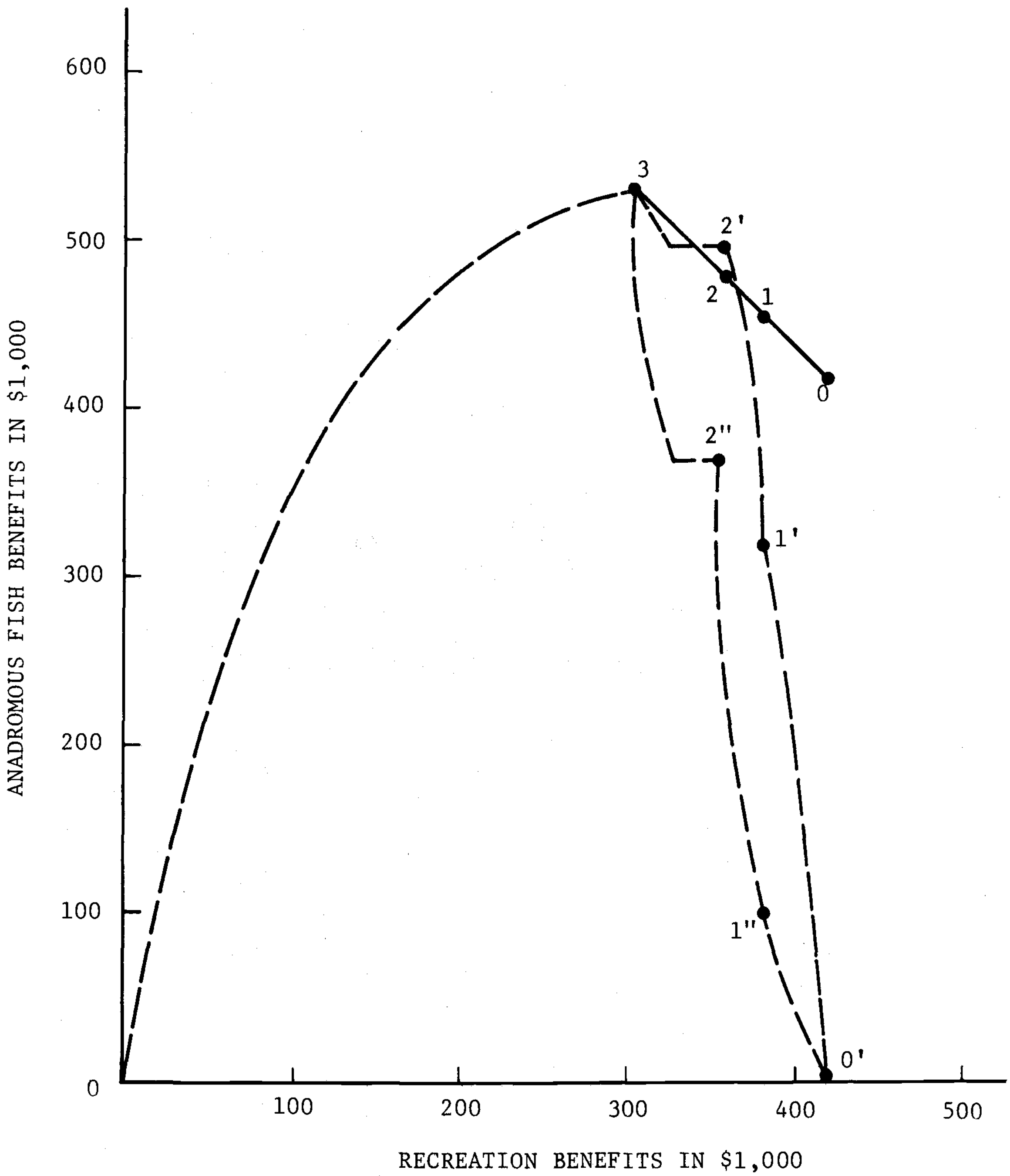

Figure 42. Iso-Revenues and Expected Revenues for Fish Enhancement and Reservoir Recreation, Proposed Holley Project.

fish benefits for strategy three. Thus, the recreational benefits are assumed known for points two, one, and zero. The fish benefits associated with each of these points represent the amount that would have to occur to make a particular strategy at least as attractive as strategy three.

The iso-revenue line forms a base or standard for comparing the expected revenue mix with the assumed known point on the iso-revenue line, point three. Any strategy that would result in a revenue mix represented by a point above and to the right of the solid line would be preferred over any point on or below the line. The most preferred point would be that which lies the greatest vertical distance above the iso-revenue line.[37]

The shape of the revenue possibilities curve in Figure 42 (dotted lines), from production of the two products, can be speculated upon. The dotted curve to the left of point three (maximum fish production) would result from strategies that would keep the reservoir at a low level during the summer months. Strategies of this type would result in decreasing anadromous fish and recreation benefits, hence the revenue possibilities curve approaches the left-hand origin.

At the right hand side of Figure 42, the dotted curve again approaches the origin. This represents a maximum level of

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[37]This assumes that the objective is to maximize revenue between these two project purposes.

recreation benefits resulting from a strategy requiring adherence to the rule curve. This strategy would result in a near duplication of without project downstream conditions and no fish enhancement benefits.

Two of the fishery consultants indicated that the predicted stream temperatures associated with strategy one would produce little or no fish enhancement over natural conditions. Thus, the attainable level of fish enhancement associated with strategy one probably lies close to the horizontal axis. For purposes of demonstration and because of the uncertainty involved, the dotted lines at 1' and 1'' are assumed to be the upper and lower bounds of fish enhancement revenue associated with strategy one.

The status of point two is also uncertain. While the fishery experts indicated that the level of fish production associated with the stream temperatures of strategy two would be less than under the temperature regime predicted by strategy three, the actual level of fish enhancement revenue generated by strategy two could lie above or below the iso-revenue line at point two, say at 2' or 2''.

The initial simulations indicated that strategies requiring a greater withdrawal rate than strategy two, but less than strategy three, would not improve the water temperature conditions over strategy two. Therefore, the level of fish production would not be expected to be greater than that produced under strategy two, until the magnitude of reservoir withdrawal approached that of strategy three. This is the reason that the dotted line to the left of points 2' and 2'' are nearly horizontal, indicating nearly constant fish production and decreasing recreation benefits due to an increasing reservoir withdrawal rate.

Figure 42 indicates that either strategy two or strategy three would produce the greatest total revenue from the two project purposes. It is important to note that if 2' were the relevant point for strategy two, then more total revenue could be produced by not attempting to maximize fish production as at point three.

It should also be pointed out that by comparing the figures in Tables 8 and 11, the separable benefit-cost ratio for recreation is shown to be favorable for strategy one under both 1967 and 1968 conditions, but is unfavorable for strategies two and three. Although the total project benefit-cost ratio would still be favorable, due to the unfavorable separable recreation B-C ratios generated by strategies two and three, the procedure used by the Corps of Engineers would require the rejection of this particular project.

An alternative project would be considered without recreation as a project purpose. This alternative project would be identical to the one analyzed in this study, with the exception of no recreation facilities at the reservoir (U.S. Army Corps of Engineers, 1970). The separable benefit-cost ratios and total benefit-cost ratio for the low flow purposes would be shown in this demonstration to be favorable at an interest rate of 4 and 7/8 percent over a 100-year life.

Summary

This chapter has investigated the expected effects of the predicted river temperature and reservoir drawdown on the product mix and resulting benefits between anadromous fish enhancement and recreation.

It was shown that the maximum level of fish production might be approached by increasing the magnitude of reservoir withdrawal over those withdrawals proposed by the Corps of Engineers. However, this would reduce the benefits expected to accrue to reservoir recreation, thus making this latter project purpose unfeasible.

Finally, it was demonstrated that if the project were built and irrigation requirements were treated as a given, the objective of maximizing fish production would not necessarily produce the maximum benefits resulting from the low flow purposes.

VIII. SUMMARY AND CONCLUSIONS

Summary of Problem

A problem confronting the Corps of Engineers and other water resources planners in the Pacific Northwest is the prediction of anadromous fish enhancement benefits that would result from a proposed high dam project. These benefits are expected from augmenting natural streamflows with relatively cold reservoir withdrawals. The resulting increased streamflows and decreased river temperatures downstream are expected to sustain larger salmon populations than would occur without an impoundment.

In at least one project plan, the Holley Project for the Calapooia River, the Corps estimated the increase in salmon production from the expected downstream flows and water temperatures of the reservoir withdrawals. The agency did not address the summer season problem of the cool water at the base of the dam increasing in temperature as it travels downstream.

Because the river temperatures can be a limiting factor in salmon production, one approach to evaluating the effects of high dams on these anadromous fish is to predict the downstream river temperatures and streamflows resulting from high dam construction. Fish production could then be estimated from the values of these parameters. This was the approach used in this research in an attempt to estimate the fish enhancement benefits associated with the proposed Holley Dam.

The Holley project is multi-purpose in nature. Therefore, project purposes in addition to anadromous fish enhancement were considered in this study. For example, water released from the reservoir for downstream purposes (fish and irrigation) could be used to maintain reservoir elevations, which correlate with reservoir recreation use. Thus, a competitive nature exists between low flow purposes. Trade-offs become a consideration in allocating reservoir water; consequently, the objective of the economic analysis in this study became the maximization of low flow project purposes.

Summary of Water Temperature Model

A computer simulation model was developed to continuously predict reservoir and downstream temperatures and mass flows. The reservoir submodel constructed for this study was based upon the stratified reservoir model developed at MIT. A downstream bulk flow river temperature and routing algorithm, solar flux submodel, and several decision submodels were constructed and incorporated into the system.

It is not known if the total cost of operating the continuous model constructed in this study is greater than the total cost of independently operating the M. I. T. reservoir model and a traditional river temperature model. The computation costs of operating only the river temperature submodel of this study would be higher than for a traditional river model. However, operation of the general water temperature model of this study would not require reading, storing, and handling data two or more times as would be necessary using separate reservoir and river temperature models.

The model developed in this study provides more flexibility than do the traditional models reviewed. For instance, the water temperature model permits determination of all values of mass flows, depths, and water temperatures at any designated point in time for all points along a river, and for all reservoir elevations. The model structure allows decision routines to be called at each time interval update for determining reservoir withdrawals, the amount of withdrawal from each of three reservoir outlets, and the channel withdrawal for irrigation.

Three specific areas of water-related research can be singled out where this work appears to have the most potential for impact. The first problem relates to predicting or monitoring water quality on a continuous basis. The model could be operated by agency personnel who have little understanding of computer programming or the internal logic of the model. Also, the model is structured so that any water quality parameter, not just temperature, could be simulated with the downstream routing algorithm.

The second area is in the realm of project planning and estimation of benefits, particularly those benefits associated with water quality.

Finally, the third problem area is the evaluation of operational efficiency of existing multiple objective water resources projects. A research effort on this problem might involve an initial postdiction study of an existing project. The economic output resulting from this initial study could then be compared with the output from a prediction study. The same historical physical parameters could be used for the prediction study, but the rules for water allocation would be changed to determine if and how the economic output and distribution of this output would be altered or increased under various management policies.

Conclusions of Holley Project Analysis

Although the results of the computer simulations of the proposed Holley Project cannot be interpreted as conclusive, a number of important findings emerged. These are:

1. It was predicted that a greater mass of cold water would be available for release from Holley Reservoir than was predicted by either the Corps of Engineers or the Federal Water Quality Administration.

2. Within a certain range, an increase in reservoir withdrawal rate will likely decrease the average river depth, resulting in higher river temperatures.
3. No reservoir withdrawal strategy could be found that was capable of maintaining the downstream river temperatures within the optimum range for Pacific salmon.
4. The temperatures of the reservoir withdrawals would have little or no effect upon either the maximum or minimum river temperatures beyond 24 miles below the dam site.
5. There is a low probability (1/50,000) that the reservoir rule curve could be adhered to throughout any summer season while releasing the minimum downstream flow requirements for anadromous fish.
6. Because the Calapooia River divides into two channels, each approximately eight miles in length, river temperatures of the magnitudes now being recorded at Holley and Albany would be present in the two channels under the minimum withdrawal requirements specified by the Corps of Engineers.

The conclusions associated with the economic analysis are:

1. Attempts to maximize fish enhancement benefits may not yield the maximum total revenue from the proposed project.

2. It is doubtful that the level of anadromous fish and recreation benefits predicted by the Corps of Engineers can be achieved.

Limitations and Implications for Future Research

The reliability of the information reported in this study is dependent upon the assumptions and data used in the model. Consequently, several limitations need to be explicitly stated where improvements in data and/or model specification could increase the predictive capabilities of the water temperature model.

Improvements in the hydrometeorological data would probably have yielded more accurate predictions of the physical parameters modeled in this study. For example, dye studies could have provided additional information on water speeds which could have been compared to, or substituted for, the water speeds calculated by the model.

Data limitations also prevented a complete validation analysis of the ability of the model to accurately predict river temperatures below the proposed Holley reservoir. Additional water temperature and streamflow recording gauges located below the dam site would have provided the necessary data to complete the validation of the Calapooia basin river temperature submodel.

The availability of additional computer core storage would have allowed for the inclusion of stochastic hydrometeorological elements, resulting in a more realistic time series data set.

The economic analysis in this study is limited in that it is partial in nature and deals with limited information describing fish production. Analytical consideration is restricted to the low flow project purposes of anadromous fish enhancement, recreation, and irrigation, with the latter treated as a given. Reservoir recreation use is assumed to be a function of time (year) and beach length only. The production of anadromous fish is assumed to be a function of time (year), river temperatures, and streamflows only.

Reliability of the economic information generated by the simulation model could be improved by future research efforts into several areas. First, the procedure for estimating recreation benefits could be improved by including prices and incomes in a dynamic algorithm to investigate future recreation demand for a proposed project.

Second, the inclusion of flood control as a project purpose in the model structure would broaden the economic analysis and would then depict a more realistic situation, as flood control is the major purpose in most high dam projects. Also, flood control is not independent of the low flow project purposes as was assumed in this study.

Finally, this study demonstrated that considerable resources can be used to predict the values of environmental factors that help determine salmonid production. However, reliable estimates of the level of salmonid production resulting from high dams will not be forthcoming until more explicit information describing the interaction of environmental factors and their effects on the production process becomes available.

The complexities of the natural bio-system, coupled with exogenous man-induced controls, suggest that a dynamic systems approach, using the information generated by the model developed in this study, could significantly increase the reliability of anadromous fish production estimates.

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TOTAL OUTFLOW=   240.000          OUTFLOW TEMP=     8.087          DAY OF YEAR=  192          TOTAL INFLOW=  71.00000
UNITS IN METERS, DAYS, KCALS, KGRAMS, AND DEG. CENT. UNLESS STATED OTHERWISE.
ELAPSED TIME=  88.00                     ACTUAL SURFACE ELEVATION=      0            INFLOW TEMPERATURE= 19.64
NO. OF TIME STEPS=   8                   SURFACE ELEVATION USED=   205.82            AIR TEMPERATURE= 22.43
NO. OF GRID POINTS= 29                   ELEVATION OF INFLOW= 202.00                 RELATIVE HUMIDITY=  .63
LEVEL OF INFLOW= 27                      EVAPORATION FLUX= 3.37271E 03               INSOLATION FLUX= 8.58864E 03
MIXING DEPTH= 2.25                       ATMOSPHERIC RADIATION=        0E 00         WIND SPEED= 2.59
HEAT LOSS FLUX= 4.91710E 03              RADIATION FLUX= 1.54439E 03                 INFLOW RATE=    173706.9
EPSILON= 3.4161E-05                      WITHDRAWAL THICKNESS=  39.00                OUTFLOW STD. DEV.=  9.95
OUTLET ELEVATION= 205.00000               OUTFLOW RATE=          0                    OUTFLOW TEMPERATURE=      0C
OUTLET ELEVATION= 196.00000               OUTFLOW RATE=          0                    OUTFLOW TEMPERATURE=      0C
OUTLET ELEVATION= 166.00000               OUTFLOW RATE=  587178.1                     OUTFLOW TEMPERATURE=  8.09C
MAX OUTFLOW VELOCITY=          0
MAX OUTFLOW VELOCITY=          0
MAX OUTFLOW VELOCITY=    100.423
MAX INFLOW VELOCITY=     15.480
ENERGY RATIO= -1.529     ENERGY STORED= 2.29716E 12     ENERGY INFLOW= 5.56532E 12   ENERGY OUTFLOW= 6.23319E 12
TEMP OF MIXING LAYER= 19.28              MIXED INFLOW TEMP= 19.46                    TOTAL ENERGY RATIO=  1.24686
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J	ELEV	TEMP(C)	J	ELEV	TEMP(C)	J	ELEV	TEMP(C)	J	ELEV	TEMP(C)	J	ELEV	TEMP(C)	J	ELEV	TEMP(C)	J	ELEV	TEMP(C)
1	163.0	7.29	11	178.0	8.00	21	193.0	9.63												
2	164.5	7.30	12	179.5	8.01	22	194.5	10.47												
3	166.0	7.31	13	181.0	8.03	23	196.0	11.66												
4	167.5	7.32	14	182.5	8.05	24	197.5	13.27												
5	169.0	7.33	15	184.0	8.09	25	199.0	15.36												
6	170.5	7.34	16	185.5	8.15	26	200.5	17.66												
7	172.0	7.35	17	187.0	8.25	27	202.0	20.00												
8	173.5	7.36	18	188.5	8.41	28	203.5	22.45												
9	175.0	7.37	19	190.0	8.66	29	205.0	23.74												
10	176.5	7.39	20	191.5	9.05															

DRY BULB= 61.00000 REL. HUM.= .90667 WIND= 2.30000 CLOUDS= 0 DAILY FLUX= 7729.77707

DAY	TIME PRD.	FLOW PRD.	SUB PRD.	STA.	TEMP F	TEMP C	FLUX	VEL.	FLOW	DEPTH	TRAV TIME	REACH LENGTH	ACCUM LENGTH
88	16	16	1	2	46.594	8.108	0	2.022	240.000	1.272	.962	7000.000	1.326
88	16	16	1	3	46.684	8.158	0	4.978	240.000	.965	.636	11400.000	3.485
88	16	15	1	4	47.056	8.364	0	1.875	240.000	2.096	1.733	11700.000	5.701
88	16	15	1	5	47.098	8.388	0	1.489	240.000	2.780	.093	500.000	5.795
88	16	15	1	6	47.854	8.808	0	2.715	240.000	1.564	1.627	15900.000	8.807
88	16	15	1	7	48.524	9.180	0	1.885	240.000	1.246	.793	5380.000	9.826
88	16	15	2	8	48.525	9.181	0	2.498	150.000	1.630	.001	10.000	9.828
88	16	15	2	9	48.526	9.181	0	4.947	150.000	.932	.001	10.000	9.830
88	16	14	3	10	49.643	9.802	0	4.205	150.000	.904	1.120	16950.000	13.040
88	16	14	3	11	50.204	10.113	0	2.124	150.000	1.701	.438	3350.000	13.674
88	16	13	3	12	54.077	12.265	0	1.494	150.000	1.478	3.000	16800.000	16.856
88	16	13	2	13	56.236	13.464	0	.606	150.000	2.029	1.421	3100.000	17.443
88	16	8	3	14	63.573	17.540	0	.369	150.000	2.624	3.000	17800.000	20.814
88	16	4	3	15	64.286	17.937	0	.415	150.000	2.473	3.000	17800.000	24.186

DRY BULB= 57.00000 REL. HUM.= .96000 WIND= 1.15000 CLOUDS= 0 DAILY FLUX= 7729.77707

DAY	TIME PRD.	FLOW PRD.	SUB PRD.	STA.	TEMP F	TEMP C	FLUX	VEL.	FLOW	DEPTH	TRAV TIME	REACH LENGTH	ACCUM LENGTH
88	17	17	1	2	46.714	8.174	171.151	2.022	240.000	1.272	.962	7000.000	1.326
88	17	17	1	3	46.823	8.235	171.151	4.978	240.000	.965	.636	11400.000	3.485
88	17	16	1	4	46.911	8.284	171.151	1.875	240.000	2.096	1.733	11700.000	5.701
88	17	16	1	5	46.922	8.290	171.151	1.489	240.000	2.780	.093	500.000	5.795
88	17	16	1	6	47.221	8.456	171.151	2.715	240.000	1.564	1.627	15900.000	8.807
88	17	16	1	7	47.506	8.615	171.151	1.885	240.000	1.246	.793	5380.000	9.826
88	17	16	2	8	47.507	8.615	171.151	2.498	150.000	1.630	.001	10.000	9.828
88	17	16	2	9	47.507	8.615	171.151	4.947	150.000	.932	.001	10.000	9.830
88	17	15	3	10	48.098	8.944	171.151	4.205	150.000	.904	1.120	16950.000	13.040
88	17	15	3	11	48.365	9.092	171.151	2.124	150.000	1.701	.438	3350.000	13.674
88	17	14	3	12	50.569	10.316	171.151	1.494	150.000	1.478	3.000	16800.000	16.856
88	17	14	2	13	52.262	11.256	171.151	.606	150.000	2.029	1.421	3100.000	17.443
88	17	9	3	14	64.250	17.917	171.151	.369	150.000	2.624	3.000	17800.000	20.814
88	17	5	3	15	62.311	16.839	171.151	.415	150.000	2.473	3.000	17800.000	24.186

DRY BULB= 65.00000 REL. HUM.= .78667 WIND= 1.53333 CLOUDS= 0 DAILY FLUX= 7729.77707

DAY	TIME PRD.	FLOW PRD.	SUB PRD.	STA.	TEMP F	TEMP C	FLUX	VEL.	FLOW	DEPTH	TRAV TIME	REACH LENGTH	ACCUM LENGTH
88	18	18	1	2	48.787	9.326	1447.347	2.022	240.000	1.272	.962	7000.000	1.326
88	18	18	1	3	50.252	10.140	1447.347	4.978	240.000	.965	.636	11400.000	3.485
88	18	17	1	4	50.664	10.369	1447.347	1.875	240.000	2.096	1.733	11700.000	5.701
88	18	17	1	5	50.641	10.356	1447.347	1.489	240.000	2.780	.093	500.000	5.795
88	18	17	1	6	51.621	10.901	1447.347	2.715	240.000	1.564	1.627	15900.000	8.807
88	18	17	1	7	52.248	11.249	1447.347	1.885	240.000	1.246	.793	5380.000	9.826
88	18	17	2	8	52.248	11.249	1447.347	2.498	150.000	1.630	.001	10.000	9.828
88	18	17	2	9	52.249	11.249	1447.347	4.947	150.000	.932	.001	10.000	9.830
88	18	16	3	10	53.961	12.201	1447.347	4.205	150.000	.904	1.120	16950.000	13.040
88	18	16	3	11	53.802	12.112	1447.347	2.124	150.000	1.701	.438	3350.000	13.674
88	18	15	3	12	54.314	12.397	1447.347	1.494	150.000	1.478	3.000	16800.000	16.856
88	18	15	2	13	54.451	12.473	1447.347	.606	150.000	2.029	1.421	3100.000	17.443
88	18	10	3	14	66.425	19.125	1447.347	.369	150.000	2.624	3.000	17800.000	20.814
88	18	6	3	15	64.270	17.928	1447.347	.415	150.000	2.473	3.000	17800.000	24.186

DRY BULB= 77.66667 REL. HUM.= .54333 WIND= 6.90000 CLOUDS= 0 DAILY FLUX= 7729.77707

DAY	TIME PRD.	FLOW PRD.	SUB PRD.	STA.	TEMP F	TEMP C	FLUX	VEL.	FLOW	DEPTH	TRAV TIME	REACH LENGTH	ACCUM LENGTH
88	19	19	1	2	50.697	10.387	2441.733	2.022	240.000	1.272	.962	7000.000	1.326
88	19	19	1	3	53.762	12.090	2441.733	4.978	240.000	.965	.636	11400.000	3.485
88	19	18	1	4	56.010	13.339	2441.733	1.875	240.000	2.096	1.733	11700.000	5.701
88	19	18	1	5	56.007	13.337	2441.733	1.489	240.000	2.780	.093	500.000	5.795
88	19	18	1	6	58.376	14.653	2441.733	2.715	240.000	1.564	1.627	15900.000	8.807
88	19	18	1	7	59.644	15.358	2441.733	1.885	240.000	1.246	.793	5380.000	9.826
88	19	18	2	8	59.644	15.358	2441.733	2.498	150.000	1.630	.001	10.000	9.828
88	19	18	2	9	59.645	15.358	2441.733	4.947	150.000	.932	.001	10.000	9.830
88	19	17	3	10	62.795	17.108	2441.733	4.205	150.000	.904	1.120	16950.000	13.040
88	19	17	3	11	62.623	17.013	2441.733	2.124	150.000	1.701	.438	3350.000	13.674
88	19	16	3	12	63.996	17.776	2441.733	1.494	150.000	1.478	3.000	16800.000	16.856
88	19	16	2	13	62.593	16.996	2441.733	.606	150.000	2.029	1.421	3100.000	17.443
88	19	11	3	14	68.883	20.491	2441.733	.369	150.000	2.624	3.000	17800.000	20.814
88	19	7	2	15	69.364	20.758	2441.733	.415	150.000	2.473	3.000	17800.000	24.186

DRY BULB= 86.66667 REL. HUM.= .40333 WIND= 8.81667 CLOUDS= 0 DAILY FLUX= 7729.77707

DAY	TIME PRD.	FLOW PRD.	SUB PRD.	STA.	TEMP F	TEMP C	FLUX	VEL.	FLOW	DEPTH	TRAV TIME	REACH LENGTH	ACCUM LENGTH
88	20	20	1	2	50.333	10.463	2350.575	2.022	240.000	1.272	.962	7000.000	1.326
88	20	20	1	3	54.347	12.415	2350.575	4.978	240.000	.965	.636	11400.000	3.485
88	20	19	1	4	58.242	14.579	2350.575	1.875	240.000	2.096	1.733	11700.000	5.701
88	20	19	1	5	58.331	14.629	2350.575	1.489	240.000	2.780	.093	500.000	5.795
88	20	19	1	6	62.097	16.721	2350.575	2.715	240.000	1.564	1.627	15900.000	8.807
88	20	19	1	7	63.910	17.728	2350.575	1.885	240.000	1.246	.793	5380.000	9.826
88	20	19	2	8	63.911	17.728	2350.575	2.498	150.000	1.630	.001	10.000	9.828
88	20	19	2	9	63.913	17.729	2350.575	4.947	150.000	.932	.001	10.000	9.830
88	20	18	3	10	67.584	19.769	2350.575	4.205	150.000	.904	1.120	16950.000	13.040
88	20	18	3	11	67.826	19.903	2350.575	2.124	150.000	1.701	.438	3350.000	13.674
88	20	17	3	12	71.171	21.762	2350.575	1.494	150.000	1.478	3.000	16800.000	16.856
88	20	17	2	13	70.243	21.246	2350.575	.606	150.000	2.029	1.421	3100.000	17.443
88	20	12	3	14	70.248	21.249	2350.575	.369	150.000	2.624	3.000	17800.000	20.814
88	20	8	3	15	74.171	23.428	2350.575	.415	150.000	2.473	3.000	17800.000	24.186

DRY BULB= 89.00000 REL. HUM.= .36000 WIND= 9.58333 CLOUDS= 0 DAILY FLUX= 7729.77707

DAY	TIME PRD.	FLOW PRD.	SUB PRD.	STA.	TEMP F	TEMP C	FLUX	VEL.	FLOW	DEPTH	TRAV TIME	REACH LENGTH	ACCUM LENGTH
88	21	21	1	2	49.214	9.563	1233.567	2.022	240.000	1.272	.962	7000.000	1.326
88	21	21	1	3	51.743	10.968	1233.567	4.978	240.000	.965	.636	11400.000	3.485
88	21	20	1	4	55.745	13.192	1233.567	1.875	240.000	2.096	1.733	11700.000	5.701
88	21	20	1	5	55.916	13.287	1233.567	1.489	240.000	2.780	.093	500.000	5.795
88	21	20	1	6	59.965	15.536	1233.567	2.715	240.000	1.564	1.627	15900.000	8.807
88	21	20	1	7	61.967	16.648	1233.567	1.885	240.000	1.246	.793	5380.000	9.826
88	21	20	2	8	61.969	16.649	1233.567	2.498	150.000	1.630	.001	10.000	9.828
88	21	20	2	9	61.970	16.650	1233.567	4.947	150.000	.932	.001	10.000	9.830
88	21	19	3	10	65.158	18.421	1233.567	4.205	150.000	.904	1.120	16950.000	13.040
88	21	19	3	11	65.926	18.848	1233.567	2.124	150.000	1.701	.438	3350.000	13.674
88	21	18	3	12	70.453	21.363	1233.567	1.494	150.000	1.478	3.000	16800.000	16.856
88	21	18	2	13	71.350	21.861	1233.567	.606	150.000	2.029	1.421	3100.000	17.443
88	21	13	3	14	68.931	20.517	1233.567	.369	150.000	2.624	3.000	17800.000	20.814
88	21	9	3	15	75.047	23.915	1233.567	.415	150.000	2.473	3.000	17800.000	24.186

DRY BULB= 77.33333 REL. HUM.= .42667 WIND= 12.26667 CLOUDS= 0 DAILY FLUX= 7729.77707

DAY	TIME PRD.	FLOW PRD.	SUB PRD.	STA.	TEMP F	TEMP C	FLUX	VEL.	FLOW	DEPTH	TRAV TIME	REACH LENGTH	ACCUM LENGTH
88	22	22	1	2	46.951	8.306	85.404	2.022	240.000	1.272	.962	7000.000	1.326
88	22	22	1	3	47.734	8.741	85.404	4.978	240.000	.965	.636	11400.000	3.485
88	22	21	1	4	50.257	10.143	85.404	1.875	240.000	2.096	1.733	11700.000	5.701
88	22	21	1	5	50.435	10.242	85.404	1.489	240.000	2.780	.093	500.000	5.795
88	22	21	1	6	53.333	11.852	85.404	2.715	240.000	1.564	1.627	15900.000	8.807
88	22	21	1	7	54.925	12.736	85.404	1.885	240.000	1.246	.793	5380.000	9.826
88	22	21	2	8	54.927	12.737	85.404	2.498	150.000	1.630	.001	10.000	9.828
88	22	21	2	9	54.928	12.738	85.404	4.947	150.000	.932	.001	10.000	9.830
88	22	20	3	10	56.876	13.820	85.404	4.205	150.000	.904	1.120	16950.000	13.040
88	22	20	3	11	57.927	14.404	85.404	2.124	150.000	1.701	.438	3350.000	13.674
88	22	19	3	12	62.436	16.909	85.404	1.494	150.000	1.478	3.000	16800.000	16.856
88	22	19	2	13	64.946	18.303	85.404	.606	150.000	2.029	1.421	3100.000	17.443
88	22	14	3	14	65.095	18.386	85.404	.369	150.000	2.624	3.000	17800.000	20.814
88	22	10	3	15	70.560	21.422	85.404	.415	150.000	2.473	3.000	17800.000	24.186

DRY BULB= 65.33333 REL. HUM.= .63667 WIND= 3.83333 CLOUDS= 0 DAILY FLUX= 7729.77707

DAY	TIME PRD.	FLOW PRD.	SUB PRD.	STA.	TEMP F	TEMP C	FLUX	VEL.	FLOW	DEPTH	TRAV TIME	REACH LENGTH	ACCUM LENGTH
88	23	23	1	2	46.479	8.044	0	2.022	240.000	1.272	.962	7000.000	1.326
88	23	23	1	3	46.512	8.062	0	4.978	240.000	.965	.636	11400.000	3.485
88	23	22	1	4	47.112	8.396	0	1.875	240.000	2.096	1.733	11700.000	5.701
88	23	22	1	5	47.206	8.448	0	1.489	240.000	2.780	.093	500.000	5.795
88	23	22	1	6	48.745	9.303	0	2.715	240.000	1.564	1.627	15900.000	8.807
88	23	22	1	7	49.798	9.888	0	1.885	240.000	1.246	.793	5380.000	9.826
88	23	22	2	8	49.800	9.889	0	2.498	150.000	1.630	.001	10.000	9.828
88	23	22	2	9	49.801	9.889	0	4.947	150.000	.932	.001	10.000	9.830
88	23	21	3	10	51.294	10.719	0	4.205	150.000	.904	1.120	16950.000	13.040
88	23	21	3	11	52.016	11.120	0	2.124	150.000	1.701	.438	3350.000	13.674
88	23	20	3	12	56.714	13.730	0	1.494	150.000	1.478	3.000	16800.000	16.856
88	23	20	2	13	58.850	14.917	0	.606	150.000	2.029	1.421	3100.000	17.443
88	23	15	3	14	64.020	17.789	0	.369	150.000	2.624	3.000	17800.000	20.814
88	23	11	3	15	67.499	19.722	0	.415	150.000	2.473	3.000	17800.000	24.186