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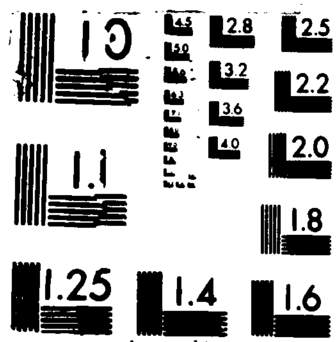
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WATER OPERATIONS TECHNICAL SUPPORT PROGRAM

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PROCEEDINGS: CE WORKSHOP ON RESERVOIR RELEASES

Hydraulics Laboratory

DEPARTMENT OF THE ARMY
Waterways Experiment Station, Corps of Engineers
PO Box 631, Vicksburg, Mississippi 39180-0631

July 1987
Final Report

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During 28-30 October 1986, a workshop on reservoir releases was conducted in Atlanta, Georgia. The objective of the workshop was to provide a forum for the exchange of information related to reservoir release water quality problems, potential solution techniques, and evaluation or predictive techniques. Papers were solicited from across the Corps of Engineers (CE), the Bureau of Reclamation, and the Tennessee Valley Authority on reservoir releases and enhancement/improvement techniques.

Twenty-seven papers were presented in five categories: (1) Water Quality Policy; (2) Measurement, Evaluation, and Prediction Techniques; (3) Operational and Tailwater Techniques; (4) In-Structure Techniques; and (5) In-Reservoir Techniques. Generalized and specific case studies were topics in each category.

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EXECUTIVE SUMMARY

During 28-30 October 1986, the Office, Chief of Engineers (OCE), under the Water Operations Technical Support (WOTS) Program sponsored a workshop on reservoir releases, which was conducted by the US Army Engineer Waterways Experiment Station (WES). OCE, WES, and Divisions and Districts from across the Corps of Engineers (CE) and other Federal and local agencies were represented by participants and presenters at the workshop. A list of workshop attendees is included in the workshop proceedings. The objective of the workshop was to provide a forum for the exchange of information related to reservoir release water quality problems and potential evaluation and solution techniques and methodologies to improve reservoir releases. Papers were solicited from across the CE, the Bureau of Reclamation, and the Tennessee Valley Authority (TVA). Examination of the Table of Contents and the workshop Agenda will reveal the breadth of the topics discussed in the twenty-seven papers presented at the workshop. In overview, the papers were presented in five categories: (1) Water Quality Policy; (2) Measurement, Evaluation, and Prediction Techniques; (3) Operational and Tailwater Techniques; (4) In-Structure Techniques; and (5) In-Reservoir Techniques. Generalized and specific case studies of reservoir release water quality problems and solutions were presented.

In general terms, papers in Session I: Water Quality Policy, covered some of the issues that have impacted and shaped the existing posture regarding reservoir release water quality. Representatives of the OCE and TVA presented papers on each agency's policies regarding the quality of reservoir releases. Both agencies have clear objectives to maintain suitable aquatic habitat downstream of reservoir projects and to improve the release quality when possible within the guidelines of authorized project purposes. Specifically identified as a developing policy question is the challenge to develop a strong policy of nondegradation of release water quality at projects where non-Federal hydropower is proposed. TVA and CE, based on the papers in this session, appear to be in transition from primarily identification and assessment of water quality problems to emphasizing the monitoring and management of water quality. To these ends, remote satellite data acquisition equipment is being developed and deployed to permit the effective use of fiscal and manpower resources to achieve the monitoring/management objectives.

Papers in Sessions IIA and IIB: Measurement, Evaluation, and Prediction Techniques, contained generalized topics related to existing capability to model and thereby evaluate various aspects of water quality in reservoirs and regulated streams. Quality parameters of interest were temperature, dissolved oxygen (DO), and suspended sediment. Other papers dealt with the effects of and methods to evaluate the impacts of reservoir releases on tailwaters. A general paper on modeling provided an overview of several approaches available to evaluate reservoir release water quality with a brief review of two applications of these modeling techniques. A case study was presented on the application of a one-dimensional model for simulating the transport of suspended sediment into and through a reservoir. Other papers presented case studies on the evaluation of a modified reservoir withdrawal structure, the potential impacts on in-lake and release temperatures of a pool raise and storage reallocation, and the application of a reservoir system model for real-time control of DO. Two papers focused on predicting the effects of release improvement on project tailwaters below several TVA reservoir projects and Buford Dam, a CE project, on the Chattahoochee River. Of particular interest in these two papers were the effects of providing minimum flows downstream of a reservoir and the application of an aquatic habitat model PHABSIM.

The papers in Session III: Operational and Tailwater Techniques, presented case studies on the data collection and analysis to determine the causes of low DO downstream of a hydropower project. A case study was presented on a project where provisions were made to provide minimum flows downstream of a hydropower project. Two papers presented applications of a numerical model of selective withdrawal that can predict release quality characteristics and can provide guidance on the operation of a multilevel outlet structure.

In Session IV: In-Structure Techniques, the results of generalized research and specific applications of the concept of blending in a single wet well to control release water temperature were presented. Additionally, several applications of turbine venting to increase the DO content of releases from hydropower projects were discussed.

In the last session, Session V: In-Reservoir Techniques, in-lake aeration and oxygenation systems were the topics of several papers that outlined experiences at TVA and CE projects. At both, the oxygenation system was designed to improve the reservoir DO concentration in a local area. At the CE project, the water that was improved was for release quality maintenance. At the TVA project, the locally improved water was to provide a haven for striped bass. Case studies were presented relating results of applications of localized destratification for release improvement. These studies specifically dealt with the use of an epilimnetic pump and early experiences with a pneumatic destratification system. Another paper developed the design of a local hydraulic destratification system. Of particular importance was a paper from the Bureau of Reclamation that outlined a design procedure for pneumatic destratification systems. Additionally, the results of operating a retrofitted selective withdrawal structure were presented.

Appreciation must be expressed to OCE for sponsoring the workshop through the WOTS Program; to the session chairmen for efficiently presiding over their topic areas; to the presenters who provided the information and technology transfer; to Mr. Dennis Barnett who was the local point of contact from the South Atlantic Division; to Mr. William N. Rushing who made all the physical arrangements; and to personnel of the WES Hydraulics Laboratory who conducted the workshop.

PREFACE

This report presents the proceedings of the CE Workshop on Reservoir Releases that was held in Atlanta, Georgia, 28-30 October 1986. The workshop was funded by the Water Operations Technical Support (WOTS) Program, which is sponsored by the Office, Chief of Engineers (OCE). The workshop was organized and conducted by personnel of the US Army Engineer Waterways Experiment Station (WES). Mr. Steven C. Wilhelms of the Reservoir Water Quality Branch (RWQB), WES Hydraulics Laboratory, and Dr. Richard E. Price, Acting Chief, RWQB, were the principal conference coordinators. Mr. William N. Rushing, Assistant Program Manager of WOTS, WES Environmental Laboratory, made arrangements for the workshop facilities and served as point of contact for submission of abstracts and papers. Messrs. Wilhelms and Rushing and Dr. Price were assisted in workshop administration by Ms. Laurin I. Yates. Messrs. Jeffery P. Holland, Chief, RWQB, John L. Grace, Jr., Chief, Hydraulic Structures Division, and F. A. Herrmann, Jr., Chief, Hydraulics Laboratory, directed the effort. Mr. Wilhelms prepared the Executive Summary and organized the remainder of this report.

Dr. Jerome L. Mahloch, Environmental Laboratory, WES, was Manager of the WOTS Program. The Technical Monitor of the WOTS Program for OCE was Mr. David P. Buelow.

COL Dwayne G. Lee, CE, was the Commander and Director of WES. Dr. Robert W. Whalin was Technical Director.

TABLE OF CONTENTS

AGENDA
Workshop on Reservoir Releases
28-30 October 1986
Hyatt Regency Ravinia
Atlanta, Georgia

Monday, 27 October

Time	Topic	Speaker	Office
1830	Reception (Cash Bar)		

Tuesday, 28 October

0745-0830	Registration		
0830-0845	Welcome	BG C. Ernest Edgar, III	SAD
0845-0915	Workshop Overview	Richard Price	WES
SESSION I:	**Water Quality Policy**		
CHAIRMAN:	**Dennis Barnett, SAD**		
0915-0945	Corps' Policy Review	David P. Buelow	OCE
0945-1015	Policy Issues Associated with Reservoir Release	John S. Crossman	TVA
1015-1030	Break		
1030-1100	TVA Reservoir Improvements: An Overview	Bevan W. Brown	TVA
1100-1130	Status of The Water Quality Control Mission in ORD and Needs for Research Support	Mark Anthony	ORD
1130-1200	Panel Discussion/Questions		
1200-1300	Lunch		
SESSION IIA:	**Measurement, Evaluation, and Prediction Techniques**		
CHAIRMAN:	**Mark Dortch, WES**		
1300-1330	WESTEX Revisited: Predicted Verses Observed Reservoir Turbidity	J. D. Graham	NPP
1330-1400	Water Quality Modeling of Regulated Streams	M. S. Dortch	WES
1400-1415	Break		

Tuesday, 28 October (con)

Time	Topic	Speakers	Office
1415-1445	Effects of Aeration and Minimum Flow on the Biota and Fishery of Norris Tailwater	W. M. Seawell	TVA
1445-1515	Predicting Effects of Reregulation Downstream of Buford Dam, Georgia	John M. Nestler	WES
1515-1530	Break		
1530-1600	Panel Discussion/Questions		
1600-1615	Announcements		

Wednesday, 29 October

SESSION IIB: Measurement, Evaluation, and Prediction Techniques
Chairman: Jeff Holland, WES

Time	Topic	Speakers	Office
0800-0830	HEC-5Q: A Handy Tool or Monkey Wrench	Richard E. Punnett	ORH
0830-0900	Lake Greeson and Little Missouri River Modeling Studies	D. R. Johnson	LMK
0900-0930	Howard A. Hanson, Washington, Temperature Analysis Mathematical Modeling Investigation	M. L. Schneider	WES
0930-1000	Water Quality Monitoring Using Satellite Data Transmission	Steven D. Hiebert	USBR
1000-1015	Break		
1015-1045	Panel Discussion/Questions		

SESSION III: Operational and Tailwater Techniques
Chairman: Richard Punnett, ORH

Time	Topic	Speakers	Office
1045-1115	Dissolved Oxygen Studies Below Walter F. George Dam	Diane I. Findley Kenneth Day	SAM
1115-1145	Providing Minimum Flows Below Hydropower Projects	J. S. Adams, Jr.	TVA
1145-1215	Selective Withdrawal Structure Operation	S. C. Wilhelms	WES
1215-1315	Lunch		
1315-1345	Application of the SELCIDE Model in the Nashville District	Jack Brown	ORN

Wednesday, 29 October, (con)

Time	Topic	Speakers	Office
1345-1355	System Spill Allocation for the Control of Dissolved Gas Saturation on the Columbia River	Jeffrey D. Hanson	NPP
1355-1415	Panel Discussion/Questions		
1415-1430	Break		
SESSION IV:	**In-Structure Techniques**		
Chairman:	**Jerry Mahloch, WES**		
1430-1500	Recent Advances in Blending Technology and Their Application	Stacy Howington	WES
1500-1530	Single Wet Well Blending at Applegate Lake, Oregon	Jeffrey D. Hanson	NPP
1530-1600	Recent Developments in Turbine Aeration	E. Dean Harshbarger	TVA
1600-1630	Panel Discussion/Questions		
1830	Cocktails and Dinner at CHEQUERS (Dutch Treat)		

Thursday, 30 October

0800-0815	Announcements		
SESSION V:	**In-Reservoir Techniques**		
Chairman:	**Harold Sansing, ORN**		
0815-0845	Oxygenation of Releases from Richard B. Russell Dam	Gary V. Mauldin	SAS
0845-0915	In-Reservoir Aeration Systems	C. E. Bohac	TVA
0915-0945	Epilimnetic Pumps to Improve Reservoir Releases	M. H. Mobley	TVA
0945-1000	Break		
1000-1030	Local Destratification System for Mark Twain Lake	Steven C. Wilhelms Thomas J. Furdek	WES LMS
1030-1100	Lake Allatoona - An Early Experience in Destratification Revisited	N. D. McClure	SAM
1100-1130	Pneumatic Diffusers	P. L. Johnson	USBR
1130-1200	Retro-Fitting for High-Level Releases to Improve Downstream Quality	Richard E. Punnett	ORH

Thursday, 30 October (con)

Time	Topic	Speakers	Office
1200-1230	Panel Discussion/Questions		
1230-1330	Lunch		
1330-1430	Summary		
1430-1500	Feed-Back		
1500	Adjourn		

POLICY ISSUES
ASSOCIATED WITH RESERVOIR RELEASES

by

John S. Crossman, Ph.D.
Environmental Scientist, Environmental Quality Staff
Tennessee Valley Authority
Knoxville, Tennessee

ABSTRACT

Reservoir releases from the Tennessee Valley Authority's (TVA) dams are adversely affecting approximately 340 miles of stream during late summer, low flow conditions. The parameter of greatest concern and the one most often failing to meet State and Federal water quality criteria is dissolved oxygen (DO). Unacceptably low DO concentrations in combination with reservoir operations that leave stream beds dry for up to 45 days per year have severely impacted tailwaters and have virtually eliminated any potential they offer. Because of changed public priorities, TVA is in the process of addressing and/or accommodating the nonstatutory demands being made on TVA's reservoir system. However, major policy questions that still need to be addressed include: (1) are reservoir releases being improved to restore beneficial uses or to develop the resource, (2) should the cost of improvements be paid by the ratepayer or taxpayer, and (3) what level of improvement should TVA strive for?

INTRODUCTION

The TVA integrated water control system of 40 dams comprises what is generally considered the most regulated river system in the world. Developed in accordance with a comprehensive plan mandated by the TVA Act, the Nation's fifth largest river is managed primarily for flood control and navigation, and as consistent with these statutory purposes it also provides hydroelectric power. Largely completed by the late 1950s, TVA's multipurpose reservoir system has been one of the cornerstones of the region's success in economic growth and development. Estimated benefits provided by the reservoir system for the 1934-83 period (expressed in 1982 dollars) are:

- Flood Control Benefits: Total flood control benefits from the TVA system are estimated to be $4,127 million . . . based on the cost of flood damages prevented.

- Navigation Benefits: Total navigation benefits of the TVA reservoir system are estimated to be $3,655 million . . . [as determined] by comparing the cost of moving freight via the Tennessee River to the next cheapest mode and route of transporation.

- Recreation Benefits: Total estimated benefits associated with recreational activities made possible by the reservoir system are $6,363 million.

- Hydroelectric Benefits: The total value of the benefits associated with the production of electricity by TVA's system of dams and reservoirs is between $8.3 billion and $12.9 billion depending on the method used to measure them, i.e., based on TVA average wholesale rates or national average wholesale rates (Anonymous TVA, 1985).[1]

As the Valley's economy has improved and public priorities have changed, TVA has successfully modified its operations to accommodate to some degree many of the nonstatutory demands made on the system. For example, TVA currently operates its system to meet water supply needs, to control mosquitos and aquatic vegetation, to reduce the adverse effects of downstream effluent releases such as thermal discharges, and to provide for recreational use. Among other things, these changes have helped eradicate malaria from the Valley, they support a fisheries valued at $425 million per year, and they support a recreation industry that draws 70 million visits per year (Mills and Jones, 1986).

NEW EXPECTIONS - TVA REGION

A study by TVA's Reservoir Development Branch indicates that the recreational use of TVA's reservoir system is expected to grow by over 10 million visits during the 1980s (Anonymous TVA, 1979). If the Labor Department's Bureau of Labor Statistics trend on high employment growth in amusement and recreation services holds true for the TVA area, recreation will be a growth industry deserving greater consideration in the way the Valley's water resources are managed.

In addition to the expected recreational demands, environmental concerns related to wastewater treatment and meeting water quality standards are placing new demands on the quality and quantity of releases from water resource projects. For example, in Tennessee

1. The real (inflation adjusted) rate of return is in the 7-10 percent range which is roughly double the return society received on investments made during the same period.

the governor's Safe Growth Team has noted that economic growth may be compromised by the inability of many communities to provide adequate wastewater treatment. This was summarized in a 1982 report to Governor Alexander in which the Safe Growth Team reported that:

> At present there are 89 communities in Tennessee where the inadequacies of wastewater treatment facilities are so great that water quality standards are regularly violated in the streams receiving these wastes. Approximately 630,000 Tennesseans live in these communities. A much larger number of our citizens are affected because of the widespread recreational use of the waters of the State (Smith, 1982).

When considered in conjunction with the following observations, it is no wonder that states are making water issues a priority.

- o During the 1975-82 period, the Nation was spending between $4 and $7 billion per year on municipal waste treatment facilities with approximately 75 percent of the monies being provided by the Environmental Protection Agency's construction grants program. Today this program has been scaled back to $2.4 billion per year and the Federal share has been reduced from 75 to 55 percent.

- o It is estimated that it will cost $1.2 billion to meet the wastewater treatment needs in just the Tennessee Valley region through the year 2000.

- o EPA's new Municipal Compliance Policy is placing new demands on the States. It specifically calls for enforcement actions against municipalities not complying with appropriate provisions of the Clean Water Act whether Federal wastewater treatment funds are available or not (Wyatt, 1986).

RESOURCE MANAGEMENT CHALLENGES

As clean water related issues gain increased public attention, water resource management agencies should expect questions on how they plan to address and/or accommodate these issues while meeting the project uses authorized by Congress. Recent initiatives supporting this observation include:

- o In Tennessee, State water quality and natural resource agencies have indicated an increased concern about water quality and instream flows downstream of all dams in the State. The Tennessee Division of Water Pollution Control has proposed the adoption of language in their water quality standards regulations specifying that the State's use designations and DO criteria, i.e., 5.0 mg/l for warmwater fisheries and 6.0 mg/l for coldwater fisheries, are applicable to all tailwaters " . . . up to the toe of the dam." The Tennessee Water Quality Control Board and the Tennessee Wildlife Resources Commission have also adopted resolu..ons urging TVA to move beyond its experimental releases improvement program and mitigate the impact of its reservoir releases.

- o EPA has developed a draft guidance document clarifying the need to control nonpoint sources of pollutants through nonpoint source controls, best management practices (BMP's), and how BMP's should be implemented to meet water quality standards.

- o Pending hydropower licensing, omnibus water resources, and clean water bills in the House and Senate indicate that increased emphasis on environmental and energy conservation requirements is likely to be placed on new as well as existing hydropower projects. The most environmentally significant legislation involves hydropower relicensing in which Congressmen Markey, Dingell, and others are recommending changes to the Federal Power Act that require the Federal Energy Regulatory Commission, in deciding whether to issue a license for any project, to give equal consideration to energy conservation, fish and wildlife amenities (including related spawning grounds and habitat), recreational opportunities, and the preservation of environmental quality to the power and development purposes.

TVA'S RESERVOIR RELEASES SITUATION

In August 1978, an interoffice task force prepared a special executive report for the TVA Board of Directors on the impact of TVA's projects on downstream uses and water quality (Anonymous TVA, 1978). One of the most significant findings of the report was that releases from TVA's dams were adversely impacting approximately 340 miles of stream. The parameter of greatest concern and the one most often failing to meet State and Federal water quality criteria was dissolved oxygen (DO). Unacceptably low concentrations of DO in combination with operations that leave stream beds dry for extended periods of time had virtually eliminated the tailwater resource and any development potential it offered.

The number of days that releases from TVA's dams failed to meet protective criteria varied from a minimum of three days at Upper Bear Creek to a maximum of 183 days per year at Tims Ford. The most serious problems were associated with the east Tennessee tributary projects. These projects are deep, stratified impoundments with long retention times and highly variable releases. They are also located in moderately to highly developed watersheds.

TVA's efforts to mitigate these problems has primarily involved a 2-prong technological and resource management approach, i.e., mitigating technologies aimed at improving DO concentrations at the dam and efforts to control pollution in the reservoir watershed. The technological work has involved a major experimental program to identify inexpensive, efficient aeration techniques that can be used at TVA dams. The resource management effort addresses the reduction of upstream pollution.

This has resulted in the development of reservoir water quality management plans. One of the major components of these plans involves modeling the relationships between reservoir inputs (point and nonpoint source contributions to reservoirs) and reservoir water quality. Although point and nonpoint sources of pollutants are not the only cause of hypolimnetic DO depletion, control of these pollutant loads should improve reservoir DO concentrations.

In addition to the technological and resource management activities described, TVA has undertaken institutional initiatives to address issues related to the DO problem. One of the most promising endeavors is a Memorandum of Understanding between the State of Tennessee and TVA which recognizes TVA, "as the area management agency with respect to nonpoint sources of pollution originating from properties in TVA custody or under TVA control which are situated in the State of Tennessee." As the designated management agency, TVA has agreed to work with the State and other authorized agencies to achieve implementation of the Tennessee Water Quality Management Plan. Efforts being pursued under this agreement include:

- Incorporating waste treatment management plans into all TVA land use plans.
- Incorporating requirements for nonpoint source control measures into all future licenses, leases, and easements for property in TVA custody or control including any renewals of existing licenses, leases, and easements.
- Incorporating requirements for nonpoint source control measures into approvals given under Section 26a of the TVA Act.

TVA has also taken an increasely active role in the review of State water quality standards. This year, TVA has urged the State of Tennessee to control more effectively the pollutants that are causing significant cultural eutrophication and sediment buildup problems in Tennessee's streams and reservoirs. Water bodies with severe use impairments should be identified as "Nutrient Sensitive Waters" or "Erosion Sensitive Watershed." Following this designation, TVA recommended implementation of watershed pollutant control programs that emphasize the control or treatment of point and nonpoint source contributions at their source (Rivers, 1986).

POLICY DISCUSSION

The issues raised strongly suggest a need to reexamine resource management and pollution control strategies, in particular those related to water quality and quantity programs. Better integration of these programs is not only an important policy issue in the Tennessee Valley, but nationally as originally noted by the Water Resource Council's (WRC) in their "Second National Water Assessment," nearly 10 years ago. In the policy section of that report, WRC stated:

> Perhaps the most significant deficiency in the past management of water resources was the failure to adequately consider water quality in many areas. Accordingly, it has been necessary to make major investments in an effort to bring water quality up to an acceptable level. Aggressive programs directed at water quality management should have been initiated before major problems arose as a result of industrial and agricultural development and urbanization. Even now, as an all out attempt to alleviate water pollution problems proceeds, the integration of water quality planning and management with other aspects of water resources development tends to be overlooked. Without integration, the water quality goals will be more difficult, and more costly to meet. The Second National Water Assessment did not address the issue of flow requirements for water quality management (Anonymous WRC, 1978).

The country has taken steps to factor reservoir releases into the design of new projects. For example, in the planning of new projects, Federal agencies now routinely factor in needs to maintain downstream assimilative capacity, to protect fisheries, and to accommodate downstream recreation. There has been less success in resolving these issues at existing projects. Efforts to address these issues, i.e., at existing projects, under the national water quality standards program, the sections 402 and 404 permitting activities, and the section 208 water quality management program of the Clean Water Act have not been particularly effective. Instead it seems that litigation or the threat of litigation over hydropower relicensing has been the principle vehicle for resolving these issues.

Whether litigation or some other option is the appropriate vehicle for addressing the reservoir releases situation at existing projects, only a future historian can say with any certainty. At TVA we are addressing it as a technological issue requiring the combined resources and expertise of both resource management and regulatory agencies. In that regard, we have identified the following policy issues that are the subject of an ongoing policy debate within TVA:

- Are the downstream improvements in DO and flows being undertaken to restore beneficial uses or to develop the tailwater resource?
- Should the cost of improvements be paid for by the ratepayer in accordance with the polluter-pays principle or the taxpayer through Congressional appropriations in support of the uses prescribed in the original project authorization?
- What water quality objective or what level of DO improvement should TVA strive for?

How ever these internal TVA issues are resolved, the following policy issues deserve the attention of everyone interested in water resources. Given the need to better integrate water quality and water resource programs, if water resource projects are achieving State approved minimum flows and water quality objectives:

- o Should construction grants decisions for funding waste treatment projects beyond secondary treatment be coordinated with water resource agencies?
- o Should EPA and States modify their policies making additional flows and increased DO levels eligible for funding under the construction grants or revolving loan programs? (Note - This change should be consistent with section 102(b) of the Clean Water Act, i.e., water resource "shall not be provided as a substitute for adequate treatment or other methods for controlling waste at the source.")
- o Should flow related permits be encouraged?

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TVA RESERVOIR RELEASE IMPROVEMENTS: AN OVERVIEW

Bevan W. Brown

Director, Division of Air and Water Resources, Tennessee Valley Authority, Knoxville, TN

ABSTRACT

A brief overview of TVA efforts to improve the quality of reservoir releases from major dams in the Tennessee Valley. The program involves improvement of dissolved oxygen levels in reservoir releases and steps to provide a minimum stream flow below TVA dams. These and other efforts are, however, not to be understood in isolation but as part of a total concern for water quality in the region. Without a commitment to total water quality, such technological fixes would simply be stopgap measures, doomed to be overwhelmed eventually by the pressures of industrial expansion and population growth.

INTRODUCTION

TVA and others around the country have been aware for many years that our dams were creating water quality problems. Two factors are involved, of course. One is the natural tend;ncy of deep bodies of water to stratify during the summer. The other is the design of power components which use the deeper waters of a reservoir for hydroelectric generation. These deeper waters tend to be low in oxygen through much of the summer and early fall.

A number of devices have been examined through the years to try to correct the problem. Several years ago TVA looked into ways of introducing oxygen into the water near the turbine intakes. But, there was no sense of urgency on anybody's part to deal with low dissolved oxygen in reservoir releases. The situation was there. We knew it was there. And we knew, in the back of our minds, that someday it probably would have to be dealt with. We were, however, somewhat like Scarlet O'Hara in "Gone With The Wind"--it was something we would think about tomorrow.

THE CURRENT TVA PROGRAM

It was not until the start of the 1980s, or thereabouts, that we at TVA really got serious about dealing with oxygen deficiency in reservoir releases. We began experimenting with some technology that had been tried in Alabama, and we thought we had found a way to make it work without costing our power system a fortune.

Improving the oxygen content of reservoir releases is the first part of a two-pronged effort TVA has been pursuing for about seven years. You'll hear more about the technical aspects of this program from others in TVA.

I'll just give a brief outline of where we stand right now. But mainly I want to consider some of the reasons why we believe water quality issues must be among our top priorities in water resource management.

At Norris Dam, we are working with hub baffles to increase air injection into water turning hydroelectric turbines. The baffles installed at existing aeration vents on the turbine hub are able to increase oxygen content of the water by 2.5 to 3 milligrams per liter. The baffles are bolted onto the turbine hub in the spring and removed in the fall of the year when they are no longer needed.

At Tims Ford Dam in middle Tennessee, we are using a large air compressor to force air into the water flow. The effect is about the same as we are getting with hub baffles at Norris.

At South Holston Dam in upper east Tennessee, we found that a simple modification of the existing aeration system gave pretty good results. Oxygen levels are increased around 2 milligrams per liter.

And at Douglas Dam we have begun a study of Garton pumps to overcome the problem of low dissolved oxygen in turbine releases into the French Broad River. These pumps look something like airplane propellers. They are suspended ten feet deep beneath a floating platform, and are protected by a heavy wire cage. The idea is to force surface water deep enough into the reservoir near the turbine intakes so that the well oxygenated water from the surface flows through the powerhouse and into the stream below the dam.

Our work has convinced us that there is no simple, off-the-shelf method of improving oxygen content of reservoir releases. We have found that hub baffles work pretty well one place but not at all on another, similar generating unit somewhere else. Air compressors can do the job at some installations but are unsuited for other facilities. Each installation has its own characteristics and requires an individual solution. It may be that oxygen injection may be the best solution at some facilities. We have not given up on that procedure.

In fact, we are exploring direct oxygen injection at Cherokee Reservoir as part of an attempt to create fish sanctuaries for striped bass. These fish suffer considerable stress during the summer unless they can find an area of cold water with adequate dissolved oxygen.

It is also becoming clear that what we are doing now may be only a transient phase in dealing with oxygen depletion in reservoir releases. The ultimate solution probably lies with the design of the replacement units in powerhouses all across the Nation. We have studied a number of TVA installations in recent years, looking at the older units and at the feasibility of replacing them with newer designs. There is no doubt that many older TVA hydroelectric units can be replaced with units that will not only produce greater amounts of electric power with the same amount of water but that aeration can be designed into the new equipment.

The second part of our efforts to improve water quality in streams below TVA dams involves sustainable minimum flows. Below Norris Dam we have constructed a small reregulating weir that is designed to ensure a minimum flow of 200 cubic feet per second. This is accomplished pretty much through normal operation of the hydroelectric facilities. By impounding water behind a rock dam during power generation and releasing it in a controlled manner when the turbines are not operating, we can provide a reliable minimum flow in the Clinch River. There are times when no hydroelectric generation is scheduled for an extended period that we have to pulse a turbine for an hour twice a day to supply the needed water.

At Tims Ford Dam, we have installed a small hydro unit that generates when the big power unit is not operating. It releases 80 cubic feet of water per second to the stream below the dam.

And below Tellico Dam, we are providing 8 cubic feet per second of cold, well aerated air to an arm of Watts Bar Reservoir to create a sanctuary for striped bass. A siphon has been constructed to carry the water over the dam and into the old river bed below Tellico. To hold the cold water, an underwater barrier has been built near where the Little Tennessee and the Tennessee River join below Fort Loudoun Dam. This sanctuary appears to have been used by considerable numbers of stripers and other fish this past summer.

OUTCOME AND OUTLOOK

Obviously, improving oxygen content of water below TVA dams and guaranteeing a minimum flow in the stream provide fishery benefits. Fish and the aquatic organisms on which they feed need oxygen and minimum amounts of flowing water in order to thrive. Our experience below Norris Dam indicates that some cold water fisheries in the Tennessee Valley can be greatly improved through a combination of oxygen improvement and provisions for a minimum flow. Not only do the fish have better quality water in which to live but insects and other aquatic life important to fish thrive better under such conditions.

This is especially important in tributary streams. On the Tennessee River from Knoxville to Paducah, Kentucky, maintenance of navigation depths ensures that there will always be considerable water below mainstream dams. But in some tributary streams, there are times when the flow is little more than a trickle. Full development of any kind of fishery is impossible under such conditions.

There is a broader range of considerations than fishery development, however, behind our efforts at reservoir release improvement. Just about everyone in the Tennessee Valley region -- all 7 million people -- have a stake in water quality improvement and protection in one sense or another. The economic well-being of the region is wrapped up in the health of its water resources. An abundance of water has long been a major selling point for the Tennessee Valley when industrial recruiters go looking for prospects.

But abundance is often a relative matter. That is one of the things we are learning from two years of drought in the Tennessee Valley. We still have a lot of water in the region, but that water is now under considerable stress. Fish and mussels are dying in some places where we had no idea that severe water quality problems existed.

Some of the problems may come from sources of pollution that are poorly understood or undocumented. Some of the problems may arise in the reduced capacity of the water to assimilate natural and manmade inputs of one kind or another. And this probably results from a combination of low dissolved oxygen and higher than usual water temperatures.

Some things, of course, we can do little about under the conditions that govern our operations. But fundamental reservoir release improvements are within our capacity. And because one reservoir generally feeds into another in a downstream pattern, improvements at certain key dams can make significant improvements in water quality in one, two, perhaps three or more reservoirs downstream.

THE BALANCED PICTURE

More than 50 years of TVA experience have confirmed many times over that quality natural resources are essential to the economic well-being of a region. When the agency went to work in the 1930s, we immediately put people to work restoring lands that had been abused for centuries. This restoration had a bonus effect of improving water quality in the Tennessee Valley.

Now we are hard at work on a number of water quality issues, some far older than TVA, and some as new as the latest consumer product or industrial process. Reservoir release improvements are an important part of the overall water quality effort. But let me make it clear, they do not stand alone. Unless we guard water quality from all other threats, any improvement we make in dissolved oxygen and more reliable stream flow will only be stopgap. Stresses of industrial change and population growth will soon overwhelm whatever we do unless we can see the whole water quality picture.

STATUS OF THE WATER QUALITY CONTROL MISSION IN ORD AND NEEDS FOR RESEARCH SUPPORT

MARK ANTHONY, PhD

Chief, Water Quality Section
Reservoir Control Center
U.S. Army Engineer Division, Ohio River

Water quality control is a significant, integral aspect of our water control mission in the Ohio River Division. The location and resources of this basin resulted in early extensive development and subsequent major problems with polluted streams. The Corps of Engineers was involved in two major studies of pollution in this basin (1920's and 1930's) prior to any reservoir construction in the basin. While flood control was the primary focus of the 1938 Flood Control Act which authorized construction of a large reservoir system in this basin, water pollution control was also cited as a major benefit. Subsequent authorization of specific projects have established water quality control storage as well as storage for low-flow augmentation. A major improvement in water quality control resulted from incorporating selective withdrawal capability at storage projects constructed during the last 20 years. This feature allows us to more closely comply with stream standards than at projects with only a capability for bottom releases.

While our districts were collecting at least some water quality data for reservoir regulation guidance during the 1960's, an intensive effort was initiated in 1967. A long-range plan was carefully developed along with specific objectives and priorities and an organized data collection effort began. The basic strategy was to acquire an adequate data base at all projects, employ interpretative techniques abetted by an understanding of hydrodynamics and hydrometeorological conditions in order to anticipate and respond to the quality of storage when scheduling reservoir releases. The success of this strategy along with very careful coordination with state agencies has allowed the Ohio River Division (ORD) to avoid controversy, litigation and crash mitigation efforts.

We, at the present time, are attempting to further refine all of the procedures associated with water quality control. We now have access to a multitude of more precise and eloquent procedures developed through the Environmental Water Quality Operations Study Research Programs, many of which have already been integrated into our program. However, further integration or utilization of this improved technology is proving difficult because of the limited size of district staffs and the level of difficulty posed by the complexity and sophistication of some of the latest computerized analytical techniques. Staffing constraints force districts to rely more and more on direct support by research personnel for initial application of water management procedures. Thus, there is a dilemma: districts require the services of a skilled and also staff-limited research staff whose primary mission is to continue research toward identified and justifiable objectives and needs.

The location and resources of the Ohio River Basin promoted early and rapid settlement and the Ohio River provided relatively easy transportation to the west. By 1800, Pittsburgh was a thriving center of industry including iron and steel, boat building, coal, lumber, and glass. Thousands of flatboats were built to transport settlers and materials on one way journeys as far as New Orleans. In the early 1800's, bulk coal shipments by flat boat were routine. The largest of these boats was 175 feet long, 26 feet wide, and drew 8 feet when loaded with 24,000 bushels of coal. They were floated downstream only on high water stages. In 1847, the peak year, more than 2,200 flat boats from the Ohio Valley landed in New Orleans.

The first Ohio River steamboat was built at Pittsburgh in 1811 and the first Ohio River towboat taking coal to New Orleans delivered 2,280 tons in four barges in 1854. Steamboats provided an enormous impetus to Ohio Valley industrial development and accelerated commercial growth. The success of this means of transportation led to the first Inland Waterways Improvement Act in 1824 which directed that experiments be conducted to determine the best method of coping with sandbars in the Ohio River and that measures be taken to remove snags obstructing navigation on the Ohio and Mississippi Rivers. The Army Corps of Engineers was assigned responsibility for implementing this Act.

In 1824 the Ohio River was much obstructed throughout its length by snags, rocks, and sand and gravel bars. The width of channel varied exceedingly and the low-flow depth varied from a minimum of one-foot above Cincinnati to two feet below Cincinnati. When the depth over the worst shoals was three feet or more, the river was navigable throughout its entire length for steamboats, flat boats and keel boats.

Improvements initiated in 1825 consisted of removal of rocks and snags, placement of wing dikes and back channel dams. This approach was continued and expanded until construction of a system of 50 locks and dams with a nine-foot navigation channel completed in 1929. By 1950, this system had become obsolete because of capacity and maintenance costs. Beginning in 1954, 13 new modern navigation structures were built to replace 39 of the old structures. The present navigation system consists of 20 locks and dams on the Ohio main stem. Tributary navigation projects include the Allegheny River (eight structures), Monongahela River (nine structures), Kanawha River (four structures), Kentucky River (14 structures), Green River (four structures), Cumberland River (four structures), and Tennessee River (10 structures).

As settlement proceeded, Ohio Basin floods became a major problem and a controversial issue. A proposal by Charles Ellet, Jr. in 1849 outlined improvement of the Mississippi and Ohio Rivers by means of storage reservoirs. He maintained that reservoir sites should first be established on the Allegheny, Monongahela and other Ohio River tributaries. His plan suggested that such reservoirs would control flooding, and low flow navigation and produce power. The opposing position (including the Corps of Engineers') considered levee systems to be a much more feasible approach. A series of devastating flood events led to the 1936 Flood Control Act and responsibility on the part of the Corps for planning, constructing and operating flood control reservoirs.

Ohio River water quality degradation had become a serious problem by the early 1900's. A report published in 1912 by the Pittsburgh Flood Commission contained the results of surveys, investigations and studies concerning the causes of damages by and methods of relief from floods in the Allegheny, Monongahela and Ohio Rivers. This report recommended building a system of storage reservoirs to control flooding and provide benefits to navigation, sanitation, water supply and hydropower. Human sewage alone was overpowering during low flows. Industry, mining and deforestation also contributed major problems. Exposed metal (boats, lock gates, boilers, etc.) had an extremely short life. For example, three-eighths inch steel plates were reduced to a knife edge in a year's time. Increasing pollution and typhoid epidemics soon led to additional studies. The Corps was involved in two major Ohio River pollution studies during the 1920's and 1930's. These studies led to the establishment of the Ohio River Valley Sanitation Commission in 1946 and to the addition of water pollution control as a major benefit for the Ohio River Reservoir System authorized by the 1938 Flood Control Act.

The Corps currently operates 76 storage reservoirs in the Ohio River Basin. These projects are located on 21 tributaries. The primary authorized purpose is flood control at all projects. Other authorized purposes include recreation at 74 projects, water quality at 39 projects, fish and wildlife at 35 projects, water supply at 16 projects, hydropower at six projects and navigation at 1 project. These projects are located on 21 tributaries. Thirty-six projects have selective withdrawal capability, 64 projects have minimal summer releases, 18 projects have downstream temperature objectives. Reservoir systems operation is critical during both low and high flows in the basin and is highly controversial during drought events when requirements to initiate early drawdown conflict with desires for stable recreation pools. The original plan of operation for this system assumed that main stem Ohio and lower Mississippi low flow needs, including navigation, would be met with system releases from conservation storage. Subsequent authorizations that established recreation as a project purpose have decreased the flexibility that was established by the original plan.

By the mid-1960's the Corps was operating more than 45 reservoirs in the Ohio Basin and was immersed with water pollution issues. Public sensitivity concerning pollution was rapidly increasing in proportion to post-war affluence and increased leisure time. Several of the existing reservoirs and many under construction or being planned were impacted by coal mining activities. Hundreds of miles of tributary streams were biological deserts and the Ohio River was heavily polluted. The navigation modernization program, replacement of many low head locks and dams with a few highlift structures, had adversely impacted reaeration and made dissolved oxygen conditions much worse. Efforts were being initiated to draft legislation to establish water quality standards and fund extensive new treatment facilities.

A Water Quality Section was formally established in the ORD Water Control Branch in 1967 and shortly thereafter in the district Hydrology and Hydraulics Branches. The primary mission was to advise reservoirs regulation elements concerning water quality issues. At that time, there was no published guidance regarding procedures for man-made lakes. A long range plan and specific objectives were developed. Objectives were to acquire adequate knowledge and understanding so as to improve and maintain the water quality in reservoirs to enhance fish and wildlife and recreation uses, control the quality of releases to protect downstream fisheries and water supply, maintain the best possible quality conditions in the lower tributaries and main stem Ohio and to mitigate the low-flow impacts of the highlift navigation structures.

Based on the experience of a multi-disciplinary staff and data collected previously by the districts and other agencies, a multi-faceted approach was implemented. Limnological procedures complimented with knowledge of hydrodynamics and hydrometeoro-logical expertise provided a protocol for evaluating existing reservoirs and performing pre-impoundment studies. Reservoir temperature models helped with interpretation of data and understanding of existing reservoir conditions, as well as prediction of conditions in future projects. These models along with physical modeling by the Waterways Experiment Station (WES) improved the design of selective withdrawal structures. While we had achieved a reasonable degree of success in understanding and control of problems by the mid-1970's, the after shocks of the Dredge Material Research Program and spinoff from the Environmental and Water Quality Operational Studies Program (EWQOS) enhanced the efficiency and effectiveness of our procedures.

Ohio River Basin water quality conditions are significantly improved today. A quality fishery now exists throughout most of the Ohio River from Pittsburgh to Cairo and hundreds of miles of tributary streams have recovered. While several of our reservoirs are borderline at times because of watershed problems, most support at least a reasonable fishery and releases comply reasonably with water quality standards most of the time. Certainly much credit for this improvement is due to the massive outlays for improved wastewater treatment and enforcement of effluent standards. Nevertheless, intentional and inadvertent operations on our part occasionally kill fish in streams that were barren 10 years ago and changes in systems operation could easily induce extensive degradation because of accumulative impacts as flow passes from pool to pool.

Conditions are still far from pristine in the basin and political and economic changes can easily reverse what has been accomplished. State and local governments are willing to allow some degradation as a part of the competition to attract industry. The acid rain issue threatens costly changes for industry. Enforcement of coal mining regulations has been relayed but, fortunately, reduced coal production and recent closures of steel mills have significantly enhanced water quality in several stream reaches. The treatment of abandoned mine wastes has been reduced and sewage treatment is still short of optimal. The new secondary treatment facility at Cincinnati, at best, provides primary treatment because of design defects. Consumption of fish caught in parts of the Ohio River should be limited because of concentrations of toxic materials in the flesh.

Our strategy for day-to-day guidance of reservoir operation to control water quality has been relatively simple in concept. Collect enough of a data base over a long enough period of years so that the response of each lake (water chemistry, hydrodynamics, etc.) to any hydrometeorological event is clearly understood. Subsequently, data collection would be substantially reduced to a level that verifies that current conditions correspond to those anticipated in relation to hydrometeorological events. We had assumed that computerized procedures, remote sensing techniques, and limited data collection would support control efforts.

In practice, this strategy is only partly successful. Our data collection efforts have been reduced substantially and the actual needs for data and priorities regarding specific problem areas have guided this reduction. However, manpower limitations have become a major issue. Our districts have never acquired a desirable skill level regarding use of computerized procedures for water quality (or for water control). While we have successfully utilized modeling and made reasonable headway in developing data management and interpretive procedures, recent manpower cuts have had a very negative impact.

We cannot complete an objective of modifying and integrating the multitude of useful computerized procedures developed by EWQOS. We feel that we need these procedures in order to fully utilize an extensive data base and to effectively manage a program that faces increasing pressures from conflicting water user groups. For example, add-on hydropower at storage and navigation projects will cause at least some degradation and may result in significant accumulative impacts unless we upgrade water control and water quality control procedures.

There is little possibility that ORD will be able to fully utilize the extensive list of EWQOS software products regardless of the advantages and needs unless staff numbers and skills are is increased. We must rely more and more on direct support by research personnel or consultants for application and other support. Even this alternative is constrained by both research laboratory and district resource limitations. The effective use of contractors also requires a high level of technical skill and time in order to communicate objectives to the contractor and evaluate the return product.

Another problem that concerns us is the degree of sophistication and especially the data required by some of the models already developed or proposed for further research. Even if district manpower were at optimal levels, any model that will be used for operating reservoirs and reservoir systems for water quality purposes must be as simplistic as possible in regard to data requirements and computer time.

The development of models for day-to-day regulation of reservoirs and reservoir systems requires in-depth knowledge of district regulation procedures and of projects to be regulated. The orientation required far exceeds the historical data or the period of record and regulation manuals. The verification and calibration of the greatest model in the world is just the beginning of the tedious, frustrating and perhaps lengthy effort required if it is to become a useful, reliable tool for real-time regulation of reservoirs.

The reservoir system and navigation improvements have had a major impact in terms of the history of the Ohio River Basin. Since most of the feasible storage projects have been constructed, the future needs must be met by fine tuning water management procedures and changes in water use must be accommodated by project and operational modifications. The addition of hydropower is one such change that will stress the operating flexibility of the system. The operational demands of the system already suggest that 24-hour/7-day water control staffing is necessary. We have only partially met expanding needs for operating models for reservoirs and tributary systems including water quality.

We will require increasing support from research elements for more than routine technology transfer. We need support in terms of applying water management models and implementing available software for precise identification of data needs. We need additional research, including simplified operational models and use of remote sensing, to acquire water quality data to support reservoir regulation. We need instruments that are compatible with data collection platforms and the Geostationary Operational Environmental Satellite (GOES) that are relatively maintenance free. We need guidance regarding reaeration in relation to hydropower development at storage reservoirs and navigation structures. We need to improve selective withdrawal capability for hydropower releases. These needs obviously exceed the resources of the research laboratory elements and, unless agency priorities are changed, water management in the Ohio River Basin is likely to experience unnecessary controversy.

WESTEX REVISITED PREDICTED VERSUS
OBSERVED RESERVOIR TURBIDITY

D W Larson, Ph.D.[1] and J D Graham[2]

[1] Limnologist, Hydraulics and Hydrology Branch, Portland District, U S A E , Portland, Oregon.

[2] Hydrologist, Hydraulics and Hydrology Branch, Portland District, U S A E., Portland, Oregon.

ABSTRACT

Reservoir turbidity was simulated with a mathematical model called WESTEX during preimpoundment water quality studies of the Rogue River Basin in southwestern Oregon. The main objective of this simulation was to predict the turbidity of reservoir release flows for a variety of operating schemes and watershed hydrological conditions. A comparison between observed data after the project became operational with simulated data shows that the WESTEX predictions had been very high and, hence, that the reservoir's turbidity potential had been greatly overestimated. WESTEX's unreliability raises questions about the value of turbidity assessments which depend solely on mathematical modeling. This paper offers suggestions as to how studies to determine turbidity potential might be made more reliable and effective.

INTRODUCTION

WESTEX is a reservoir temperature and turbidity simulation model that was developed for Portland District, Corps of Engineers in the early 1970's (Fontane et al. 1973) This model was used during preimpoundment water quality studies to estimate the turbidity potential of three large multipurpose dams in the Rogue River Basin, Oregon (U S. Army Corps of Engineers, 1974).

WESTEX's development and use, costing over one million dollars, was one of several commitments made by Portland District to ensure protection of the Rogue River from the possible disruptive effects of dams. Another commitment was the installation of reservoir intake towers with multi-level ports for selective withdrawal capability (Figure 1) to avoid the discharge of poor quality water or, alternately, to discharge reservoir water capable of enhancing water quality downstream.

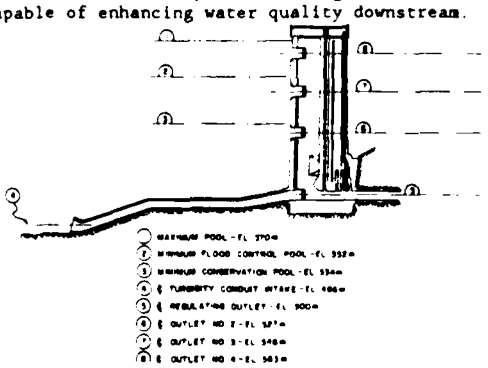


Figure 1. Cross-section view of intake tower at Lost Creek Lake, Oregon.

At Lost Creek Reservoir, selective withdrawal capability was increased substantially by the addition of a turbidity conduit (Figure 1), facetiously called an "elephant trunk," for rapid, downstream disposal of turbid inflows whenever river turbidities throughout the Rogue River were naturally high because of frequent winter storms and high streamflow conditions (Larson, 1982). These commitments were certainly appropriate and necessary, not only for safeguarding the Rogue River's famous and much-valued fish runs (Larson, 1984), but because most of the lower Rogue River had been officially designated as "wild and scenic" and, therefore, would require careful operation at upstream dams to assure that its essential attributes would be preserved.

A major objective of the WESTEX study, hence, was to predict release-flow turbidities for several combinations of yearly runoff conditions and selective withdrawal operations. In this paper we compare WESTEX predictions with 4 years of observed (postimpoundment) reservoir and release-flow turbidity data, collected routinely at Lost Creek Reservoir, to assess the reliability and usefulness of the WESTEX model. We also report on the selective withdrawal manipulations at Lost Creek Reservoir to minimize the problem of turbidity downstream and in the impoundment. Finally, we discuss why WESTEX performed so poorly and suggest ways to improve the model's capacity to predict water quality impacts.

WESTEX ADAPTED TO TURBIDITY PREDICTIONS

WESTEX was originally designed to simulate thermal and chemical stratification in a reservoir, and to predict temperatures and dissolved-solids concentrations for reservoir-release flows (Clay and Fruh, 1970). The model's execution of three of its mathematical components-- (1) vertical placement of reservoir inflows, (2) the extent of vertical mixing, and (3) the depth of reservoir withdrawal-- relies almost solely upon reservoir density stratification, either physically or chemically induced. For example, the model directs reservoir inflows into that reservoir stratum whose density equals the density of the inflow The model then vertically mixes the reservoir, to whatever extent necessary, to attain a stable density gradient. This stability must be obtained before WESTEX can calculate the depth of reservoir withdrawal.

Fontane et al (1973) justified the use of WESTEX as a reservoir turbidity model on the underlying assumption that suspended sediment, which causes turbidity, can be budgeted for and routed through a reservoir system, similar to the budgeting of heat and dissolved solids for other water quality predictions. The model's output, predicted concentrations of suspended matter in the reservoir and in outflows, is converted to turbidity predictions by linear regression, establishing a "correlation" between turbidity and suspended matter (U.S. Army Corps of Engineers, 1974).

WESTEX CALIBRATION AND VERIFICATION

WESTEX was calibrated with actual turbidity data collected by Klingeman et al. (1971) at Hills Creek Reservoir, Oregon. Hills Creek had been chosen for WESTEX verification simply because it was the only reservoir among Portland District's 16 reservoir projects with turbidity data. Unfortunately, Hills Creek was unique in another respect. The reservoir contained a deepwater layer of relatively turbid water (Figure 2), which caused considerable turbidity downstream whenever the deeper water was released from the project.

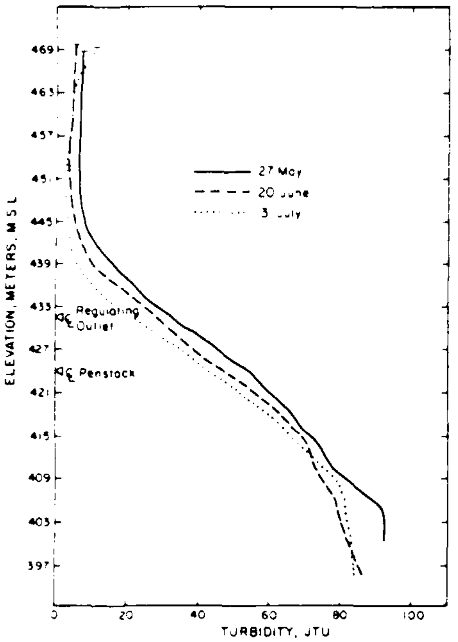


Figure 2. Vertical turbidity profiles for Hills Creek Reservoir, Oregon, 1971 (Klingeman et al. 1971).

Deepwater turbidity had persisted for years in Hills Creek owing to the reservoir's poor flushing ability and the predominance of extremely fine-particle, hence slow settling, montmorillonite clay in suspended loads entering the reservoir (Klingeman et al. 1971; Larson, 1979). Interestingly, it was the Hills Creek turbidity situation which had prompted Portland District to undertake the preimpoundment turbidity assessment for Lost Creek Reservoir, using the WESTEX turbidity model.

Several assumptions were required before the Hills Creek Reservoir turbidity structure could be simulated with WESTEX (Fontane et al. 1973; U.S. Army Corps of Engineers, 1974). First, the reservoir's initial thermal and turbidity profiles were assumed to be uniformly 4.4° and 5 NTU, respectively. Second, it was assumed, because of the characteristically slow settling of Hills Creek clays, that soils and other particulate matter in the reservoir's water column would be held in suspension, i.e., have a zero settling rate. Third, stormflows greater than 2000 cfs were mathematically plunged to the reservoir bottom, followed by a 50 percent reduction in reservoir turbidity. Fourth, coefficients for heat diffusion and light extinction were assumed to be 0.1 and 0.05, respectively; other parameters, such as heat absorption and heat entrainment were also assumed.

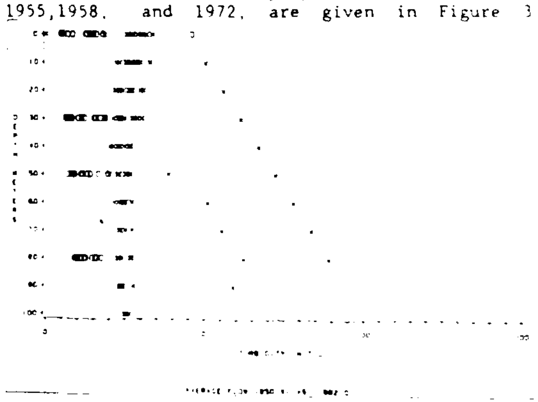
Fontane et al. (1973), after a series of computer trials, best-fit WESTEX to the Hills Creek Reservoir prototype. They stated that they "obtained reasonable agreement between the predicted and observed turbidity profiles." The verified WESTEX turbidity model, calibrated to fit the extreme turbidity gradient in Hills Creek (Figure 2), was now ready for application at Lost Creek Reservoir.

LOST CREEK RESERVOIR TURBIDITY PREDICTIONS

Three hydrologically different years were selected for WESTEX turbidity simulations: 1950 (average runoff), 1955 (low runoff), and 1958 (high runoff). Additionally, a spring freshet in March 1972 was modeled to estimate the effect of highly turbid inflows on the reservoir's turbidity structure during reservoir refill (U.S. Army Corps of Engineers, 1974). A second regression analysis, flow versus turbidity, was used to compute reservoir inflow turbidities for the particular years of study. This "correlation," perhaps as tenuous as the relationship between turbidity and suspended solids (APHA, 1985), was based on approximately 35 turbidity and flow measurements recorded in 1971-72 (U.S. Army Corps of Engineers, 1974).

Turbidity simulations were also made under three project operational schemes, designated as plans A, B, and C, in combination with various assumed or actual watershed hydrological conditions (U.S. Army Corps of Engineers, 1974). In Plan A, all dam outlets were utilized year-round for the sole purpose of meeting water temperature requirements downstream. In plans B and C, the same temperature objectives were desired, but there was also an attempt to meet turbidity objectives by using only the dam's low-elevation outlet, the "elephant trunk," during the winter when flows, and thus turbidities, in the Rogue River Basin are highest. Plans B and C differ from one another in scheduled use of the low-elevation outlet. Under Plan B, the outlet would be used from November through February; Plan C would carry this use on through April, to pass highly turbid spring freshets. The predicted data considered in this report were generated under Plan C, which the Corps eventually recommended for Lost Creek (U.S. Army Corps of Engineers, 1974).

Reservoir turbidity predictions for 1950, 1955, 1958, and 1972, are given in Figure 3.



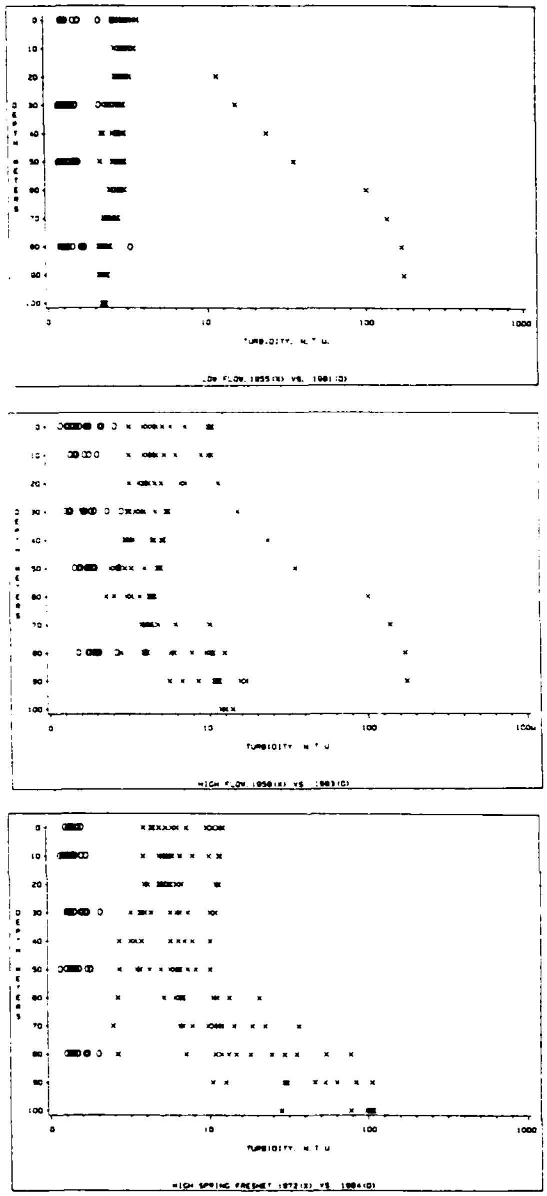


Figure 3. Observed (O) versus predicted (X) turbidity profiles in Lost Creek Reservoir, Oregon.

Predicted profiles are compared with their respective observed years, i.e., 1982, 1981, 1983, and 1984. As demonstrated, WESTEX reservoir turbidity predictions were mostly 1 or 2 orders of magnitude above observed conditions (Figure 3).

Similarly, predicted outflow turbidities were higher than values actually observed, but those differences were considerably less (Table 1) than those for reservoir turbidity; in fact, the differences in outflow turbidity may be insignificant in some cases.

Table 1. Observed versus Predicted Outflow Turbidities (NTU). Average Monthly Values, Plan C. U.S. Geological Survey Records

Month	1982/1950	1981/1955	1983/1958	1984/1972
Jan	4/4.5	1/5.0	2/5.0	5/12.5
Feb	5/4.5	1/4.0	2/8.0	4/26.5
Mar	5/4.5	2/4.0	3/9.0	5/32.5
Apr	2/4.5	2/4.0	2/7.0	4/28.0
May	2/5.5	2/4.5	1/9.0	3/13.5
Jun	1/6.3	1/5.0	2/7.5	2/9.0
Jul	0/5.8	2/4.5	2/6.5	3/8.0
Aug	1/5.0	1/4.0	3/6.5	3/8.0
Sep	1/4.5	1/4.0	2/5.0	3/6.5
Oct	1/4.5	1/4.0	2/4.0	2/6.0
Nov	1/6.5	1/3.5	2/5.0	3/13.5
Dec	2/9.5	5/25 5	6/4.5	2/7.0

Average monthly turbidity values such as those compared in Table 1, however, are probably not indicative or representative of day-to-day outflow turbidity conditions, which may vary substantially over a 30-day period (Figure 4). Thus, predicted versus observed comparisons may be uninstructive, at least for outflow conditions

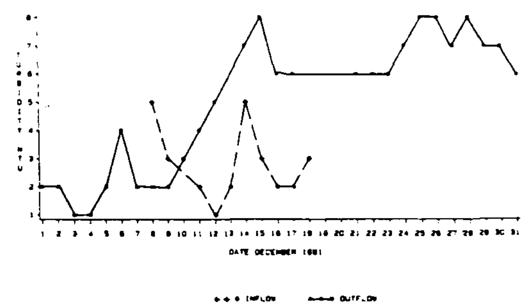


Figure 4. Inflow versus outflow turbidities, Lost Creek Reservoir, Oregon.

There are perhaps several reasons for WESTEX's generally poor predictive record at Lost Creek. Certainly, using Hills Creek turbidity data to calibrate and verify the model, especially various assumptions made to obtain correlation between observed and predicted turbidity profiles, contributed to the model's questionable performance. In addition, the model generally predicted that highly turbid stormwaters entering Lost Creek Reservoir would be retained indefinitely, when, in fact, these flows pass quickly through the impoundment and on downstream. That this quick outflow occurs is suggested by both Figure 5 and data in Table 2, which show that turbidity values for the period of record (1976-present) are nearly as high at the downstream station as they are upstream of the reservoir.

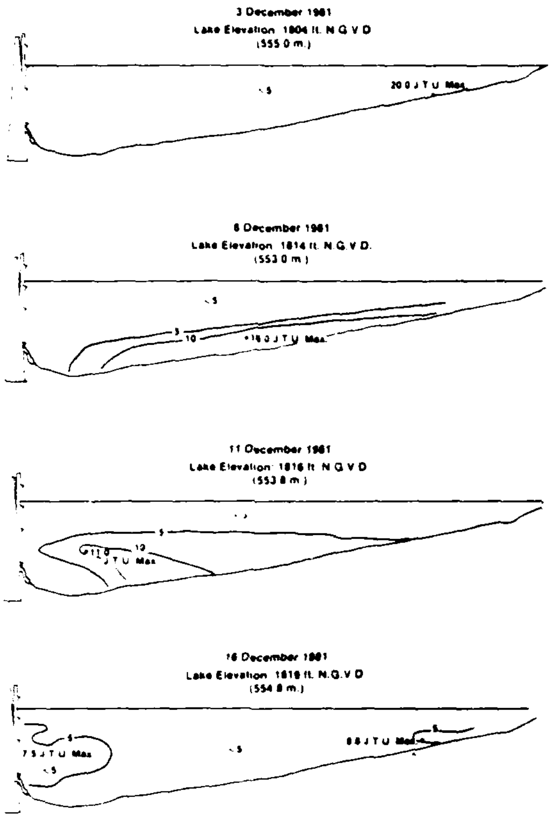


Figure 5. Turbidity-density currents in Lost Creek Reservoir, Oregon, 1981.

Table 2. Statistical Comparison by Month of Inflow and Outflow Turbidity Data From U.S. Geological Survey Records.

STATION	MON	NO OBS	NO MISS	MEAN TUR NTU	MIN TUR NTU	MAX TUR NTU
14330000 (UPSTREAM)	Jan	229	50	3	0	32
	Feb	210	32	5	0	59
	Mar	287	15	5	0	101
	Apr	284	8	3	0	21
	May	289	14	4	0	47
	Jun	247	13	4	0	85
	Jul	306	4	3	0	41
	Aug	295	8	1	0	12
	Sep	248	16	1	0	40
	Oct	270	9	1	0	22
	Nov	180	32	3	0	77
	Dec	129	65	3	0	55
14335075 (DOWNSTREAM)	Jan	314	43	4	0	190
	Feb	239	56	3	0	35
	Mar	330	27	3	0	55
	Apr	279	19	2	0	20
	May	316	25	2	0	15
	Jun	302	12	2	0	15
	Jul	365	14	1	0	3
	Aug	382	7	1	0	[illegible]
	Sep	347	27	2	0	10
	Oct	374	21	1	0	15
	Nov	316	28	2	0	80
	Dec	323	33	4	0	50

As seen in Figure 5, a plume of sediment appears in the upstream portion of the reservoir on 3 December, then moves as a concentrated mass along the bottom of the reservoir, reaching the outlet on 8 December. At the outlet, a portion of the sediment plume is discharged downstream (as shown in preceding Figure 4). The estimated velocity of this density current was approximately 0.1 fps (feet per second) during the monitoring period.

Conventional applications of analytical techniques such as Churchill (1947), or applications of reservoir hydraulics and sediment settling velocity concepts would indicate that nearly all of the fine sediment would remain in the impoundment area. In actuality, the hydrodynamics of density currents are not well known. According to a density current study on Lake Mead (Howard, 1953), those currents consist chiefly of particles smaller than 20 microns, whose settling velocity is approximately 0.001 fps. These fine sediments would require only 1 percent of the mean velocity of 0.1 fps in the vertical to stay in suspension. Thus, we should not be surprised to find that turbidity-density currents tend to move through *Lost Creek Reservoir*, and that tendency has been efficiently demonstrated.

Little information about particle-size distribution of sediments yielded to Lost Creek Reservoir is available. At the very least, all inflowing sediment of sand-size or larger is deposited within the backwater or delta areas of the reservoir (Brune, 1953, Churchill, 1948). The fate of sediment finer than sand-sized (<0.62 mm) is not so evident. Intensive monitoring of a storm event in December 1981 showed that fine sediment apparently follows the bottom of the reservoir as a density current. Figure 5 tracks a turbidity-density current from the upstream end of the reservoir to the dam axis.

RECOMMENDED ACTIVITIES FOR A RESERVOIR TURBIDITY STUDY

A primary feature of any sediment study should be a sediment budget for the watershed of study. A sediment budget can be defined as conceptualization of the type and magnitude of sediment sources and the linkage and timing of sediment transport through the watershed (Swanson and Fredriksen, 1982.) A reservoir, if its turbidity regimen is to be accurately predicted, cannot be evaluated in isolation. A thorough turbidity study must include an evaluation of sediment yields to the reservoir as well as sediment transport and deposition within the pool.

A reservoir's sediment input is found by studying its tributary watershed's hydrology, soils, geology, and erosional charecteristics. Geologic, soil, and hydrologic resource maps show what material might be yielded to a stream channel

The nature of erosional processes depends largely on land-use, rainfall intensity, and slope, such as rill and gully erosion on croplands or debris avalanches and debris torrents in steep, forested watersheds. Aerial photos are the best means to identify active areas of erosion, especially if coverage of the reservoir's watershed has been repeated.

The timing and particle size of sediment yielded to a reservoir site is identified by stream gaging and sediment sampling. The intent of these activities is to find a statistical relationship between sediment transport and water discharge, by sediment size. This statistical relationship can then readily be checked for persistence throughout a water year and, of course, over a period of several water years. Mineralogic identification of the finer materials such as clay and non-inert silts is particularly needed, so that their flocculation or dispersion when they enter the reservoir can be determined. As with any study, the longer the duration and the wider the spectrum of the flows sampled, the more conclusive the results.

The mathematical/statistical results of the sediment discharge/water discharge relationship can then be integrated with a flow duration curve to determine the estimated average annual sediment yield to the proposed reservoir (Vanoni, 1977).

The proposed land use of a watershed is an important factor of expected reservoir sedimentation. Changes in land use, such as conversion of grazing to crop lands or timber harvesting, may increase sediment yields considerably. Therefore, a reservoir sedimentation study should include analysis of anticipated as well as present sediment yields.

The processes of sediment transport and deposition in a reservoir are not completely known at the present state of art. There are uncertainties in defining effective flow and dead storage in a reservoir, density *stratification*, secondary currents, and transport of fine sediments within a pool. Sediments coarser than silts (> 0.62 mm) generally settle in the backwater and delta areas of the reservoir. These coarser sediments can be effectively and readily studied using a hydraulics-based model such as HEC-6 (U.S. Army Corps of Engineers, 1977).

The fate of fine and non-inert sediments, as mentioned, is less well defined. Obtaining estimates of retention of fine sediments in some cases may be as simple as using a nomograph, such as Churchill (1948) or Brune (1953); other studies may require complex procedures such as x-ray diffraction of sediments, sediment chemistry, and numerical modeling. If numerical modeling, such as WESTEX is used, sediment transport processes must be defined. In the previous study of Lost Creek Reservoir, in which WESTEX modeling was used, the possibilities of density currents and the effectiveness of the turbidity conduit were not taken into account. The projections derived from this study suggested that Lost Creek would release chronically turbid water. This the project has never done.

There is a danger in making environmental impact predictions: The process often becomes an end in itself. Preimpoundment turbidity predictions, in this case, are merely planning aids that will require testing in the postimpoundment period to establish their authenticity (Hecky et al 1984). As Nielson (1967) eloquently stated, "Predictions are easily made; it is accuracy in a prediction which is difficult."

ACKNOWLEDGMENT

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WATER QUALITY MODELING OF
REGULATED STREAMS

M. S. Dortch[1], P.E., J. L. Martin[2], PhD, M. J. Zimmerman[3], PhD, and D. E. Hamlin[4]

[1]Supervisory Hydraulic Engineer, USAE Waterways Experiment Station, Vicksburg, MS
[2]Civil Engineer, USAE Waterways Experiment Station, Vicksburg, MS
[3]Hydrologist, USAE Waterways Experiment Station, Vicksburg, MS
[4]Bioengineer, USAE Waterways Experiment Station, Vicksburg, MS

ABSTRACT

Environmental consequences of construction of new water resource projects or changes in operations of existing projects must be addressed. A variety of tools developed during the Environmental and Water Quality Operational Studies (EWQOS) program may be used to assess impacts of regulation on stream water quality. These techniques range from simple analytical models to more sophisticated unsteady flow water quality models. Model applications have been made to a number of systems in order to address riverine management issues. Several of these applications are discussed to provide an overview of tools available and their possible uses.

INTRODUCTION

Like our other natural resources, we find that water, too, is limited. To meet this challenge, the Corps of Engineers (CE) is seeking out every obtainable benefit from our existing water resource projects. Not only are our dams and reservoirs valuable resources, but the tailwaters below them are also becoming increasingly important. Consequently, more environmental concern is being focused on regulated stream environments such as reservoir tailwaters and waterways.

A variety of modeling tools have been developed and applied to study effects of project modifications on riverine and stream water quality. These tools range from relatively simple analytical models to more sophisticated unsteady flow water quality models. This paper provides an overview of these various tools with a brief description of applications.

ANALYTICAL MODELS

Some riverine and stream water quality issues can be addressed with analytical models that have closed solutions solvable with a hand-held calculator. Questions about the initial spread and dilution of a pollutant discharged into a receiving stream can be determined with such methods. In general, these analytical models are referred to as integral jet-plume models. Typically these models address the question of whether a pollutant has been sufficiently diluted over a reach of concern. An example is the spread and dilution of return flow from a confined dredged material disposal facility or other types of effluents (see Figure 1).

The integral jet-plume solution used depends on physical characteristics of the discharge and receiving stream. Guidance on selection and application of these models can be found in a publication by Holley and Jirka (1986). Although application of these analytical models is relatively simple, selection of an appropriate procedure can be a tedious task if one is not familiar with the various models and their assumptions. The best way to make these models readily useable by even a novice would be to develop them and their protocols into an expert system through

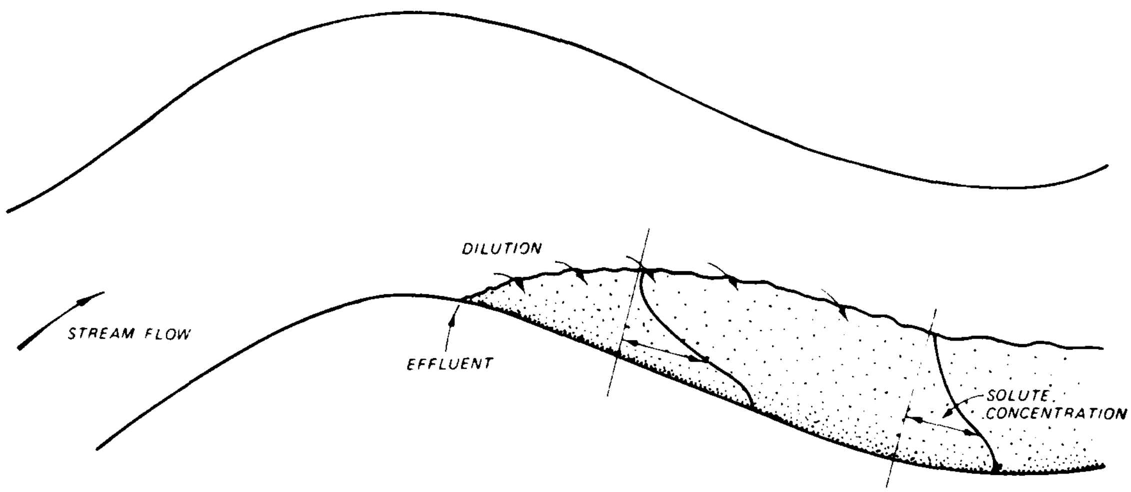

Figure 1. Two-Dimensional Spread and Concentration Distribution of an Effluent in a Stream

Artificial Intelligence. This development, which has been proposed by several researchers familiar with this field, would lead the user through the proper model selection and use.

After a discharge has fully mixed with the receiving stream, the mass transport equation is used to compute downstream transport and biochemical transformations of water quality constituents. Most stream water quality issues can and should be addressed with the one-dimensional (1-D) mass transport equation. In other words, the only significant changes in water quality occur along the longitudinal axis (stream-wise direction), and cross-sectional homogeneity is assumed. If steady, uniform flow can be assumed, and if constituent kinetic processes can be adequately described with rather simplistic relationships, such as first order decay, then analytical solutions are still possible. The stream model, STEADY, is a good example of using an analytical solution procedure to provide a relatively simple method of modeling temperature and dissolved oxygen (DO) in streams.

STEADY (Martin 1986a) is based upon 1-D, longitudinal, steady-state, analytical methods for determining stream-wise variations in temperature and DO. STEADY allows comparisons of different flow regimes, inflow loadings and meteorological conditions on the spatial distribution of water temperatures and dissolved oxygen concentrations in flowing systems under steady-state conditions. It is easily applied and requires minimal input data.

While STEADY is based upon analytical solutions, its coding allows simulation of a series of piece-wise non-uniform segments which make up a river subreach. A series of subreaches with different, but steady, flows can make up the total river reach modeled. The effects of withdrawals, branches, and tributaries can be simulated.

STEADY was originally developed for the Nashville District by the Waterways Experiment Station (WES) to allow a reconnaissance level assessment of temperature variations above and below a proposed reregulation dam downstream of Wolf Creek Dam, Kentucky. In subsequent studies, the unsteady flow model, CE-QUAL-RIV1, was applied to the same system. Comparisons indicated that STEADY provided water temperature predictions nearly identical to time-averaged values from the unsteady model. STEADY is considered appropriate for other applications where steady-state or time-averaged predictions are useful. The model has also been applied by the Walla Walla District to assess temperature variations in river reaches affected by proposed drainage from Lake Malheur, Oregon.

When assumptions that facilitate analytical solutions become inappropriate or do not allow for enough flexibility, it becomes necessary to resort to numerical water quality models. Numerical water quality models for rivers and streams vary widely due to the amount of detail allowed, the number and type of water quality constituents, and whether or not the model allows for time-varying conditions. Discussions that follow provide an overview of two 1-D, numerical, stream water quality models used by WES on CE projects.

QUAL-II

QUAL-II is a 1-D (longitudinal) stream water quality model having branching capability (Roesner et al. 1977) and is maintained by the EPA. The basic equation to be solved is the time dependent water quality constituent transport equation which allows for changes in each constituent due to advection, dispersion, and sources and sinks.

QUAL-II subdivides a stream system into basic sections called reaches. Each reach represents a portion of the river having similar channel geometry, hydraulic characteristics, and chemical and biological coefficients. Reaches are further divided into equally spaced units called computational elements or nodes. The constituent transport equation is solved simultaneously (implicitly) for all elements in the system.

Hydraulic conditions (flow rate and depth) used within the constituent transport equations are determined from steady, non-uniform flow by satisfying continuity and using either stage-discharge relationships or solution of Manning's equation with channel geometry information. Steady flow implies that flow, velocity, width, and depth at a given point in the stream are constant with time. Non-uniform flow allows these factors to change longitudinally from reach to reach.

QUAL-II was applied by WES (Hamlin and Nestler 1986) to 108 miles of the Rogue River downstream of Lost Creek Dam in Oregon. The study objective was to provide the Portland District with a model that could predict the downstream effects on water temperatures of the Lost Creek Dam operation. Specific temperature ranges are critical to Rogue River salmon abundance. The model was calibrated and verified with data collected from 1978 to 1981. For this study, the model was modified to allow for variable discharge updates at inflow boundaries. Use of this feature provided acceptable results as long as flow updates were gradual with respect to the system's travel time. Flow changes in this system were gradual, thus this model was selected as opposed to using the CE-QUAL-RIV1 model. The model was considered to be quite accurate for temperature predictions as Figure 2 demonstrates for the Merlin gage (River Mile 86) during 1979.

One problem encountered in the Rogue River study was the lack of flow and temperature data for many of the tributaries. The ungaged tributaries were estimated to contribute 30 to 50 percent of the flow in the river depending on the time period modeled. To solve this problem, regression equations for temperature and flow based on STEADY (Martin 1986a) and basin characteristics, respectively, were developed for these tributaries. This development substantially improved model accuracy.

The Rogue River model has been developed within an interactive, user-friendly system so that the Portland District can easily apply the model for various operational scenarios.

At the time this paper was prepared, the QUAL-II model was also being applied by WES to the Red River in Louisiana for the Vicksburg District (Martin 1986b). The objective of the Red River application was to assess the impact of a

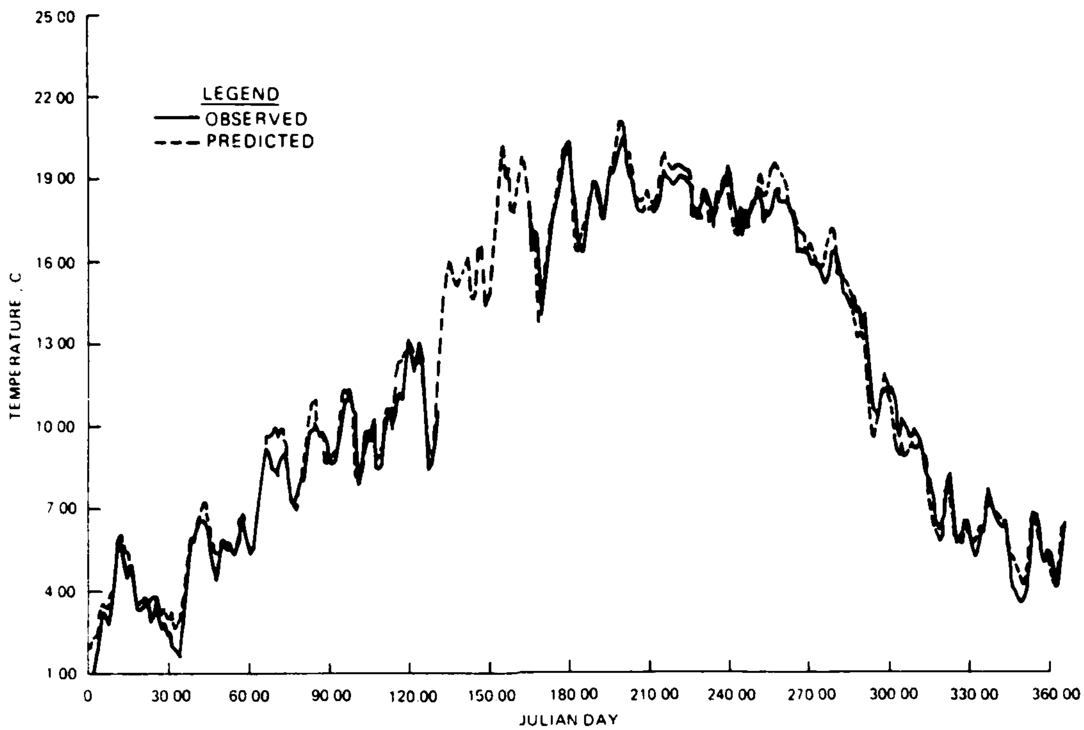

Figure 2. Comparison of Predicted Temperature Using QUAL-II with Observations for the Rogue River at the Merlin Gage during 1979

series of lock and dams on DO concentrations in the waterway. The model was calibrated with data collected during 1979 and 1985. The 1979 and 1985 data were collected prior to and following, respectively, the construction of Lock and Dam 1. The model was then used to assess the effects of the ongoing staged construction of a series of five lock and dams and the effects of the completed project. The application allowed assessment of structural reaeration requirements for maintaining DO concentrations in the waterway. The effects of present as well as projected future loadings were assessed.

CE-QUAL-RIV1

CE-QUAL-RIV1, a 1-D, unsteady flow, riverine water quality model (Bedford et al. 1983), actually is comprised of two sub-models: a hydrodynamic model, RIV1H, which can stand alone and a water quality model, RIV1Q, which requires output from RIV1H or another routing model to drive it.

RIV1H uses the four point, implicit, finite difference scheme to solve for flows and elevations. The model's formulation allows unequal steps in time and space and simulation of branched river systems with multiple hydraulic control structures (flow regulating structures such as weirs and dams).

RIV1Q uses a two point, fourth order accurate scheme to calculate advective transport. This allows sharp gradients in water quality constituents to be accurately resolved. The program can simulate ten water quality variables: temperature, dissolved oxygen, CBOD, organic nitrogen, ammonia, nitrite plus nitrate, phosphate, dissolved iron and manganese, and coliform bacteria. Reaeration may take place via stream reaeration, wind-driven reaeration, and reaeration through control structures. Temperature may be modeled with a direct energy balance method or equilibrium temperature calculations.

This model's versatility in simulating time-varying flows and water quality has lead to its use in a variety of situations. Three applications presented here cover a wide range of systems and constraints. The Chattahoochee River study (Zimmerman and Dortch 1986), conducted for the Savannah District, addressed potential effects on water quality of building a proposed reregulation dam below a peaking hydropower dam (Buford Dam) to meet projected water supply needs for the Atlanta area. The Wolf Creek Dam study (Martin 1986c) on the Cumberland River in Kentucky (Nashville District) addressed water quality impacts associated with proposed modifications of adding peaking production and a downstream reregulation dam to an existing baseload hydropower project. The on-going modeling effort for the Lower Ohio River Multipurpose Study (LORMS) of the Louisville District addresses the water quality impacts of retrofitting for hydropower generation six lock and dams on a major inland waterway.

In the Chattahoochee River study, CE-QUAL-RIV1 simulated unsteady flow and water quality conditions over 50 river miles. Concerns addressed in the study included possible development of unsuitably high temperatures, low dissolved oxygen concentrations, and high concentrations of dissolved manganese and iron. Summer and autumn water quality conditions were modeled as they represented times when highest temperatures and lowest dissolved oxygen concentrations, respectively, could be expected. The model was calibrated and verified using prototype data for most of the model variables and a variety of conditions. Figure 3 demonstrates how well the model predicted stages, dye concentrations, and temperatures as a power wave moved downstream. After the model was fully calibrated and verified, Zimmerman and Dortch (1986) made numerous simulations to compare various conditions with and without the proposed reregulation dam and to investigate various reregulation dam design and operational alternatives.

The CE-QUAL-RIV1 model was applied to a 16 mile reach of the Cumberland River below Wolf Creek Dam, Kentucky. Hydropower modifications have been proposed which include uprating existing units, a change from base load to peaking operation, and construction of downstream reregulation dam to attenuate the power wave. Like the Chattahoochee River study, the highly unsteady releases from Wolf Creek Dam required application of an unsteady flow model, such as CE-QUAL-RIV1, to examine time-varying conditions. The objective of the model study was to assess the impact of the proposed modifications on water quality and fisheries habitat. Fisheries habitat was evaluated by Curtis et al. (1986) using the PHABASIM system (Milhous et al. 1984) driven by the unsteady flow output of RIV1H.

The LORMS application, currently in progress, examines how proposed retrofitting a series of lock and dams for hydropower may influence water quality, especially dissolved oxygen, along about 500 miles of the lower Ohio River. The system is being modeled for mid through late summer conditions, when severest water quality impacts may be expected. To facilitate this application, RIV1Q is being coupled to another unsteady flow hydraulic model that was previously applied by the Louisville District to this river reach. CE-QUAL-RIV1 was chosen for this study because the retrofitted hydropower may involve peaking operations, and the multiple lock and dams can be handled more easily with the capabilities of this model.

CE-QUAL-RIV1 has already proved itself a valuable and reliable tool for predicting water quality in a wide variety of riverine and reservoir tailwater situations. While unsteady flow modeling is a primary attribute of the code, steady flows can be simulated as well. Different approaches to predicting temperature are easily implemented, and the code includes several methods for modeling reaeration. The ten water quality variables presently in the program cover most common water quality modeling situations, and select variables can be modeled without having to model all ten.

CONCLUSIONS

No single method is appropriate for addressing all water quality issues in regulated streams. Differences in stream environments and management issues require that a variety of tools be available for assessing existing or predicting future impacts of reservoir releases and stream regulation. The various stream water quality models discussed here have provided an effective means of addressing reservoir tailwater and stream water quality issues. Each particular model presented has its own merits and limitations and should be selected based on the specific needs of an application.

Additional research is presently underway to better identify and describe factors influencing the quality of reservoir tailwaters. An additional tool under development is the system for Management and Analysis of Tailwater Quality (MATQ), which will provide a user-friendly method of estimating, with limited input data requirements, effects of reservoir releases on downstream water quality. The need for such information and techniques to aid in understanding and managing our reservoir tailwaters and regulated streams becomes increasingly important as we attempt to increase benefits we obtain from our projects.

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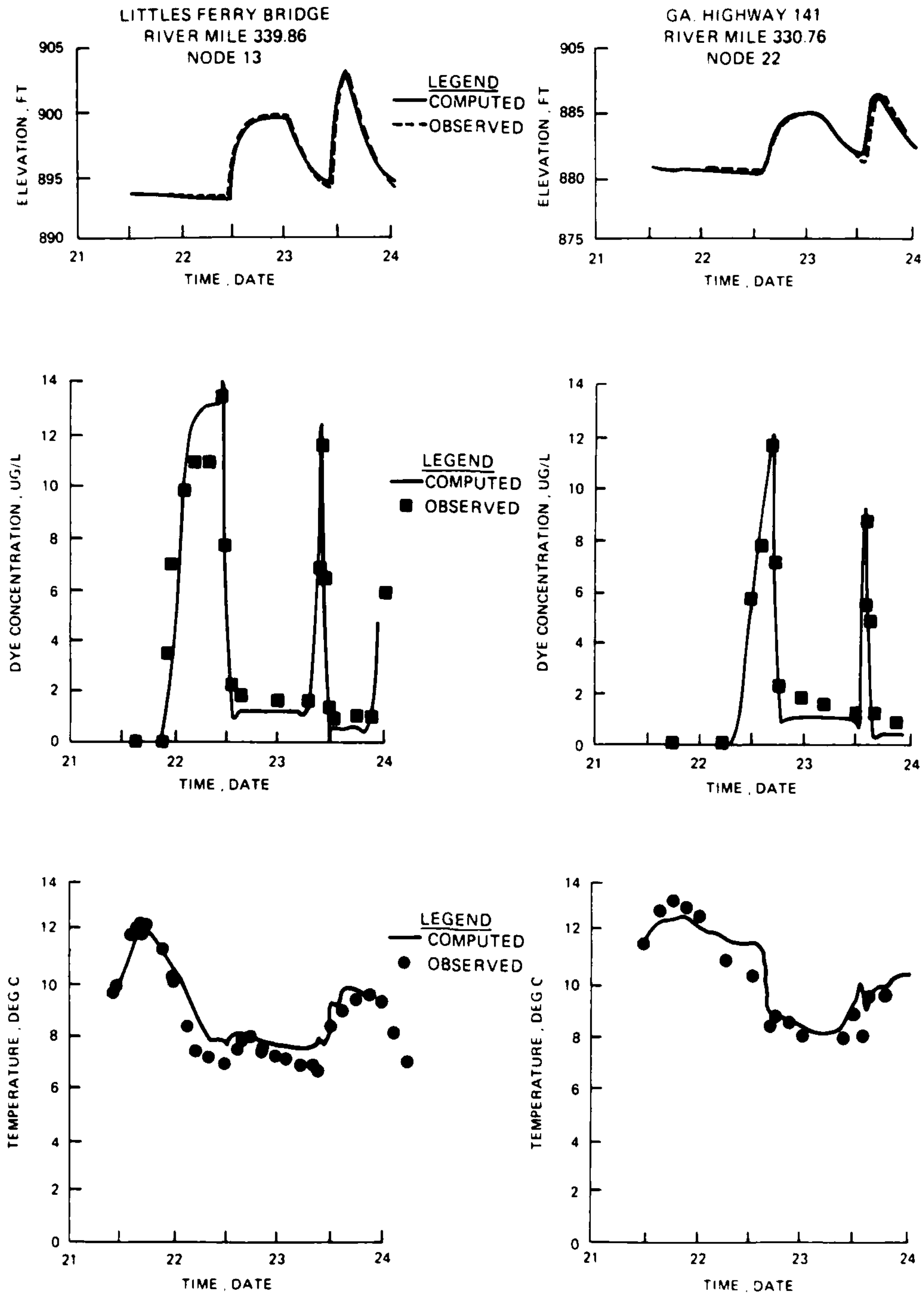

Figure 3. Comparison of Predicted Stage, Dye Concentration, and Temperature Using CE-QUAL-RIV1 with Observations at Two Stations on the Chattahoochee River for a Dynamic Flow Event during March 21-24, 1976

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Effects of Aeration and Minimum Flow on the Biota and Fishery of Norris Tailwater

by Donley M. Hill
and William M. Seawell

Tennessee Valley Authority
Knoxville, Tennessee

ABSTRACT

Low dissolved oxygen (DO) in the releases from Norris Reservoir was improved through turbine venting in 1981, and minimum flows were increased in 1984. Subsequently, several beneficial changes have occurred in the tailwater. Invertebrates sensitive to environmental stress have increased, and trout condition has improved somewhat during the season when low DO is most pronounced. These improved conditions have allowed an increase in number of trout stocked and in fishing use of the tailwater. A trophy fishery for brown trout has developed and is receiving widespread publicity. Possible reasons for the lack of complete recovery of aquatic life are presented.

INTRODUCTION

Hypolimnetic discharges from storage reservoirs can significantly affect downstream communities of fish and benthic invertebrates through altered stream temperatures, extreme and frequent fluctuations in flow, inadequate minimum flows, increased heavy metal concentrations, and reduced stream organic matter. Some of these effects can be positive (Axon, 1976). Cold hypolimnetic releases have created many miles of big river trout water where previously none existed. However, several conditions caused by these releases are detrimental to both fish and benthic invertebrates, principally low dissolved oxygen (DO), inadequate minimum flows, and associated phenomena. Low DO concentrations have been demonstrated to limit growth and survival of fish in laboratory studies (USEPA, 1986); and similar effects are believed to occur in some tailwaters.

Invertebrates such as midges, isopods, and amphipods are tolerant and are often very abundant in streams with low DO (Davis, 1975). Benthic invertebrates such as mayflies, stoneflies, and caddisflies are, however, sensitive to low DO and are not abundant in affected waters. Inadequate minimum flows can cause substrate drying, reduce available habitat, and favor invertebrates which are adaptable to these conditions. These invertebrates also include many of the same types that are tolerant of low DO, i.e., midges, isopods, and amphipods. (Brown et al., 1968).

Adverse effects of low DO on fish may include loss of appetite, depressed growth and condition, increased susceptibility to disease and predation, and in extreme cases, mortality. Low DO also limits the number of fish species in a stream and the size and diversity of fish food organisms. Low stream flows below tailwaters reduce water depths and velocity and change the amount of preferred habitat for individual species and can reduce the number and weight of fish a stream can support both by direct influence on their physical habitat and by affecting the food base.

Approximately 191 miles of the tailwaters of 10 tributary reservoirs operated by the Tennessee Valley Authority are affected by low DO and/or lack of minimum releases. In these stream reaches, fish and aquatic invertebrates are limited, and recreational use is held well below its potential.

A study of the Norris tailwater from 1971-1977 (Boles, 1980) identified low DO and lack of adequate minimum flow as the primary factors limiting further development of the trout fishery. Partly as a result of these findings, TVA's Reservoir Releases Improvements Program (TVA, 1980) began an effort to improve dissolved oxygen and flow conditions in the tailwater. In 1980 a hub baffle system was installed and tested intermittently on one of the two Norris turbines. This aeration system increased minimum DO in the releases by an average of about 0.7 mg/l in 1981, and 2-3 mg/l from 1982-1985. The system was operated when DO in the releases fell below 4 mg/l.

Construction of a flow reregulation weir approximately 3.2 km (2 mi) downstream of Norris Dam was completed in May 1985, and water stored behind the weir, supplied by pulsed flow from the hydropower units, has since provided a minimum flow of approximately 5.7 m^3/s (200 ft^3/sec) downstream. Approximately 1/2 hour of flow from one turbine is required to fill the pool behind the weir which supplies flow downstream for twelve hours.

The purpose of this paper is to describe changes in fish, invertebrate populations, and angler use in Norris tailwater that have accompanied aeration of releases and operation of the flow reregulation weir.

MATERIALS AND METHODS

Description of Study Area

Norris Dam impounds a storage reservoir located at mile 79.8 of the Clinch River. It was impounded in 1936, covers 13,841 ha (34,200 ac) at normal full pool level, and has a drainage area of 7,539 km^2 (2,912 mi^2). Water is normally released from the reservoir's hypolim- nion through two hydroelectric units. Maximum two-unit discharge is approximately 241 m^3/sec (8,500 $ft.^3/sec$). Flow is controlled primar- ily for peak power generation to meet power loads.

DO varies seasonally in the releases from Norris Dam, with lowest concentrations (as low as 0.6 mg/l) usually recorded in the early autumn. A typical DO and temperature, and discharge cycle is shown in Figure 1.

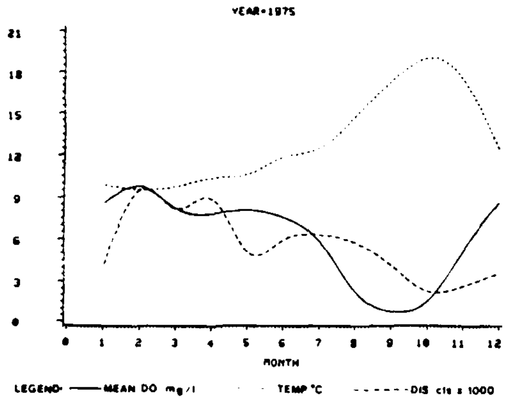

Figure 1. Representative DO, temperature, and discharge trend in turbine discharge from Norris Reservoir for 1975.

The tailwater extends approximately 22.5 km (14 mi) downstream from Norris Dam to the headwaters of Melton Hill Reservoir. Average width of the river at two-unit flow from Norris Dam is approximately 131.6 m (432 ft), while under minimum flow conditions width is 94.5 m (310 ft). Water level in the river fluctuates about 1.8 m (6 ft) between periods of two-turbine generation and no generation. Average unregulated flow at the dam site (1903-1969) was 120.8 m^3/sec (4,624 ft^3/sec). Stream gradient is about 0.6 m (2 ft) per mile with a normal sequence of pools and riffles. Bedrock is the primary substrate in the tailwater, with small patches of rubble, gravel, and cobble. Extensive growth of filamentous green algae, primarily Oedogonium with small pockets of Cladophora, covers much of the bottom.

The river is used for boat, bank, and wade fishing. Most bank and wade fishing takes place when the turbines are off and access is limited primarily to the upper 6.4 km (4 mi) below Norris Dam and a short section 22.5 km (14 mi) downstream of the dam. Most boat fishing takes place when the turbines are operating. At full flows the entire 22.5 km may be floated.

Benthos

Four quantitative samples for benthic invertebrates were collected at each of 5 tailwater locations (Figure 2) with an unmodified 0.09 m^2 (1 ft^2) surber sampler with a 750 micron mesh net. Samples were collected from shoal areas less than 0.3 m deep from substrate consisting mostly of cobble and/or rubble. Samples were fixed in a 10 percent formalin solution. After sorting, invertebrates were preserved in alcohol, identified (usually to genus), and counted.

Fish Population Sampling

Samples were collected with chemical fish toxicants (sodium cyanide or rotenone) at river miles 77.1 and 70.5 from 1971-1977 and 1980-1985. River mile 70.5 was not sampled in 1973 or 1977. Sampling was conducted twice annually in summer and autumn). Fish collected in these samples were identified, weighed, and measured.

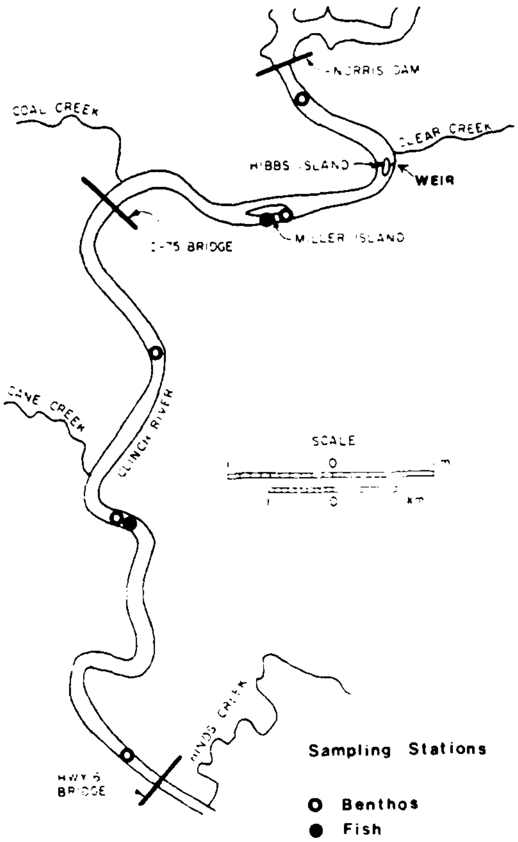

Figure 2. Sampling stations for benthos and fish in Norris tailwater.

Creel

A roving creel census based on non uniform probability design developed by Dr. Don Hayne, North Carolina State University was used. Sampling was conducted in accessible areas of the tailwater five (5) days a week. All weekends and holidays were included; then enough weekdays are randomly selected to complete the needed number of workdays during a period. Census days were divided into two work periods, sunrise until noon, and noon until sunset. The period censused for each day was chosen by random selection. The time for a count of anglers was then chosen by random selection. At this preselected time the clerk counted all persons fishing in accessible areas. During the remainder of the day the clerk interviewed anglers. Information collected included time fished, number caught, and length and weight of individual fish.

The fishing pressure count was then expanded to estimate total fishing pressure for the day. Harvest estimates were made for each period as the product of mean daily effort and estimated mean catch per effort. Estimates from individual periods are summed to produce monthly, quarterly, and annual estimates.

Trout Condition

Relative condition, proposed by LeCren (1957) to eliminate trends in condition associated with length, was used as a measure of the changes in plumpness (condition) of fish harvested by anglers. Condition was used to examine changes in growth rates after aeration of releases and increased minimum flow.

RESULTS AND DISCUSSION

Physical Changes

Aeration and flow enhancement measurably improved physical habitat in Norris tailwater. The minimum dissolved oxygen concentrations in the releases which typically occur during August through October were elevated by as much as 3 mg/ℓ immediately below the dam and the effects diminished downstream (Figure 3). Flow, temperature, and water quality modeling (Beard and Hauser, 1986) under low flow conditions (pulsing to maintain weir pool) typical of weekend operations indicate that the increased minimum flow:

1. Decreased daily maximum temperature by 0.5 - 1° C.
2. Increased daily minimum DO by about 1 mg/ℓ below the weir.
3. Decreased daily maximum DO by about 2 mg/ℓ below the weir.

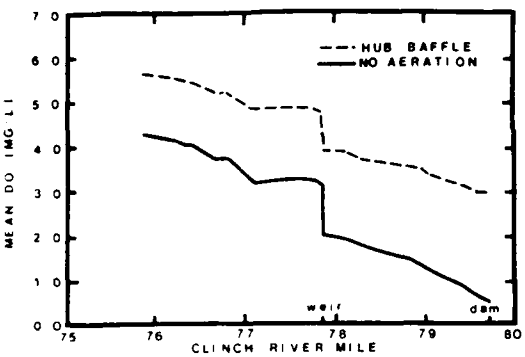

Figure 3. Mean DO (Mg/P) during a "typical" 24-hour period with 2-unit discharge (8,000 cfs) from 10 a.m. until 10 p.m. with and without hub baffles (after Beard and Hauser, 1986).

Under high flow conditions water residence times are considerably reduced, dampening diurnal variations in DO and temperature.

Flow model simulations under minimum flow conditions also indicate the wetted area of the stream is increased about 25 percent as a result of impoundment upstream of the weir and about 10 percent downstream from a sustained minimum flow.

Effects on Benthic Fauna

Although the Norris tailwater benthic fauna community continues to be dominated by chironomids, isopods, and amphipods, less tolerant taxa are occurring more frequently (Figure 4). Caddisflies, crayfish, snails, stoneflies, and mayflies, all desirable food for trout, now occur more frequently at all stations in the tailwater except immediately below the dam. Caddisflies, crayfish, and snails are also beginning to influence the benthic fauna community structure as their percentage composition increases relative to other taxa (Table 1). The overall numerical abundance of bottom fauna organisms in samples taken has not changed significantly since aeration and flow enhancement, even though these desireable forms are more abundant.

The delayed, and as yet incomplete recovery of the Norris tailwater benthic fauna community may be because DO is still too low to allow survival and reproduction of some sensitive benthic species, a shortage of colonizers, or because full recovery simply takes longer than expected. A tranplant of additional invertebrates was made into the tailwater in 1985 in an effort to increase the speed of recovery. The results of this transplant are being monitored.

Table 1. Percent composition of 10 taxa of bottom fauna organisms in Norris Tailwater from 1971-1985.

Taxonomic Group	Community Percent Composition 1971-1975	1980-1984	1985
Caddisflies	0.27	1.07	3.43
Mayflies	0.01	0.01	0.01
Stoneflies	0.01	0.01	0.01
Black flies	9.11	3.98	6.54
Midges	59.98	70.74	59.99
Crayfish	0.00	0.03	0.04
Snails	0.05	0.12	0.43
Amphipods	5.76	6.13	11.13
Isopods	17.60	9.26	12.60
Others	7.21	8.65	5.72

Response of the Fishery

Prior to 1971, the Norris tailwater trout fishery was not intensively managed, and fishing pressure was relatively light (estimated at 10,000 hours per year). Along with increased stocking of both fingerling and catchable rainbow and brown trout (Table 2), angling pressure has increased significantly during recent years (Figure 5). Effort during 1980-83, when DO improvements were made and public awareness of the fishery at Norris increased, was 17 percent greater than the 1970s (average of 54,394 hours/year for 1980-83 vs average of 46,520 hours/year for the 1970s), an increase of approximately 17 percent.

During 1984-85 following establishment of a 5.7m/sec (200 ft^3/sec) minimum flow, angling pressure again showed a 79 percent increase (average of 83,388 hours/year) over the 1970s and was also significantly greater than the

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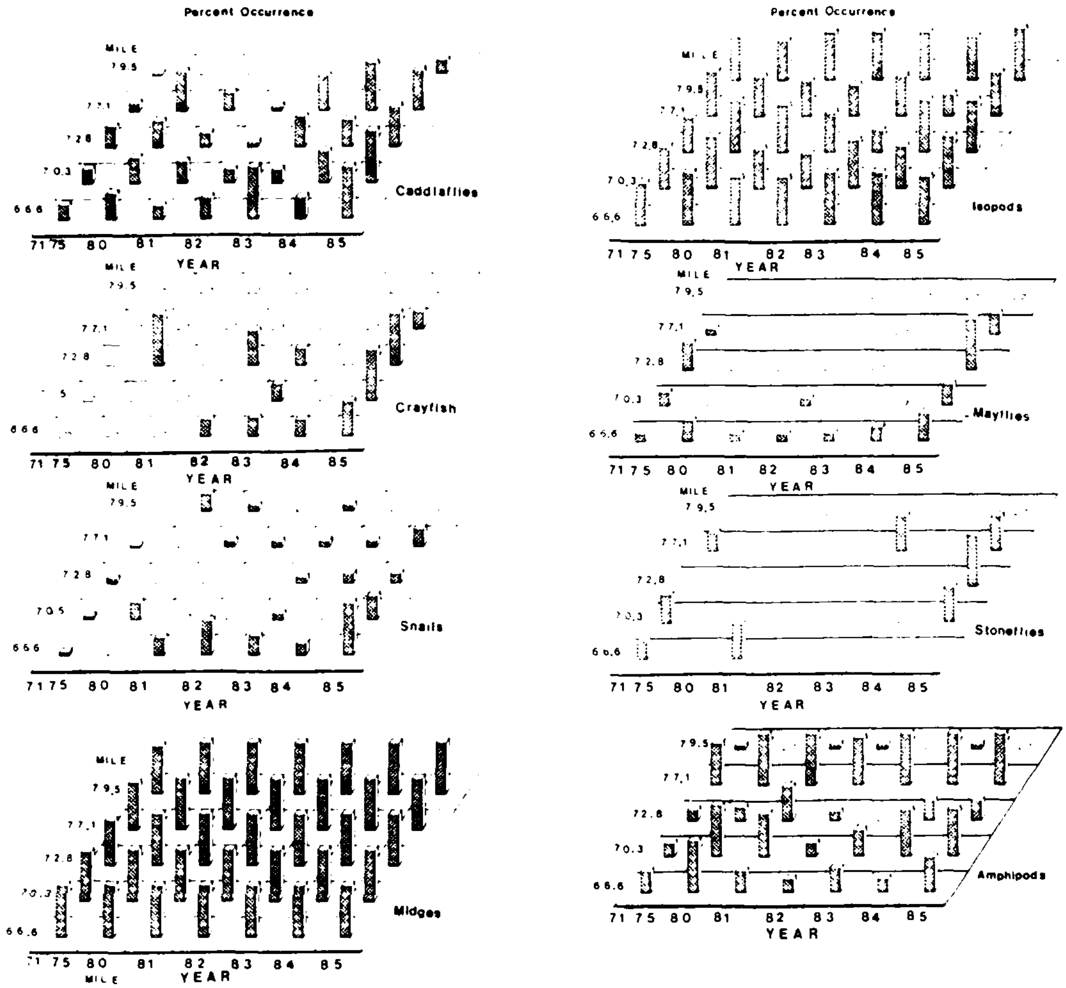

Figure 4. *Frequency of occurrence (percentage of samples which contained one or more individuals of a particular order) of major orders of Norris tailwater bottom fauna organisms. Graphic scales are for relative comparisons within an order and are not comparable among orders.*

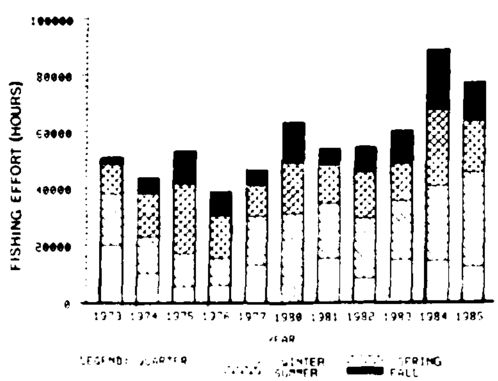

Figure 5. Norris tailwater fishing effort (fisherman hours per year), 1973-1977 and 1980-1985.

1980-83 period. Expansion of angling pressure is probably attributable to a number of factors, including: public interest and awareness of efforts to enhance the fishery at Norris (i.e., DO improvements; increase in stocking of rainbow trout; and successful establishment of a trophy brown trout fishery); provision of more stable, esthetically pleasing, and more "fishable" waters by maintenance of a minimum flow; and additional access created by the flow reregulation weir.

During the 11-year creel period (1973-77, 1980-85) an estimated 254,170 trout of all species weighing 37,323 kg were harvested from the tailwater. Total annual harvest of trout increased significantly (Table 3). Average harvest was 15,656 fish (2,404 kg) per year in the 1970s and 27,640 fish (4,217 kg) per year in the 1980s. Increased total annual harvest was related primarily to two factors, increased total fishing effort and increased stocking of catchable-sized fish which both occurred under improved DO and/or flow conditions during the 1980s.

Table 2. Trout stocking record, Norris tailwater 1973-77 and 1980-85. Numbers include both rainbow and brown trout.

Year	Total Number of Catchables	Total Number of Fingerlings
1973	48,290	0
1974	14,842	244,285
1975	0	209,359
1976	0	159,989
1977	10,001	180,256
1980	37,743	160,000
1981	10,000	145,723
1982	30,433	134,400
1983	36,733	159,926
1984	29,084	180,000
1985	37,759	159,572

The number or weight of fish harvested in a unit of time (catch rate) is probably the best measure of changes in a fishery. Figure 6 shows yearly catch rates in Norris tailwater. Average annual catch rates improved from 0.34 fish/hour in 1973-77 to 0.43 fish/hour for 1980-85. Aeration of the releases and establishment of a minimum flow had little direct effect on catch rate. These improved catch rates were however highly correlated with increased annual stocking of catchable sized trout, demonstrating that the fishery was improved by altered stocking strategies which took place concurrently with, and are impossible to separate from, improved DO and flow conditions.

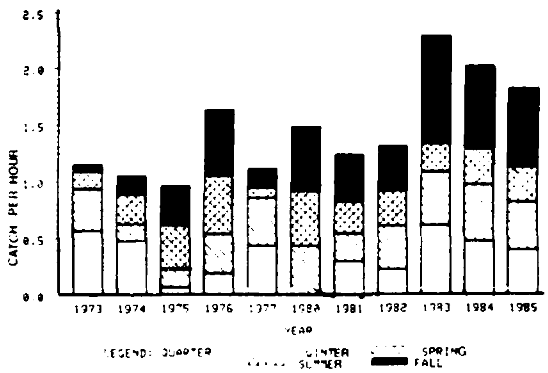

Figure 6. Norris tailwater angler success (number of fish caught per hour of effort) during 1973-1977 and 1980-1985.

Table 3. Estimated annual harvest for the Norris Tailwater Trout fishery, 1973-77 and 1980-85.

Year	Total Harvest (Number)	Total Harvest Kilograms
1973	19,591	3,424
1974	11,538	1,761
1975	15,661	2,307
1976	16,871	2,251
1977	14,617	2,300
1980	20,416	3,395
1981	15,540	1,562
1982	18,389	3,602
1983	33,436	3,124
1984	44,085	8,098
1985	33,976	5,518
Total	254,120	37,316
x	23,102	3,392

Average drop in LeCren condition of trout between summer and autumn is shown in Figure 7. There is an apparent trend toward improving trout condition following aeration, although there are no statistically significant differences between pre- and post-aeration years. The best year on record occurred during 1985. Flow is apparently a major factor affecting this condition, since high turbine discharges expose fish to low DO for long periods, even with turbine aeration. Record low flows occurred in summer and autumn 1985, while 1984 had extremely high flows.

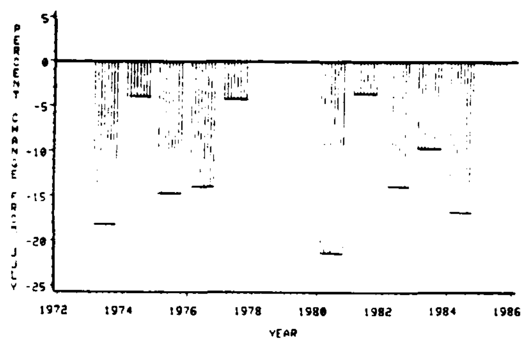

Figure 7. Percent change in LeCren condition of trout in Norris tailwater population samples between July and October.

All actions in concert have dramatically improved the fishery in Norris tailwater. Recovery of invertebrates is still not complete, and growth of individual trout has been only minimally affected, indicating a need for additional increases in dissolved oxygen in the releases and possibly increased minimum flow. Additional access would also allow increases in fishing use in the tailwater.

Because no dramatic improvements in fish condition have taken place after aeration, we conclude that DO has not improved sufficiently to produce measurable changes in fish growth. Recent experiments in large channels at TVAs Aquatic Research Lab (TVA, unpublished) indicate that under fluctuating conditions, mean DO concentrations of around 6 mg/ℓ are required to bring about significant improvements in rainbow trout growth. Until such concentrations are achieved in Norris tailwater, measurable improvements in fish condition are not likely to occur. This finding is further substantiated by the fact that condition of trout in Norris tailwater starts to decline when DO in the releases falls below about 6 mg/ℓ.

SUMMARY AND CONCLUSIONS

Seasonal aeration of the releases from Norris Reservoir beginning in 1981 and an increased minimum flow of 5.7 m^3/sec (200 ft^3/sec) dramatically improved dissolved oxygen and flow conditions in the 22.5 km (14 mi) reach of the tailwater. With these improvements some beneficial changes in the tailwater biota have also occurred. Invertebrates which are more sensitive to environmental perturbations and desireable as fish food (caddisflies, mayflies, stoneflies, crayfish and snails) began to occur more frequently throughout the tailwater beginning in 1985. The delayed and incomplete recovery may be a result of DO levels which are still below threshold levels for survival and reproduction of some sensitive benthic species, the scarcity of available colonizers, or a longer than expected time required for full recovery.

Fishing pressure, catch rate, and total annual harvests, increased significantly during aeration and following increased minimum flow in the tailwater. These increases are believed to be due to an increased public awareness of efforts to improve the fishery; improved DO during critical periods of the year; more stable, esthetically pleasing flow conditions; increased trout stocking; and improved access around the weir.

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PREDICTING EFFECTS OF REREGULATION DOWNSTREAM OF BUFORD DAM, GEORGIA

John M. Nestler

Research Ecologist, USAE Waterways Experiment Station, Vicksburg, Miss

ABSTRACT

This presentation describes a case-history application of the Physical Habitat Simulation (PHABSIM) system, developed by the US Fish and Wildlife Service. The application resolved potential instream flow conflicts between needs for water supply and trout habitat that would result from operation of a reregulation dam downstream of Buford Dam, Chattahoochee River, Georgia. This study documents how potential downstream conflicts between project operation and fishery concerns can be approached, formalized and resolved from both an institutional and technical standpoint. The study results indicate that multiple downstream uses of the releases from the reregulation dam can be accommodated with relatively minor changes in project operation.

INTRODUCTION

As part of water resources development and management, reservoir projects are operated for flood control, water supply, navigation, irrigation, power generation, and other beneficial uses. However, operation of reservoir projects, particularly peaking projects, can cause considerable modification from preimpoundment conditions in the downstream reaches. Peaking hydropower operation can have potentially severe effects on the tailwater ecosystem because of extreme short-term flow and water quality alterations and long-term changes in channel morphology. These effects are restricted more to peaking operation than other types of project operation because of the rapid changes in flow associated with meeting peak demand for power and because most peaking projects are deep release. Deep releases usually depart more from equilibrium conditions than surface releases, particularly in the summer.

One alternative often identified to decrease the detrimental effects of peaking operation is to construct a smaller dam downstream of the peaking hydropower project to reregulate the releases. The smaller reregulation dam attenuates or eliminates the peaking releases and instead releases a more constant flow into the river. However, the benefits and trade-offs associated with reregulation have not been well quantified.

This case-history study presents results from a cooperative study performed by the USAE Waterways Experiment Station and the US Fish and Wildlife Service to predict the effects of reregulation of the Chattahoochee River below Buford Dam, Georgia on downstream trout habitat (Nestler et al. 1985) using the PHABSIM system. This study was limited to trout (juvenile and adult brown trout - _Salmo trutta_, adult rainbow trout - _S. gairdneri_, adult brook trout - _Salvelinus fontinalis_) although many other species of fish also occur in the Chattahoochee River (Hess 1980) because of the value of the trout fishery.

Study Background

In 1974 county governments in the Atlanta vicinity determined that demands on the Chattahoochee River for water supply plus the streamflow required for water quality nearly equaled the minimum low flow release from Buford Dam (Atlanta Regional Commission 1981). Projected increases in water supply demand in the following years could not be supplied under the existing release schedule in the river. In response to the anticipated shortage of water, regional planners suggested a number of alternatives to increase the base flow in the Chattahoochee River. The preferred alternative for providing future water supply was construction of a reregulation (rereg) dam about 6.3 river miles downstream Buford Dam. The proposed rereg dam would release a much more constant flow than the peaking flows presently released from Buford Dam (generally, a maximum release of approximately 8000 cubic feet per second (cfs) or minimum release of about 550 cfs) by storing the generation releases from Buford Dam for gradual release during non-generation periods. The anticipated minimum release from the rereg dam would be approximately 1050 cfs. Considerable concern was expressed that flow modifications caused by operation of the rereg dam (and resultant water quality changes) to meet increased demands for water supply within this reach of the Chattahoochee River could have a negative effect on the valuable trout fishery downstream of Buford Dam.

Site Description

The reaches of river investigated in this study (Figure 1) are bounded upstream by Buford Dam (River Mile 348.3) and downstream by the confluence with Peachtree Creek (RM 300.5). Three study reaches were identified based on the presence of actual or potential hydraulic control structures. The results of this study are presented separately for each of the major reaches of the Chattahoochee River: (a) from Buford Dam to the site of the potential rereg dam, (b) from the site of the potential rereg dam to the Headwaters of Bull Sluice Lake and (c) from Morgan Falls Dam (a small Georgia Power dam that creates Bull Sluice Lake) to the confluence of Peachtree Creek (river mile 300.5). The storage capacity of Morgan Falls Dam has been substantially reduced by siltation to the extent that daily

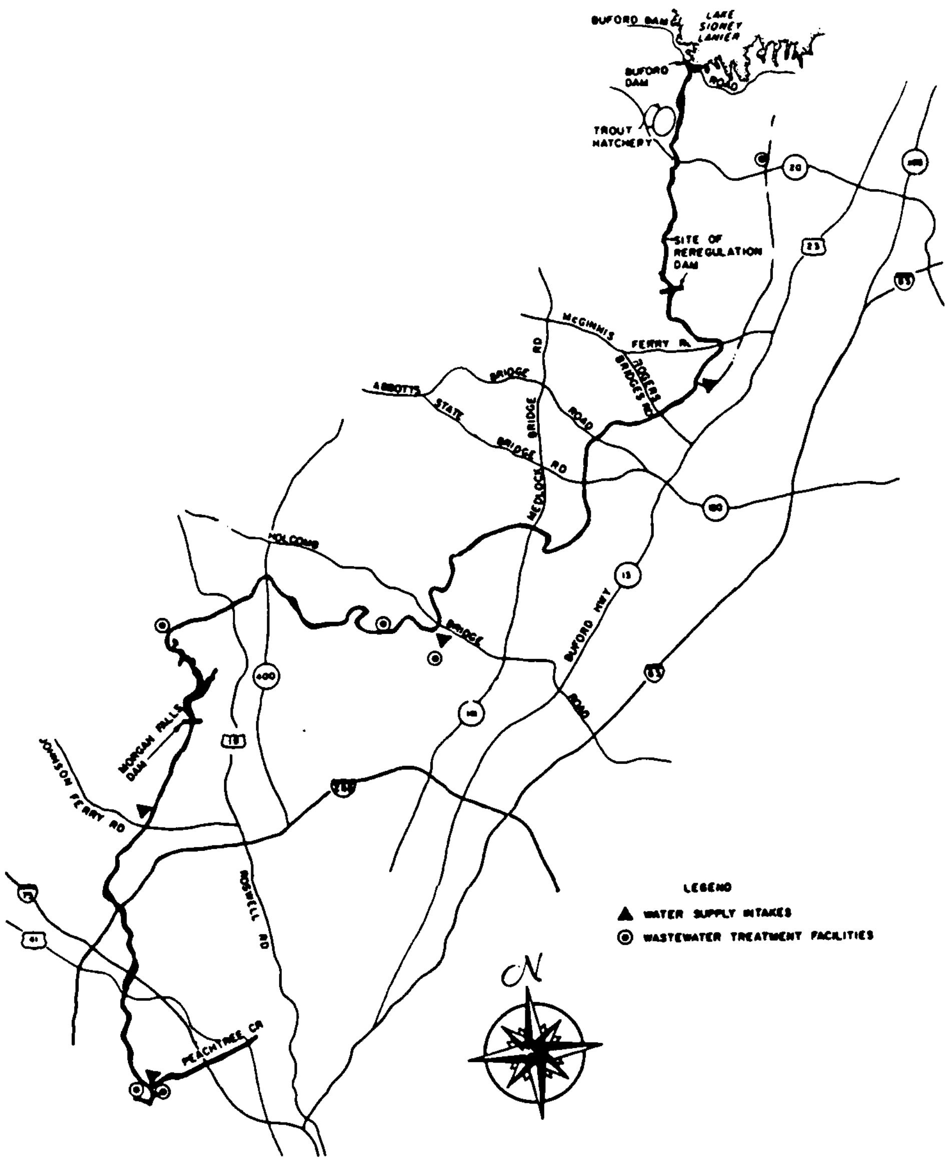

Figure 1. Map showing major features of study area.

operation is largely determined by the flows entering Bull Sluice Lake.

Operation of Buford Dam for peaking hydropower has considerably altered the preimpoundment flow regime. Buford Dam is peaked for a minimum of 11 hours per week with peak releases of about 8000 cfs occurring during the afternoon and evenings of weekdays. Minimum releases near 550 cfs are discharged during all other time periods. On autumn weekends, increased flows may be released from the dam to improve water quality conditions at the trout hatchery located near Buford Dam.

The year-round availability of cool water has allowed the development of a valuable "put-and-take" trout fishery in this formerly warm water river. Harvestable-size rainbow, brook, and brown trout are stocked at numerous points in the Chattahoochee River by the Georgia Game and Fish Division. Juvenile brown trout are stocked downstream of Morgan Falls Dam by the Georgia Game and Fish Division with the assistance of Trout Unlimited. Over 100,000 catchable-size trout are stocked in most years. Long-term survival of stocked brown trout has produced trophy-sized fish, particularly downstream from Morgan Falls Dam.

MATERIALS AND METHODS

General Approach

An examination and evaluation of the issues related to the proposed reregulation of Buford Dam releases indicated that, with some modification, the PHABSIM system could be used as a tool to predict and manage fish habitat in the

Chattahoochee River. PHABSIM was selected for the following reasons:

1) it is generally accepted by many agencies as a defensible method to assess the downstream effects of reservoir operation on fish habitat;

2) the results of an instream flow study using PHABSIM are particularly amenable to resolving potential water resources conflicts;

3) it is incremental, that is, it relates small changes in operation to changes in habitat for target life stages;

4) it is well-documented and supported;

5) it is flexible; the organization of the system is such that changes can be made easily as the state-of-the-art of fish habitat simulation increases and additional studies provide more information to the analysis.

PHABSIM Description

Background

The PHABSIM system is based on the observation that most species of fish prefer certain combinations of depth, velocity, and cover and tend to avoid other combinations of these parameters. If the relative values of different depths and velocities for each species are known and the hydraulic conditions within the channel can be described for different discharges, then it becomes possible to determine both the quality and quantity of habitat for each species of fish at these different discharges. Thus, an instream flow study consists of three essential steps. The first part involves a description of the depth, velocity, and cover available in the river at discrete discharges. The second part is the development of criteria for each species of fish. The last part of an instream flow study is to combine parts one and two for each discharge of interest to derive an estimate of the value or worth of the river for each fish species at each discharge. Figure 2 depicts a conceptualization of the major steps involved in performing an instream flow study. The discussion above provides only a summary of the conceptual basis of the PHABSIM system. Depending upon the problem at hand, the user may choose any of a number of different options and approaches to complete an analysis. The next sections will detail how these general steps were applied to the Chattahoochee River.

Suitability Curves

Criteria (or suitability) curves for each variable (depth, velocity, and cover) must be identified or generated to relate cell-by-cell flow conditions in the study river reach to useability by a target life stage. That is, the value of conditions in cell(i) for a particular target species or life stage can be assessed if the criteria (the value of different velocities, depths, and substrates) are either known initially or identifiable with further study. Figure 3 presents the criteria curves used in this analysis. Note that the range of potential values vary from zero (no value) to one (ideal conditions) for each variable. For most applications of PHABSIM, depth, velocity, and cover are assumed to be independent variables, even though

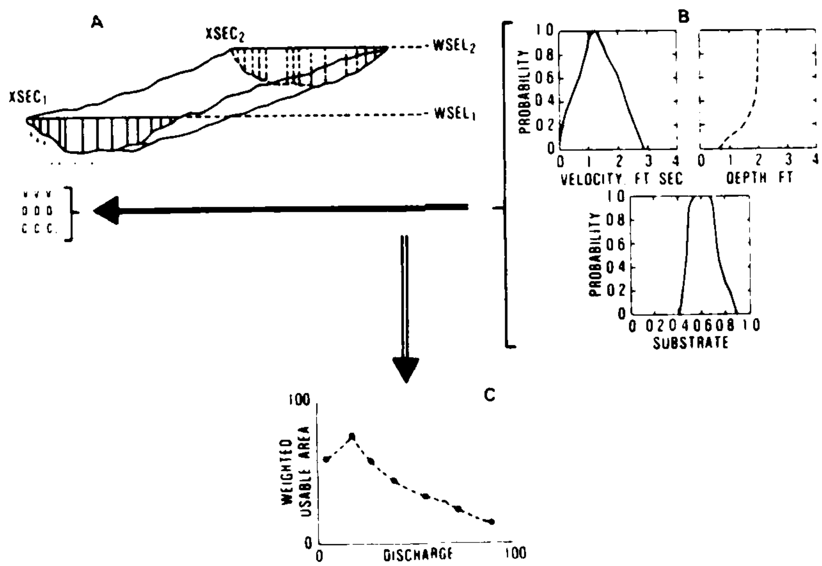

Figure 2. Conceptualization of how PHABSIM makes habitat predictions. First, as depicted in subfigure A, depth D(i) and velocity V(i) and cover conditions, C(i), in cell(i) are measured or simulated for a given discharge. These conditions are then evaluated relative to criteria in subfigure B to generate a single habitat value for each cell. The habitat values for all cells in the study reach are summed to obtain a single habitat value for the given discharge, depicted as a single point in subfigure C. This procedure is repeated for different discharges to obtain the graph in subfigure C.

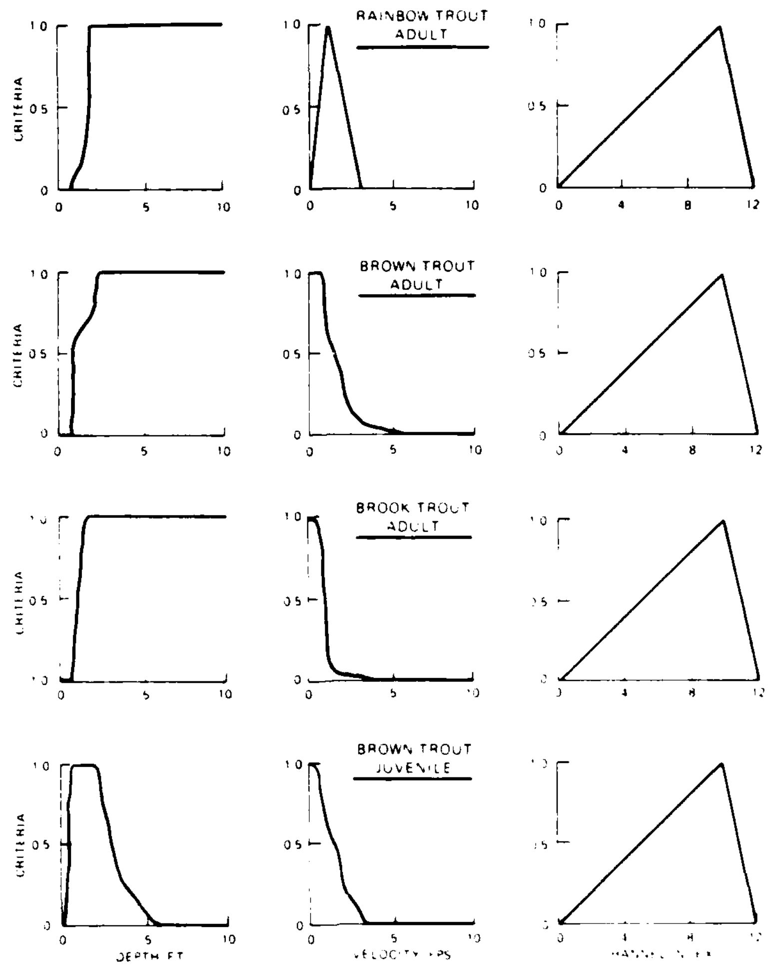

Figure 3. Plots of suitability functions for velocity, depth, and cover for target species analyzed in the Chattahoochee River Instream Flow Study. Note that suitability values range from 0 (no value) to 1.0 (optimal value).

these variables are known to be correlated. If necessary, bivariate representations of depth and velocity can be used; however, field data must then be collected to develop bivariate criteria since almost all published criteria curves present depth and velocity as independent variables. Additional information on interpreting, evaluating, and generating criteria curves can be found in Bovee (1986).

Estimating Habitat

In a typical application of PHABSIM, the values for depth, velocity, and substrate in cell(i) are each evaluated relative to the criteria for the target life stage to generate a weighting factor for the surface area of river represented by cell(i)

$$W(i) = suit(d) \times suit(v) \times suit(c) \quad (1)$$

where

W(i) - weighting factor for cell(i),
suit(d) - suitability of the depth in cell(i) for a given discharge for target life stage,
suit(v) - suitability of the velocity in cell(i) for a given discharge for target life stage,
suit(c) - suitability of the cover in cell(i) for target life stage.

The amount of river area (WUA) available for a target life stage in cell(i) can be represented as

$$WUA(i) = area(i) \times W(i) \quad (2)$$

where

WUA(i) - total weighted useable area of the river surface area represented by cell(i),
area(i) - area of river represented by cell(i),
W(i) - weighting factor for cell(i).

The total Weighted Useable Area (WUA) in the study reach available for use by the target life stage for a given discharge can then be represented as the sum of the weighted areas of each cell or

Nestler

$$\text{WUA} = \sum_{i=1}^{n} \text{WUA}(i) \tag{3}$$

where

WUA - total WUA for a given life stage,
WUA(i) - WUA in cell(i),
n - number of cells in the river reach of interest,
Σ - summation symbol.

This formulation allows estimation of a single habitat value for a river reach that is a function of discharge for the target life stage. The habitat available at other discharges can then be calculated in a similar fashion to generate a habitat versus discharge relationship. Additional information on different ways of combining individual criteria curves to generate a composite criteria or weighting value can be obtained in Milhous et al. (1984).

This discussion presents only the most fundamental underpinnings of PHABSIM. Many other techniques, options, and programs are available that provide for complete analyses of water development projects including comparisons of different operational or structural alternatives, time series analysis, and other types of habitat analyses. The reader should consult Instream Flow Information Paper No. 11 (Milhous et al. 1984) and No. 12 (Bovee 1985) for more details on the use and application of PHABSIM.

Flow and Channel Geometry Description

Collection of detailed field data was restricted primarily to shoal areas, particularly downstream of Morgan Falls Dam, because of their value for recreation and trout habitat. Reaches that were considered to provide less trout habitat were described using historical information previously collected either by the US Geological Survey (Faye and Cherry 1980) or by the Corps of Engineers (1974). Field data were collected by the US Geological Survey using standard methods of stream gaging.

A range of techniques are available in the PHABSIM system to simulate cell velocities and water surface elevation in a stream. The Water Surface Profile (WSP) Program (see Milhous et al. 1984 for more detailed information) was used where only cross section geometry data were available and cell water velocity information was unavailable. For the reaches investigated as part of the field effort for this study a stage-discharge relationship, developed separately for each cross section, was used to determine the water surface elevations at different discharges of interest (see Trihey and Wegner 1981 for more details). The IFG4 program was then used to determine the velocities within the stre .. channel given the stage-discharge relationship (see Milhous et al. 1984 for more information on the IFG4 program). Sensitivity analyses indicated that selection of an alternative hydraulic approach would not have had a significant impact on the final results.

Trout Suitability Criteria Curves

The criteria for the trout were based initially on information available in Bovee (1978). However, the curves of Bovee (1978) were based on trout behavior in trout streams in western states and their use, without modification, on a highly managed, upper piedmont, put-and-take trout fishery downstream from a peaking hydropower project in the southeast would be problematic, at best. The criteria were modified (Figure 1) through consultation with two experts designated by the Georgia Game and Fish Division to make them more applicable to the Chattahoochee River. The suitability criteria for substrate, developed specifically for this study, are shown in Table 1.

Table 1. The Channel Index Used for the Chattahoochee River Trout Fishery

Channel Index	Suitability	Description
1.0	0.10	all sand, no cover
1.5	0.15	gravel, no cover
2.5	0.25	sand, some cover
3.0	0.30	sand, extensive cover
4.0	0.40	gravel, extensive cover
5.0	0.50	cobble (75-254 mm), some cover
6.0	0.60	boulder (> 254 mm), some cover
7.0	0.70	bedrock, some cover
8.0	0.80	cobble, extensive cover
9.0	0.90	bedrock, extensive cover
10.0	1.00	boulder, extensive cover
11.0	0.50	upland vegetation

Time series analyses of habitat and flow, ordinarily part of an instream flow study, were not performed in this analysis because: a) natural reproduction was not a source of trout recruitment; b) comparisons between peaking flows (existing conditions) and steady flows (project condition) were not possible, and c) because little opportunity was available to modify releases from Buford Dam on a seasonal basis.

RESULTS

For the sake of brevity, the results of this study are presented in weighted useable areas (WUA) for total reach summaries (Figure 4). The summary figures present the mean weighted useable area for a length of river 1000 feet long for each of the three major reaches. This form of presentation allows for comparison of reaches of unequal length. Total habitat for each reach can be calculated as WUA per 1000 feet times a reach multiplier.

Trout Habitat

The general results for trout of all life-stages for the major reaches were remarkably similar (Figure 4). In all cases, habitat for each species peaked at a discharge under 2,000 cfs and then declined to a minimum at the highest simulated discharge of 12,000 cfs.

The four species life-stages investigated in this report could be placed into two groups. The WUA-discharge relationships for adult rainbow trout and adult brown trout were generally similar. The WUA-discharge curve of both peaked at approximately 1,500 cfs and then declined to a minimum at 12,000 cfs. For the second group, brook trout and juvenile brown trout, the WUA-discharge relationships peaked at or under 1,000 cfs and declined to a minimum at 12,000 cfs. In general, the amount of habitat available for adult brook trout was less than that available for either adult brown trout or adult rainbow trout.

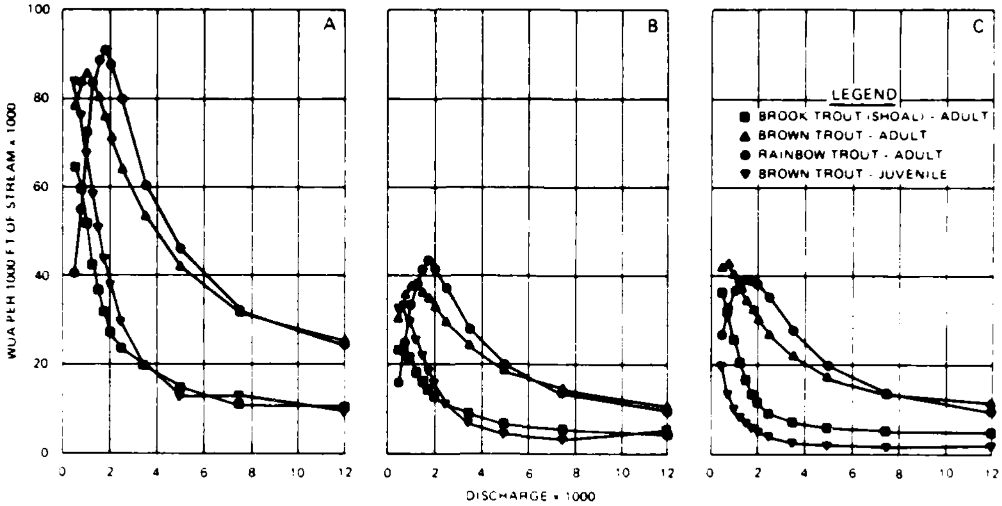

Figure 4. Relationship between WUA (presented as the average weighted useable area for 1000 feet of stream channel) and discharge for trout life stages for - A) Morgan Falls to Peachtree Creek B) site of proposed rereg dam to headwaters of Bull Sluice Lake C) Buford Dam to the site of the proposed rereg dam.

The habitat value of the different major reaches differed significantly. The largest area of habitat for all trout species and life-stages was found below Morgan Falls Dam, primarily because of the steep stream gradient, increased river width, and numerous shoals that occur in this reach. Rocks, boulders, and fractured bedrock in the shoal areas provide abundant cover for trout. Two additional areas of prime habitat, shoals at river mile 328.6 and 319.5, occur within the major reach between the site of the proposed reregulation dam and the headwaters of Bull Sluice Lake.

Adult Brook Trout Habitat

Habitat for adult brook trout in the three major reaches peaked at a discharge near or under 1000 cfs (Figure 4). For the rereg pool reach and below Morgan Falls Dam, habitat dropped substantially below the optimal discharge. Habitat availability as a function of discharge dropped more gradually in the reach of river between the site of the proposed rereg dam and the headwaters of Bull Sluice Lake. Although the habitat for adult brook trout for each major reach peaked at low discharges, there were several noticeable exceptions. Habitat in shoal areas usually peaked at discharges near 1500 cfs.

Adult Rainbow Trout Habitat

Habitat for adult rainbow trout in the three major reaches peaked at a discharge between 1,500 and 2,000 cfs and declined to a minimum at 12,000 cfs (Figure 4). The habitat available in all of the subreaches also follows the same trend. There did not appear to be major differences in the habitat-discharge relationship for this species between shoal and non-shoal areas. The habitat value of the major reach below Morgan Falls Dam was greater than that of the two upstream reaches, primarily because of increased cover and width of the extensive shoals found in this reach.

Adult Brown Trout Habitat

Habitat for adult brown trout followed the same general pattern observed for adult rainbow trout, except that it peaked at a slightly lower discharge, usually around 1000 cfs (Figure 4). Again, the habitat-discharge relationship declined from the peak at 1000 cfs to a minimum at 12,000 cfs. The habitat value of the major reach below Morgan Falls Dam was greater than that of the two upstream reaches, primarily because of the increased cover associated with the extensive shoals found in this reach. There did not appear to be a major difference in the habitat-discharge relationship among the subreaches that comprised the major reaches, although the shoal areas tended to have a more flattened peak.

Juvenile Brown Trout Habitat

The habitat-discharge relationship observed for juvenile brown trout was similar to that observed for brook trout. In general, habitat for each major reach peaked at or near a discharge of 500 cfs. The habitat value (Figure 4) of the major reach below Morgan Falls Dam was greater than that of the other reaches, again, primarily because of increased cover provided by the shoals and increased width of river. Habitat in the shoal areas either peaked at a discharge above 500 cfs but below 1500 cfs or the rate of decline in habitat with increasing discharge was less pronounced than in the total reach summaries.

DISCUSSION

The potential effects of flow alterations in the Chattahoochee River can be broadly classified as fish habitat modifications resulting from changes in depths and velocities; water quality changes caused by construction of the proposed rereg dam (operation of a rereg dam will slow the travel time of water through the system thereby resulting in more warming of the water and increased water temperatures over current conditions); changes in cover in the channel; and changes in the channel itself (i.e. bank sloughing).

Under current operating conditions, trout habitat at any point within the entire study length of the Chattahoochee River varies between optimum and near-optimum at the lower flows (550 to 1050 cfs, depending upon location in the river) to a minimum at the higher discharges (near 10,000 cfs depending upon discharge from Buford Dam and local inflows). Additionally, habitat can vary from a maximum to a minimum several times in a 24 hour period. Thus, fish habitat may be optimal for much of the day and minimal for several hours. Under the proposed revised operating schedule, the minimum in habitat that occurs on a daily basis will be eliminated and the overall flow regime will more nearly approximate the optimum flow required by the four trout life-stages. The benefit to the fishery obtained by this change could not be assessed at the time this study was completed because relative differences in habitat value between a steady flow and a fluctuating flow could not be defensibly quantified; although, from a qualitative standpoint considerable information exists suggesting that daily fluctuating flows are more detrimental to fishes than steady flows. In addition, steady flows in the river will be more conducive to increased primary (algae and aquatic macrophytes) and secondary (aquatic macroinvertebrates) production than the fluctuating flows. Neither aquatic vegetation nor aquatic macroinvertebrates will be subjected to alternate scour and stranding by fluctuating water levels under the revised operation to meet water supply needs. From a fish habitat standpoint, the revised flow in the river obtained by eliminating the peak flows associated with demand for power will be beneficial for the reaches downstream of the site of the proposed rereg dam, assuming that detrimental water quality conditions do not occur.

The fish habitat benefits derived from flow alterations vary somewhat by lifestage. The habitat available for juvenile brown trout and adult brook trout is negligible at discharges above 4,000 cfs. Thus the elimination of the daily peaking flows would be of considerable benefit to the habitat available to these two species. This is particularly true if the success of these two lifestages is limited by the lack of habitat at the high daily discharges. This may, in fact, be the case based on discussions with representatives of the Georgia Game and Fish Division. Brook trout are not stocked in appreciable numbers since this species has not provided the return rate (harvest) of rainbow and brown trout. Juvenile brown trout are stocked in the Chattahoochee River below Morgan Falls Dam. Based on the PHABSIM analysis presented in this report, habitat at a higher flow (1000-1500 cfs) is available at the shoal areas. An upward shift in the current minimum low flow to a more constant release may cause an overall moderate decline in habitat available at the lower discharges since both of these lifestages have optimum habitat at a discharge lower than 1000 cfs in all of the major reaches. However, since the optimum is somewhat higher in some of the subreaches, particularly in the widest shoal areas, some areas of good habitat will be available at discharges above the current minimum flow.

Firm conclusions concerning the effect of flow modification cannot be reached for sections downstream of Morgan Falls, since it is operated independently by Georgia Power Company. If Morgan Falls Dam is operated as a run-of-the-river project, in which discharges equal inflows, then the trout fishery downstream from Morgan Falls Dam will be enhanced in much the same manner as the reach between Bull Sluice Lake and the site of the proposed rereg dam. However, if Morgan Falls Dam is operated in pond-and-generate or in peaking mode, the effects of flow modifications could be considerably different.

If no rereg dam is constructed and flows necessary to meet water requirements are obtained by modifying Buford Dam, then the effects on the trout fishery will be similar to the effects downstream from the site of the proposed rereg dam. However, if a rereg dam is constructed, firm conclusions cannot be made for the reach between Buford Dam and the site of the proposed rereg dam since neither the size, storage-capacity/elevation relationship, nor operation of the rereg dam is currently known.

The general effects of operating a rereg dam on this reach will be determined by how low the water level drops within the pool of the rereg dam. If the pool of the rereg dam falls enough to dewater the shoals in this reach and minimum flows from Buford Dam are stopped, then much of the prime trout habitat in this major reach may be lost. The effects further downstream but within the pool of the rereg dam cannot be estimated since the details of operation are unknown.

Water quality, particularly temperature, is a major concern downstream of Morgan Falls Dam and, in fact, in the summer may be of greater concern than the depths and velocities available for trout habitat, since lethal temperatures can occur at very low flows. Water quality modifications in the Chattahoochee River caused by operation of a rereg dam were addressed in a separate study (Zimmerman and Dortch 1986).

ACKNOWLEDGEMENTS

The author thanks the individuals who assisted in the Chattahoochee River Instream Flow Study. Without their help this paper could not have been completed. Dr. Robert Milhous, Mr. Jay Troxel, and Ms. Janet Fritschen are coauthors of the technical report (Nestler et al. 1986) upon which this paper is based. Messrs. Tim Hess and Chris Martin of the Georgia Game and Fish Division provided valuable assistance in all phases of the study. Mr. Ken Hulick of the U.S. National Park Service provided logistical support for many of the site visits made by the study team. Mr. Tim Hale of the U.S. Geological Survey directed collection of the field data under often trying conditions.

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HEC-5Q: A HANDY TOOL OR MONKEY WRENCH?

Richard E. Punnett, PhD

Hydraulic Engineer, Corps of Engineers, Huntington District, WV

ABSTRACT

A HEC-5Q model was developed and tested for the Kanawha River, West Virginia. The model included three upstream reservoirs. The system was regulated for dissolved oxygen at a point near the mouth of the river. Historically, the regulation method was based upon a temperature flow relationship. The HEC-5Q model was tested during the 1986 summer season as a real-time, operational guide. Data inputs included real-time flow conditions (received via satellite) and the future flow conditions (from HEC-1 forecasting). A comparison of the augmentation requirements as predicted by the two methods was made.

AUTHOR'S NOTE

The written paper was not available at the time of publication. Therefore, the following synopsis was provided in lieu of the full report. However, a detailed report on the model development, calibration, and verification was given in Special Projects Report No. 86-5 by the Hydrologic Engineering Center, Davis, California.

SYNOPSIS

The Kanawha River system has a watershed of 12,300 square miles; the Corps of Engineers, Huntington District, regulates three lakes and three navigation pools within the system for multi-purposes (including low-flow augmentation for dilution). Temperature, dissolved oxygen, and flow were the three most important regulation parameters for determining the flow augmentation requirements of the river. The HEC-5Q model was tested as a real-time tool for predicting the flow augmentation requirements. The inputs for the model included initial conditions and predictions of natural flows (which were available from HEC-1 model outputs) and weather conditions. Because of the bulk of required initial conditions, the uncertainties associated with the forecasted inputs, and the complexity of the system, there was an initial concern that the model usage would be too cumbersome and inaccurate -- hence, a "monkey wrench." However, most of the required inputs were already available in compatible data files and the forecasted values were sufficiently accurate.

Five-day forecasts were generated by the HEC-5Q model. For the 1986 summer period, the model accurately predicted the regulation parameters. Additionally, the model provided a lot of insight into the effects of travel time, weather patterns, changes in flowrates, and changes in oxygen demands. The preliminary evaluation (based on one season) was that the HEC-5Q model was a "handy tool."

LAKE GREESON AND LITTLE MISSOURI
RIVER MODELING STUDIES

D. R. Johnson, Supervisory Environmental Engineer, USAE, Vicksburg District, Vicksubrg, Miss.

ABSTRACT

Lake Greeson, Arkansas, was modeled with the numerical model CE-QUAL-R1. The modeling study evaluated the merit of attaching steel plates to the trash racks in order to create a skimmer weir. The plan was considered as part of the Arkansas Lakes Interim Study as a means to increase the temperature and dissolved oxygen in the releases from Narrows Dam. The numerical model, CE-QUAL-RIV1 was then used to evaluate the effects of warmer releases upon the water quality of the Little Missouri River. The results of the modeling studies were favorable, indicating likely success of the plating proposal to improve water quality in the Little Missouri River.

INTRODUCTION

Since 1971, the Vicksburg District has received considerable communication from the Department of the Interior, U.S. Fish and Wildlife Service (USFWS), the Federal Power Commission (now Federal Energy Regulatory Commission [FERC]), the Arkansas Game and Fish Commission (AGFC), and other agencies concerning the effect of cold hypolimnetic releases upon downstream water quality and fisheries in the Ouachita River Basin in Arkansas. The releases are from three District hydroelectric dams: (1) Narrows Dam on the Little Missouri River; (2) Blakely Mountain Dam on the Ouachita River; and (3) DeGray Dam on the Caddo River. In 1977, Congress authorized the Arkansas Lakes Interim Study (ALIS)--a 5-year basin-wide comprehensive study. The study was to identify what problems existed and to recommend solutions or mitigations to these problems.

Vicksburg District initiated a 4-year in situ water quality monitoring study which utilized 18 recording monitors located upstream and downstream of each reservoir. Analysis of the in situ data confirmed that low temperature and low dissolved oxygen levels were found downstream of the reservoirs. The months of September and October showed the greatest temperature and dissolved oxygen deficits. These problems were most severe downstream of Narrows Dam on the Little Missouri River, where the temperature was as much as 16 C less than above the reservoir and dissolved oxygen levels stayed below the Arkansas state criteria of 5 mg/l for 3 months of the year.

In addition to the in situ monitoring, a fisheries study was conducted. The study concluded that fisheries downstream of all three reservoirs had less species diversity than upstream and that both numbers and percent of game fish were reduced from above the reservoirs. As with the water quality monitoring, the problem was most severe downstream of Narrows Dam.

As part of the ALIS, a variety of techniques were considered to alleviate the cold-low DO discharges from Narrows Dam. Among the techniques considered were: (1) selective withdrawal; (2) submerged weirs; (3) destratification; and, (4) aeration. The proposal with the greatest merit was to plate the lower portion of the trash racks to elevation 520 NGVD. The plating would form a skimmer weir, withdrawing lake water from a zone 30 feet above the existing withdrawal zone. The WES selective withdrawal model, SELECT, was used to determine if the elevated withdrawal zone would be sufficient to increase the temperature to the target temperatures provided by the USFWS. Although the SELECT study showed the USFWS target temperatures could not be met, it was determined that the temperatures would be close enough to warrant a more detailed modeling study.

Reservoir Modeling Study

The Water Quality Section determined that a one-dimensional, vertically segmented, reservoir model would be appropriate for the study. CE-QUAL-R1 and the thermal sub-model CE-THERM-R1 were selected. A one-dimensional model was chosen because conditions in the outflow were of major interest and these are best approximated from conditions near the dam.

Four years were used for the thermal calibration and verification of Therm. Those years were 1974, 1975, 1976, and 1983. These years included 2 "average" years (1974, 1975) and 1 hot, dry year (1976) and a wet year (1983). The three earlier years had been modeled previously with ECOTHERM, and the data decks were modified to the format needed for THERM. The last year, 1983, was the best year in that the most data were available for verification. In 1983, weekly temperature profiles had been taken immediately up-lake of the dam and in situ monitors were located on the inflow and outflow. THERM was calibrated using the 1976 data and verified with the other years. Figure 1 is a plot of the predicted temperature versus depth and the measured temperature versus depth for 1983. In general, THERM predicted temperatures well for all years; however, a large storm on 2 July 1983, created a bulge in the metalimnion during the 1983 simulations. The storm inundated the monitoring station and no temperature data were available for a period of 7 days. During that period, the model had some difficulty placing the water in the appropriate layers. The difficulty is likely due to incorrect water temperature estimates for the missing period.

Statistical analysis of the predicted versus observed temperatures was done with the

Figure 1
Temperature
Simulated versus Observed

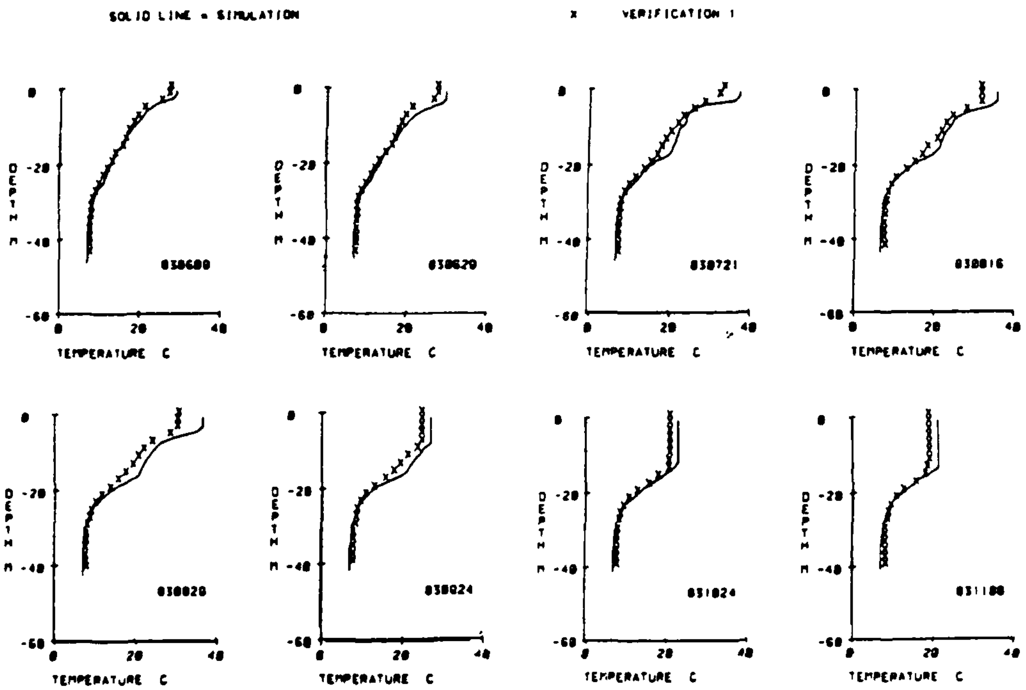


Table 1
Results of CE-Therm-Lake Greeson

Year		1974		1975		1976		1983		Average	
Elevation Withdrawal		493 Ft NGVD	520 Ft NGVD	493 Ft NGVD	520 Ft NGVD	493 Ft NGVD	520 Ft NGVD	493 Ft NGVD	520 Ft NGVD	493 Ft NGVD	520 Ft NGVD
	USFWS Target Temps	C	C	C	C	C	C	C	C	C	C
Jan 15		7.2	7.2	9.5	9.5	6.9	6.9				
Feb 1		8.0	8.7	8.3	8.9	6.3	6.4			7.5	8.0
Mar 1	12	8.3	8.2	8.0	8.0	9.5	10.7			8.6	9.0
Apr 1	18	12.0	13.4	10.6	10.9	12.5	15.0			11.7	13.7
May 1	22	15.3	19.5	12.2	18.4	13.1	17.9			13.5	18.6
Jun 1	26	16.4	23.7	12.7	23.6	14.0	21.5	13.7	18.0	14.2	21.7
15	28	18.6	24.5	19.1	24.6	14.0	32.1	15.2	19.2		
Jul 1	30	24.1	26.0	19.2	27.9	15.5	24.8	15.8	22.8	20.2	25.3
Aug 1		25.9	30.5	15.8	30.0	21.3	29.3	21.8	26.7	21.1	29.1
Sep 1		26.3	29.2	20.4	28.6	23.7	28.5	23.4	30.2	23.4	29.1
Oct 1		21.6	21.8	21.4	21.8	23.5	24.1	24.1	25.9	22.7	23.4
Nov 1		18.9	18.6	19.1	17.6	14.9	13.5	22.2	20.7	18.9	17.6
Dec 1		12.6	12.2	11.5	10.6	8.7	8.3	13.7	12.4	1.63	10.9

reliability index (Leggett and Williams, 1981). A perfect match would give a reliability index (RI) of 1.0. The RI's for the four years simulation were 1974 - 1.13; 1975 - 1.12; 1976 - 1.12; 1983 - 1.13. The inflow temperatures for 1983 were from average daily values. Only occasional measured values were available for other years; therefore, inflow temperatures were estimated based on air temperature and observed water temperature.

After the calibration of the model was completed, the withdrawal port elevation was increased 10 meters for the four model years. Table 1 lists the release temperature predicted for the normal versus elevated withdawal zones. The temperature values listed indicated that even with plating, the USFWS target temperatures would not be met. However, after plating, the temperature would be met after a 1-month delay. It is also important to note that release temperatures from November through March are relatively unchanged. Thus, little impact on the winter put and take trout fishery is anticipated.

The second parameter of interest in the modeling study was dissolved oxygen. Although in-lake profiles for temperature and dissolved oxygen (DO) are available for 10 years, sufficient

Figure 2
Dissolved Oxygen
Simulated versus Observed

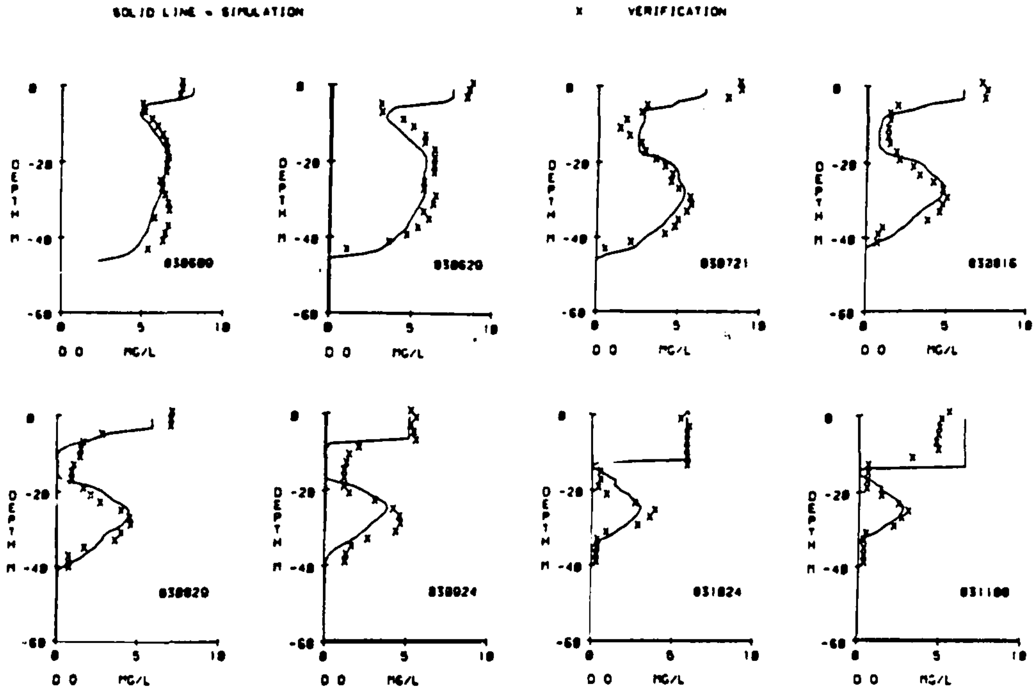
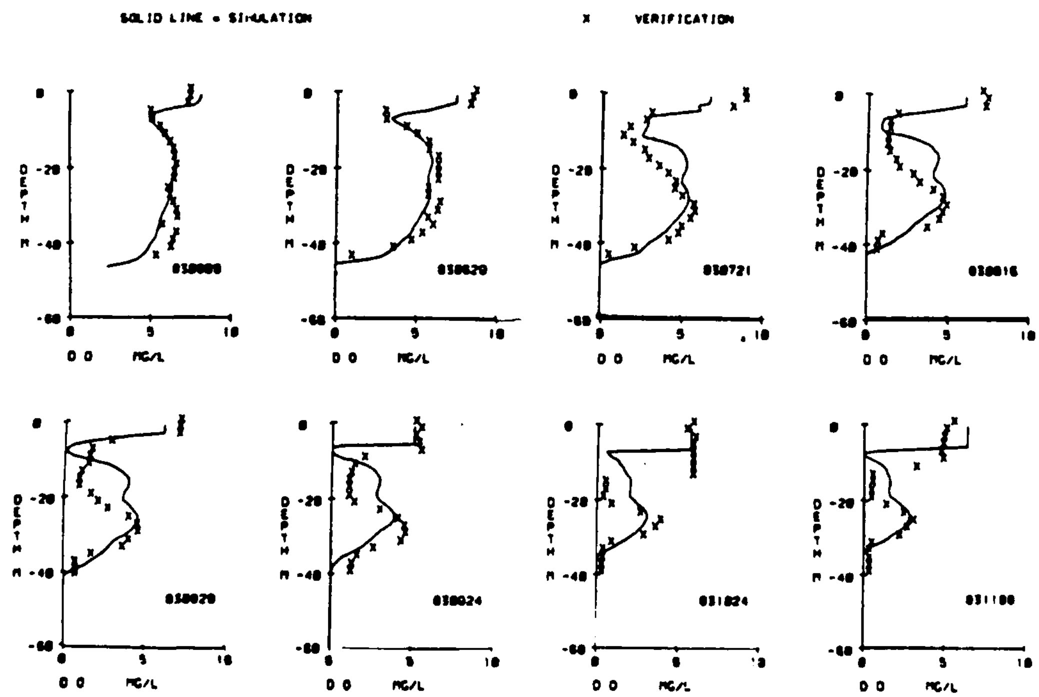


Figure 3
Dissolved Oxygen
Plated Simulation vs Observed



Johnson

chemistry data were only available for 1983. Weekly DO profiles were available for that entire year, while monthly profiles were available for many other parameters. CE-QUAL-R1 was calibrated and verified on DeGray Lake, a Corps reservoir 40 miles east of Lake Greeson. As the reservoirs are similar, the coefficients for the DeGray simulations were used for the Lake Greeson study. Thus the 1983 model runs on Lake Greeson should be considered verification runs instead of calibration runs.

Modeling the DO in Lake Greeson was more complex than modeling the temperature, because more factors were involved. Using the available data plus making estimates for such factors as sediment oxygen demand and percent liable organic carbon from the DeGray study, reasonable predictions of DO levels were made. The RI for DO for 1983 was 1.79. Plots of the simulated versus the observed DO are on Figure 2. The plots indicate that Lake Greeson has a severe metalimnetic oxygen minimum. The metalimnetic zone of oxygen depletion coincided with the elevation of the hydropower penstocks and was the reason for the low DO levels in the releases from the reservoir.

Raising the elevation of the withdrawal zone not only increased the release temperatures, but also improved the DO levels within the reservoir and downstream of the reservoir. The plots of DO versus depth in the reservoir with an elevated withdrawal zone are shown on Figure 3. When the plots in Figures 2 and 3 are compared, it is apparent that the zone of metalimnetic oxygen depletion has been reduced with the elevated withdrawal zone. In addition, the overall rate of hypolimnetic oxygen depletion was reduced with plated withdrawals.

The reservoir modeling study showed that the proposal to plate the trash rakes to elevation 520 NGVD would effectively improve downstream water quality. The impacts to the lake would be a deeper hypolimnion, but one with more DO. A FWS study of fish in DeGray Lake under similar conditions found no negative impacts to fisheries as a result of increased withdrawal elevations.

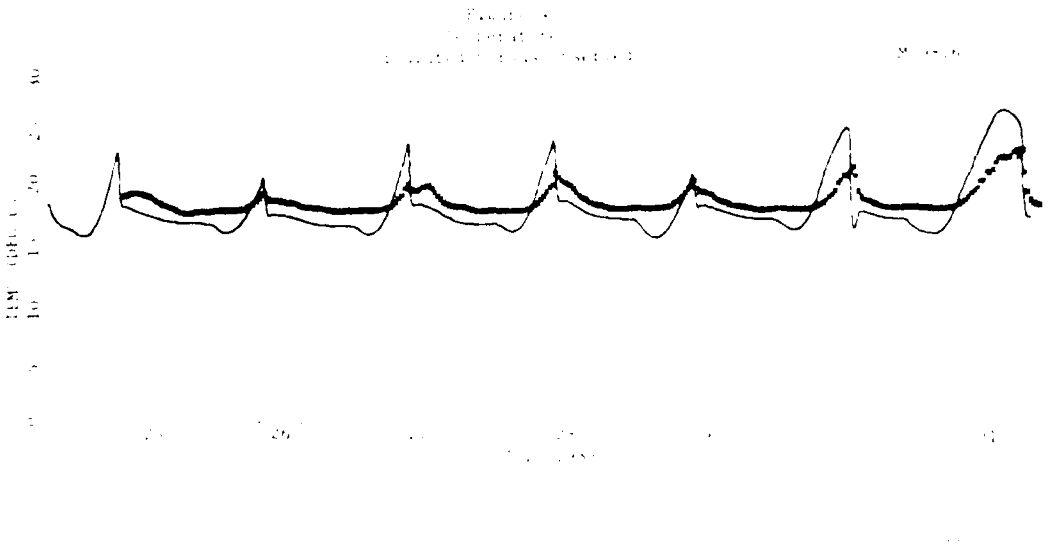
Riverine Modeling Study

After completion of the Lake Greeson Modeling Study, a riverine modeling study of the Little Missouri River (LMR) was initiated. A 12-mile reach of the LMR immediately downstream of Lake Greeson was studied. The objectives of the study were to evaluate the temperature and dissolved oxygen conditions downstream of Lake Greeson under a variety of release conditions and to consider the use of control structures to augment low flows. Because Narrows Dam is a peaking hydropower facility, a dynamic riverine model was necessary to simulate hydraulic conditions in the LMR. CE-QUAL-RIV1, a one-dimensional, longitudinally segmented model, was selected.

Two weeks in 1983 were selected for input in the modeling study. The first, in late July, was characterized by long periods of hydropower generation with low temperatures, and moderately low DO. The second week selected was in early September and was characterized by short generation periods, warmer temperatures and very low DO. The second week represented a worst-case situation with regard to DO levels.

CE-QUAL-RIVI is composed of two separate codes—hydraulic and water quality. Although CE-QUAL-RIV-I is a dynamic model, the hdyraulic code was not able to handle the peaking hydropower flows well. The instantaneous jump in flow from 15 cfs to 2000 cfs was too much for the code. Most runs were made using 100 cfs as the low flow and stepping the discharge up to the peak flow over ten, 5 minute time periods. The effect of this adjustment on the downstream hydrograph was small.

The biological code handled ten constituents: temperature, carbonaceous biochemical oxygen demand, organic nitrogen, ammoniacal nitrogen, nitrate nitrogen, phosphate, dissolved oxygen, iron, manganese and coliform bacteria. As with the reservoir modeling, temperature and dissolved oxygen were of primary interest. Comparisons of the simulated versus the observed temperature at node 15, river mile 98.6, are plotted in Figure 4. In the July simulation, the model predicted a greater response to the water during periods of no generation, but during generation, predicted a smaller increase in temperature than was observed. In the September simulation (Figure 4) the model again predicted a greater response to the weather during periods of no generation than was observed; however, model response during generation was good.



The calibration of CE-QUAL-RIVI for dissolved oxygen was not as straight forward as for temperature. RIV-I had two stream reaeration equations which could be selected. The first was a generalized equation suggested by Rathbun and Bennett. The second was the equation developed by Tsivoglou-Wallace. Both equations were tried. Rathbun-Bennett provided too little reaeration during generation and too much during not-generation (Figure 5). Tsivoglou's formula provided too much reaeration all the time, but when the constant was reduced from .0537 to .016, the reaeration was acceptable (Figure 6).

apparent anomaly was likely the result of higher saturation values in the colder waters of the existing condition releases.

CONCLUSIONS

Corps of Engineers-developed numerical water quality models CE-QUAL-RI and CE-QUAL-RI were found to give good results and were reasonably easy to use. The major difficulty in using both models was obtaining all the necessary data to accurately run the models. Both studies were conducted using existing data, which were at times

Figure 5
Bennett-Rathbun Reaeration
versus Observed Data

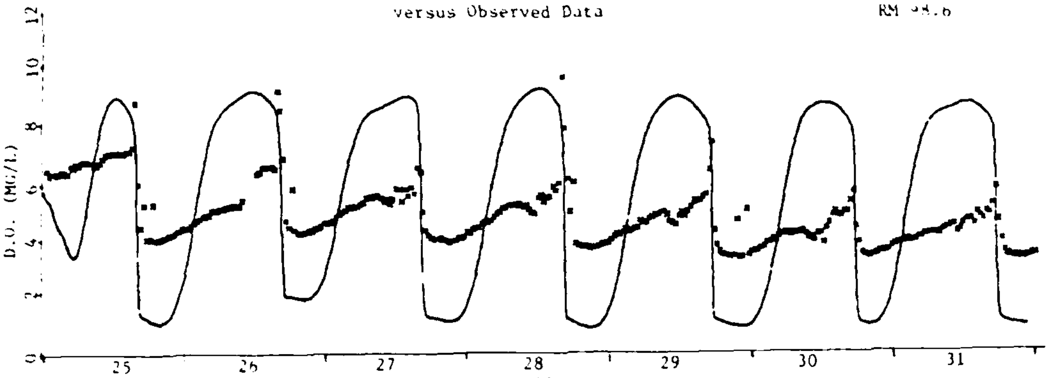
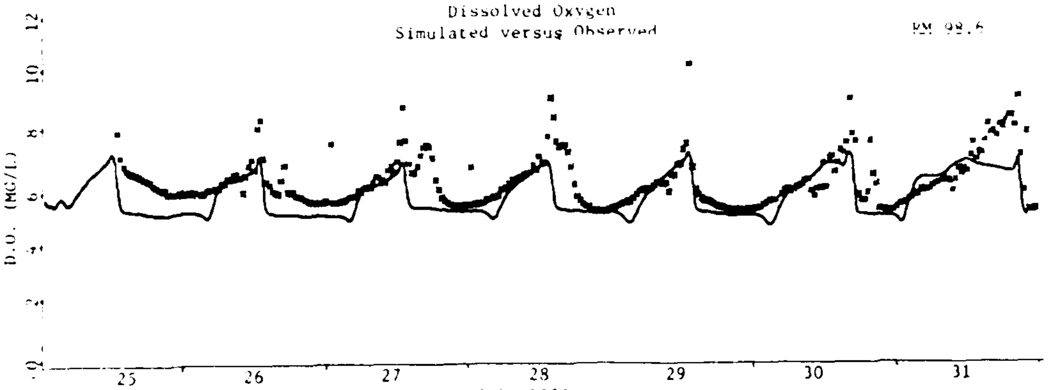


Figure 6
Dissolved Oxygen
Simulated versus Observed



Once the model was calibrated, riverine conditions between normal and plated releases were compared. Release water quality with plated releases was derived from the CE-QUAL-RI output from the appropriate week of the 1983 model run. Comparison of the temperature and dissolved oxygen of the two conditions are plotted on Figures 7 and 8. RIV-I predicted the temperatures would range from 20° C to 31° C. As with the calibration simulations, daily water temperature variations were likely exaggerated. Observed water temperatures displayed a maximum of 5° C daily range and averaged a 3°C daily range. Dissolved oxygen levels from plated releases were substantially increased during generation but were somewhat less during non-generation. This

outdated or incomplete. These are constraints that most Districts must occasionally handle. The favorable results of the studies indicate no major impacts to the reservoir as a result of plating the trash racks and that the water quality in the Little Missouri River would be measurably improved.

ACKNOWLEDGEMENTS

The Water Quality Modeling Group, Waterways Experiment Station, provided assistance in model selection and technical subjects in support of these modeling studies. Dr. J. Wolsinski and D. Hamlin provided assistance with CE-QUAL-RI and M. Dortch and Dr. J. Martin provided assistance with CE-QUAL-RIV1.

Figure 7
Temperature
Normal versus Plated Releases RM 98.6

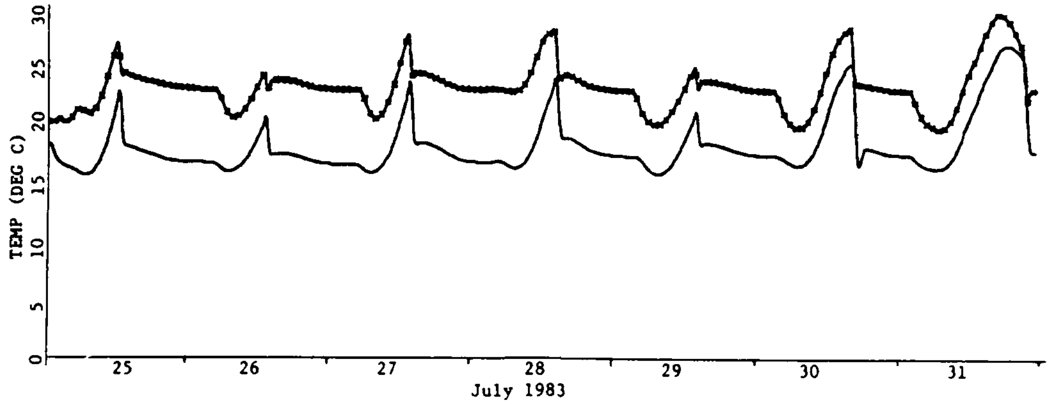
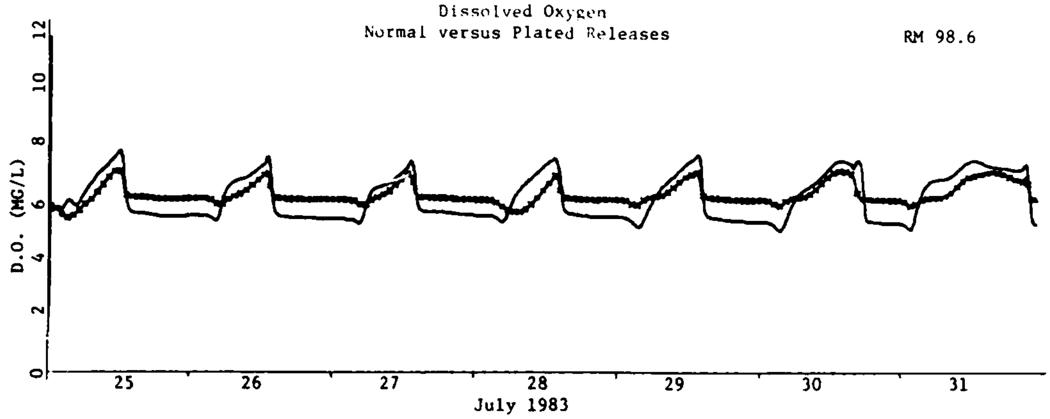


Figure 8
Dissolved Oxygen
Normal versus Plated Releases RM 98.6



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Johnson

HOWARD A. HANSON RESERVOIR, WASHINGTON, TEMPERATURE ANALYSIS MATHEMATICAL MODEL INVESTIGATION

by Michael L. Schneider[1] and Richard E. Price[2]

[1]Research Hydraulic Engineer, USAE Waterways Experiment Station
[2]Research Physical Scientist, USAE Waterways Experiment Station

ABSTRACT

The US Army Engineer District, Seattle (NPS), is presently evaluating proposed additional water storage at Howard A. Hanson Reservoir in Washington State. This proposed project will involve raising the existing pool approximately 12 m (40 ft). This investigation examined the impacts of raising the conservation pool on the reservoir thermal profiles and release temperatures for several study years. The mathematical model used in this study examined impacts of raising the pool with and without structural modification to the existing outlet works. Optimization procedures were used with the mathematical model to provide optimum number and elevations for the additional ports. Results indicate significant improvement in release temperatures could be achieved with a multilevel outlet structure.

INTRODUCTION

Purpose and Scope of Study

The Howard A. Hanson project was authorized by Congress on 17 May 1950 to provide standard flood protection and minimum flow requirements for fishery enhancement in the Green River. The US Army Engineer District, Seattle (NPS), is presently evaluating a proposed additional water storage modification to the project involving the raising of the maximum conservation pool by 12 m (40 ft). This study was conducted to investigate the thermal characteristics of the existing reservoir and releases and to assess changes in these characteristics resulting from the proposed storage reallocation.

Project Description

Howard A. Hanson Dam is located 105 km (65 miles) upstream from the mouth of the Green River and 56 km (35 miles) east of the city of Tacoma in western Washington, as shown in Figure 1. The project drains 572 km^2 (221 square miles) of protected watershed in the Cascade Mountains. The earth- and rock-fill dam reaches a height of 71 m (235 ft) above the streambed. The tainter gate controlled spillway is located in the right abutment of the dam with a maximum discharge capacity of 3,030 m^3/sec (107,000 ft^3/sec) at maximum project pool (el 371 m (1,220 NGVD)). Normal releases are passed through a 6.3-m (19-ft) horseshoe-shaped sluiceway controlled by regulating tainter gates located at the bottom of the pool. The sluiceway releases about 634 m^3/sec (22,000 ft^3/sec) at maximum project pool. Low-flow releases are made through a 1.22-m (48-in.) bypass intake located about 12 m (40 ft) above the bottom of the pool.

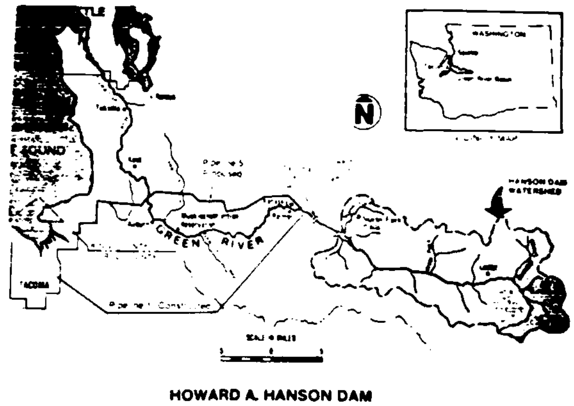

Figure 1. Location of Howard A. Hanson Reservoir

This outlet has a capacity of about 14.2 m^3/sec (500 ft^3/sec) at maximum conservation pool (el 348 m (1,141 NGVD)). The existing outlet tower is shown in Figure 2.

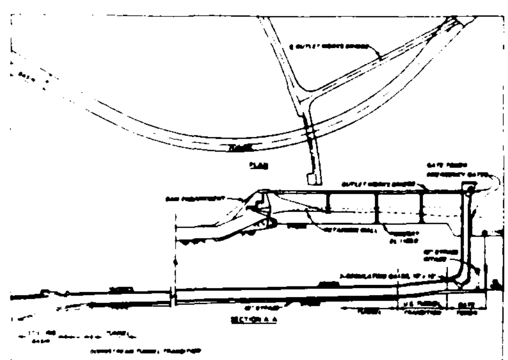

Figure 2. Howard A. Hanson outlet works

The reservoir operation rule curve is designed to prevent flooding downstream in the winter months and to augment low flows during the summer and fall for fishery enhancement. The reservoir is maintained at a depth of about 9 m (30 ft) during the nonconservation period of the year from 1 October through 31 March except during unusually wet conditions. The average yearly rainfall in the drainage basin is 2.3 m (89 in.) with 75 percent of the precipitation occurring during this nonconservation season. Runoff hydrographs are characterized by frequent short-duration, sharply peaked events during the winter months followed by longer duration, smaller peaked hydrographs associated with snowmelt.

Beginning 1 April the reservoir begins filling to a maximum conservation period depth of 30.5 m (1141 NGVD), as shown in Figure 3, with water of as low a turbidity as possible.

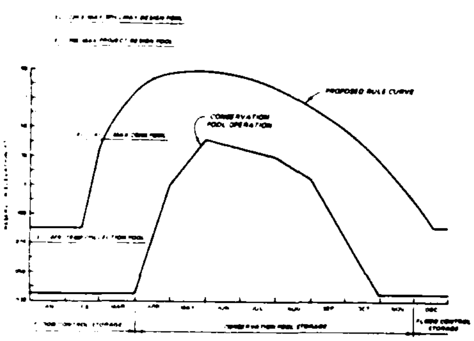

Figure 3. Existing and proposed rule curve

Inflowing water impounded by Howard A. Hanson Reservoir is of good quality with low concentrations of dissolved minerals and nutrients. Turbidity levels in the tailwaters are the only parameter known to exceed water quality standards.

The reservoir impounds 9.8 km^3 (25,650 acre-ft) at full conservation pool with a surface area of 0.9 million m^2 (732 acres). During the summer-fall low-flow period, the pool is generally drawn down from minimum release requirements consisting of 3.1 m^3/sec (110 ft^3/sec) for fishery enhancement and 3.2 m^3/sec (113 ft^3/sec) for water supply for the city of Tacoma. The city of Tacoma operates a concrete water supply diversion dam approximately 5.6 km (3.5 miles) downstream of the project. No other treatment except chlorination is required of this water supply. However, during high-flow events, water in the Green River is too turbid for usage by the city without dilution. The impoundment also becomes thermally stratified in the late spring and early summer once a pool is established. Low-level releases provide for downstream temperatures slightly cooler than those which occur naturally in the Green River. During a portion of the year, this is beneficial for most of the downstream fisheries.

The Green River is one of Washington State's primary producers of salmon and steelhead. The completion of Howard A. Hanson Dam in 1961 precluded the migration of fish upstream of the project, but downstream passage is possible through the existing sluiceway. There is a strong commitment by both State and Federal resource agencies to preserve and enhance the anadromous fishery resources in the Green River Basin. The water resources demands in western Washington, however, are changing. The city of Tacoma has requested additional storage in Howard A. Hanson Reservoir for the purpose of water supply while the State of Washington would like to further augment Green River flows during the summer and late fall to enhance the fishery in the lower Green and Duwamish Rivers. These additional water supply demands would involve raising the maximum conservation pool 12.2 m (40 ft) and impounding almost three times the existing conservation storage. Over 2.02 km^2 (500 acres) of land would additionally be inundated regularly by this proposed change. Specific concerns about raising the pool center on the impacts to the reservoir thermal stratification. With a deeper maximum conservation pool, a stronger stratification is possible. This stratification may alter release temperatures significantly and ultimately impact the steelhead salmon habitat downstream in the Green River. If this appears likely, a selective withdrawal structure may be needed to provide adequate control of release temperatures along with an operational plan to minimize depletion of desired thermal resources in the reservoir and control release temperature fluctuations. In-reservoir impacts may also be significant since upstream fisheries are being developed.

Approach

The approach taken in the investigation of proposed storage modifications to Howard A. Hanson Reservoir involved the application of a mathematical thermal reservoir model entitled WESTEX (Holland 1982). The model was adjusted to historical data from 1982 and verified through data collected during 1979 and 1983. The impact of changing the storage allocation in the reservoir subject to the existing outlet tower was investigated by comparison of predicted release temperatures to release temperature objectives. Release temperature objectives providing an optimal environment for the varied downstream fisheries were specified by NPS to equal 14.4° C (58° F) throughout the year. Addition of selective withdrawal capability to the existing outlet was simulated to predict impacts on in-reservoir thermal profiles and release temperatures. Although other water quality characteristics may be affected by changes to the operating schedule, the influence on temperature was of primary concern in this study.

MATHEMATICAL METHODOLOGY

The downstream release and in-lake temperature characteristics for Howard A. Hanson Reservoir were modeled using a one-dimensional thermal simulation model. The model WESTEX used in this investigation was developed at the US Army Engineer Waterways Experiment Station (WES) based on results of Clay and Fruh (1970), Edinger and Geyer (1965), Dake and Harleman (1966), and Bohan and Grace (1973). The WESTEX model can be used for examining the balance of thermal energy imposed on a reservoir. This one-dimensional model includes computational methods for predicting dynamic changes in thermal content of a body of water through simulation of heat transfer at the air-water interface, heat advection due to inflows and outflows, and internal dispersion of thermal energy. The reservoir is conceptualized as consisting of a series of homogeneous layers stacked vertically. The time history of thermal energy in each layer is determined through solving for conservation of mass and energy at each time increment subject to an equation of state regarding density. The boundary conditions at the water surface, inflow, and outflow regions are required to conduct these simulations. The computational procedure for the withdrawal zone allows prediction of release temperature. Mathematical optimization routines have been added to this model enabling the systematic evaluation of optimal outlet configurations subject to specified release water quality objectives. A more detailed discussion of the WESTEX model may be found in Holland (1982).

Thermal Model Inputs

The WESTEX model required input data concerning the physical, meteorological, and hydrologic characteristics of Howard A. Hanson Reservoir. Hydrologic input included daily values of reservoir inflow and outflow volume and inflow temperature. The meteorological data (air temperature, cloud cover, relative humidity, and wind velocity) were used to compute surface heat exchange at the air-water interface. Physical characteristics included the stage-storage relationship of the reservoir and the rating curves for the outlet structure.

Study Years

The years studied in this investigation were determined in consultation with NPS and were based on the inflow during the conservation period. Historical events of varying return periods were modeled to study reservoir thermal properties under a wide range of hydrometeorological conditions. The calendar year 1979 was chosen as representative of a low-flow year, 1972 as a high flow year, and 1982 as an average-flow year. The years 1983 and 1985 were added as additional study years because of available field data. Simulations were run January through December, although the primary period of concern was during the conservative period after spring filling.

Meteorological and Hydrological Data

Meteorological data required by the model are daily average values for wet and dry bulb temperatures, wind speed, and cloud cover. These data were available from the US Air Force Environmental Technical Applications Center at Scott Air Force Base (USAF-ETAC), Illinois, for the Tacoma, Washington, Airport weather station, which is the nearest meteorological station to the Howard A. Hanson Dam. Meteorological data received from the USAF-ETAC were averaged on a daily basis. Equilibrium temperatures, surface heat exchange coefficients, and daily average solar radiation quantities for the years of study were computed using the HEATEX program (Heat Exchange Program 722-F5-E1010).

Hydrologic data provided by NPS consisted of daily discharge from the project and pool level fluctuations. The average daily inflow was computed from these data. The existing operating schedule for the project as well as the proposed rule curve were also provided.

Temperature Data

Historic inflow temperatures on the Green River were available for the years 1970, 1973, and 1985, at Humphreys, Washington. Since inflow temperatures for the years of study were not available, a multiple regression technique was used to develop a statistical model for inflow temperature. The dependent variable, water temperature, was expressed as a function of air temperature and inflow rate.

Daily release temperature data from the dam were monitored at the city of Tacoma's water supply intake located approximately 5.6 km (3.5 miles) downstream from the project. These data proved to be an unreliable measure of release temperatures from the project through a comparison with tailrace temperatures collected in the summer of 1985. Therefore, with the exception of 1985, release temperature data were not used in measuring the performance of the numerical model.

Physical Characteristics

The properties of the existing outlet at Howard A. Hanson Reservoir were required data for simulation of historical events. The port elevations, dimensions, and rating curves were obtained from project records. Operating conditions for given historic events were also required during model adjustment and verification. A third order polynomial was fitted to the rating curve of the existing water quality port to calculate the capacity of a port at a given submergence. This calculation was required because of the significant pool fluctuations which occur throughout the year.

The area-capacity curve for Howard A. Hanson Reservoir as furnished by NPS indicated the lake storage curve is typical of mountainous terrain. Only 7 percent of the storage at maximum conservation pool occurred at or below the elevation of the low-flow outlet, indicating little storage in the lower levels of the reservoir.

MODEL ADJUSTMENT

The WESTEX model requires determination of coefficients characterizing certain reservoir processes. The hydrodynamic processes representing entrainment of inflows and internal mixing resulting from circulation within the reservoir are approximated through the application of mixing coefficients. Other coefficients influence the distribution of thermal energy absorbed into the pool. These model coefficients were modified until modeled conditions approached field observations for the year 1982. Temperature profiles were predicted for days in which observed prototype data were available.

The 1982 simulation was initiated on 1 January with an initial uniform temperature of 4° C (39.2° F). Conditions during the non-conservation period of the year were nearly isothermal with an average depth of only 9.1 m (30 ft). Storm events during this period resulted in significant fluctuation in the pool level as indicated in Figure 4. Storage was quickly released after these events to provide additional flood control benefits. Generally, spring filling began on the receding side of the snowmelt hydrograph to minimize total lake turbidity. Once the pool was established, thermal stratification developed rapidly as indicated by the nearly vertical temperature contours during June and July shown in Figure 4. The maximum stratification occurred during the early summer when cool inflows were still available. As the summer progressed, surface temperatures ranged up to 20° C (68° F). Release water temperatures were significantly cooler than objective temperatures in the spring and early summer. Release water temperatures increased linearly from 6° C (42.8° F) at the beginning of April to 14° C (57.2° F) by the end of June. Temperatures exceeding 14° C (57.2° F) were released throughout most of the summer and

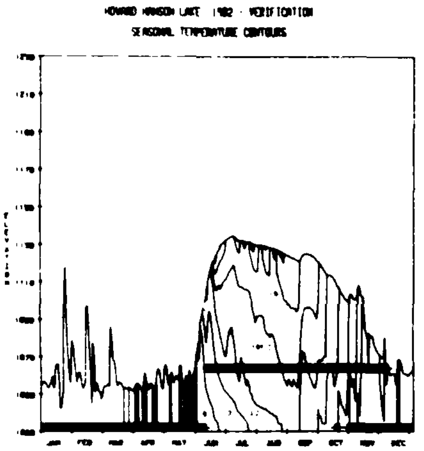

Figure 4. Modeled temperature and water-surface fluctuation for 1982

early fall. At the beginning of September, lake overturn began with the remaining stratification quickly dissipating. The low-flow outlet was operated until the pool is drawn down near the end of November.

The predicted versus measured temperature profiles for the adjustment of the model for 1982 are shown in Figure 5. Modeled and observed

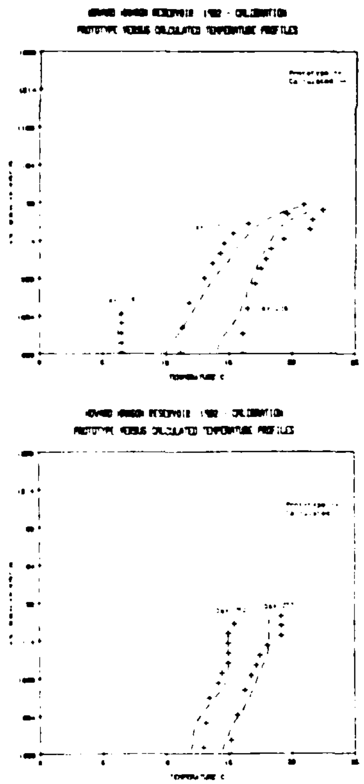

Figure 5. Prototype versus calculated temperature profiles for 1982

results were generally within 1° C (33.8° F) throughout the simulation period. The modeled results were slightly warmer than the observed results on day 175 (24 June) because of the synthesized inflow temperatures. The mixed layer depth was modeled a little deeper on day 277 (4 October). This could be a result of using wind speeds from the Tacoma airport that were not indicative of those at the project.

The temperature releases from the project are highly variable during the nonconservation portion of the year and reflect the rapidly changing weather conditions. Spring filling initiates the stabilization of release temperatures with a near-linear increase in project release temperatures. Only large hydrologic events significantly influence release temperature during this period. The releases are dominated primarily by the meteorological warming of the reservoir. Since the releases come from the hypolimnion, tailwater temperatures are consistently cooler than the naturally occurring stream temperatures during the spring and early summer. Later in the summer, release temperature exceeds objective temperature releases for a duration of several months.

Model Verification

The mathematical model was verified through comparison of predicted and observed thermal conditions for the years 1983 and 1979 using model coefficients determined during the adjustment phase. The degree of agreement between observed and simulated thermal properties was similar to the results for 1982. Early in the year, prior to stratification, measured and simulated temperature profiles deviated as much as 8° C (46.4° F) because the reservoir processes were advectively dominated and inflow temperatures were synthesized. By day 175, which is well into the pooled conservation period, the predicted and observed temperature profiles deviated by no more than 1° C (33.8° F). This degree of agreement between modeled and observed temperature characteristics remained throughout the stratified period and into the fall overturn. Because observed release temperature data were unreliable, verification could not be made to predicted releases.

RESULTS

Proposed Storage Reallocation

The proposed operational changes enabling additional storage at Howard A. Hanson Reservoir were incorporated into the release schedule for the study years subject to the existing outlet configuration. The proposed rule curve specifies spring storage to begin the first of March and to reach a maximum conservation pool of about 12.2 m (40 ft) higher than existing conditions. The minimum low-flow release from the project is proposed to almost double to 11.7 m^3/sec (412 ft^3/sec; 212 ft^3/sec for Tacoma water supply and 200 ft^3/sec for low-flow augmentation for fisheries enhancement). The proposed rule curve is shown in Figure 3.

The effects of raising the pool are most significant during the spring and early summer. The day-to-day release temperature fluctuations are not as prevalent under the proposed conditions because of the insulation properties of the deeper pool. Release temperatures during most of the spring are several degrees cooler than the existing conditions and hence several degrees further removed from the objective temperature. The maximum release temperature during the summer is also lower than existing conditions. In the fall, the effects of raising the pool will slightly warm (less than 1° C (1.8° F)) releases from Howard A. Hanson Reservoir because of the larger heat content in the raised pool.

The comparison of raised pool predictions with observed thermal profiles indicates that the raised pool condition begins to retain more thermal energy due to the increased surface area. The surface and bottom temperatures are not radically altered by the storage modification, but the volume of the hypolimnion and epilimnion is greatly changed. The existing outlet maintains hypolimnetic releases, thereby preventing the conservation of cooler water resources. The breakup of stratification in the fall is also different from the existing conditions in that the deeper pool retains more thermal energy and hence has a warmer profile. Isothermal conditions are also reached at a later date with the raised pool.

The comparison of objective release temperatures with predicted release temperature for the raised pool scenario indicated most late winter and early spring releases were below the objective by up to 5.0° C (9.0° F). The maximum release temperature will be reduced up to 1° C (1.8° F) during the summer, but the duration of temperatures exceeding the objective will be extended into the fall by several weeks.

Impacts of Selective Withdrawal

The existing project releases allow for establishment of a certain quality environment downstream from the dam. If the raised pool is put into effect, changes would be expected in the downstream environment as it seeks a new equilibrium in response to the modified release temperatures. The Green River downstream of Howard A. Hanson Dam supports a rich anadromous fishery resource. If the anticipated response of the downstream environment to modified release temperatures is unacceptable to resource managers, then several alternatives are available to minimize these impacts. One alternative is the incorporation of a multilevel selective withdrawal system to allow release of water to meet specified objectives. The chief advantage of a multilevel selective withdrawal system is flexibility in meeting release water quality objectives over a wide range of operating conditions.

The need for a multilevel outlet to compensate for pool raising on Cowanesque Lake was investigated by Holland (1982). Although his study involved reallocation of flood storage, additional intakes were needed to meet existing release temperatures. Similar conclusions were reached by Dortch (1981) in his investigation of Kinzua Dam in Pennsylvania and by Peters (1978) in his report of modifications to Flaming Gorge Dam in Utah. In the investigation of release water quality from Sutton Dam (George, Dortch, and Tate 1980), a riser was designed to improve water quality releases.

The addition of a port or ports higher in the pool at Howard A. Hanson Reservoir should allow releases to better meet release temperature objectives downstream. The existing outlet works could be modified to allow releases from the epilimnion and still allow the existing sluiceway releases to operate independently. It was assumed that the flow control for low flow outlets would remain the same. The capacity of this system is then defined by the rating curve of the 1.22-m (48-in.) bypass. Hypolimnetic releases could also be maintained through the existing sluiceway by throttling the control gates or through the addition of an independent low-level outlet. This design allows independent operation of two levels of release and does not consider the potential of single system blending.

Optimization of Outlet Structure Design

To arrive at an efficient outlet structure design for the raised pool, the number and location of additional intakes needed to meet release temperature objectives must be determined. The design of the outlet structure is greatly simplified through the coupling of mathematical water quality models like WESTEX to mathematical optimization techniques. This combination enables the consideration of numerous hydrologic, hydraulic, meteorological, biological, chemical, and operation conditions in the formulation of tower design. Prior to the implementation of such optimization techniques, selective withdrawal intake configurations were based on judgment and experience of the design engineer. Optimized outlet configurations may involve fewer ports, as compared to traditionally accepted designs, to meet a given downstream temperature objective, thereby reducing both operational complexity and the costs associated with design, construction, and maintenance. Additionally, the use of optimization techniques should further enhance tower design by allowing systematic evaluation of the flexibility needed in the design for multiple or anticipated quality objectives.

The purpose of the mathematical optimization procedure is to systematically screen numerous outlet tower designs in terms of performance in meeting a specified release water quality criterion. The goal of releasing water with a temperature of 14.4° C (57.9° F) was expressed earlier. This objective was modified slightly to represent the cyclical nature of available thermal resources. The objective temperature was defined as the naturally occurring Green River stream temperature as defined by a sine function up to a maximum temperature of 14.4° C (57.9° F). Employing a cyclical objective temperature avoids temporal biasing of optimization results characteristic of constant release temperature objectives.

A satisfactory measure of system performance must also be specified if optimum outlet configurations are to be determined. The objective function is a mathematical reflection of how well one possible decision (i.e., number and location of outlets) meets a given set of objectives. The objective function chosen in this study was the sum of squares of deviations between predicted release and target temperatures during the conservation period. Minimization of the objective function yields the optimal location of additional ports for release temperature control. This form of objective function was chosen since its minimization tends to produce outlet configurations which reduce the magnitude of objective deviations experienced. The formula chosen to represent the objective function is project dependent and may include mathematical representations of such factors as state and Federal water quality regulations, temporal weighting of deviations, or numerous water quality constituents

(Dortch and Holland 1984). Since the addition of another intake to the system will generally result in reducing the objective function value, the incremental benefits of project releases must be weighted against the additional costs of this port. Often, however, the incorporation of additional selective withdrawal ports is dictated by the need to meet legislated quality standards rather than the desire to improve on existing good quality releases.

Results of Single Port Addition

Stream temperatures are generally warmer than reservoir releases in the spring and cooler in the fall. For releases from Howard A. Hanson Reservoir to match more closely the naturally occurring stream temperatures, an outlet located in the epilimnion is required. This outlet will enable warmer releases earlier in the spring, thereby saving cooler water for release later in the fall. However, there are several drawbacks associated with a single outlet located in the upper epilimnetic region of the reservoir. Pool level fluctuations dictated by the proposed rule curve may render an upper level port inactive over a large portion of the year. The subsequent transition from upper to lower release may result in significant day-to-day fluctuations in temperature releases.

During the conservation period in the late spring and early summer months, there is a rapid warming of the surface waters at Howard A. Hanson Reservoir resulting from the longer retention time associated with reservoir inflows as well as greater solar energy. To achieve warmer spring releases, an outlet located in the epilimnion is required. Results from the outlet design simulations for the normal and wet years indicate that this port should be located at an elevation about 36.6 m (120 ft) above the reservoir bottom. This elevation corresponded to the minimum objective function value computed by the optimizer for the host of intake elevations considered. The region bounding the optimal port location exhibited almost no change in the objective function values or thermal release characteristics. The significant reduction of the objective function value for each year compared to that computed based on the existing system indicates the inadequacy of the present outlet configuration in meeting release target temperatures subject to the proposed rule curve.

As midsummer approaches, releases strictly from the upper level are warmer than the target release temperatures. Lower-level releases are required at this point to withdraw water selectively at the objective temperature as shown in Figure 6. This type of operating condition is continued throughout the summer with increasing rates of hypolimnetic releases to meet release temperature criteria. Daily gate changes theoretically would result in meeting the daily release temperature objective throughout this period. Lower level releases in turn deplete colder water reserves. By the time the upper port is no longer submerged in mid-August, the hypolimnion has reached a maximum temperature of about 12° C (53.6° F). At this point, the bottom level release becomes the sole outlet with the flexibility of selective withdrawal eliminated for the remainder of the year. Temperature releases will deviate from objective release temperatures during this period.

Design simulations for the dry year (1979) yielded an optimal port elevation slightly lower than the other years modeled. The objective function values for this year were significantly higher because of the larger heat content in the reservoir and the depletion of colder water

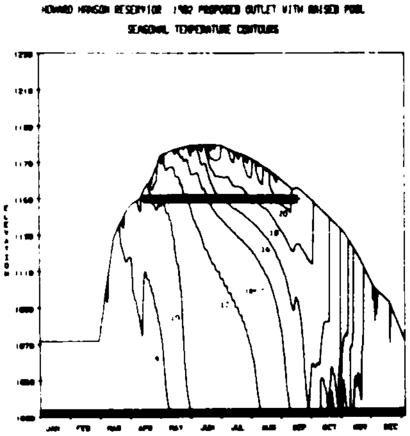

Figure 6. Modeled temperature and water-surface fluctuations for proposed outlet and rule curve

reserves earlier in the summer which resulted in large deviations of release from objective temperatures in the fall. Locating the upper level port lower in the pool caused the hypolimnetic waters to remain cooler, thereby reducing deviation in the late summer when lower level releases are required. The objective function values are relatively insensitive to port location within a wide band of elevations.

Two Additional Ports

The incorporation of two additional levels of ports to the outlet tower has the potential for extending selective withdrawal capabilities for longer periods in the fall. The upper level port could be used to withdraw surface water in the spring and early summer, and an intermediate level port could be employed when surface water is above release temperature objectives and during the fall when the upper port is out of the pool.

Design simulations with two additional ports indicated ports located at elevations 30.5 m (100 ft) and 39.6 m (130 ft) above the bottom, respectively, resulted in the optimal release temperature characteristics. The improvement in the objective function value with the existing ports, one additional port, and two additional ports is shown in Figure 7. A significant improvement in meeting objective release temperatures resulted from adding one additional port, but very little improvement resulted from two additional ports. The second port does not significantly improve releases because very little stratification exists when the pool drops below the upper port, thus minimizing any benefits associated with retaining selective withdrawal capabilities.

CONCLUSIONS AND RECOMMENDATIONS

Modifying the operating policy at Howard A. Hanson Reservoir by initiating spring storage earlier and increasing the maximum conservation pool will impact the project releases and in-reservoir thermal characteristics. The increased

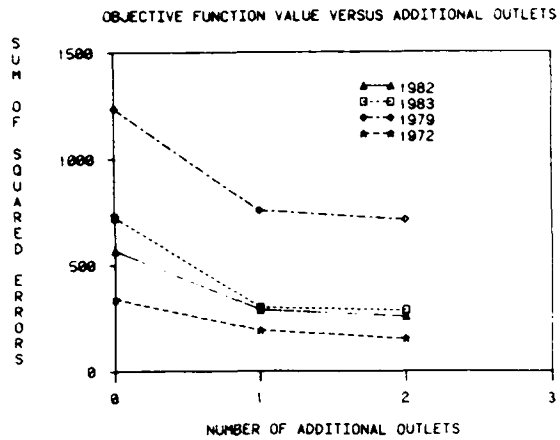

Figure 7. Objective function value versus number of ports

volume and surface area of the reservoir result in a significant increase in total heat content of the reservoir. The deeper conservation pool provides for larger temperature gradients to develop from top to bottom, resulting in stronger stratification and increased water column stability. Release temperatures from the raised pool through the existing outlet works will be cooler during the spring and summer months but slightly warmer during the late summer and fall. The maximum release temperature would be reduced if the proposed storage reallocation is implemented. The existing outlet configuration has little flexibility in altering the release water quality characteristics from the project. During low-flow years, late summer and fall release temperatures may significantly exceed downstream temperature objectives. The addition of selective withdrawal capability through epilimnetic releases provides a means of effectively managing the thermal resources in the reservoir. The location of an additional port in the epilimnion allows warmer surface water to be released earlier in the spring while conserving cooler water resources. This cooler water can be blended with surface water later in the summer to meet downstream release target temperatures. For certain low-flow years, Howard A. Hanson Reservoir will be resource limited. For these events it is critical to manage the available thermal resources to minimize the damage to the downstream environment. Dynamic optimization procedures when used in conjunction with the numerical reservoir model can also provide operational guidance for mitigating the damage caused by release temperatures deviating from project objectives.

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Water Quality Monitoring Using Satellite Data Transmission

Steven D. Hiebert

Biologist, U.S. Bureau of Reclamation, Engineering and Research Center, Denver, Colorado

ABSTRACT

Instruments to monitor and transmit total dissolved gas, dissolved oxygen, water temperature, pH, conductivity, and oxidation-reduction potential (ORP) data have been installed at two sites below Yellowtail Afterbay dam on the Bighorn River, Montana. Data collected with these instruments are being used as part of a long-term study of the effects of nitrogen supersaturation on fish and aquatic macroinvertebrates. Water quality measuring instruments are interfaced with a satellite transmittal system so that data gathered are transmitted and readily available for review and analysis using a receiving station computer. Power supply and instrument circuit modifications have been made to ensure the compatability of the systems components. These electronic modifications were the result of field experimentation and recommendations by instrument manufacturers. The correlation of continually monitored dissolved gases and other water quality parameters with dam operations will contribute information to be used in considering modifications in operations which would enhance conditions for aquatic fauna downstream of the dam.

INTRODUCTION

The Bureau of Reclamation and the Fish and Wildlife Service entered into an interagency agreement to study nitrogen supersaturation problems affecting trout below Yellowtail Afterbay Dam on the Bighorn River in south central Montana. The high concentrations of nitrogen gas in the river cause gas bubble disease in the rainbow and brown trout. Gas bubble disease (GBD) in fish is a pathological process due to excess gas (usually nitrogen) taken in through the gills and forcing its way out through the tissues when reaching equilibrium with atmospheric pressure. The nearest corresponding process in humans is known as decompression sickness or the bends.

Releases from the afterbay dam have been documented since 1973 as the source of the supersaturation in the Bighorn River (Bureau of Reclamation, 1973). The afterbay dam is approximately 3.5 km (2.2 miles) downstream of the 160 m (525 ft) high Yellowtail Dam and Powerplant. The afterbay dam is a concrete diversion structure 415 m (1360 ft) in length and 22 m (75 ft) in height and functions to provide uniform daily flow in the Bighorn River by buffering the peaking power generation flows from the Yellowtail Powerplant. The afterbay dam includes structures such as spillways, a sluiceway, and a canal diversion headworks (figure 1). The spillway has an ogee crest controlled by radial gates. The sluiceway consists of three bays and is controlled by three 3.1 by 2.4 m (10 by 8 ft) slide gates (Young, 1982). Supersaturation conditions result when entrained air in water released from the sluiceway gates plunges to depths in the stilling basin pool. Usually supersaturated dissolved gases contained in water can be dissipated in a short period of time through natural turbulence in the river; however, the river stretch below Yellowtail Afterbay Dam is tranquil, which interferes with this dissipation process and the flow remains supersaturated for several kilometers downstream.

To meet the objectives of the nitrogen supersaturation study, ambient water condition data including flow, hydrology, and water quality needed to be monitored in order to correlate this information with gas supersaturation and the subsequent effects on the fish and invertebrates. Gas tensionometers that measure total gas pressure, water temperature, and dissolved oxygen pressure plus multiparameter water quality instruments that measure dissolved oxygen concentrations, pH, water temperature, conductivity, and ORP were selected to monitor <u>in situ</u> water quality at each river site.

Satellite telemetry was identified as the best method to monitor and record the data from the water quality instruments. Data transmitted at regular intervals in real time to a computer data base eliminated the delays and costs associated with strip chart and internal instrument memory data recording and storage. The satellite-telemetered data is stored directly to a computer with no delays caused by retrieval of a strip chart or instrument memory, and monitoring of events and analysis can be performed in the office. This paper presents information and recommendations on satellite telemetry of water quality data from instruments used as part of the gas supersaturation study conducted by various Federal agencies on the Bighorn River, Montana.

DATA TRANSMISSION SYSTEM

The satellite data collection system used for this study was one in use by the Bureau of Reclamation for transmission of stream stageand reservoir elevations within the Missouri Basin Region. The Bighorn River river stage site and two water quality sites were added to the existing system for this study. One water quality site is located 0.2 km below the afterbay along

Figure 1. - Yellowtail Afterbay Dam

[illegible] and the other site is located [illegible] downstream from the afterbay.

At each site on the river, a data collection [illegible] was set up along with control [illegible] the water quality instruments enclosed [illegible] The DCP module consists [illegible] low-powered [illegible] a precision clock, a sensor [illegible] digital, analog [illegible] memory [illegible]

[illegible]

with a cable-attached underwater probe. The underwater portion of the tensionometer consists of 3 sensors: an electronic pressure transducer connected to a wound 4-meter length of gas permeable tubing that measures the total gas pressure, a [illegible] electrode simultaneously [illegible] oxygen [illegible] measures the [illegible] dissolved oxygen [illegible] and a thermistor that measures the water temperature.

[illegible]

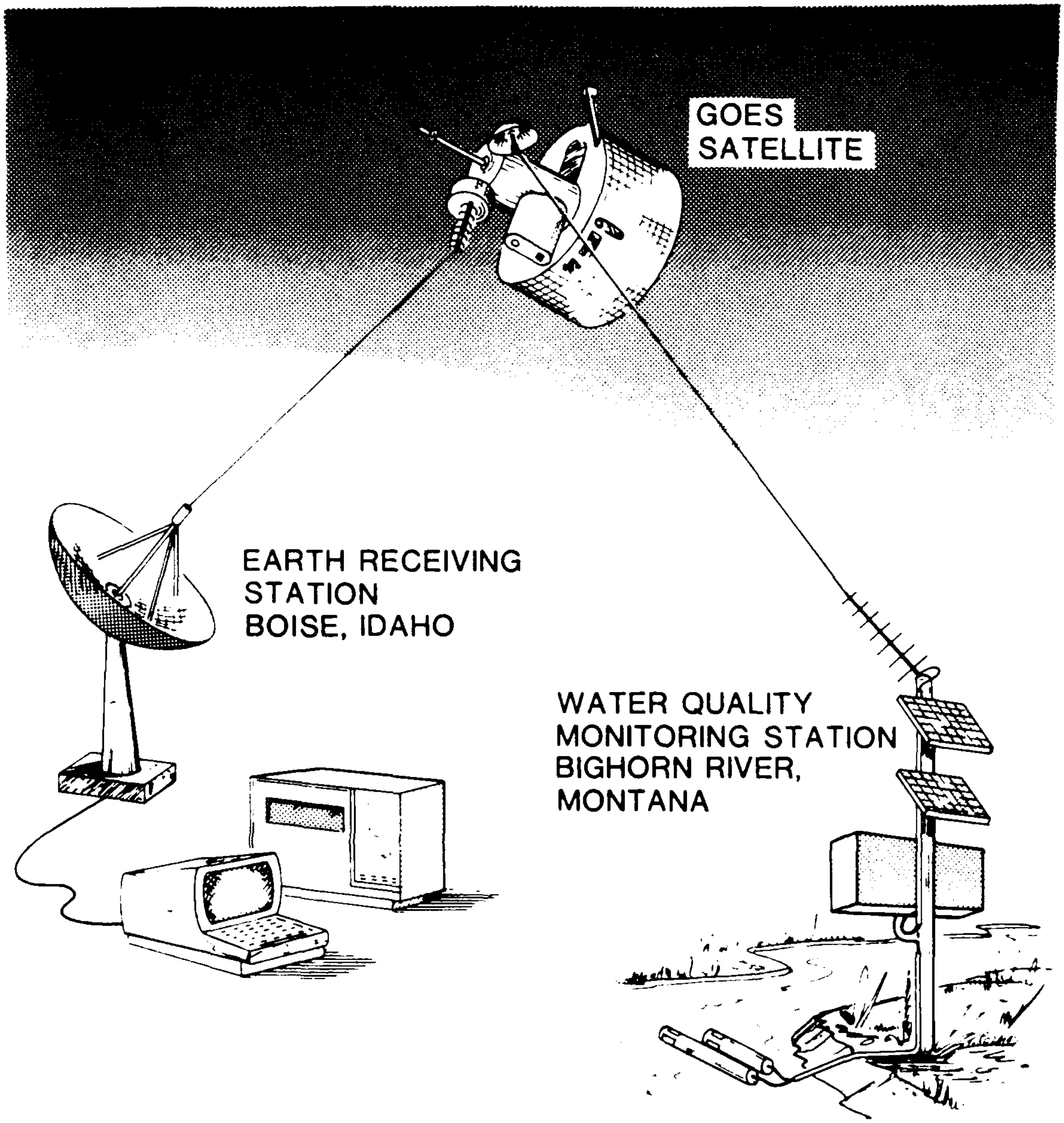

Figure 2. - Diagram of Satellite Data Transmission System

ments varied between instruments and between parameters. Initially calibration was performed weekly for most parameters. Currently up to 3 weeks is sufficient between calibrations for many parameters. As discussed later in the report, the ability to monitor the parameters in real-time helps establish a realistic calibration and maintenance schedule.

The DCP was initially programmed in the field by a portable computer to collect water quality data every 30 minutes after a 2.5 minute minute warmup time. This data is stored in the DCP for 4 hours before being transmitted to the GOES. All DCP stored data is sent in about a 20-second transmission and returned to the earth station in Boise, Idaho, about 20 seconds later (Figure 2). The earth station computer stores the data which can be accessed over a phone line from a field terminal. Once the DCP has been programmed with parameter codes and scales and tested to operate in the required time slot, no further programming is normally necessary.

Modifications of water quality instrument electronics and power supplies were necessary following difficulties in interfacing to the DCP. Solutions to the problems included diagnosis with a voltmeter, monitoring of satellite signal information, and consultation with electronic technicians.

Initial difficulties with the system setup were experienced in the power supply interface with the multiparameter probe. All instruments at each site were initially powered by one 110 ampere hour 12 V battery with photovoltaic panel recharge. Through the dissolved gas pressure and multiparameter water quality transmitters, a ground loop was created that fed back voltage and damaged components in one of the control units. This problem was corrected by isolating the multiparameter probe with a separate 33-ampere hour battery and photovoltaic panel.

Problems with the interface of the multiparameter water quality probe and the DCP were caused by the output signal matching of the two systems. The DCP could accept digital, analog, and switch closing inputs. There was also programmable scaling on the analog inputs. The multiparameter water quality units analog output scale were different on each parameter measured, none of which was based on the DCP default scale of 0 to 5 volts. The DCP had errors in handling these scales along with the fluctuating voltages from some parameters. To correct these problems, the water quality monitoring equipment manufacture was asked to change the electronics to produce a filtered output with a linear, analog scale corresponding to the output readout.

Excessive power consumption (above specified), low light intensity, and cold diminished the battery and recharge system over the winter. These problems were solved, following load tests on the battery, by the addition of a larger capacity battery.

Impedence differences between the DCP and the multiparameter probe caused a small voltage offset during collection which is corrected with the earth station computer programming. This voltage offset from the field has been diagnosed from each parameter and then programmed to be added to the incoming data by the receiving computer. This problem is still being worked on with the intent being no voltage offset and consequent computer correction.

All of the previously discussed problems were discovered in the field and most were primarily caused by non standard (non DCP manufacturer product) instruments being installed to the DCP. The manufacturers of all the electronic equipment were helpful and informative in solving these interfacing problems. The most efficient way to solve these problems would have been for the manufacturers representatives to get together at the sites or to be able to connect and bench test this equipment before field installation.

MONITORING AND PRELIMINARY RESULTS

All data transmitted is stored at the central receiving site and is available for analysis at any time. Data can be obtained from the field seconds after a transmission. With each transmission, a system status is sent that includes battery voltage, signal strengths, and transmission errors. This information is helpful in diagnosing field equipment or satellite problems.

The data can be retrieved from the computer in two basic formats, short term and long term. The short term format provides every parameter value at every collection. These data can be retrieved by site, by day in any time frame. Short term format is kept on hard disc resident memory for 6 months and then archived to tape. The long term format provides a daily average read out of each parameter. This information can be retrieved in 15-day increments. Long term data is stored on resident memory for immediate access at any time.

An initial value of the satellite transmission and recording of data has been to monitor instrument performance over time and detect malfunctions and calibration drifts. Data from this has indicated that the multiparameter probe should be serviced about every 10 days to clear algae and accumulated aquatic macrophytes from the probes. The gas tensionometer needed a less intensive maintenance schedule; however, on few occasions was observed measuring hydrostatic pressure rather than dissolved gas pressure after several weeks when moisture accumulated behind the gas permeable membrane (silastic tubing). Continuous monitoring and comparison of the two different type dissolved oxygen probes tentatively indicate that the three electrode, oxygen producing-reducing probe is more accurate for long-term monitoring in that it requires less maintenance and fewer and smaller calibration adjustments as opposed to the two electrode polarographic dissolved oxygen sensor on the multiparameter probe. The value of knowing immediate instrument malfunctions is obvious in the course of gathering reliable, accurate data for extended periods of time.

In preliminary analysis of several parameters and dam operations some trends are shown in dissolved gas pressure in the river. As found in previous investigations, and confirmed with the continuous monitoring instrumentation, the regulation of the sluiceway gates on the afterbay dam positively correlates with total gas measurements taken immediately below the afterbay at the 0.2 km site. A negative correlation of total gas pressure was observed with the operation of the radial gates (White et al., 1986). Consequently, a mixed flow or release consisting of a portion of the water passing through the sluiceway and a portion passing across the spillway produces less supersaturation than the same total volume through the sluiceway. Unfortunately, this operation [illegible] supersaturation constrains fluctuation [illegible] the afterbay and therefore potential peak [illegible] generation by Yellowtail Dam. In [illegible] tests, where flows through the afterbay [illegible] were reduced and then incrementally [illegible] as mentioned, there was a [illegible] of increasing total gas with [illegible] a leveling off of [illegible] above approximately [illegible] no apparent increase [illegible] continued [illegible] be a function [illegible] variables [illegible]

By parameters [illegible] mid-July, the [illegible] pressures have been between May [illegible] the peak in the early week [illegible] May [illegible] off. The discharge [illegible] with the spring runoff starting in mid-June. The conductivity in the river increased to the highest level in early May. The pH has remained relatively constant. Dissolved oxygen has remained constant at the upstream site below the afterbay and varied at the 4.5 km site in daily fluctuations due to photosynthetic production. The dissolved oxygen makes up a greater portion of the total gas in moving downstream. At The 4.5 km site, the oxygen saturation exceeded 138 percent during sunny afternoons whereas at the site 0.2 km below the afterbay oxygen did not exceed 114 percent saturation.

CONCLUSIONS

The technology of satellite telemetery is a valuable tool in monitoring water quality among other parameters. The collection of data in real time and direct computer storage along with time and monetary savings in not having to retrieve field tape or instrument memory has proven to be useful in data aquisition for this study. The set up of a satellite data collection system requires some preliminary identification of necessary electronic equipment with specifications of accuracy, service and compatability with the satellite system. The electronic equipment manufacturers should be informed of exact application of their equipment so changes can be made before field trials.

Continuous monitoring of water quality is vital to this study as collection of field data during diel periods as well as annual cycles is necessary to accurately determine effects of changes in the environment. In addition, because of power dispatch office and schedule changes with several generator units in the Yellowtail Dam being off line for portions of the year, afte bay levels and discharge may not have been typical thus far. In monitoring and being able to identify non-typical years, much more information on a long term basis is necessary.

Previous studies have been performed and models have been used to identify problems in an attempt to eliminate or reduce the air entrained and consequent supersaturation by the afterbay sluiceway. These studies elucidated other problems arising with installation, costs, and operation of structures or ineffective reductions of supersaturation (Denson and Loomis, 1985). This research with monitoring equipment will provide valuable information to be used in further modification or operation of the dam. In addition, the investigations have been structured so that the results will have maximum application to other locations where gas supersaturation problems exist but the various habitat parameters cannot be easily sampled.

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DISSOLVED OXYGEN STUDIES BELOW WALTER F. GEORGE DAM

Diane I. Findley, PhD[1] and Kenneth Day[2]

[1]Ecologist, US Army Corps of Engineers, Mobile District
[2]Park Manager, US Army Corps of Engineers, Mobile District

ABSTRACT

A special study of conditions within and downstream from Walter F. George Lake (AL-GA) was conducted from 13-19 August 1986, in response to numerous fish kill incidents that have occurred in the tailrace. The study focused on the suspected cause of the problem, low dissolved oxygen, and on an evaluation of possible corrective measures, such as spillway gate releases and generation startup procedures. Results indicated that extremely low dissolved oxygen water apparently leaked into the tailrace and moved up through the water column and subsequently downstream. A compressed turbine startup sequence proved to be the most beneficial startup method. Releases through the spillway gates during nongeneration periods offered the most feasible results in preventing or alleviating downstream depressed dissolved oxygen conditions.

INTRODUCTION

Project Location and Description

The Walter F. George (WFG) Lake is formed along the Chattahoochee River by the WFG Lock and Dam (L&D) which is located near Fort Gaines, Georgia. The impounding structure consists of a concrete dam, a fourteen-gate spillway, and single lift lock. The WFG Lock is the second highest lift east of the Mississippi River. The WFG Powerhouse with four generating units is located on the opposite bank from the lock. The lake is 121 km (75 mi) from the mouth of the river, has a surface area of 13 292 ha (45 181 ac), and is 23.3 m (93.0 ft) deep at the deepest point. Normal pool elevation is 57.9 m (190.0 ft) National Geodetic Vertical Datum (NGVD). Authorized primarily for navigation and hydroelectric power generation, associated purposes include flood control, streamflow regulation to provide a nine-foot navigation channel, outdoor recreation, and fish and wildlife conservation. The WFG Powerhouse is operated as a peaking facility. Characteristically, hydroelectric power is generated 3 to 6 hours daily, Monday through Friday.

Approximately 47 river km (29 river mi) downstream of WFG L&D is the George W. Andrews (GWA) L&D. This structure consists of a concrete dam with a fixed spillway and a single lift lock. The GWA L&D is a single purpose navigation project intended only to provide sufficient depth for authorized navigation. The lake created by GWA L&D has a surface area of 623 ha (1 540 ac).

History Of The Problem

Construction of both the WFG and the GWA projects was essentially complete in 1963. Problems with fish kills in the WFG tailrace area during periods of stratification soon followed. A number of interim measures have been implemented to combat the problems. The first of these measures, Standard Operating Procedures (SOP), was developed in 1970. Basic provisions of this initial effort were to provide for water releases through spillway gates when fish were observed distressed or dying, to monitor conditions for indications of causal effects, and to verify that corrective actions were effective.

In 1972, an automatic water quality monitor was installed downstream of WFG Lock and Dam. This monitor, which is presently located about 457 m (1 500 ft) below the dam on the west bank of the river, monitors dissolved oxygen (DO), pH, temperature, and conductivity.

The same basic provisions of the 1970 SOP were incorporated into a subsequent SOP revision which was implemented in 1982. On 30 July 1985, a major fish kill occurred affecting approximately 100,000 fish as estimated by State of Georgia fisheries personnel. It was this incident which resulted in increased monitoring of tailrace conditions and culminated in this study. Through this increased monitoring, several trends were observed in the tailrace area. When water levels dropped following generation, fish were trapped in a drainage channel which paralleled the lock wall. The fish would subsequently die and be swept downstream during the next generation period. A similar situation occurred in the stilling basin which parallels the face of the WFG spillway. This shallow area has a short vertical wall along the downstream edge which serves to trap fish as water levels drop. Another situation noted was fish entrainment during the startup of generation. On several occasions small numbers of dead fish were observed below the WFG Dam shortly after the initiation of generation. Laboratory examination of specimens revealed that the fish had ruptured gas bladders and gas bubbles trapped in the fins and gills which indicated a rapid movement from depths to the surface.

Observations during the spring revealed that fish, primarily shad, which are anadromous in nature, tended to enter the lock in large numbers through the downstream valves. When these fish remained inside the chamber for extended periods, the DO was depleted and the fish died. By far the most common situation which also produces the highest incidence of mortality, is the occurrence of extremely low DO throughout the tailrace area. This situation occurs frequently during the spring and summer, generally in the early morning hours between 2400 hours and 0700 hours, and over the weekend.

As a result of the 1985 fish kill, another SOP was implemented later that year which contained the basic provision of releasing water from the spillway gates once conditions deteriorated. There were, however, some provisions of note in this procedure. The SOP was prepared recognizing that although the problem had existed for several years, little was actually known about the cause. In an attempt to better understand the problem, the 1985 SOP required significantly increased monitoring requirements. Notable among these was the direct transmittal of the monitor's DO information to a digital display in the WFG Powerhouse and the inclusion of measures to provide for spillway releases prior to visible fish distress or fish kill.

Some of the provisions for increased monitoring contained in the 1985 SOP were revised as more was learned about the situation and to accommodate the need to reduce the overwhelming burden it placed on field personnel. The last revision, dated June 1985, will remain in effect with some modifications until early 1987. At that time, the SOP will be modified to reflect recommendations developed from this study.

METHODS

This concentrated investigation was conducted from 13-19 August 1986, to better describe the effects of operational procedures such as generation startup sequences, spillway releases and lock discharges on DO and temperature in the immediate tailrace and at locations farther downstream. It was anticipated that data from the study would identify operational regimes which when combined with the naturally occurring processes would achieve the highest possible DO levels.

A temperature and DO profile was measured at midstream and at the right and left quarter points of the channel, except as indicated below, at each of the following stations. The depth intervals for each profile ranged from 0.2 m (0.5 ft) to 1.0 m (3.0 ft) at all stations, except Station 1. At Station 1 in WFG Lake, the data were collected at 1.5 m (5.0 ft) or 3.0 m (10.0 ft) intervals through the 28.3 m (93.0 ft) depth of the lake. Pool elevation during the study averaged 56.2 m (184.5 ft) NGVD.

Station 1 - 244 m (800 ft) upstream of WFG L&D (single point vertical profile)

Station 1A - 8 m (25 ft) downstream of WFG L&D at powerhouse draft tube exit area (single point vertical profile)

Station 1B - 122 m (400 ft) downstream of WFG L&D (single point vertical profile)

Station 2 - 244 m (800 ft) downstream of WFG L&D

Station 3 - 457 m (1 500 ft) downstream of WFG L&D at water quality monitor

Station 3A - 579 m (1 900 ft) downstream of WFG L&D (single point vertical profile)

Station 4 - 2.4 km (1.5 mi) downstream of WFG L&D

Station 5 - 26.6 km (16.5 mi) downstream of WFG L&D

Station 6 - 42.3 km (26.3 mi) downstream of WFG L&D

Stations 1, 2, 3, 4, 5, and 6 were sampled until 1400 hours 15 August. With the understanding acquired from these initial tests, Stations 4, 5, and 6 were relocated closer to the dam (Stations 1A, 1B, and 3A) for the remainder of the study.

Measurements were taken using Yellow Springs Instruments (YSI) DO meters. Air calibration of the meters was verified with the azide-modified Winkler titration technique. Temperature readings were verified with mercury thermometers. In addition to the YSI meters, a Hydrolab water quality monitor was used at selected locations to measure temperature, DO, pH, conductivity, and oxidation-reduction potential. Generally, data at each station were collected for at least one hour prior to operational changes, such as generation startup, and continued until the transient hydraulic effects of the event had dissipated. Data were collected at 20-minute intervals for all the tests, except the 24-hour test period on 15-16 August when the frequency for collection varied depending on project operations but provided as a minimum one profile per hour. Station 1 profiles were collected generally once per hour throughout the study.

DISCUSSION OF RESULTS

Data from Station 1 were typical for large stratified bodies of water (Wetzel, 1975). During the study, surface readings varied from 6.0 to 8.0 mg/l, middepth readings ranged from 3.0 to 4.0 mg/l and to below 1.0 mg/l at 15.2 m (50.0 ft) and continued to drop until readings were virtually 0 mg/l from 21.3 m (70.0 ft) to 28.3 m (93.0 ft), the lake bottom. Figure 1 illustrates the effects at middepth at Station 1, and surface changes at Stations 2 and 3, when each of the three functional turbines were brought on-line in one-hour intervals. (Turbine 4 was nonfunctional due to servicing.) The DO level at middepth of Station 1, a point approximately 6.7 m (22.0 ft) above the turbine intake area, showed an appreciable decrease after the second turbine was brought on-line. DO levels continued to fall until the units were turned off at 1700 hours. DO soon recovered to pregeneration concentrations at Station 1.

Releases from spillway gate 12, which originate from a depth of 9.1 m (30.0 ft) in the lake, had no effect on the middepth DO concentration at Station 1. However, as shown in Figure 1, the surface DO readings at Stations 2 and 3 displayed immediate increases that continued even after the gate was closed. It should be noted that the initial increases in tailwater DO from extremely low concentrations achieved with a gate opening of

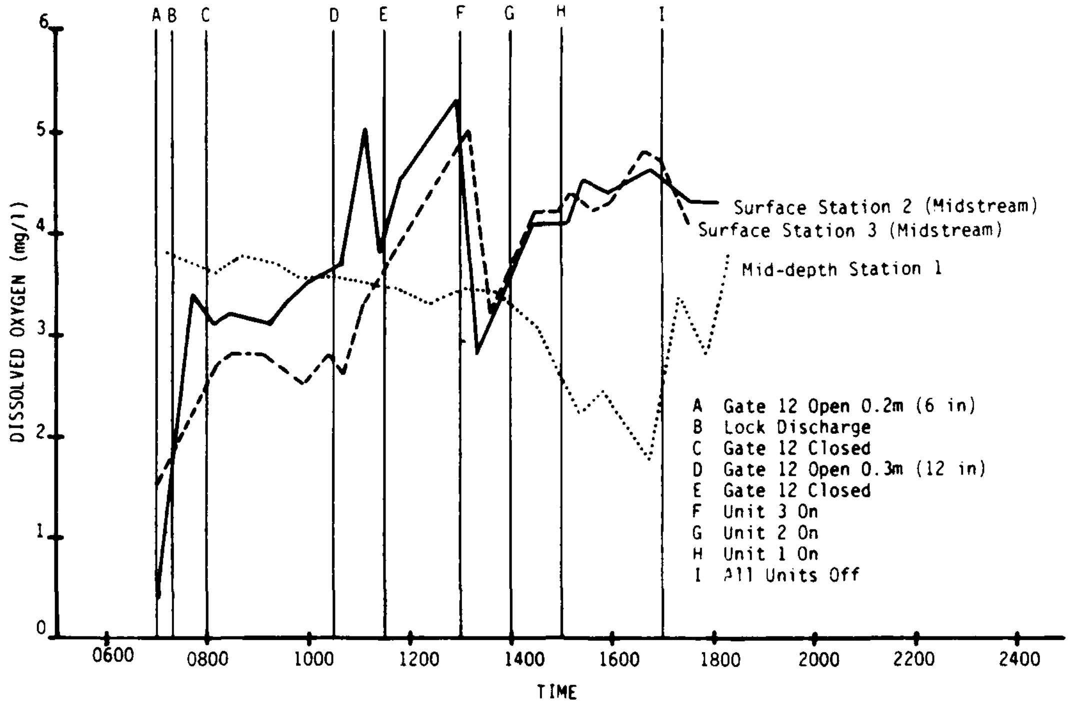

Figure 1. Dissolved oxygen concentrations at stations 1, 2, and 3 on 14 August 1986

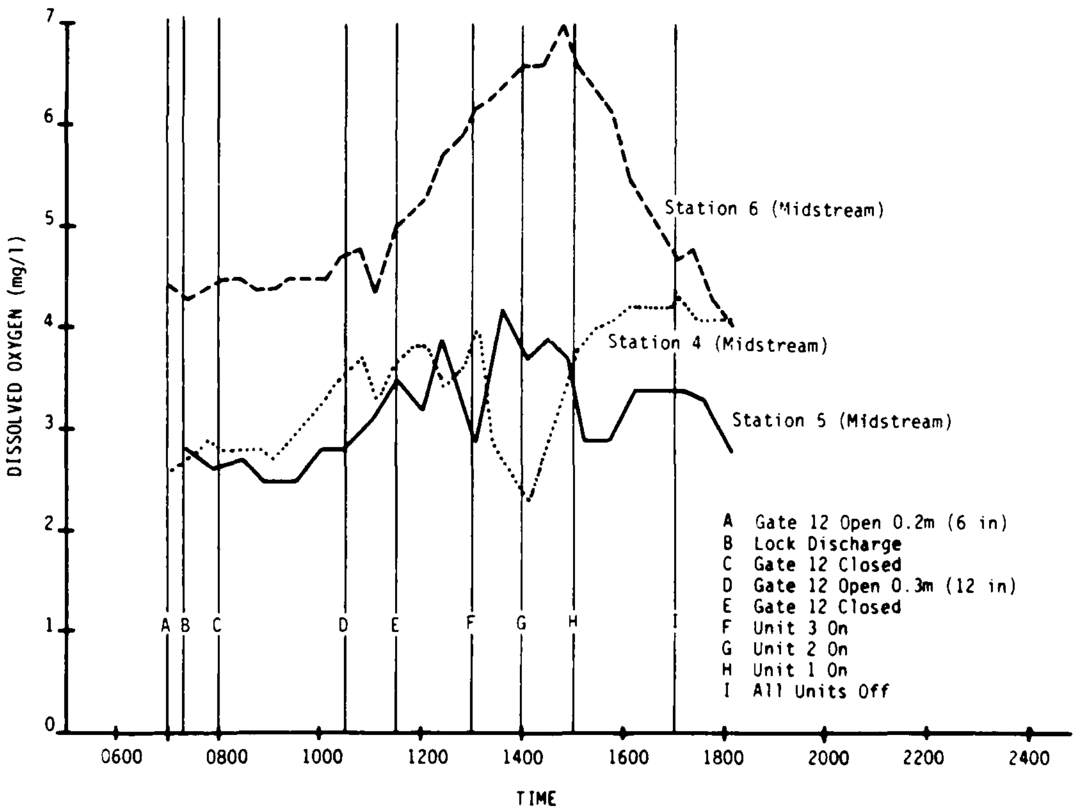

Figure 2. Surface dissolved oxygen concentrations at stations 4, 5, and 6 on 14 August 1986

0.2 m (0.5 ft) were much more dramatic than the increases achieved from a 0.3 m (1.0 ft) gate opening after the DO level had reached 3.0 mg/l.

Stations 2 and 3 displayed a sharp drop in DO levels when the first turbine was brought on-line followed by a gradual increase as the two remaining units were brought on-line. However, the DO never completely recovered to the pre-generation concentrations observed after releases were made from gate 12.

Figure 2 illustrates the DO concentrations at Stations 4, 5, and 6 for the same time frame as depicted on Figure 1. Generally, variations in DO concentrations observed at the upper tailrace stations were also observable at Station 4. DO changes noted at Station 4, however, were not as dramatic, and were shifted with respect to time and rate of discharge. Fluctuations in DO readings at Stations 5 and 6 could not be specifically attributed to operational discharges as they were at stations in the immediate tailrace area. It was noted that DO at Station 6 was higher than upstream stations by as much as 2.0 mg/l during the mid-day hours and extremely low readings (<2.0 mg/l) were not observed during the study. This correlates with Strain (1980).

Approximately two hours after generation ceased on 14 August, DO and temperature readings were measured in the draft tube exit area immediately below the powerhouse. These measurements revealed that DO ranged from 3.0 mg/l at the surface to 2.0 mg/l or less from 3 m (10.0 ft) below the surface to 11 m (36.0 ft), the bottom. Based on the previous experience of the technical advisor from the Waterways Experiment Station, Mr. Steven C. Wilhelms, at Mark Twain Lake, Missouri, it was hypothesized that the cause of the low DO water in the tailrace was due to leakage through the turbines during nongeneration periods. This poor quality water would gradually displace the water from generation in the tailrace and would move downstream. Therefore, testing and data collection were oriented toward testing this hypothesis beginning 15 August at 1700 hours through the remainder of the study. A test of an interim procedure as a means of preventing the buildup of extremely low DO water was conducted. This procedure provided spillway releases overnight from 1500 to 0700 hours and indicated good effectiveness. At Station 2 following an overnight release of 0.1 m (0.25 ft) from gate 12, DO readings were between 3.0 to 4.0 mg/l at all depths compared to readings of 1.0 mg/l or less when the releases were not provided. Lower readings of 1.0 mg/l or less, however, were measured directly in front of the turbine discharge bays.

Figure 3 depicts DO changes in the three upper stations resulting from spillway releases from gate 14 which was opened to a height of 0.2 m (0.5 ft) and 0.3 m (1.0 ft). For the purpose of the study gate 14 had been modified by inserting stoplogs on the upstream side of the gate to a depth of 1.8 m (6.0 ft) from the surface. These stoplogs created a weir effect and allowed gate 14 to discharge surface water rather than water from the depth of the gate opening, 9.1 m (30.0 ft). The effects of discharges from gate 14 on DO paralleled data collected during discharges from the unmodified gate 12 (Figure 1).

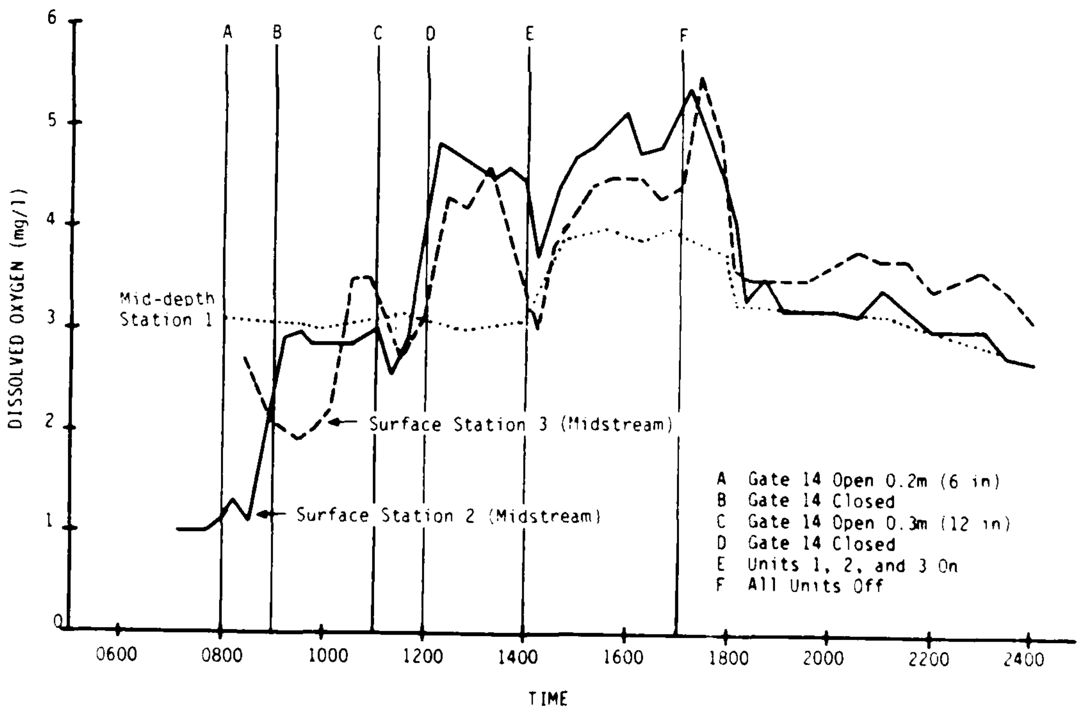

Figure 3. Dissolved oxygen concentrations at stations 1, 2, and 3 on 15 August 1986

Figure 3 also depicts generation startup with all three functional turbine units started sequentially within minutes of one another. This startup began at approximately 1400 hours and shows that DO at the middepth of Station 1 registered a corresponding rise over the duration of the generation period. This rise seemingly is due to an expanded withdrawal zone which encompasses more highly oxygenated water closer to the lake surface. Over the same period the surface DO at Stations 2 and 3 showed an immediate but short-lived sag of about 1.0 mg/l followed by an equally sudden rise of about 2.0 mg/l. Stations 2 and 3 also registered a general improvement in DO over the 3-hour generation period (to approximately 5.0 mg/l) followed by a drop of about 2.0 mg/l after generation ceased. Continued monitoring at these stations through 0300 hours on 16 August revealed a progression of low DO readings (<1.0 mg/l) from near the bottom at Station 1A, both up through the water column and downstream. By the end of the monitoring period this slug of low DO water had reached the surface at Stations 1A and 2, and to middepth at Station 3.

On 17 August the procedures outlined in the June 1986 SOP were tested for effectiveness. In summary, these provisions require spillway releases from gate 12, opened at 0.1 m (0.5 ft), to relieve fish distress situations or to increase DO concentrations in the immediate tailrace following a 3-hour period of below 2.0 mg/l readings or a single reading of 1.5 mg/l at the water quality monitor. If the initial one-hour release from gate 12 does not relieve the situation, the SOP further specifies that gate 10 be opened the same height and both gates provide releases for one hour.

Responses to low DO levels summarized above were based on initial data recorded by the automatic water quality monitor and verified by titration tests performed by WFG personnel. Therefore, discussions of results regarding SOP procedures are focused on data collected at Station 3. However, it was noted throughout the study that DO concentrations near the water quality monitor intake were highly variable compared to readings taken at other points across the river at this location. Also, comparisons of monitor and YSI results revealed that the DO monitor generally indicated higher readings than those obtained with the YSI meter.

The test of the SOP began following a period of about 15 hours when the DO from Station 3 upstream was less that 1.0 mg/l. At Station 3 midstream DO levels throughout the water column remained below 1.0 mg/l during the one-hour period when gate 12 was opened singularly. In accordance with the 1986 SOP, after one hour gate 10 was opened. At the two-hour mark in the operation the surface DO had recovered to above 4.0 mg/l, the middepth value had improved to slightly above 1.0 mg/l, and the bottom DO had remained below 0.5 mg/l. Releases from both gates continued for an additional 30 minutes at which time surface DO was up to 6.0 mg/l; however, middepth and bottom DO readings remained unchanged from concentrations measured at the two-hour mark. Within a few minutes of gate closure middepth concentration reached over 4.0 mg/l and the bottom reading increased to about 2.0 mg/l. Approximately 1 hour following the gate closures described above, there was a lock discharge which resulted in a sharp, short-lived depression in DO concentrations at all depths of the midstream sampling point at Station 3.

CONCLUSIONS

Observations at the middepth of Station 1 upstream and the surface at Stations 2, 3, and 4 in the tailrace, during the startup of generation indicated that irrespective of the startup sequence, there is an initial drop in DO levels in the immediate tailrace associated with the start of hydropower production. The severity and duration of this DO drop is influenced by the startup procedure. Data indicate that compressed starting of the generators, i.e., all units sequentially put on line within minutes of one another, minimizes both the acuteness and the duration of the DO drop. This seems to be due to the increased expansion of the zone of withdrawal within the lake, thus capturing more of the epilimnetic waters.

Data collected during spillway gate openings indicated that comparatively there is no difference between releases from gate 14 modified with stoplogs and other gates. Additionally, the data indicated that when DO levels were severely depressed, the spillway releases resulted in more dramatic improvements compared to releases when tailwater DO was 3.0 mg/l or better. Manipulations with spillway gate heights indicate that releases result in improvements in DO without respect to gate height, however, these improvements occur more rapidly with increased discharges either through increased gate heights or by opening additional gates. Data collected during spillway discharges indicate that the slug of low DO water that accumulates in the immediate tailrace is apparently pushed downstream by releases and DO is not significantly improved. Further, overnight releases through the spillway gates seem to prevent significant accumulations of low DO water and instead both mixes and transports this water downstream except in a very localized area in the vicinity of the turbine discharge bays.

Intensive monitoring in the tailrace area substantiated the hypothesis that extremely low DO water moved into the downstream area from the turbine discharge bays. Using conductivity as a conservative tracer, data collected with the Hydrolab in the lake clearly showed that the low DO water being observed in the tailrace during nongeneration periods was available at the elevation of the powerhouse intakes. Conductivity and pH were measured at the draft tube exits and were found to be identical to those values measured in the lake at the intakes. Further, data collected during the study indicated that the extremely low DO water accumulates in the tailrace at varying rates. During an overnight monitoring period the slug of low DO water had moved downstream only to the middepth at Station 3. On other occasions when monitoring began in the early morning, results indicated that the low DO water was uniformly distributed throughout the water column at Station 3 and had moved farther downstream. It is hypothesized, therefore, that the leakage varies in rate of discharge and is most likely attributed to improper or incomplete closure of wicket gates. Additionally, groundwater discharges in the tailrace area may also provide a source of extremely low DO water.

Comparisons between data collected in the river at the location of the monitor intake (Station 3), titrations using the azide-modified Winkler technique, and data obtained from the automatic water quality monitor at the same time indicated that the monitor frequently recorded erroneous DO data. In consideration of the hypothesis regarding leakage, a monitor would best provide data on which to base actions described in the SOP if it were located closer to the powerhouse.

RECOMMENDATIONS

(1) Provide for the maximum use of compressed starting procedures for hydropower production during the months when the lake is stratified.

(2) Adjust the automatic monitor position to a site closer to the powerhouse where the low DO problems originate and resolve the discrepancies noted between the monitor readings and readings obtained by other methods.

(3) Revise the SOP to provide for immediate action (spillway releases) when low DO conditions (3.0 mg/l or less) are detected which are not the result of an operational release producing a short-term drop in concentrations. Or, as an alternative, provide for continuous low volume (0.1 m (0.25 ft) gate opening) spillway discharges during nongeneration for the months the lake is stratified to prevent the extremely low DO conditions from developing.

(4) Conduct further investigations to determine the source and circulation patterns of the low DO water in the tailrace. Evaluate structural solutions to stop the leakage and/or other alternatives. If any alternative is implemented, further study to determine the necessity for an SOP as described above should be conducted.

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PROVIDING MINIMUM FLOWS BELOW HYDROPOWER PROJECTS

H. Morgan Goranflo, Jr.
and
J. Stephens Adams, Jr.

Tennessee Valley Authority
Engineering Laboratory
P.O. Drawer E
Norris, Tennessee 37828

ABSTRACT

Many hydropower facilities are operated only during a portion of the day to meet peak system power demands. Sometimes, during portions of the year when system power demands are low, no releases are made for several days at a time. Such periods without flow can seriously damage aquatic life in the tailwater.

This paper discusses two projects where continuous minimum flows are provided year around. The first is the Norris project where a minimum flow of 5.7 m^3/s (200 cfs) is provided by a reregulating dam located approximately 2.4 km (1.5 mi) below the main dam. The other is the Tims Ford project where a minimum flow of 2.3 m^3/s (80 cfs) will be provided through a small hydroturbine operating during periods when the main turbine is shut down. The paper will address the alternatives evaluated at each project and the design, construction, and operation of the minimum flow systems.

NORRIS MINIMUM FLOWS

General

TVA's Norris project is located on the Clinch River at river mile 79.8 in Campbell and Anderson Counties, Tennessee, about 32 km (20 mi) northwest of Knoxville. The dam is a concrete gravity structure approximately 580 m (1,900 ft) wide and 81 m (265 ft) high. The powerhouse contains two generating units rated at 50.4 MW each. The hydraulic turbines are the vertical Francis type and have a discharge capacity of about 120 m^3/s (4,200 cfs) each at a rated head of 50 m (165 ft).

Flow conditions for several miles downstream from the dam are essentially determined by controlled releases from Norris Dam. An insignificant amount is contributed from uncontrolled runoff. Leakage through Norris Dam produces minimum flows of 0.5 to 1.8 m^3/s (17 to 65 cfs) when the turbines are off.

With a turbine discharge capacity approximately twice the average annual inflow, 12 hours of peaking per day are needed on the average to pass inflow in a normal year of operation. Actual daily schedules vary depending on system load requirements and reservoir levels. Summertime peaking releases normally begin about midmorning and continue for the remainder of the daylight hours. Wintertime load peaks tend to occur in the morning and again in the evening. Unless the Norris pool is well above its normal operating level, the turbines are normally not operated during offpeak hours. Extended periods of zero discharge covering several days to several weeks can occur in the spring and fall periods to conserve water in Norris Reservoir when power demands and reservoir levels are low, or during flood control operations.

These daily flow fluctuations and the prolonged shutoffs combined with the low dissolved oxygen concentration in the discharge (late June through early November) are considered to have deleterious effects on aquatic life, river recreation, water quality and aesthetic values of the downstream river reaches. By venting turbine releases when needed and providing a minimum release of at least 5.7 m^3/sec (200 cfs), it was felt that these adverse effects could be mitigated or eliminated altogether.

Alternatives Considered

Various schemes were investigated for providing the desired minimum flow. These included releasing through the existing sluices, adding a small hydroplant, and constructing a reregulating structure downstream of the dam. Venting of turbine releases when necessary would be a part of each alternative.

Sluice releases to provide continuous minimum flows when the turbines were not operating were evaluated for several minimum flow values. For the period October through March, 5.7 m^3/s (200 cfs) was acceptable with higher values desired during daylight hours in the warm months. Estimated annual costs ranged from \$1.13 to \$1.70 million (1980 dollars) and were due to power losses from releases bypassing the generating units.

A new small hydro unit to be constructed on the left bank was also considered. The powerhouse would contain a single vertical Francis turbine connected to a generator rated at 8,000 kW. The unit's rated discharge would be 17.0 m^3/s (600 cfs) and could discharge as little as 5.7 m^3/s (200 cfs). Costs for this option would include an initial investment of \$14.1 million and an annual cost of \$0.47 million. The annual cost was primarily due to a shift of some peak generation to offpeak hours.

Two schemes for constructing reregulating structures below Norris Dam were seriously considered. The first was a collapsible dam which would regulate the release of stored water by maintaining a constant depth of flow over one or more gates. When the turbines were operating, the gates would be lowered to prevent a tailwater increase at the dam. The initial cost of a collapsible barrier dam was estimated to be $5.8 million. Annual costs would range from $0.67 to $0.94 million, depending on the minimum flow assumed.

The second scheme was a low weir which would have a series of sluices installed at its base to retard emptying of the channel following turbine shutdown. Short turbine pulses every few hours would be required to recharge the channel until generation resumed. However, the weir would cause a permanent tailwater rise at Norris Dam, resulting in a loss of capacity and energy generation from the hydroplant. Estimated costs for the weir were $0.5 million initially, and annual expenditures of $0.04 million.

Construction of a rock-filled gabion weir approximately 2.4 km (1.5 mi) downstream of Norris Dam was determined to be the most attractive alternative. Total costs for this scheme were lower than any other alternative considered by far. Also, since the project was considered to be experimental, a rock weir could easily be removed if major problems were encountered during testing.

Design

The preliminary design process involved examining several different weir locations, sizes, and shapes to determine a solution that best met the project design objectives. The most important objectives considered were maintaining a minimum flow of 5.7 m^3/sec (200 cfs), avoiding significant impacts on the Norris Dam hydroplant, providing safe floating and fishing conditions, and minimizing project costs. Improvement with respect to one objective often meant a reduction with respect to one or more of the other objectives. Numerous tradeoffs between objectives were evaluated using mathematical and physical modeling to find the best overall solution. For example, it was important to locate the weir as close to the dam as possible to minimize the amount of uncontrolled local inflow that could deposit sediment or debris in the weir pool and at the same time provide improved hydraulic conditions over the entire tailwater. However, if the weir was located too close to the dam, then headlosses would occur due to the higher tailwater elevation caused by the weir backwater.

The hydraulic effects of the weir were examined through the use of a mathematical computer model and a physical model. The computer model was used to determine headlosses (due to tailwater rise) at Norris Dam and to determine how long a minimum discharge of 5.7 m^3/s (200 cfs) could be sustained without storage replenishment from Norris turbine releases. Data collected during operation of the physical model was used to calculate computer model discharge coefficients for flow over the weir and to evaluate boating risks.

A mathematical model for a section of the Clinch River from the dam to about 6.4 km (4 mi) downstream was developed. Simulation results showed that a flow of about 5.7 m^3/s (200 cfs) could be maintained below the weir for about 12 hours without turbine releases. Also, tailwater elevation below Norris Dam would increase by about 0.09 m (0.3 ft).

A physical model was constructed at the TVA Engineering Laboratory. The primary concern addressed by the model study was navigation over the weir by small boats or canoes. Several weir geometries were tested, with most performing satisfactorily at one or two of the expected flowrates, but being unacceptable at other flows. Recommendations were to construct and observe a 6.4-m (21-ft) wide weir in the field, and if the performance was not satisfactory, then the structure could be modified into a 9.1-m (30-ft) configuration. Discharge coefficients were also computed and provided for the mathematical model.

Regulation of weir releases would be by butterfly valves controlled by floats installed in some of the discharge pipes. After turbine operation ended, flow would pass over the weir and through the uncontrolled pipes. As the water level dropped below the top of the weir, the valves would begin to open to maintain desired releases. Without controls, 5.7 m^3/s (200 cfs) could not be maintained without either increasing the weir height or increasing pulsing frequency. Either option would cause a significant increase in annual power losses at Norris.

Physical Features

The weir was constructed of galvanized steel gabion baskets filled with 10 to 20 cm (4 to 8 in.) washed limestone rock. It is 1.5 m (5 ft) high, 6.4 m (21 ft) in width, and contains 54 30-cm (12-in.) diameter steel pipes for discharge control. The 122-m (400-ft) long weir is divided by Hibbs Island into a 46-m (150-ft) long west channel section and a 76-m (250-ft) long east channel section. To reduce construction costs, a loose rock core was used. Twenty of the fifty-four steel pipes were fitted with float actuated butterfly valves to control weir releases. Trashracks were placed upstream of all controlled pipes to reduce potential damages to the flotation device from debris. An impermeable membrane was placed within the upstream portion of the weir to minimize leakage. Figure 1 is a photograph of the completed weir structure.

Figure 1. Norris Weir

Construction

The weir was constructed in three stages from fall 1983 until spring 1984. Total construction time, excluding delays, was approximately three months. The west channel section of the weir was built in the first stage. Flows (mostly leakage) were diverted to the east channel through pipes contained in a small dam built for this purpose. The dam was also used as an access road to the west channel construction site. After the weir section in the west channel was completed, flows were diverted through discharge pipes in the finished section while work proceeded in the east channel. An 11-m (36-ft) long section was left uncompleted until the remaining weir was constructed to its full height. This enabled the pool level to drop sufficiently for construction personnel to enter the river three hours after unit operation ended. The last stage of construction was closing the open section over a two day weekend period with no turbine releases. All work was accomplished during periods the turbines were not operating. During the fall, power demands required that construction take place at night on the 11 p.m. to 7 a.m. shift. All other work was performed during daylight hours.

Operation

The weir is designed to sustain a minimum discharge of approximately 5.7 m³/s (200 cfs) for 12 hours after turbine releases end. Based on historical records, adequate streamflow below the weir can be maintained on a typical day of operation. However, during extended periods of zero release, a one-turbine pulse of 30 minute duration is required twice daily to replenish storage behind the weir.

The weir as constructed did not provide satisfactory regulation. Field tests revealed that there was too much leakage and the flow decreased too rapidly following turbine shutoff. Seventeen of the uncontrolled pipes were plugged, and leakage was reduced by sealing the top 15 cm (6 in.) of the weir. Field tests to evaluate the weir's hydraulic characteristics were again conducted, and overall flow patterns were determined to be satisfactory. The water surface profile over the weir appeared smooth, and no large recirculating eddies were observed. The measured increase in tailwater elevation varied between 0.05 and 0.11 m (0.18 and 0.35 ft).

Three hours after the turbine releases stop, the pool elevation drops to the top of the weir. From hours 3 through 7, a flow of 4.5 m³/s (160 cfs) is maintained through the weir, primarily by the uncontrolled pipes. The valved pipes begin to contribute more and more as the pool level drops, steadily increasing total discharge to about 6.1 m³/s (215 cfs) at hour 10. The flow gradually returns to 4.5 m³/s (160 cfs) by the end of the 12 hour period. Leakage through the abutments supplements pipe discharge to effectively maintain a constant release.

Results

Conditions in the tailwater have improved due to weir construction and aeration of turbine releases. Fish food organisms, such as mayflies, caddisflies, stoneflies, snails, and crayfish preferred by trout have grown in number. Also increasing in abundance have been minnow species, a part of the diet of the large brown trout. Trout condition, a measure of the plumpness of an individual fish and an indicator of growth rate, has also improved. From 1980-83, conditions of rainbow trout dropped an average of 12 percent between summer and fall. In 1985, the condition did not drop at all.

An increase in trout stocking in addition to improvements in tailwater conditions has resulted in substantial growth in fishing pressure and recreation. Plans are to increase stocking even further, raising expectations for greater use in the future.

TIMS FORD MINIMUM FLOWS

Alternatives for supplying instantaneous minimum flows in the Elk River below TVA's Tims Ford Dam were evaluated. Due to thermal stratification in Tims Ford Reservoir, turbine and sluice releases at the dam provide cold water ranging from 5°C in the winter to 16°C in the fall. These temperatures offer the opportunity to establish and maintain a 64-km (40-mi) long cold water fishery in the tailwater. Supplying minimum flows in the river channel at all times would help maintain an adequate food base and suitable flow and depth conditions for trout. Costs and benefits for four alternatives were estimated, and implementation of the most favorable alternative is nearing completion.

Project Description

Tims Ford Dam is located at Elk River Mile 133.8, in southcentral Tennessee. This location, within two hours driving time of the large population areas of Huntsville, Alabama; Chattanooga, Tennessee; and Nashville, Tennessee, offers an opportunity to maintain and enhance a cold water fishery unique to this region.

Site description

Tims Ford Dam was closed in December 1970 and a single diagonal-flow, fixed blade generating unit went into commercial operation in March 1972. The unit is rated at 45 MW at a net head of 36.2 m (120 ft). The dam is also equipped with a 0.41-m (3-ft) diameter low-level supplementary water release sluice with rated discharge capacity of 7.1 m³/s (250 cfs). Below the dam, the Elk River flows through Fayetteville, Tennessee, approximately at river mile 90. The Fayetteville Water Treatment Plant pump intake is located at river mile 93.9. To maintain proper submergence of the pump intake, a river stage corresponding to one foot at a nearby USGS gage is required. Currently, this corresponds to a flow of about 4.1 m³/s (145 cfs). The unregulated drainage area between Tims Ford Dam and the gage is 773 km² (298 mi²), and the estimated 20-year recurrence interval, 3-day minimum local inflow from this area is 1.25 m³/s (44 cfs).

Current operations

Tims Ford is a multipurpose project, and as such, is operated for authorized purposes of flood control, recreation, power production, and water supply. The single generating unit is generally limited to a discharge rate of 110 m³/s (3,900 cfs) due to downstream considerations. The average annual adjusted flow at Tims Ford is 27 m³/s (955 cfs), which means that the unit on average can be expected to operate about 25 percent of the time. Based on historical records, the average number of "zero release hours per day" varies from 12.3 in December to 19.6 in October.

The supplementary water sluice is used from Memorial Day weekend through September to provide for water supply at Fayetteville. These flows are provided on weekends, since limited reservoir inflows during this period are generally passed through the generating unit on higher load weekdays only. During the summer and early fall months, when two hours of turbine use per day is common, the unit is often operated for an hour in midmorning and again for an hour in late afternoon to provide for a more even distribution of flow at Fayetteville to ensure that the stage requirement is met.

The generating unit at Tims Ford is operated locally from within the powerhouse. The powerhouse is normally staffed from 6 a.m. until 10 p.m. on weekdays only. Turbine use on weekends requires personnel to be called in. The sluice can be operated either locally or by remote control from a neighboring project. Evaluation of various alternatives described later in this paper is based on not having remote operation capability for the turbine. An air compressor was installed at the plant in 1984 and used during the fall of 1984 and 1985 when the natural dissolved oxygen concentration in the turbine releases dropped below 4 mg/L. Use of the compressor is planned for the future during similar low DO periods.

From examination of available records, leakage from the reservoir rim and through the unit was estimated to be 0.6 m^3/s (20 cfs). This water enters the river channel in the proximity of the dam. Leakage will vary with headwater elevation, being somewhat lower in winter and higher in summer.

Current visitation to the Elk River from Tims Ford Dam to Fayetteville was estimated to be about 9,000 annual trips. Two thousand are canoeing trips and 7,000 are fishing trips. Recreational access to the river is provided at seven different locations owned by TVA, and with one exception, they are maintained by the Elk River Development Authority. Currently, the State stocks about 70,000 trout annually in the Elk River below the dam.

Evaluation Criteria

The impact of the various minimum flow alternatives on power production at Tims Ford was based on the comparison of projected plant output for each alternative vs. simulated output for the base case, or historical conditions. Historical flow and reservoir elevation data for the period 1973 through 1983 were used to simulate daily peak and offpeak generation. These values were used to compute monthly averages for the period for comparative purposes. Historical reservoir elevations were used as much as possible, although for some alternatives the reservoir was necessarily lowered during dry conditions in order to meet the minimum flow requirements. In those cases, computed elevations were forced to converge to historical elevations as soon as possible by reducing releases in succeeding periods, thereby incurring either a power loss or seasonal shift of power. Maximum drawdowns below historical levels were noted in the results for each alternative.

The economic analysis for each alternative was based on the U.S. Water Resources Council recommended discount rate of 8 [illegible] percent and a project life of 25 years. All costs and benefits were expressed in 1985 dollars.

The fishing benefits evaluation was based on converting projected visitation increases to dollar values, based on the unit day value method established by the [illegible] Water Resources Council. This procedure utilizes a number of subjective factors to consider in assigning points. These factors include: (1) recreation experience, (2) availability of opportunity, (3) carrying capacity, (4) accessibility, and (5) environmental quality. Out of a possible 100 points, the Elk River was judged to merit a range of 65-69 points. In 1985 price levels, this converted to a range in the value of visits from \$16.50 to \$17.50 per unit day. For the purpose of this analysis, about 75 percent of the increase in benefits was attributable to enhanced minimum flows, and about 25 percent to enhanced dissolved oxygen levels.

Current fishing use estimates were supplied by personnel from the Tennessee Wildlife Resources Agency who are most familiar with fishing in the tailwater. Their estimates represented the best available information but have yet to be confirmed by actual pressure estimates based on field data.

In response to these improvements in the tailwater, stocking would be increased in each of the alternatives in proportion to the expected biological improvements and increase in angler use expected. Potential for increased fisherman use was calculated based on the following factors provided by the various alternatives: improved food base, increased production, increased carrying capacity, and improved conditions for bank and boat fishing.

For evaluation of the alternatives, the minimum flow was targeted to be 2.3 m^3/s (80 cfs). This selection was based on field visits to the tailwater sites by various personnel during periods of operation of the sluice at that rate. It was felt that this rate would provide acceptable flow conditions for the fishery as well as maintain suitable conditions for floating. This release, plus the estimated leakage and minimum local inflow as described previously, would also be adequate to provide the required flow at Fayetteville in most cases. It was apparent at the beginning of the study that minimum flows much larger than these would severely impact power production, and would limit flexibility in selecting alternatives.

Alternatives for Maintaining Minimum Flows

Four alternatives are described in this section. Each was evaluated based on the above criteria.

Sluicing alternative

This alternative used the existing sluice to provide a discharge at Tims Ford of 2.3 m^3/s (80 cfs) any time the generating unit was not operating. This alternative required no capital expenditure since no physical modifications to the sluice or other plant structures was required.

The impact on power generation included a net loss in both peak and offpeak generation, due to the diversion of water which would otherwise have been passed through the turbine. Annual average losses totaled [illegible] MWh peak and 84[illegible] MWh offpeak and were estimated to have a present worth replacement cost of \$[illegible].

Fishing would probably not be affected by this alternative. The flow provided was viewed as a minimum and would not attract additional users. This alternative, as well as the others, was [illegible] heavy increases in fishing pressure on the river immediately below the dam, at a highway bridge access. Due to rapid water rise when the turbine is started, additional warning signs at the site would have been warranted.

The impacts on water quality would be positive. The sluice releases would maintain lower overall temperature in the river between turbine pulses. The releases would be well aerated due to aspiration through the sluice vent system. Recent tests showed dissolved oxygen increases from 1 mg/L at the sluice intake to 10 mg/L just downstream from the sluice outlet in the tailrace.

For the period of record analyzed, the impact on reservoir elevations was minor. The simulations indicated, even in the drier years, a projected drawdown of only one foot below levels observed historically.

Fishery benefits were calculated as described previously. Benefits accrued from provisions of desirable DO concentrations and flow conditions in the upper tailwater. Those improvements would increase the food base, improve the ability of trout to feed during the late summer and early autumn, and improve the minimum water depths and velocities. These benefits accrued primarily in the upper reaches of the tailwater where the change from existing flow conditions would be most dramatic. It was estimated that this alternative would increase total annual visitation to 24,000 within five years. On a present worth basis, this corresponded to an increase in benefit of \$1,820,000 to \$1,924,000 over the life of the project.

Small generating unit alternative

This alternative would include the addition of a small generating unit to the existing powerhouse (see Figure 2). The unit would use the existing sluice pipe for water supply. A surplus cooling tower makeup pump/motor would be installed and operated in reverse as a turbine/generator. This equipment was available from a cancelled TVA nuclear project. Preliminary estimates of performance for this unit, operating as a turbine, indicated full gate releases of 2.2 to 2.5 m^3/s (78 - 90 cfs), with generator outputs ranging between 470 and 830 kW, depending on the reservoir pool elevation.

As in the case of the sluicing alternative, this unit would be operated any time the existing unit was not, thereby providing an instantaneous minimum flow at the project. Both units would be operated concurrently only during flood control operations to evacuate water from the reservoir.

This alternative would result in a small gain of total generation but a shift of peak energy to offpeak energy. This was due to the new unit releasing water during offpeak hours which otherwise would be passed through the larger unit only during peak hours. The expected annual generation gain was 600 MWh, with 1,930 MWh being shifted from peak to offpeak use. The net effect of these impacts was a present worth power cost of \$315,000. Nevertheless, this alternative had the least impact on power production of any of the alternatives studied.

Most other impacts were the same as described for the sluice alternative. The main exception

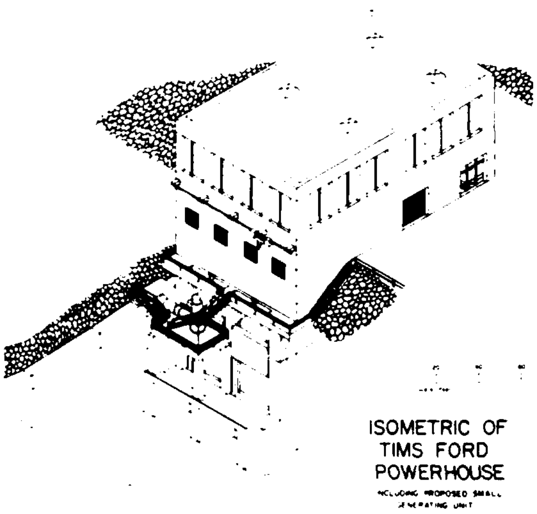

Figure 2. Tims Ford Powerhouse

was the dissolved oxygen concentration expected in the tailrace during the operation of this unit. It is expected that some natural aeration could be induced in the draft tube of the unit, however provisions would be made for forced air injection by a small compressor. In either case, DO uptake would not be as great as that occurring with the sluice releases although it was anticipated that adequate amounts would be provided when the natural DO content dropped below 4 mg/L.

Fishery benefit increases for this alternative were projected to be the same as for the sluicing alternative.

The project capital cost was estimated to be \$860,000, with a projected present worth O&M cost of \$103,000. An additional \$40,000 was estimated for outage costs during the construction period, since use of the existing unit would not be possible during early stages of construction.

Reregulating weir alternative

This alternative would involve the construction of a low weir some distance downstream from Tims Ford Dam. This weir was similar in purpose and scope to the Norris (Clinch River) weir described previously in this paper. Two alternate weir sizes were studied: one which could provide flows of 2.8 m^3/s (100 cfs) for twelve hours duration; and one which could provide the same flow for twenty-four hours duration. Due to limited access to the river in the first several miles below the dam, the proposed weir site was located 4 km (2.5 mi) below the dam. This location is owned by TVA and allows public access to the river.

The option to provide for 24 hours storage required a weir size that was deemed unacceptable due to safety concerns. The final weir selection was based on providing only 12-hours storage. The cost estimate for this option was based on a similar design as the weir below Norris Dam, and included 15 valved and 10 unregulated pipes. To replenish the water behind the weir would require a pulse of water from the generating unit at the dam of 25 minutes in duration once every 12 hours. Because the powerhouse is not manned on weekends, it was more cost effective to sluice the water rather than call in operators to make the two pulses 12 hours apart.

The impact on power production was a net loss of total generation, mostly from peak periods, resulting from the sluice discharge required on weekends. This loss was estimated to be 640 MWh. It was estimated that the weir would raise the tailwater elevation about .06 m (.2 ft) during normal operation of the generating unit. The estimated net present worth replacement cost of these impacts was $431,000.

Most other impacts were the same as described for the other alternatives. However, it was predicted that increased biological benefits would not occur between the weir and the dam. Thus the increase in annual visitation was projected to be lower, resulting in a total present worth benefit increase ranging from $1,169,000 to $1,241,000.

The weir was also evaluated as to possible backwater impacts upstream from the weir to the dam. It was calculated that the impoundment effect of the weir would result in water surface profiles which exceeded the flowage easement boundaries originally acquired between the weir site and the dam, for normal release rates of 110 m^3/s (3,900 cfs). Therefore the cost estimate included the purchase of additional flowage easements where appropriate.

The estimated capital cost was $673,000, with an additional $40,000 required for outage cost and a projected present worth O&M cost of $52,000.

Pulsing alternative

This alternative required that the generating unit at Tims Ford be operated in a pulsing mode to provide minimum flows in the reaches below the dam. This option did not provide for instantaneous minimum flows at the dam, but due to natural wave attenuation and depending on the time interval and pulse duration selected, would provide minimum flows of varying magnitudes in the channel downstream. For sites nearest the dam, resultant hydrographs would be sharply spiked, with high flows during the pulse and rapidly diminishing to zero shortly after the pulse ceases. At intermediate locations downstream a noticeable rise in flowrate would occur sometime after the pulse, but appreciable minimum flows would be maintained. At locations further downstream, the resulting flows would be essentially steady.

The pulse duration, 15 minutes, and interval between pulses, 3 hours, selected for evaluation were based on maintaining an instantaneous flow of about 2.8 m^3/s (100 cfs) at the proposed weir site. Minimum flows upstream from this site would be less (only 0.6 m^3/s (20 cfs) at the immediate tailwater), while the instantaneous minimum flows downstream would be larger, ranging to 9.8 m^3/s (345 cfs) at Fayetteville.

Impacts on power production would include a net gain in total generation and significant shifts of peak to offpeak generation. The gain (430 MWh) is because no water would be released through the sluice. The shift to offpeak power results from the unit being operated for pulses in the offpeak period. Annually, this shift would total 9,490 MWh. The present worth of the net replacement cost due to these impacts is $2,111,000.

No benefits were claimed for the higher net flows in the lower reaches of the tailwater. This is because the river was not considered as a prime recreational floatway. There could be a negative impact on safety for those persons using the river in the upper reaches, particularly near the dam. Water surface elevations would rise rapidly during the pulse, up to 2 m (6.2 ft). This would happen eight times per day, in contrast to current operations with only one or two pulses.

Constant raising and lowering of the water surface in the channel reaches immediately below the dam could increase the rate of bank erosion there. However, the reaches further downstream which have historically been eroding at a faster rate would not be affected as much because of channel attenuation.

For the period of record analyzed, upstream reservoir elevations could be reduced significantly. In the drier years, projected reservoir drawdowns up to 2.9 m (9.5 ft) below levels observed historically were computed. This could seriously impact recreation on the reservoir depending on the timing of the drawdown, with August and September being the most critical times.

There were no capital costs associated with this option. No attempt was made to estimate increased O&M costs due to much more frequent starting and stopping of the unit. However, operation costs would increase substantially. Because the powerhouse is currently manned from 6 a.m. to 10 p.m. during the weekdays, additional operators would be required to provide for operations 24 hours per day, 7 days per week. The present worth cost of this increased staff was estimated to be $1,240,000.

Fishery benefits were reduced due to the variability of flow in the reaches immediately below the dam, which would not provide biological benefits or good conditions for bank or float fishing. Benefits were increased further downstream because the higher minimum flowrates would provide increased water depths, wetter substrate, and more stable water temperatures. The estimated present worth benefits were $962,000 to $1,024,000.

Comparison of Alternatives

A quantitative and qualitative comparison of alternatives is shown in Table 1. The economic analyses show B/C ratios of approximately the same magnitude for alternatives one, two, and three. The lower B/C for the weir alternative is due to the reduction in benefits because minimum flows are not provided in the reach between the dam and weir site. Based on the study, the small unit alternative had the least adverse impact on power production and the most attractive B/C ratio. Although the plan indicates a relatively high initial cost, it provides an example of an innovative small hydro installation. It will allow a chance for the evaluation of the effectiveness of this type of installation in maintaining and enhancing a designated cold water fishery.

TABLE 2

Comparison of Alternatives

	Alternative			
	Sluice	Small Unit	Weir	Pulsing
COSTS				
Construction	0	860	673	0
Unit Outage	0	40	40	0
Operation and Maintenance	0	103	52	1,240
Power Losses	1,550	315	431	2,111
Stocking	155	155	103	103
Total	1,705	1,473	1,299	3,454
BENEFITS				
Coldwater Fishery	1,820-1,924	1,820-1,924	1,169-1,241	962-1,024
NET BENEFITS	115 - 219	347 - 451	(130)-(58)	(2,492)-(2,430)
BENEFIT/COST RATIO	1.07 - 1.13	1.24 - 1.31	.90 - .96	.28 - .30

Note: All amounts are present worth values expressed in thousands of 1985 dollars. Amounts in parentheses are negative values.

OTHER CONSIDERATIONS	Sluice	Small Unit	Weir	Pulsing
Potential negative impact on bank erosion	No	No	No	Yes
Maintain minimum flow in upper 3 miles	Yes	Yes	No	No
Includes improved access for public	No	No	Yes	No
Additional safety concerns	Yes	Yes	Yes	Yes
Requires land easements or land purchase	No	No	Yes	No
Flexibility in changing minimum flows	Yes	No	Yes	Yes
Minimum flows are aerated	Yes	Yes[1]	Yes[2]	Yes[2]
Recreational floating enhanced	No	No	No	Yes
Possible adverse impact on reservoir levels	No	No	No	Yes

[1]Aeration provided by natural induction or small compressor when DO<4.0 mg/L
[2]Aeration provided by large compressor when DO<4.0 mg/L

Three primary factors impacted greatly the selection of this alternative: (1) the relatively large amount of time the unit would be run (average of 18 hours/day), (2) the physical availability of a usable water supply for the unit, thereby minimizing civil costs, and (3) the availability of the unit as a surplus item at a greatly reduced cost

Implementation

Implementation of the small unit alternative began in the summer of 1985. An earth and rock cofferdam was placed in the tailrace for the purpose of dewatering the existing draft tube. The top of the draft tube was used to support the operating platform for the new unit. After a period of several weeks, the cofferdam was removed, allowing normal operations with the existing unit. The penstock for the new unit was tied into the concrete encased sluice pipe, which runs laterally along the tailrace. Butterfly valves were installed between the new unit and the sluice pipe, and downstream of the penstock on the sluice pipe for directional control of water, enabling the sluice to be used for providing flows during maintenance on the new unit.

The platform and penstock work had been completed by late fall of 1985. Work resumed in 1986, consisting of installation of electrical tie-in to the switchyard, wiring of controls and instrumentation, and modifications to the shaft and bearings on the unit. A small air compressor is being installed to enhance oxygenation, with air being injected into the penstock just upstream of the unit. Preoperational testing is expected to begin in October 1986, with online operation anticipated to follow soon thereafter.

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SELECTIVE WITHDRAWAL STRUCTURE OPERATION

Jack E. Davis[1] and Steven C. Wilhelms[2]

[1]Research Hydraulic Engineer, USAEWES, Vicksburg, Miss.
[2]Research Hydraulic Engineer, USAEWES, Vicksburg, Miss.

ABSTRACT

Reservoir personnel operating multilevel selective withdrawal outlet structures often have difficulty determining the daily outlet operations required to yield a desired release temperature. Therefore, we have compared four techniques designed to determine day-to-day outlet structure operations that will yield a particular reservoir release temperature given a specific outlet works configuration and various reservoir conditions. Data from a stratification season of approximately 8 months were used in the comparison. Given the prescribed optimal operating scheme of each technique, a mathematical description of selective withdrawal was used to predict the resulting project release temperatures. The release temperatures produced by each scheme's prescribed operations were compared to the target release temperature, and the most accurate technique was identified. The US Army Corps of Engineers (CE) program SELECT with a port-selection capability was considered the best of the four techniques. SELECT is a one-dimensional, user-oriented numerical model of withdrawal and daily operations that is executable on a microcomputer, as well as on a mainframe computer. The program is capable of being an integral part of an automated reservoir data collection and structure operation system.

INTRODUCTION

Background

As a result of increased public awareness of our environment and the formulation of state and Federal legislation, CE reservoir projects are being operated with an emphasis on specific water quality objectives. Our emphasis in this paper is on reservoir releases and ways to meet specified release temperatures. While temperature is not the only water quality parameter of interest in reservoir releases, it is certainly an important one. For example, the viability of a fishery downstream of a reservoir may depend on the release of water with a certain maximum or minimum temperature. Further, there could be limits on the rate of change of the water temperature during operational changes. Attempting to accurately meet requests accurately for particular release temperatures places a heavy burden on project operators.

At most projects where releases can be controlled, the outlet works are multilevel selective withdrawal structures. Operators often find it difficult to determine how to operate the structure to achieve a target or objective release temperature. Experience may tell them that under given temperature conditions in the reservoir, certain outlet operations will approximate the objective release temperature; but when the release requirements are stringent, experience alone may not be sufficient.

The major questions that must be answered for accurately operating an outlet structure to achieve an objective release temperature with a certain flow rate are (1) which outlets should be opened, and (2) if multiple levels are used, how should the total flow rate be distributed between them? To answer these questions, one must know the temperature of the water being withdrawn through each outlet. With this information, the flow rate for each outlet could be determined so that, when the releases mix, the total release has a temperature equal to the objective release temperature.

As a result of density stratification in a reservoir, upper and lower limits are formed by the process commonly called selective withdrawal (Wilhelms 1986) because a selective region of water (the withdrawal zone) is withdrawn from the reservoir for release. Only the water between the limits is withdrawn (Figure 1). If the withdrawal zone limits can be determined, and if the temperature and withdrawal distributions are known throughout the withdrawal zone, the resulting release temperature from the reservoir can be predicted. The selective withdrawal processes have been described mathematically (Smith et al. 1986) and incorporated into the computer program SELECT (Davis et al. 1986), making the program capable of accurately calculating the temperature of water withdrawn through an outlet given the release rate and reservoir temperature or density stratification.

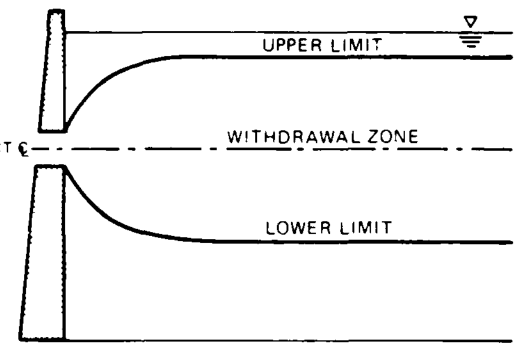

Figure 1. Selective withdrawal zone.

Objective and Scope

A port selection capability has been incorporated into the one-dimensional numerical model of withdrawal, SELECT, making the computer code a predictive tool for operational guidance (Holland and Wilhelms 1985). This paper compares the effectiveness of four techniques (including SELECT) at choosing outlet operations required to meet release temperature objectives. In addition to SELECT, the techniques include the simplest technique, called the Closest-Port Method (CPM), the Proportional Distance Method (PDM), and the Proportional Temperature Method (PTM). The PDM and the CPM are completely independent of water temperature at the outlet elevation and are functions of the proximity of the outlets to the elevation of the objective release temperature in the reservoir. The PTM estimates the release temperature from an outlet as the temperature at that outlet's center-line elevation. These methods may not be the only techniques used to determine outlet operations, but they are methods which may commonly be used at some reservoir projects.

DESCRIPTION OF METHODS AND DATA

Closest-Port Method

For the CPM, one determines the elevation of the target or objective release temperature in the reservoir and then selects the submerged outlet closest to that elevation for operation. All of the release (up to the hydraulic maximum) is passed through that outlet. For example, from Figure 2, Port 1 is closest to the target temperature; therefore, the total release would pass through that outlet if the CPM is used for operational guidance.

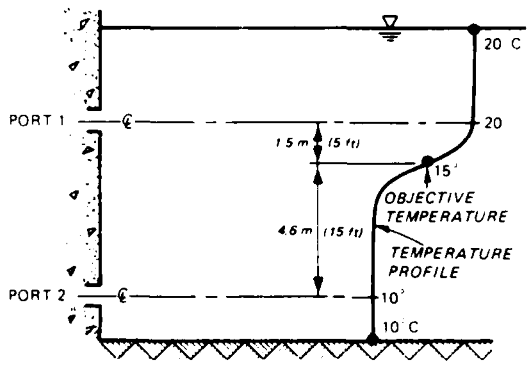

Figure 2. Outlet elevations relative to temperature profile.

Proportional Distance Method

For the PDM, one must determine the elevation of the target release temperature in the reservoir and then select the outlets that lie immediately above and below that elevation for operation. The portion of the total release for each outlet is a function of how close the outlet is to the elevation of the target release temperature. For example, Figure 2 indicates that Ports 1 and 2 are the ports immediately above and below the elevation of the target temperature. Port 1 is 5 ft away from the target temperature; Port 2 is 15 ft away. A simple distance ratio is used to divide the total release between the ports. The flow rate for the upper outlet would be

$$Q_U = \left(1 - \frac{D_U}{D_T}\right) Q_T$$

and the flow rate for the lower outlet is

$$Q_L = \left(1 - \frac{D_L}{D_T}\right) Q_T$$

where

Q_U = flow rate through the upper outlet

D_U = distance between the upper outlet and the elevation of the objective release temperature

D_T = distance between outlets

Q_T = total flow rate through the outlet works

Q_L = flow rate through the lower outlet

D_L = distance between the lower outlet and the elevation of the objective release temperature

Equations 1 and 2 indicate that Port 1 should release 75 percent of the total discharge while Port 2 should release 25 percent. Note that the outlet release temperature does not influence the computation of the flow rates. In fact, the only influence exerted by temperature for both the CPM and PDM is the selection of the ports to be operated and then only as a function of proximity to the objective temperature.

Proportional Temperature Method

With the PTM, one must determine the outlets immediately above and below the elevation of the objective release temperature in the reservoir. The temperature at the center-line elevation of each of the outlets must then be determined. The portion of the total release to be released through each outlet for maintenance of the release objective is based on a ratio of the temperature differences between the outlets and the objective temperature. Mathematically described, the flow rate through the upper outlet would be

$$Q_U = \left(1 - \frac{\Delta T_U}{\Delta T_T}\right) Q_T \quad (3)$$

and the flow rate through the lower outlet would be

$$Q_L = \left(1 - \frac{\Delta T_L}{\Delta T_T}\right) Q_T \quad (4)$$

where

ΔT_U = temperature difference between water at the upper outlet elevation and the objective release temperature

ΔT_T = temperature difference between water at the upper and lower outlet elevations

ΔT_L = temperature difference between water at the lower outlet elevation and the objective release temperature

For example, using the PTM Figure 2 indicates that Ports 1 and 2 should each release 50 percent of the total release. Note, in this case, that the approximation of the outlet release temperatures with outlet center-line temperatures significantly influenced the computation of the flow rates compared to the CPM and PDM methods.

SELECT

As previously discussed, SELECT uses a more sophisticated approach to determine port operation. Initially the PTM is used as the estimate for outlet operation and flow rate proportioning. The PTM operational scheme is then modified to account for effects of stratified flow. The following steps are involved in SELECT's solution algorithm (all performed by the computer):

a. SELECT determines the outlets to open and computes an initial estimate of the outlet flow rates using the PTM.

b. The estimated flow rates are then used to determine the outlet release temperatures using the description of selective withdrawal.

3. These release temperatures are then used to redistribute the flow between the ports using the same ratio equations as in the PTM.

In almost every case that has been evaluated, this single iteration has been sufficient to accurately determine outlet operations.

Data Used for Comparison

The above methods were used to determine outlet operation over an 8-month period for an observed set of reservoir conditions. The reservoir data were typical of stratification patterns from meteorologically dominated reservoirs. Figure 3 shows an isothermal plot of

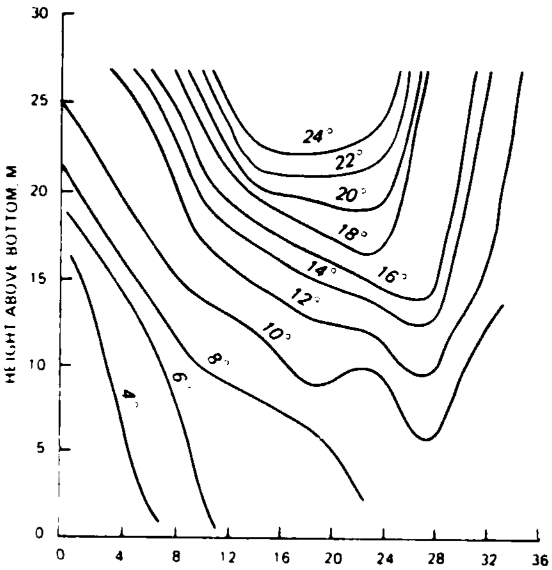

Figure 3. Isotherms in test case reservoir.

the reservoir temperature profiles for the 8 months in the comparison. Structure operation (outlet modifications) was changed weekly, and was based on a monthly temperature profile and specified weekly release temperature objective and discharge. The target temperatures for the 8-month stratification season are shown in Figure 4.

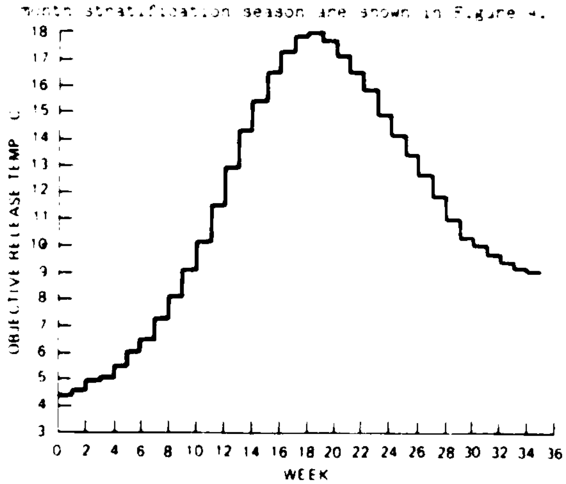

Figure 4. Objective temperature for release.

The example outlet structure consisted of two separate wet wells containing two ports per wet well at staggered elevations (Figure 5). The outlet structure allowed two outlets to be used (one in each wet well) simultaneously for the blending of differing reservoir water temperatures needed to meet the temperature objectives.

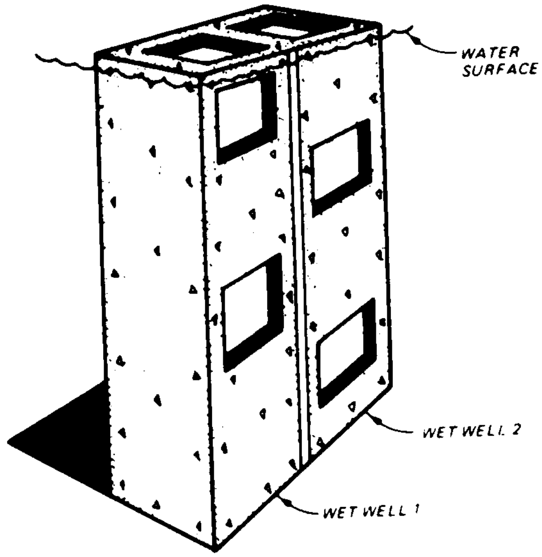

Figure 5. Withdrawal structure configuration.

RESULTS AND CONCLUSIONS

The results of these four methods were compared to evaluate their capabilities for selecting the outlet operation to achieve a particular release temperature under a given set of reservoir and operating conditions. These decision-making techniques were compared by using a numerical description of the withdrawal processes to determine release characteristics as opposed to a physical model or prototype measurements. For the CPM, PDM, and PTM techniques of decision-making, the ports for operation was selected and port discharge was determined by hand computations. This information and reservoir conditions were used in the numerical description of withdrawal to determine the release characteristics. The decision-making algorithm in SELECT essentially reproduced the PTM and then adjusted the withdrawal rate through each port to meet the release objective more closely.

Each of the port selection techniques was used in a simulation of 8 months of operation of the outlet structure previously described. The release temperatures resulting from these simulations were compared with the release target temperatures in Figure 4. With the CPM, the standard error* between the simulated and objective release temperatures for the entire 8 month period was [illegible] °C with a maximum error of

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* standard error = $\sqrt{\frac{\Sigma (T_o - T_r)^2}{N - 1}}$

where

T_o = objective temperature

T_r = release temperature

N = number of days in the period of operation

Davis, et al.

3.5° C. The PDM showed a standard error of 0.4° C and a maximum error of 1.3° C. The PTM had a standard error of 0.1° C and a maximum error of 0.5° C. Simulation of structure operation with the decision-making algorithm in SELECT resulted in a standard error of 0° C with a single nonzero error of -0.1° C.

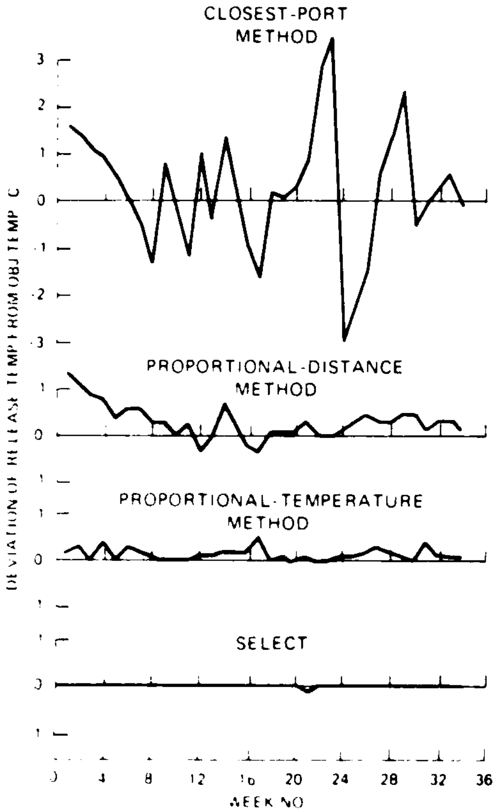

Figure 1. Comparison of deviation of release temperatures from target for each port selection technique.

The results indicated that, as the techniques incorporated more comprehensive concepts that more closely account for the dominant physical processes, the results improved greatly. The CPM, which selected only one outlet, yielded the worst results. The PTM, which incorporated outlet release temperatures, was more accurate. SELECT, which incorporated outlet release temperature plus the known effects of stratified flow, yielded even more accurate operation than the PTM. Based on the needs of individual projects and downstream interests, SELECT, which is convenient and accurate, should be considered for use, particularly when specific release temperatures are critical.

It should be noted that we would not expect an operational error of 0.0° C if SELECT were in use at a reservoir, but we would expect the error to be smaller than if one of the other techniques were used. To qualify these results, one must recognize how they were derived. A mathematical description of selective withdrawal was used to determine the withdrawal characteristics for the CPM, PDM, and PTM operations. SELECT also used that description. Thus, the results of 0.0° C average error for SELECT indicated that the port selection algorithm is able to determine the port operation and flow proportions to meet objective temperature exactly when the mathematical description of withdrawal accurately defines the actual outflow characteristics of the structure. Significant differences between the actual release temperature and objective temperature would occur only if the actual withdrawal characteristics of the project differed greatly from the mathematical description of outflow. In this case, however, all methods mentioned would probably have substantial error, with SELECT generally having the least error of the four.

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APPLICATION OF THE SELCIDE MODEL
IN THE NASHVILLE DISTRICT

R. B. Sneed

Hydraulic Engineer, U. S. Army Engineer District Nashville, Nashville, Tenn.

ABSTRACT

The author discusses application of the SELCIDE model developed at the Waterways Experiment Station to Corps of Engineers projects in the Nashville District. The need to accurately route water quality through a system of ten impoundments on the Cumberland River system is emphasized. The ability to predict the physical and chemical properties of water released from both tributary and main-stem projects using SELCIDE is presented with a series of four case studies. Included in the examples of the application of SELCIDE are a navigation project, a tributary hydropower project, a mainstem hydropower project, and a tributary project with selective withdrawal capability.

INTRODUCTION

The CE has long recognized the need to be able to accurately predict the physical and chemical properties of water released from their projects. This is particularly true in the Nashville District where a system of ten tributary and main-stem projects are operated in the Cumberland River Basin. The ability to route water quality through the system depends in a large part in being able to accurately assess the quality of spillway and turbine releases from both tributary and main-stem projects. The Nashville District has for a number of years accomplished this with one or another version of what is now titled SELCIDE. SELCIDE allows the user to make predictions of release water quality for both planned and existing projects and is thus both a useful planning tool as well as an operational tool.

SELCIDE as it exists today has its roots in the earlier SELECT model developed at WES by Bohan and Grace (1973). Initial physical model investigations were begun in 1966. These studies were carried out by injecting red dye in a stratified water column and filming the effect of varying the quantity of discharge and the location of the discharge port on the zone of withdrawal. From this work it was determined that the variables of primary concern in describing the upper and lower limits of withdrawal were the orifice size, the velocity through the orifice, the density profile, and the location of the orifice with respect to the density profile. These relationships were then described mathematically for the case where boundaries are not encountered in terms of the densimetric Froude number. It was also determined that the upper and lower withdrawal zone limits are independent of each other and one or both can extend to the boundary without affecting the other. A similar approach for flow over submerged weirs identified the velocity over the weir, the density profile, and the location of the weir with respect to the density profile as the important variables. The data were correlated to the densimetric Froude number as in the case of flow through an orifice.

The SELECT model underwent several modifications through its development. However, most of these changes involved the format of required input data and the presentation of model results, and the basic theory behind the model remained intact. In 1985 the DECIDE option was added to the SELECT code, thus the name SELCIDE was derived from the SELECT code with the DECIDE option added. With SELECT it was possible to determine the quality of release water given the location of outlet ports and the headwater profile. The DECIDE option allows the user to specify an objective value and the model will determine which ports to open and the relative flow distribution between them. The SELCIDE model can still be used in the SELECT mode by omitting from the input file the specific lines which initiate the DECIDE option.

PROJECT APPLICATIONS

An effort is made herein to present examples of the application of SELCIDE to projects in the Nashville District. These examples represent a wide range of projects in terms of their size and complexity. Individual applications were selected to demonstrate the flexibility of the SELCIDE model. As will be noted later only one of the examples involves the DECIDE option. This is due to the fact that only one Nashville District project has selective withdrawal capability.

Bay Springs Lock And Dam

Bay Springs Lake is the northernmost impoundment on the Tennessee-Tombigbee Waterway. Bay Springs Lock and Dam was designed and built by the Nashville District and is operated by the Mobile District. Water released from the lake can originate from one of two sources: a low flow diversion channel used during prolonged periods of zero or limited lockages to augment downstream flows and release from the 25.6 m (84 ft) lift lock. Bay Springs Lock and Dam has no facility to make spillway releases. Lake elevations are controlled by Pickwick Lock and Dam on the Tennessee River whose pool is connected to Bay Springs Lake through the 43.5 km (27 mi) Divide Cut channel.

It was recognized during the design phase of the project that owing to the long retention time in Bay Springs Lake strong thermal stratification patterns would develop. The intake for the lock is located near the bottom and release of water from the lower layers of the lake would not meet downstream water quality objectives. It was determined that a submerged weir located in the lock approach would effectively skim the higher quality water from the upper layers of the lake.

Wilhelms (1976) used SELECT to evaluate the effects of different weir crest elevations and a range of lockage rates on the temperature and dissolved oxygen of lock releases. Results from the model were compared with an objective temperature band developed for Mackeys Creek under pre-project conditions. The investigation found that the objective temperature band could not be met all times of the year owing to a shortage of cool oxygenated water during the summer. This shortcoming was most evident for model runs simulating low lockage rates. The project was designed with the weir crest 6.1 m (20 ft) below summer pool. This location resulted in lock release temperatures warmer than the objective during certain times of the year; however, acceptable dissolved oxygen conditions were maintained during the entire simulation period.

Wolf Creek Dam

Wolf Creek Dam is located on the Cumberland River in Russell County, Kentucky, 742 km (460.9 mi) above its confluence with the Ohio River. Lake Cumberland which is the impoundment formed by Wolf Creek Dam provides flood control for nearly one-third of the Cumberland River Basin. Lake Cumberland is characterized by a rocky shoreline and a steep-sided channel where water depths exceed 30 m (100 ft) some 97 km (60 mi) upstream of the dam. Wolf Creek Dam was completed in 1950.

Lake Cumberland experiences an annual cycle of thermal stratification which begins in late March or April and usually persists until December. At this point the lake is mixed and remains mixed until stratification patterns again develop in the spring. Releases from Lake Cumberland are through six hydropower units whose penstocks are located in the lower layers of the lake. Outflow temperatures normally range from a low of 6°C (43°F) in the winter to a high of 15°C (59°F) in the early fall. Dissolved oxygen concentrations in project releases vary between a maximum of 12 mg/l in March to a minimum of 5 mg/l in October. The assured release of cold water with an acceptable oxygen content during the warmer months has led to the development of a put-and-take trout fishery in the Wolf Creek tailwater.

The Nashville District (1985) is currently evaluating the possibility of increasing the hydropower capability of Wolf Creek Dam by either uprating the six existing hydropower units or adding an additional four units or a combination of both. Water quality studies were performed to ensure that the proposed project will have the capability to meet water quality objectives. These objectives include maintaining existing water quality conditions in Lake Cumberland, meeting state water quality criteria in releases from Wolf Creek Dam, and maintaining the Wolf Creek tailwater as coldwater habitat suitable for a put-and-take trout fishery. If either the seasonal operation or withdrawal zone at Wolf Creek were significantly altered, both lake and outflow water quality could be affected. Since one of the fundamental assumptions of the overall study was that the seasonal operation of the project would not change, then the remaining concern was any impact on the withdrawal zone.

Potential impacts of the uprate and the new powerhouse on the withdrawal pattern were evaluated with SELCIDE. Although SELCIDE is not structured to handle the inclined penstocks found at Wolf Creek, a reasonable approximation of the withdrawal zone was computed by raising the centerline elevation of the penstocks a few feet in the input data. At Wolf Creek the centerline elevation of the existing penstocks is 189.1 m (620 ft). The effective centerline elevation, 190.9 m (626 ft), was determined through a trial and error process which involved entering observed temperature profiles and manipulating centerline elevations until computed outflow temperatures agreed with observed outflow temperatures. Since the size, elevation, and inclination of the penstocks in the proposed powerhouse alternatives will be the same as the existing penstocks, the effective centerline elevation will also be the same.

Once the outlet port characteristics were established, SELCIDE was executed for typical temperature and dissolved oxygen profiles for each month under different flow conditions. Selected flows which were modeled correspond to minimum and maximum flows for the existing powerhouse, the existing powerhouse with uprated units, and for the maximum proposed power plant expansion. Owing to the proximity of the proposed powerhouse to the left bank, which is perpendicular to the dam, there was concern that the withdrawal zone would be significantly altered due to the restricted angle of withdrawal. This was accounted for by specifying an angle of withdrawal of 90 in the input data for the new powerhouse.

Discharge distribution curves were generated with SELCIDE for each combination of hydropower capability and flow for the months of April, August, and October, which were considered typical spring, summer, and fall withdrawal patterns. Since minimum flows under existing and proposed conditions are virtually equal, computed withdrawal zones for these conditions were similar. The greatest difference was for the proposed new powerhouse, where the restricted angle of withdrawal tended to expand the withdrawal zone somewhat. Under maximum flows the withdrawal zone always extends from surface to bottom and the shapes for the three hydropower configurations were very similar. Changes in the withdrawal zone of the magnitude found in this study would have only minor effects on water quality. Since the proposed project will not significantly change either the seasonal operation or withdrawal pattern, there will be no significant changes to water quality conditions in Lake Cumberland or the outflows from Wolf Creek Dam.

Old Hickory Lock And Dam

Old Hickory Lock and Dam is a main-stem project located on the Cumberland River just upstream of Nashville. The Old Hickory project provides a limited amount of flood control in addition to its hydroelectric and navigation benefits. The Old Hickory pool extends upstream nearly 160 km (100 mi) to the point where it backs up to Cordell Hull Lock and Dam. Stratification normally persists in Old Hickory during the period June through December; however, it is generally limited to the lower third of the lake. The Old Hickory tailwater has been recognized as the critical point on the Cumberland River main-stem in terms of maintaining acceptable dissolved oxygen conditions.

Experience has shown that as long as the dissolved oxygen concentration in Old Hickory releases is above the state standard of 5.0 mg/l the downstream reaches of the river will maintain or increase this level. The dissolved oxygen of Old Hickory outflows is dependent on the residence time in the lake which in turn is dependent on how the upstream storage projects are operated. In order to model the effects of project operations on Old Hickory releases it was necessary to be able to predict release water quality for given lake conditions. A withdrawal zone study was performed using SELCIDE. The objectives of the study were to define the withdrawal zone of the lake for instantaneous flows, simulate outflow conditions from both power and spillway releases, and establish the effective withdrawal zone for predicted average daily flows.

The ability of SELCIDE to accurately predict outflow conditions for a relatively shallow main-stem project had not previously been tested in the Nashville District. Hydraulically, SELCIDE treats ports as point sinks; therefore, port dimensions are not used in the model's computations. The port dimensions are used to perform an internal check on the validity of the point sink assumption. This check is based on a ratio of the thickness of the theoretical withdrawal zone to the vertical dimension of the port. For the case of Old Hickory, while the vertical dimension is approximately 10 m (33 ft) and the lake is only about 23 m (75 ft) deep at the dam the point sink assumption was not violated. A minor program modification was required to compute the theoretical withdrawal zone limits.

SELCIDE was able to closely reproduce observed outflow temperature and dissolved oxygen values. Under all conditions, including when the lake was destratified, the upper and lower withdrawal zone limits extended from the surface to the lake bottom. When Old Hickory goes from one to two units power generation significantly more water is withdrawn from the upper layers of the lake and consequently a lesser percentage is withdrawn from the lower layers. The withdrawal zone flattens out when power generation goes from two to three units and is reflected in a more uniform withdrawal zone throughout the water column. There are only minor differences in the withdrawal zone when Old Hickory goes from three to the maximum of four units power generation. Spillway releases were modeled by treating the spillway gates which at Old Hickory are tainter gates and have a crest elevation some 11.9 m (39 ft) below normal pool as a second port vertically separated from those used for power generation.

Martins Fork Dam

Martins Fork Dam is located on Martins Fork approximately 16 km (10 mi) south of the City of Harlan, Kentucky. Martins Fork, Clover Fork, and Poor Fork come together near Harlan and form the headwaters of the Cumberland River. The Martins Fork project is unique to the Nashville District in many ways. In addition to being the only Nashville District project with selective withdrawal capability it is the smallest project, the only project without hydropower, and the only project with authorized water quality benefits. The Martins Fork project is operated to meet downstream water quality objectives. A guaranteed instantaneous minimum release of 0.14 cms (5 cfs) is made from the project. During certain times of the year the project may operate at the minimum release for periods of several weeks.

An objective temperature band was developed from data collected prior to construction of Martins Fork Dam. The project is now operated to reproduce this objective temperature band as long as acceptable dissolved oxygen conditions are maintained. The selective withdrawal system at Martins Fork consists of three gated conduits located at discrete elevations on the face of the dam. There are bypass lines associated with each gate for passing low flows.

The operation of the selective withdrawal system at Martins Fork is monitored on a daily basis with SELCIDE. A water quality monitoring system was incorporated into the design of the project. The monitoring system consists of four monitor intakes located at discrete elevations on the face of the dam. Information collected from these monitors is transmitted through the GOES satellite system to the Harris 1000 computer in the Nashville Reservoir Control Center. A program is then activated that pulls the objective temperature from a file, integrates the monitor data, and executes SELCIDE. The output from SELCIDE gives the flow distribution from the three ports needed to meet the objective temperature. This entire process is accomplished without input from the user. There is a GOES water quality monitor located a short distance downstream of the dam that is used to check the effectiveness of gate settings.

CONCLUSIONS

SELCIDE and its predecessor SELECT have been successfully applied to a wide range of projects in the Nashville District. These projects include tributary storage projects, main-stem navigation projects, a navigation lock, and a project with selective withdrawal. SELCIDE has been used to define the withdrawal zone in lakes, assess the impact of changes in project operation on the withdrawal zone, and to predict the quality of releases for both existing and future conditions. Outflow conditions predicted by SELCIDE compare well with field data. Typically, predicted temperatures are within 0.3°C (0.5°F) of observed values, although differences exceeding 1.0°C (1.8°F) have been observed. Differences between predicted and observed dissolved oxygen values tend to be somewhat greater. This can be explained in part by the differing turbine reaeration characteristics found among Corps of Engineers projects. These differences range up to about 1.5 mg/l, however differences of 0.5 mg/l are more the norm.

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SYSTEM SPILL ALLOCATION FOR THE CONTROL OF DISSOLVED GAS SATURATION ON THE COLUMBIA RIVER

Bolyvong Tanovan, PhD, P.E.[1]

[1]Chief, Water Quality Section, North Pacific Division
Portland, Oregon

ABSTRACT

Spill schedules for the Columbia/Snake River dams are developed and dispatched from the Corps North Pacific Division's Reservoir Control Center in Portland. During the spring and summer, spill is usually required not only because of high runoff in excess of project hydraulic capacity and reduced power demands but also to provide a safer passage route for fish past mainstem dams. This has often resulted in high dissolved gas saturation levels that are detrimental to fish. A system optimization procedure has been developed for use with an existing N_2 simulation model to determine where and how much to spill to minimize high localized gas problems and ensure the system power output is met. The combined analytical tool consists of three major components to simulate N_2 saturation in spillway-stilling basins, in reservoirs, and to control the procedure for developing optimal spill allocation. The model, code-named GASSPIL, is directly linked to a real-time data base system and can be used for planning as well as actual operational spill management.

INTRODUCTION

Spill at hydroelectric dams is routinely required when the river discharge exceeds what can be passed through the turbines in the dam powerhouse. In the Pacific Northwest, this usually occurs in the spring and early summer when snow in the mountainous areas melts and [illegible] runoff far in excess of the hydraulic capacity of the powerhouses, at a time when juvenile fish start to migrate toward the ocean. [illegible] during periods of lower runoff when power [illegible] sufficient to require the full turbine generating capacity.

In recent years, with the addition of the Pacific Northwest Electric Power Planning and Conservation Act of 1980, spill has been increased to improve the survival of fish migrating downstream through the spillway rather than the turbines. The fishery agencies and Indian Tribes may request that a given percentage of the river flow be spilled at prespecified projects during set days and hours of the juvenile fish migration season. The agencies and Tribes have [illegible] the way [illegible] spill is allocated [illegible] by providing a recommended [illegible]. These new considerations are part of the additional constraints [illegible] reservoir regulation have to take into account in scheduling spring and summer spill. They are expected to [illegible] for some time, pending installation of fish bypass and screening systems to divert fish from the turbines.

Spill is not an efficient use of water because of the power loss it entails. For maximum benefit to fisheries, its timing has to coincide with the movement of the ocean-bound juvenile fish and the upriver-bound adult fish. Night-time spill mostly benefits the juvenile fish and day-time spill, the adult fish. Furthermore, ill-conceived spill could be detrimental to those fish it was designed to benefit in the first place because of the dissolved gas supersaturation it creates. Water released over the spillway plunges deep into the stilling basin below the dam and causes entrapped air to dissolve into the water at levels higher than saturation. As a result, supersaturated N_2 gas will enter the juvenile fish, creating a condition similar to the "diver's bends". In severe cases, an embolism will eventually kill the fish.

This problem is not localized at or around any given dam per se, although some dams are more critical than others because of their heights, spill amount and stilling basin design. As most of the Columbia River and some of the Lower Snake River is a series of stair-step impoundments [illegible] the water moving downstream [illegible] sufficiently [illegible] of the [illegible] of the stilling basin of the upstream dam [illegible] entire [illegible] dissolved gas saturation [illegible].

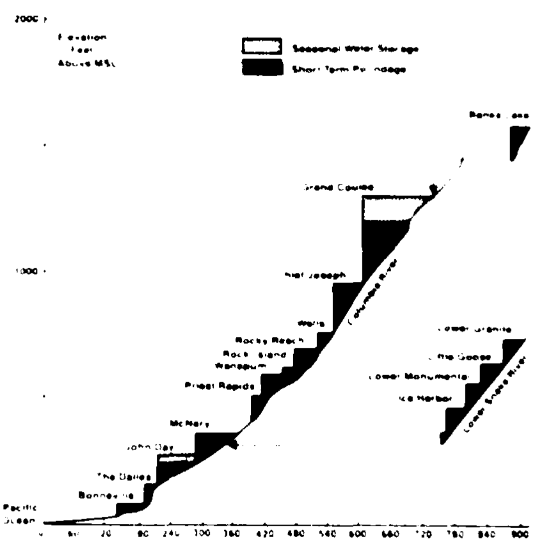
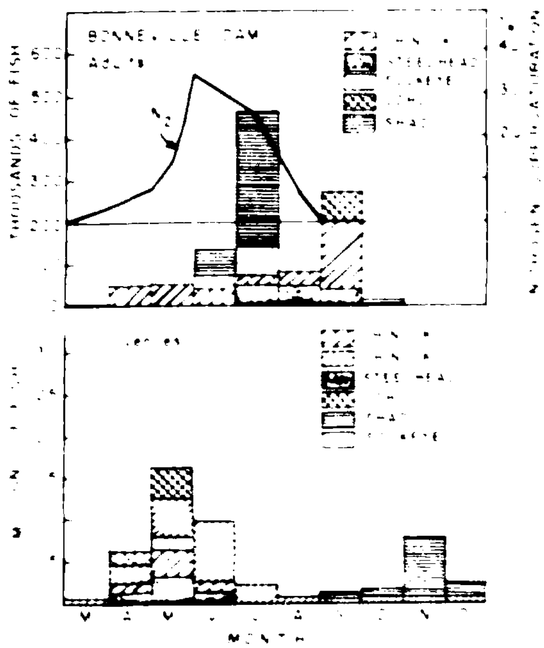


Figure 1. Profile of the Columbia River Main Stem and Lower Snake River



[illegible] Th[illegible] and [illegible] and [illegible]

[illegible] main [illegible] Northwest [illegible] Engineers [illegible] Harbor [illegible] and [illegible] single [illegible] the [illegible] and [illegible] operational [illegible] the fishery [illegible]

[illegible] monitoring net- [illegible] which 13 are [illegible] that collect and [illegible] to the Reservoir [illegible] explored several [illegible] and indirectly [illegible] saturation problem. A [illegible] implemented, some of them [illegible] using more [illegible] new fish bypass sys- [illegible] units, to reduce the [illegible] operating "flip-lip"

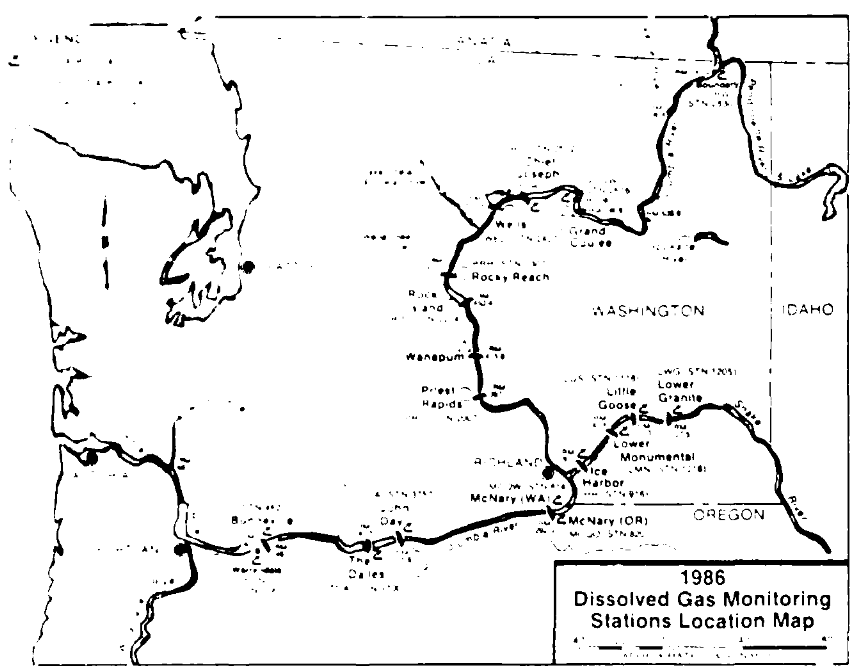
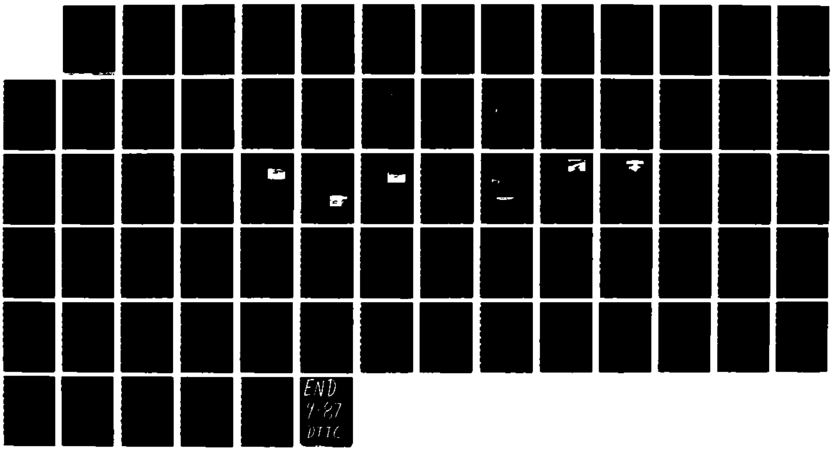
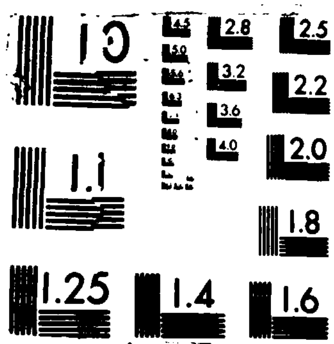


Figure 3

AD-A183 860 WATER OPERATIONS TECHNICAL SUPPORT PROGRAM PROCEEDINGS 2/2
CE WORKSHOP ON RE (U) ARMY ENGINEER WATERWAYS
EXPERIMENT STATION VICKSBURG MS ENVIR JUL 87
UNCLASSIFIED WES/MP/E-87-3 F/G 13/2 NL




1.0
1.1
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4.5
5.0

deflectors in the spillway-stilling basin, transferring power loads to high-dissolved gas producing dams, and altering spill patterns at individual projects, to minimize N_2 mass entrainment; and (d) collecting and transporting juvenile salmonids around the supersaturated river reaches, to avoid exposure to dissolved gas.

The general area of spill (and load) transfer is attractive because it involves no costly structural modifications and can be based on a relatively simple monitoring network. The GASSPIL model has been developed within this framework as a management tool to predict the power and dissolved gas impacts of the spill-for-fish passage and spill priority lists and, if necessary, to suggest other acceptable alternate spill schedules.

SPILLWAY-STILLING BASIN AND RESERVOIR SUBMODELS

The conceptual representation of the spillway-stilling basin combination and the reservoir used in GASSPIL is the same as that originally used by Water Resources Engineers, Inc. (1971) in the N_2 Gas Model for the Columbia River they developed for the Corps (See Figure 4 and 5). The spill is assumed to be uniform over the entire width of the spillway, and the reservoir is treated as slow moving stream fully mixed in its vertical and transverse dimensions -- a condition that fits the low-head, run-of-river type dams and reservoirs found in the lower Snake and Columbia system. Simulation of the complete system is accomplished through successive application of the spillway-stilling basin and reservoir submodels, as each dam and reservoir set is considered to behave as if it were functionally independent of all other dam and reservoir sets of the system.

Mathematically, the stilling basin concentration is expressed as a function of saturation concentration, forebay concentration, discharge per foot along the spillway crest, length of the stilling basin and entrainment coefficient. In the reservoir submodel, N_2 concentration in a given segment of the reservoir depends on time interval and rate of concentration change due to internal and external advection, air-water interface and instream eddy diffusion processes. Data requirements including the following:

Spillway-Stilling Basin

Time independent data

1. length of stilling basin
2. elevation of the stilling basin floor
3. average spillway gate width
4. specific gravity of air-water mixture, and
5. two empirical coefficients describing N_2 exchange in the stilling basin.

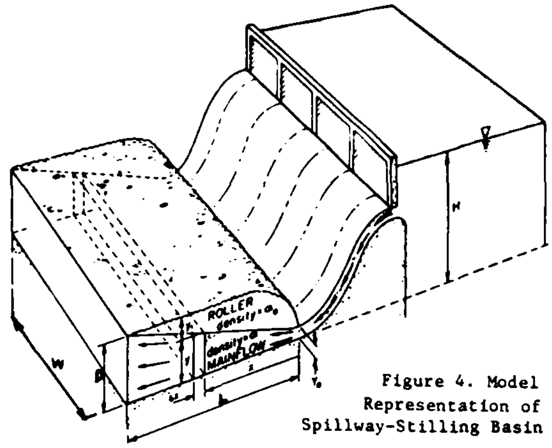

Figure 4. Model Representation of Spillway-Stilling Basin

Time dependent data

1. total hydraulic discharge at the project
2. spillway discharge
3. forebay water surface elevation
4. tailwater water elevation in the stilling basin
5. number of spillway gates open
6. forebay N_2 concentration, and
7. forebay water temperature

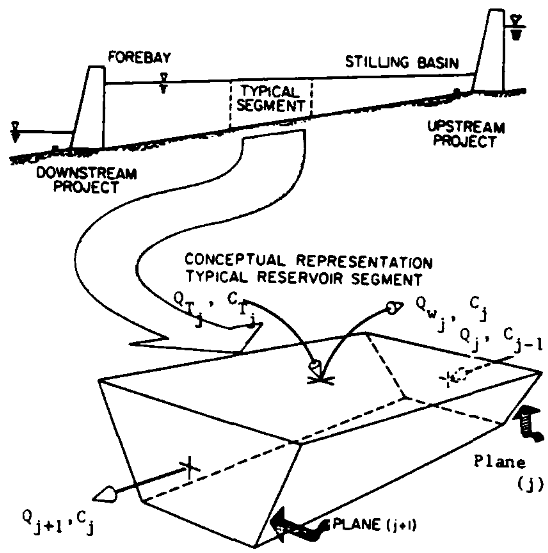

Figure 5. Model Representation of the Reservoir System

Tanovan

Reservoir Simulation

Time independent data

1. total reservoir length
2. volume-stage relationship for the reservoir
3. reservoir's effective bottom elevation at its upstream and downstream boundaries
4. ratio of top to bottom width for the assumed reservoir trapezoidal cross-section
5. system parameter relating effective water surface area to the planar projection of the reservoir's surface area, and
6. initial longitudinal N_2 profile in the reservoir

Time dependent data

1. total reservoir inflow
2. total reservoir outflow
3. water surface elevation at the upstream and downstream boundaries of the reservoir
4. average water temperature in the reservoir, and
5. concentration of N_2 entering the reservoir by upstream inflow.

The two submodels were modified by OTT Water Engineers, under contract to the Corps of Engineers, to allow for simulation/optimization starting with any dams and/or reservoirs in the system, in case the model user is only interested in a given reach of the stream. They can now operate in both historical and forecast modes using, as the case may be, either known input extracted directly from a real-time data base or artificial data corresponding to an expected hydraulic condition and a tentative spill schedule. In this manner, simulation runs are made possible to answer a wide range of "what if" questions of interest.

GENERAL APPROACH TO SYSTEM SPILL OPTIMIZATION

Given a fixed total discharge past each project in each time interval, the goal of the optimization is to distribute spill among the dams so as to minimize dissolved gas saturation while meeting a pre-specified total system power production for the same time interval.

A typical relationship between spill and the mass of N_2 entrained at each dam is shown in Figure 6, in which the quantity of spill is varied while all other variables (forebay and tailwater surface elevations, water temperature, and forebay N_2 concentration) are held constant. This relationship varies from one project to another, depending on the design of the dam's stilling basin and the way the spillway gates are brought into operation. It may sometimes have a saw tooth appearance when the resulting changes in hydraulic conditions are rather abrupt.

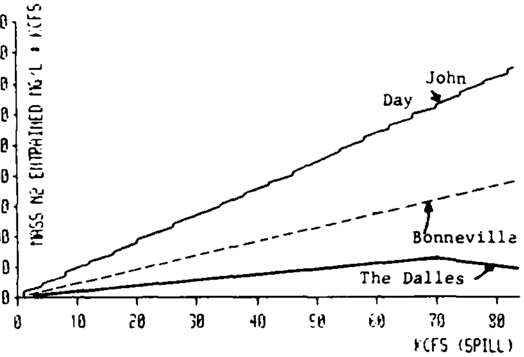

Figure 6. N_2 Entrainment versus Spill

Discontinuity in the functional relationship between spill and mass of N_2 entrained at some projects precludes the use of standard optimization methods such as linear programming. Furthermore, simulation is performed one project at a time and all time steps must be completed at that project before proceeding to the next downstream project. This is tantamount to assuming that N_2 concentrations in each time interval are independent. Therefore, since adjustment or redistribution of spill in any time interval at one project can produce adverse changes in N_2 concentration in later time intervals at downstream projects, the optimization amounts to the minimization of the maximum of the dissolved N_2 saturation levels at the head of each reservoir pool in each time interval. There is no direct account for time delay and decay function between the upper and the lower ends of the pool.

The system power demand used to drive the optimization can be specified in one of two ways. Power demand can be inferred from the hydraulic data input, assuming that the flow available for power production is the total flow less spill and less diversion, and using data relating discharge and head to power generation stored for each project in the model. Power demand can also be specified directly to the model as a separate input. For time steps shorter than a day, load curves giving hourly and daily power demand patterns are used to convert daily power demands into the required hourly values. Spill amount is adjusted so that system-wide power production matches demand. If power production does not match demand, then power production is increased or decreased at the various projects on a priority basis, the priority being another pre-specified input.

The first step of the optimization is to identify a single target project at which N_2 entrainment (and, hence, concentration) caused by the spill determined earlier is to be reduced. The project choosen is either (1) that with the highest level of saturation at which N_2 is being entrained, or (2) as specified in a spill priority list. The mass entrainment versus spill relationship for the selected project is used to estimate the change in spill required to reduce N_2 saturation to the target level, at the target project. In all cases, the change in spill is constrained to a maximum of 50 percent of the flow to avoid unduly large spill variations.

Reduction (increase) of spill at any project must be compensated by corresponding increase (decrease) in power generation flows since the total flow past each project in each computational interval is fixed. The increase in power production brought about by a decrease in spill at the target project is thus computed. To maintain constant system-wide power production for that time step, there must be a decrease in power production (increase in spill) at other projects in the system.

Redistribution of excess power capability is carried out by assigning additional spill (reduction in power production) at projects with N_2 saturation below target. Spill is increased until the excess power production is eliminated. If the increase in spill results in N_2 saturation above the target saturation, then the redistribution of spill is considered to be unsuccessful. In this case, the process of reducing N_2 saturation at a selected target and redistributing spill among other projects is repeated but with a smaller reduction in the N_2 levels at the target project.

When a redistribution of spill has been successful, the original spills stored in a temporary work array are updated for use in future iterations of an optimization run. This also provides the user with protection against inadvertent computer failure during long optimization runs and the option of further refining the system operation based on information from a previous run.

A sample summary output illustrating an optimization run with four iterations for 9 one-day time intervals and a limited six-dam system is provided in Table 1. It can be seen that even with four iterations, significant reductions in dissolved gas saturation levels have been achieved, particularly at John Day and The Dalles Dams. The lack of change at Priest Rapdis Dam and Ice Harbor is largely the result of the high N_2 concentration originally specified for the upstream boundary conditions.

Table 1. Summary Results of GASSPIL Run: N_2 % Concentration Below The Projects

Projects	Time Steps (day) 1	2	3	4	5	6	7	8	9
Priest Rapids	118.8	124.5	121.5	118.2	118.1	118.7	122.6	119.7	120.1
	118.8	124.5	121.3	118.2	118.1	118.7	122.6	119.7	120.1
Ice Harbor	115.4	112.8	114.5	125.4	130.2	129.9	128.9	128.8	128.8
	115.4	114.5	114.6	124.3	128.9	128.6	128.1	128.3	128.2
McNary	116.9	117.0	113.8	111.6	112.2	113.2	118.1	118.6	120.6
	116.9	117.0	116.1	113.6	112.6	113.3	117.8	118.3	120.2
John Day	125.0	126.1	126.7	125.6	124.2	124.3	123.5	121.5	121.5
	117.2	118.2	116.8	121.4	124.2	120.4	121.5	120.1	122.5
The Dalles	113.5	122.4	120.6	121.8	122.1	120.4	123.4	119.3	119.8
	112.4	115.7	113.1	113.9	118.5	120.2	123.4	117.7	118.5
Bonneville	118.7	118.9	117.5	120.8	120.5	123.3	124.1	122.1	123.5
	116.0	116.8	116.4	116.1	115.8	118.0	121.3	120.3	121.3

NOTE: The first line numbers are N_2 concentrations (as a percentage of the saturation concentration) below the specified project at the start of the optimization. The second line numbers are the corresponding numbers at the end of the 4th optimization.

APPLICATION OF GASSPIL MODEL

The following sequence of events that occurred in 1986 is given to illustrate the various steps involved in actual spill scheduling and GASSPIL model application. Assume today is Monday April 14, 1986.

(1) Inflow forecasts have been issued for the following week of April 15 - April 21 for Grand Coulee on the Columbia River and Lower Granite on the Snake River, as well as for the tributaries downstream from those two dams. As a result, inflows to all reservoirs in the system were assumed known for the next seven days.

(2) Based on their best forecast of power loads, the staff of the Bonneville Power Administration (BPA) planned the schedule of hydrosystem generation operations for the week of April 15-21, 1986. They looked at the inflow forecasts and computed the potential power output for the system. They realized they were not going to be able to use all that power for every hour of the day, every day of the week, especially during the night hours of the week-days and for the most hours during the week-end. The resulting surplus power, SMW, was computed based on the difference between the power generation potential, PMW, and the forecasted load, PLOAD.

(3) Enter the anadromous fish. As prescribed by the Power Planning Council, the Corps has agreed to spill at several dams to achieve a 90 percent fish survival at all but one of its lower eight Columbia/Snake River dams. Specific spill requirements to meet the Council's objective had been worked out in the winter of 1986 in collaboration with all agencies concerned. Details of the Corps' 1986 Juvenile Fish Passage Plan are summarized in Table 2.

Table 2. Summary of the 1986 Corps Spill Plan

1. Bonneville: Optional spill only. No sooner than date of 10% fish passage and for as long as spill is required at The Dalles. Limit operations of the second powerhouse as specified below.

   8pm-6am: Shut down the second powerhouse. Unlimited spill when flows exceed 120 kcfs.

   6am-8pm: Second powerhouse may be activated if needed to limit spill to 75 kcfs or for fishery research.

Flow	1st pwh	2nd pwh	spillway
0<Q<120	Q	-	-
120<Q<195	120	-	Q-120
195<Q<325	120	Q-195	75
325<Q	120	130	Q-250

2. The Dalles: 3.6 to 4 kcfs through sluiceway, 16hr/daytime during juvenile fish passage season. In addition, between 80% fish passage date (typically between Apr 15 - Jun 11 for spring run, and Jun 15 - Aug 21 for summer run) and Aug 15: as soon as 30,000 yearling (or subyearling) have passed John Day, spill 10% of instantaneous Q between 6pm-6am. This will protect 80% of the fish run.

3. John Day: Between dates of 80% of summer run juvenile fish passage (typically between Jun 7 - Aug 21) and Aug 15 (or when less than 30,000 fish for three consecutive days, or when 90% fish have passed the project): spill 36% instantaneous Q between 6pm and 6am.

4. McNary: No requirement (optional spill only).

5. Ice Harbor: Use 2.7 kcfs through sluiceway 24 hours/day during fish migration season.

(4) The fishery agencies and Tribes had been requesting spill-for-fish passage list almost every week to sepcify their recommended levels of spill at the projects. For the forthcoming week, their spill-for-fish passage was as summarized in Table 3, based on fish movement in the area.

Table 3. Requested Spill-For-Fish Passage April 14, 1986

Lower Monumental:	Starting April 15, spill 31% of daily average flow between 6pm-6am.
Ice Harbor:	Starting April 18, spill 31% of daily average flow between 6pm-6am.

The agencies and Tribes recognized that the spill levels in Table 3 exceed those assumed necessary to obtain the 90 percent survival objective established by the Northwest Power Planning Council. However, whenever there is a surplus of federal firm power, or when federal non-firm exists, they routinely request that the Corps make every effort to provide more generous spill.

A few days earlier, on April 10, the agencies and Tribes have already submitted a spill priority list for distribution of surplus spill. That list (See Table 4), also based on natural and hatchery fish movement, was still in effect at this time. It was to be implemented only after spill-for-fish passage levels have been satisfied.

Table 4. Spill Priority List April 10, 1986

Priority	Dams	Spill Limits
1	Lower Monumental	50% Q
2	Ice Harbor	50% Q
3	John Day	50% Q
4	The Dalles	50% Q
5	Bonneville	75 kcfs (daytime) no limit (night-time)
6	The Dalles	no limit
7	Lower Monumental	no limit
8	Ice Harbor	no limit
9	McNary	50% Q
10	Lower Granite	50% Q
11	Little Goose	50% Q

(5) The Reservoir Control Center's staff, in the North Pacific Division's Water Management Branch, reviewed the agencies' recommended spill levels and priorities in the light of other operational constraints. Final decisions had to account for these recommendations, the Corps committed spill percent, dissolved gas saturation, flood control requirements, size of migrating fish runs, etc.

A few potential problems surfaced because the agencies' recommended spill for fish passage levels exceeded the Corps' commitment; these spill levels could also generate inadmissible dissolved gas saturation levels. Likewise, the spill priority list needed to be checked as to its practicability and system impacts on power and dissolved gas.

The basis for decision-making included best predictions on the impacts of the proposed spill schedules, using analytical tools such as the GASSPIL Model and other means to determine how much the spill should be at the dams desired to help fish migration.

The needed input for GASSPIL runs to provide these predictions included:

a. Initial TDG percent at the forebays of all dams (at time T=1).

b. Inflows at each dam for every time step of the week long study period.

c. Predicted forebay and tailwater elevations at all projects. These were assumed approximately constant during the study period since run-of-the-river dams basically pass all inflow and, therefore, sustain no storage change.

d. Daily load projections in MW for T=1 to T=7. Hourly loads for each hour and each day of the week will be computed by the model using the built-in load shapes. They will be used as load requirement in the optimization.

e. Minimum spill percent at each dam.

f. Spill priority list, for use in controlling the sequence of the model optimization and specifying the maximum spill levels.

The model output first included a summary condition reflecting the input, especially the flows, loads, surplus spill amounts, spill-for-fish passage and spill priority lists, and the power generation potential. At the end of each optimization iteration, a list of maximum dissolved gas saturation levels reached in each pool with date and time of their occurrence, and the spill at each dam was given. When the run was completed, the series of standard outputs developed by the original model for the dams and the reservoirs followed. Also, the spill amount (in percent of the project total discharge) for each dam was provided, along with the maximum dissolved gas saturation levels.

(6) Based on all the information developed above and following consultation with BPA and the Corps staff biologists, appropriate spill schedules were finalized. As it turned out, the Reservoir Control Center decided in this case not to implement the agencies' recommended spill-for-fish-passage levels, leaving them at the Corps-committed spill levels. It also modified the spill priority list to put spill at Chief Josepn and Grand Coulee prior to spilling at McNary and Lower Granite, which have fish transportation facilities. The agencies and Tribes were informed of these decisions, and necessary instructions subsequently teletyped to the projects for implementation.

CONCLUSION

The North Pacific Division of the Corps of Engineers has direct responsibility for efficiently managing the water resources of the Columbia River basin, including the task of ensuring that spill at its dams does not generate dissolved saturation levels in excess of existing state and federal standards. The multi-purpose, often conflicting, nature of the water uses in this region makes water resources allocation a real, day-to-day challenge. A model such as GASSPIL that can quickly predict the dissolved gas saturation and system power impacts of the fishery agencies and Indian Tribes' spill-for-fish passage requests and spill priority lists (or other alternate spill schedules) is a very useful management tool. It provides a better understanding of how the Columbia System works and furnishes the basic justifications needed to support a Corps spill schedule that may be different from that requested by the agencies. Efforts will continue to be made to refine the model to make it an even better tool for operational use.

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Roesner, L.A. and Norton, W.R. 1971. "A Nitrogen Gas (N_2) Model for the Lower Columbia River", Report No. 1-350, Water Resources Engineers, Inc. Walnut Creek, CA.

SIMULTANEOUS MULTIPLE-LEVEL WITHDRAWAL THROUGH SINGLE WET WELL STRUCTURES FOR DOWNSTREAM WATER QUALITY MAINTENANCE

Stacy E. Howington[1]

[1]Research Hydraulic Engineer, Waterways Experiment Station, Vicksburg, MS

ABSTRACT

The concept of selectively withdrawing from multiple levels simultaneously in a stratified reservoir is presented and its usefulness discussed. The problem of predicting the flow distribution between the withdrawal levels under various operating conditions is outlined. A brief theory from previous work is expanded to increase the applicability of the technology. The application of the expanded theory is shown and the results are compared to observed data.

BACKGROUND

Thermal stratification is a common occurrence in many lakes and reservoirs, especially during the warmer months when inflows and outflows are generally low, heat influx is high, and other climatic components such as wind are favorable for stratification development and maintenance. Density stratification accompanies the thermal stratification due to the physical properties of water. An increase in temperature results in a decrease in density for water above 4° C. Therefore, the surface waters in a reservoir accepting heat from solar radiation and the atmosphere become lighter and tend to remain at the surface. Conversely, the bottom waters do not receive as much heat and therefore remain more dense. Density stratification limits the vertical movement of water within the reservoir since additional energy is required to overcome buoyant forces. This limiting of vertical motion may also cause chemical stratification as the mixing between layers is reduced and the hypolimnetic water no longer exchanges with the surface.

The effects of density stratification, the limited vertical movement of the water, may be used in a positive manner. A specific quality of water might be selected from the vertical variety of qualities and released, while not withdrawing from the entire vertical range in the pool. This technique, which is very common in release water quality maintenance, is known as selective withdrawal.

Sometimes, one vertical level of withdrawal from a stratified pool is adequate, but often two or more levels are needed. This situation may arise when single-level operation cannot withdraw the desired quality and quantity of water. Traditionally, simultaneous multiple-level withdrawal has been accomplished through the use of dual wet well outlet structures. In these cases, one level of withdrawal is selected in each wet well, and the two qualities are mixed in the release conduit and stilling basin downstream of the individual wet well service gates.

Dual wet well type operations, however, are not always possible. In recent years, the addition of hydropower to existing outlet structures has become an attractive source of energy. In many cases, this process involves the transfer of flow control from the service gates to the turbine downstream. To continue multiple-level operations as before would require the mixing of water from multiple withdrawal levels upstream of a single point of flow control. For the purpose of this publication, this process will be referred to as blending. The difficulty associated with blending is that the amount of flow withdrawn from each level is not strictly controlled and can be influenced by the density stratification in the pool. The selective withdrawal characteristics of the structure will not be altered, only the amount of flow which is withdrawn through each port.

Another situation in which blending can occur is multiple-level withdrawal in a single wet well structure. Single wet well structures have been employed in some cases for economic reasons or in situations where individual flow control on each potential simultaneous level of withdrawal is not feasible. These situations include selective withdrawal addition to existing hydropower facilities and the use of greater than two levels of simultaneous withdrawal in a single wet well.

Blending operations may even be desired at dual wet well projects with separate flow controls on each well to gain additional flexibility. Potentially, more flow could be passed through the water quality system (avoiding use of the flood-control outlet for some discharges) if more than two ports were used. Also, additional capability might be gained in meeting downstream quality objectives by using more than one level of withdrawal in each of the individual wet wells.

THEORY

Single wet well blending can be approached from an analytical standpoint as seen in Howington (1986). Consider the simple case in Figure 1a. This is a single wet well structure with two ports open. The density stratification is perfectly two layer represented by ρ and $\rho+\Delta\rho$. One open port resides in each of the two homogeneous layers. The same pattern of density stratification exists in the wet well as in the pool. The outlet from the wet well is located at the bottom of the structure. Figure 1a. represents the "no-flow" condition.

When a small discharge is initiated from the structure, the system will respond by withdrawing the total discharge through the lower port as shown in Figure 1b. This is due to the buoyancy effects of the pool density stratification. The

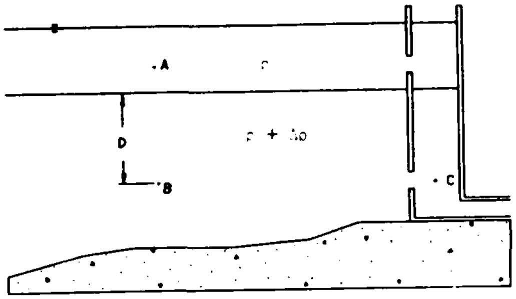

Figure 1a. No discharge

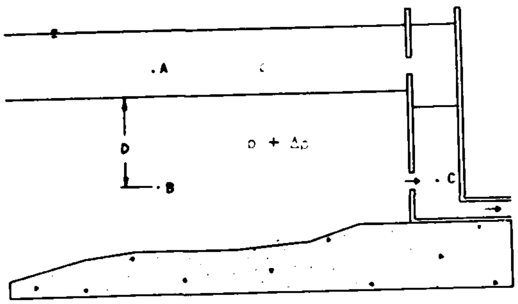

Figure 1b. Small discharge

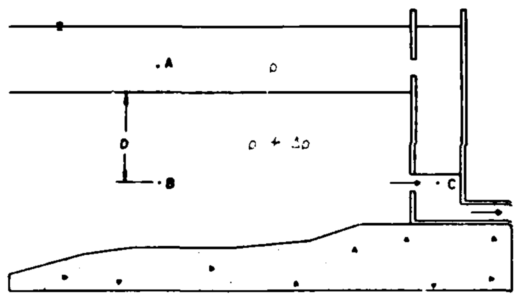

Figure 1c. Critical discharge

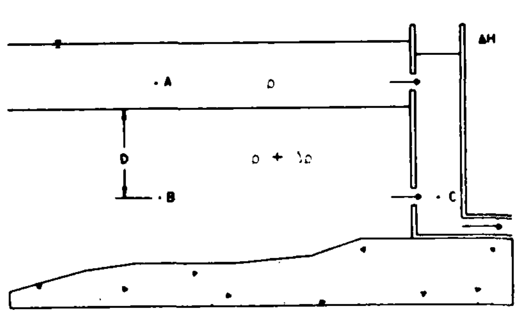

Figure 1d. Blending

Figure 1. A single wet well system responding theoretically to increasing discharge

lighter surface water is buoyed up by the more dense bottom water. More work would be required to pull the lighter water down than to withdraw the entire amount through the lower port. The thermocline (which is also the pycnocline) will be depressed in the wet well until an equilibrium has been reached. The drop of the thermocline in the wet well is a reflection of the energy loss as flow enters the lower port. This small discharge will induce only enough flow through the upper port to fill the void created by the receding thermocline. At this equilibrium condition, no flow from the upper port will be released from the system. The water-surface elevation in the wet well will be the same as the reservoir pool elevation. This situation is referred to as density blockage since the density effects prevent flow through the upper port. This phenomenon has been observed in a laboratory environment using an approximately two-layer stratification with the upper layer dyed.

As the total discharge is increased, so is the energy loss across the lower port, and the thermocline in the wet well is further lowered. This trend obviously cannot continue beyond the point where the thermocline reaches the elevation of the lower port without inducing flow from the upper port. This critical equilibrium point, seen in Figure 1c, is termed incipient blending and the associated discharge is termed critical discharge. At this equilibrium point, the potential energy associated with the stratification is exactly offset by the energy lost by the flow entering the lower port.

As the discharge is further increased, blending occurs with some amount of flow coming through each of the two port elevations (Figure 1d). However, density stratification still plays an important role in downstream quality maintenance since it affects the flow distribution between the port elevations.

To describe the processes involved in blending, the Bernoulli equation, as seen in Brater and King (1976), is employed. For flows less than or equal to critical discharge, the Bernoulli equation can be written from a point in the pool (point B in Figure 1c) to a point in the wet well at the elevation of the lower port (point C). For flows less than critical discharge, the equilibrium elevation of the thermocline in the wet well can be determined from this formulation. If the thermocline is established at the elevation of the lower port, the critical discharge can be determined from the following application of the Bernoulli equation:

$$ZB + \frac{PB}{WB} + \frac{VB^2}{2g} = ZC + \frac{PC}{WC} + \frac{VC^2}{2g} + HL_{B-C} \qquad (1)$$

$$HL_{B-C} = \Delta\rho * D \qquad (2)$$

$$Q_C = \sqrt{\frac{2g*A_\ell^2*HL_{B-C}}{K_\ell}} \qquad (3)$$

where

ZB, ZC = elevation of points B and C referenced to datum, m

PB, PC = pressure at B and C, N/m^2

WB, WC = specific weight at B and C, N/m^3

VB, VC = velocity at B and C, m/sec

g = gravitational acceleration, m/sec^2

HL_{B-C} = head lost between points B and C, m

$\Delta\rho$ = density difference between the layers, kg/m^3

D = distance between the thermocline and point B, m

Q_c = the discharge at incipient blending, m^3/sec

A_ℓ = lower port area, m^2

k_ℓ = head loss coefficient across lower port

For flows greater than critical discharge, Bernoulli's equation can be written through each port. Referring to Figure 1d, Bernoulli's equation can be written from point A, which is at the elevation of the upper port, to point C and from point B to point C. Given this, and using Equation 1, the ratio of flows between the ports can be determined from the following:

$$ZA + \frac{PA}{WA} + \frac{VA^2}{2g} = ZC + \frac{PC}{WC} + \frac{VC^2}{2g} + HL_{A-C} \quad (4)$$

$$HL_{A-C} = \Delta H \quad (5)$$

$$HL_{B-C} = \frac{\Delta\rho * D}{\rho + \Delta\rho} + \frac{HL_{A-C} * \rho}{\rho + \Delta\rho} \quad (6)$$

where

HL_{A-C} = head loss between points A and C, m

ΔH = water-surface drop in the wet well, m

ρ = density of the upper layer, kg/m^3

Equation 6 indicates that the head loss across the lower port must always be greater than the head loss across the upper port by approximately the constant amount of potential energy associated with the particular density stratification.

To determine the flow distribution between the ports, an iterative technique must be employed. First, an estimate of the water-surface drop in the wet well is made. This equals the head loss across the upper port. Equation 6 is used to determine the head loss across the lower port. The known relationship between head loss and discharge for each port can then be used to determine the individual port flows. The two flows are computed and summed, as seen in Equation 7, and compared to the known total discharge. The estimate of upper port head loss is adjusted until the summed discharges agree with the known total discharge.

$$Q = \sqrt{\frac{2g * A_u * HL_{A-C}}{k_u}} + \sqrt{\frac{2g * A_\ell * HL_{B-C}}{k_\ell}} \quad (7)$$

where

Q = total discharge, m^3/sec

A_u = upper port area, m^2

k_u = head loss coefficient across upper port

A very similar procedure can be followed for the analysis of blending in a linearly or an arbitrarily stratified environment as seen in Figure 2. The only difference arises in the

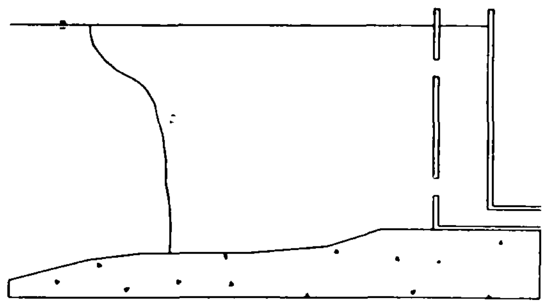

Figure 2. A nonlinear, non-two-layer density stratification.

development of the density potential energy term. Once again, Bernoulli's equation can be written across the lower port for flows less than or equal to critical discharge. At incipient blending, Equation 8 results. For flows greater than the critical discharge, the resulting formulation is given in Equation 9. The discharges through the individual ports can be determined using the same general procedure as used before in Equations 6 and 7.

$$HL_{B-C} = \frac{-1}{\rho_B} * \int_B^A \rho(z)\, dz \quad (8)$$

where z = elevation referenced to datum, m.

$$HL_{B-C} = \frac{-1}{\rho_B} * \int_B^A \rho(z)\, dz + \frac{HL_{A-C} * \rho_A}{\rho_B} \quad (9)$$

where

ρ_B = density at point B, kg/m^3

$\rho(z)$ = density as a function of elevation, kg/m^3

ρ_A = density at point A, kg/m^3

A more meaningful representation of the density potential energy term can be seen in Figure 3. The potential energy is computed by vertically integrating the density of the water in the pool between the two port elevations. The area between the reservoir density and the upper port density and between the two port elevations (shaded in the figure) constitutes the density potential energy term for this situation. Figure 3 represents the density effect for a two-port

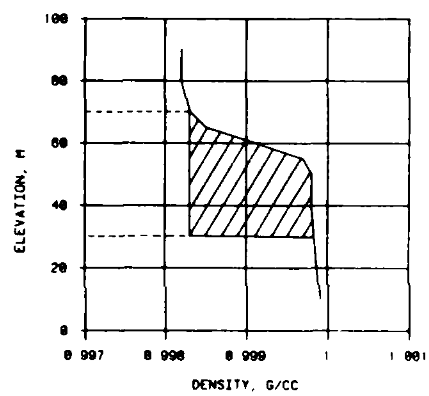

Figure 3. Graphical representation of the density potential energy term.

Howington

configuration with one port at elevation 30 m and the other at elevation 70 m.

A more complex situation can develop with the simultaneous operation of more than two levels of ports in a single wet well under a stratified condition. This problem can also be approached using Bernoulli's equation. In this situation, a stratification pattern exists within the wet well under blending conditions. This pattern is caused by the withdrawal of a potentially different density through each of the open ports which can influence the flow distribution significantly.

The water-surface drop in the wet well is approximately equal to the head loss across the upper port. It is also an unknown component in the head loss terms for every other port within the single wet well. Only one water-surface drop exists which will provide a flow distribution consistent with both the theory and continuity.

The solution method to determine the flow distribution is iterative and very similar to that used in the two-level analysis shown earlier. The head loss across the highest open port (call this port level 1), which approximately equals the water-surface drop in the wet well, is estimated. Then, the head loss across the next lower open port (port level 2) is computed. From the theory, the head loss across the lower port is known to be approximately equal to the port level 1 head loss plus the energy associated with the stratification difference between the wet well and the pool between port levels 1 and 2. The port level 3 (the next lower open port) head loss is equal to the port level 2 head loss plus the density potential energy between port levels 2 and 3. This process continues until all the losses are known. The individual flows are computed, summed and compared to the known discharge. The water-surface drop estimate is adjusted and the process repeated. The equation which is solved follows. The head loss term in Equation 10 includes the density potential energy associated with the stratification difference between the wet well and the pool between withdrawal levels n and n-1.

$$Q = \sum_{n=1}^{NP} \sqrt{\frac{2g*A_n^2*HL_n}{k_n}} \qquad (10)$$

where

- NP = number of simultaneous withdrawal levels
- A_n = area of port n opening, m^2
- HL_n = head loss across port n, m
- k_n = head loss coefficient across port n

Once the total discharge has been matched in this iteration, the individual port flows are then determined, as follows, knowing the port head loss.

$$Qn = \sqrt{\frac{2g*A_n^2*HL_n}{kn}} \qquad (11)$$

where Qn is the discharge through port level n, m^3/sec.

The process described in Equations 10 and 11 is theoretically applicable for any number of simultaneous withdrawal levels (including two) within a single wet well with any stratification pattern in the pool.

ASSUMPTIONS AND LIMITATIONS

In developing the preceding theory, several assumptions were necessary.

a. Bernoulli's equation is applicable.

b. No hydraulic blockage exists.

c. Losses other than entrance losses are negligible.

d. The density of the water in the pool at the center-line elevation of the port is the density that will be withdrawn through that port.

e. Lower port velocity jet mixing is unimportant.

The Bernoulli equation has some assumptions which are inherent in its formulation. When the equation is applied along streamlines, as it was is this application, the fluid flow must be incompressible and steady. These assumptions are valid for this application.

The existence of hydraulic blockage was proposed as a potential problem in blending. It was proposed that hydraulic blockage might theoretically occur under stratified or unstratified conditions when the velocity jet through the lower port could create a turbulent zone in the wet well which would be impassable by flow from the upper port. Hydraulic blockage, to our knowledge, has not been observed in outlet structure operations.

Energy losses other than entrance losses are assumed to be negligible. Entrance losses include contraction and expansion losses associated with flow from the pool, through the port, into the wet well, and with velocity jet impingement turbulence in the wet well. The only losses which are being neglected are those frictional losses encountered by the flow as it passes from the upper port elevations downward through the wet well. In most cases these losses are minor, making this an appropriate assumption.

Another assumption is that the density of the water at the elevation of the port is the density of the water which will be withdrawn through that intake. From selective withdrawal, it is known that this is generally, but not always, a good assumption. This potential problem will be accounted for once the blending technology is incorporated with selective withdrawal technology.

The last assumption is that the turbulence associated with the velocity entering the lower port does not affect blending. This is true in most situations, but the theoretically computed critical discharge will be affected. The effects of this assumption can be seen in Figure 4. The theory assumes that the thermocline in the wet well will drop to the elevation of the lower port undisturbed. However, the turbulent mixing caused by the lower port velocity jet will break up the thermocline at some discharge which is lower than the theoretical critical discharge, and therefore induce a small amount of flow through the upper port. The theoretical critical discharge is a conservative estimate of the amount of flow necessary to overcome density blockage since the actual critical discharge is always lower.

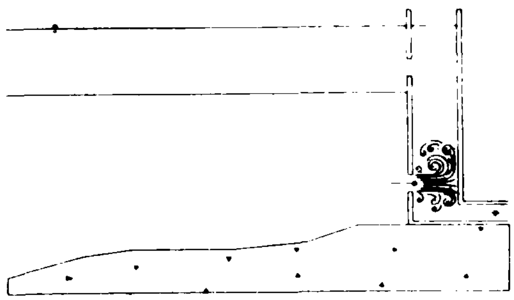

Figure 4. Turbulent mixing associated with the lower port velocity jet impinging on the walls of the wet well

RESULTS

The effects of density can be illustrated by Figure 5. This figure shows the flow through the lower port of a two-port, single wet well

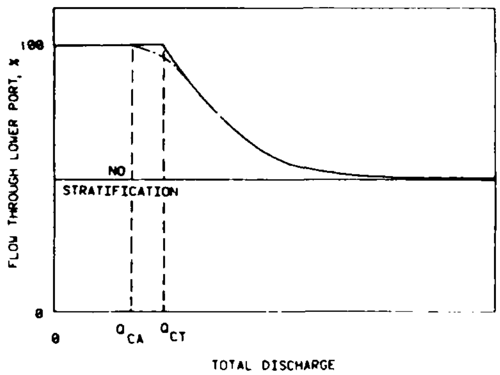

Figure 5. The effects of density for one stratification pattern and port configuration with varying discharge

structure in relation to the total discharge from the structure for a single stratification condition. This figure was developed using two ports of equal areas and constant, equal loss coefficients. The horizontal line labeled "no stratification" indicates equal flow (50 percent flow) between the two ports for the homogeneous density condition at all flow rates. The solid curve represents operation in a density-stratified environment. The density effects at any discharge can be seen as the distance between the solid curve and the equal-flow line since the curve represents density stratification and the equal-flow line represents homogeneity. At low flows, the density impact is large with density blockage occurring. In the figure, density blockage is represented by the horizontal portion of the solid curve which begins at zero discharge and extends to the theoretical critical discharge, Q_{CT}. When the flow surpasses Q_{CT}, flow through the upper port begins and the flow through the lower port quickly drops below 100 percent.

As the total discharge is increased, the impacts of density decrease and the percentage of the total flow passing through the lower port drops. This continues until the constant effects of density are masked by the [illegible] head loss terms. In the figure [illegible] the higher discharges where the equal-flow [illegible] the solid curve converge at [illegible] percent flow through each port. The actual [illegible] under stratified conditions, as observed in the laboratory and in the field, is roughly indicated by the dashed line which separates from the solid curve at the actual critical discharge, Q_{CA}, and rejoins it at a flow rate slightly higher than Q_{CT}. The actual critical discharge differs slightly from the theoretical critical discharge, but the theory applies very well over the remaining range of discharges. The theoretical critical discharge will always be a conservative approximation in that the actual discharge necessary to overcome density blockage is always slightly less.

Head loss is obviously important in describing the blending process. Therefore, the prospect of using partial port closure (port throttling) to control head loss, and thereby blending, was investigated. Figure 6 shows example results. The solid curve represents the same condition seen in the previous figure. The dashed curve represents the same condition as the solid curve, only with the lower port throttled by 50 percent. The head loss term was assumed not to vary with gate closure to demonstrate more effectively the effects of throttling. The horizontal lines, as before, represent the unstratified conditions.

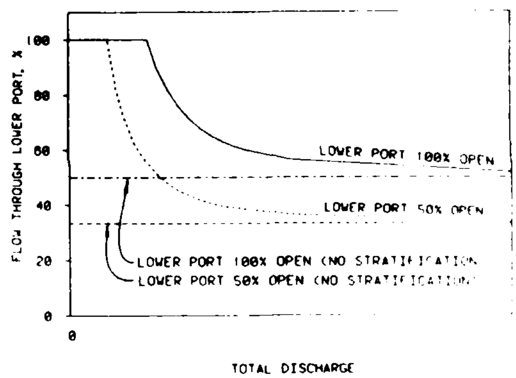

Figure 6. Total discharge versus lower port discharge for the full open and the throttled conditions

Port throttling results in a dramatic decrease in the critical discharge. This can be seen in the figure by the dashed curve, which drops below 100 percent flow through the lower port at a much lower total discharge than that required by the solid curve. Another result is an overall decrease in the effects of density in the intermediate discharges. Intermediate discharges are those between critical discharge and the discharge at which density influences become negligible. This is evidenced in the figure by the distance between stratified and unstratified conditions. For the same intermediate discharge, this distance is smaller for the throttled condition than for the full open condition. The reduction in the critical discharge could be anticipated. The head loss for the throttled condition is much greater than that of the unthrottled condition for the same total discharge. Therefore, the density potential

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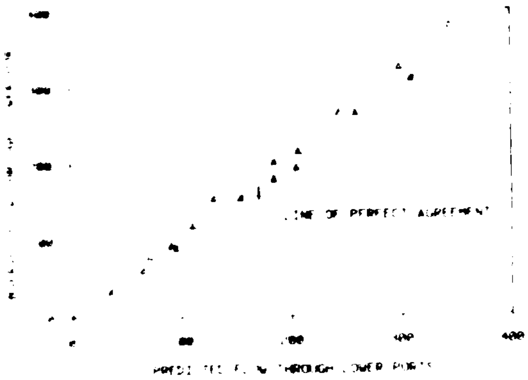
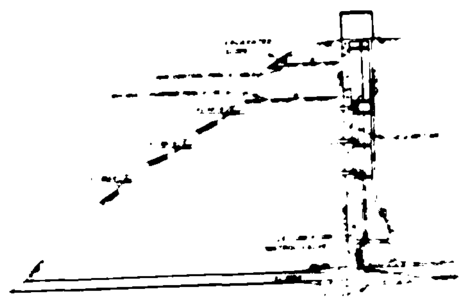


[illegible] predicted versus observed [illegible] through the lower ports [illegible]

[illegible] the figure indicates, the [illegible]

A [illegible] different [illegible] from that at [illegible] was encountered at Warm Springs Dam [illegible] tests were [illegible] the structure is located [illegible] the three water quality intake [illegible] considerable distance from the [illegible] wet well.



[illegible] Warm Springs Dam single wet well intake structure

The structure has the [illegible] moderately strong [illegible], the flow was established [illegible] with the upper and lower water quality intakes open. Although [illegible] the minimum discharge for [illegible] was not [illegible] because, even at [illegible] discharge, the energy [illegible] the flow [illegible] entered the lower intake, passed through the [illegible] and the valve, and entered the wet well [illegible] to overcome the potential energy [illegible] quantitative results from [illegible] presently available, but a qualitative analysis indicated general agreement with the [illegible] established trends. As [illegible] was increased for a particular test, the percentage of flow through the lower port decreased. This was evidenced by the increase in the release temperature. Port throttling had the

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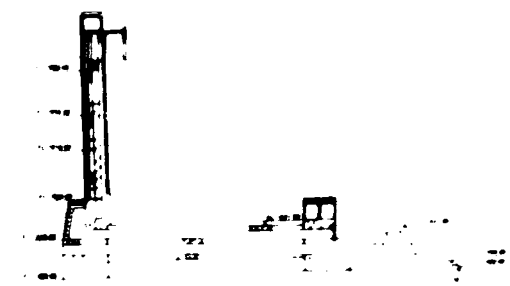

[illegible] single wet well intake structure

[illegible] meeting the desired [illegible] port elevations.

An example of the predicted versus the observed discharges [illegible] shown in Figure [illegible]. The [illegible] the predicted flow ratio between the [illegible] expected impact as well. The temperature of the releases increased as the lower port was throttled for constant discharge.

CONCLUSIONS

The theory for single wet well blending has now been expanded to account for arbitrary stratification patterns and for multiple (greater than two) simultaneous levels of ports operating. The limitations of the theory appear to be minor. Turbulence in the wet well does not appreciably affect blending except in a very limited range of discharges near critical discharge. The implications of this discrepancy appear minor as well.

The theory applies well in all cases tested thus far. These cases include a variety of sizes and types of intake structures. For those situations in which quantitative comparisons are possible, the theory is accurately predicting flow distributions between the port elevations. In those cases where only qualitative analyses are possible, the trends of the observed data and the theory compare favorably. Further testing of the theory is planned under an ongoing research effort. Situations are being sought in which the theory does a less than adequate job of prediction so that these can be accounted for in the formulation.

Head loss is the driving mechanism for blending. The generation of head loss by port gate throttling adds a great deal of flexibility to blending operations. However, creation of increased head losses is contrary to hydropower purposes and, under extreme conditions of port throttling, head loss can potentially become significant. The result could be a situation which requires compromise between water quality and hydropower interests. This would be most likely to occur when the flow distribution desired is predominantly through the upper port and the stratification is strong. Port throttling should be considered in the design of new intake structures or in the modification of old structures if blending capability is needed. Throttling of the gates is not mandatory in blending operations but is highly desirable from a water quality management perspective.

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SINGLE WET WELL BLENDING AT APPLEGATE LAKE, OREGON

Jeffrey D. Hanson[1] and Richard A. Cassidy, P.H.[2]

[1]Hydrologist, USACE, Portland District, Portland, Oregon
[2]Supervisory Environmental Engineer, USACE, Portland District, Portland, Oregon

ABSTRACT

In 1983, the Portland District performed experimental reservoir releases using one half of the dual wet well selective withdrawal system at Applegate Lake to evaluate the potential for blending water in a 1.5 m (5 ft) diameter single wet well. The reservoir water located in the epilimnion was approximately 18°C (65°F) and the hypolimnion water, 27 m (90 ft) deeper in the reservoir, was near 9°C (49°F). Successful water temperature blending was achieved for two days with a wide variety of gate opening combinations, at the test release flow of 14.2 m^3/s (500 ft^3/s).

In 1986, an operational study was performed to test single wet well mixing at 4.8 m^3/s (170 ft^3/s). A constant water temperature near the target release temperature of 12.8°C (55°F) was maintained for eight days. The reservoir water temperatures at the operating ports were 16° to 18°C (60° to 65°F) near the surface and 7° to 8°C (44° to 46°F) near the bottom.

INTRODUCTION

Selective withdrawal systems provide the capacity to remove water from one of a number of different levels in a reservoir, or from a combination of two levels, to utilize the best quality water in a reservoir for discharge downstream.

Conceptually, the intent of a selective withdrawal system is to provide the flexibility to choose better quality water for release compared to a traditional fixed, low-level withdrawal system. For instance, the temperature of water released from a reservoir can be more closely controlled to resemble natural temperatures using a selective withdrawal system. Control of the temperature of water being released, therefore, can have a significant long-term effect on the biota of that regulated stream.

The use of the dual wet well selective withdrawal structure at the Portland District's Applegate Lake has provided good control of release water temperature. However, a few operating conditions have occurred where water temperature requests were greater than the dual wet well system could provide. The Reservoir Regulation and Water Quality Section has tested the capability of blending water from two levels in one of the Applegate Lake selective withdrawal wet wells for two different operating conditions.

WATERSHED DESCRIPTION

The Applegate River Basin is a subbasin of the Rogue River, a coastal stream in southern Oregon (Figure 1). The confluence of the two rivers is approximately 8 kilometers (5 miles) west of Grants Pass, at the upstream terminus of the 135 km (84 mi) portion of the Rogue River that is designated a national "wild and scenic" river.

Applegate Lake is located on the Applegate River, 74.5 km (46 mi) upstream of the Rogue River, near the California-Oregon border. It is a 1.01 x 10^8 m^3 (82,200 ac-ft) multiple-purpose project authorized for flood control, irrigation, fish and wildlife, recreation, and water quality enhancement in the Applegate Valley.

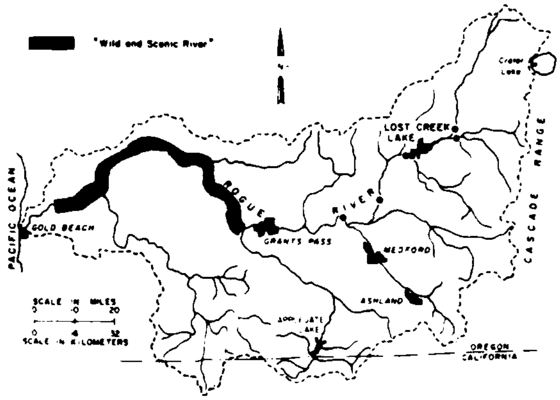

Figure 1. Project Location, Rogue River Basin, Oregon

The project has a butterfly-shaped watershed and controls the rainfall and snowmelt runoff from approximately 580 km^2 (225 mi^2) of the Applegate River Basin. The headwaters of the Applegate Lake watershed are at the heavily timbered crestline along the Siskiyou Range of California and Oregon, usually above 1,525 m (5,000 ft), National Geodetic Vertical Datum (NGVD). Streams have steep gradients varying from 4.2 to 6.3 m/km and flow through narrow channels cut deeply into intrusive rock.

The Applegate River Basin experiences mild, wet winters and warm, dry summers. A predominately westerly flow of moist air from the Pacific Ocean during the winter produces a large proportion of snow at higher elevations and rain at lower elevations (U. S. Army Engineers, 1983). Rare summer rainstorms of short duration and small areal coverage occur because of local convective activity that develop as thunderstorms. The normal annual precipitation is approximately 114.3 centimeters (cm) or 45 inches, ranging from 76.2 cm (30 in) at the project to nearly 152.4 cm (60 in) in the headwaters where significant orographic lifting occurs. Usually about 75 percent of the annual precipitation occurs from November to March, and less than 2 percent occurs during July and August.

Fall chinook and coho salmon, along with steelhead trout, are the major anadromous fish that utilize the Applegate River system. Resident rainbow and cutthroat trout are other sport fish present in the watershed. Before the construction of Applegate Project, approximately 15,000 fall chinook spawned annually in the mainstem Applegate River. Also, approximately 5,000 coho salmon spawned annually in tributary streams. Additionally, about 10,000 steelhead trout utilized both the mainstem and tributaries of the Applegate River for spawning before the dam was built (Fish and Wildlife Service, 1961). Rainbow trout are stocked annually in the watershed.

In the recent past, the aquatic habitat of the drainage basin was affected by significant water diversions for irrigation. Low flow and concomitant increases in water temperature were considered harmful to the fish. Consequently, the Applegate Dam was designed with a multiple level withdrawal structure to provide better quality outflow.

DESCRIPTION OF THE SELECTIVE WITHDRAWAL STRUCTURE

Applegate Dam was constructed with a dual wet well selective withdrawal structure capable of removing water from six different levels in the reservoir. The withdrawal structure consists of a 72.2 m (237 ft) high, freestanding concrete tower and a 144.5 m (474 ft) long, 3.6 m (12 ft) wide service deck providing access to the intake tower (Figure 2).

The base of the tower contains two regulating outlets for control of high flows, a regulating gate chamber, and a trash rack structure. The tower consists of two vertical wet walls for water temperature control and a single dry well connecting the equipment room with the regulating gate chamber. The two water temperature control wet wells are served by five intake ports, two serving one wet well and three serving the other. The low level regulating outlets serve as the sixth level of withdrawal.

The bellmouth intakes for the regulating outlets are on the face of the base, behind the trash structure of vertical and horizontal trashbars. Each outlet is controlled by two 1.4 x 1.8 m (4.6 x 6.0 ft) slide gates used for regulating discharges and emergency closures. Water passes through the regulating outlet gates into twin 1.4 x 1.8 m (4.6 x 6.0 ft) rectangular conduits that transition in 7.7 m (25 ft) to a 4.4 x 2.7 m (14.5 x 9 ft) oblong concrete cut and cover

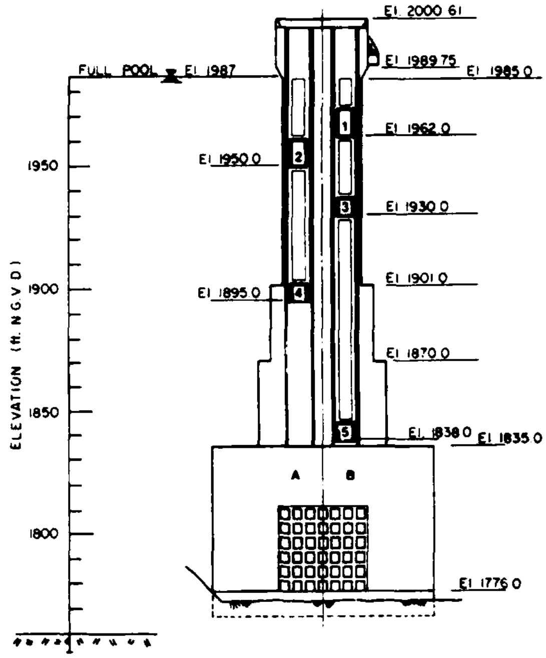

Figure 2. Applegate Lake Selective Withdrawal Structure Schematic

conduit approximately 243 m (800 ft) long, connecting the withdrawal structure to the downstream stilling basin through the dam embankment. Flows of up to 161 m^3/s (528 ft^3/s) will pass through the regulating outlets. Downstream of the cut and cover conduit, an open channel flares into a 9.1 x 85.3 m (30 x 280 ft) rectangular primary and secondary stilling basin.

Reservoir water used for temperature control can be drawn into the temperature control wet wells from any one of up to five levels, or mixed from different levels. The top two intake ports 1.5 x 3.0 m (5 x 10 ft) high and are rated at 14.1 m^3/s (46 ft^3/s). The other three intake ports are 1.5 x 1.8 m (5 x 6 ft) and are designed for 8.5 m^3/s (28 ft^3/s). They are operated in either a fully open or closed position. Water passes through the ports into either or both of the 2.1 x 1.4 m (6.9 x 4.6 ft) wet wells. The wet wells transition into a gate chamber area regulated by 0.6 x 0.8 m (2 x 2.6 ft) high tandem sliding wet well control gates. The wet wells then merge into the regulating outlet conduit for downstream release.

Also, a 0.77 m (2.5 ft) diameter fish facility water supply pipeline takes water from the right wet well (B) and delivers it to the fish collection facility by the stilling basin. The pipeline, with a capacity of 5.7 m^3/s (107 ft^3/s), is controlled by tandem 0.6 m (2 ft) diameter ball valves fully open or closed.

METHODOLOGY

Two separate studies, a test study and an operational study, of water temperature mixing in a small, single wet well using intake ports at different levels, were conducted. The test study was conducted for a two-day period on 5-6 October 1983, and the operational study covered an eight-day period between 5 and 12 September 1986. In both instances, the downstream gaging station, Applegate River near Copper, located 1 km (0.6 mi) downstream of Applegate Dam, was the control point.

A profile of reservoir water temperatures were taken near the selected withdrawal structure prior to each test study.

OCTOBER 1983 TEST STUDY

This test study was the first time since the project began operating in December 1981 that lake elevations, drawdown rate, lake stratification, and downstream fish spawning, were suitable for conducting a single wet well mixing study.

Because of the possible impact on the anadromous fishery downstream of the project, the Oregon Department of Fish and Wildlife (ODFW) was informed of a the test and the possibility of a temperature fluctuation between 9°C and 19°C (48° and 66°F) over the two day period. Approval was given with the stipulation that the period of maximum temperature outflow be minimized.

The plan was to use intake ports (IP) 3 & 5 on wet well (WW) B for temperature mixing (Figure 2). Initially, only IP 3 was to be open. WW A was to be completely closed, although IP 4 would be left open in case WW A was needed in an emergency. All gate settings were to be held for a 2-hour period to insure temperature and flow stablization downstream. Various port combinations would be tested by keeping IP 3 fully open and opening IP 5 in 0.3 m (1 ft) increments.

During the test, the wet well control gate (WTC) B would remain open at 0.8 m (2.5 ft) releasing near maximum flows for wet well B. The flow during the test was held near 14.2 m^3/s (500 ft^3/s).

The downstream gaging station, Applegate River near Copper, was used as the control point. The GOES satellite data collection platform (DCP) at that gage was reprogrammed to transmit in 10 minute increments. The two gaging stations farther downstream at Applegate and Wilderville were similarly reprogrammed to give downstream effects of the study for future use. A water temperature initial profile taken by project personnel on 4 October 1983, before the test began, is shown on Figure 3.

Operational Procedures During Test Study

Tables 1 & 2 give chronological tabulations of events, gate settings, flows, and temperatures for each step of the test. These tables are enhanced by Figures 4 & 5 which show the effects downstream at the Applegate River near Copper gaging station.

Since the intake ports were designed to operate in fully-open or fully-closed positions, and the test operation would be at partial gate openings, the ports were observed for gate hanger cable slacking, increased gate vibrations, abnormal wet well water surface level changes, excessive noise, etc. At no time during the study were any of the above peculiarities noted.

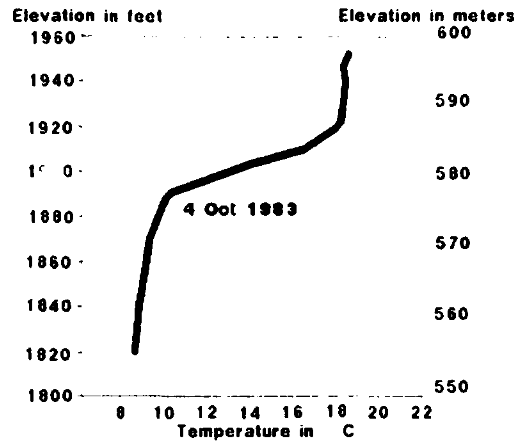

Figure 3. Applegate Lake Temperature Profile 4 October 1983

Day 1 Test

IP 3 was operated fully open at a 1.8 m (6.0 ft) setting, while the IP 5 opening varied from 0.0 m to 0.9 m (0.0 ft to 3.0 ft) in 0.3 m (1 ft) increments. Each setting was held for approximately 2 hours to stabilize downstream temperature and flow at the Copper gage. During the first day's test, outflow from the project was maintained by setting the wet well control gate (WTC) B at 2.56 feet (495 ft^3/s).

As shown on Figure 4, definite mixing occurs when more than one intake port is used in a small wet well. At each incremental change in IP 5 (the lower port) the outflow temperature indicated a noticeable temperature drop. The incremental changes also had a constant effect downstream, showing up at the Copper gage 15 minutes later and stabilizing 50 to 60 minutes after the change.

A minor increase in flow was noted with the successive IP 5 openings while WTC B remained fixed at 0.8 m (2.56 ft). This could be attributed to a higher pressure head resulting from using the lower gate.

At the end of the day, the outflow temperature was 14.3°C (57.8°F), near ODFW's desired target temperature and the temperature expected using the 4 October lake temperature profile shown on Figure 3. IP 3 was left fully open overnight at 1.8 m (6.0 ft) and IP 5 at 0.9 m (3.0 ft) using only WW B, to observe if any mixing change caused by density or temperature differences in the well would occur overnight.

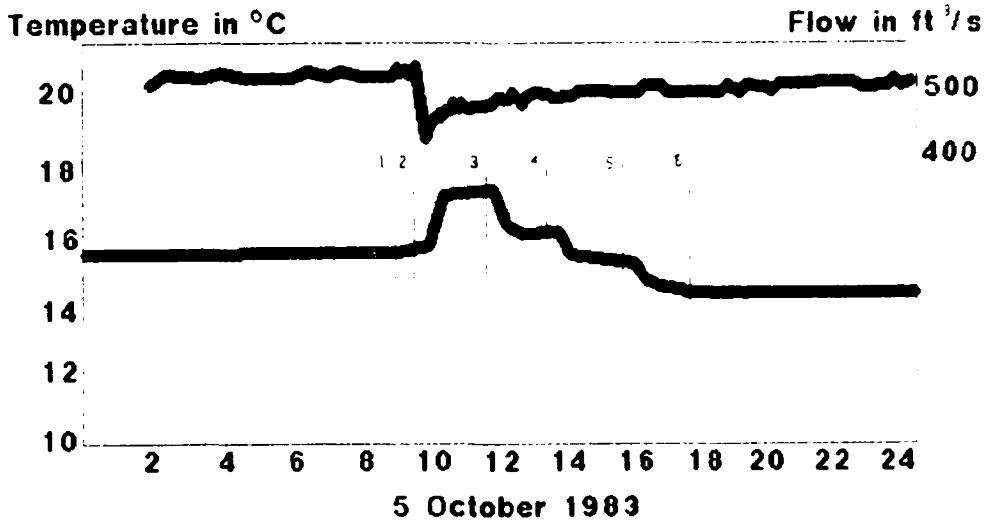

Figure 4. 1983 Test Study - Effect of Gate Changes on Temperature and Flow - Day 1

Day 2 Test

Although WTC B gate had settled from 0.78 m to 0.77 m (2.56 ft to 2.53 ft) overnight, the flow had slightly increased from 14.0 m^3/s (495 ft^3/s) to 14.4 m^3/s (510 ft^3/s). The temperature at the Copper gage only varied from 14.35 to 14.20°C (57.8 to 57.6°F), indicating an extremely stable condition downstream. *This was attributed to* overnight cooling downstream rather than a change in the temperature of water from the wet well.

On the second day, an attempt was made to shift the major portion of the flow to the lower gate and close the upper gate, followed by partially opening the upper gate. Shifting from one wet well to the other was necessary to completely close IP 3. At the end of the study it was also necessary to shift wet wells to meet operational conditions desired by ODFW. As explained later, we found shifting of wet wells was not necessary for structural integrity due to the absence of possible intake gate problems.

IP 5 was incrementally opened from 0.9 to 1.8 m (3.0 to 6.0 ft). Temperature was noticeably affected, because the increased gate opening provided additional cooler water from the lower level intake port (Figure 5).

The final step of the test was to close IP 3 and run the total flow through IP 5. In closing IP 3, what was thought to be a velocity problem, prevented closure beyond 0.2 m (0.8 ft) while discharging 14.3 m^3/s (505 ft^3/s) through WW B. Attempting to close IP 3 resulted in a decrease in temperature at the Copper gage.

To close IP 3 entirely, the outflow was shifted from WW B to WW A (IP 4) to provide a static condition in WW B. This operation resulted in a temporary increase in water temperature at Copper of approximately 1°C. Because of the short timing of the gate adjustments, water temperatures most likely did not stabilize enough at the gage to give accurate effects of these changes. With WW B static, IP 3 could still not be closed beyond 0.2 m (0.8 ft). Most likely the gate was held open by debris. Since IP 3 could not be fully closed, it was opened to 0.6 m (2.0 ft) and IP 5 remained fully open. The outflow was then shifted from WW A to WW B.

A significant change in flow occurred with this last operation. Prior to the change, WTC B was set at 0.77 m (2.52 ft) with a downstream flow of 14.3 m^3/s (505 ft^3/s). After the change, the WTC B was at 0.78 m (2.55 ft), but the flow had dropped to 13.0 m^3/s (460 ft^3/s). The change occurred over a long enough time for downstream conditions to stabilize. This could be considered an indication of inaccuracies in gate settings or downstream gage height readings. It illustrates the inability to finely control outflows.

The next step was to set the project back to normal operational conditions. Because ODFW's desired target temperature was for 14.0°C (57.2°F) water, the pre-test outflow water temperature conditions were not re-established. To reach 14.0°C, an increase of 3.5°C was required in the final step of the test. The final gate settings were based on a lake temperature profile taken 6 October. From that profile, shown on Figure 3, it was decided that IP's 3 & 4 and WTC's A & B should be opened. The first operation was to close IP 5 with WTC B fully open. No problems, such as vibration or cable slacking were encountered while closing IP 5 with a maximum flow in the wet well.

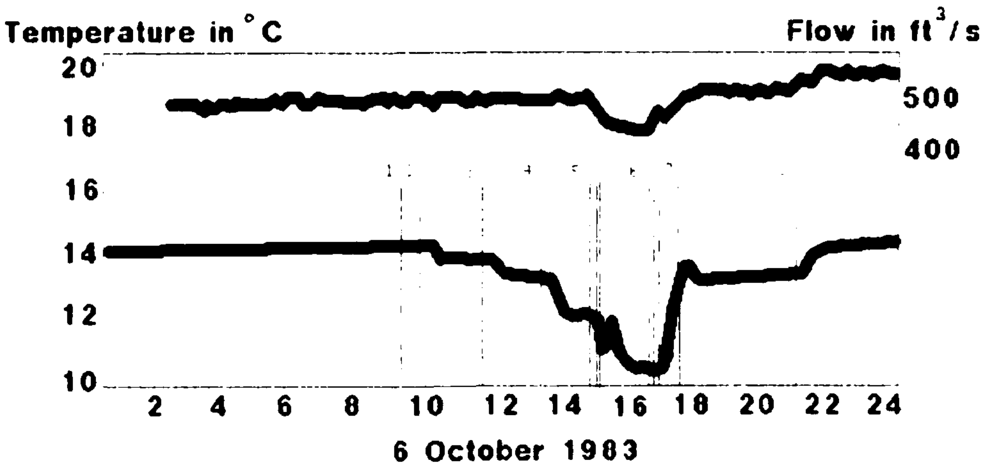

Figure 5. 1983 Test Study - Effect of Gate Changes on Temperature and Flow - Day 2

To control for 14.0°C outflow temperature, WTC B was set to 0.32 m (1.05 ft) and WTC A to 0.62 m (2.05 ft). After 30 minutes the downstream temperature was found to be rising faster than expected or desired. WTC A was raised to 0.76 m (2.48 ft) and WTC B lowered to 0.17 m (0.57 ft) to slow the rate of increase in temperature. Two and one-half hours later, WTC A was lowered to 0.61 m (2.0 ft) and WTC B was raised to 0.37 m (1.2 ft) as a final adjustment for 14.0°C (57.2°F) outflow.

Throughout the test, design flow criteria were exceeded at the WTC's and IP's. Design flow for the wet wells, with the head on the gates during the study, is 13.0 m^3/s (460 ft^3/s) with a WTC gate setting of 0.8 m (2.67 ft). The most that the WTC gates could be opened during the test was 0.79 m (2.58 ft), which gave a flow slightly greater than 14.2 m^3/s (500 ft^3/s). The design flow of the IP's is 8.5 m^3/s (300 ft^3/s), but when IP 3 or IP 4 were used alone during the test, the release was near 14.2 m^3/s (500 ft^3/s). When operating WW A alone, a very noisy situation, similar to being close to a moving train occurred. Also, substantial vibration was evident. This might be attributed to high flows, but WW B did not have the same problem.

Conclusion of Test Results

From the data collected during the test, noticeable temperature changes indicate that mixing did indeed take place in the small, single wet well at Applegate Lake using intake ports at different levels. The amount of flow provided by each intake port at the various openings was not determinable. There appeared to be no immediate or short-term structural or operating problems using the two intake ports in a single wet well. In fact, well noise and vibration was considerably less with two intakes in use, than when a single intake port was used. Although the test was conducted at a single flow, any problems anticipated while making gate changes, would most likely have occurred at this near maximum flow rather than at low flows when velocities are less.

Results from this test show that it is feasibile to operate multiple intake ports in a small, single wet well for temperature mixing and control.

TABLE 1

OCTOBER 1983 TEST STUDY OPERATING PROCEDURE - DAY 1

STEP	OPERATION CONDUCTED	TIME	WET WELL WTC A ft	WET WELL WTC B ft	FLOW ft^3/s	INTAKE PORT 3 ft	INTAKE PORT 4 ft	INTAKE PORT 5 ft	COPPER TEMP °C
1.	Initiate Day 1 Test	0930	1.08	2.06	523	6.0	6.0	0.0	15.6
2.	Set Initial Test Conditions	0933	CLOSED	2.56	477	6.0	N/A	0.0	17.25
3.	Begin Using Ports 3 and 5	1137	CLOSED	2.56	491	6.0	N/A	1.1	16.05
4.	Open Port 5 To Next Increment	1323	CLOSED	2.56	495	6.0	N/A	2.0	15.20
5.	Open Port 5 To Next Increment	1534	CLOSED	2.56	495	6.0	N/A	3.0	14.35
6.	Ended Day 1 Test	1730	CLOSED	2.56	495	6.0	N/A	3.0	14.30

TABLE 2

OCTOBER 1983 TEST STUDY OPERATING PROCEDURE - DAY 2

STEP	OPERATION CONDUCTED	TIME	WET WELL WTC A ft	WET WELL WTC B ft	FLOW ft^3/s	INTAKE PORT 3 ft	INTAKE PORT 4 ft	INTAKE PORT 5 ft	COPPER TEMP ^oC
1.	Initiate Day 2 Test	0915	CLOSED	2.53	505	6.0	N/A	3.0	14.20
2.	Open Port 5 To Next Increment	0929	CLOSED	2.53	510	6.0	N/A	4.0	13.85
3.	Open Port 5 To Next Increment	1125	CLOSED	2.53	505	6.0	N/A	5.0	13.22
4.	Open Port 5 To Full Opening	1313	CLOSED	2.52	505	6.0	N/A	6.0	12.10
5.	Set Final Test Condition	1443	CLOSED	2.52	505	0.8	N/A	6.0	11.20
a.	Changing Wet Wells To Close Port 3	1445 1452	OPENING 2.55	CLOSING CLOSED		0.8	6.0	N/A	12.00
b.	Adjust to Final Test	1455	CLOSING	OPENING		2.0	N/A	6.0	
c.	Adjustment Completed	1503	CLOSED	2.55	460	2.0	N/A	6.0	10.55
6.	Ended Day 2 Test and Set Outflow to 14^o	1632	CLOSED	2.55		6.0	N/A	CLOSING	
a.	Changing Wet Wells	1635	OPENING	CLOSING		6.0	6.0	0.2	
b.	Set for Temperature Control	1638	2.17	0.85		6.0	6.0	0.2	
c.	Closing Wet Well to Fully Close Intake Port 5	1640	2.58	CLOSED		N/A	6.0	N/A	
d.	Adjustment to 14^oC	1650	CLOSING	OPENING					
e.	Finish Adjustment	1652	2.05	1.05		6.0	6.0	0.2	
7.	Fine Tune Adjustment	1720	2.48	0.57	520	6.0	6.0	0.2	13.15
8.	Final Adjustment and Finish Test	2100	2.00	1.20	540	6.0	6.0	0.2	14.05

SEPTEMBER 1986 OPERATIONAL STUDY

In September 1986, an opportunity to test a single wet well at the low release rate of 4.8 m^3/s (170 ft^3/s) was available. The Oregon Department of Fish and Wildlife (ODFW) had requested a target release temperature of 12.8°C (55°F) for September. The target release temperature was being achieved by blending water intake port (IP) 3 of wet well (WW) B and IP 4 of WW A. The Portland District decided to save as much of the water immediately below the thermocline as possible to be used later (Figure 6). The water near to, but below the thermocline naturally approaches 12.8°C (55°F) from thermal heating of the reservoir water during September. By mixing warmer surface water with cooler bottom water, the Reservoir Regulation and Water Quality Section felt it could provide 12.8°C water for an extended time period. IP 3 had 16° to 18°C (60° to 65°F) water available while IP 5 had 7°C (45°F) water available for mixing.

On 4 September 1986, the Reservoir Regulation and Water Quality Section requested that the project use only WW B to achieve the 12.8°C target water temperature. This was attempted by fully opening of IP 3 to 1.8 m (6 ft) and a partial opening of IP 5 to 0.9 m (3 ft). Initially there was some difficulty establishing a stable 13°C (55°F) outflow. When WW A was closed, and using only IP 3 of WW B, the release temperature climbed to over 16°C (60°F) in two hours (Figure 7). When IP 5 was opened to 0.9 m (3 ft), the release water temperature dropped to 11°C (52°F). After two hours, it appeared the release temperature had stabilized near 13°C (55°F), so project personnel departed. Later, a project operator noticed the release temperature had increased to over 18°C (65°F). Personnel returned to the selective withdrawal structure and tried a variety of port opening combinations to re-establish a release temperature near 12.8°C. By opening IP 3 to 0.6 m (2 ft) and IP 5 to 1.2 m (4 ft), the release water temperature stabilized near 14°C (57°F).

A temperature of 13° to 14°C (55° to 57°F) was maintained by releasing from WW B from 2200 hours on 4 September through 1200 hours on 12 September 1986. At that time, the dual wet well mixing procedure was established once again. Temperature readings taken from the GOES DCP data, transmitted in hourly increments (normal operational schedule) are shown on Figure 7.

Conclusion of the Operation Study

During the nearly 8-day test period, the 14°C (57°F) releases were stable and no operational problems such as gate vibrations were observed. Of interest is that at the gaging station 0.97 km (0.6 mi) downstream of the project, local downstream effects caused the diel water temperature fluctuation to be close to one degree C with a constant 14°C water being released.

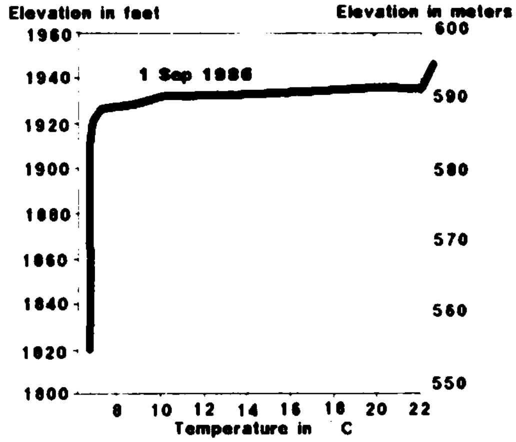

Figure 6. Applegate Lake Temperature Profile
1 September 1986

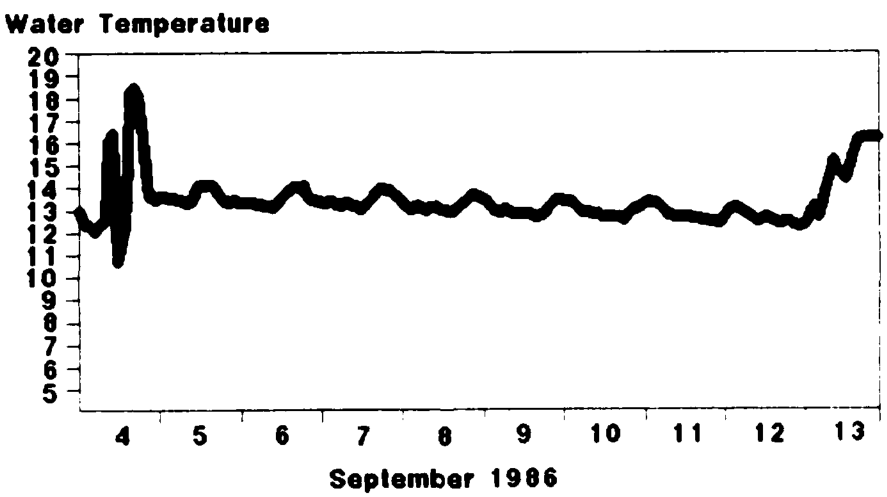

Figure 7. 1986 Operational Study - Achievement of Target Release Temperature

RECENT DEVELOPMENTS IN TURBINE AERATION

E. Dean Harshbarger

Supervisor, Research and Testing, TVA Engineering Laboratory, Norris, TN

ABSTRACT

Since the early 1970's, the Tennessee Valley Authority has been investigating methods for aerating hydroturbine discharges. Techniques which have been physically tested include small pore diffusers, draft tube air aspiration, vacuum breaker modifications, hub baffles, draft tube forced air systems and epilimnetic pumps. These tests are briefly described and the results obtained are discussed. The conclusions are that to a large extent, aeration methods are site specific and no universal solution can be applied. Efforts toward the development of new turbines specifically designed for aeration are suggested.

INTRODUCTION

The hydroturbine discharges from some 21 TVA dams have been identified as having undesirably low dissolved oxygen (DO) content during the summer and fall months. These dams involve a total of 77 turbines of various types, sizes, and designs. Since the early 1970's efforts have been made to develop reliable, economically feasible systems for increasing the DO in turbine discharges. Methods which have been investigated include: small pore diffusers to bubble oxygen just upstream from the turbine intake; aspiration of air into the turbine draft tube; modification of the vacuum breaker system to aspirate more air; forced air into the draft tube; epilimnetic pumps to force oxygenated reservoir surface water into the intakes; and development of a self-aerating turbine. Progress on aeration efforts in TVA have been reported periodically (TVA, 1981, 1983, 1984; Davis, et al., 1983; Bohac, et al., 1986; Ruane, et al., 1986). This paper reviews TVA's past efforts and updates the latest efforts to develop these aeration methods.

AERATION TECHNIQUES INVESTIGATED

Small Pore Diffusers

Small pore diffusers were extensively investigated in the early 1970's as a possible solution for DO problems at Fort Patrick Henry Dam (TVA, 1978). Fifteen to twenty different commercially available diffuser materials were evaluated in the laboratory and, based upon information gained, two were selected for use in a full-scale field test. Ten frames of about 150 diffusers each were placed near the bottom of the 70-foot deep reservoir just upstream from one of the two turbine intakes. Pure oxygen was bubbled through these diffusers from a liquid oxygen storage and evaporating facility located nearby. The system worked. DO was increased from near zero in the reservoir to about four ppm in the tailrace. The main problems with this system were the safety concerns over using oxygen and the relatively high initial and operating costs. It was estimated that an operating system to increase the DO in the discharge from both hydroturbines at Fort Patrick Henry to 5 ppm would have an initial capital cost of about $400,000 and an annual operating cost of about $110,000.

At present, a small pore diffuser system is being studied for use on the four units at Douglas Dam. The preliminary design calls for nearly 6,000, 17.5-cm (7-in.) diameter diffusers placed in an area of about 2 acres in front of the turbine intakes. A liquid oxygen storage facility capable of supplying about 54 tons of oxygen per day would be placed nearby to furnish enough oxygen to raise the discharge DO content up to 4 ppm. The storage tanks would have to be refilled every 3 days under extreme conditions of zero incoming DO and full plant discharge of 48 m³/s (16,000 cfs). Costs for the system are estimated to be about $3.5M initially and $1.0M annually.

Draft Tube Air Aspiration

To induce air into the draft tube at Norris and thus increase the DO in the turbine discharges, a flow obstruction in the form of a metal pipe, triangular in cross section, was welded to the draft tube wall about 1.5 m (5 ft) downstream from the turbine of Unit 1 (Harshbarger, 1982a). The pipe (or baffle) protruded 15 cm (6 in.) from the draft tube wall. The upstream side of the baffle served to accelerate the flow. This caused a low pressure on the downstream side to induce air. The interior of the pipe served as a passageway for air which then entered the draft tube through some 40 equally spaced 3.75-cm (1.5-in.) diameter holes in the downstream side of the baffle. Air reached the baffle via a 25-cm (10-in.) pipe through an opening cut just below the draft tube mandoor. This system aspirated enough air to increase the tailrace DO 3-4 ppm, but caused unacceptable energy losses. Imbedding the baffle into the wall of the draft tube may have reduced energy losses but was not attractive because of construction difficulties.

Another draft tube baffle scheme was installed and tested on a 35 MW Francis unit at Douglas Dam (Harshbarger, 1984d). For these tests, a 26.7-cm (10.75-in.) high, 11.9-cm (4.75-in.) deep manifold was cut into the draft tube wall about 3.6 m (12 ft) downstream from the turbine. Air was supplied to the manifold through a 14-inch pipe entering the draft tube adjacent to the mandoor. Twenty-four equally spaced 4.4-cm (1.75-in.) diameter holes were cut into the manifold for air vents into the draft tube. Over each of these vents was bolted a conical shaped baffle which protruded 12.5 cm (5 in.) from the draft tube wall. Tests indicated that these baffles reduced local pressure, but not enough to induce significant amounts of air into the draft tube.

Vacuum Breaker Modifications

Most hydroturbines are equipped with vacuum breaker systems to relieve below atmospheric pressures which may occur when flow through the turbine is suddenly stopped. These systems consist of openings in the turbine hub which lead to passageways through the headcover and then to the atmosphere via 15- to 20-cm (6- to 8-in.) diameter pipe. A spring loaded valve is placed in the air pipe to admit air if the pressure under the headcover drops below some preset value.

Tests were conducted on 19 units to determine if there was enough suction to induce air through the vacuum breaker system if the vacuum breaker valve was blocked open (Fox, 1980, 1981; Harshbarger, 1981a, 1981c, 1981d, 1981e, 1982b, 1982d, 1982e, 1983a). On a few of them, airflow was increased slightly at low wicket gate openings, but not at the gate openings where the units are usually run. Only on the 35 MW unit at South Holston Dam was vacuum breaker suction sufficient to aspirate significant amounts of air (Harshbarger, 1985b). On this unit, an additional opening was cut through the headcover to admit about 0.9 m³/s (30 cfs) of air at the normal operating wicket gate opening (near 100%). This was enough to increase DO in the tailrace on the order of 2 ppm.

Hub Baffles

To lower pressures and thus aspirate more air through the vacuum breaker system, small baffles were installed over the vacuum breaker ports on the hubs of five different turbines. Tests on two Francis type units at Cherokee Dam (Harshbarger, 1981b), one at Douglas Dam (Harshbarger, 1982c) and one at Wilson Dam (Harshbarger, 1982e) indicated that the hub baffles increased induced airflow only insignificant amounts at the wicket gate openings of interest.

Part of the problems with hub baffles appears to be attributable to changing flow patterns around the turbine hub as flow through the unit is changed. In order to design a hub baffle which would work well at all flows, particularly the flow obtained at the most efficient gate opening, TVA contracted with an outside firm to apply a mathematical model they had developed to determine the flow around the hub of the Douglas hydroturbine. This information, along with information obtained from a baffle test apparatus at the TVA Engineering Laboratory was subsequently used to design a hub baffle system for aerating the Douglas hydroturbines. This system involved relocating the vacuum breaker ports and installing 12.5-cm (5 in.) high cylindrical baffles which protruded 12.5 cm (5 in.) out from the turbine hub over each port. These baffles did induce air but not in great enough amounts to significantly increase DO in the discharge. Hub baffles were consequently discarded as a method for aeration at Douglas Dam.

On the 35 MW unit at South Holston, the vacuum breaker ports were relocated farther down the hub and then baffles were placed over the new ports. This particular unit needs air to dampen severe vibrations caused by flow through the penstock and draft tube. The baffles were designed to aspirate at gate openings of about 75 to 80 percent. At low gate openings, no air was induced and the unit vibrated severely. The baffles were subsequently removed and no additional tests were conducted since it was discovered that aeration could be accomplished by relatively simple modifications to the vacuum breaker system.

Successful aeration using hub baffles was accomplished on the two 50 MW units at Norris Dam. After several iterations (Harshbarger, 1983a, 1982d, 1984c) and a series of Laboratory tests, bolt-on type baffles were designed and have been in use for the past three years. These baffles are in the form of a cylindrical wedge cut from a section of 25-cm (10-in.) pipe and protrude out from the hub 13.75 cm (5.5 in.). With these baffles, enough air is induced to add about 4 ppm to the turbine discharges. The baffles and induced air reduce unit efficiency less than one-half of one percent. To avoid this small power loss when aeration is not needed, the baffles are bolted on in late summer and removed as soon as the incoming DO reaches 4 ppm.

The two units at Norris on which the baffles have been used have experienced an accelerated rate of cavitation damage, an expanded cavitation damaged area and cavitation damage in areas which are difficult to repair. Consideration is being given to the development of new stainless steel runners which would be less susceptible to cavitation damage and which could possibly be designed to be self aerating.

Draft Tube Forced Air System

The feasibility of using blowers to aerate by forcing air under the turbine headcover was tested on a total of 8 units (Harshbarger, 1983b, 1984a, 1984b, 1985a). For most of these tests, access holes were cut through the turbine headcover and air from portable, diesel powered compressors was supplied through temporary pipes and hoses. This method worked with varying degrees of success on nearly all turbines where it was tried. The major problem was that to install a permanent system large enough to meet desired DO conditions required large blowers and major plant modifications to install necessary piping and controls.

At Douglas Dam, blowers were also used to force air through the manifold cut into the draft tube of Unit 4 and previously used for the draft tube baffle tests (Harshbarger, 1984d). The baffles were removed for the blower tests leaving only the holes cut into the manifold. The blower system worked satisfactorily for aerating the turbine releases, but there was no room to install the required air ducts and compressor equipment on a permanent basis.

At present, a forced air system is being successfully applied at the 45 MW Tims Ford Dam (Harshbarger, 1985c). At this site, a 1.5 m³/s

(50 cfs) blower has been installed in a small shelter outside the plant and the necessary piping is routed outside the plant building to a location near the unit where a tie-in to the vacuum breaker system is made. Flow from this blower is controlled by a damper and is adjusted to maintain a DO concentration in the tailrace of 4 ppm.

Epilimnetic Pumps

Turbine intakes are usually located near the bottom of reservoirs and draw cooler, low oxygen content water when the reservoir becomes temperature stratified. At Douglas Dam, an aeration method using pumps to force highly oxygenated reservoir surface water into the turbine intakes is being tested (Mobley, 1986). Each pump consists of a 6-bladed, 4.6-m (15-ft) diameter propeller connected by a shaft and gear box to a 30-hp electric motor. The motor, gear box and controls are supported from a raft on the water surface while the pump propeller is suspended beneath the raft at a depth of 3 m (10 ft). The propeller turns at about 21 rpm and is expected to move about 1.4 m^3/s (500 cfs) of water down some 25 m (80 ft) to the turbine intake. Two pumps have been installed so far with a third expected to be in place before the reservoir stratifies next summer. Tests to evaluate the system are planned for July 1987.

CONCLUSIONS

Several techniques for aeration of turbine discharges have been investigated by TVA over the past several years. These include small pore diffusers, draft tube air aspiration, vacuum breaker system modifications, hub baffles, blowers and epilimnetic pumps. At present, a small pore diffuser system is under study for use at Douglas Dam, the vacuum breaker system has been modified to admit more air at South Holston Dam, hub baffles are in use at Norris Dam, a blower is being used to force air under the turbine headcover at Tims Ford Dam and epilimnetic pumps are under study at Douglas Dam. In addition, the development of a self aerating cavitation damage resistant turbine is being considered to replace the present turbines at Norris Dam. The conclusion is that there is no universal solution for aeration of turbine discharges. Geometry alone demands that each case be considered individually to arrive at the most economical and efficient system.

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OXYGENATION OF RELEASES FROM RICHARD B. RUSSELL DAM

James W. Gallagher, Jr.[1] and Gary V. Mauldin[2]

[1] Hydraulic Engineer, U.S. Army Corps of Engineers, Savannah District, Savannah, GA 31402-0889
[2] Hydraulic Engineer, U.S. Army Corps of Engineers, Savannah District, Savannah, GA 31402-0889

ABSTRACT

During the early planning stages of the Russell project, a major concern of the State and Federal agencies was that the project comply with State water quality standards. The Corps of Engineers committed itself to this objective and developed a diffused oxygen injection system to oxygenate the reservoir releases. At Richard B. Russell Lake, oxygen is injected into the lake through fine pore diffusers located on the bottom of the lake both at the face of the dam and 1 mile upstream from the dam. The paper documents the interagency involvement in the oxygen injection system, summarizes the results of the field tests which led to the development of the system, and describes the installation and operation of the system installed in Richard B. Russell Lake.

INTRODUCTION

The Savannah District, U.S. Army Corps of Engineers, is currently completing construction and has begun operation of the Richard B. Russell Dam and Lake project on the Savannah River between Georgia and South Carolina. The Richard B. Russell Damsite is on the Savannah River 37 miles above Clarks Hill Dam and 30 miles below Hartwell Dam. Clarks Hill and Hartwell are both Corps of Engineers projects.

The Richard B. Russell Dam and Lake is a multi-purpose project designed to provide hydropower, some flood control, recreation, and has a potential for water supply. The dam consists of a 195-foot high, 1,900-foot long concrete gravity structure flanked by two earth embankments. The project is designed as a peaking powerplant with an installed capacity of 600 megawatts. The powerhouse will contain four 75-megawatt conventional units and four 75-megawatt pump units. This installation will make the project one of the largest Corps of Engineers' hydropower facilities in the nation. During periods of maximum generation the plant will release about 60,000 CFS. During maximum pumpback operation, 30,000 CFS will be pumped from Clarks Hill Lake back into Russell. The average daily release from the project is over 3,500 CFS. At maximum power pool the Russell Lake covers 26,650 acres and impounds 1,026,000 acre-feet of water.

The lake was impounded in 1984. Power-on-line for the first of the four conventional units occurred in December 1984, and the last conventional unit became operational in January 1986. Power-on-line for the first of the four pump storage units is scheduled for February 1990. The current project cost estimate is 541 million dollars.

Like all deep lakes in the southeast, the Richard B. Russell Lake thermally stratifies in the warm summer months. During this time, the waters circulated by the wind are confined to the top 30 feet of the lake. With no means to replenish dissolved oxygen lost due to biological and chemical activity, the dissolved oxygen concentrations in the lower layer of the lake are gradually exhausted. Since the turbine intake are located in the lower layer of the lake, it is this water that is released from the project for power generation, and during the summer these waters have progressively reduced dissolved oxygen levels.

INTERAGENCY INVOLVEMENT

During the early planning stages of the Russell project, a major concern of the State and Federal agencies was that the project comply with State water quality standards. The State of Georgia was particularly emphatic on this point, and as a result, the cost sharing agreement between the State of Georgia and the Federal Government for development of the project's recreational areas includes the stipulation that the operation of the project will meet State water quality standards. Besides this commitment to Georgia, a commitment to provide 6 parts per million (ppm) dissolved oxygen in the releases from the reservoir is also explicitly stated in the Statement of Findings for the project filed pursuant to Section 404 of the Federal Water Pollution Control Act Amendments of 1972.

In July 1972, the Georgia Department of Natural Resources requested the formation of a technical committee to analyze the water quality matters relating to the Russell project. The objective of the committee was to evaluate the thermal and dissolved oxygen characteristics of the Russell project as an integral part of the Hartwell-Clarks Hill reservoir system including the following specifics:

a. Maintenance of Federal and State water quality standards.

b. Maintenance of a coldwater fishery in a 10-mile reach downstream from Hartwell Dam.

c. Development of a warm and cold water fishery within Russell Lake.

d. Maintenance of a warm and cold water fishery within Clarks Hill Lake.

With these objectives established, physical and mathematical modeling were conducted to determine travel time, level and thickness of inflows, entrainment and pumpback currents which were then input into a mathematical model which determined the dissolved oxygen and temperature regimes in the lakes and in the hydropower releases.

In its final report, the committee observed that the water quality objectives could be met with the artificial addition of oxygen. Several methods of adding oxygen were then investigated including surface aerators, diffused air injection, spillway aeration, penstock air injection, multilevel penstock intakes, submerged weirs, oxygen injection into the penstocks, side stream oxygenation, localized destratification, pulsed oxygen injection through porous diffusers into the lake at the face of the dam, and continuous oxygen injection through porous diffusers into the lake at a point several days travel time upstream of the dam. With the high oxygen and low temperature constraints, continuous oxygen injection with an on-site Government-owned cryogenic plant was identified as the most feasible alternative. Continuous oxygen injection is favored over pulsed oxygen injection because it avoids the high capital and operating costs associated with liquifying and storing gaseous oxygen.

FIELD TESTS

Between 1975 and 1980, the Savannah District, through contracts with Dr. Richard Speece of DrexelUniversity, conducted field tests of an oxygen injection system at Clarks Hill Lake. As a first step, a small scale system capable of providing sufficient oxygen for the discharge of one turbine was installed adjacent to the dam face at Clarks Hill and operated in a pulsed mode by Speece, et al,[1] in the summer of 1975. This made it possible to rapidly monitor the oxygen level in the dicharge and determine the oxygen absorption efficiency immediately.

It was concluded from these tests that it was technically feasible to dissolve oxygen in a pulsed mode that was matched to the water discharge rate. However, as mentioned earlier, the recommended method is continuous oxygen injection at an upstream point in the lake rather than pulsed oxygen injection at the face of the dam. Pulsed injection of oxygen to match the water discharge rate involves mtching the peaking discharge pattern which normally occurs less than 12 hours each weekday and even less on weekends. With on-site cryogenic oxygen being produced in the gaseous state at a uniform rate, compressio and storage would need to be provided to match the production with the usage rate. This would increase the capital costs of the oxygen production facility. Therefore, it was decided that field tests should be conducted to evaluate the feasibility of continuous injection into a diffuser system located approximately 1 mile upstream of the dam.

The field tests of the continuous injection system began the next summer[2]. The tests were divided into three phases. Phase I was an evaluation of the oxygen absorption efficiency of various diffusers. In this test, diffusers with a standard permeability of 0.5 to 2.0 feet per minute (fpm) were identified as the optimum diffusers. Phase II involved tests of racks of diffusers to determine the elevation in the water column at which the oxygenated water would come to equilibrium.

Phase III of the study involved installation of nine diffuser racks at a location approximately 1 mile upstream of the dam in water approximately 130 feet deep. The highest dissolved oxygen concentration recorded in the turbines was 4.1 ppm which occurred about 6 days after oxygen injection started. The background dissolved oxygen before oxygen injection commenced was 0.5 to 0.8 ppm. Only about 30 to 40 percent of the oxygen that was injected appeared to eventually reach the turbines. the low oxygen absorption efficiency was due to two factors. First, the diffusers on the rack were not the most efficient as determined in Phase I of the study. Second, the close semicircular spacing of the diffuser racks and high injection rates per diffuser caused localized destratification in the vicinity of the diffuser racks which resulted in the dissolved oxygen-rich water coming to equilibrium in the upper level of the lake where it was unavailble for dissolved oxygen enrichment of the turbine discharges. It was determined that improvements in the performance of the oxygen injection system could be realized by lowering the injection rate per diffuser by quadrupling the number of diffusers per rack, equipping the racks with the optimum 2 fpm diffusers, and spreading the racks across the lake cross section.

These improvements were made to the system and field tests were conducted in the summer of 1977[3]. The nine racks were fitted with 40 square feet of diffusers of 2 fpm standard permeability. The racks were placed across the lake cross section 1 mile upstream from the dam and spaced approximately 300 feet apart with the first rack located approximately 1,200 feet from shore. Oxygen was injected continuously for 30 days at a rate of 100 tons/day, and dissolved oxygen and temperature were monitored in the lake and the turbines. During this period of oxygen injection, dissolved oxygen concentrations of 4 to 5 ppm were maintained with an absorption efficiency of 50 percent. Although this represented an improvement over the results from the previous year, the goal of 6 ppm dissolved oxygen was still not achieved and the absorption efficiency was still unacceptable. Although the lake did not destratify in the vicinity of the racks, pumping of the oxygenated water occurred causing it to reach the surface where it warmed and returned to an intermediate layer generally above the turbine withdrawal zone. It was determined that the pumping was due to the four-sided diffuser configuration of the racks and that the pumping could be eliminated by employing a linear diffuser configuration.

RICHARD B. RUSSELL OXYGEN INJECTION SYSTEM

The oxygen injection system at the Richard B. Russell project is described in the Richard B. Russell Dam and Lake Design Memorandum 35 and Supplement No. 1[4 and 5]. The system at Russell has a continuous injection system located 1 mile upstream of the Russell damsite but also has supplemental injection capability at the face of the dam to be used during periods of higher than normal releases and unusually high dissolved oxygen deficits. Gaseous oxygen is supplied from a liquid oxygen storage facility on the lakeshore.

The continuous system consists of a distribution pipe from the oxygen facility to two parallel diffuser pipes suspended 5 feet off the lake bottom. The two diffuser pipes are over [illegible] feet long and are spaced 100 feet apart. For the pulsed system, a main distribution line extends from the oxygen supply site to the top of the dam. Additionally, eight feeder pipes extend from the main distribution line down the face of the dam and then connect to the diffuser lines between each intake perpendicular to the face of the dam. Each feeder pipe for the pulsed system is equipped with a motorized control valve which allows operation of any combination of pulsed diffuser lines. Power and telemetry for the motorized control valves is provided from the oxygen supply facility by means of underground cables.

A typical section of diffuser piping is shown on Figure 1. The diffuser piping consists of an 8-inch center manifold pipe made of fiberglass reinforced plastic. Flotation is attached to the manifold pipe to provide a constant positive buoyancy to the system during shutdown. Vertical and horizontal alignment of the system is secured by guying the manifold pipe to concrete anchor blocks on the lake bottow with stainless steel cables. Flanged to the manifold pipe are 20-feet sections of 4-inch schedule 80 PVC diffuser pipe. The diffusers are spaced 1 foot apart along this pipe. A control orifice is installed in each flanged connection between the manifold pipe and the diffuser pipe to ensure proper flow distribution through this system. The diffusers are 7 inches in diameter and are made of silica glass bonded together with an organic binder. The diffusers have a standard permeability of 2 fpm.

The cost of the oxygenation system was 3.9 million dollars: 1.0 million dollars for the oxygen storage facility and 2.9 million dollars for the distribution and diffusing system. The price of liquid oxygen is currently $65/ton.

Based on the best estimates currently available on the expected dissolved oxygen content of the reservoir releases, the expected daily discharge from the project and the expected oxygen absorption efficiency, approximately 5,500 tons of oxygen would have to be added annually at a maximum rate of 150 tons/day to meet the downstream dissolved oxygen objective of 6 ppm in the hydropower discharges that would occur 90 percent of the time. For the first few years of operation, oxygen will be purchased from commercial suppliers, stored in liquid oxygen storage tanks on the site, and then vaporized as needed. Although ultimate plans are to install an on-site oxygen production facility, purchased liquid oxygen will be used initially to gain detailed data on project performance and oxygen requirements. After this period, we will be in a better position to determine both the desirability of an on-site production facility and the type of facility that will be most economical to operate.

In order to monitor the impacts of Russell Dam and Lake on the three lake system, the Savannah District entered into a cooperative agreement with the Waterways Experiment Station for a 3-year water quality study. This study included 3 major objectives:

(1) to described post-impoundment water quality conditions in Richard B. Russell Lake;

(2) to document the impacts of impoundment on water quality conditions in Clarks Hill Lake and;

(3) to evaluate the effectiveness of the oxygeninjection system in ameliorating potential water quality problems in Richard B. Russell Lake and its tailwater.

Three annual reports summarizing the findings of this study will be published (6).

The oxygen injection system began initial operation in April 1985 with the onset of lake stratification and continued operating until the lake destratified in early December. Figure 2 shows a dissolved oxygen profile just downstream of the system along with a background dissolved oxygen profile taken 1-1/2 miles upstream of the system. As the figure shows, the oxygenated water remains in the lower layer of the lake where it is available for turbine discharge. The oxygen injection system has consistently maintained at least 6.0 mg/l of dissolved oxygen in the releases from Russell Dam. The total oxygen injected during 1985 was 13,300 tons which was higher than anticipated due to the large amount of oxygen demanding material found in the new lake, and a later than normal fall turncver.

The second season of oxygen injection began in May 1986. As of September 30, 6650 tons of oxygen have been injected. This is 2200 tons less than was injected for the same period in 1985. This reduction in oxygen required is due to the natural aging of the lake and to the reduced total water released from Russell Dam caused by the present drought conditions in the Southeast.

Gas analyses of the bubbles have shown that bubble size greatly influence the oxygen transfer efficiency of the system. Measured efficiencies have ranged from 25 to 96 percent with smaller bubbles averaging greater than 70 percent and larger bubbles less than 50 percent. Small bubbles result from diffusion through the diffusers while large bubbles result from leaks in the system or malfunctioning diffusers.

The oxygen injection system has been in operating for two seasons and overall the system has performed as designed. The dissolved oxygen levels released from the dam have consistently been greater than 6.0 mg/l with the average oxygen transfer efficiency around 75 percent. The largest portion of oxygen is absorbed in the lower hypolimnion which comprises most of the withdrawal from the lake. Oxygen injection has not caused local destratification which would increase the water temperature of the releases.

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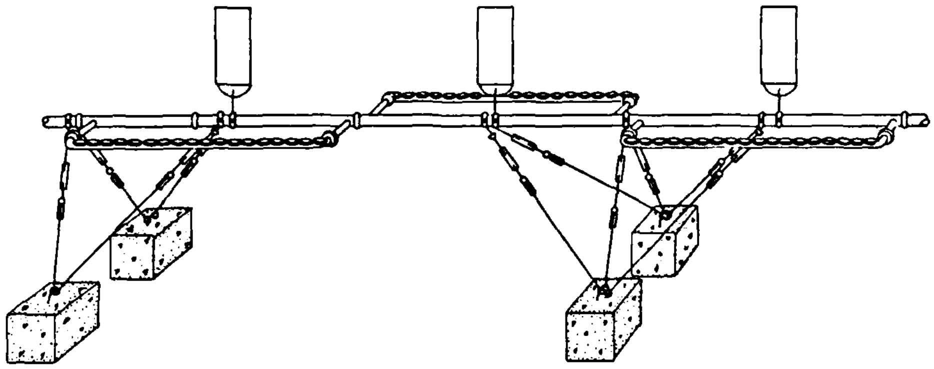

Figure 1. Typical section of diffuser piping.

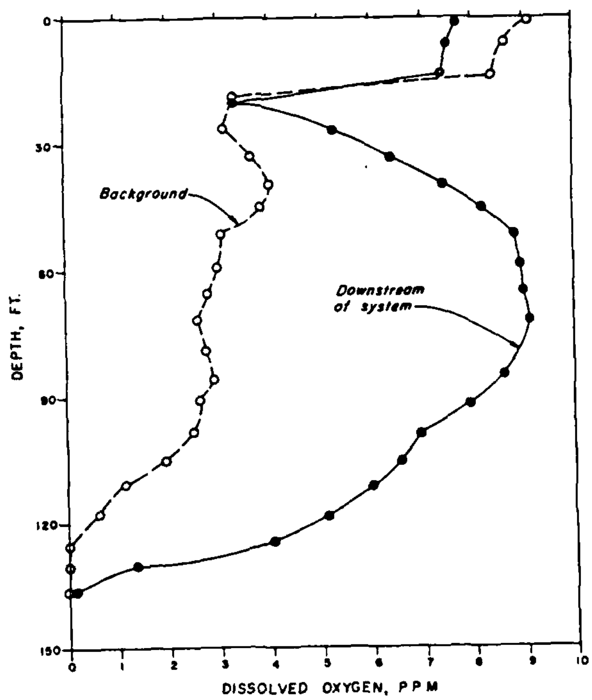

Figure 2. In-lake dissolved oxygen profiles upstream and downstream of continuous oxygen injection system.

IN-RESERVOIR AERATION SYSTEMS

C. E. Bohac, Ph.D., P.E., Environmental Engineer,
Tennessee Valley Authority
Chattanooga, Tennessee

ABSTRACT

Two types of aeration systems are are being demonstrated by the Tennessee Valley Authority. The first is an aeration/destratification system using diffused air. The system is installed at Upper Bear Creek Reservoir, Alabama. The system is designed to increase DO levels in the reservoir as well as reduce concentrations of iron, manganese, and hydrogen sulfide in the withdrawals from the reservoir.

The second system is a hypolimnetic aeration system using high purity oxygen. The system is designed to aerate only a small submerged pool of cool water in an embayment of a large reservoir. The embayment acts as a refuge for fish during times when the main reservoir body is anoxic and fish are stressed.

INTRODUCTION

There are two classes of problems arising from anoxic conditions in reservoirs. The first is that depressed reservoir oxygen levels can result in low oxygen concentrations in the releases from reservoirs. The second is that depressed reservoir oxygen levels can directly result in adverse biological and biochemical conditions in the reservoir. In some cases reservoir aeration can be used to improve the dissolved oxygen (DO) levels in the releases from the reservoir and used to modify the chemical and biological environment within the reservoir.

This paper reports on two in-reservoir aeration efforts. The first was to aerate and possibly destratify a significant portion of the Upper Bear Creek Reservoir near Haleyville, Alabama. The second was to aerate a portion of the hypolimnion of Cherokee Reservoir near Morristown, Tennessee.

UPPER BEAR CREEK RESERVOIR

Upper Bear Creek Reservoir has a volume of approximately $2.8x10^7m^3$ (23,000 acre-feet) with a surface area of about 1.2 to $1.6x10^7m^2$ (3000 to 4000) acres. Filling of the reservoir began in 1974. The reservoir provides flood protection and serves as a water supply for several communities in west-central Alabama. In addition, the reservoir provides water for the weekend operation of the Bear Creek Floatway, a major recreational feature in the area. The reservoir and outlet structure are shown in figure 1.

Figure 1. Upper Bear Creek Reservoir

The water quality problems associated with Upper Bear Creek Reservoir are low summer DO which leads to the high concentrations of iron, manganese, and sometimes hydrogen sulfide. The anoxic reservoir provides a reducing environment and the insoluble iron, manganese, and sulfide in the reservoir sediments become dissolved. The source of these three constituents is believed to be upstream coal mining.

Problems result in that the water treatment plant using the reservoir as a water supply often struggles with the removal of the iron and manganese. In addition, water released at approximately 7.1 m^3/sec (250 cfs) through an aerating valve for the floatway does not provide sufficient oxidation time for the iron and manganese. The oxidation rate for the iron and manganese are on the order of hours for the iron and days for the manganese. Therefore, the iron and manganese oxidize and precipitate in the creek, leaving it highly stained, with large growths of iron bacteria, and with precipitates coating much of the aquatic life below the stream. An additional problem occurs when the reservoir level drops below the spillway crest. In order to maintain minimum streamflow of about 0.2 to 0.3 m^3/sec (7 to 10 cfs), a small, unaerated valve is used to provide the minimum releases because the flows are too small to use the aerating valve. This operation not only results in iron and manganese problems as described above, but concentrations of hydrogen sulfide as high as 0.5 mg/L have been detected in the creek. Hydrogen sulfide concentrations greater than 0.002 mg/L can be toxic to aquatic life.

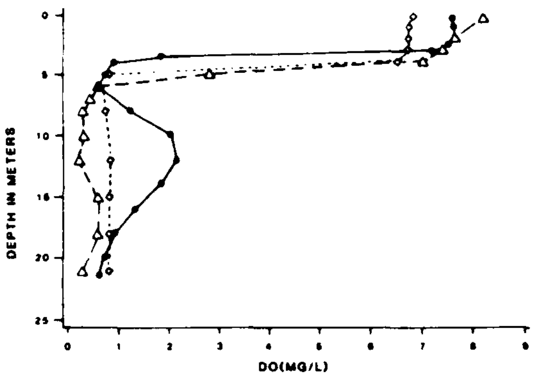

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Figure 2 Dissolved Oxygen Versus Depth Profile at Bear Creek Mile 114.8, June 25, September 3, and October 17, 1980

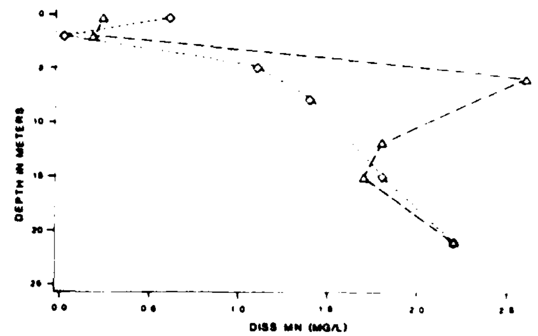

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Figure 4 Dissolved Manganese Versus Depth Profile for Bear Creek Mile 114.8 September 3 and October 17, 1980

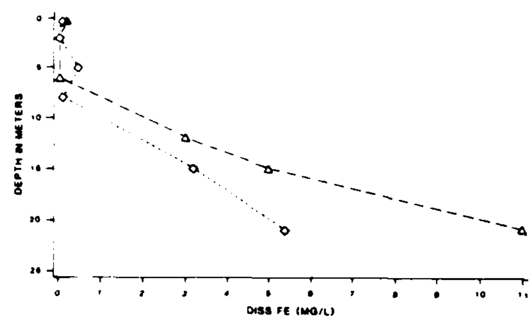

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Figure 3 Dissolved Iron Versus Depth Profile for Bear Creek Mile 114.8, September 3 and October 17, 1980

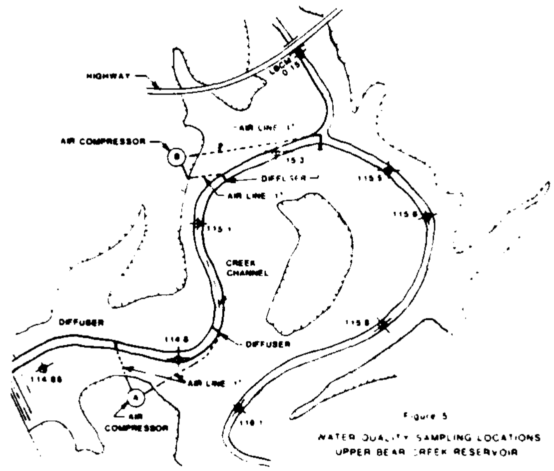

Figure 5
WATER QUALITY SAMPLING LOCATIONS
UPPER BEAR CREEK RESERVOIR

Typical profiles of DO, iron, and manganese are shown in figures 2, 3, and 4.

A diffused aeration system was installed in Upper Bear Creek Reservoir in the late spring of 1986. Four diffusers were used. The diffusers were placed across the old river channel, spaced about 150 to 300 m (500 to 1000 feet) apart. The locations are shown in figure 5. Each diffuser was constructed of 7.6 cm (3-inch) PVC pipe mounted in an aluminum frame which holds the pipe approximately 0.9 m (3 feet) above the bottom of the reservoir. Each diffuser consists of two 6.1 m (20-foot) sections of pipe joined by a coupling. 1.0-mm (0.039 inches) holes were drilled in each side of the pipes on 0.3 m (1-foot) spacings. One end of the 12.2 m (40-foot) diffuser is capped and the other fitted with a quick-connect fitting for a 2.5 cm (1-inch) air hose. Figure 6 shows the diffusers.

Figure 6. Diffusers

Two 1.4 m^3/min (50 cfm), 8.8 kg/cm^2 (125 psi) compressors, supply the air with

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one compressor for each two diffusers. The compressors are to run continuously from about the first of May through September.

System Operation

Operation of the aeration system was delayed due to a failure of the original coupling connecting the two sections of each diffuser. Additional delay was caused by two diving accidents during installation. The delays resulted in initiating operation after the reservoir had stratified.

Soon after operation began, the water treatment plant, which has its intake at the dam, experienced a change in water quality which significantly upset plant operation. The aeration system was then shut down for two weeks. The upper two diffusers which are approximately 1.2 to 1.6 kilometers (0.75 to 1.0 miles) upstream from the dam were then operated for approximately one week after which time the water treatment plant again experienced conditions of fluctuating water quality which again resulted in significant upsets of the plant's operation. The aeration system was again shut down. After three weeks the two diffusers were operated but this time for only a day and a half. Several days later, the water treatment plant again experienced an upset.

The difficulty experienced by the water treatment plant was thought to be caused by the slow oxidation and sedimentation of the manganese in the water upwelled by the aerators. Because the oxidation rate of the manganese is on the order of days, it was not removed by the time it reached the water treatment plant intake. It is hoped that under normal operation of the aeration system in which the system is started *before* the reservoir stratifies, the deep portions of the reservoir will stay aerobic and prevent the manganese from becoming soluble.

CHEROKEE RESERVOIR

During late summer, the temperature in the epilimnion of Cherokee Reservoir exceeds the optimum temperature of 22 to 24°C (72 to 75°F) for striped bass, a very important gamefish in the reservoir. The striped bass seek cooler parts of the reservoir during critical temperature periods. Unfortunately, the hypolimnion, which is cooler, contains insufficient concentrations of DO. The bass, therefore, seek refuge elsewhere to wait out the duration of the stressful conditions. The more popular refuge areas are embayments fed by cool spring water high in DO. However, there have been fish kills over the years because the refuges were not large enough for all the fish, the refuges were unable to maintain cool enough temperatures, the DO was not sufficiently high, or all of these factors.

Because of the need for additional and improved refuges, an artificial refuge was created for the striped bass in the reservoir. The refuge approach was selected due to the high cost of trying to aerate in the reservoir itself. Even if the reservoir could be aerated, it was not certain that the water would be sufficiently cool to sustain the bass.

Description of the Dam and Aeration System

A small embayment, shown in figure 7, at Holston River mile (HRM) 55 was selected for the refuge. The refuge was constructed by placing a submerged dam across the mouth of a small embayment to trap cold water in the spring of the year and retain it throughout the summer. The water trapped by the dam was aerated using liquid oxygen and a diffuser system placed on the bottom of the embayment.

Figure 7. Refuge Embayment

Submerged dam

The submerged dam is made from Hypolon strips cemented together in 1.5 m (5-foot sections. Approximately 96 m (315 feet) of the fabric is used to span the mouth of the cove. The fabric is up to 21 m (70 feet) wide in the middle where the embayment is deepest, and tapers to approximately 12 m (40 feet) along the ends. The dam is about 15 m (50 feet) high at the deepest point. The fabric is folded at the bottom to provide a tail which lies flat along the bottom of the embayment. The bottom is sealed by placing gabions filled with sandbags on top of the tail. Because of the steepness of the side slopes, the gabions are held in place by cables connected to anchor pins drilled and grouted into the reservoir side.

The top of the fabric is suspended from a 9.5 mm (3/8-inch) cable which is bolted to two pins on each side of the embayment. One pin laterally restrains the cable and the other pin vertically restrains it.

When the reservoir is unstratified, there are no loads on the dam and the fabric hangs limp. When the reservoir becomes stratified and the temperature on either side of the dam is equal, the dam remains limp. However, as the cold water is pulled from the hypolimnion for power generation, the thermocline is lowered resulting in higher temperatures at lower

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elevations in the reservoir.[1] Because the water behind the submerged dam is isolated from the effects of the turbine releases, the thermocline remains at its original depth behind the submerged weir. This, of course, causes a temperature difference to occur between the water behind the dam in the embayment and water in the main body of the reservoir. The cooler and more dense embayment water pushes against the fabric dam billowing it out much like a sail. When the maximum temperature difference is achieved, there is as much as 6360 kg (14,000 pounds) of force acting vertically on the main support cable. This load is supported with 66 underwater buoys attached to the main support cable. When the dam is unloaded (no temperature difference exists) the buoys are held down by concrete anchors on the embayment floor. As the temperature difference develops, the heavier water inside the embayment begins to pull the fabric down, relaxing the tension in the anchor lines between the buoys and the buoy anchors and allows the vertical load acting on the fabric to now be borne by the buoys. Figures 8 and 9 schematically depict the fabric dam. The design criteria for the dam are presented in table 1.

The oxygen system requires approximately one-fifth the gas flow rate of an air system and thus much less water is upwelled. This reduces the chance of losing the cold water over the top of the barrier. The diffuser system is constructed of six 0.9 m (3-foot) by 7.6 cm (3-inch), fine-pore ceramic diffusers (manufactured by Marox Corporation) mounted end to end on a galvanized pipe frame (see figures 10 and 11). There are 13 pipe frames each approximately 6.1 m (20 feet) long arranged as shown in figure 12. The oxygen is supplied to diffusers at a rate of 45 to 90 kg/day (100 to 200 lbs/day).

Because no access roads could be provided to an onshore oxygen storage site, oxygen is barged to the refuge. A 6.4 m (21-foot) pontoon boat was fitted for storing four 341 kg (750-pound) liquid oxygen tanks. The tanks are connected to a common manifold, followed by a metering system, before the oxygen is delivered to the diffusers. The oxygen requirement is low enough that vaporizing devices are not required. The oxygen barge and mooring facilities are depicted in figures 13, 14, and 15. Approximately every 7 to 21 days, depending on the oxygen flow rate, the oxygen barge is taken to a nearby boat launching facility where the empty oxygen tanks are exchanged for full ones.

Table 1

Design Data

Length of dam on area	96 m (315 ft.)
Distance between anchor pins	88 m (290 ft.)
Bottom elevation	293 m (960 ft.)
Top of dam elevation	207 m (1008 ft.)
Number of buoys and anchors	66
Number of gabions	29
Number of sandbags	1300
Length of anchor rope	1520 m (5000 ft.)
Fabric	*4 mil Hypolon®*
Maximum temperature difference	8°C (14°F)
Normal reservoir full pool elevation	328 m (1075 ft.)
Minimum reservoir full pool elevation	811 m (1020 ft.)
Diffuser length	79 m (260 ft.)
Oxygen flow	Up to 114 kg/day (250 lbs/day)
Oxygen storage	Four 341 kg (750 lb) dewars
Volume of refuge	$1.4 \times 10^5 m^3$ (117 ac-ft.)
Refuge surface area at elevation 1008.5	$2.7 \times 10^4 m^2$ (6.6 ac.)
Average depth	5.5 m (18 ft.)
Maximum depth of coldwater pool	15 m (48 ft.)

Aeration system

One of the design objectives for the aeration system was to lose as little of the cool water behind the dam as possible. Therefore, high purity oxygen was selected for the aeration system in preference to a diffused air system.

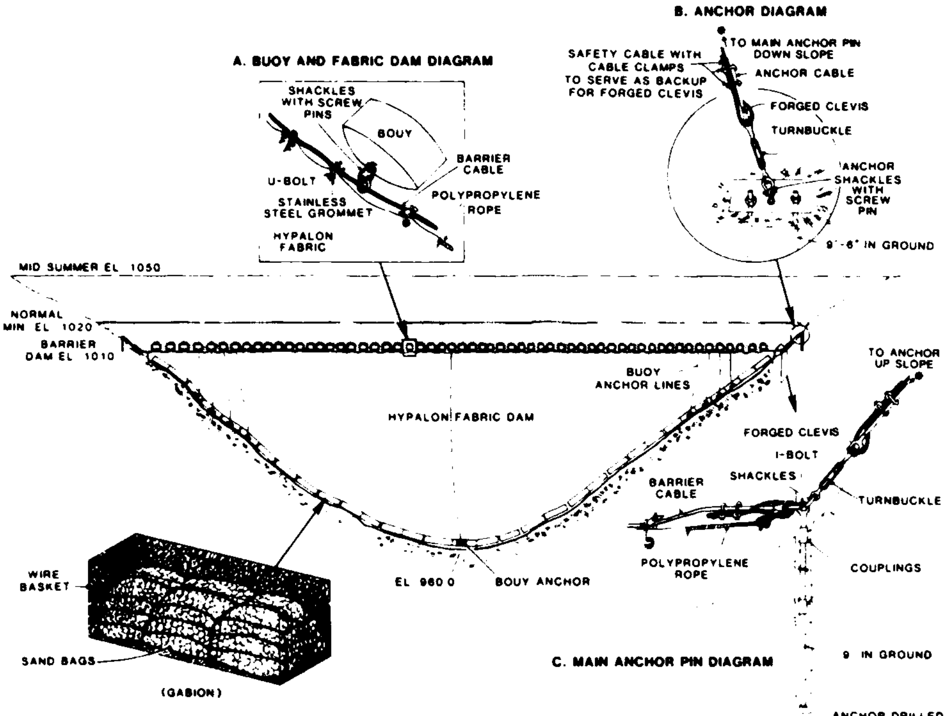
Table 1 lists the more important design features of the dam and the aeration system.

System Performance

Figure 16 shows the effect of the submerged dam and the aeration system on temperature and DO. As figure 16 indicates, the temperature in the embayment behind the dam during July was 4 to 6°C (6 to 11 °F) cooler than water at the same elevation outside the refuge. DO levels of

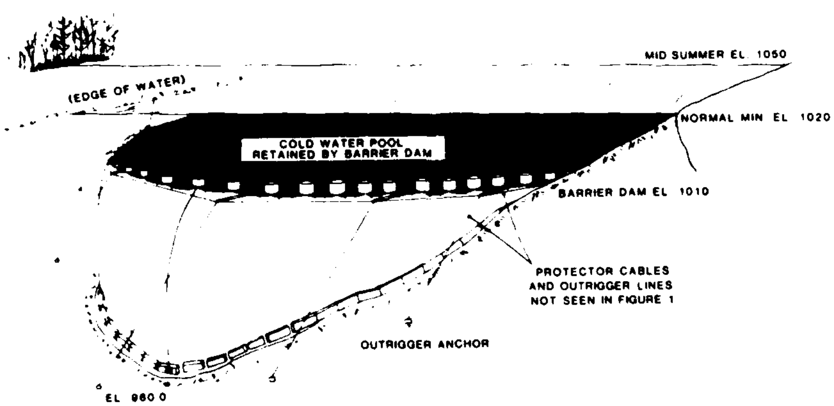
1. The intakes to the hydropower turbines at Cherokee Dam are located deep in the hypolimnion. Because Cherokee is one of the main hydropower generation facilities on the TVA system, it is used as much as possible during summer months to meet high peak air conditioning demands. The result is that the hypolimnion is generally depleted of cool water by late August.

FABRIC BARRIER DAM ; CHEROKEE RESERVOIR



Schematic of Dam

Figure 8



Concept of Dam

Figure 9

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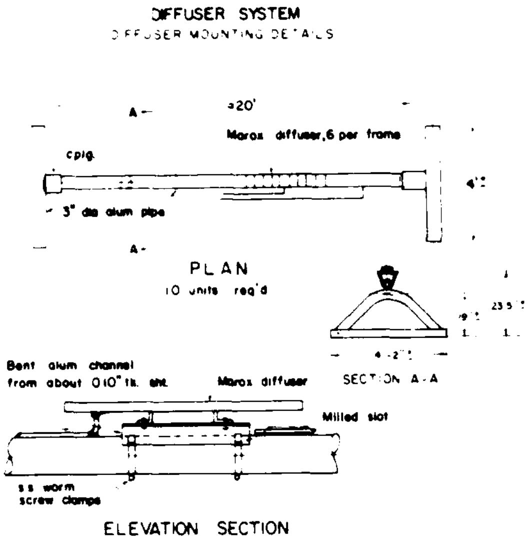

Figure 10

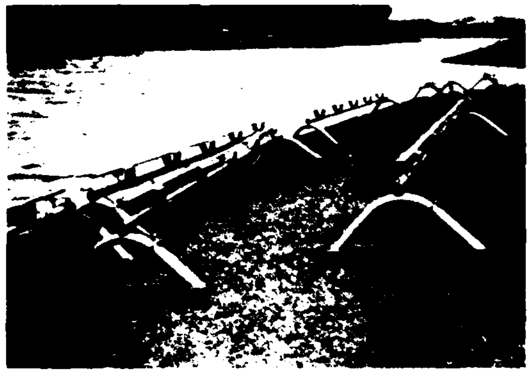

Figure 11. Oxygen Diffusers

CHEROKEE STRIPED BASS REFUGE: LOCATOR AND COLD WATER POOL MAPS

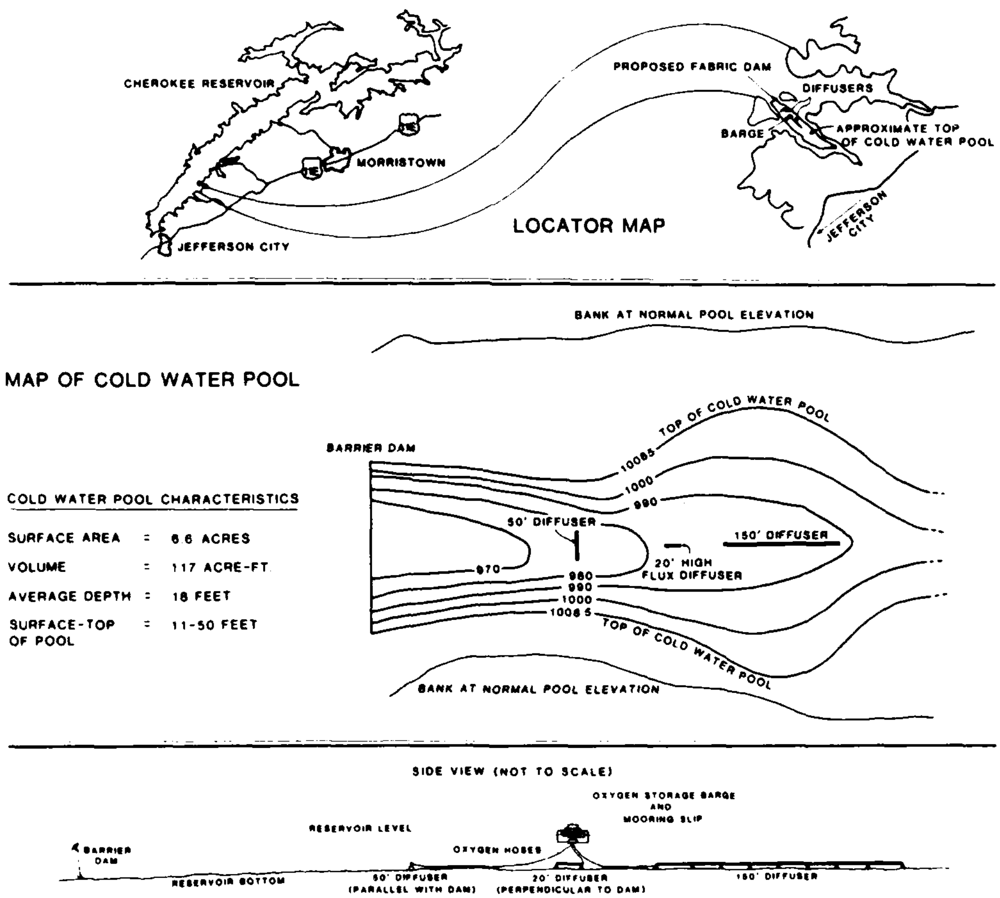

Figure 12

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OXYGEN HEADER AND OXYGEN BARGE

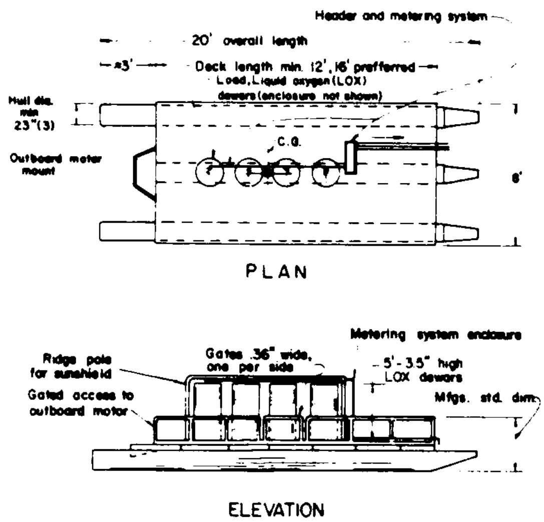

Figure 13

Figure 15. Oxygen Barge and Mooring Facility

MOORING FACILITY FOR OXYGEN BARGES

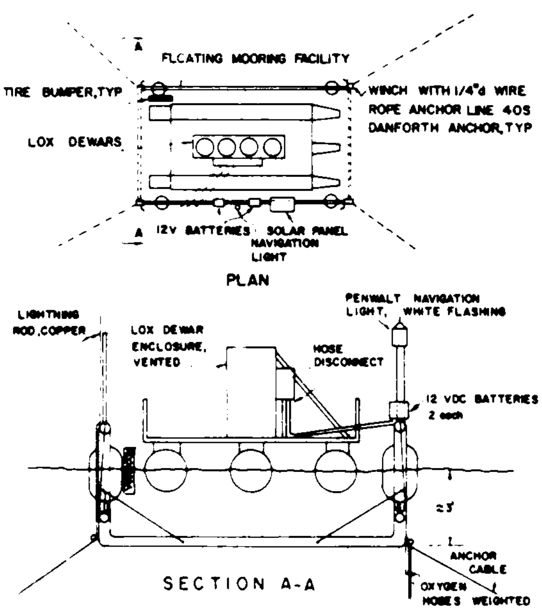

Figure 14

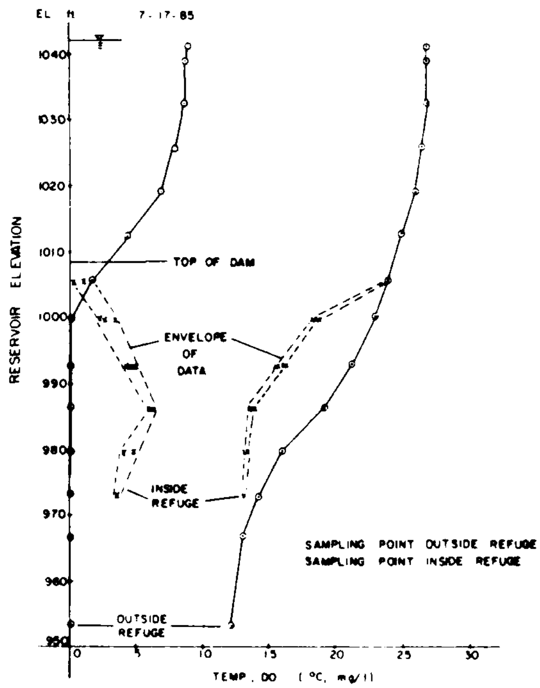

Figure 16. Temperature and DO, July 17, 1985

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up to 6 mg/L were maintained inside the refuge while DO dropped to zero outside the refuge.

By comparison to a nearby control cove of similar features, large numbers of forage fish, primarily alewife, are attracted to the artificial refuge. Although the high oxygen concentrations, cool water, and abundant food are provided, striped bass have not appeared in the cove in the same numbers that frequentsome of the natural refuges. Part of the problem might be caused by the anoxic layer that forms between the top of the dam and the bottom of the epilimnion as seen in figure 16.

The artificial refuge has been in operation for two summers. Efforts are continuing to make the artificial refuge more attractive to the striped bass.

ACKNOWLEDGMENTS

The conceptual design of the Upper Bear Creek aeration system was performed by David McIntosh. Herman Herren did the detailed design and headed up the installation team. The idea for the striped bass refuge provided by the underwater fabric dam and aeration system was that of Jim Ruane and Dick Shane. The preliminary engineering to develop the concept came from Morgan Goranflow and Steve Adams. Ron Pasch and Bill Seawell provided the design criteria for the refuge. The final design ofthe dam was performed by Dave Hegseth. Robert Hunter and his diving crew constructed the dam. David McIntosh developed the design criteria for the refuge aeration system and Marvin Smith designed the system. The aeration system was operated by David Page and Paul Smith. Fisheries data were obtained and analyzed by Jim Baker.

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EPILIMNETIC PUMPS TO IMPROVE RESERVOIR RELEASES

M. H. Mobley[1] and E. D. Harshbarger[2]

[1]Project Engineer, Tennessee Valley Authority, Engineering Laboratory, Norris, TN
[2]Supervisor, Research and Testing, Tennessee Valley Authority, Engineering Laboratory, Norris, TN

ABSTRACT

The Tennessee Valley Authority has installed high volume, low speed, axial pumps in Douglas Reservoir just upstream from the turbine intakes. The purpose of these pumps is to force highly oxygenated epilimnetic surface water into the turbine intakes when the reservoir is thermally stratified and the bottom water is low in dissolved oxygen. A description of the test installation is given, the results of preliminary tests are presented and plans for future tests are discussed.

INTRODUCTION

Low dissolved oxygen (DO) content is a common water quality problem in the hydroturbine discharges from many deep reservoirs. In addition, cold water temperatures can be a problem if the discharge area is a warm water fishery.

Most of the low temperature and dissolved oxygen problems in turbine discharges can be attributed to thermal stratification of the reservoir from which the turbine flow is obtained. Usually, the turbine intake is located at a low level and withdraws only relatively cold, poorly oxygenated hypolimnetic water. Often, highly oxygenated, warmer water is available in the epilimnion, but cannot move downward through the thermocline to reach the turbine intake because of density differences.

Professor J. E. Garton (retired), of Oklahoma State University developed the concept of using a high volume, low speed, axial pump to locally destratify the reservoir and move surface water into the turbine intake withdrawal zone. This general technique has been applied by several others (Quintero and Garton, 1973; Steichen, 1974; Strecker, 1976; Garton and Rice, 1976; Garton and Jarrell, 1976; Dortch and Williams, 1978; Garton and Peralta, 1978; Punnett, 1978; Garton and Punnett, 1980; Garton and Miller, 1982; Robinson, 1982), but has never been permanently installed in deep water, high flow situations.

The Tennessee Valley Authority is currently installing three of these "epilimnetic" pumps at Douglas Dam to investigate their use in a high flow, relatively deep reservoir application.

BACKGROUND

Douglas Dam is located at mile 32.3 of the French Broad River in east Tennessee. The city of Knoxville is located some 30 miles downstream at the confluence of the French Broad and Holston Rivers. The dam is equipped with four hydroelectric generating units which discharge about 120 m^3/s (4,000 cfs) each. Thermal stratification in Douglas reservoir is relatively strong during late summer. Surface temperatures are normally near 25°C (77°F) while temperatures at turbine intake level are about 13°C (55°F). The usual depth from surface to the intake centerline is about 25 m (80 ft) during the stratified time period. DO in the hypolimnion often drops to near zero, and therefore, low dissolved oxygen in the discharges is an annual occurrence during the summer months. This low DO contributes to waste assimulation problems in the downstream area, and is detrimental to fish and other aquatic life habitat.

Several methods of increasing the DO in the turbine discharges have been tested (Harshbarger, 1982, 1983, 1984) none of which have proven feasible from both a technical and economic viewpoint. The current tests undertaken by TVA are to provide a solution to the Douglas DO problem and at the same time to provide basic information concerning the application of such devices to other locations having large flowrates, deep turbine intakes, and fairly strong thermal stratification.

DESCRIPTION OF EQUIPMENT

Each of the three epilimnetic pumps being tested by TVA consists of a 6-bladed, 4.6-m (15-ft) diameter propeller connected by a shaft and gear box to a 30 hp electric motor. The motor, gear box and controls are supported on a 4.9 m by 4.9 m (16 ft by 16 ft) square floating platform on the water surface, while the propeller is suspended beneath the platform at a depth of approximately 3 m (10 ft). The propeller is turned at 21 rpm and is expected to move approximately 1.4 m^3/s (500 cfs) of water down some 25 m (80 ft) to the level of the turbine intakes.

Much of the machinery required for construction of the epilimnetic pumps was purchased from Union Electric Company. Robert Miller of Union Electric and Professor Garton used this equipment in their tests at Bagnell Dam on the Lake of the Ozarks in 1981 (Garton and Miller, 1982). The equipment purchased from Union Electric Company included:

1. 80:1 right angle gear reducers
2. 30 hp electric motors
3. 4,575-mm (180-in.) dia. cast aluminum cooling tower fans

4. shafts, motor/gear reducer mounting frame, guide bearings

To complete the test apparatus, TVA designed three flotation platforms consisting of square steel tubing frames, aluminum clad polystyrene flotation billets, and wooden decks and handrails.

To meet TVA safety requirements, a protective cage made of fence wire attached to a steel framework was designed to enclose the propellers and keep swimmers out of danger. The platforms were fitted with large signs on every side that warned of "danger", "high voltage", and "rotating machinery". Two lighted buoys were placed out from the platforms to warn away boaters both day and night.

A system of telescoping "stiff-legs" consisting of a 0.3 m (1 ft) diameter pipe, steel tubing and guy cables was designed to connect the group of three pumps to the face of the dam and allow flexibility in positioning the pumps for various tests. In order to respond to the 18.3 m (60 ft), fluctuations in the reservoir level, the stiff-legs were connected to the face of the dam with a sliding connection. This connection consisted of a short "I" beam section that was bolted onto the end of the stiff-legs and slid in a "C" shaped channel that was bolted to the face of the dam.

DESCRIPTION OF CONSTRUCTION

At the TVA Engineering Laboratory, each flotation platform was assembled as a unit with the gear reducer and electric motor mounted and aligned. The protective cages were built in sections to allow easy transportation and assembly. The telescoping stiff-legs were assembled complete with flotation and sliding connection to be bolted to the face of the dam. All of these components were loaded on two "WIDE LOAD" trucks for transportation to Douglas Dam.

Scaffolding was erected at a site on the North abutment of Douglas Dam to support the platforms for final assembly. When the partially assembled pump units arrived at the dam, they were lifted with a crane to support beams on top of the scaffolding. Once the platforms were located on the scaffolding, the final assembly of the submerged part of the pumps began. This included, installing the shaft, making the chain couplings on the shaft, building the supports to the lower guide bearings, installing the propeller hub and blades, setting the pitch of the propeller blades, and assembling the protective cage.

Using a crane, each completed pump was removed from the scaffolding and carried out on the deck of the dam, where it was lowered into the reservoir. The first pump to be assembled was damaged in a crane accident as it was being removed from the scaffolding. It was salvaged and reassembled using pieces of the third pump which will not be completed until replacement parts are procured. The succeeding crane lifts were done with a larger crane and went smoothly.

Boats were used to maneuver the floating pumps into position in front of the intakes of Unit 4. The two completed pumps were connected into one rigid unit, with a .3 m (1 ft) space between the floating platforms. The pumps were held out from the dam with two floating stiff legs. The telescoping stiff-legs were adjusted so that the closest edge of the platforms was 7.62 m (25 ft) from the face of the dam. This was chosen as the closest pump location so that the downward moving water pushed by the pumps would clear the top of the cylindrically shaped intakes at Douglas that extend out 6 m (20 ft). The telescoping stiff-legs were capable of extending the pumps out an additional 4.6 m (15 ft). The torque of the operating pumps was counteracted by the stiff legs and guy cables that were attached to the outside corners.

Electrical power was supplied by a 100 amp line from the dam that fed into three motor control boxes mounted on the handrails on the deck of the dam. The power cables were suspended from the control boxes out to the platforms. A system of pulleys and weights was used to tension the cables and allow movement with reservoir fluctuations.

TESTS PERFORMED AND RESULTS

Due to the time required to purchase and assemble the apparatus, the epilimnetic pumps were not installed before the reservoir destratified. However, two dye studies have been performed to check the performance of the pumps and verify test procedures for future tests.

The first test was conducted on October 3, 1986. The purpose of this test was to find out what percentage of the output of the pump was being withdrawn from the reservoir through the turbine intake under the destratified conditions. The temperature difference from the surface to the depth of the intakes had been measured to be less than 2°C (3°F). The headwater elevation (HWEL) was 951.5, 16.5 m (54 ft) above the centerline of the intake.

Only the turbine equipped with the epilimnetic pumps (Unit 4) was run for the test. The procedure was to inject a known flowrate and concentration of rhodamine dye into the water above the pump propellers and then monitor the concentrations of dye in the scrollcase, at the boil in the tailrace and in the river downstream. The concentrations at the monitoring points were then multiplied by the known flowrate during the test (from the Winter-Kennedy taps on Unit 4) to get a total mass flowrate of dye to compare with that originally injected (assuming uniform mixing). Measurements taken from a boat positioned in the river about 450 m (1,500 ft) downstream from the dam reflected a mass flowrate of dye equal to about 95 percent of the mass flowrate injected. The other two measurements at the scrollcase, and in the boil in the tailrace did not reflect nearly as much dye concentration, but it was expected that these measurements were in error because of less than adequate mixing.

During preparation for the first dye test, the epilimnetic pumps were run for about 10 minutes while the turbines were all off. It was noticed that under these conditions, a plume of mud was stirred up between the pumps and the dam. This mud plume was thought to be from the bottom, some 23 to 30.5 m (75 to 100 ft) below the level of the platforms. Since there was some question about whether the pumps were actually pushing a jet that deep, a second dye test was run.

The second dye test was run October 10, 1986, while all units were off. The HWEL was 950. The purpose of this test was to determine if the pump jet was reaching to the bottom of the reservoir under the destratified conditions. Three weighted dye sampling lines were lowered to depths of about 21 to 27 m (70 to 90 ft). Two of the lines were positioned between the pump platforms and the dam, the third line was positioned between the two platforms. Both epilimnetic pumps were started along with the pump on the sampling lines, and after allowing time for the pump jet to become established, a small amount of dye was injected near the water surface between the two units. After a delay of approximately 3 minutes the water samples showed a strong peak of dye concentration, thus showing that the pump jets were penetrating to a depth of at least 27 m (90 ft).

FUTURE TESTS

The third pump unit is anticipated to be completed sometime in the spring of 1987. The arrangement of the third pump in relation to the existing two pumps and the arrangement of the assembly with respect to the intakes will be studied in a literature search and possibly a physical model study. The purpose of the model study would be to determine the optimum positioning for penetration and maximum flow from the surface to the turbine intakes.

Once the lake has begun to stratify a series of mass flow dye tests will be run to determine the percentage of the pump jet to reach the turbine intake versus depth and stratification strength.

Once the third pump unit is installed and operating, a test will be run during strongly stratified conditions to determine the maximum improvement in the quality of the turbine discharge gained by operation of the pumps. For this test the DO and temperature of the reservoir and turbine discharge will be monitored.

CONCLUSIONS

At this time, TVA has epilimnetic pumps installed at Douglas Dam that show potential for improving the releases from the low level turbine intakes. Tests to date have been run only in low HWEL, destratified conditions; therefore, the effectiveness of the pumps under strongly stratified conditions at summer levels is still unknown. However, the pumps have performed well thus far and show promise for success next spring.

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LOCAL DESTRATIFICATION SYSTEM FOR MARK TWAIN LAKE

Steven C. Wilhelms[1] and Thomas J. Furdek[2]

[1] Research Hydraulic Engineer, USAE Waterways Experiment Station
[2] Environmental Chemist, Water Quality Section, USAED, St. Louis

ABSTRACT

Releases during start-up of generation at Mark Twain Lake (MTL) have been cold and low in dissolved oxygen. During periods of no generation, cold poor-quality water has gradually accumulated in the turbine tailbay. The authors present observed data and describe the processes that resulted in these observations. The minilake, which is the region in MTL between the main dam and the temperature control weir (TCW), restratifies when generation ceases if the lake thermocline and oxycline are above the elevation of the TCW. The authors present solutions and recommend that a hydraulic destratification system be installed to destratify the minilake. The design parameters and constraints are shown and the system design is developed. A design for the diffuser system is suggested.

INTRODUCTION

Mark Twain Lake (MTL), on the Salt River near Hannibal, Missouri, exhibits summer stratification that results in anoxic conditions in the hypolimnion. Hydropower releases, which are generally withdrawn from deep in the pool, are usually cold and low in dissolved oxygen (DO) for these lakes. Ordinarily this problem would be overcome at MTL with the temperature control weir (TCW), which is an earthen dike approximately 122 m (400 ft) upstream of Clarence Cannon Dam (CCD) (Figure 1). The TCW, with crest at el 177 m* (580 ft),

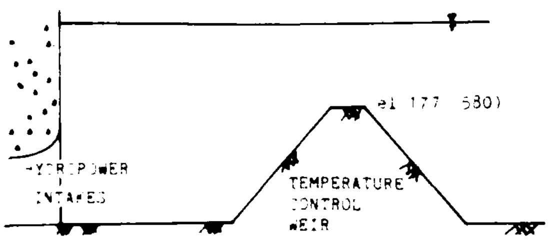

Figure 1. Schematic of CCD with TCW.

would normally act as a skimming weir to allow withdrawal of warmer higher quality epilimnetic water. However, during the last three stratification seasons (1984-86), the thermocline formed at an elevation well above the crest of the TCW, which would impair the skimming capability of the TCW. The high thermocline (relative to the TCW crest) also caused hypolimnetic conditions to be established in the area between the TCW and CCD.

* All elevations and stages cited herein are in metres (feet) referred to the National Geodetic Vertical Datum (NGVD).

Thus, when hydropower or low-level releases were made, they were cold and contained low DO. However, data collected during the 1984 turbine tests tended to confuse this scenario. The causes of and recommended solution to this problem and explanations of the apparent anomalies in the observed data are discussed in this paper.

DATA AND PROCESSES

DO and temperature data were collected at several locations at the project: (1) main lake, upstream of the TCW; (2) minilake, the region between the main dam and TCW; and (3) downstream of the powerhouse. The DO and temperature profiles shown in Figure 2 were taken on 19 August, 23 August, and 13 September 1984. Similar profiles were observed in 1985 and 1986. In 1984 turbine operation tests were being conducted that had a significant impact on the profiles observed in the minilake. Examination of Figure 2 will reveal that the minilake did not have the same temperature profile as the main lake. However, the minilake profiles were taken after turbine tests were conducted. The profiles imply that the water in the minilake was withdrawn and replaced with water from the stratum near the TCW crest elevation. Consequently, the water was warmer in the depths of the minilake than in the main lake, but was still anoxic since the oxycline was above the TCW crest.

Observations made during turbine tests confused these conclusions. In the early part of the Kaplan turbine test, the "speed, no-load" condition was established with a very low discharge rate. During this time the DO content of release water was very high, nearly as high as the DO content of the epilimnion. As the test progressed, and the discharge from the turbine increased, the DO decreased. However, after about 40 minutes of operation the DO began to increase accompanied by an increase in temperature.

During the speed, no-load portion of the test, the low discharge (about 28.3 m^3/sec (1,000 ft^3/sec)) through the Kaplan turbine would have caused the vacuum-breaker system to function to prevent cavitation. At this very low setting, large volumes of air would have been aspirated into the draft tube region just below the turbine wheel. Thus, the DO uptake as this air-water mixture traveled to the tailrace area was quite significant. As the turbine discharge increased, the amount of air aspiration decreased, resulting in lower DO uptakes and a decrease in the release DO. However, because of the relatively small volume of water between the TCW and main dam, this was a short-lived phenomenon. Once the volume of the minilake had been withdrawn, the skimming action of the TCW caused the release temperature

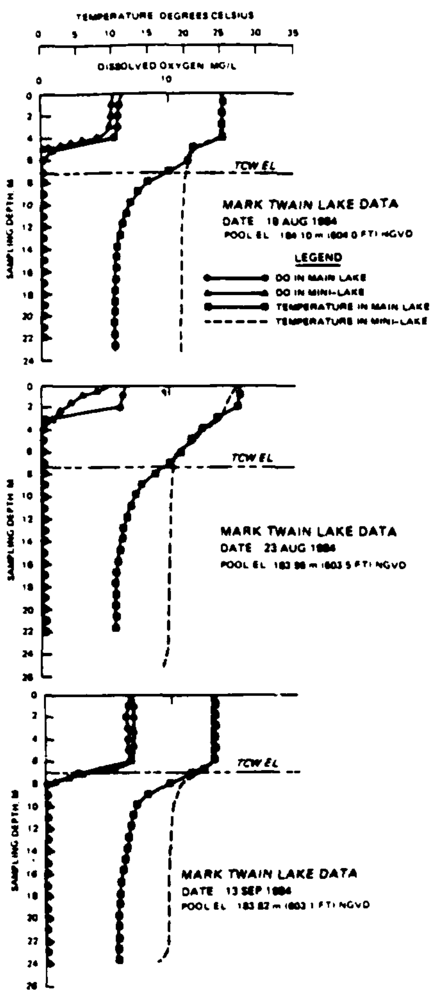

Figure 2. Mark Twain Lake temperature and DO profiles.

as well as the release DO to increase.

This conclusion was verified with data from the test of the second turbine unit at CCD: a pump-turbine. The pump-turbine unit does not have a vacuum-breaker system, and during its start-up tests, low or zero DO was observed in the releases. However, after approximately 40-50 minutes, the DO began to increase accompanied by a temperature increase. This result illustrates the impact of the minilake and the TCW.

It appears from comparison of the 1984 in-lake and minilake temperature and DO profiles and evaluation of the release DO and temperature values produced by operation of the CCD powerhouse that the major cause of the initial cold low DO release is the physical restratification of the minilake. With the thermocline and oxycline above the crest of the TCW, the minilake restratifies. This restratification results in the low DO content and cold temperature of releases until the TCW can influence the quality of the water being withdrawn.

Main lake profile data from 1985 and 1986 also showed that the thermocline and oxycline were above or near the crest of the TCW during most of the stratification period of June through October. Minilake profile data for this same period also indicated that the minilake was restratifying after generation shutdown. The temperature and DO profiles in the minilake approximated the main-lake profiles from the surface to the crest of the TCW. Below the TCW, the minilake was homogeneous with temperature and DO content of water at the TCW crest elevation.

In May 1986 a Hydrolab Data Sonde II water quality monitor was installed in the tailbay of the hydropower facility. The monitor was coupled with a data collection platform that transmits the measured values of the water quality parameters to the GOES satellite system for retrieval. Thus, a continuous record of water quality data in the turbine bay tailrace was obtained. These data showed the same trends as the 1984 data indicating that the same processes that were at work in 1984 were still affecting release quality.

An additional problem was discovered when measurements in the turbine tailbay indicated that cool hypolimnetic water with a very low or zero DO content was gradually replacing the just-released epilimnetic water after generation shutdown. After extended periods of no generation, the quality of the water in the turbine bay approximated the quality of the water in the hypolimnion of the minilake. Based on these observations, it was concluded that the source of this cold poor-quality water was leakage through the turbines and that these conditions posed a potential concern whenever the main lake thermocline and oxycline were above the elevation of the TCW.

POTENTIAL SOLUTIONS

The CCD hydropower project is a peak power generation facility. Power production is on demand and has no fixed schedule. Slight modification to ordinary operations could, at least partially, eliminate the problem of low DO in the release water. For example, at power plant start-up during the critical stratification periods, the Kaplan unit could be operated at speed, no load until the volume of the minilake has been flushed downstream. At this juncture, however, it is unknown if this could be a workable solution.

Future operating procedures at CCD are extremely important, particularly given that the project has pumped storage capabilities. Pump-back, if implemented on a regular basis, could significantly impact the stratification in the main lake and the minilake. Because of the turbulent mixing and entrainment of the pumpback jet as it flows over the TCW, some depression of the thermocline in the lake to an elevation below the TCW crest could be anticipated. This effect would result in warmer higher quality water being withdrawn into the minilake for release at the startup of generation. It is important to note that frequency of pumpback would determine the extent of these effects and that other techniques to improve release quality may have to be adopted.

Another potential solution is to raise the elevation of the TCW crest. This would prevent the restratification of the minilake with cold, low-DO hypolimnetic water since that water would be below the crest elevation and could not flow into the minilake. Additionally, the higher crest would raise the withdrawal zone to the upper levels of the lake, thereby improving the skimming action of the TCW. Operational considerations to improve release DO during start-up time should be unnecessary with this alternative. This alternative would, however, require study in a physical hydraulic model to assure that flow conditions approaching the structure would be satisfactory with the higher TCW.

Hypolimnetic aeration and oxygenation are potential solutions to the low-DO release problem. For these alternatives, the hypolimnion oxygen content would be improved by aerating or oxygenating the hypolimnion. However, neither of these alternatives would impact the transitory effects of releasing cold water during turbine start-up, nor address the potential problem of cold water leakage through the turbines and accumulation in the tailbay. Further, these alternatives would act on the hypolimnion, when in fact, the causative agents appear to be in the TCW and minilake and resulting hydrodynamics.

Total lake destratification would prevent the occurrence of cold hypolimnetic water and low DO since temperature and chemical stratification would not exist. However, the system and energy requirements to implement this alternative could be extensive, and total lake destratification may not be desirable for other project purposes. Using the concept of destratification in a localized area could potentially produce the results desired at CCD. The TCW provides the luxury of a minilake that could possibly be maintained in a destratified condition. This system and its energy requirements could be much more attractive than total lake destratification.

Turbine venting through aspiration or forced-air injection into the flow in the power project could favorably impact the release DO although this alternative would not affect release temperatures. As described earlier, Kaplan turbines aspirate air at certain operating levels. However, aspiration usually occurs only at lower discharges. Thus, such additional measures as deflector plates must be taken to induce aspiration, or compressed-air injection must be employed. This alternative may not prove to be the complete solution, but it should improve the release DO.

RECOMMENDED ALTERNATIVE

The recommended approach for the solution of the problem of transitory cold, low-DO releases at CCD is the use of a local destratification system coupled with, if necessary, the use of turbine venting. Hydraulic destratification of the minilake would prevent temperature and oxygen stratification without requiring the capital and energy outlay for a system to destratify the entire lake. Regardless of the operating procedures adopted for power production, or the level of the lake (which impacts the thermocline elevation), the temperature and DO content of the minilake could probably be maintained at an acceptable level. The use of local hydraulic destratification would avoid the potential for nitrogen supersaturation that might be encountered with a pneumatic destratification system. An additional benefit of the destratification system is the elimination of the cold poor-quality water leaking through the turbines. Since hypolimnetic water would not be present in the minilake, the leakage would not be cold or low in DO.

DESIGN PARAMETERS, CONSTRAINTS, AND GEOMETRY

To be effective in preventing the release of cold low-DO water, the destratification system must be able to break up the stratification in the minilake in the time period between generation cycles. It was anticipated that generation would start about 2:00 pm and continue until about 12:00 am; thus, there would be about 14 hours of down-time from generation shutdown until the next generation cycle. The destratification system must therefore perform its task in this 14-hour time period.

The following parameters were selected for the design:

a. Length of diffuser: 122 m (400 ft)

b. Diffuser ports: 13-mm (1/2-in.) diameter; 30.5 cm (1 ft) on centers; 400 ports

c. Time for destratification t_{80}: 50,400 sec (14 hours)

d. Volume of minilake with normal el 184 (604) V_R: 1.491×10^6 m^3 (368.5 acre-ft)

e. Density difference between[1] epilimnion and hypolimnion $\Delta\rho$: 0.00329 gm/cm^3 (0.00638 slugs/ft^3)

f. Reference density ρ: 1.0000 g/cm^3 (1.9403 slugs/ft^3)

g. Depth of the diffuser d_R: 25.6 m (84.0 ft)

h. Acceleration due to gravity g: 9.81 m/sec^2 (32.2 ft/sec^2)

Using these data and the design relationships recommended by Dortch (1979), Dortch and Holland (1980), and Holland and Dortch (1984), dimensionless time to "80 percent mixed[2]" t_{80}^* as a function of system discharge Q is

$$t_{80}^* = \frac{t_{80}Q}{V_R} = 0.00314Q \qquad (1)$$

Holland and Dortch (1984) developed a relationship between dimensionless time and the destratification densimetric Froude number F_r

$$t_{80}^* = 0.204Fr^{-0.57} = 0.204\left(\frac{V_p}{2g\,\frac{\Delta\rho}{\rho}\,d_R}\right)^{-0.57} \qquad (2)$$

[1] Analysis of 1985 prototype stratification data indicated that this density difference was maximum and occurred over a period when lake stability was maximized.

[2] The "80 percent mixed" design point approximated the fully mixed condition for the purpose of system design.

where

V_p = average velocity through diffuser port, m/sec

Substituting Equation 1 into Equation 2 and using the continuity relationship

$$Q = nA_pV_p \qquad (3)$$

where

n = number of ports in diffuser

A_p = area of individual port, m^2

the discharge required to destratify the minilake is

$$Q = 1.8\ m^3/sec\ (19.3\ ft^3/sec)$$

Using approximately 25 percent overdesign factor, the system discharge should be 2.3 m^3/sec (25 ft^3/sec). This overdesign factor should account for the most severe stratification possible as well as potential pool elevation fluctuations.

The system should be divided into five subsystems. This division should provide the most effective system since it can be operated in stages and should one system need repair, the remainder can provide the destratification needed. For this design, each subsystem would carry 0.46 m^3/sec (5 ft^3/sec) of epilimnetic water to a 24.4-m- (80-ft-) long hypolimnetic diffuser. Dortch (1979) recommended that an upward jet of epilimnetic water discharged in the hypolimnion will provide the best destratification efficiency. Therefore, the pumps, which should be submersible, should be positioned about at el 180 (592) (Figure 3). This would provide adequate

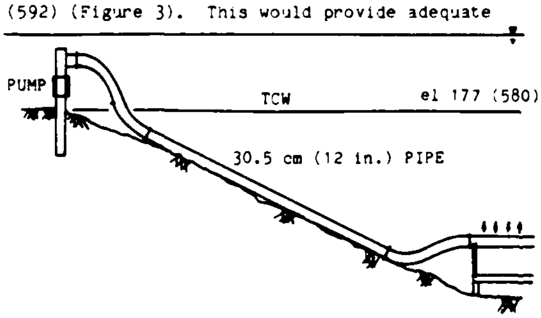

Figure 3. Pump location in MTL

submergence at low pool elevations, yet allow pumping of epilimnetic water for relatively high pool elevations during the stratification season. Complete submergence of the pumping systems would preclude the need for suction and discharge lines to enter the lake's surface, thereby relieving a potential maintenance problem because of icecover (only power and pump control conduits would enter the lake).

The diffuser should be constructed from 30.5-cm- (12-in.-) diameter pipe. There would be eighty 13-mm- (1/2-in.-) diameter ports on each subsystem diffuser. Based on head loss computations, the pump for each subsystem should be able to deliver 0.46 m^3/sec (5 ft^3/sec) at a pressure head of a minimum of 12.2 m (40 ft). To assure performance, a 25 percent overdesign factor may be applied to the pump head to account for additional losses in connections, elbows, and valves that were not included in the foregoing computations. Thus, the recommended design for each subsystem is for a pump that can produce a discharge of 0.46 m^3/sec (5 ft^3/sec) with a delivery pressure of 15.2 m (50 ft) of head.

For system installation, polyvinyl chloride (PVC) schedule 40 pipe which should provide the needed strength and corrosion resistance, should be flanged together rather than glued for ease of construction. A suggested design for the diffuser support system is presented in Figure 4. This

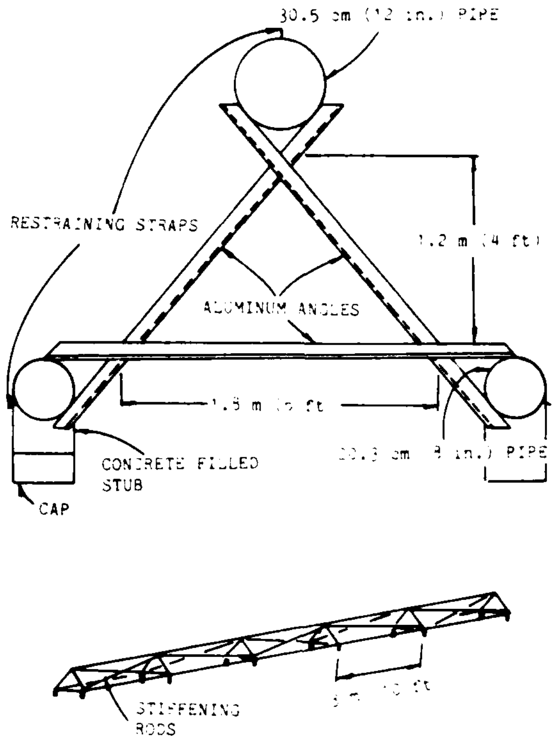

Figure 4. Suggested diffuser system design

design could be prefabricated and assembled onsite. It would provide the separation between the diffuser and lake bottom to prevent entrainment of bottom sediments. The capped PVC stubs would provide the ballast and buoyancy for sinking or retrieving the diffuser. A flexible 30.5-cm- (12-in.-) diameter hose would be used to connect the diffuser to the feedline from the pump.

The buoyancy to retrieve the diffuser system should be provided by the 20.3-cm- (8-in.-) diameter schedule 40 pipe at the lower vertexes of the triangular support brackets shown in Figure 4. Compressed air would be supplied to these pipes to cause the diffuser to float. The pipes would be flooded to allow the diffuser to sink. The ballast for the diffuser should be provided by the T sections in the flotation pipes. A capped, concrete-filled "leg" that is 61 cm (2 ft) long should be fitted into each T to provide a footing as well as added ballast.

SUMMARY

Based on analysis of temperature and DO data taken during and after hydropower operation, the transitory release of cold low-DO water is apparently due to the thermal and chemical restratification of the minilake when the thermocline and oxycline are above the crest of the TCW. For example, during generation since the TCW is skimming epilimnetic water for release, the minilake is homogeneous, the release temperature is relatively warm, and the release DO concentration is relatively high. However, when generation stops, the minilake restratifies. Because the thermocline is above the elevation of the TCW, the water at the crest (which is cooler and thus more dense than the water in the minilake) flows down the backside of the TCW and gradually fills the minilake to the crest elevation. This also causes chemical stratification of the minilake, since the water flowing into the minilake at the crest elevation is low in DO. During generation start-up, this water must be withdrawn before the skimming action of the TCW can impact the release temperature and DO.

The solution to this problem should eliminate the cause of the cold low-DO releases during generation start-up, e.g., it should prevent the restratification of the minilake. Thus, the recommended technique is a local destratification system. A hydraulic destratification system to mix the minilake would prevent or minimize the effects of stratification in the minilake.

The recommended design for the destratification system is for five subsystems. Each subsystem would consist of a pump capable of delivering 0.46 m^3/sec (5 ft^3/sec) at 15.2 m (50 ft) of pressure head feeding a 30.5-cm- (12-in.-) diameter 24.4-m- (80-ft-) long diffuser pipe with 13-mm- (1/2-in.-) diameter ports bored 30.5 cm (1-ft) on centers. This design would provide the necessary volume and momentum to satisfy the destratification requirements. PVC schedule 40 pipe should fulfill the strength and corrosion resistance requirements of the design. A suggested design of the diffuser support system is shown in Figures 3 and 4. In addition to providing the diffuser support, the piping and framework serve as ballast and anchoring weight and flotation system.

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LAKE ALLATOONA - AN EARLY EXPERIENCE IN DESTRATIFICATION REVISITED

Nathaniel D. McClure IV, P.E.

Chief, Environment and Resources Branch, Mobile District,
U.S. Army Corps of Engineers

ABSTRACT

In 1968, a rather bold experiment in lake destratification was undertaken in hope of demonstrating a viable approach for improving the dissolved oxygen (D.O.) concentrations in release waters from hydropower dams. Lake Allatoona in North Georgia was selected for a prototype installation to determine if the concept could be utilized for the then proposed West Point Project on the Chattahoochee River downstream from Atlanta. The approaches for design, fabrication, and installation of the system are presented along with a review of the State of the Art that existed at that time. The operational and testing programs are discussed and the results produced are presented. In general, the normal stratification pattern of the lake was altered to the extent that D.O. concentrations were increased at depths from 9.1 to 24.4 meters (m) (30 to 80 feet). The D.O. concentrations in the release waters were also increased. Because of the water quality improvements, especially increased D.O. concentrations, the originally envisioned temporary experimental test was operated for the past 19 years. Operational experiences and costs are discussed. A summary analysis of the overall experience is presented, including views on how the system could be improved if it were installed today.

BACKGROUND

During the 1960's, greater interest and emphasis was beginning to be placed on water quality and other environmental amenities. Prior to this time period, the design of water resource projects included very little, if any, consideration of such issues as the quality of release waters and effects of releases on downstream fishery resources. This observation is not meant to be critical, but rather reflects upon sensitivities and priorities that generally existed at both the National and local levels prior to 1970. On 1 January 1970, President Nixon signed Public Law (PL) 91-190, the National Environmental Policy Act (NEPA). This statute, which was unanimously passed by the United States Senate, illustrated that environmental matters were a National priority (Anderson, 1973). Water quality can be a vital component of the overall environmental quality, especially in conjunction with the development and operation of major water resource projects. The significance of water quality was emphasized in two Federal Statutes: PL 92-500, The Federal Water Pollution Control Act (1972) and PL 92-217, the Clean Water Act (1977). Thus, while the water quality that would be associated with a proposed reservoir under today's circumstances would be closely scrutinized and evaluated, this has not always been the case.

In 1968, evaluations were underway related to the impending construction of West Point Lake on the Chattahoochee River downstream of Atlanta, Georgia. Even though the acute environmental sensitivity currently existing was not yet manifested, at least not on a mandatory basis, there were concerns about the quality of release waters, particularly in regard to dissolved oxygen (D.O.) concentrations in the release waters. In an attempt to address these concerns, a rather bold experimentation was undertaken in the form of a demonstration project at Lake Allatoona to ascertain the viability of a diffused air pump destratification system to improve the D.O. in release waters. This paper revisits this early experimental approach and discusses the design and installation of the system, as well as its operational characteristics over the past 19 years. It should be noted that the system was not designed to be a permanent operational feature, but rather to operate for a limited time period, three to four years, while its effectiveness as a water quality management tool was evaluated. Therefore, it is remarkable that the original installation remained operational, albeit at gradually reduced efficiency, *through 1986.*

PRIOR REPORTS

A comprehensive report (USCE, 1973) on the destratification test was issued by the Savannah District. This report contains extensive water quality data collected from 1968 through 1970, as well as other information related to the effectiveness of the system. The system was operated in 1968 and 1969, but not in 1970, so water quality data could be collected for comparative purposes. As a matter of explanation as to Savannah District's involvement in projects within the Civil Works boundaries of the Mobile District (both Allatoona and West Point are Mobile District projects), the South Atlantic Division assigned the design and construction of the West Point project to Savannah District in order to balance workloads. Savannah District subsequently requested Mobile District to design, construct, and install the prototype destratification system at Allatoona Lake and to conduct water quality investigations on its effects. Savannah District then prepared the above referenced report with input from the Mobile District. This cooperative venture bears evidence to the adage "The Corps Family."

CHARACTERISTICS OF LAKE ALLATOONA

Allatoona Lake was selected for the experimental test because of its geographical proximity to West Point and the general similarity in physical and operational characteristics. While

Allatoona is on the Etowah River in the upper Mobile River basin and West Point is on the Chattahoochee River in the Apalachicola River basin, both projects operate as peaking hydropower plants, and are located within 161 kilometers (km) (100 miles) of each other. Since the prototype system was not installed at West Point, no further discussion of that project is presented and full attention will be devoted to the Lake Allatoona experience.

The Allatoona project operates for the purposes of flood control, regulation of streamflow for navigation, development of hydropower, and recreation. Construction was completed in 1955. There is a winter drawdown for flood control purposes, but the operational plan calls for attempting to maintain a constant pool from 1 May through 1 October, at elevation 840 MSL. Obviously the water surface elevation fluctuates due to hydrological variations but the 840 elevation serves as a guide for reservoir operations.

Table 1 contains vital statistics of Lake Allatoona with emphasis on characteristics related to reservoir stratification. The stratification ratio (Huber and Harleman, 1968) represents a "rule of thumb" dimensionless indicator of the type stratification a reservoir can be expected to exhibit. Lake Allatoona exhibits a relatively strong classical three-layered stratification system as generally described by Ford et al. (1986).

Table 1

Lake Allatoona Characteristics

Characteristic	Amount	Units
Drainage Area	2,949(1,110)	Sq.km(sq.mi)
Maximum Depth[1/]	45.7 (150)	m (feet)
Surface Area[1/]	4,900(11,860)	hectares (acres)
Storage Capacity[1/]	453.3×10^6 (367,500)	cu. m. (acre-ft)
Average Annual Discharge	46.84(1,654)	cu.m/s (cfs)
Stratification Ratio[2/]	3.3	Dimensionless

[1/]Normal summer pool elevation 286 m (840 feet) above mean sea level (MSL)

[2/]The Stratification Ratio (Huber and Harleman, 1968) is computed by dividing the average annual discharge, integrated over the entire year and expressed in acre-feet by the storage volume in the same units.

POWERHOUSE

The powerhouse contains two 36,000 kilowatt main units and a 2,000 kilowatt service unit. The main unit penstocks are 6.1 m (20 feet) in diameter with the centerline of these intakes 27.4 m (90 feet) below the normal summer pool, while the service unit is 1.67 m (5.5 feet) in diameter with its centerline 16.7 m (55 feet) below the normal summer pool. A minimum discharge of 5.66 cum/s (200 cfs) via the service unit exists at all times that the main units are not operated. The main units, with a discharge of approximately 113.3 cum/s (4,000 cfs) each, discharge for generally short durations from 2 to 8 hours to meet peaking electric power demands within a power grid system. As discussed in more detail later, the physical characteristics of the reservoir, powerplant configurations, and operational patterns all contribute to the water quality, especially the D.O. concentration of the release waters.

STATE OF THE ART

The concept of reservoir destratification or mixing was not unique in the 1963 time frame; however, the utilization of a diffused air pump in a large reservoir, such as Allatoona, was a new venture. Dr. James M. Symons of the National Environmental Research Center served as a consultant providing guidance on the overall size and configuration of the system. Previous work by Dr. Symons (Symons et al., 1967) illustrates the "State of the Art" at the time the design for Lake Allatoona was initiated. Destratification efforts on eight lakes are described. The largest lake had a storage volume of 41,617,953 cu.m. (33,740 acre-feet) or less than 10 percent of the volume of Lake Allatoona. Therefore, the Lake Allatoona experiment was an order of magnitude larger than any known previous efforts.

SYSTEM DESIGN

Several factors influenced the design of the system, including cost, capability of performing the installation without elaborate equipment, availability of materials to meet time constraints, and uncertainties related to the manner in which the compressed air system would perform under various operating conditions. Five 60 horsepower, electrically powered, rotary type compressors were selected. Each compressor had a minimum capacity of 250 cfm free air at 100 psig. Once the determination was made on the number and size of compressors to be utilized, the design focused on the method to distribute the compressed air into the lake in the most effective manner. As a first step, diffusers were evaluated. The selected diffuser was about 0.61 m (2 feet) in length with an outside diameter of 7.62 centimeters (cm) (3 inches). The diffusers basically consisted of a corrugated metal core covered by 0.16 cm (1/16-inch) diameter saran cord which was wrapped around the metal core to assure the air exited the diffuser in relatively small bubbles. Because of the capacity of the individual diffusers, a total of 200 was required. Five arrays with 40 diffusers each were designed to spread the air diffusion in a controlled fashion (Figure 1).

Figure 2 shows the elevation and plan view of the arrays. The arrays were configured to suspend the diffusers 3.05 m (10 feet) above the bottom in order to keep the diffusers out of the bottom sediments, yet keep them relatively close to the bottom for greater pumping efficiency. A system consisting of an anchor and a submerged buoy was incorporated below and above the crossarms, respectively. A network of cables was utilized to hold the diffusers level. The cables served the dual purposes of fixing the elevation of the diffusers and in absorbing the stresses associated with lifting the arrays for installation, so undue forces would not be exerted on the diffuser crossarms. The combination buoy-anchor system had to function with the diffuser arms full of water and with the added buoyancy from the air in the arms when the system was in operation. Another interesting aspect was the manner in which styrofoam performed at water depths of 42.7 m (140 feet). Conventional styrofoam compressed about 20 to 25 percent of its volume at atmospheric pressure when lowered to this depth. Obviously this greatly affected its buoyancy and a high density styrofoam especially formulated for the Navy to be utilized in submarines was selected because of its ability to maintain much of its original volume, and thus buoyancy, even at significant depths.

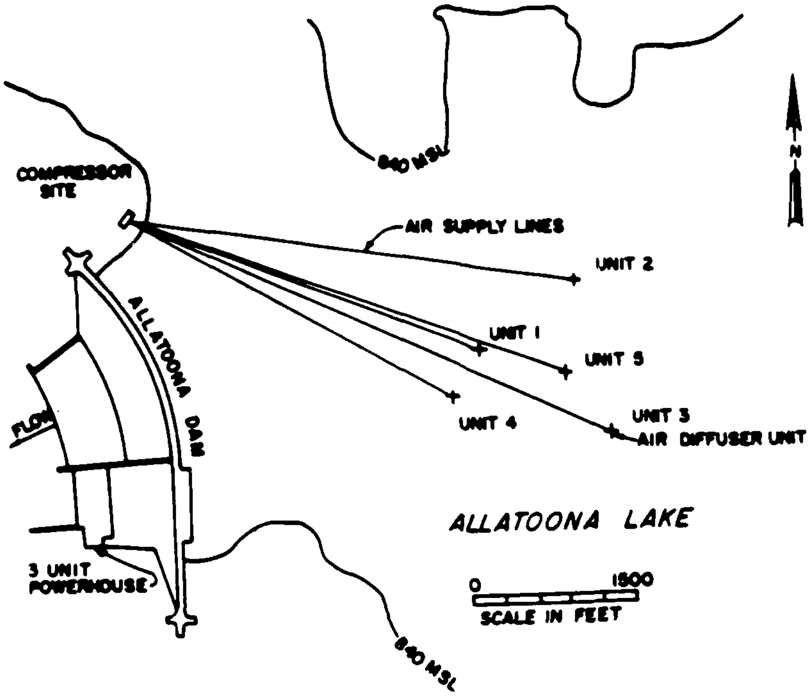


FIGURE 1. Location of Air Diffuser Units

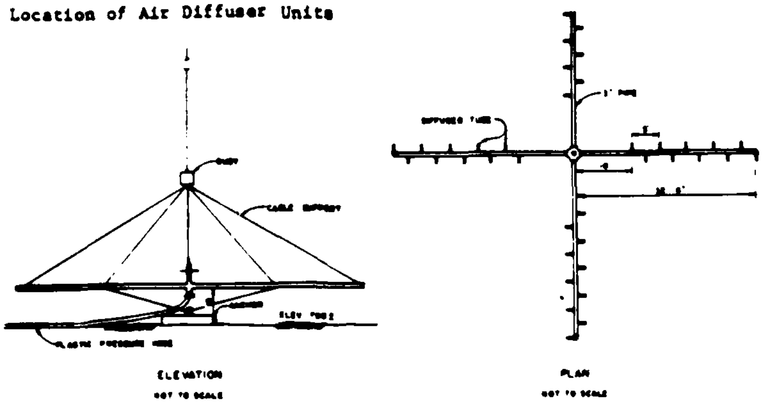


FIGURE 2. Air Diffuser Unit Elevation.

Air Diffuser Unit Plan.

McClure

AIR SUPPLY

The bank of compressors to supply the air was placed in a fenced area completely exposed to the ambient weather conditions. A house or shed was not considered necessary since the system was only to be operated for a couple of years. A manifold was designed so any number of compressors could be operated to serve the five arrays in an individual or collective fashion. Pressure gauges and flowmeters were incorporated to facilitate a balanced distribution of air to the individual arrays. The manifold played an important role during periods when one or more compressors were out of operation for maintenance or other purposes by evenly distributing the available air to the five arrays. Also, valves were included so any of the arrays could be shut off if a malfunction developed.

INSTALLATION

Installation of the arrays and the 5.1 cm (two-inch) galvanized feeder pipes created a special set of challenges to be overcome. The furthermost array was about 0.54 km (one-third of a mile) from the compressors and the feeder pipe had to essentially conform to the contours of the steep shoreline and the irregular lake bottom. The supply lines were initially strung out along the lake surface with buoys attached to keep it on the surface. The far end of the line (away from the compressors) was secured to a somewhat flexible plastic tube connected to the center of the array, which had previously been lowered into position on the lake bottom. (The arrays were assembled on top of the dam; lowered over the upstream face and into the water. A motorized raft with a power winch was connected to the cable above the submerged buoy and transported the array suspended below the water surface to a designated location in the lake. The winch then lowered the the array assembly to the bottom while the plastic tubing was manually fed out as the array descended, retaining the unconnected end to be attached to the supply line.) An innovative swivel joint was incorporated incrementally along the supply line to facilitate the lowering process and to allow the supply line to better conform to the irregularities of the bottom. The swivel joint consisted of two 90 degree elbows turned to face each other and connected by a short nipple. Thus, the joint was somewhat rigid, but nevertheless provided enough flexibility to relieve excessive stress from the pipe joints. A union joint was incorporated to facilitate connection of pipe sections. An earlier attempt to install the supply line with only standard couplings used to connect the 6.1 m (20-foot) sections of galvanized pipe failed because the stresses encountered in lowering the pipe to the bottom pulled the pipe out of the couplings.

The installation crew (all Corps employees) consisted of a fabrication team that preassembled the arrays and pipe sections on the dam and a team that accomplished the actual installation. The installation team utilized two 5.5 m (18-foot) work boats and a motorized raft and included seven or eight people. The installation process was evolutionary based on trials and errors and experimentation. Once the process was fully developed, it went fairly smoothly. Some interesting problems were encountered such as having almost 0.54 km (one-third of a mile) of supply line stretched out on the lake surface, suspended by 200 liter (55 gallon) drums serving as buoys, and a strong wind coming up. The result was a supply line that resembled a pretzel which had several sheared joints. Another problem encountered was a kinked plastic tube leading to an array which required retrieval of the array and replacement of the plastic pipe. Considering the equipment that was available, the inexperience of the installation crew (no one had ever been involved in this type effort), and the physical setting that existed, it is amazing that the system was successfully installed. The Corps motto "Essayons" was certainly apropos to this effort. The cost of the system installed was $83,633 (USCE, 1973).

SYSTEM OPERATION

The system has been in operation since 1968 continuously with the exception of 1970 and 1974. The two years that the system were not operated serve very well as a base so the effectiveness of the system can be measured. Table 2 presents a summary of the operational history. Since the installation was to be temporary, commercial power was utilized even though it was more expensive. Special arrangements needed at the powerhouse to provide government power could not be accomplished to meet the operation startup in 1968. In 1977, the system was switched to government power. The switch resulted in a reduction in cost of power from $200.00 per day to $21.50 per day. An undersized circuit breaker would only accommodate four compressors during 1968. A replacement prior to the 1969 season permitted full operation of the system. The compressors operated continuously, as indicated in Table 2, except for brief periods when single compressors were taken off line for maintenance. Major repairs were generally accomplished during the off season. As noted, one compressor had to be removed from operation during 1981; a second compressor went out after 1985. A third compressor became unserviceable in September 1986. It is not certain whether it is permanently out of commission.

WATER QUALITY IMPROVEMENT

The initial evaluation of the system (USCE, 1973) led to the conclusion that the D.O. concentrations in the release water was definitely improved by the operation of the system. The D.O. was increased in the reservoir at depths from 9.1 to 24.4 m (30 to 80 feet), with some increase below 24.4 m (80 feet). The fall overturn was also observed to occur a month to six weeks earlier than normal. The increase in D.O. in the reservoir manifested itself in higher D.O. concentrations in the release waters. In 1969, with the system in full operation, the D.O. content of peak flows exceeded 4 mg/l for the entire summer and fall periods. Low flow concentrations exceeded 4 mg/l, except for one week in late August. While the report (USCE, 1973) provides this summary, observed values at the water quality monitor in Appendix B of the report indicates values below 4.0 mg/l did exist for short periods during the hours of darkness. There was a temperature increase of 2 to 4 degrees centigrade associated with increase in D.O. for the months of June through August.

A new continuous water quality monitor was installed about 1 km (0.6 miles) downstream of the dam in the early 1970's. Although the record is not complete due to various operational problems, the monitor data does allow a detailed analysis of what happens to the D.O. regimen in the release water throughout an entire day, as well as providing data for statistical analysis for longer

Table 2

Record of Operation[1]

Year	Began	Ended	Remarks
1968	5/9	9/30	Initial year of Operation. Power from commercial source. Operated at 80 percent capacity. 693,760 KWH[2]
1969	3/17	9/30	Full operation 1,091,840 KWH[2]
1970			Did no operation for comparison purposes.
1971	7/22	10/4	
1972	4/6	9/27	
1973	5/22	9/24	
1974			Did not operate.
1975	4/23	10/3	
1976	4/5	9/3	
1977	4/11	9/29	Switched to Government Power.
1978	4/3	9/29	
1979	4/2	10/1	
1980	5/4	9/29	
1981	4/7	9/30	One compressor permanently removed from operation 9/15/81.
1982	4/5	9/17	
1983	4/4	8/5	
1984	4/1	9/30	
1985	4/1	9/30	
1986	4/1	9/30	One array out of operation. Three compressors operating most of year. Two operational after 9/17/86.

[1]System consisted of 5 sixty horsepower compressors. Each compressor has a minimum capacity of 250 cubic feet per second of 100 psig.

[2]Kilowatt hours (KWH) electrical energy presented for comparison of energy input. Other years of operation are proportional to periods of operation and number of compressors operating.

periods. Since the destratification system did not operate during 1974, a very convenient base for comparison exists. Recognizing there would be natural variations from year to year due to hydrological, climatological, and operational differences, a comparison of 1973 to 1974 is nevertheless illustrative of the general trend of improvement in D.O. produced by the destratification system. Table 3 presents summary data from 1973 and 1974. Both monthly minimum and average monthly D.O. values are higher with the system in operation. However, there are a number of times that the D.O. is less than 4 mg/l which was not the case stated for 1969, as discussed above. It is not clear whether this is due to actual quality differences or better instrumentation.

Table 3

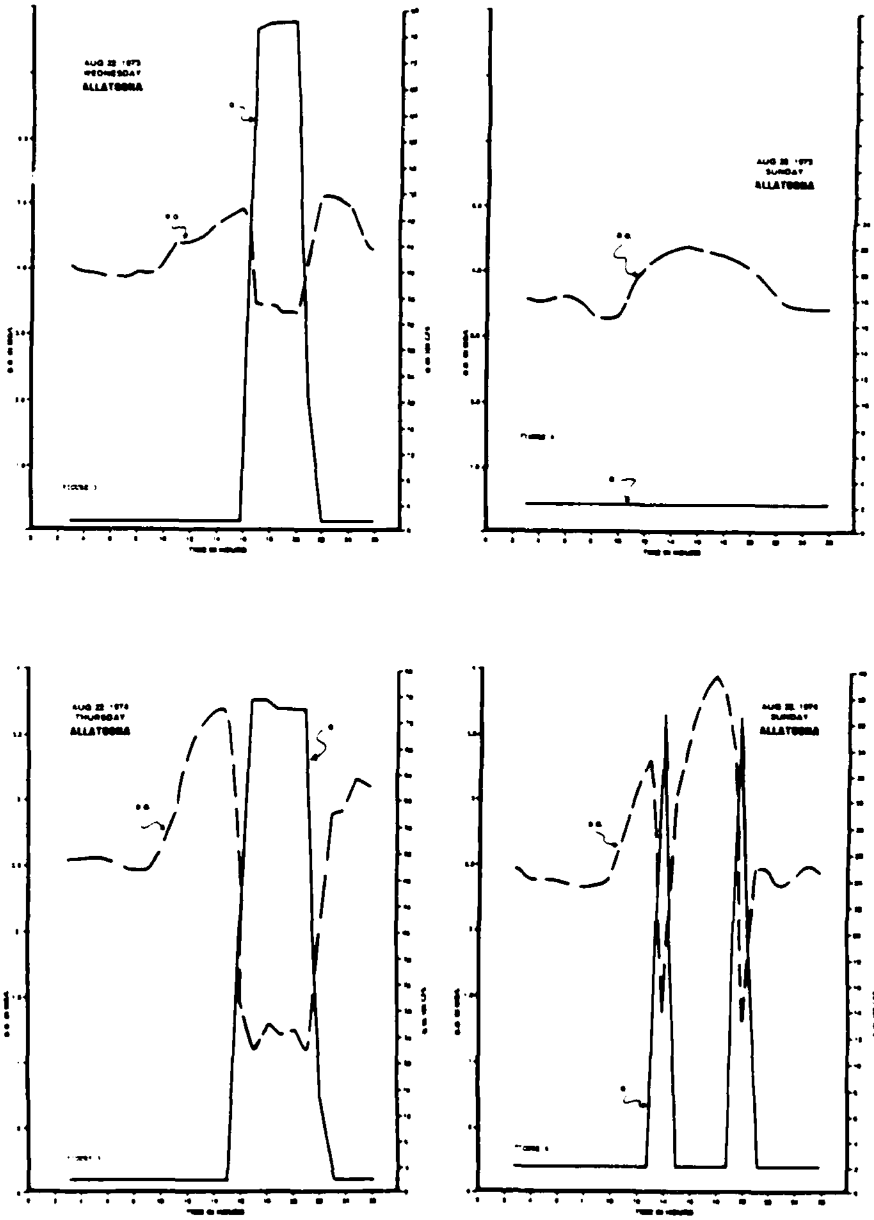
Comparison of Dissolved Oxygen (DO) Values mg/l In Release Waters1/

	Average D. O.			Minimum D. O.		
	Jul	Aug	Sep	Jul	Aug	Sep
1973	3.9	4.2	3.4	2.3	2.2	1.4
1974 2/	3.6	2.2	3.0	0.8	0.4	0.9

1/ Data from water quality monitor - miles downstream of Allatoona Dam.

2/ System did not operate in 1974.

Figures 3-6 demonstrate the D.O. regimens for selected days in late August of 1973 and 1974 associated with typical operational scenarios. The conditions depicted generally represent worst case scenarios since it is late in the season and the lake is fully stratified. Figure 3 shows how the D.O. responds to the operation of the large turbines (both operating) because the zone of withdrawal is from lower in the lake where the D.O. levels are depressed. The D.O. is increasing from an overnight low of about 4 mg/l and is



approaching 5 mg/l, apparently due to photosynthetic oxygenation, when the peaking plant comes on line and the discharge increases dramatically from 5.66 to 220.9 cum/s (200 to 7,800 cfs). Correspondingly the D.O. level drops to about 3.4 mg/l. When the period of peaking generation ends the D.O. increases then gradually decreases due to respiration during the evening hours. This operational pattern is contrasted to a weekend day in Figure 4, where no peak discharge is made. The D.O.'s associated with the 5.66 cum/s (200 cfs) release in Figure 4 is somewhat lower than in Figure 3, but it appears that diurnal variation of approximately 1 mg/l is consistent.

Figures 5 and 6 depict similar time frames and situations in 1974, when the destratification system was not operated. Not only was the D.O. associated with the 5.66 cum/S (200 cfs) discharge lower when compared to 1973, but the depression in D.O. reached about 1.2 mg/l contrasted to a minimum of 3.4 mg/l in 1973. Figure 6 represents a situation when only one of the large generators is put on line for two separate two-hour periods. Again, the response of the D.O. is graphically displayed.

Another way to compare the impact of the destratification is the pounds per day of D.O. in the release water. The computed values are 15,150; 2,025; 7,954, and 2,864 kilograms (33,330; 4,455; 17,500; and 6,300 pounds) per day for Figures 3 through 6, respectively. On this basis the pounds of D.O. available in Figure 5, for waste assimilation and for use by aquatic organisms, was only 52 percent of that in Figure 3. The average discharge in Figure 3 was 46 cum/s (1,625 cfs), which was 1.25 times more than in Figure 5, so while part of the difference can be explained by the increased discharge, the destratification system still made a significant contribution to the D.O. regimen.

PROPOSED REPLACEMENT

In 1984, plans and specifications were issued to replace and upgrade the Allatoona destratification system. The design was essentially the same as the original with a few improvements such as:

a. A building to house the compressors.

b. Three 100 horsepower compressors to deliver 14.2 cum/minute (500 SCFM) each at 100 psig (an increase in total air of 250 SCFM).

c. Upgraded diffuser tubes.

d. Revised submerged float.

e. Upgrading all galvanized pipe in the arrays and supply lines to stainless steel.

Bids were opened with a low bid of $413,378 and a contract was awarded in 1984. In January 1985, the successful contractor submitted a Value Engineering proposal to substitute a "Continuous Laminar Flow Inversion/Oxygen" system for the designed concept. The Mobile District requested the assistance of Waterways Experiment Station (WES) in evaluating the proposal. This evaluation produced two significant conclusions. First, (WES, 1985) the contractor proposal was found to be questionable or lacking from a number of perspectives and rejection was recommended. Secondly, it was concluded that the destratification design could be improved in several areas, such as diffuser location, type diffuser, lowering diffuser loading rates, and spacing, as well as the configuration of diffusers. Based on these findings the replacement contract was terminated and consideration was given to a redesign of the destratification of the system.

CONCLUSIONS

The Allatoona destratification experiment is considered to be a success even though it did not destratify the lake and most likely its effectiveness could be improved upon. A system designed to perform for two to three years has operated for 19 years. While the State of Georgia standards for D.O. of 5 mg/l are not always met, the D.O. concentration in the release waters has been increased substantially. As of this date, although there have been discussions with WES on how the system could be improved, redesign has not been initiated due to higher priority efforts. However, the existing system was operated throughout 1986.

It is possible that the system design could be improved with a resulting increase in D.O. in the release waters. It is not known if the anticipated increase would be sufficient to produce a D.O. regimen that would meet State Standards at all times. Other measures, such as turbine venting, could also be incorporated if determined to be necessary, engineeringly feasible, and cost effective.

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PNEUMATIC DIFFUSERS

P. L. Johnson, P.E.

Hydraulic Engineer, Bureau of Reclamation, Denver, Colorado

ABSTRACT

Pneumatic diffusers are devices which aerate and destratify reservoirs through the use of free air bubble plumes. They are widely used to enhance the quality of reservoir water and reservoir releases. Experience indicates that pneumatic diffusers offer a proven and cost effective treatment technique. They function by upwelling oxygen depleted hypolimnion water toward the surface where it mixes with oxygen rich metalimnion and epilimnion water. Use of pneumatic diffusers results in varying degrees of destratification depending on the design.

This paper is a limited overview of pneumatic diffusers. The paper includes discussion of basic theory, possible resulting reservoir impacts, design, and documented field experience.

INTRODUCTION

Numerous devices and techniques are available to either treat or rectify the problems associated with lake stratification and hypolimnion oxygen depletion. Options include operational approaches such as use of deep level outlets to minimize hypolimnion residence times, reaeration of releases through use of draft tube aeration, turbulent stilling basins, or use of release structures that would actively aerate the flow, use of selective withdrawal to prevent the release of anaerobic water, aeration of tailrace flows through use of structures such as rock or baffled weirs, or aeration of the reservoir. There are numerous options available for in-reservoir aeration. Options include hypolimnion aeration devices and diffused molecular oxygen which are expensive but which do not disturb the reservoir temperature stratification and destratification dependent devices including hydraulic guns, mechanical pumping, and pneumatic diffusers.

The appropriate device or technique for use at any site is a function of the specific site with its biological, chemical, and physical characteristics and of the objectives of the reaeration treatment. The influence of the device on both temperature and dissolved oxygen should be considered.

The pneumatic diffuser is the subject of this report. Pneumatic or compressed air diffusers release air bubbles at depth within a reservoir. They mix and aerate the reservoir through the influence of the rising bubbles as means of heat and momentum interaction between the bubble plume and the surrounding oxygen depleted hypolimnion. The hypolimnion water is upwelled toward the surface where it mixes with oxygen rich metalimnion and epilimnion water. The mixed water sinks to an intermediate level dictated by its temperature. Stratified circulation cells that extend over large areas of the reservoir result. Pneumatic diffusers can effectively and economically aerate hypolimnion water. In many cases the number and extent of surface algal blooms are reduced. Use of the device does yield at least partial destratification with surface cooling and hypolimnion warming.

The impact of a diffuser on a reservoir is a function of the specific reservoir and the specific diffuser with its bubble plume. A diffuser should be designed to fit the needs of a reservoir with its specific temperature and density structure and with its oxygen demand. To evaluate reaeration needs, reservoir parameters including reservoir volume, reservoir geometric configuration, temperature and density stratification, oxygen demand, inflows, and outflows must be considered. In addition, the performance of a diffuser is a function of diffuser length, diffuser configuration, diffuser position within the reservoir, diffuser submergence, the size and spacing between diffuser orifices, and the air discharge per orifice.

RESERVOIR PARAMETERS

To define the initial reservoir conditions from which modification would be made and to define the extent of the desired modifications, the physical and chemical characteristics of the reservoir should be understood.

Reservoir Volume and Geometry

Reservoir volume and volume distribution as a function of depth are parameters required to define both the stability of the reservoir and the total oxygen demand of the reservoir. Stability is a measure of the density stratification. It is the energy required, with 100 percent efficient mixing, to fully mix the reservoir. Stability is defined as the work required to move the total reservoir mass from the elevation of the center of gravity to the elevation of the centroid of the reservoir. Destratification impacts can be evaluated as a reduction of stability. To find the elevations of both the center of gravity and the centroid, the volume-depth distribution must be known. In addition, to find the elevation of the center of gravity, the density distribution with depth and, thus, the temperature distribution with depth, must also be known.

The reservoir volume to depth relationship is also used along with observed or predicted D.O. profiles to determine the total D.O. within the reservoir. To do this, the product of the D.O. concentrations and reservoir volume are integrated over the reservoir depth. Comparison of total D.O. values through the stratified season yields evaluation of total oxygen demand within the unmodified reservoir. If this demand is then compared to a similar D.O. decline rate for the modified or aerated reservoir (which reflects minimum acceptable D.O. levels), than a required diffuser aeration rate is found. This aeration rate is used in conjunction with diffuser reaeration efficiencies to size the diffuser system.

Reservoir Density Stratification

Mixing action due to a pneumatic diffuser results from shear and drag energy transfer from the rising bubble curtain to the surrounding water. This yields an upwelling of the deep, dense, low oxygen water. This water rises into warmer, less dense water where gravitational effects tend to pull the upwelled water downward. The upwelled water is therefore influenced by momentum established by the bubble plume and by the negative buoyancy which results due to temperature differences. If the bubble plume is concentrated containing large bubbles, or if the temperature and density stratification is weak, the bubble curtain may dominate over the negative buoyancy and the deep water will be upwelled to the surface. However, with a dispersed curtain made up of small bubbles with reduced momentum, or with a strong density stratification, velocities established in the upwelled water may not be sufficient to completely overcome the negative buoyancy. Under these conditions, the upwelled water will detach from the bubble curtain and sink back to depth without reaching the surface. Reaeration is thus influenced in that strong upwelling to the surface yields fairly localized but relatively intense mixing between the upwelled hypolimnion water and the epilimnion and thermocline water (the main source of hypolimnion reaeration). Also, if the hypolimnion water is supersaturated with any gases (nitrogen, hydrogen sulfide), the upwelled water would be more readily stripped of the gas under these conditions. Weaker upwelling which may not reach the surface could result in mixing with only the thermocline and lower epilimnion water. This mixing would be at a reduced intensity level and would result in reduced localized aeration and degasing. Weaker upwelling, however, results from the use of a more dispersed bubble plume which also results in the mixing occurring over a larger area. The findings of a study by Johnson (1980) indicates that the widespread plume yields substantially more oxygenation and destratification per unit of energy consumed. Thus, although localized destratification and aeration are reduced, total destratification and aeration are increased.

In addition to differences in efficiencies, differences in mixing patterns result due to variations in bubble plume density and reservoir stratification. This is shown on figure 1 where typical midsummer temperature and D.O. profiles are shown for a reservoir with no reaeration, for the same reservoir with point source diffuser operation (concentrated bubble plume), and for the reservoir with line diffuser operation (dispersed bubble plume). The point source diffuser caused breakdown of the thermocline with warming of the hypolimnion resulting in nearly linear temperature and D.O. profiles from the diffuser to the surface. This indicates that substantial mixing is occurring throughout the water column, although the mixing is not sufficient to create a homogeneous water body. On the other hand, the line diffuser, although still showing substantial mixing influence, did not break down the thermocline as much. This pattern was observed over a wide range of line diffuser airflow rates. Note that the mixed hypolimnion is fairly homogeneous and that a distinct thermocline and epilimnion exist. This indicates that the upwelled water detaches from the bubble curtain in the thermocline and that the diffuser is mixing the hypolimnion well. A second thermocline and oxycline is developed at the diffuser elevation. Below this is undisturbed hypolimnion.

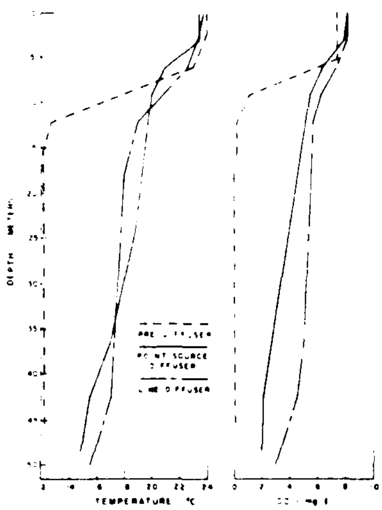

Figure 1. - Reservoir profiles.

Reservoir Flowthrough

Flowthrough represents a source of mixing which in some cases may be so substantial that reservoir stratification will not form. A rule of thumb indicates that if the total annual flowthrough is less than 10 times the total reservoir storage capacity, the reservoir will stratify, and if the total annual flowthrough is greater than 20 times the total storage capacity, the reservoir will be mixed. Of course, this is quite rough and does not define intermediate conditions. However, it should be noted that even if stratification does occur, substantial flowthrough will weaken the stratification and likely increase hypolimnion D.O. levels.

Stratified inflows or outflows may result in either freshening of oxygen-depleted water in the reservoir or removal of oxygen-depleted water from the reservoir. Stratified inflows will settle in the reservoir at the elevation where the reservoir water has the same density. If the inflow, which may be high in D.O., settles in an oxygen-depleted zone, then aeration or freshening of that zone could result. Likewise, through the use of selective withdrawal, hypolimnion water may be constantly removed from the reservoir and released to the downstream channel.

In some cases where flowthroughs are adequate, this inflow-outflow mechanism may be used to maintain satisfactory hypolimnion D.O. levels with no additional reaeration.

The influence of stratified inflows and outflows on reservoir temperature stratification and thus reservoir stability can be evaluated through the use of any of numerous mathematical thermal and hydrodynamic simulation models. In recent years, efforts have been made to extend these models to obtain prediction of D.O. profiles within the reservoir. Both the temperature and D.O. prediction models require substantial reservoir and inflow physical and chemical data as well as atmospheric data. Their predictive accuracy is limited, but certainly sufficient to describe likely temperature and D.O. development patterns.

DIFFUSER AND BUBBLE PLUME PARAMETERS

The influence of a pneumatic diffuser on a reservoir is strongly controlled by the size, physical nature, and positioning of the bubble plume. Numerous parameters influence the bubble plume. These parameters should be understood if an optimum design is to be obtained.

Submergence

The depth of diffuser submergence controls the vertical height of the bubble plume and, therefore, controls the path length of interaction between the bubble plume and the water body. Work done by Straub (1959), Kobus (1968), and Bulson (1961) disagrees on the exact relationship but indicates that the quantity of water upwelled is a function of unit air discharge rate (air discharge per unit length of diffuser) and submergence. These parameters influence both the cross-sectional area of the plume and the upward velocities within the plume. Cederwall and Ditmars (1970), in their analysis of the Kobus data, find the plume half-width equal to:

$$b = \frac{\sqrt{2}\; g q_0 \; H_0}{\sqrt{\pi}\;(1 + \lambda^2)\; U_b^3} \; \frac{W^2}{V} \tag{1}$$

where q_0 is the unit airflow rate at atmospheric pressure, H_0 is the barometric head, λ is the plume spreading ratio (Kobus found this to be practically a constant over the airflow range he studied; a reasonable assumed value is 0.2), U_b is the bubble slip velocity (which can be considered equal to the terminal rise velocity of a single bubble), and W^2/V is a parameter that is evaluated using figure 2. To use figure 2, the parameters Z and G must be evaluated:

$$Z = \frac{H_0 + H - X}{H_0 + H} \tag{2}$$

and

$$G = \frac{\sqrt{2}\;(H_0 + H) \;\; \alpha\; (1 + \gamma^2) \;\; U_b^3}{g \; q_0 \;\; H_0} \tag{3}$$

where H is the submergence, X is the vertical distance from the diffuser to the station of interest, and α is a coefficient of entrainment that is a function of unit airflow rate as indicated on figure 3.

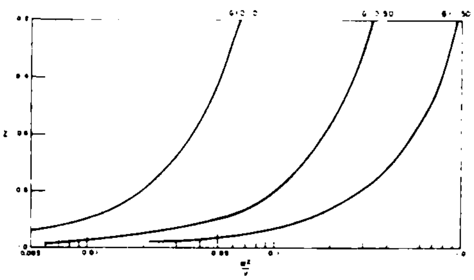

Figure 2. - W^2/V evaluation.

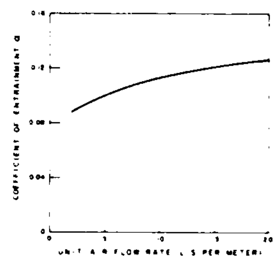

Figure 3. - Coefficient of entrainment.

Field observations indicate that the Cederwall and Ditmars equations give a reasonable prediction of plume half-width.

With a known plume cross-sectional area, in particular at the top of the hypolimnion, and with a known average velocity across the plume section, upwelled discharges and, in particular, upwelled discharges from the hypolimnion can be computed. The velocity distribution in upwelled plumes is a function of bubble density and size. For smaller bubbles less than 15 mm in diameter it appears that a Gaussian velocity distribution is representative. Cederwall and Ditmars give the centerline upwelled velocity (U_m) to be:

$$U_m = (1 + \lambda^2)^{1/2} \;\; U_b \, \frac{V}{W} \tag{4}$$

where V/W is a parameter that is evaluated using the parameters Z and G in conjunction with figure 4.

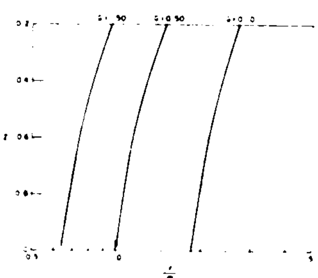

Figure 4. - V/W evaluation.

They note that for a line diffuser in a homogeneous fluid, the velocity distribution through the rising plume may be approximated by:

$$U = U_m e^{-Y^2/b^2} \qquad (5)$$

where Y is the lateral coordinate of the local plume velocity U. An integration of this velocity distribution over the plume width yields the upwelled unit discharge:

$$q = \sqrt{\pi}\, U_m b \qquad (6)$$

Depending on reservoir density stratification strength, there may be small or significant reductions from predicted upwelled discharge. The presented analysis may, however, be used to obtain an approximate evaluation of the effect of submergence and guide design modifications.

To maximize diffuser impact, the diffuser should be located as deep within the reservoir as possible. Therefore, depth of submergence is largely a function of the reservoir depth. At the same time, the diffuser must be kept a sufficient distance above the bottom to prevent mixing of bottom sediment. Experience indicates that line diffusers have an influence to a depth approximately 5 meters below the diffuser. Five meters thus appears to be an adequate distance to place the diffuser above the bottom.

Diffuser Configuration

Diffuser configuration controls the shape of the resulting bubble plume. A straight-line diffuser will yield a single-plane bubble plume while a circular diffuser will yield a cylindrical bubble plume, and a single point diffuser will yield a single-column bubble plume. The shape of the plume affects the area of the plume that comes into effective shear and gas transfer interaction with the main body of the reservoir. A single-plane bubble plume, if centrally located in the reservoir, will have effective gas transfer and shear on both of its sides. This offers an optimum potential for interaction with the reservoir. The literature contains limited information on the influence of bubble plume shape. Cederwall and Ditmars (1970) analyzed both line and point diffusers. Their analysis, which is based on a homogeneous reservoir, when applied over airflow discharge ranges used at field sites indicates that for equal total airflow rates, the line diffuser will yield between 2.5 and 3.5 times more upwelled hypolimnion water at the thermocline than a point source. No information was found that compared straight-line diffusers to line diffusers of other configurations.

In addition, the diffuser should be located such that both sides of the bubble plume have access to the main reservoir body. The diffuser should not be placed where the bubble plume would isolate a small portion (for example, a bay) of the reservoir from the main body. In such a location, one side of the bubble plume might have reduced influence which could result in a decrease of total diffuser efficiency.

Bubble Size and Distribution

As noted, bubble size and distribution will substantially influence the dynamic characteristics of the bubble plume. This in turn will affect diffuser efficiency and resulting reservoir impact. Bubble size and distribution are a function of the size of the diffuser orifices, the airflow rates through the orifices, and the spacing between the orifices. Bubble size and distribution affect the water-air interfacial area, the bubble rise velocities in the plume and, in general, the gas and energy transfer from the plume to the reservoir. Smaller bubbles have higher surface area to volume ratios and develop proportionally more drag than larger bubbles. Thus, they have better gas and energy transfer characteristics. If these bubbles are widely dispersed, they will have increased potential for contact with the reservoir body and their effectiveness will be further improved. Concentrated bubble plumes develop higher velocity center cores which have reduced interaction with the reservoir body. Thus, the relative energy and gas transfer potential of the plume are reduced.

Similar research studies have been conducted by Neilson (1972), Holroyd and Parker (1949), and Camp (1963) which have evaluated the aeration potential of pneumatic aerators. These studies were conducted in small volume laboratory test facilities. The tests were run with homogeneous oxygen-depleted water. A limited range of bubble sizes were evaluated and very low density bubble plumes were used. These tests evaluated as separate factors the aeration across the reservoir water surface, the aeration from the rising bubbles, and the aeration that results due to bubble bursting at the reservoir surface. These studies indicate that for typical pneumatic aerator sites with 5 mm to 15 mm bubble diameters that gas transfer across the reservoir water surface comprises more than 95 percent of the total aeration gas transfer. These studies indicate that direct gas transfer from the rising bubbles is small and that bubble diameters would have to be substantially reduced to make this a significant factor.

Upwelled Discharge

Bubble size and distribution also influence the velocity distribution within the plume, the rate of plume spread, the rate of surrounding reservoir water entrainment, the resulting upwelled water discharge, and thus aeration and reservoir destratification. Bubble size influences bubble shape, the drag interaction between the bubble and the water, and consequently, the rise velocity of the bubble. Larger bubbles yield higher rise velocities and reduced relative drag and energy transfer. Bulson (1961) and Wilkinson (1979) show that for large airflow rates with unit discharges greater than 15.8 std. (L/s)/m of diffuser, velocity distributions vary markedly away from Gaussian with increased centerline velocities, while with smaller bubbles and reduced air concentrations the velocity distribution is nearly Gaussian. Centerline velocities within the plume are higher than the terminal rise velocities of individual bubbles. As previously observed, velocities within the plume are also a function of submergence. Evaluation of these velocities is discussed in the submergence presentation earlier in this report. As theorized by Taylor and as experimentally verified by Straub, et al (1959), Baines and Hamilton (1959), and Evans (1955), the maximum velocity in the plume for reduced airflow rates varies with the airflow rate to the 0.33 power. Both Neilson (1972) and Wilkinson (1979) note that bubble plumes with smaller bubbles and reduced airflow rates yield higher upwelled water discharge to airflow rate ratios. Thus, they conclude that this type of plume has a more efficient upwelling action and that unit airflow rates should be kept low. From Kobus's work, Wilkinson (1979) concludes that unit airflow rates should be held below 10.0 (L/s)/m of diffuser.

Minimum Diffuser Differential Pressure

To keep the diffuser purged of water and to maintain relatively uniform airflow over the diffuser lengths, a minimum differential pressure of at least 10 kPa should be maintained across the diffuser orifices. This has been confirmed by field observations.

Bubble Size

A theoretical analysis by Van Krevelen and Hoftijzer (1950) may be used to analyze the size of bubble produced by a diffuser. They show that for small air flow rates, bubble size at or immediately above the diffuser is solely a function of orifice diameter. For larger air discharges, bubble size is found to be a function of the air discharge at the orifice to the 0.4 power. The findings of this analysis are summarized on figure 5. For bubbles smaller than 7 millimeters in diameter, this analysis was verified by experimental work by Coppock and Meiklejohn (1951). Field observations approximately confirm these findings. Differences in orifice orientation and/or flow conditions past the diffuser may cause variations.

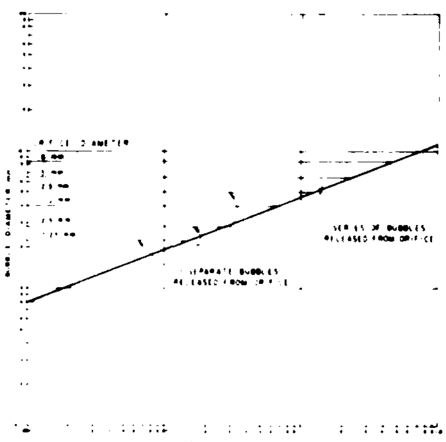

Figure 5. - Bubble diameter.

Bubble Rise Velocity

Haberman and Morton (1954) show that the terminal rise velocity of individual bubbles in unfiltered water is solely a function of bubble volume. They find that bubble shape changes with volume from spheres to ellipsoids to spherical caps. A graphical relationship is developed (fig. 6) that relates terminal velocity to equivalent bubble radius. The equivalent radius is found by converting the bubble volume, for whatever shape the bubble might be, into a sphere and determining the radius.

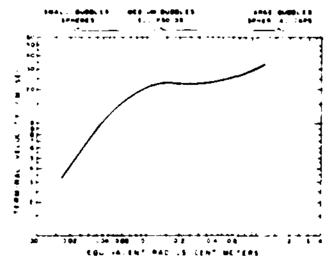

Figure 6. - Terminal velocity.

In summary, a diffuser that creates a bubble plume made up of small, well dispersed bubbles will yield increased oxygen transfer from the rising bubble, increased upwelled water per unit volume of air and thus, increased reservoir surface aeration and increased destratification. All of this results in response to the increased total bubble surface area, increased relative drag, and increased plume entrainment that occur due to the small, dispersed bubbles. Good bubble dispersion is obtained through the use of a fairly long diffuser which allows for substantial spacing between orifices. A 0.3-meter center-to-center spacing has been used successfully at several sites. Small bubbles are obtained through the use of small orifices which operate with a minimum differential pressure. One-millimeter-diameter orifices represent a minimum diameter that can be easily field drilled. A differential pressure of 10 kPa appears to be about the minimum that could be used while keeping the diffuser flushed of water and maintaining fairly uniform air dispersion. The orifices can be drilled in the diffuser sides and alternated from side to side. Limited laboratory flume studies show this arrangement to yield increased entrainment over that obtained with the orifices placed in the top of the diffuser pipe. Alternating the orifices from side to side balances the forces exerted by the air releases on the diffuser.

SAMPLE DESIGN

As an initial step, the size or extent of the problem to be treated is defined. To do this, the size of the impoundment to be treated and thus the volume of water to be treated should be determined. Likewise, the expected oxygen demand in the untreated impoundment and desired oxygen decline rates in the treated reservoir should be defined. Expected oxygen demand in the untreated impoundment may be evaluated through observation of historical data for that impoundment, through observation of the oxygen response in similar impoundments, or through the use of D.O. prediction mathematical models. It should be noted that oxygen demand observed from historical data or from similar reservoirs will vary over the short term, for example, because of the decay of algae blooms or flooding; and over the long term, for example, because of reservoir maturing or seasonal variations. A decision must be made as to whether reaeration system design should be based on typical oxygen decline rates or on some extreme value. Sizing a system based on an extreme decline rate will yield a system that is oversized for most cases and thus may have both excessive capital and operating costs. However, sizing a system based on a typical or mean decline rate will yield a system that is unable to meet all desired demands. The design D.O. reaeration rate selected generally depends on how critical the reaeration is.

It appears that historical data for the reservoir of interest may supply the best estimate of initial untreated D.O. demand. Similar reservoirs can supply a good estimate of untreated conditions. However, care should be taken to ensure sufficient similarity. The comparison impoundment should be in the same vicinity as the impoundment of interest, should experience fairly similar climatic conditions, and should be of similar depth or at least deep enough to allow similar thermocline and hypolimnion development. The comparison reservoir should experience similar inflow and release discharges. The relative influence of the flowthrough should be similar and thus the relative magnitude of the discharges versus reservoir volume and the stratified flow response of the flows in the reservoirs should be similar. This also implies the need, where multiple release structures exist, to have similar operating characteristics for the two sites. Finally, for a good comparison of D.O. response, the oxygen demand of the two reservoir hypolimnion should be similar. This generally implies that the impoundments have similar nutrient characteristics, biological productivity, and that they are biologically managed in similar ways.

As previously noted math models may also be used to predict initial or untreated reservoir states. Use of the models requires substantial data bases; the models are best applied where sufficient data exist for verification. With limited input data and with no historic profiles to help fit the model, only approximate predictions and guidance can be obtained.

After determining the untreated D.O. state of the reservoir, minimum acceptable D.O. levels should be selected. Desired uses of the water should be considered in this selection. For example, if the objective is to prevent the development of anaerobic conditions, a level of 2 mg/l might be selected. However, if the objective is to maintain a trout fishery, a minimum acceptable hypolimnion D.O. level of 5 mg/l might be selected. Noting then that the epilimnion water will tend to be saturated in D.O., and considering the degree of destratification or variation away from a traditional two-layer density profile (discussed earlier in the paper) that would result, estimated treated D.O. profiles can be obtained. These profiles are an epilimnion-hypolimnion composite with a transition between the two layers. Hypolimnion D.O. levels will decline from saturation at the start of the stratification season to minimum acceptable values just prior to fall turnover. By noting the total D.O. mass decline for the treated reservoir (as indicated by the developed profiles), and dividing by the expected stratification season length, an acceptable total D.O. mass decline rate is obtained. For example, if it is found that an acceptable total D.O. mass decline of 5×10^5 kg O_2 could occur over the stratified season and if the expected stratified season length is 200 days, an acceptable total D.O. decline rate of 2,500 kg O_2/day could be tolerated. It should be noted that if destratification accompanies the reaeration then the stratified season will be shorter than it would be in the untreated reservoir.

A similar computational process can be done for the untreated reservoir. The total D.O. mass content of the reservoir at the start of the stratified season can be computed. Likewise, a total D.O. content at the end of the stratified season or when the hypolimnion goes anaerobic can also be computed. Note that once the hypolimnion goes anaerobic, the total oxygen decline rate changes because there is no oxygen left to be depleted. Again, by taking the difference between the total D.O. mass at the start of the season and the depleted total D.O. mass, the total D.O. mass decline that occurs in the untreated reservoir is found. When this is divided by the time period, the stratified season length, or the time length to an anaerobic hypolimnion, the total oxygen decline rate in the untreated reservoir is found (for example, 3 700 kg O_2/day). The difference between total D.O. decline rates (3 700 to 2 500 kg O_2/day) represents a reaeration rate that must be supplied by the device.

It should be noted that the untreated D.O. levels in the reservoirs which were obtained either from historic data, a similar reservoir, or mathematical models include the influence of inflows and releases on reservoir D.O. This does not however consider oxygen demand of the oxygen-depleted flowthrough. In these cases, it may be required to size the reaeration system to treat both the reservoir and the flowthrough. Noting flowthrough volumes and desired D.O. levels, estimates of required additional reaeration for the flowthrough can be obtained.

With a knowledge of the required total oxygenation or reaeration rate (1 200 kg O_2/day for the example), the reaeration system may then be sized. The pneumatic diffuser efficiencies shown in figures 7 and 8 may be referred to. The efficiencies were obtained for a straight-line diffuser submerged at a depth of 46 m. Air was supplied to the diffusers with rotary screw, electric motor-driven, single-stage compressors. Energy losses through the distribution piping and diffuser system were less than 4 percent of the available energy in the air flow at the compressors. The efficiencies shown in figures 7 and 8 reflect this particular compressor and distribution system design. Compressor selection and distribution system design can significantly influence efficiencies. Thus to directly apply the efficiencies shown in figures 7 and 8 the compressor and distribution system should be similar to that mentioned above. Variations from this design can be computationally compensated for. In figures 7 and 8 the efficiency bands labeled "Diffuser System" should be used. These efficiency bands reflect uncertainty in efficiency evaluation and thus uncertainty in actual efficiency values. By referring to the efficiency data shown in figure 7, and knowing the required reaeration rate (1 200 kg O_2/day) and a selected unit air flow rate (for example 0.6 std.L/s/m of diffuser) a reaeration efficiency can be determined (3.5 kg O_2/kWh) and a required energy consumption rate obtained (343 kWh per day). For the previously described compressor and distribution system this energy consumption rate corresponds to a system supplied by a 19.5-horsepower, 37 std.L/s capacity compressor.

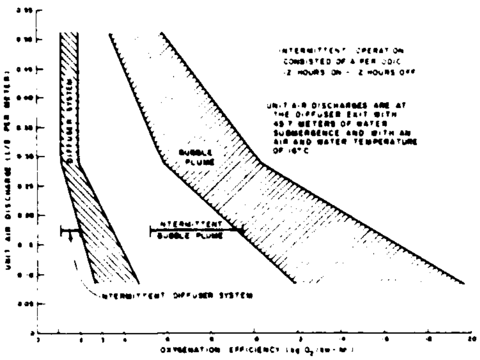

Figure 7. - Oxygenation efficiency.

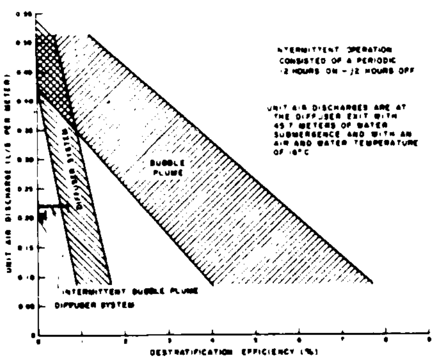

Figure 8. - Destratification efficiency.

This sizing was based on an assumed unit air flow rate of 0.6 std.L/s/m of diffuser. This is approximately the minimum unit air flow rate that could be effectively supplied by the diffuser as designed. As previously noted, findings indicated that more widely dispersed bubbles (smaller unit air flow rates) yield higher reaeration efficiencies. The 37 std.L/s air discharge with a unit air flow rate of 0.6 std.L/s/m yields a line diffuser that is 61.7-m long. As previously noted, it is recommended that this diffuser be constructed with 1.0-mm-diameter orifices placed at 0.3-m centers.

This design assumes diffuser submergence to be 46 m (the depth that the efficiency curves were developed at). If reservoir geometry dictates a diffuser submergence of for example 60 m, then diffuser length, unit air flow rate, and thus total air flow rate can be adjusted to compensate. To do this, equations 1, 2, 3, 4 and 6 and figures 2, 3, 4, and 5 can be used. If the upwelled discharge is computed at the approximate bottom of the thermocline for the 60-m submergence and compared to the corresponding upwelled discharge with 46 m of submergence, a compensation factor can be evaluated. If the bottom of the thermocline is predicted to be at 9 m depth (51 m above the diffuser) a computed upwelled unit discharge at that location in the plume would be 1.44 m^3/s. Using the same equations and figures the upwelled discharge at the 9-m depth (37-m above the diffuser) is 0.97 $m^3/s/m$ with 46 m of diffuser submergence. Thus the increased submergence would yield a 48 percent increase in upwelled hypolimnion discharge. To obtain the desired impact, the 0.6 std.L/s/m unit air discharge could be maintained and the diffuser length reduced by 33 percent. This would yield a 41.6-m long diffuser with a total required air discharge of 24.9 std.L/s.

A similar computational process could be undertaken if diffuser submergence was to be less than 46 m. For this case analysis indicates the need for either a longer diffuser or increased unit air discharges. Increased unit discharges are used at the expense of reduced reaeration and destratification efficiencies.

With the diffuser sized, numerous other design problems can be addressed. Included is sizing of the distribution and diffuser piping; selection of valves, instrumentation, compressors, compressor shelters, and system security; and design of anchors, floats, and retrieval gear. It is strongly recommended that the designer contact or visit existing installations. Much fabrication, operation, and maintenance knowledge can be obtained through experience.

Two final factors should be considered in reaeration system design. First, since destratification results from system operation, satisfactory reservoir temperatures may or may not be obtained. Typically, destratification cools the epilimnion and warms the hypolimnion. If the reservoir is intended for a temperature dependent use, such as for a cold water fishery, then a conflict may result. Sufficient reaeration to yield the desired D.O. levels may produce unacceptable temperatures. Figure 8 shows pneumatic diffuser destratification efficiencies as a function of unit air discharge. The efficiencies were obtained using the previously described diffuser design. A technique similar to the one presented in this paper for reaeration may be followed to evaluate the destratification influence. Initial untreated temperature profiles can be determined, untreated reservoir stabilities computed, destratification influence on stability evaluated (using system size determined from the reaeration computations and appropriate destratification efficiencies), and the impact on temperature profiles found. If the temperature impact is unacceptable, a treatment device that would yield less or no destratification could be selected.

A final consideration is potential nitrogen supersaturation development within the reservoir. Supersaturation may develop either from direct gas transfer from air bubbles or from the warming of the water that results with destratification. Warming lowers the saturation concentration. Thus warming can yield supersaturation even with no additional gas transfer. Fast and Hulquist (1982) show the degree of supersaturation development to be a function of diffuser design and mixing intensity. In many cases, nitrogen supersaturation development does not pose a problem. Because of submergence, dissolved gas levels at depth within the reservoirs are typically well below saturation levels. Likewise, high turbulence releases or high turbulence tailrace flows can strip the excess gas from the release water and alleviate the problem. Only where low turbulence submerged releases (such as power releases) are made does the supersaturation pose a problem. Where supersaturation is a problem, hypolimnion aeration may be required.

FIELD EXPERIENCE

Pneumatic diffusers are widely used, sometimes successfully, sometimes not. Frequently the cause of unsuccessful application is lack of understanding of basic reservoir hydrodynamics, of bubble plume hydrodynamics, and of the interaction between the two. For example, diffusers have been installed at relatively shallow reservoir sites at which the hypolimnion thickness is small. Bubble plume interaction with the hypolimnion is very limited and consequently the quantity of hypolimnion water upwelled is small. The result is metalimnion and epilimnion mixing with very little hypolimnion influence. At other sites extremely concentrated bubble plumes have been used which, because of the inefficient use of the air, have been ineffective.

There are several sites for which well documented reports on successful application are available. These reports contain not only details on the aeration system design but also details on physical, chemical, and biological reservoir impact. Some of the better references are those by Barnett (1975); Bowles, Powling, and Burns (1979), and Fast (1968, 1971).

CONCLUSIONS

Pneumatic diffusers offer a proven technique for reaeration of reservoir water. As with all reaeration options pneumatic diffusers have specific traits that may or may not make the device desirable for specific applications. Depending on the site and the reaeration objective, pneumatic diffusers can offer a cost effective, dependable treatment option. Through a process of definition of specific reaeration objectives and consideration of various parameters that influence diffuser performance, a diffuser system can be designed to meet the needs of a specific site. Pneumatic diffusers achieve efficient reaeration through reservoir destratification. This results in at least a partial vertical mixing of the reservoir which results in epilimnion cooling and hypolimnion warming. Destratification may result in reduction of surface algal blooms and development of nitrogen supersaturation in the hypolimnion. Nitrogen supersaturation may or may not pose a problem depending primarily on the type of reservoir release.

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RETRO-FITTING FOR HIGH-LEVEL RELEASES TO IMPROVE DOWNSTREAM QUALITY

Richard E. Punnett, PhD

Hydraulic Engineer, Corps of Engineers, Huntington District, WV

ABSTRACT

Historically, the downstream releases from Sutton Lake, West Virginia, experienced turbidity and temperature problems because of hypolimnetic releases. To provide for epilimnetic releases, a semi-circular conduit was retro-fitted to the face of an existing dam. Improvements in the quality of the releases have been noted. Operational guidelines, which are a product of interagency coordination, have been developed.

AUTHOR'S NOTE

The written paper was not available at the time of publication. Therefore, the following synopsis was provided in lieu of the full report.

SYNOPSIS

After Sutton Dam, West Virginia, began operations in 1961, a distinct turbidity problem was incurred which adversely affected downstream recreation. The dam had five flood conduits located at the bottom of the lake and did not have a high-level low-flow outlet. Following a storm event, the initial release water had low turbidity even though the downstream tributaries were turbid. After a few days of flood releases, the outflow became turbid and remained turbid for extended periods of time (long after the downstream tributaries became clear). Turbid inflows into Sutton Lake were passing through but the time delay resulted in a few days of high-volume, clear outflows which were followed by many days of low-volume, turbid outflows. Downstream recreation, especially sport fishing, was degraded for several weeks following a single flood event.

To solve the downstream turbidity problem, a semi-circular "riser" was fastened to the face of the dam in front of one of the existing flood gates. The design process included a numerical lake model (to predict temperature changes) and a physical model (to ensure surface vortices did not occur).

The riser met the primary objectives of providing control of the downstream turbidity and providing a warmer outflow temperature. Because of the change in withdrawal zones (from about 100 feet to about 14 feet deep), the thermocline zone became shallower and narrower. The accompanying change in the thermal-density profile caused inflows to enter the lake as an interflow nearer the surface. Thus, a temporary increase in near-surface turbidity was evident after flood events.

Coordination between the West Virginia Department of Natural Resources and the Corps yielded a plan of operation that met the desires of both agencies. During low-flow periods, the riser was used exclusively. When outflows exceeded 1200 cfs, the riser was closed and the bottom gates were used. When outflows exceeded 8000 cfs (summer channel capacity), the riser and all gates were used evenly. The plan allowed for hypolimnetic releases during flood events and conserved the epilimnion for use during low-flow periods without hampering outflow rates or requiring excessive gate operations.

APPENDIX A - LIST OF ATTENDEES

CE WORKSHOP ON RESERVOIR RELEASES
28-30 October 1986
Hyatt Regency Ravinia
Atlanta, Georgia

LIST OF ATTENDEES

Name	Address	Telephone
Steve Adams	Tennessee Valley Authority Engineering Laboratory P.O. Drawer E Norris, TN 37828	615-632-1961
Edward Andrews	USAED*, North Atlantic ATTN: NADEN-TH/90 Church Street New York, NY 10007-9998	FTS 264-7459 212-264-7459
Mark Anthony	USAED, Ohio River ATTN: ORDED-W/P.O. Box 1159 Cincinnati, OH 45202-1159	FTS 684-3070 513-684-3070
Robert Bank	USAED, Baltimore ATTN: NABEN-RW/P.O. Box 1715 Baltimore, MD 21203-1715	FTS 922-4893
Brian L. Barels	Nebraska Public Power District 1414 15th Street Columbus, NE 68601	402-563-5335
Alex Barna	USAED, Pittsburgh ATTN: ORPED-HS/1000 Liberty Avenue Pittsburgh, PA 15222-4186	FTS 722-6860 412-644-6860
Dennis Barnett	USAED, South Atlantic ATTN: SADPD-R 510 Title Building 30 Pryor Street, S. W. Atlanta, GA 30335-6801	FTS 242-4580 404-331-4580
Robert A. Biel	USAED, Louisville ATTN: ORLED-H/P.O. Box 59 Louisville, KY 40201-0059	FTS 352-5640 502-582-5640
David P. Bierl	USAED, Rock Island ATTN: NCRED-HQ Clock Tower Building, P.O. Box 2004 Rock Island, IL 61204-2004	FTS 386-6361 309-788-6361
Chuck Bohac	Tennessee Valley Authority 270 Haneg Building Chattanooga, TN 37401	615-751-7319

* U.S. Army Engineer Division/District

LIST OF ATTENDEES (Continued)

Name	Address	Telephone
Bevan Brown	Tennessee Valley Authority 140 Evans Building Knoxville, TN 37902	615-632-6770
David R. Brown	USAED, Southwestern ATTN: SWDED-WA/1114 Commerce Street Dallas, TX 75242-0216	FTS 729-2384 214-767-2384
Jack Brown	USAED, Nashville ATTN: ORNED-E/P.O. Box 1070 Nashville, TN 37202-1070	FTS 852-5637 615-736-5637
Terry M. Brown	USAED, Wilmington ATTN: SAWEN-HA/P.O. Box 1890 Wilmington, NC 28402-1890	FTS 671-4761 919-343-4761
Dave Buelow	Office, Chief of Engineers ATTN: DAEN-CWH-W 20 Massachusetts Avenue, N.W. Washington, DC 20314-1000	FTS 272-8512 202-272-8512
Joe Carroll	USAE Waterways Experiment Station Richard B. Russell Limnological Laboratory P.O. Box 533 Calhoun Falls, SC 29628	803-447-8561
John Crossman	Tennessee Valley Authority 217 Summer Place Building Knoxville, TN 37902	615-632-6657
Jack L. Davis	Tennessee Valley Authority 252LB-K Knoxville, TN 37902	615-632-7564
Kenneth Day	USAED, Mobile ATTN: SAMOP-R/P.O. Box 2288 Mobile, AL 36628-0001	
Mark S. Dortch	USAE Waterways Experiment Station ATTN: WESES-Q/P.O. Box 631 Vicksburg, MS 39180-0631	FTS 542-3517 601-634-3517
Frank [illegible] Dragoun	Central Nebraska Public Power and Irrigation District P.O. Box 356 Holdrege, NE 68949-0356	308-995-8601
[illegible]	USAED, Mobile ATTN: SAMPD-EI P.O. Box 2288 Mobile, AL 36628-0001	FTS 537-3857 205-694-3857

LIST OF ATTENDEES (Continued)

Name	Address	Telephone
Thomas J. Furdek	USAED, St. Louis ATTN: LMSED-HQ Foot of Arsenal Street St. Louis, MO 63118	FTS 273-4008 314-263-4008
Jim Gallagher	USAED, Savannah ATTN: SASEN-HA/P.O. Box 889 Savannah, GA 31402-0889	FTS 248-5515 912-944-5515
James Graham	USAED, Portland ATTN: NPPEN-HH-S/P.O. Box 2946 Portland, OR 97208-2946	FTS 423-6444 503-221-6444
John Hains	USAE Waterways Experiment Station Richard B. Russell Limnological Laboratory P.O. Box 533 Calhoun Falls, SC 29628	803-447-8561
Brad Hall	USAED, Seattle ATTN: NPSEN-HH-HC/P.O. Box C-3755 Seattle, WA 98124-2255	FTS 446-3597 206-764-3597
Dorothy E. Hamlin	USAE Waterways Experiment Station ATTN: WESES-Q/P.O. Box 631 Vicksburg, MS 39180-0631	FTS 542-2676 601-634-2676
Jeffrey Hanson	USAED, Portland ATTN: NPPEN-HH-R/P.O. Box 2946 Portland, OR 97208-2946	FTS 423-6468 503-221-6468
E. Dean Harshbarger	Tennessee Valley Authority Engineering Laboratory P.O. Drawer E Norris, TN 37828	615-632-1947
Gary E. Hauser	Tennessee Valley Authority Engineering Laboratory P.O. Drawer E Norris, TN 37828	615-632-1888
Jim Helms	USAED, Seattle ATTN: NPSEN-HH-WM/P.O. Box C-3755 Seattle, WA 98124-2255	FTS 446-3544 206-764-3544
Steve Hiebert	U.S. Bureau of Reclamation MC-1522 P.O. Box 25007/Denver Federal Center Denver, CO 80225	FTS 776-6009 303-236-6009

LIST OF ATTENDEES (Continued)

Name	Address	Telephone
Jeff Holland	USAE Waterways Experiment Station ATTN: WESHS-R/P.O. Box 631 Vicksburg, MS 39180-0631	FTS 542-2644 601-634-2644
Bert Holler	USAED, South Atlantic ATTN: SADEN-TH 510 Title Building 30 Pryor Street, S.W. Atlanta, GA 30303-6801	FTS 242-4260 404-331-4260
Stacy Howington	USAE Waterways Experiment Station ATTN: WESHS-R/P.O. Box 631 Vicksburg, MS 39180-0631	FTS 542-2939 601-634-2939
Linda Jenks	USAED, Omaha ATTN: MROED-HC 6014 U.S. Post Office and Courthouse Omaha, NE 68102-4978	FTS 864-4608 402-221-4608
David R. Johnson	USAED, Vicksburg ATTN: LMKED-HW/P.O. Box 60 Vicksburg, MS 39180-0060	FTS 542-7221 601-634-7221
Perry Johnson	U.S. Bureau of Reclamation Mail Code D-1531 P.O. Box 25007 Denver, CO 80225	FTS 776-6160
Robert Kennedy	USAE Waterways Experiment Station ATTN: WESES-A/P.O. Box 631 Vicksburg, MS 39180-0631	FTS 542-3659 601-634-3659
Don Kraus	Central Nebraska Public Power and Irrigation District P.O. Box 356 Holdrege, NE 68949-0356	308-995-8601
Lynn Lamar	Office, Chief of Engineers ATTN: DAEN-CWH-W 20 Massachusetts Avenue, N.W. Washington, DC 20314-1000	FTS 272-8513 202-272-8513
Dale Leggett	USAED, Little Rock ATTN: SWLPL-A/P.O. Box 867 Little Rock, AR 72203-0867	FTS 740-5834 501-378-5834
Jerry Mahloch	USAE Waterways Experiment Station ATTN: WESEP-W/P.O. Box 631 Vicksburg, MS 39180-0631	FTS 542-3635 601-634-3635

LIST OF ATTENDEES (Continued)

Name	Address	Telephone
William P. Mathis	USAED, Little Rock ATTN: SWLPL-A/P.O. Box 867 Little Rock, AR 72203-0867	FTS 740-5033 501-378-5033
Gary Mauldin	USAED, Savannah ATTN: SASEN-HA/P.O. Box 889 Savannah, GA 31402-0889	FTS 248-5512 912-944-5512
N. D. McClure	USAED, Mobile ATTN: SAMPD-E/P.O. Box 2288 Mobile, AL 36628-0001	FTS 537-2666 205-690-2666
Mark F. McKevitt	USAED, Savannah ATTN: SASPD-EI P.O. Box 889 Savannah, GA 31402-0889	FTS 248-5389 912-944-5389
Mark H. Mobley	Tennessee Valley Authority Engineering Laboratory P.O. Drawer E Norris, TN 37828	615-632-1939
Heinz J. Mueller	Environmental Protection Agency, Region IV Environmental Assessment Branch 345 Courtland Street Atlanta, GA 30365	FTS 257-1901 404-347-1901
Patrick Neichter	USAED, Louisville ATTN: ORLED-H P.O. Box 59 Louisville, KY 40201-0059	FTS 352-6[illegible]9 502-582-6[illegible]9
John Nestler	USAE Waterways Experiment Station ATTN: WESES-Q P.O. Box 631 Vicksburg, MS 39180-0631	[illegible] [illegible]
Rudy Ny	USAED, South Atlantic ATTN: SADPD-R 510 Title Building 30 Pryor Street, S.W. Atlanta, GA 30303-6801	[illegible] 46[illegible] 404-[illegible]-46[illegible]
Phillip M. Payonk	USAED, Wilmington ATTN: SAWPD-EW P.O. Box 1890 Wilmington, NC 28402-1890	[illegible] 4589 919-[illegible]-4589
Sue Perry	Federal Energy Regulatory Commission RB308, Office of Hydropower Licensing 825 N. Capitol Street, N.E. Washington, DC 20002-4232	[illegible] [illegible]

LIST OF ATTENDEES (Continued)

Name	Address	Telephone
Allen Piner	USAED, Wilmington ATTN: SAWEN-HA/P.O. Box 1890 Wilmington, NC 28402-1890	FTS 671-4762 919-343-4762
Richard E. Price	USAE Waterways Experiment Station ATTN: WESHS-R/P.O. Box 631 Vicksburg, MS 39180-0631	FTS 542-2440 601-634-2440
Richard E. Punnett	USAED, Huntington 502 8th Street Huntington, WV 25701-2070	FTS 924-5604 304-592-5604
Jim Robinson	USAED, Norfolk ATTN: NAOPL-H/803 Front Street Norfolk, VA 23510-1096	FTS 827-3774 804-441-3774
Richard Roline	U.S. Bureau of Reclamation P.O. Box 25007/Denver Federal Center Denver, CO 80225	FTS 776-6005 303-236-6009
Bill Rushing	USAE Waterways Experiment Station ATTN: WESEP-W/P.O. Box 631 Vicksburg, MS 39180-0631	FTS 542-3542 601-634-3542
Harold [illegible]. Sansing	USAED, Nashville ATTN: ORNED-E/P.O. Box 1070 Nashville, TN 37202-1070	FTS 852-5675 615-736-5675
George A. Sauls	USAED, Philadelphia ATTN: NAPEN-H/2nd & Chestnut Streets Philadelphia, PA 19106-2991	215-597-6829
John Schnagl	Federal Energy Regulatory Commission 400 1st Street, N.W. Washington, DC 20001-2027	FTS 376-9791 202-376-9791
Mike Schneider	USAE Waterways Experiment Station ATTN: WESHS-R/P.O. Box 631 Vicksburg, MS 39180-0631	FTS 542-3424 601-634-3424
Steve Schreiner	USAE Waterways Experiment Station Russell Limnological Laboratory P.O. Box 533 Calhoun Falls, SC 29628	803-447-8561
W[illegible] Seawe	Tennessee Valley Authority, 132 SPB Knoxville, TN 37902	615-632-3243
[illegible]ack W. Simmons	Tennessee Valley Authority 251 84E Lookout Place Chattanooga, TN 37401	615-751-6023

LIST OF ATTENDEES (Concluded)

Name	Address	Telephone
Linda Stafford	USAED, Pittsburgh ATTN: ORPED-HW/1000 Liberty Avenue Pittsburgh, PA 15222-4186	FTS 722-4645 412-644-4645
John A. Swenson	USAED, Philadelphia ATTN: NAPEN-H/2nd & Chestnut Streets Philadelphia, PA 19106-2991	215-597-6832
Lloyd O. Timblin, Jr.	U.S. Bureau of Reclamation Mail Code D-1520/P.O. Box 25007 Denver, CO 80225	FTS 776-5991 303-236-5991
Jim Vearil	USAED, Jacksonville ATTN: SAJEN-HW/P.O. Box 4970 Jacksonville, FL 32232-0019	FTS 946-2116 904-791-2116
Wayne Wagner	USAED, Seattle ATTN: NPSEN-HH-WM/P.O. Box C-3755 Seattle, WA 98124-2255	FTS 446-3542 206-764-3542
Tsong Wei	USAED, Missouri River ATTN: MRDED-TH P.O. Box 103, Downtown Station Omaha, NE 68101-0103	FTS 864-7325 402-221-7325
Paul M. White	USAED, Huntington ATTN: ORHOP-R/502 8th Street Huntington, WV 25705-2070	FTS 924-5488
Steve Wilhelms	USAE Waterways Experiment Station ATTN: WESHS-R/P.O. Box 631 Vicksburg, MS 39180-0631	FTS 542-2475 601-634-2475
Laurin Yates	USAE Waterways Experiment Station ATTN: WESHS-S/P.O. Box 631 Vicksburg, MS 39180-0631	FTS 542-2344 601-634-2344
Tom Yourk	USAED, Savannah ATTN: SASPD-EI/P.O. Box 889 Savannah, GA 31402-0889	FTS 248-5793 912-944-5793
Marc Zimmerman	USAE Waterways Experiment Station ATTN: WESES-Q/P.O. Box 631 Vicksburg, MS 39180-0631	FTS 542-3784 601-634-3784

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