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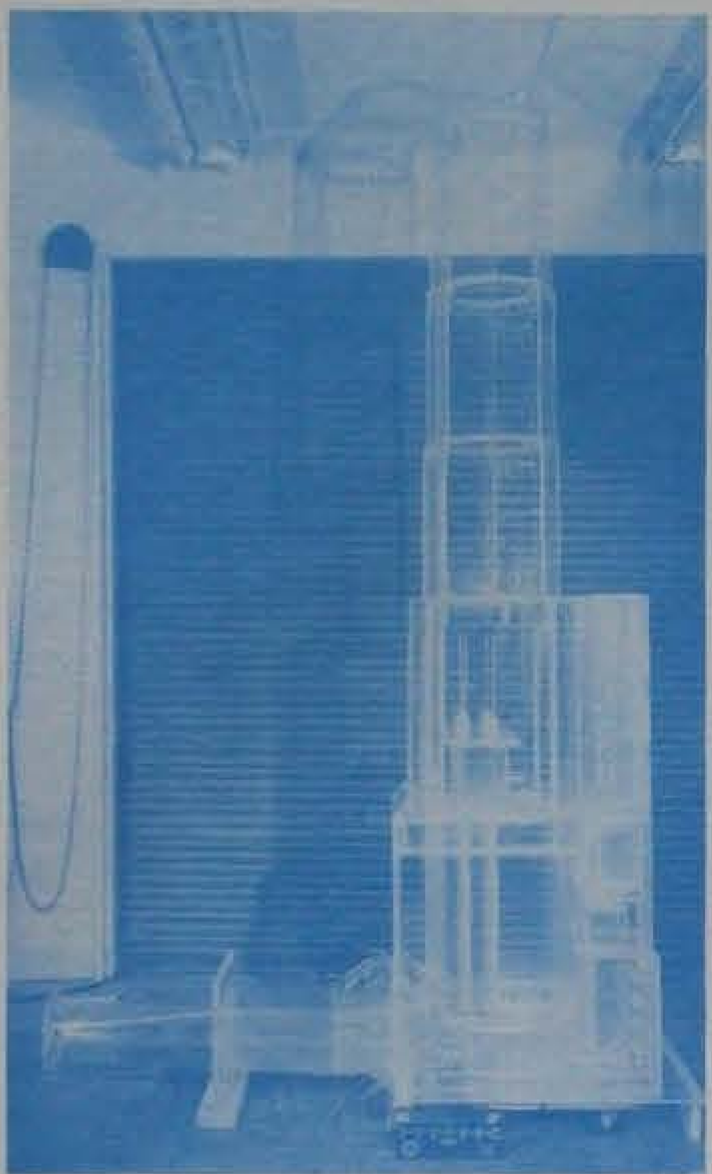

HYDRAULICS LABORATORY

MISCELLANEOUS PAPER HL-86-3

PROCEEDINGS: CE WORKSHOP ON DESIGN AND OPERATION OF SELECTIVE WITHDRAWAL INTAKE STRUCTURES

Hydraulics Laboratory

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

May 1986

Final Report

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During 24-28 June 1985, a workhop on the design and operation of selective withdrawal structures was conducted in San Francisco, Calif. The objective of the workshop was to assemble the personnel within the Corps of Engineers (CE) who have had experience in the design and operation of selective withdrawal structures and thereby provide a forum for information exchange and identification of common problems arising in either design or operation of these structures. Papers were solicited from across the CE and the resulting mix of topics represents a significant portion of the available information and experience within the CE on design and operation of selective withdrawal structures.

At the workshop, two items of structure design were highlighted in presentations and discussions: discharge capacity and multilevel withdrawal flexibility. Limited capacity and limited flexibility were identified as the most common constraints in meeting

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19. ABSTRACT (Continued).

particular water quality objectives. Another common problem cited in the papers was the long-term modification of the project's operational objectives and, as a consequence, the lack of capability to meet the present objectives with the existing structure. Case studies of structural and operational modifications to meet release objectives were presented.

10. Flood Control Hydraulic Research Program Work Unit 31717

EXECUTIVE SUMMARY

During 24-28 June 1985, the Office, Chief of Engineers (OCE) sponsored and the Waterways Experiment Station (WES) conducted a workshop on the design and operation of selective withdrawal structures. OCE, WES, Divisions and Districts from across the Corps of Engineers (CE) were represented by the participants and presenters at the workshop. A list of the attendees and their respective offices is included in the workshop proceedings. The objective of the workshop was to assemble the personnel within the CE who have had experience in the design and operation of selective withdrawal structures and thereby provide a forum for information exchange and identification of common problems arising in either design or operation of these structures. To facilitate distribution of the available design information and operational guidance, papers were solicited from across the CE. The resulting mix of topics represents a significant portion of the available information and experience within the CE on design and operation of selective withdrawal structures.

Examination of the Table of Contents and the paper abstracts shown in Appendix A in the proceedings will reveal that a large range of topics were discussed. Presentations covered the basic concepts of selective withdrawal, specific project designs and operational experiences, original and modified objectives of structure operation, blending of selective withdrawal flows in a single wet well, and the present resources available to assist in the design and operation of a selective withdrawal structure. Papers were presented on the design of structures that have been in existence for a relatively long time and some that are under construction. Structural modifications of existing structures to achieve a particular withdrawal characteristic were discussed. Changes in project objectives and the resulting impact on operations compared with original operational criteria were presented. A generalized analytical description of blending in a single wet well was presented. Basic tools for evaluating withdrawal characteristics, project operations, and structure design were presented. These included discussions about physical hydraulic models and the numerical models SELECT, SELCIDE, and CE-RES-OPT. A field trip to the Warm Springs Project was conducted to give first-hand experience to participants about the size, construction, and potential operations of a single wet well structure.

The two major items of structure design highlighted in the presentations and discussions were discharge capacity and multilevel flexibility. For many existing structures, the discharge capacity of the selective withdrawal system is very low relative to required releases. Thus, at times, ideal operations of the structure cannot be achieved because of this hydraulic constraint. In some cases, there is an insufficient number of selective withdrawal outlets or they may be located at inappropriate elevations. If releases of a particular temperature are desired, without sufficient flexibility to select the outlet elevation, the release temperature objective cannot be met. For the particular structures where these constraints occur, it seems that economics governed the processes for selecting the number and locations of intakes and the overall size of the selective withdrawal system. However, in defense of these designs, the objectives for which the project is currently operated are different than the operational objectives for the original design. In some cases, the postconstruction environment has revealed a desirable quality which has resulted in modification of the proposed operational objectives in order to maintain that quality.

For some structures, where modified operational objectives were significantly different from original objectives, structural modifications have been implemented or are being considered to provide the operational capability to meet the desired release objectives. In one instance a skimming weir was installed just upstream of the outlets on the dam face (Sutton Dam Riser), thereby allowing the release of higher quality surface water. In other cases, where current operational criteria could not meet desired release objectives, operational criteria were modified to allow more flexibility to meet the release objectives. For example, delaying fall drawdown until turnover occurs thereby improving the water quality in the lower level of the reservoir will improve the quality of the water released from a low-level outlet.

Based on a consensus of opinion, the final conclusion of those attending the workshop was that this type of forum should be conducted periodically for educational purposes. Operational and structural alternatives to meet water quality objectives are being formulated and implemented at many projects. The workshop provided a forum for the exchange of this information and experience. Hence, it was considered to be in the best interest of the CE to conduct this workshop again at some future date.

Appreciation must be expressed to OCE for sponsoring the workshop; to the presenters who provided the subjects of discussion; to Messrs. Dick DiBuono and Harold Huff, the local organizers from the South Pacific Division (SPD) and Sacramento District, respectively; to Mr. Steve Phillips and the Resource Management Staff at Warm Springs Reservoir; to Mr. Ted Albrecht, SPD Retired; and to the personnel of the WES Hydraulics Laboratory who conducted the workshop.

PREFACE

This report summarizes the results of the CE Workshop on the Design and Operation of Selective Withdrawal Structures held in San Francisco, California, 24-28 June 1985. The Workshop was funded by the Flood Control Hydraulics (FCH) Research Program Work Unit entitled "Design Configurations of Multi-Level Intakes," which was sponsored by the Office, Chief of Engineers (OCE). The Technical Monitor of the FCH Research Program for OCE was Mr. Tom Munsey. Mr. Burton Boyd, Chief, Hydraulic Analysis Division, Hydraulics Laboratory (HL), was Program Manager of the FCH Research Program.

Mr. John L. Grace, Jr., Chief, Hydraulic Structures Division, and Mr. Frank A. Herrmann, Jr., Chief, HL, directed the effort. The workshop was organized and conducted by Mr. Jeffery P. Holland, Chief, Reservoir Water Quality Branch. Mr. Holland was assisted by Messrs. Stacy E. Howington and Steven C. Wilhelms and Miss Laurin I. Yates. Mr. Dick DiBuono, South Pacific Division, assisted with local arrangements; Messrs. Harold Huff and Steve Phillips, Sacramento District, assisted with field trip arrangements. Papers were prepared and presented by personnel from across the CE. Mr. Wilhelms organized this report which was edited by Mrs. Beth F. Vavra, Publications and Graphic Arts Division.

Director of WES was COL Allen F. Grum, USA. Technical Director was Dr. Robert W. Whalin.

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CONVERSION FACTORS, NON-SI TO SI (METRIC)
UNITS OF MEASUREMENT

Non-SI units of measurement used in this report can be converted to SI (metric) units as follows:

Multiply	By	To Obtain
acres	4,046.873	square metres
acre-feet	1,233.482	cubic metres
cubic feet per second	0.02831685	cubic metres per second
Fahrenheit degrees	5/9	Celsius degrees or Kelvin*
feet	0.03048	metres
gallons per minute	0.06308	litres per second
inches	2.54	centimetres
miles (US Statute)	1.609347	kilometres
ounces (mass)	28.34952	grams
pounds (mass)	0.4535924	kilograms
pounds (mass) per cubic foot	16.01846	kilograms per cubic metre
pounds (force) per square foot	47.88026	pascals
pounds (force) per cubic foot	7.9395	Newtons per cubic metre
square feet per day	0.09290304	square metres per day
square miles (US statute)	2.589988	square kilometres

*To obtain Celsius (C) temperature readings from Fahrenheit (F) readings, use the following formula: $C = (5/9)(F - 32)$. To obtain Kelvin (K) readings, use: $K = (5/9)(F - 32) + 273.15$.

CE Workshop on Selective Withdrawal Intake Structure
Design and Operation
24-27 June 1985
Bellevue Hotel
San Francisco, California

LIST OF ATTENDEES

Name	Office	FTS Number
Ted Albrecht	Consultant (SPD Ret)	415-829-0139
Chandra Alloju	Fort Worth District	334-2210
Boniface Bigornia	Los Angeles District	894-6979
David Brown	Southwestern Division	329-2384
Dave Buelow	Ohio River Division	684-3070
Dick Cassidy	Portland District	423-6469
Dick DiBuono	South Pacific Division	556-2033
Earl Eiker	OCE	272-8509
Jim Gallagher	Savannah District	248-5510
John C. Gribar	Pittsburgh District	722-6820
Floyd Hall	Portland District	423-6407
Dale Hart	Waterways Exp. Station	542-2258
Jim Helms	Seattle District	399-3544
Jeff Holland	Waterways Exp. Station	542-2644
Stacy Howington	Waterways Exp. Station	542-2939
Harold Huff	Sacramento District	448-3583
Michael Koryak	Pittsburgh District	722-6831
Carl F. Kress	South Pacific Division	556-8549
Gordon Lance	Louisville District	352-5764
Kenneth Lee	Baltimore District	922-4893
Walter M. Linder	Kansas City District	758-3854
Mark Lindgren	Walla Walla District	434-6518
Len Lipski	Philadelphia District	597-6829
Warren Mellema	Missouri River Division	864-7323
Margaret Morehead	Little Rock District	740-6180
Tom Munsey	OCE	272-8504
Sam Powell	OCE	272-8501
Richard Price	Vicksburg District	542-5697
Richard Punnett	Huntington District	924-5248
Sam Sang	Huntsville Division	873-5190
Dennis Seibel	Baltimore District	922-4840
Bolyvong Tanovan	North Pacific Division	423-3764
Ronald L. Turner	Fort Worth District	334-2222
Frank Vovk	Omaha District	864-4611
Steve Wilhelms	Waterways Exp. Station	542-2475
Don Wood	New England Division	839-7601

AGENDA
CE Workshop on Selective Withdrawal Intake Structure
Design and Operation
24-27 June 1985
Bellevue Hotel
San Francisco, California

Monday, 24 June

Time	Topic	Speaker	Office
0730-0830	Registration		
0830	Welcome		SPD
0845	Administration	Jeff Holland/ Dick DiBuono	WES SPD
0855	OCE Overview	Sam Powell	OCE
0910	Selective Withdrawal: Basic Concepts	Steve Wilhelms	WES
0940	Break		
1010	Hydraulic Design of the Selective Withdrawal Structures in the Rogue River Basin	Floyd Hall	NPP
1040	Reservoir Regulation and Selective Withdrawal in Oregon	Dick Cassidy	NPP
1110	Break		
1130	Operation of Selective Withdrawal Facilities Libby Dam, Montana	Jim Helms	NPS
1200	Lunch		
1340	Alternatives for Improving Reservoir Water Quality	Margaret Morehead	SWL
1410	Break		
1440	A Review of Selective Withdrawal Performance in the Fort Worth District	Ronald Turner	SWF
1500	Modeling of Selective Withdrawal Intake Structures	Chandra Alloju	SWF
1530	Adjourn		

Tuesday, 25 June

0830	Bloomington Dam, and Warm Springs Water Quality Outlets	Frank Vovk	MRO
0900	Hydraulic Design: Bloomington Lake and F. E. Walter Dam Projects', Selective Withdrawal Structures	Dennis Seibel	NAB
0930	Break		
1000	Discussion: Past and Proposed Hydraulic Testing of Prototype Intake Towers	Dale Hart	WES

Tuesday, 25 June (con)

Time	Topic	Speaker	Office
1040	Break		
1110	Determination of Selective Withdrawal System Capacity for Intake Tower Design	Ken Lee	NAB
1140	START OF WES SELECTIVE WITHDRAWAL TUTORIAL		
1140	SELECT: The Numerical Model	Steve Wilhelms	WES
1210	Lunch		
1340	SELECT Example	Steve Wilhelms/ Stacy Howington	WES
1410	Blending in a Single Wet Well	Stacy Howington	WES
1440	Break		
1500	Operational Tools	Jeff Holland	WES
1530	Overview of Warm Springs	Harold Huff	SPK
1600	Adjourn		

Wednesday, 26 June

Time	Topic
0830	Field Trip to Warm Springs
1700	Return to Hotel

Thursday, 27 June

Time	Topic	Speaker	Office
0830	Field Measurements at Intake Structures	Dale Hart	WES
0900	Thoughts and Considerations for Hydraulic Design	Ted Albrecht	SPD (ret
0930	Break		
0950	Selective Withdrawal Needs for Lake Greeson, Arkansas Intake Structure	Richard Price	LMK
1020	Selective Withdrawal Structure Operation Experiences in ORD	Dave Buelow	ORD
1050	Break		
1110	Overview of Pittsburgh District Selective Withdrawal Operation Experiences	Mike Koryak	ORP
1140	Selective Withdrawal From Any Level Between Minimum Pool and Spillway Elevation at Stonewall Jackson Dam, West Virginia	John Gribar	ORP
1210	Lunch		
1340	Selective Experiences: Alias Withdrawal Pains	Richard Punnett	ORH

Thursday, 27 June (con)

Time	Topic	Speaker	Office
1410	Design and Performance of Skimming Weirs in the Kansas City District	Walt Linder	MRK
1440	Break		
1500	Small Group Discussions of Selective Withdrawal Issues Led by OCE Technical Monitors		
	Water Quality	Earl Eiker	OCE
	Hydraulic Design	Tom Munsey	OCE
		Sam Powell	OCE
1600	Summary and Concluding Remarks	Jeff Holland	WES

SELECTIVE WITHDRAWAL: BASIC CONCEPTS

Steven C. Wilhelms*

ABSTRACT. An intuitive approach is used to explain the stratified flow phenomenon known as selective withdrawal. Basic definitions and descriptions are given for the geometries, stratification conditions, and other processes that impact the selective withdrawal characteristics of ports and weirs. Equations that describe the establishment of a withdrawal zone are presented for various density stratifications. The basis for the numerical predictive model SELECT is presented. Application of SELECT to predict release concentrations of water quality constituents is discussed.

INTRODUCTION. Selective withdrawal as a concept is very simple. It is the capability to predict the flow distribution caused by release of water from a stratified impoundment and selectively apply that capability to withdraw water of a desired quality. However, the simplicity ends with that description. The real complexity of describing selective withdrawal is aptly illustrated by the extensive research that has been conducted over the last 40 years. This research includes a substantial research effort by the Corps of Engineers. The objective of these efforts has not been just to describe selective withdrawal but to develop techniques to predict withdrawal characteristics and then apply those techniques in evaluation and engineering design of a project.

DEFINITIONS. Let us consider an intuitive approach to simply describe the selective withdrawal process. Before we continue, some definitions are needed:

a. outlet - any device, regardless of shape or size, that is used to withdraw water from a reservoir.

b. stratification - the result of the declining impact with depth of thermal input to the surface of a reservoir or the impact of dissolved or suspended solids in the body of a reservoir that results in a vertical density gradient from the surface to the bottom.

c. stratified flow - any flow situation where density stratification influences or modifies the flow pattern which would be observed for the non-stratified condition.

d. withdrawal zone - the vertical extent of water in a stratified reservoir that moves toward and is withdrawn through an outlet.

e. point sink - conceptualization of an outlet in which the dimensions of the outlet are small in relation to the withdrawal zone thickness. Water is withdrawn from three dimensions into a "point."

f. line sink - conceptualization of an outlet in which the vertical dimension of the outlet is small relative to the withdrawal zone thickness but the width of the outlet is infinite (or as wide as the reservoir). Water is

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withdrawn from two dimensions into a "line."

g. two-layer stratification - stratification condition with two homogeneous layers of different densities separated by a sharp, distinct, horizontal interface.

h. linear stratification - stratification with a constant change in density per unit depth from top to bottom of the reservoir.

i. chemical stratification - usually the result of impeded circulation within an impoundment because of thermal or density stratification. Chemical and biological processes often reduce oxygen or concentrations release other constituents when stratification impedes transport of oxygen from the surface to the lower levels of the reservoir.

j. weir - outlet device that "skims" fluid usually from the surface layer of a stratified reservoir.

With these definitions, let us examine the stratified flow conditions and processes that affect selective withdrawal.

CONCEPTS IN SELECTIVE WITHDRAWAL. First let us consider point sink withdrawal from a reservoir with two-layer stratification (Figure 1a). Consider the energy of the discharge as it passes through the outlet. During release, we are converting potential energy (represented by the head in the reservoir) into inertial energy (velocity) and losses in the outlet. How much energy can we convert without withdrawing water from the lower layer?....What is the "critical" discharge at which withdrawal is incipient from the lower layer? That critical point occurs when the inertial energy and the losses are equal to the buoyancy or potential energy of the lower layer. Craya[2] and Gariel[4] mathematically described this critical discharge with

$$\frac{Q_c}{\sqrt{\frac{\Delta\rho}{\rho} g h^5}} = 2.54 \qquad (1)$$

where

Q_c = critical discharge from one layer without withdrawing fluid from other layer

ρ = density of upper layer

g = gravitational acceleration

and other variables are defined in Figure 1a.

Thus the critical discharge can be predicted given the geometry of the outlet relative to the stratification and the strength of the stratification. It must be noted that this predictive equation is completely valid for an idealized two-layer stratification withdrawal through a point sink on a wall. If the interface is diffuse, however, then Equation 1 will overpredict the critical discharge.

It is unfortunate, but rarely does a reservoir stratify with the idealized conditions of two-layer stratification. We must therefore examine additional stratification conditions to discern their impact on the formation of the withdrawal zone. Consider the withdrawal through a point sink from a linearly stratified impoundment, i.e. $d\rho/dz$ = Constant (Figure 1b). What causes the formation of an upper and lower limit of withdrawal? Again, remember the

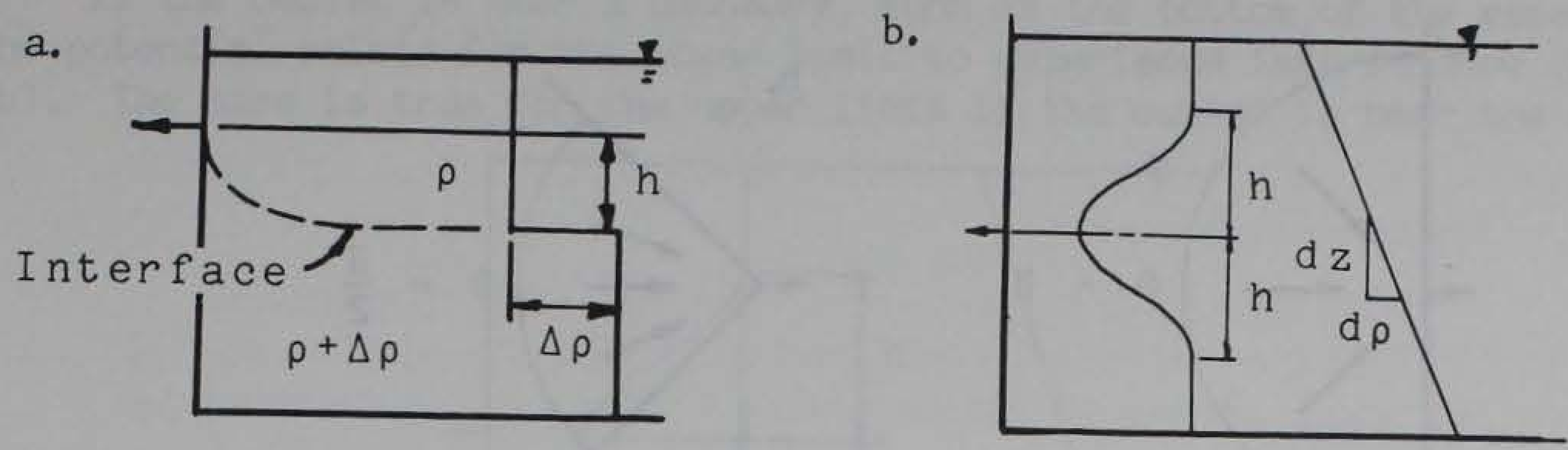

Figure 1. Withdrawal from Two-Layer (a) and Linear (b) Stratification

buoyancy of the fluid in the water column. The potential energy of the water above the upper limit (or below the lower limit) is greater than the energy that is being converted to inertial energy and hydraulic losses. There is not enough energy conversion to "pull" the water above the upper limit down or the lower water, below the lower limit, up to be withdrawn through the outlet. This is called "intermediate" withdrawal because the vertical limits of the withdrawal zone form freely in the pool; there is no "boundary interference." Smith et al. (1985) characterized this withdrawal with

$$\frac{Q}{\sqrt{\frac{g}{\rho}\frac{d\rho}{dz}h^6}} = 1.0 \qquad (2)$$

where

Q = the discharge through the outlet

ρ = the density at the elevation of the outlet

and the other variables are defined in Figure 1b.

Notice the different description (coefficient) for this stratification compared with the two-layer description (Equation 1). The reason for this "difference" lies in the amount of work that has to be performed to withdraw water. At some small distance above the outlet, there is a small density difference for linear stratification; for two-layer stratification, there is no such difference for this same distance. Therefore more work has to be performed to withdraw the water from a linearly stratified reservoir than from the two-layer system. Thus, with a finite amount of energy available to do work, less discharge can be withdrawn for linear stratification than two-layer stratification. This is why the two-layer description will overpredict the critical discharge for a diffuse (linear) interface between the layers.

Smith et al.[7] also introduced the concept of the "withdrawal angle" Θ which impacts the vertical location of limit formation and modifies Equation 2 to become

$$\frac{Q}{h^3\sqrt{\frac{g}{\rho}\frac{d\rho}{dz}}} = \frac{\Theta}{\Pi} \qquad (3)$$

For example, consider the plan view of withdrawal shown in Figure 2a. For a given linear stratification the discharge Q is described by Equation 3 with $\Theta = \Pi$. What would be the discharge from the geometry shown in Figure 2b given the same stratification? Intuitively, the withdrawal rate would be half the discharge discussed in the previous example. Equation 3 also indicates that the discharge would be $Q/2$.

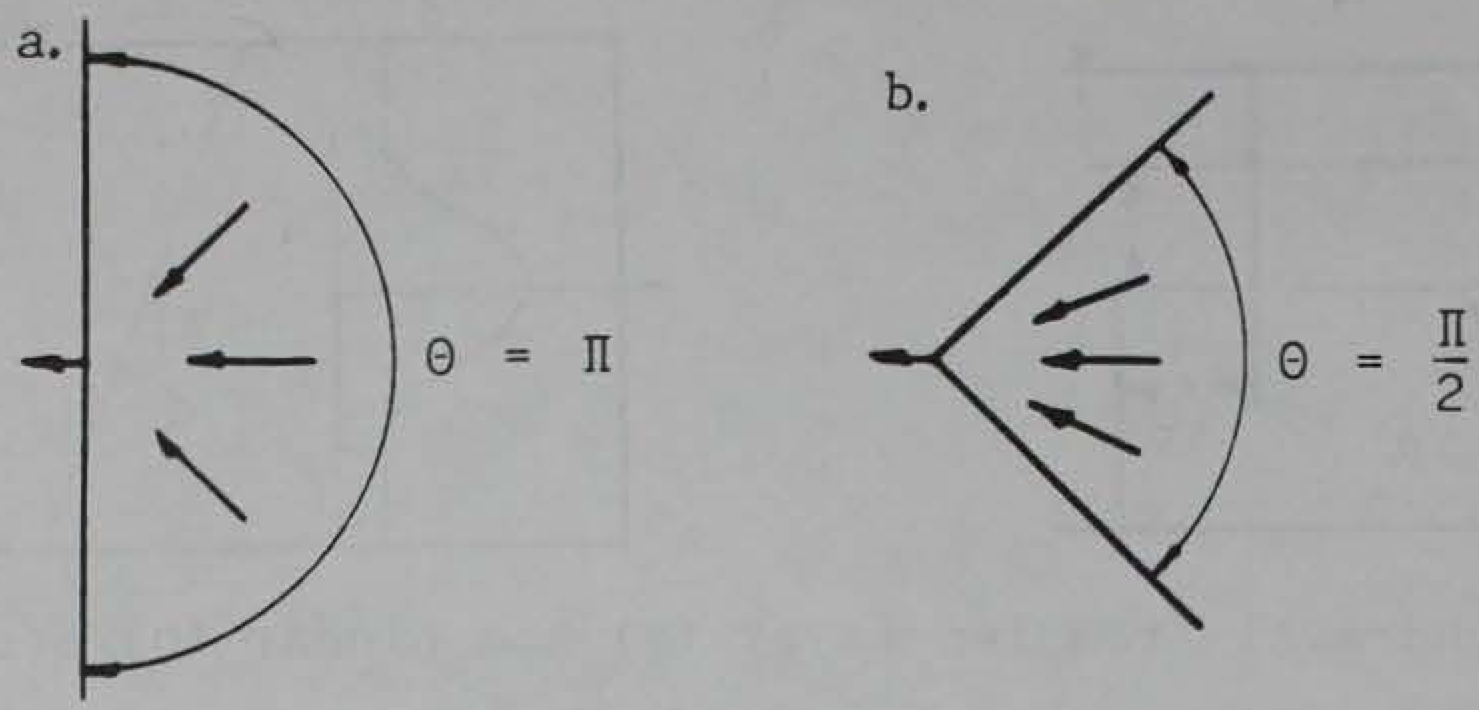

Figure 2. Withdrawal Angle Examples

As with the two-layer stratification, linear stratification is an idealized situation that may not occur very often in an impoundment. Usually, the thermal structure of an impoundment will result in stratification with linear, homogeneous, and perhaps two-layer sections. However, the processes that impact the formation of the withdrawal zone are the same as those for linear stratification. Bohan and Grace[1] conducted withdrawal experiments with stratification similar to that expected in CE impoundments. As explained by Smith et al.[7], Bohan and Grace implicitly analyzed their experimental tests for linear or arbitrary stratification from the outlet to the limit. Although some scatter exists in their data (because of the non-linear density profiles), their results can also be described by Equation 3. Therefore Equation 3 could be used to predict the limits of withdrawal from a CE impoundment given the stratification conditions, outlet configuration, and withdrawal rate.

In addition to the idealized outlet geometry of the "point sink," the "line sink" has been investigated by several researchers. The mathematical expression from which the withdrawal limits can be calculated is similar to the descriptions of the point sink. The line sink concept may be applicable to prototype situations when several closely spaced outlets (at the same elevation) are operated, giving the appearance of a "line" outlet, such as for a hydropower project.

SPECIAL CONSIDERATIONS. What happens to the withdrawal zone if the outlet is near the surface or the bottom of the reservoir? What happens when more than one outlet withdraws water; is the withdrawal zone the same with one outlet as with two multilevel outlets? Both of these scenarios can potentially interfere with the formation of limits that would be described by Equation 3. This interference would cause the limits to form at elevations different from those for intermediate withdrawal without interference.

For instance, consider multilevel withdrawal with two outlets withdrawing water (Figure 3a). The overlap of the upper limit of outlet No. 2 with the lower limit of outlet No. 1 will impact the formation of these two limits. Each port is trying to withdraw the water in the shaded area in Figure 3a. Thus each limit would have to "shift" farther away from its respective outlet to increase the withdrawal from the shaded area, thereby accounting for the withdrawal from that region into each outlet. Conservation of energy for the system dictates that more flow must come from the area of interference. Thus the inertial energy and hydraulic losses at the outlet are balanced by the potential energy of the water that is being withdrawn.

If the outlet is near a boundary, such as the bottom of the reservoir, then the potential exists for the lower limit to experience interference (Figure 3b). The same is true for the upper limit if the outlet is near the reservoir

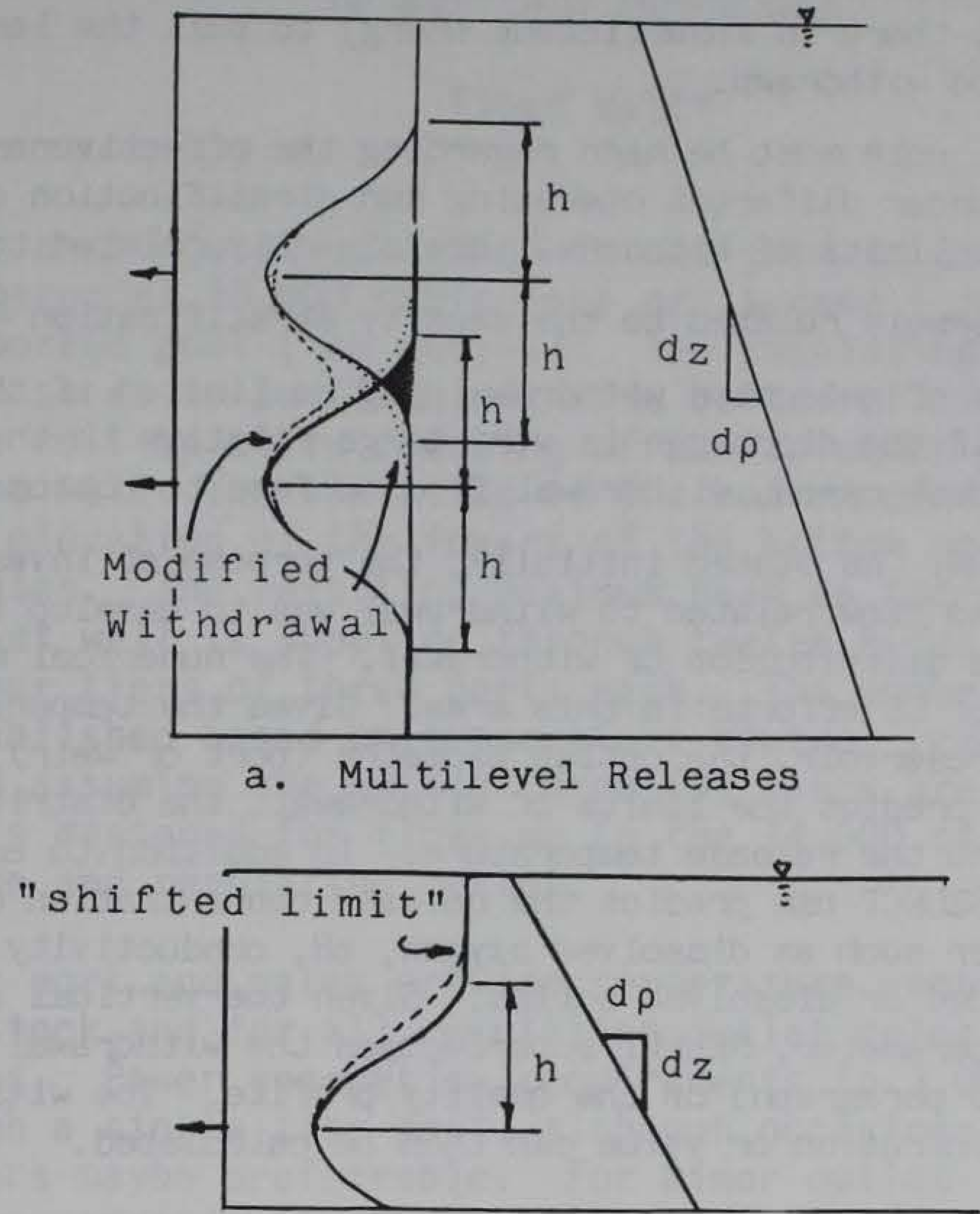

Figure 3. Withdrawal Zone Interference

surface. For bottom interference, the outlet would like to withdraw water from lower in the pool (in accordance with Equation 3). However, since the bottom interferes with the free formation of the lower limit, then the upper limit must form higher in the pool. Consider again the conversion of energy from potential (stratification) to inertial (velocity) and hydraulic losses. If interference inhibits limit formation, then the inertial energy and losses have not been balanced by potential energy. Therefore the upper limit is forced higher in the pool.

OTHER OUTLET DEVICES. Grace[5] and Harleman et al.[6] investigated the withdrawal characteristics of a submerged weir. In most applications, a weir is used to skim water either from the surface or the bottom of a reservoir for temperature control. For effecting a surface withdrawal, Figure 4 shows the geometric relationship between the weir elevation and the pycnocline (density

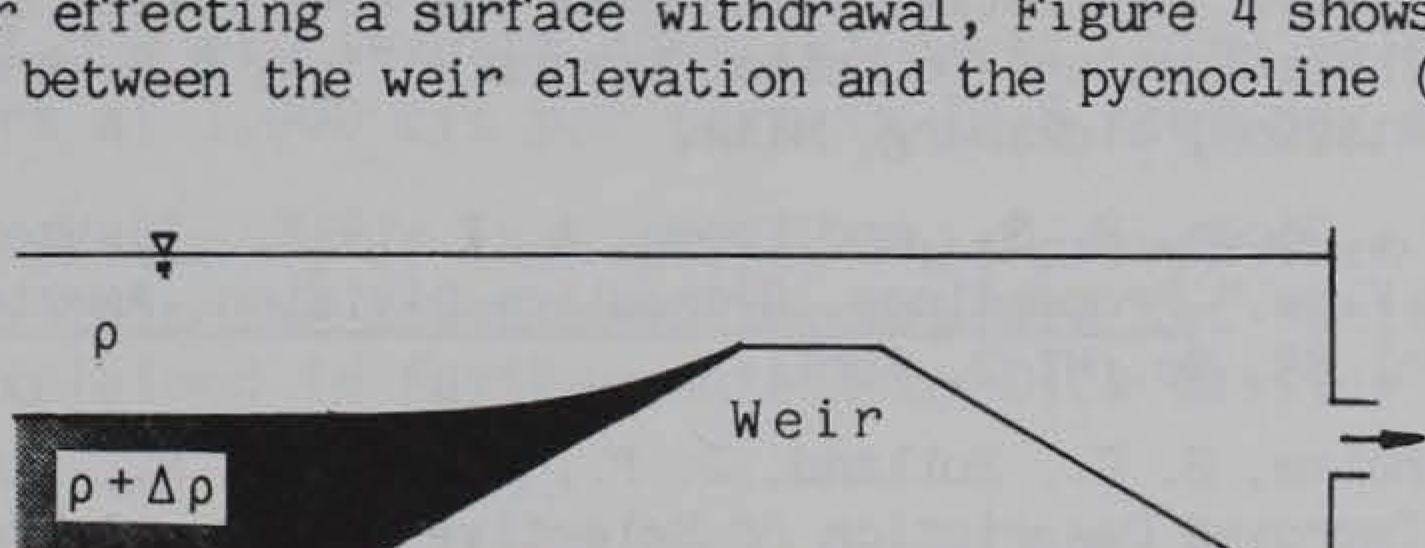

Figure 4. Weir Withdrawal

discontinuity or thermocline). Similar to point sink withdrawal, the energy at the withdrawal device (weir crest) is insufficient to "pull" the dense (cold) water up to the crest and out. For an "inverted" skimming weir designed to withdraw bottom water, there is insufficient energy to pull the less dense (warm) water down to be withdrawn.

SPECIAL NOTE. A final note must be made regarding the effectiveness of selective withdrawal under different operating and stratification conditions. You will note that the limits of withdrawal are directly related to the discharge Q and inversely related to the density stratification $\frac{d\rho}{dz}$ (Equation 3). The effectiveness of selective withdrawal may be limited if the stratification is very weak or if the discharge is very large relative to the strength of stratification. In these cases, withdrawal from surface to bottom may occur.

RESULTS AND APPLICATION. As stated initially, the purpose of investigating the mechanics of stratified flow related to withdrawal was to develop techniques to predict the limits and distribution of withdrawal. The numerical model SELECT[3] is a primary product of CE efforts in this area. Given the temperature or density profile of a reservoir, the outlet geometry (port or weir), and discharge, SELECT can predict the limits of withdrawal, the distribution flow between the limits, and the release temperature. In addition to estimating release temperature, SELECT can predict the release concentration of any in-lake water quality parameter such as dissolved oxygen, pH, conductivity, iron, manganese, and suspended or dissolved solids. Given the vertical distribution of the water quality parameter, SELECT superimposes the withdrawal distribution (discussed in previous paragraph) on the quality profile. The withdrawal-weighted release concentration or value can then be calculated.

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HYDRAULIC DESIGN OF THE SELECTIVE WITHDRAWAL STRUCTURES OF THE LOST CREEK DAM

Floyd Hall*

The intake tower and regulating outlet is designed to pass the normal maximum evacuation discharge of 10,000 cubic feet per second (cfs) at the 15 percent flood control storage pool (see encl-1). The regulating outlet intake invert has been lowered so as to be flush with the bottom of the 33-foot-diameter intake tower wet well to allow flushing of debris which may accumulate at the bottom of the well. The elevation of the bottom of the well is dictated by the required elevation of the invert of the bottom port for temperature control of releases. The resulting maximum head on the outlet invert is 232-feet. The wet well is supplied through twelve 8-feet-wide by 15-feet-high intake ports, four tiers of three ports each. The number of tiers and elevation of each was established based on temperature control requirements. The regulating outlet was sized assuming the bottom three (3) tiers are in use. The intake tower wet well is designed for flows up to the 14,000 cfs combined capacity of the powerhouse and regulating outlet.

The intake tower port and gates provide temperature control for all releases through the penstock and for all regulating outlet releases except major flood evacuations. Power generation requirements to 3,000 cfs will normally be passed through a single tier port, although occasionally use of ports from two (2) tiers maybe preferrable. For minor outlet releases in addition to power releases, additional ports will be opened to limit the discharges to approximate 1,000 cfs per port. The normal maximum evacuation discharge of 10,000 cfs will normally be drawn through the lower nine (9) ports although all twelve (12) ports might be used at high pool levels. The 10,000 cfs release will normally be divided 2,300 - 3,000 cfs to the powerhouse and 7,000 - 7,700 cfs to the regulating outlet. Assuming an outlet release of 7,700 cfs, a powerhouse release of 2,300 cfs and an equal distribution of inflow from the nine (9) lower most ports, the average verticle velocity in the 33-feet-diameter wet well just above the outlet entrance will be approximately 5 feet per second. For releases of the full 10,000 cfs through the oulet the corresponding velocity would be 8 feet per second. The ports have been rated as orifices with a coefficient of 0.9 assuming an error reduction for the 1-foot-wide single verticle trash bar. The head loss, assuming no debris buildup, will vary from 1.7 feet for 3,000 cfs through three (3) ports, to 2.1 feet for 10,000 cfs through nine (9) ports, and 4.2 feet for the rare condition of 14,000 cfs through nine (9) ports. The velocity though the net area of the port at 1,000 cfs per port is 10.6 feet per second (fps).

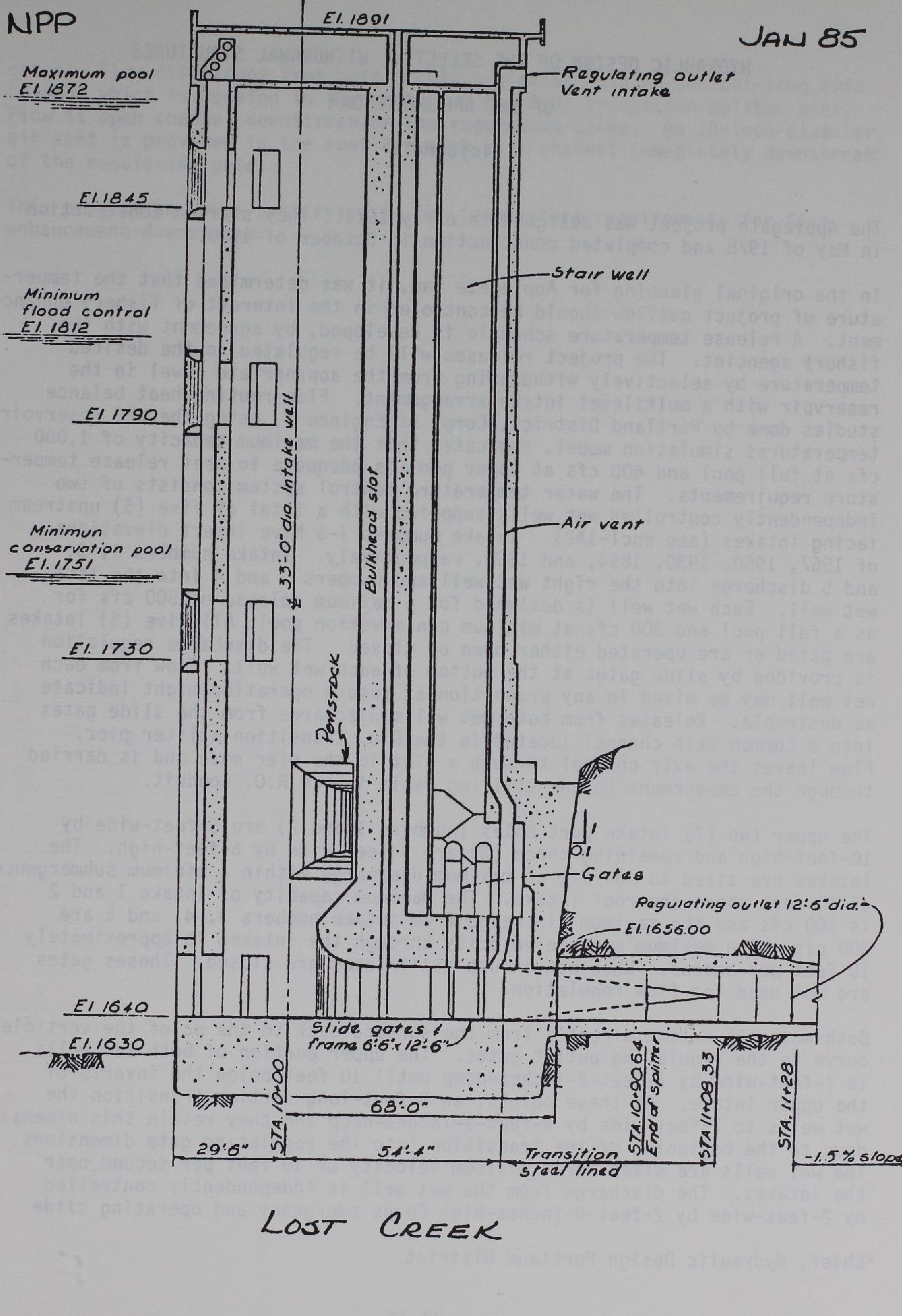
The prototype monitoring of this project has proven that the project can be operated to maintain the downstream temperature set forth by the agencies. This will be explained in further detail by Dick Cassidy who follows my presentation.

A 400-feet-long 12½-feet-diameter turbidity conduit (elephant's trunk) was a late add on to withdraw turbidity from the lower range of the pool. This

*Chief, Hydraulic Design Portland District

conduit was connected to the center lower most port and designed for a flow of 1,150 cfs, resulting in a maximum velocity of 9.37 fps within the conduit. This project was designed in 1967-1968, start construction in June of 1972, and completion construction in October of 1976.

We originally designed the project for dissipating energy below the regulating outlet by utilizing a stilling basin. In the late 1960s, we became aware of supersaturation of nitrogen of which after sampling high nitrogen supersaturation in the stilling basins created a real problem at this project, in that we had a multimillion dollar fish hatchery just downstream of this project that uses the project tailrace as it's source of water supply requiring a drastic redesign or modification to provide acceptable water to the hatchery. After much research of the northwest projects, the results of the flip bucket design at Lucky Peak Dam produced a key to reducing nitrogen supersaturation to between 104% to 105%. This Lucky Peak flip bucket design was modified and replaced the stilling basin design at Lost Creek Dam. It was redesigned with a flip bucket wherein we assured the agencies that nitrogen supersaturation would not be above 106%. With the assurance from us, the Corps, that if we did not reach this goal, we would do whatever was necessary to provide the proper quality of water for the hatchery. Monitoring the downstream nitrogen has proven to retain the supersaturation within these limits.


NPP
JAN 85
El. 1891
Maximum pool
El. 1872
Regulating outlet
Vent intake
El. 1845
Stair well
Minimum
flood control
El. 1812
El 1790
33'-0" dia. Intake well
Bulkhead slot
Air vent
Minimun
conservation pool
El. 1751
El. 1730
Penstock
1'
0.1
Gates
Regulating outlet 12'-6" dia.
El. 1656.00
El 1640
El. 1630
Slide gates & frame 6'6"x12'6"
STA. 10+00
STA. 10+90.64
End of splitter
STA. 11+08.33
STA. 11+28
68'-0"
29'-6"
54'-4"
Transition
steel lined
-1.5% slope
LOST CREEK

HYDRAULIC DESIGN OF THE SELECTIVE WITHDRAWAL STRUCTURES OF THE APPLEGATE DAM

Floyd Hall*

The Applegate project was designed in early 1971. They started construction in May of 1978 and completed construction in October of 1980.

In the original planning for Applegate Dam, it was determined that the temperature of project outflow should be controled in the interest of fishery enhancement. A release temperature schedule is developed, by agreement with the fishery agencies. The project releases will be regulated to the desired temperature by selectively withdrawing from the appropriate level in the reservoir with a multilevel intake arrangement. Flow routing heat balance studies done by Portland District, Corps of Engineers, using the WRE reservoir temperatures simulation model, indicated that the maximum capacity of 1,000 cfs at full pool and 600 cfs at lower pool is adequate to meet release temperature requirements. The water temperature control system consists of two independently controlled wet wells supplied with a total of five (5) upstream facing intakes (see encl-1&2). Intake numbers 1-5 have invert elevations of 1967, 1950, 1930, 1894, and 1838, respectively. Intake numbers 1, 3, and 5 discharge into the right wet well and numbers 2 and 4 into the left wet well. Each wet well is designed for a maximum release of 500 cfs for as a full pool and 300 cfs at minimum conservation pool. All five (5) intakes are gated or are operated either open or closed. The discharge regulation is provided by slide gates at the bottom of each wet well. Flow from each wet well may be mixed in any proportion at future operation might indicate as desirable. Releases from both wet wells discharge from the slide gates into a common exit channel located in the R.O. transition spliter pier. Flow leaves the exit channel through a slot in the pier nose and is carried through the embankment to the stilling basin in the R.O. conduit.

The upper two (2) intake port holes (numbers 1 and 2) are 5-feet-wide by 10-feet-high and remaining three (3) are 5-feet-wide by 6-feet-high. The intakes are sized to operate at maximum discharge within a minimum submergence of 10 feet above the roof intake. The maximum capacity of intake 1 and 2 is 500 cfs and the maximum discharge from intake numbers 3, 4, and 5 are 300 cfs. The maximum average velocity through the intakes is approximately 10 feet per second. When not in use all intakes are closed. Theses gates are not used for flow regulation.

Both wet well extend directly from the surface deck to the pc of the verticle curve to the regulating outlet gates. The upper portion of both wet wells is 7-feet-wide by 7-feet-2-inches-deep until 10 feet below the inverts of the upper intake. At these points, an 8-foot-long section transition the wet wells to 7-feet-wide by 4-feet-9-inches-deep and they retain this dimension down to the beginning of the transition into the regulating gate dimensions. The wet wells are sized for a maximum velocity of 10 feet per second near the intakes. The discharge from the wet well is independently controlled by 2-feet-wide by 2-feet-8-inches-high Corps emergency and operating slide

*Chief, Hydraulic Design Portland District

gates. Flow discharges from both regulating gates into an 80-foot-long exit channel which is located in the center of the R.O. transition spliter pier. Flow is open channel downstream of the regulating gates. An 18-inch-diameter air vent is provided in the roof for each exit channel immediately downstream of the regulating gate.

This project has been satisfying in the temperature requirements for fish enhancement downstream.

NPP

APPLEGATE

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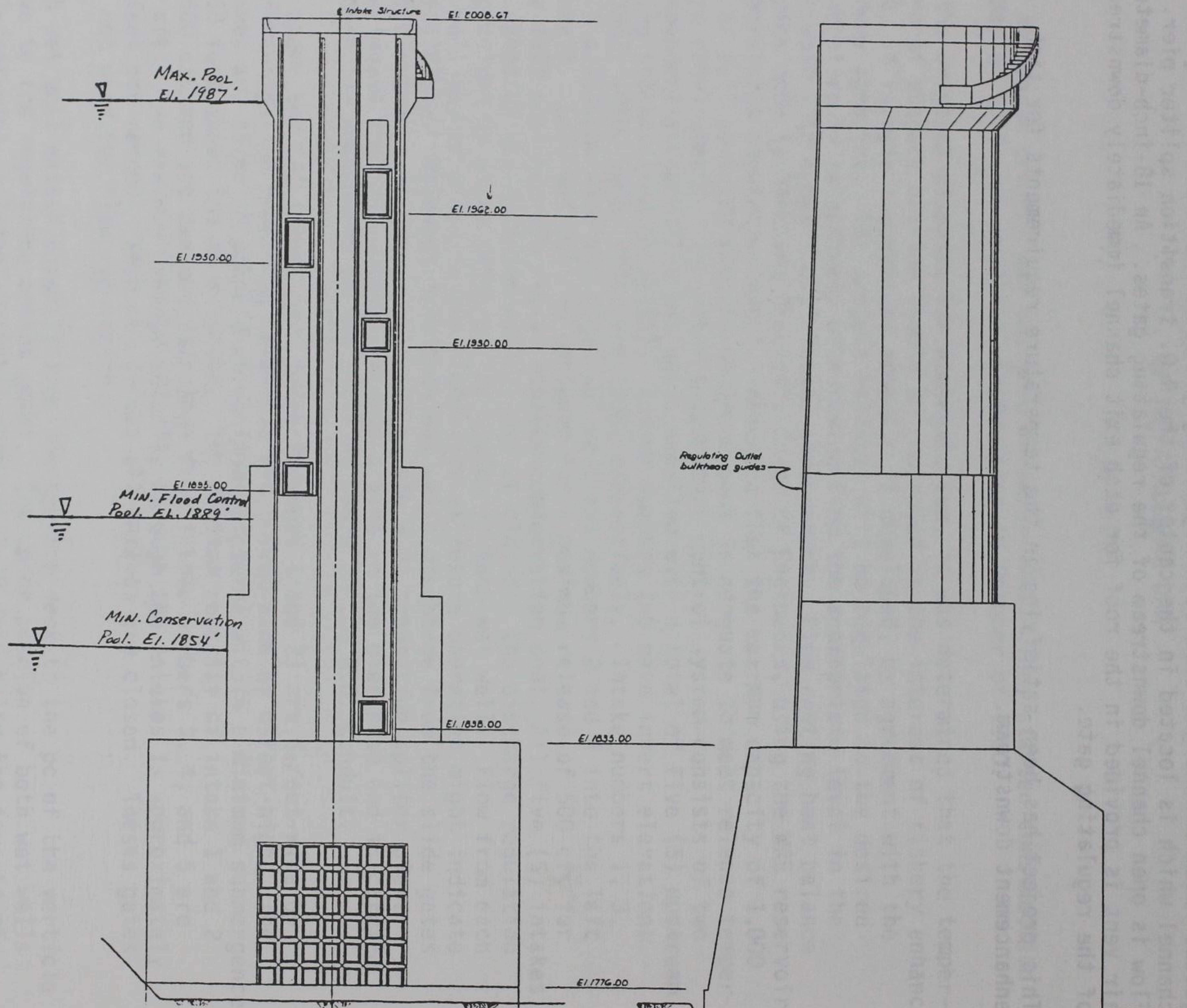

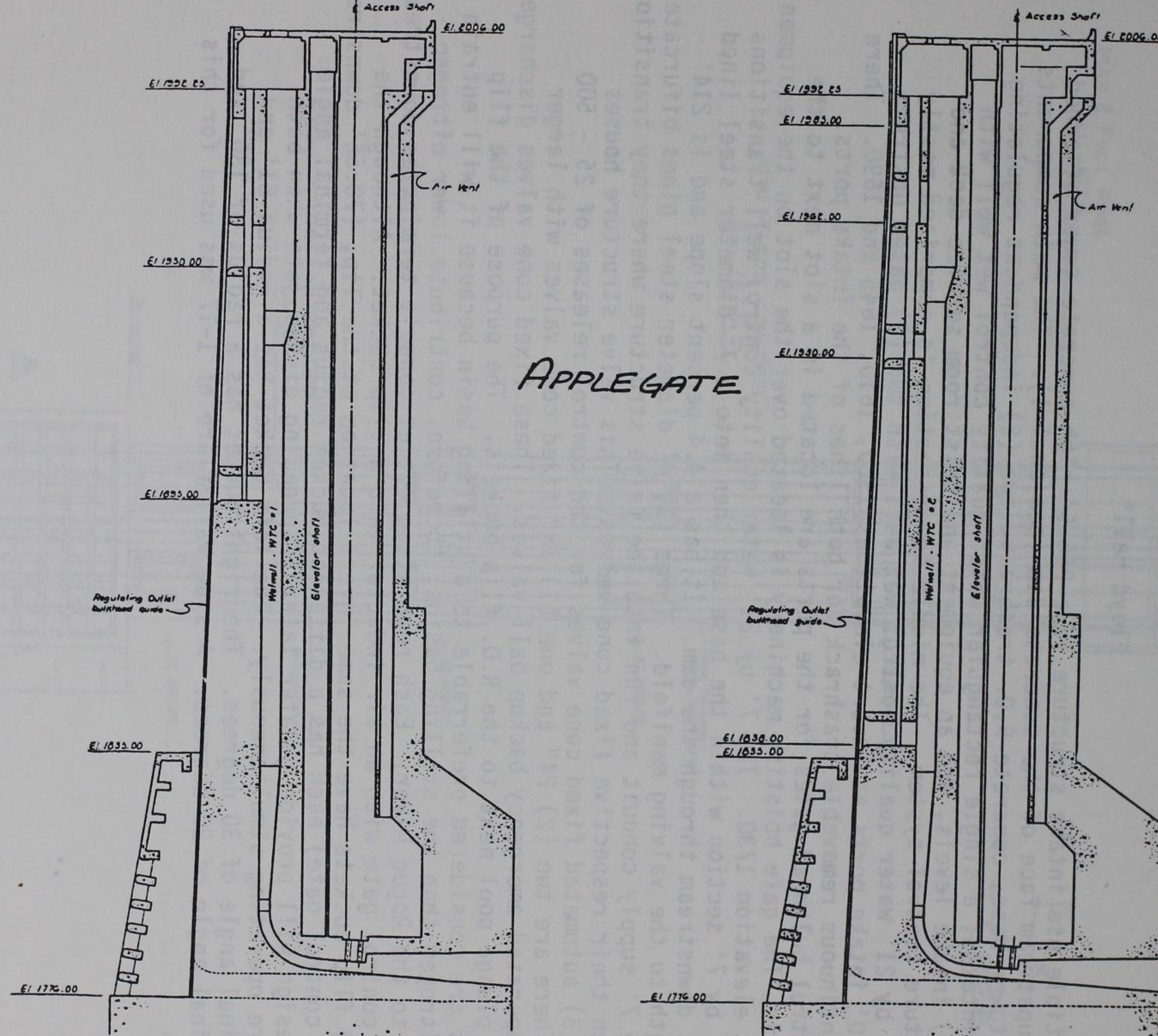
Access Shaft
El 2006.00
El 1992.25
El 1930.00
El 1855.00
Air Vent
Wetwell - WTC #1
Elevator shaft
Regulating Outlet bulkhead guide
El 1835.00
El 1776.00
APPLEGATE
Access Shaft
El 2006.00
El 1992.25
El 1985.00
El 1962.00
El 1930.00
Air Vent
Wetwell - WTC #2
Elevator shaft
Regulating Outlet bulkhead guide
El 1838.00
El 1835.00
El 1776.00

NPP

JAN 85

HYDRAULIC DESIGN OF THE SELECTIVE WITHDRAWAL STRUCTURES AT THE ELK CREEK DAM

Floyd Hall*

The multiple use intake structure is a 266.5' high concrete tower attached to the upstream face of the concrete dam (see encl-1). The main components of the tower base, concrete R.O. trashrack, two (2) independent regulating outlet intakes, a single rectangular water quality control wet well with four (4) intake levels, and an equipment and hoist room service deck and temperature control system. The middle tower section is comprised mostly by a 7' by 21' water quality control wet well with two (2) gated upstream 5' by 10' intake ports at invert elevations 1560, 1610, 1645 and 1690. There is a continuous removable trashrack for both lines of the intake ports. The control bulkhead gates for the ports are located in a slot next to the wet well. The gate hoisting machinery is located over the slot on the equipment deck at elevation 1730. The 7' by 21' water quality control well transitions to a 7' by 7' section within the base and then into a 7' diameter steel lined conduit downstream through the dam. It has a 4.4 percent slope and is 214' in length to the valving manifold. Three (3) 4' diameter steel pipes bifurcate off the 7' supply conduit and run into the valve structure where they transition to match their respective fixed cone valves. This valve structure houses three (3) automated fixed cone valves for the control releases of 25 - 500 cfs. There are two (2) 24" and one (1) 18" fixed cone valves with larger motor operated emergency backup ball valves. These fixed cone valves discharge into a plunge pool next to the R.O. flip buckets. The purpose of the flip buckets are considered preferrable to a stilling basin because it will entrain less nitrogen than the stilling basins, therefore, contribute lower nitrogen levels to the Rogue River. Each regulating outlet conduit terminates downstream of the control gate with an air inductor into a flip bucket. Although the two (2) flip buckets have the same invert location at the lip, (78.56' downstream of the control gate) each has a different bucket radius and terminal angle. This design will provide greater lateral spreading of the jets when both R.O. are operating simultaneously. The left bucket has a radius 81' and a terminal angle of 30 degrees. The right bucket has a radius of 180' and a terminal angle of 20 degrees. A side wall flare of 1-17 was used for this design.

*Chief, Hydraulic Design Portland District

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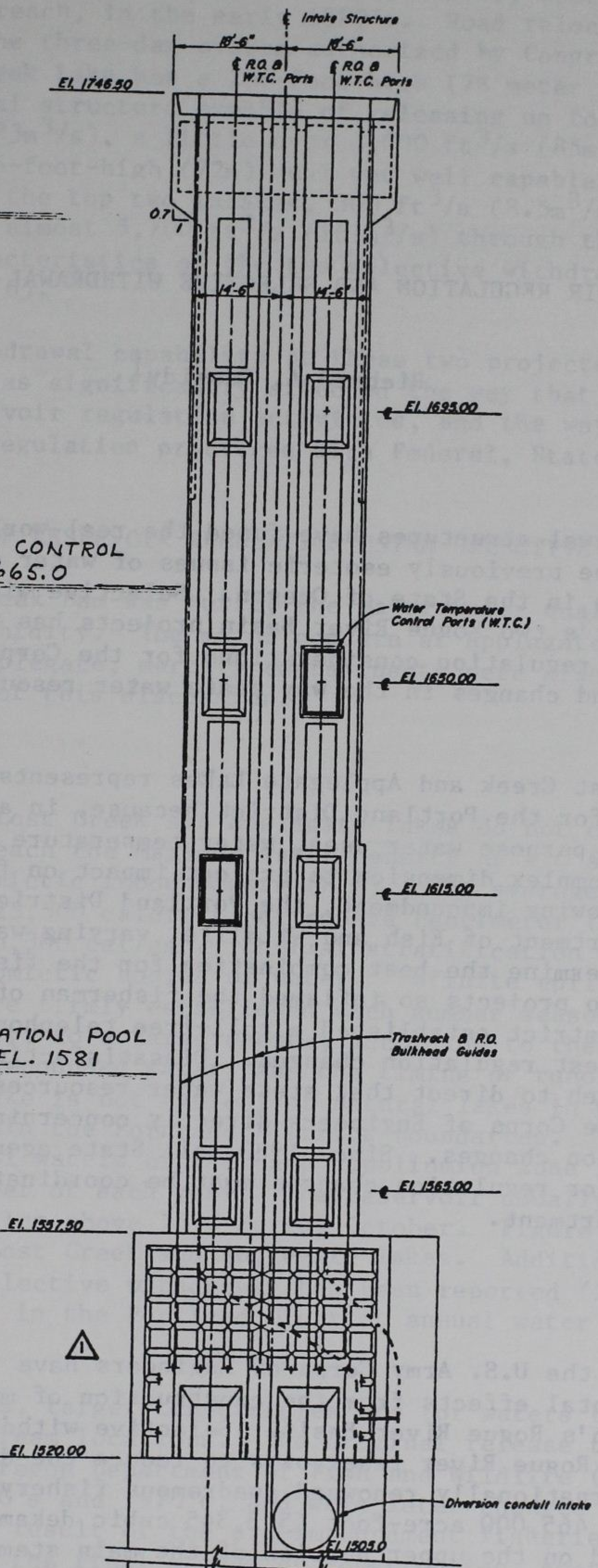
ELK CREEK

MAX & FULL POOL EL. 1726

MIN FLOOD CONTROL EL. 1665.0

MIN. CONSERVATION POOL EL. 1581



RESERVOIR REGULATION AND SELECTIVE WITHDRAWAL IN OREGON

Richard A. Cassidy[1]

ABSTRACT

Selective withdrawal structures have fused the real world considerations of water quantity with the previously esoteric issues of water quality to form dynamic water resource issues in the State of Oregon. Selective withdrawal capability in the Portland District's two Rogue River Basin projects has not only increased the day-to-day reservoir regulation considerations for the Corps of Engineers, but has also made profound changes in the way state water resources agencies consider release changes.

Regulation of Lost Creek and Applegate Lakes represents another level of water resource management for the Portland District because, in addition to managing conflicting multiple purpose water uses, water temperature and turbidity controls have added another complex dimension to project impact on fisheries. During the first few years following impoundment, the Portland District worked closely with the Oregon Department of Fish and Wildlife, varying water temperature and outflows to help determine the best combination for the fishery. Manipulation of releases from the two projects so inflamed the fisherman of the Rogue River Basin that the Portland District established a toll-free telephone service to inform fishermen of the latest regulation changes. Dissatisfaction ultimately led Governor Victor Atiyeh to direct that state water resources related agencies could no longer contact the Corps of Engineers directly concerning requests for unscheduled regulation changes. Since 1983, all State agency requests to the Corps of Engineers for regulation changes must be coordinated through the Oregon Water Resources Department.

INTRODUCTION

Innovations by the U.S. Army Corps of Engineers have reduced some negative long-term environmental effects from the construction of multiple purpose reservoirs in Oregon's Rogue River Basin. Selective withdrawal capability was incorporated in two Rogue River reservoirs to reduce the dams' effects on the river system's internationally renowned anadromous fishery, and on the riverine water quality. The 465,000 acre-foot (573,345 cubic dekameter [dm^3]) Lost Creek Lake was constructed on the upper portion of the main stem Rogue River during the mid-and-late 1970's. The 82,200 acre-foot (101,390 dm^3) Applegate Lake was

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constructed on a major tributary of the Rogue River, upstream of a designated "wild and scenic" reach, in the early 1980's. Road relocation for Elk Creek Lake, the third dam of the three-dam system authorized by Congress in 1962, is currently underway. Lost Creek Lake has a 256-foot-high (78 meter [m]) single wet well selective withdrawal structure capable of releasing up to 10,000 cubic feet per second (ft.^{3}/s) (283m^3/s), a little over 3,000 ft^3/s (85m^3/s) per level. Applegate Lake has a 236-foot-high (72m) dual wet well capable of releasing 500 ft^3/s (14.1m^3/s) through the top two intakes, 300 ft^3/s (8.5m^3/s) through the lower three intakes, and almost 5,700 ft^3/s (161m^3/s) through the regulation outlet. The hydraulic characteristics of the two selective withdrawal structures have been described by Hall (6).

Selective withdrawal capability at these two projects has worked well. That feature, however, has significantly affected the way that the North Pacific Division conducts reservoir regulation activities, and the way Portland District coordinates reservoir regulation practices with Federal, State, and County agencies, and the public.

TECHNICAL ASPECTS OF RESERVOIR REGULATION USING SELECTIVE WITHDRAWAL

Before Lost Creek Dam was built, the major water quality concerns were water temperature and turbidity. The major concern at Applegate Lake was mostly water temperature. At Applegate, mercury contaminants were also an issue, but that concern is not a part of this discussion.

Water Temperature

The waters of Lost Creek and Applegate lakes do not cool enough during the winter seasons to reach the maximum water density at 4°C and, therefore, are classified as warm monomictic impoundments (8,9). The average annual heat budget for Lost Creek Lake is 25,500 calories per square centimeter (cal/cm^2), that for Applegate Lake is 26,300 cal/cm^2. Thermal stratification in Lost Creek Lake is typical of warm monomictic bodies of water. Definite epilimnion, metalimnion, and hypolimnion zones are firmly established each summer season and the levels of water withdrawal vary, to a large degree, according to the amount of solar heating, the volume of impounded water, and the timing of runoff. Thermal stratification at Applegate Lake is not typical of natural lakes in western Oregon, or of many reservoirs within the Portland District boundaries. To meet target water temperatures,the cool waters of the deep hypolinmion zone in the reservoir are evacuated by September of each year. The reservoir usually exhibits nearly isothermal characteristics above 15°C during October. Figure 1 shows the withdrawal levels utilized at Lost Creek and Applegate lakes. Additional information on water quality and selective withdrawal has been reported (2, 5) and greater details are provided in the Portland District annual water quality reports (8,9).

At both projects, target temperatures are for waters being released from the dams, not for downstream locations. The original release target temperatures were recommended by the Oregon Department of Fish and Wildlife (ODFW). They were proposed during the 1960's and 1970's, and were later changed during construction of Lost Creek Lake as a result of the pre-impoundment fisheries studies. While Lost Creek Lake was under the final stages of construction, another set of target temperatures was recommended by ODFW as the preferred outflow water temperatures. The Lost Creek and Applegate selective withdrawal structures were designed to meet the original target temperatures of the 1960's and 1970's. Only the system's versatility allows the new target temperatures to be approached (2, 3). The Rogue River Basin target temperatures are specific only for a portion of the year and general the rest of the year. For instance, starting in April, the Lost Creek target water temperatures are based on the average weekly natural stream temperature. Beginning May 1, as warm an outflow as possible is desired until the outflow reaches 55°F (12.8°C). Water releases are then kept at 55°F throughout the summer season. On September 1, target water temperatures again become the average, weekly pre-impoundment stream temperatures.

Both the single wet well Lost Creek withdrawal system and the dual wet well Applegate withdrawal system adequately meet target release temperatures (2, 3, 5, 8, 9). Decisions on the use of water withdrawal levels are based on the latest reservoir water temperature profiles and real-time outflow temperature measurements. Mathematical modeling for operational use is not utilized. Target temperatures at Applegate Lake are exceeded during a portion of the fall season because there is not sufficient cool water in the small hypolimnion volume (9). The Applegate fall season target temperature goals have, therefore, been revised by the Oregon Department of Fish and Wildlife.

Turbidity

The Lost Creek lake selective withdrawal structure has been operational for almost 9 years. There have been only two high runoff events that have caused turbidity flows high enough to use the low level withdrawal conduit to evacuate the low level turbid water. The largest event in 1977 caused peak inflows of 66 JTU water to enter the impoundment. The second largest event occurred in 1980 and caused peak inflows of 30 JTU water. The low level withdrawal conduit, often called the elephant trunk because of its shape, has performed well on both occasions, evacuating most of the turbid water in about 30 to 45 days (1, 8).

MANAGEMENT ASPECTS OF RESERVOIR REGULATION USING SELECTIVE WITHDRAWAL

Corps Coordination

Reservoir regulation practices for the two Rogue River Basin reservoirs have evolved since completion of Lost Creek Lake in 1977 and Applegate Lake in 1981. At both projects, the Portland District was responsible for all pre-project water quality studies, plus all water quality studies performed during the filling process. Because there was a significant drought in 1977, Lost Creek Lake did not fill until the summer of 1978 (4,7). The Portland District was not scheduled to transfer regulation responsibility to the North Pacific Division Reservoir Control Center until the impoundment was filled for the first time. Consequently, for almost 2 years, the Portland District actively participated in the reservoir regulation aspects of on-going fishery research in the downstream Rogue River. In both 1977 and 1978, the research included experimenting with different flow and water temperature combinations. When the project filled and regulation was officially transferred, the Portland District requested a change from the North Pacific Division practice of regulating all Portland District reservoirs. Because of necessary close coordination with the ODFW and because of water quality issues associated with the new reservoir, the Portland District recommended that regulation responsiblity during the conservation release season (late spring through mid-fall) remain with the district. Approval was granted, with the

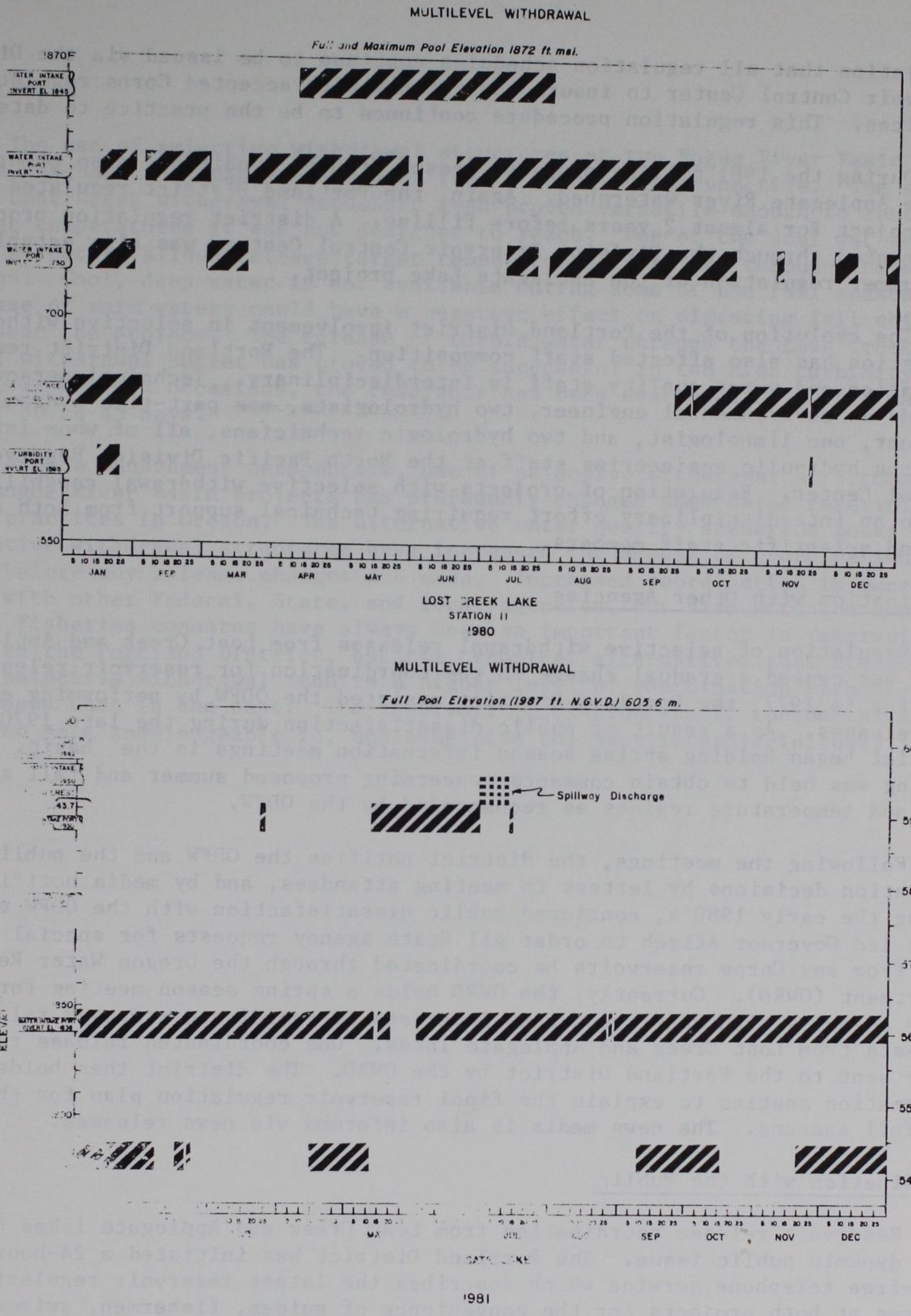

Figure 1. Examples of the withdrawal of water from selected levels at Lost Creek and Applegate Lakes, Oregon.

stipulation that all regulation schedules continue to be issued via the Division Reservoir Control Center to insure consistency with accepted Corps regulation practices. This regulation procedure continues to be the practice to date.

During the 1981 filling process, lesser drought conditions also occurred in the Applegate River watershed. Again, the Portland District regulated the new project for almost 2 years before filling. A district regulation procedure, implemented through the Division Reservoir Control Center, was also established for summer regulation of the Applegate Lake project.

The evolution of the Portland District involvement in selective withdrawal regulation has also affected staff composition. The Portland District reservoir regulation and water quality staff is interdisciplinary. Technical personnel include an environmental engineer, two hydrologists, one part-time hydraulic engineer, one limnologist, and two hydrologic technicians, all of whom interact with the hydraulic engineering staff at the North Pacific Division Reservoir Control Center. Regulation of projects with selective withdrawal capability has become an interdisciplinary effort requiring technical support from both engineering and scientific staff members.

Coordination with Other Agencies

Regulation of selective withdrawal releases from Lost Creek and Applegate lakes has caused a gradual change in the coordination for reservoir releases. Starting in 1977, the Portland District assisted the ODFW by performing experimental releases. As a result of public dissatisfaction during the late 1970's, the district began holding spring season information meetings in the basin. The meeting was held to obtain comments concerning proposed summer and fall season flow and temperature regimes as recommended by the ODFW.

Following the meetings, the district notifies the ODFW and the public of its regulation decisions by letters to meeting attendees, and by media notification. During the early 1980's, continued public dissatisfaction with the ODFW experiments led Governor Atiyeh to order all State agency requests for special regulation from any Corps reservoirs be coordinated through the Oregon Water Resources Department (OWRD). Currently, the OWRD holds a spring season meeting for all State agencies with water resources interests to coordinate proposed release requests from Lost Creek and Applegate lakes. One coordinated release request is later sent to the Portland District by the OWRD. The district then holds a public information meeting to explain the final reservoir regulation plan for the summer and fall seasons. The news media is also informed via news releases.

Coordination with the Public

Reservoir release coordination from Lost Creek and Applegate lakes has been a very dynamic public issue. The Portland District has initiated a 24-hour-per-day toll-free telephone service which describes the latest reservoir regulation changes at both projects for the convenience of guides, fishermen, swimmers, boaters, and the general public using the Rogue River. Additionally, the traditional practice of media notification of regulation changes accompanies every major change. An estimated three to five non-flood related news releases concerning reservoir regulation changes in the Rogue River Basin are issued by the Portland District every year.

CONCLUSION

The use of selective withdrawal structures at two Rogue River Basin reservoirs has been reasonably successful from a technical perspective. The single wet well Lost Creek withdrawal structure even proved versatile enough to meet release target temperatures it was not designed to reach. Use of the dual wet well Applegate structure allows release target temperatures to be met through the summer season. Cool, deep water is not available during some of the fall season and the release of warm waters could have a negative effect on migrating fall chinook salmon near the project. The release of turbid water through the Lost Creek low level withdrawal outlet has proved to be successful in two high turbidity runoff events. On both occasions, the reservoir has been evacuated of turbid water in about 30 to 45 days.

From a management perspective, use of selective withdrawal structures at the Rogue River Basin projects has profoundly changed some Corps reservoir regulation practices in Oregon. The alternative water quality scenarios possible with selective withdrawal structures have increased the amount of coordination necessary before any release changes are made. Increased coordination is necessary not only with other Federal, State, and local agencies, but also with the general public. Fisheries concerns have always been an important factor in reservoir regulation in the Portland District, but the technical alternatives that are possible with selective withdrawal require a higher level of coordination with fisheries interests than in the past. The reservoir regulation staff, concomitantly, must be even more knowledgeable in, and sensitive to, many interdisciplinary issues.

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Operation of Selective Withdrawal Facilities
Libby Dam, Montana

James W. Helms

Introduction

The Kootenai River is a major tributary to the Columbia River. The river originates in Canada, flows through northwestern Montana and northeastern Idaho, and re-enters Canada, passes through Kootenay Lake, and eventually enters the Columbia River at Castlegar, British Columbia.

Construction of Libby Dam on the Kootenai River, near Libby, Montana, began in 1966. Pre-impoundment water quality studies initiated in 1967 predicted that annual loadings of nitrogen and phosophorus would be 4 to 10 times the value that could cause eutrophication of the proposed reservoir.

The original project design did not include a selective withdrawal facility. In recognition of the potential by serious downstream impacts of releases from an eutrophic reservoir, studies were initiated to incorporate such a system into the dam then under construction. Provisions were made to permit selective withdrawal through the turbine penstocks when the power units came on line.

Major dam construction was completed, and Lake Koocanusa was impounded on 21 March 1972. The selective withdrawal system became operational in 1977.

History of the Libby Dam Project

Libby Dam and its reservoir, Lake Koocanusa, were designed as a multipurpose project to provide flood control, power, and recreational benefits. As the project involves both upstream storage and downstream effects, not only in the United States, but in Canada, the project was built under an international treaty for the cooperative water resource development of the Columbia River Basin.

To monitor the effects of construction and operation of Libby Dam, the U.S. Army Corps of Engineers, the Federal Water Pollution Control Administration (now the Environmental Protection Agency), the U.S. Geological Survey and the Montana Departments of Health and Fish and Game cooperated in developing a water quality monitoring program both above and below the dam site.

Libby Dam Selective Withdrawal Structure

Libby Dam is located on the Kootenai River in northwestern Montana, 219 miles (350 km) upstream from the confluence of the Kootenai and Columbia Rivers and about 17 miles (27 km) upstream from the town of Libby, Montana. The dam is a concrete gravity structure rising some 427 ft. (130m) above bedrock with a top length of 3055 ft. (931m). The dam has two spillways with crests at elevations of 2405 ft. (733m) above sea level, three sluices with inverts at elevation 2200 ft. (671m), and eight penstocks (five currently in operation) with inverts at 2222 ft. (677m).

The selective withdrawal system consists of a series of vertical piers and horizontal bulkheads which permit the withdrawal of water from the reservoir at elevations ranging from 172 feet (52m) from the penstock invert to within about 20 ft. (6m) of the surface at full pool, normally, however, the 10-foot-high (3m) bulkheads are not stacked closer than 50 ft (15m) from the water surface.

The 22 bulkheads atop each of the five powerhouse intakes are stacked by gantry crane into the guide slots until the desired water is diverted over the top of the stacked bulkheads into the 20-foot diameter (6m) penstock intakes. The right- and left-hand sections of the structure operate separately with each section serving four penstocks.

Lake Koocanusa

Lake Koocanusa is a relatively large, long, and narrow reservoir with a wide range of volume and surface area. At full pool, the lake has a surface area of about 46,500 acres ($1.9x10^8 m^2$) and a volume of over 5,800,000 acre-feet (7.24 km^3). The lake has a mean depth of 126 ft. (38m) and a maximum depth of 350 ft. (107m) in the forebay. Normal pool fluctuation is about 129 ft. (39m) in response to the cyclic pattern of May-June inflow and winter drawdown. Lake Koocanusa generally has a maximum volume during the July through September conservation period and a minimum during the March-April flood control season.

The flood storage function of Lake Koocanusa, in conjunction with large seasonal inflows and withdrawals, results in wide fluctuations in the reservoir content. Mean annual retention time is short, ranging from about 0.20 to 0.68 years. As a consequence, the reservoir displays complex hydrodynamics resulting in a weak thermal structure that allows mixing deep into the water column.

Phytoplankton are circulated out of the euphotic zone diminishing their productivity. Woods and Falter (1982)(7) attribute this circulation as one of the key influences in restraining productivity in Lake Koocanusa. The nutrient loading model used by Bonde and Bush(1) to predict a potentially eutrophic reservoir did not account for the physical limitations imposed on the system.

Nutrient loadings have been reduced by cleanup procedures instigated in Canada in 1975, and concerns over eutrophication have largely been alleviated. Woods and Falter(7) reported that primary productivity of Lake Koocanusa from 1972 through 1975 averaged only 28.8g C m^{-2} $year^{-1}$. McMillan (1979)(4) ranked this value near the bottom of Wetzel's oligatrophic range and compared it with Goldmans classification of Lake Tahoe as ultraoligatrophic with a value of 55g C $m^{-2} year^{-1}$.

Kootenai River

Libby Dam has reversed the natural flow regime of the river below the dam. Historically, the highest flows had occurred from April through July with median peak flows of 60,000 cfs during May and June. Minimum discharges of about 2,000 cfs had occurred during the winter and early spring. Long term annual average discharge is 12,000 cfs. Downstream at its mouth, the annual discharge of the Kootenai River is 30,650 cfs making the Kootenai the second largest tributary to the Columbia, exceeded in volume only by the Snake River.

Since impoundment, low flows now occur during the refill period from April through July. During the remainder of the year, flows generally range from the operational minimum of 4,000 cfs to a maximum of 23,000 cfs in response to power demands. The daily flow regime, which was relatively stable under natural conditions, now fluctuates due to hydropower operations. Releases fluctuation are limited to four vertical feet a day from April through September and to six feet a day during the remainder of the year.

The change in temperature of the Kootenai River below Libby Dam has been the result of both the dampening affect of the large reservoir and also a function of the dam outlet through which water is released. The sluiceways were used almost exclusively from the initial impoundment in 1972 until the first power unit came on line in August of 1975 and penstock releases commenced. Prior to June 1977, the penstocks intercepted deep reservoir water and the release temperatures seldom exceeded 54°F (12°C) unless spillway releases were made. After that date, the selective withdrawal system was used to supply water to the penstocks.

May and Huston (1979)(3) reported that the impoundment of the Kootenai River in 1972 by Libby Dam not only altered the flow regimes, temperature patterns, sediment loads and water quality but greatly altered the aquatic environment which resulted in changes in periphyton, aquatic insects, and fish population. Periphyton biomass and productivity increased. Insect diversity decreased but the biomass of insects increased dramatically, being highest near the dam. Fish diversity decreased, but of the remaining population, rainbow trout and mountain whitefish increased in both numbers and size.

Selective Withdrawal Operation

The selective withdrawal system is operated to: (a) provide release waters of adequate dissolved oxygen; (b) prevent the release of toxic hydrogen sulfides; (c) preclude turbid releases from zones with high concentrations of dead and decaying algae; and (d) increase downstream temperatures in the summer months and reduce them in the winter--thereby duplicating, to the extent possible, the natural temperatures in the river prior to the dam.

The selective withdrawal system is typically placed into service in April with installation of the first series of bulkheads. Bulkheads are then placed as rapidly as possible while maintaining a 50-foot interval between the top of the highest bulkhead and the pool surface. Withdrawing water no closer than 50 feet from the pool surface reduces escapement of fish from the reservoir.

A rule curve developed to duplicate pre-impoundment river conditions was used in 1977 during the first year of selective withdrawal. A meeting of interested resource agencies in 1978 revealed that the selective withdrawal system could not only be operated to achieve pre-dam conditions, but with modification of the rule curve, be made to optimize trout production temperatures. To accomplish this objective, the pre-dam rule curve was modified to reach and maintain an optimum maximum temperature of 58°F (14.4°C) to be held as long as possible into the fall months. Bulkheads are removed as necessary to maintain temperatures below this value and to maintain the distance between the pool surface and the top bulkhead. This plan increases the number of degree days above 32° F by approximately 30%.

When the reservoir becomes isothermal--generally by the end of October--the bulkheads are removed so that discharge will come from deep in the reservoir during winter months. This prevents any buildup of poor quality hypolimnetic water.

Because the earlier concerns over eutrophication, algal blooms, depleted oxygen and hydrogen sulfide production failed to materialize, the selective withdrawal system is now being regulated to provide the best temperature pattern possible for the downstream Kootenai River without affecting the biota of Lake Koocanusa. Temperature control has significantly increased the total biomass carrying capacity, particularly aquatic insects and fish in the Kootenai River below Libby Dam (Huston, et al., 1983)(2). This increased biomass has resulted in more fish for anglers.

Impacts on Aquatic Insects

Altering the flow regime, temperature patterns, sediment loads, and water quality of the Kootenai River has changed the composition of the aquatic insect population from a stonefly, mayfly, caddisfly dipteran complex to one dominated by a few mayfly and dipteran taxa(2).

It is recognized that a seasonal temperature cycle is essential for the maintenance of most aquatic communities and that many insects have strict temperature requirements, alterations of which can have drastic effects. Constant temperatures may eliminate species which depend upon a temperature maxima or minima to break diapause or stimulate hatching, growth, or emergence. Insect life histories are often dependent upon temperature summation data. Species for which the number of degree days is inadequate may not reach maturity and may consequently be eliminated.

The degree-day concept has been used by many researchers to study insect life histories. Mean daily temperatures can be summed for a given period of time (week, month, season, or year) to provide a relative comparison of the cumulative heat load or energy input for different years. Assuming pre-impoundment conditions as 100%, a significant increased energy input can be shown for post-impoundment releases and another increase with selective withdrawal operations.

Impacts on Fish

Following impoundment in 1972, until the turbines came on line in 1975, water was released from Libby Dam by way of the sluices and spillways. This mode of release caused gas supersaturation that may have limited both insect and fish populations below the dam. After 1975 water was released primarily through the penstocks and rainbow trout and mountain whitefish populations increased by over 300 percent (3).

The marked improvement in rainbow trout and mountain whitefish population downstream of Libby Dam has been attributed to (2) the interactions of several environmental factors including:improved water temperatures for trout growth; reduction in sediment loads; higher flows from August through March; and pollution abatement in Canada.

Decreased competition for the available food sources may also have contributed to the dominance of these two species. May and Huston (3) reported that cutthroat trout populations have declined markedly since 1975, indicating that few have escaped from the reservoir since turbine operation in 1975 and selective withdrawal system implementation in 1977.

Peamouth and squawfish have shown a similar population decline. This has been attributed to low water temperatures in the spring and early summer adversely affecting reproductive success, since these species spawn when water temperatures approach 55°F.(3)

Water temperatures similarly impact rainbow trout and numerous studies have been made on their temperature preference. May and Huston (3) summarized these studies which showed that temperature preference was directly related to the age of the fish and its recent thermal acclimation. These studies showed: (a.) the preferred temperature of a 15 month old rainbow trout was 52°F; (b.) rainbow fingerlings preferred 63°F which was also the temperature of maximum growth for juvenile rainbow with excess feed; and (c.) rainbow trout over one year old grow best when temperatures are about 50-54°F.

Rainbow trout are primarily draft feeders generally selecting food items from the water column or from the water surface while mountain whitefish are much more substrate oriented (3). Although rainbow trout of all sizes generally feed on a wider variety of food than mountain whitefish, there is a considerable overlap in the food habits of these two species, most evident for fish less than 8 inches (20cm) in length and generally confined to their utilization of chironomidae (3).

State of Montana fishery studies as reported by Huston, et al. (2), show from trend estimates that rainbow trout numbers increased markedly for two years after impoundment, leveled out through 1977, and again increased markedly in 1978 and 1979 following implementation of the selective withdrawal system. Growth rates of fish decreased slightly during this latter period but were still generally above those representing pre-impoundment conditions. Two-year-old fish averaged 9.4 inches (24cm) in 1972, 12 inches (30cm) in 1977, and 10.2 inches (26cm) in 1978 (2). The Kootenai River below Libby Dam is currently classified as one of the most productive trout streams in Montana and a "blue ribbon fishing stream".

As a result, fishing pressure has increased significantly on the free-flowing section of the Kootenai River since 1968 when there was an estimated at 116 angler-days per mile to the 1978 level of 1600 per mile, (2). People desire to fish when the flow in the Kootenai River is below 8000 cfs. Prior to impoundment, flow in the river was below 8000 cfs on an average of 43 days during the April through September season. After impoundment, fishing opportunities during this period increased to 94 days, an increase of 51 days.

Creel census data indicate that rainbow trout compromised 49 percent of the catch in 1975, 92 percent in 1977, and 95 percent in 1978. Cutthroat trout dropped from 51 percent in 1975 to only 4 percent in 1978. The catch rate of .64 fish per hour and the average size of the rainbow trout creeled at 11.4 inches rank the Kootenai River as one of the better wild trout fisheries in Montana. The largest fish caught was an 18-pound, 1-ounce rainbow caught in 1977 near the mouth of the Fisher River, about 3 miles below the dam.

Summary

Construction of Libby Dam and the impoundment of the Kootenai River has significantly changed the riverine environment below the project. The interaction of many environmental components has produced conditions which have been favorable for rainbow trout. Operating the selective withdrawal system to stimulate biological activity by increasing the heat budget while maintaining preferred release water temperatures, optimizes conditions for this species. This preferential enhancement has resulted in a very productive fishery providing excellent angling for sports fishermen both in the reservoir and in the free-flowing segment of the Kootenai River.

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Alternatives for Improving Reservior Water Quality

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Low dissolved oxygen (DO) concentrations are a common characteristic of waters released from hydroelectric powerplants which draw from deep reservoirs. The dissolved oxygen deficiency in the hypolimnion results from normal biochemical reactions which gradually consume oxygen from deep waters trapped by seasonal thermal stratification. The deficiency becomes most critical in the fall shortly before the reservoir destratifies due to natural cooling. The Corps of Engineers, as well as other Federal and some private agencies, have attempted numerous corrective measures at various reservoirs. None can assure both consistent success and cost-effectiveness. Thus, solutions must be developed on a case-by-case basis.

Since 1970, the Corps of Engineers Little Rock District has tested a number of alternatives to solve the dissolved oxygen problem at Table Rock Lake. The Table Rock dam and powerhouse facilities are located at mile 528.8 of the White River in Southwest Missouri. The dam is approximately 6 miles southwest of the town of Branson, Missouri. The Flood Control Act of 1941 authorized the construction of Table Rock Dam as a multipurpose facility for power production, flood control and other beneficial purposes. Recreation on Table Rock Lake and trout fishing in its tailwater Lake Taneycomo have contributed to the development of a tourism-based economy in Branson.

In the interim, a large number of studies have been conducted to better characterize factors affecting the problem and to evaluate possible solutions. The objective of the studies was to develop an alternative which would maintain a minimum of 6.0 mg/l dissolved oxygen and not exceed 68°F temperature in order to meet the Missouri state water quality standard for Lake Taneycomo.

Another objective of the studies was to develop an alternative which would maximize the net economic benefits. A relationship between dissolved oxygen concentrations and angler success was used to determine the economic benefits. The benefits varied depending upon the DO concentration which an alternative could attain.

The alternatives considered usually involved costs such as: investment costs, operating costs and forgone revenues from power generation. Almost all of the alternatives considered would have an impact on either generating capacity or efficiency. Some impact

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both. The turbine rating curves were used along with "typical" generating patterns to estimate the impact of an alternative on forgone revenues from power generation.

Generating conditions at Table Rock Dam have a substantial impact on dissolved oxygen concentrations in downstream Lake Taneycomo. The Table Rock powerhouse is operated primarily for peaking power generation, thus hydropower releases are usually largest during the daytime and on weekdays. Minimum dissolved oxygen concentrations can occur during both maximum and minimum release conditions. When maximum hydropower releases are made, there is little or no air aspiration through the turbines and substandard dissolved oxygen concentrations can result when Table Rock Lake is stratified. Minimum dissolved oxygen concentrations also occur when the main turbine units are shut down. Under nongenerating conditions, water released to the lake consists of about 95 cfs leakage past the main turbine units and approximately 20 cfs of releases from house unit generation. Many of the alternatives considered for improving dissolved oxygen concentrations are effective only during generating conditions.

Dissolved oxygen concentrations in Table Rock Lake vary with depth and with time . Periodic measurements of D.O. concentrations at various depths at a location about 1,000 feet upstream from the powerhouse have been recorded. From this type of data and from a penstock withdrawal zone pattern, determined by field measurements , the DO concentrations of powerhouse withdrawals throughout several seasons have been calculated. Seasonal concentrations are not consistent year by year, but appear to vary with weather conditions and other factors. A dissolved oxygen deficiency design curve was derived from the minimum concentrations observed. This design curve was used to determine the degree of improvement which would result from implementation of some of the alternatives, when appropriate.

The alternatives to increase the dissolved oxygen content of hydropower releases while concurrently complying with temperature standards consist of either blending high DO warm epilimnetic water with cool hypolimnetic water or by adding oxygen to the hypolimnetic water. The oxygen added can be obtained using either atmospheric oxygen or molecular oxygen. The alternatives considered were divided into three basic types:

1. Reaction with Molecular Oxygen.
2. Reaction with Atmospheric Oxygen.
3. Blending Alternatives.

The successful solution of the problem is complicated by the magnitude of flow rates of the turbine discharge. The 16,000 cfs upper design limit is equivalent to nearly 7.2 million gallons per minute. This is far in excess the capacity of commercially available industrial equipment for mixing or adding oxygen for water treatment.

The alternatives for adding oxygen to the hypolimnetic water were broken into two major categories depending upon the source of the oxygen. Both types of alternatives are subject to the same physical laws which affect mass transfer. An equation can be derived from Fick's law which states that the oxygen transfer efficiency decreases exponentially as the saturation concentration is approached. A series of curves can be derived based upon the DO deficiency design curve and the oxygen solubility curve. These curves can be used to evaluate the effectiveness of the alternative in meeting the minimum dissolved oxygen requirements.

Alternatives which use molecular oxygen can be expected to have much higher oxygen absorption efficiencies than those which use air. The actual absorption efficiencies will vary with the process conditions and the size of the upstream deficit. Further testing prior to design and installation is essential since absorption efficiencies are a major factor in the cost of molecular oxygen processes. For expediency, the oxygen requirements for the alternatives were based on estimated absorption efficiencies. The four alternatives for reacting hypolimnetic water with molecular oxygen are shown below in Table 1.

Table 1

Alternatives Utilizing Molecular Oxygen

Alternative	Description
1. Lake Oxygen Injection	Oxygen injected into hypolimnion upstream of dam, similar to the insulation at the Richard B. Russell Project in Savannah District. Cost exceeds benefits.
2. Powerhouse Oxygen Injection	Limited testing in 1973 indicated that 4.0 mg/l DO can definitely be maintained but it is not certain whether 6.0 mg/l DO can be attained. This alternative may be more cost-effective in combination with another alternative.
3. Sidestream Oxygen	A portion of the flow is withdrawn, pressurized, and oxygen is injected. The oxygen saturated sidestream is then dispersed into the main flow. Cost greatly exceeds benefit.
4. House Unit Oxygen Injection	Theoretically impractical. Even if the house unit discharge could be increased to near saturation levels, there would be little improvement in the receiving stream due to the nearly fivefold dilution by low DO leakage from the main turbine units.

In accordance with Dalton's Law of Partial Pressures, any alternative which utilizes air to increase dissolved oxygen levels will also result in increased dissolved nitrogen concentrations. Since the total amount of gas which can be dissolved in water under given conditions is relatively constant, the presence of nitrogen depresses the solubility of oxygen. Because of these effects, the use of air to increase dissolved oxygen concentrations is less efficient than molecular oxygen. Any alternative which utilizes air to increase dissolved oxygen levels has the potential to cause nitrogen supersaturation.

The alternatives which utilize atmospheric oxygen to increase dissolved oxygen concentrations fall into the following categories:

1. Inducing air flow into the water stream by creating a relative vacuum at the discharge side of the turbine wheel. The amount of air flow that can be induced is affected by tailwater levels and, thus, by generating levels.

2. Structural modifications to enhance the naturally occurring reaeration process.

3. Injecting pressurized air into the water via a mechanical device. The alternatives for reacting hypolimnetic water with atmospheric oxygen are shown in Table 2.

Table 2

Alternatives Utilizing Atmospheric Oxygen

Alternative	Description
1. Limited Capacity Operation	Currently used to induce air aspiration during generation. Effectiveness varies with generating level. Can meet 4.0 mg/l DO minimum, when sufficiently restricted.
2. Turbine Vent Modifications	The success of turbine vent modifications varies depending upon the turbine design. Modifications would probably not be practical for the Table Rock turbines due to limitations in the venting system.
3. Turbine Design Changes	Costs make retrofitting impractical for an existing installation.
4. Tailwater Stage Reduction	Removing the flashboards at Ozark Beach Dam would result in only a slight increase in natural reaeration through Lake Taneycomo.

Table 2 (Con.)

Alternatives Utilizing Atmospheric Oxygen

Alternative	Description
5. Stepped Reaeration Weir	It would be difficult to design a structure which would result in high dissolved oxygen uptakes due to the nappe thickness and submerged tailwater conditions. The head loss would result in an energy cost greater than the annual benefit.
6. Baffle Block Reaeration	Oxygen uptake is proportional to energy dissipation. Because there is very little energy left to dissipate at the discharge side of the turbine, it is unlikely that a substantial improvement in DO concentrations would occur.
7. Lake Aeration	Air injection into the hypolimnion of Table Rock Lake was tested in 1971. There was only a small improvement in dissolved oxygen concentrations.
8. Powerhouse Air Injection	Limited tests at Table Rock in 1971 and 1972. Can probably meet 4.0 mg/l minimum. Additional testing required to determine aeration effectiveness and efficiency losses over the operating range.
9. Draft Tube Venting Ring	High tailwater levels will inhibit air aspiration. Effectiveness varies with operating level.
10. Downstream Channel Aerators	Unsightly, high maintenance, noisy, high power consumption, high capital cost. Will not practically exceed 4.0 mg/l DO.
11. Downstream Channel Diffusers	It is economically impractical to inject air into Lake Taneycomo due to the low absorption rates in shallow water.

A number of alternatives were considered which involve blending warm, high DO water from the surface with cold, oxygen-deficient hypolimnetic water. The blending alternatives fall into three categories: changing inflows to Table Rock Reservoir; partial or complete upstream lake destratification; and selective withdrawal methods. The blending alternatives are described in Table 3.

Table 3

Blending Alternatives

Alternative	Description
1. Reservoir Destratification	Would result in meeting the DO standard, but would exceed the temperature standard.
2. Upstream Submerged Weir	No suitable withdrawal mixture could be found in the region of the thermocline which would meet both the temperature and dissolved oxygen standards.
3. Selective Withdrawal Structure	Preliminary mathematical model results indicate that this alternative could meet both the temperature and DO standards. Additional model testing required prior to implementation.
4. Modified Trash Rack Withdrawal	Blinds inserted into the trash rack could not sufficiently shift the withdrawal zone in this instance.
5. Supplemental Lake Releases	Spillway or sluiceway releases reaerate rapidly as energy is dissipated but result in large amounts of hydropower generation foregone.
6. Change Release Patterns	Would result in only a modest improvement in water quality since upstream Beaver Dam discharges from a hypolimnion and other inflows would not change. Mathematical and physical modeling needed.
7. Localized Mixing	At high flow rates, it would be impractical to pump epilimnion water to the penstock intakes.

The WQRRS model was used to simulate a selective withdrawal procedure at Table Rock Dam. The model primarily used water from the epilimnion. Water from the hypolimnion was used to adjust the temperature. The results indicated that a selective withdrawal procedure can meet both the temperature and dissolved oxygen standards. Selective withdrawal structures are the only alternative evaluated which can meet both the temperature and dissolved oxygen standards for downstream Lake Taneycomo. Supplemental math and physical model studies are necessary to further develop the withdrawal structure design, and to determine the range of withdrawal elevations required, and to evaluate the effects of the changed water quality on the downstream trout fishery.

A REVIEW OF SELECTIVE WITHDRAWAL PERFORMANCE IN THE FORT WORTH DISTRICT

Ronald L. Turner*

The Fort Worth District has twenty-two lakes existing at the present time, with two more under construction and one additional lake in the design stage. Geographically, these lakes are spread across the District, with average annual rainfall on the drainage basins varying from less than 20 inches in the western part of Texas to more than 50 inches in the eastern part of the state. These lakes were constructed over a time span of 37 years, with the earliest deliberate impoundment date in 1948. Table 1 is a chronological listing of the lakes by deliberate impoundment date. The depth of the conservation pool above the flood control intake invert elevation is shown in column 4, and the depth to the sill elevation of the various selective withdrawal gates for those structures which have selective withdrawal capability, is shown in column 5. The chronological listing readily divides the construction of the lakes into three periods. These are the 1950's, the 1960's, and the 1980's. Of the ten lakes constructed in the 1950's period, four had higher level withdrawal capability. Two of these could be considered to have true selective withdrawal capability, and the other two have only one elevation at which water can be drawn into the selective withdrawal wet well. Of the eight lakes constructed in the 1960's period, only Lake Waco was provided with any selective withdrawal capability. Of the lakes constructed in the period since 1980, all but one has or will have selective withdrawal capability. A review of the Design Memoranda for the structures constructed during the earlier period indicates that the decision to provide selective withdrawal capability was made by the water user, rather than the Corps of Engineers. The reports for the three structures with true selective capability all indicate that the cities which participated with the Corps as the local sponsor of the project had not only requested that selective withdrawal be furnished, but had also provided the Corps with the sill elevations of the selective withdrawal gates.

The earliest intake structures with true selective withdrawal capability were those at Lake Grapevine and Lake Lewisville. These lakes, located north and upstream of Dallas and used by Dallas as a water supply source, were constructed in the late 1940's and early 1950's. The deliberate impoundment date for Lake Grapevine was in 1952, and for Lake Lewisville, 1953, so a substantial period of record exists for both. The other two lakes built during the 1950's time period and provided with some selective withdrawal capability were Lake Belton, located in central Texas, and Lake Benbrook, located upstream of and southwest of Fort Worth. Each of these two lakes have a single elevation at which water can be withdrawn from the lake into the low flow wet well. Neither indicated in the DM that a request had been made that selective withdrawal capability be provided. A more detailed discussion of several of the District's lakes will provide a representative indication of the performance of the withdrawal systems in the District.

Lake Lewisville, which has one of the longest periods of record, began deliberate impoundment in 1953, but because of a drouth in Texas at the time, did not actually fill up until

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TABLE 1

LAKES OF THE FORT WORTH DISTRICT

1	2	3	4	5
NO.	NAME	IMPOUNDMENT DATE	CONSERVATION POOL DEPTH, FT.	DEPTH TO S.W. GATE SILL, FT.
1	HORDS CREEK	1948	44	NOTE 1
2	TOWN BLUFF	1951	29	
3	WHITNEY	1951	84	
4	O.C. FISHER	1952	68	
5	GRAPEVINE	1952	60	15, 22.5, 34.5, 34.5
6	BENBROOK	1952	72	38
7	LAVON	1953	39	
8	LEWISVILLE	1953	67	12, 19, 34, 34
9	BELTON	1954	110	54
10	WRIGHT PATMAN	1956	27	
11	LAKE O' THE PINES	1957	29	
12	NAVARRO MILLS	1963	25.5	
13	PROCTOR	1963	34	
14	CANYON	1964	134	
15	WACO	1965	55	10, 14, 31, 47
16	SAM RAYBURN	1965	59	
17	BARDWELL	1965	30	
18	SOMERVILLE	1967	32	
19	STILLHOUSE HOLLOW	1968	107	
20	GRANGER	1980	55	8, 14, 20, 26
21	GEORGETOWN	1980	71	14, 28, 42, 56
22	AQUILLA	1983	35	
23	JOE POOL	1985 (NOTE 2)	56	9, 18, 27, 36
24	RAY ROBERTS	1987 (NOTE 2)	81.5	14.5,29.5,44.5,58.5
25	COOPER	1991 (NOTE 2)	42	20.6, 30.6

NOTES:

1. Structures having selective withdrawal capability are indicated by entries in colunm 5. The various entries indicate depths from the conservation normal pool to the gate sill elevations for the various selective intake gates.

2. These projects under construction or in design. Dates indicate scheduled deliberate impoundment dates.

1957. The normal conservation pool depth is 67 feet. The outlet works consists of an intake tower with three 6.5 X 13 feet flood control gates emptying into a sixteen feet diameter conduit. The low flow capability is furnished by two wet wells, each emptying into a separate 5-feet diameter conduits with butterfly gates on the downstream end. These conduits are connected on the downstream end with a cross-over pipe. Selective intake gates are provided to the wet wells, with each having a gate with sill at a depth of 34 feet, and each having a gate with a higher elevation . The right side wet well selector gate sill is at a depth of 19 feet; the left at 12 feet. The project is used to supply water to the city of Dallas, and the water is passed through the outlet works structure to a pump station located downstream. The project personnel reported no occurrances of water quality problems from discharges through the low flow conduits during the time they had been there. The project also furnishes water to a fish hatchery located immediately downstream of the dam. The hatchery gets its water from the cross-over pipe at the downstream end of the twin five-feet diameter conduits. The project personnel indicated that no water quality problems had been reported from the hatchery. This would indicate that the water delivered through the selective system had furnished water which had a satisfactory oxygen content, since the hatchery takes its water upstream of the reaeration furnished by the stilling basin. However, during tests conducted by project and district personnel a few years ago to evaluate the source of gate vibrations in the low flow facilities, the lower gates (depth 34 feet) were opened and did deliver water with significant hydrogen sulfide odors.

Lake Belton was constructed in the early 1950's, with deliberate impoundment in March 1954. The normal conservation pool depth is 110 feet. The outlet works consists of an intake tower with three 7X22-feet flood control gates and a 22-feet diameter conduit. Low flow capability is provided by a single wet well with the intake gate sill at elevation 540, 54 feet below the normal conservation pool elevation. The wet well empties into the conduit from the side, directly behind the flood control gates. Because of the spray and turbulence provided at this entry point, considerable aeration of the flow takes place. Belton also passes water downstream of the stilling basin for use by the city of Belton. Poor water quality would be noticed by the user and reported to project personnel. The project manager, who has been there many years, indicated that they had no quality problems as long as they were releasing through the low flow system. He did report, however, that water released through the flood control gates at the 110-feet depth , as well as the water which leaked through the flood control gates contained significant hydrogen sulfide odor. The sulfur in the water is also believed to support bacterial growth which attacked and weakened the concrete in the conduit and stilling basin. The investigation into this possibility is continuing.

Canyon and Stillhouse Hollow Lakes will be considered together because of their many similarities. Both are located in hilly country and have narrow valleys relative to depth when compared with other Texas lakes. Canyon has a conservation pool depth of 134 feet, land Sstillhouse Hollow has a conservation pool depth of 107 feet. Canyon has a conduit diameter of 10 feet; Stillhouse Hollow 12 feet. Neither lake has any selective withdrawal capability. Canyon has a "put and take" trout fishery during the colder months, taking advantage of the colder water available at that depth. Both project managers indicated that no water quality problems from releases through the lake have been experienced. Canyon is currently under design by the river authority for an add-on hydropower plant. Its capacity will be about 250 cfs, and it will be a run of the river plant. Concern has been expressed to the project manager by the state fish and wildlife agency that adding the hydropower, and losing the effect of reaeration as the flows pass through the stilling basin will cause dissolved oxygen deficiencies in releases made through the power plant.

Waco Lake, located in the central Texas plains, was constructed in the early 1960's with deliberate impoundment in 1965. At the request of the city of Waco, the outlet works structure was equipped with selective withdrawal capability. The depth of the conservation pool is 55 feet; selective withdrawal gate sills are located at depth of 10, 14, 31, and 47 feet. The structure is constructed very similar to that at Lewisville, with the city water intakes located on the cross-over pipe upstream of the downstream gates. The city has experienced water quality problems for several years, and has added an aeration device immediately upstream of the intake structure. The quality problem they are experiencing are due to foul taste and odor of the water, and come from the quality of water in the lake. They have also added an aeration device in the river at the headwaters of the lake in an attempt to decrease the poor quality effects. The selective withdrawal capability has permitted them to withdraw water from different elevations in an attempt to find strata with less effects of the taste and odor.

Sam Rayburn Lake is located on the Angelina River in east Texas, with a drainage basin located in the 45 to 50-inch rainfall area of Texas. The Dam was constructed in the early 1960's with deliberate impoundment in 1965, giving about 20 years of record. The outlet works consists of two gate controlled conduits with dimensions of 10 X 20 feet, with invert elevation at a depth of 59 feet below the top of normal pool. The dam has hydropower generation capability, with 2-26,000 KW generating units. The majority of the water passed through the dam is utilized in generating, so that the flood control conduits are used infrequently. The dam has no selective withdrawal capability. Historical water quality data collected from the Angelina River below Sam Rayburn Dam indicate that dissolved oxygen violations of the state water quality standards have occurred since 1972. It has been concluded based on these observations that the violations were caused by the oxygen depleted water being released from the hypolimnion of Sam Rayburn Lake. Separate studies by the Fort Worth District, the local river authority, and the U. S. Geological Survey of the river downstream from the dam confirmed this conclusion. The state standard for this reach of the river is 5 mg/l. The data indicate that D. O. concentrations below the standard commonly occur, with occasional readings below 2.0. Tests in the lake during the same period, the months of July and August when the problems are most severe, indicate that the lake was strongly stratified at about a depth of 35 feet, with the D. O. at greater depths essentially zero. Various alternatives were considered to control the releases of water low in dissolved oxygen from the lake. These included destratification and aeration within the lake; structural modification to the outlet facilities; modification to the turbines; installation of downstream aeration devises; and operational procedures which might relieve the problem. Turbine venting tests were conducted in the field. The tests experienced some success at low flow rates, but were unsuccessful at flows above about 2000 cfs. The inability to vent at higher discharges was related to the design of the turbines. Since the turbines are seldom operated at flow rates of less than 4400 cfs, the venting alternative was not considered viable. The field data collected during these tests indicated that levels of D. O. below about 3 occurred for short time periods (four to six minutes) after generating began, with the D. O. in the river quickly recovering to a level greater than four for the duration of the generating cycle. A review of historical data indicated that no fish kills had occurred as a result of the low D. O. levels; in fact the river downstream of the dam supports an excellent fishery, with a gratifying diversity in the fish population. The expensive costs associated with the dubious abililty of proposed solutions to provide positive improvement in the condition of the river downstream led to the conclusion that modifications would not be constructed at that time.

Ray Roberts Lake is located north of Dallas and upstream of Lake Lewisville on the Elm Fork of the Trinity River. It is currently under construction, with a completion date set for 1987. The intake structure has two 6.5 X 13 feet gates discharging into a 13-feet diameter conduit. The invert elevation of the conduit is at a depth of 81.5 feet below the conservation pool normal water surface. The intake structure has a single wet well with 4 selector gates, with sill elevations at depths of 14.5, 29.5, 44.5, and 58.5 feet below the normal pool elevation. The wet well empties into a single 60-inch diameter conduit located underneath the flood control conduit, first passing through the center pier as a rectangular conduit. The 60-inch conduit continues underneath the flood control conduit to a point just upstream of the stilling basin headwall, where it turns to the right. It passes out from underneath the conduit to a valve box, which is provided with the capability of delivering the discharge back into the stilling basin, or on to a future hydropower plant. Ray Roberts Lake was recognized as having potential for the addition of a small hydropower plant during the design of the project. The city of Denton made application for the hydropower license during design, and requested the Fort Worth District to provide capability for addition of hydropower at a later date. The hydropower plant will be a continuous generation plant with a capacity of about 250 cfs. The water use for the lake will be by Dallas and Denton, and will be picked up downstream of the dam, so that the yield of the lake will be available for hydropower generation. In the water-short areas of the state it is almost a necessity that the water use be downstream for hydropower to be feasible. The flows for generation will be passed through the low flow selector gates to avoid the type of water quality problems experienced at Sam Rayburn. The size of the selector gates were increased during the design of the hydropower capability, to reduce gate and trash rack losses so that they would be available for the purpose of hydropower. The valve box directs low flows not used for generation back into the stilling basin through the right stilling basin wall. By considering water quality, low flow flood control releases, and hydropower needs concurrently in the design, a facility was provided which should adequately meet the needs of all the purposes.

This review of the lakes in the Fort Worth District leads to the conclusion that the selective withdrawal facilities on those lakes for which they were provided, have justified the cost of their construction. Several of the lakes which have no facilities for selective withdrawal have not been discussed individually. The project managers on those lakes which were interviewed indicated no problems with poor water quality from those lakes, except most did agree that some odor could be detected in the late fall when the lake turned over. Therefore, it would seem that the decision not to include the selective withdrawal facilities of the shallower lakes would also be justified. Aquilla lake, the only shallow lake for which a temperature model was used in the design, was not equipped with selective withdrawal facilities because the study indicated that the lake could be successfully operated without them.

MODELING OF SELECTIVE WITHDRAWAL INTAKE STRUCTURES

Chandra Alloju* M. ASCE

Introduction

A thermal simulation model[1] was used in the design of selective withdrawal structure at four projects in the Fort Worth District. Two of these projects are Georgetown and Granger Lakes, which were built in 1980. The two other projects with selective withdrawal capability, Joe Pool and Ray Roberts Lakes, are currently under construction. In this presentation an attempt will be made to describe how the thermal simulation model was applied in the design of selective withdrawal facility at Joe Pool Lake.

Project Description

Joe Pool Lake is located at river mile 11.2 on Mountain Creek, a tributary to West Fork of the Trinity River, about 10 miles Southwest of the city of Dallas. This project is authorized as a multipurpose project for flood control, water conservation, recreation, and fish and wildlife enhancement. The drainage area above the dam site is 232 square miles. The lake is formed by a rolled earth fill embankment, rising about 108.5 feet above the streambed, will have a maximum depth of 66 feet at conservation pool, a surface area of 7,470 acres and a volume of 176,900 acre-feet. The outlet works is provided with a selective withdrawal low flow system capable of withdrawing water from 4 levels and through the flood control conduit.

Input to the Model:

- Hydrologic data
- Meteorologic data
- Inflow Stream Temperature

Hydrologic Data

Monthly flows at the dam site have been estimated for the period 1924 through 1965. Since meteorological data is only available for the years 1949 to present, the period 1949 through 1965 was used to determine years representing wet, dry and average flow conditions. From this information, the years 1949, 1954 and 1961 were respectively selected as wet, dry and average flow years.

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Monthly and daily flows at the dam site for the period 1949 through 1965 were based upon observed reservoir levels and records of gate operation of Mountain Creek Reservoir located just downstream from Joe Pool Lake. A drainage area factor was applied to convert flows at the Mountain Creek Reservoir to flows at Joe Pool Dam site. Mean daily outflows, satisfying water management objective were obtained from hydrologic routings based on the proposed plan of regulation.

Meteorological Data

Meteorological data required consisted of dew point temperature, air temperature, sky cover and wind speed. This data was obtained from the Dallas station, located about 15 miles from the project, and is the closest National Weather Service Station with available data.

Inflow Stream Temperatures

Daily stream temperature data for the Mountain Creek are not available, however, daily stream temperatures are available for the West Fork of the Trinity River at Grand Prairie, Texas for the years 1968, 1970, and 1971. This is the nearest station with several years of stream temperature data and is located approximately 10 miles northeast of the project. To generate mean daily inflow temperatures for the three study years at Joe Pool Dam site, a multiple linear regression equation responsive to air temperature, stream flow and measured stream flow temperature at the Grand Prairie gage was calibrated. Using this equation, mean daily inflow temperatures for Joe Pool Dam site were calculated for the three study years. A fifth order polynomial curve was then fitted to the average of the computed daily stream temperatures for the same period. This yielded an equation which defined a curve considered representative of the natural stream temperatures for the releases from Joe Pool Lake.

Hydrologic Evaluation

The selective withdrawal system capacity of low flow outlet that is less than or equal to the period of record flow 95% of the time was determined to be 300 c.f.s. This number was based upon a period of record (1924-1965) routing for a minimum release requirement of either 5 c.f.s. or 35 c.f.s. (dependable yield).

Thermal Simulation of Joe Pool Lake[2]

The lake was simulated from April through October for all three study years. A sensitivity analysis was made of the three variables that control the mechanism for development of thermal stratification in the model. These are "D", diffision, "β", the fraction of radiation absorbed in the three feet of water in an impoundment, and "λ", the average absorption coefficient of impounded water.

Verification tests to determine the most reasonable values for these variables for Joe Pool Lake were conducted using observed temperature data for Belton and Lewisville Lakes.

The values used in simulating the observed lake temperature profiles for Belton and Lewisville Lakes are given in table 1 below.

Table 1

	Belton Lake	Lewisville Lake
D	3.5 ft^2/day	5.0 ft^2/day
β	0.75	0.75
λ	0.3 ft^{-1}	0.3ft^{-1}

Because of geographic location and relatively high wind speeds in the area of the proposed dam site the following values were chosen for Joe Pool Lake: $D = 5.0\ ft^2/day$, $\beta = 0.75$, $\lambda = 0.3\ ft^{-1}$. The pertinent data for the three projects are given in Table 2.

Table 2

Project	Joe Pool	Lewisville Lake	Belton Lake
Top of Conservation Pool	522	515.0	594.0
Surface Area (acres)	7,470	23,210	12,420
Capacity acre-feet	176,900	457,600	441,980
Max. depth* (ft.)	66	80	124.0
Ave. depth* (ft.)	24	20	36
Mean Annual Inflows (ac-ft)	58,977	467,100	488,300
Inflow/Volume	0.328	1.021	1.091
D	5.0	5.0	3.50
β	0.75	0.75	0.75
λ	0.30	0.30	0.30

* Based on conservation pool.

In order to determine the need for a selective withdrawal facility for Joe Pool Lake, three outlet configurations were investigated and thermal simulation of the lake was computed for the study years 1949, 1954, 1961 for each configuration. The first configuration consisting of only a flood control conduit with invert elevation 466.0 yielded release temperatures which satisfied the objective temperatures $\pm 5°F$ only 60% of the time.

The second configuration consisted of flood control conduit with invert elevation 466.0 and four 3x5 foot low flow ports with invert elevations at 486, 495, 504, and 513. The third configuration consisted of a flood control conduit with invert elevation at 466 and three 3X5 foot low flow ports with elevations at 486, 498, and 510.

A comparison of release temperatures for the second and third configuration showed that they were both equally capable of meeting the downstream temperatures within ±5°F of the objective temperature. However, due to the location of trash rack cap, three ports configuration was not practical. Therefore four ports configuration was selected. With a four port configuration selective withdrawal system, target temperatures were met 94% of the time, compared to 60% of the time without the selective withdrawal system.

The predicted thermal stratification in the proposed lake for the three study years showed little difference with minimum releases of 35 c.f.s. and 5 c.f.s. respectively. The lake reacted the same in all the study years. For the dry year 1954, stratification began in late May, continued through early July and began to breakup in early August. However, for the wet and average years the impoundment began stratifying in early April, had a stable stratification in summer months and began breaking up in September. The stable stratification was due to a longer retention time of about three years.

Operation

To determine if release temperatures utilizing multilevel ports satisfied downstream temperature criteria, we monitored release temperatures at Georgetown Lake project which was built in 1980. This project initially was closed in March 1980 and during the first year of operation the project did not completely fill. In 1981, however, the project operated to the design criteria under the filling plan. During this period the outflow temperatures at the project were monitored. The required downstream temperature was maintained by utilizing all the outlets at different times. During releases from Georgetown Lake an average temperature was maintained conducive to temperatures in the North Fork of the San Gabriel River before the project was built. Temperature records indicated that the release temperatures were within ±5° of objective temperature.

Although selective withdrawal outlet structures that have been built recently are designed primarily for downstream temperature control, they are seldom used for this purpose. The reason is that all these projects are operated primarily for flood control and water supply. Flood control releases are made either through the flood conduit or spillway gates. Water supply releases, instead of being released through low flow multilevel outlets, are pumped out directly into treatment plants. With this method of operation, these projects are still able to support some very good quality fishlife below them. Maintenance of downstream temperature control is not critical at any of the projects. The multilevel withdrawal capability is there just in case of a need for mixing upper level water with lower level water to achieve desired release temperatures. In general, it has been our policy to release water from the highest port for water quality benefit.

Key Words: selective withdrawal, thermal simulation, stream temperature, objective temperature, stratification, diffusion, radiation, absorption coefficient, routing.

APPENDIX - REFERENCES

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BLOOMINGTON DAM, POTOMAC RIVER WATER QUALITY OUTLET

By Frank Vovk* and Laverne S. Horihan** M. ASCE

ABSTRACT. The criteria available and the development of rationale for the design of the multilevel outlet facility for selective withdrawal to achieve the desired water quality conditions for use downstream from Bloomington Dam are discussed. The justification for selection of the water quality system features are presented, and recommendations for future projects are furnished.

DESCRIPTION. Bloomington Dam is located on the North Branch Potomac River; the river forms the border between western Maryland and northeastern West Virginia. The damsite is located about 8 miles upstream from the confluence of the North Branch Potomac River with the Savage River at Bloomington, Maryland.

The outlet works, located in the right abutment, consists of a 2,092 foot-long, 16 1/3 foot diameter tunnel which discharges into a hydraulic-jump stilling basin. The control tower contains multiple intakes that provide a means for obtaining water quality control of its releases for municipal and industrial uses. Details for the intake control tower are shown on Figure 1.

LOW-FLOW RELEASE SYSTEM. Two 6-foot diameter vertical wet wells that are located in the control shaft connect to individual 6-foot diameter inlet pipes equipped with butterfly valves. Two 2- by 3-foot electrically operated slide gates are located at the bottom of the wet wells and provide a capability for fine regulation. The discharge openings downstream from the regulating gates are joined into one rectangular sluice which is located in the enlarged outlet works center pier. The low-flow release jet enters the outlet works tunnel at the end of the pier where a lift, 2 feet above the tunnel invert is provided by a slight curvature. This detail was utilized to ensure atmospheric pressure around the issuing jet at the end of pier, as well as to prevent cavitation damages to the tunnel floor. The bends and transition areas were carefully selected to guarantee positive pressures. Releases through one unit provide sufficient discharge to meet minimum downstream requirements. The low-flow control gate is normally kept in a throttled position to both create a back pressure in the inlet pipes and keep the jet from falling into the wet well. Two individual low-flow outlet systems were constructed, each with five inlet pipes at symetric elevations.

RESERVOIR TEMPERATURE STRATIFICATION. During the preliminary study stage, it was contemplated that nine intake ports would be built into the control tower. A subsequent study relative to the heat budget and zone of withdrawal was completed, and the results of this study showed that five intake ports would be adequate to provide the quality of water required for downstream use. The proposed elevation and location of the intake ports of the planned withdrawal structure for the Bloomington project were evaluated through the use of both a thermal model developed by Water Resources Engineers, Inc. (WRE) of Walnut

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Creek, California and a density current analysis technique presented in U.S. Army Corps of Engineers' Waterways Experiment Station Technical Report H-69-10, "Mechanics of Flow from Stratified Reservoirs in the Interest of Water Quality," dated July 1969. The model developed by WRE uses input information that includes the reservoir size, shape, and orientation; the temperature and volume of inflow water; and meteorological data to compute a heat budget for the reservoir which permits computation of the thermal stratification pattern to be expected during the summer months. It then simulates withdrawal to meet a downstream temperature criterion. A density current analysis is used to more accurately define the withdrawal zones from which each port draws its water during the period of operation.

The only temperature requirement imposed on the released water was that downstream temperatures should be low enough to support a cold-water fishery throughout the spring and summer months. The temperature objective curve is shown on Figure 2A. Thermal stratification normally prevails in the reservoir during the March through October period and gradually breaks down until the reservoir becomes isothermal about the first of November. Temperature variations during the 1962 critical stratification buildup and breakdown periods are shown on Figures 2B and 2C.

Evaluation of study results indicated that three outlets would meet the assumed thermal requirements; however, it was recommended that the five-outlet feature of the selective withdrawal structure be retained in view of the uncertainty of the nature of chemical stratification in the reservoir. Water quality benefits now need to be reevaluated because of the water pollution control legislation that has developed since formulation of the project.

In order to comply with the Corps criteria which allows only 1 foot of drop between the reservoir and wet well for a discharge of 300 cubic feet per second (cfs) and 4 feet of drop for a maximum flow of 570 cfs, 8-foot inlet pipes should have been provided. The vertical wet well pipe should also have been 8 feet in diameter, but this would have created a space problem in the intake tower. For that reason, and because two low-flow systems were adopted, 6-foot diameter inlet pipes and vertical wet wells were built into the system.

DISCHARGE CAPACITY. The discharge rating curve, presented on Figure 3, shows that a minimum discharge of 300 cfs will be assured at a pool elevation of 1352.5 feet mean sea level (msl) with one unit in service. About 570 cfs can be released through one unit with a pool elevation at 1500 feet msl. The head loss coefficients for the low-flow outlet are shown in Table 1. The total head loss coefficients for determining discharge through the wet well are presented in Table 2.

RECOMMENDATIONS. Based on the experience gained during a short period of operation, changes suggested are (a) use of an 8-foot diameter inlet and wet well system; (b) increase the size of the air vents; (c) investigate replacement of butterfly valves; and (d) isolate the common bulkhead opening.

TABLE 1 - OUTLET WORKS LOW FLOW, LOSS ASSUMPTION

6 Foot Diameter Inlet Pipe

Description	Formula	Loss Coefficient
1. Entrance	0.50	0.50
2. Emergency gate slots	0.20	0.20
3. Bend; 60^{o} angle	0.17	0.17
4. Butterfly	0.20	0.20
5. Friction in inlet pipe n = 0.012; L = 24'	$\frac{(n^2)(2g)(L)}{2.2082\ R^{4/3}}$	
5-foot dia.		0.0756
6-foot dia.		0.0600
7-foot dia.		0.0477
8-foot dia.		0.0400
6. Velocity head		1.0000

(in terms of velocity head for inlet pipe)

Total K = 2.1456
Total K = 2.1300
Total K = 2.1177
Total K = 2.1100

6 Foot Diameter Wet Well Below Elevation 1342

Description	Formula	Loss Coefficient
1. Gate passage coefficient	0.10	0.10
2. Bend; 75^{o} angle	$0.130(AG/AAVE)^2$	0.0387
3. Contraction to gate	$(0.1)(0.43)(AG/AAVE)^2$	0.0128
4. Two 30^{o} bends	$0.236/28.27)^2$	0.0104
5. Contraction from (1) 6' pipe to (4' X 4') square	$(0.1)(0.39)(6/16)^2$	0.0055
6. Friction gate to 1290.66 n = 0.012; L = 22.46 ft	$\frac{(n)^2(2g)(L)}{2.2082\ R^{4/3}}$	0.1268
7. Friction 1290.66 to 1342.0 n = 0.012; L = 51.34 ft	"	0.0056
8. Velocity head		1.0000

(in terms of velocity head for 2' X 3' gate)

Total K = 1.2998

TABLE 2 - SUMMARY OF TOTAL HEAD LOSS AND DISCHARGE EQUATION COEFFICIENTS

Inlet Pipe Elev	N	L FT	TOTAL K_1	K^1	TOTAL K_2	K^{11}
1342	0	0	1.2998	42.21	1.3957	40.74
1375	1	33	1.3534	41.37	1.4493	39.97
1400	2	58	1.4062	40.58	1.5021	39.26
1426	3	84	1.4390	39.84	1.5349	38.85
1449	4	107	1.5116	39.14	1.6074	37.97

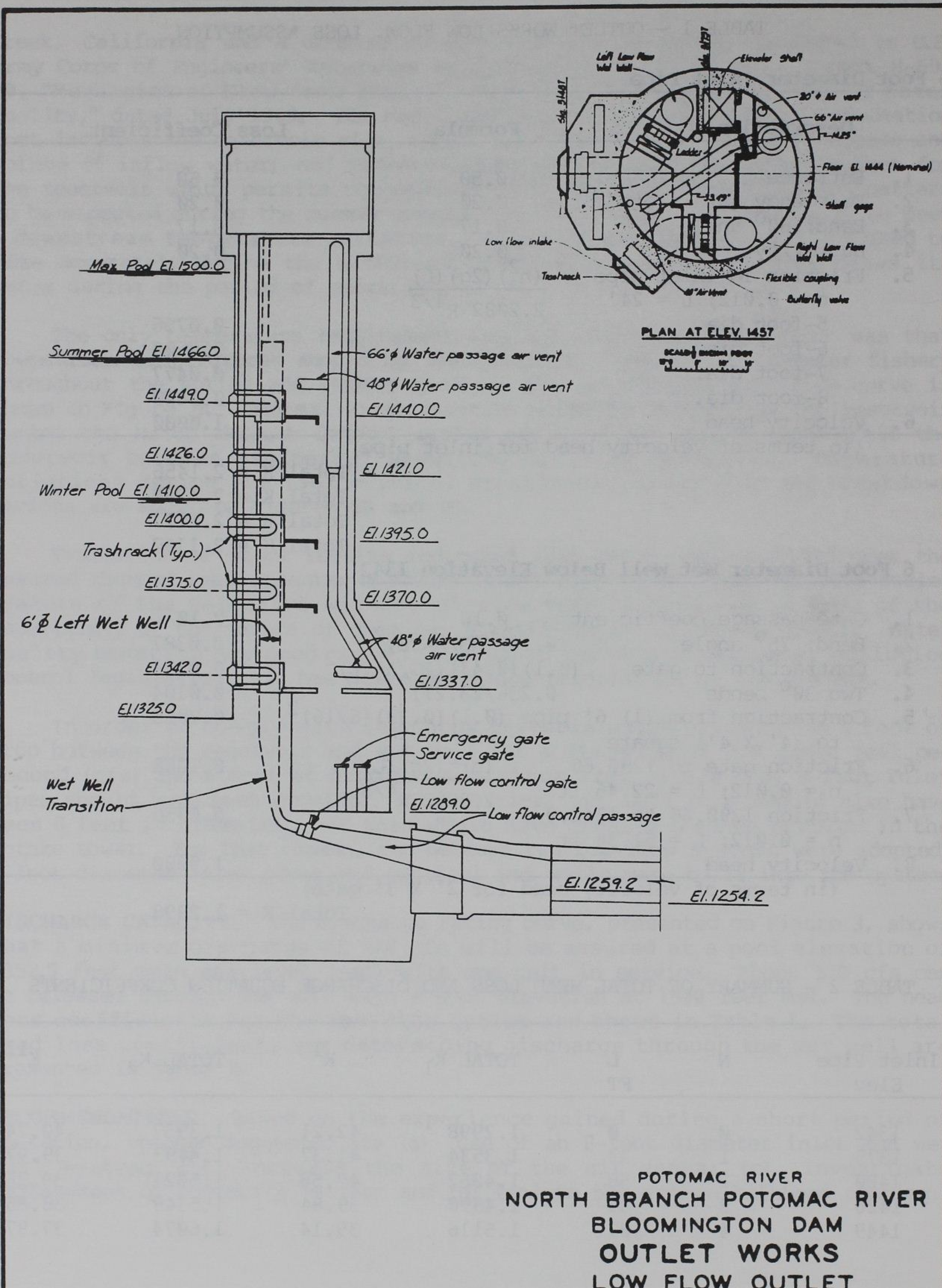
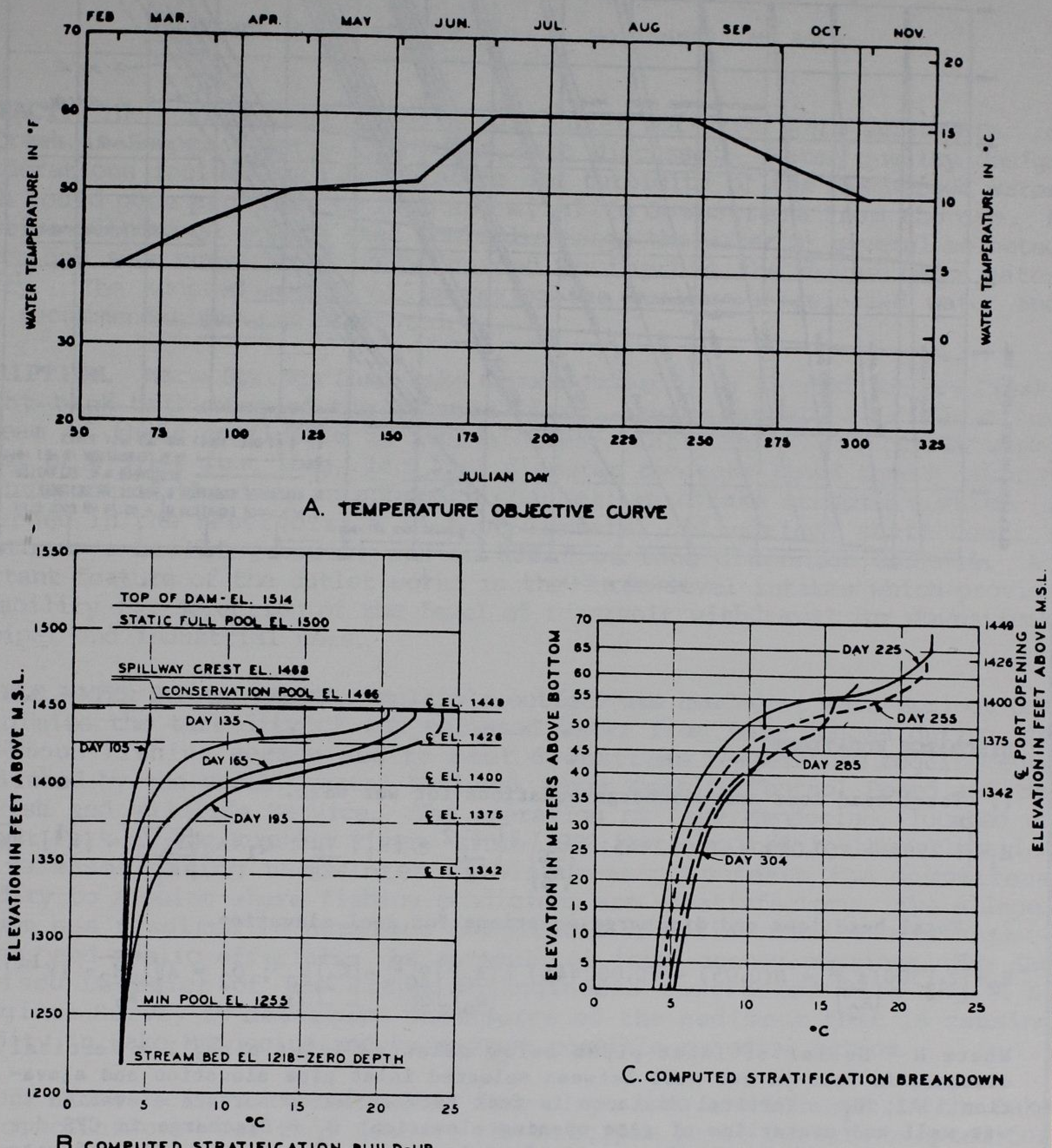

Max Pool El. 1500.0
Summer Pool El. 1466.0
66"ϕ Water passage air vent
48"ϕ Water passage air vent
El. 1449.0
El. 1440.0
El. 1426.0
El. 1421.0
Winter Pool El. 1410.0
El. 1400.0
El. 1395.0
Trashrack (Typ.)
El. 1375.0
El. 1370.0
6'ϕ Left Wet Well
48"ϕ Water passage air vent
El. 1342.0
El. 1337.0
El. 1325.0
Emergency gate
Service gate
Low flow control gate
Wet Well Transition
El. 1289.0
Low flow control passage
El. 1259.2
El. 1254.2
Elevator shaft
30"ϕ Air vent
Ladder
Floor El. 1444 (Nominal)
Shaft gage
Low flow intake
Trashrack
Right Low Flow Wet Well
Flexible coupling
Butterfly valve
PLAN AT ELEV. 1457
POTOMAC RIVER
NORTH BRANCH POTOMAC RIVER
BLOOMINGTON DAM
OUTLET WORKS
LOW FLOW OUTLET
U.S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS OMAHA, NEBRASKA

FIGURE 1



A. TEMPERATURE OBJECTIVE CURVE

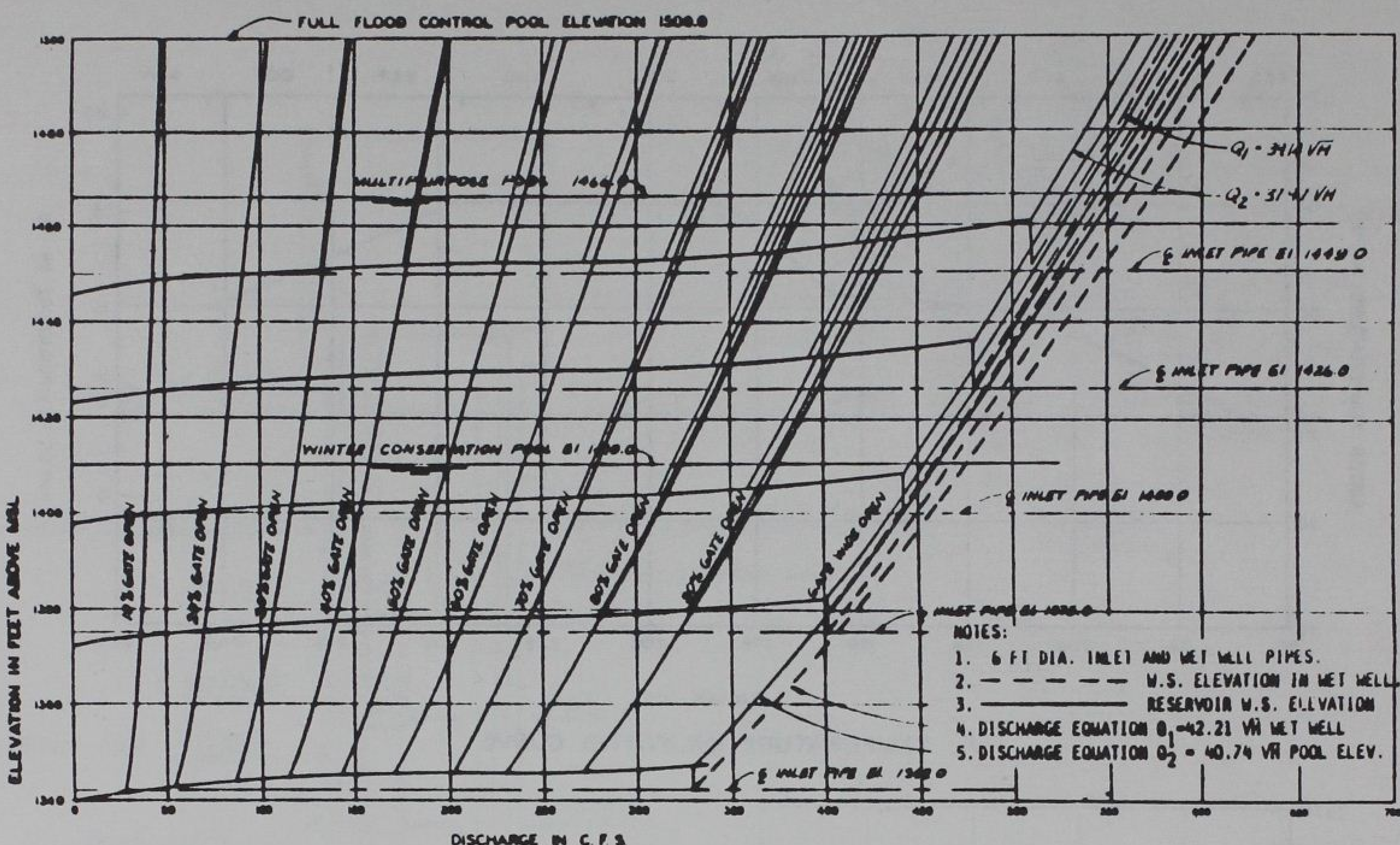
B. COMPUTED STRATIFICATION BUILD-UP

C. COMPUTED STRATIFICATION BREAKDOWN

POTOMAC RIVER
NORTH BRANCH POTOMAC RIVER
BLOOMINGTON DAM
OUTLET WORKS
STRATIFICATION PATTERN

U.S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS OMAHA, NEBRASKA

FIGURE 2



DISCHARGE EQUATIONS

1. Total head loss and discharge equations for wet well.

$$H_1 = \left[1.2998 + N(0.05) + (0.002445)\,L\left(\frac{A_G}{A_6}\right)^2\right]\frac{V_G{}^2}{2g} = [K_1]\,\frac{V_G{}^2}{2g};\quad Q_1 = A_G\sqrt{\frac{2gH_1}{K_1}} = [K^1]\,\sqrt{H_1}$$

2. Total head loss and discharge equations for pool elevation.

$$H_2 = \left[2.1300\left(\frac{A_G}{A_G}\right)^2 + N(0.05) + (0.002445)\,L\left(\frac{A_G}{A_6}\right)^2\right]\frac{V_G{}^2}{2g} = \left[K_2\right]\frac{V_G{}^2}{2g};\quad Q_2 = A\sqrt{\frac{2gH_2}{K_2}} = [K^{11}]\,\sqrt{H_2}$$

Where N = Number of inlet pipes below selected inlet pipe; L = Vertical distance in feet in wet well between selected inlet pipe elevation and elevation 1342; H_1 = Vertical distance in feet between water surface elevation in wet well and centerline of gate opening elevation; Q_1 = Discharge in CFS for wet well; H_2 = Vertical distance in feet between pool and centerline of gate opening elevation; Q_2 = Discharge in CFS for pool elevation.

POTOMAC RIVER
NORTH BRANCH POTOMAC RIVER
BLOOMINGTON DAM
OUTLET WORKS
LOW FLOW OUTLETS
DISCHARGE RATING CURVES
U.S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS OMAHA, NEBRASKA

FIGURE 3

WATER QUALITY OUTLET WARM SPRINGS DAM

By Frank Vovk* and Laverne S. Horihan**, M. ASCE

ABSTRACT. The analysis and design of the outlet works for Warm Springs Dam on Dry Creek in Sonoma County, California, are discussed. Water quality design considerations include both temperature and turbidity of the discharged water which could pose problems to fish and wildlife downstream from the dam. A selective withdrawal system that would discharge the water at several selected elevations was considered to be needed to improve the downstream water quality. The adopted method of improving the quality of released water and final recommendations are presented.

DESCRIPTION. Warm Springs Dam, Lake Sonoma Project, is located on Dry Creek, a right-bank tributary of the Russian River, approximately 14 river miles upstream of their confluence in Sonoma County, California. The outlet works consist of a 3,140 foot long, 14.5 foot diameter concrete-lined tunnel through the abutment of the dam; an approach channel; an intake structure which is submerged in the reservoir; a 30-foot diameter cylindrical shaft control structure; a stilling basin; and an 800-foot long discharge channel. An important feature of the outlet works is the three-level intakes which provide a capability for selection of the level of reservoir withdrawal for downstream municipal and industrial uses.

MULTIPLE WATER SUPPLY INLETS. Multiple outlets are needed at Warm Springs Dam to minimize the turbidity of the released water from Lake Sonoma during the anadromous fishing season and to meet downstream temperature requirements established by the North Coastal Regional Water Quality Control Board and the U.S. Fish and Wildlife Service. The operation of Lake Mendocino, located on the East Fork of the Russian River, since 1958 has resulted in the allegation that releases during the winter fishing season increase the downstream turbidity to a point where fishing conditions are unsatisfactory. The alleged problem has resulted in numerous complaints from fishermen, sports organizations, and public officials. As a result of inter-agency meetings, the San Francisco District of the Corps of Engineers contracted with the U. S. Geological Survey to determine the source of the sediment that is causing turbidity in Lake Mendocino and to suggest possible remedial action.

Dry Creek sediment contains less fine clays than are inherent in the Lake Mendocino inflow. For this reason, the multiple level outlets adopted for Warm Springs Dam have proven to be useful for a much longer period than might be expected at Lake Mendocino. Dry Creek inflows clear up rapidly following high discharges into Lake Sonoma, but storage of turbid floodwaters in the conservation pool results in releases with greater turbidity than before Warm Springs Dam was constructed. Therefore, provisions were made for selectivity in the elevation from which releases are drawn. Inclusion of multiple-level outlets in the intake shaft were requested by the Environmental Protection Agency, Water Quality Office, and the U.S. Fish and Wildlife Service. The

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North Coastal Regional Water Quality Control Board established water quality standards which include the turbidity and temperature parameters for the tidal reach of the Russian River.

LOW-FLOW OUTLET SYSTEM. The low-flow outlet consists of a submerged intake structure which has inlets at three levels, a 5-foot diameter pipe, bulkhead slots, and a butterfly valve for water quality selection. The lengths of these inlet pipes vary depending upon the elevation of the intakes. These inlet pipes are connected to one 6-foot diameter vertical steel wet well located within the control shaft. A single gate passage with a 2- by 3.5-foot electrically operated slide gate is provided for fine discharge regulation. The bends and transition area were carefully chosen to avoid negative pressures. A low-flow control gate is used in a throttled position to prevent the free-falling jet from dropping from the inlet pipes into the wet well. The discharge opening downstream from the regulating gate is located in the enlarged outlet works center pier. The jet enters the outlet works tunnel at the end of this pier; at which location, a small lift above the tunnel invert is provided to assure atmospheric pressure around the issuing jet. A detail of the low flow outlet system is shown on Figure 1.

DESIGN CHANGES. The water quality system presented in the General Design Memorandum was retained except that only one wet well was constructed. This resulted from a change in the design of the water supply to the fish hatchery. The change was from a flow-through system from the reservoir to a system where water is obtained by pumping from the stream. Temperature prediction studies, using the Corps' Hydrologic Engineering Center (HEC) computer program, indicated that the change in hatchery design concept and the addition of the low-flow inlet at the service gate level also eliminated the need for an inlet at the 270-foot level. The constructed outlets are at elevation 352, 391, and 431 feet mean sea level (msl). During initial reservoir filling, the low-flow releases were passed through a 30-inch diameter pipe connecting the wet well at elevation 274 feet msl. A water quality monitoring system was also installed. It consists of twelve 3/4-inch diameter plastic pipes leading from the reservoir at 12 different elevations to the control tower manifold system. With this system, the water temperature and other water quality parameters can be determined at each referenced elevation. Ordinary maintenance of the wet well is accomplished by use of the service gates as a by-pass when water quality conditions are favorable. See Figure 1.

DISCHARGE EQUATIONS. A head loss equation was developed for each section of the low-flow outlet systems. Finally, one equation, expressed in terms of regulating gate velocity head, was determined. Similarly, discharge equations to determine the water surface elevation in wet well and pool elevation, were developed.

The maximum discharge demand for municipal and industrial water is 300 cubic feet per second (cfs). The flow enters the wet well through the three level inlet pipes. The intake structures for the inlet pipes are large enough to assure low entrance flow velocities. This low velocity is believed to be desirable to improve the probability of withdrawal of water from a limited stratum of the reservoir. The discharge rating curves for the 5-foot diameter inlet pipe discharging freely into the wet well and for back pressure flow conditions for the system are shown on Figure 2.

WATER QUALITY CONTROL. The multilevel intake was constructed to provide a system by which the water temperature and turbidity could be satisfactorily regulated. The use of butterfly valves for the outlet works was approved at a September 21, 1973 meeting held in San Francisco at the South Pacific Division Office with the stipulation that they be operated either fully open or fully closed. A portion of the discharged water is to be used for the fish hatchery. The San Francisco District proposed to regulate the quality of water taken from the reservoir by having one valve fully open and another partially open. Studies made by the Omaha District, with a flow of 300 cfs, indicated that there would be no problem in operating the gates this way; however, it was felt that there could be some surging in the wet well. This could develop an unsteady force on the butterfly valve which would cause the valve to flutter. Tests of head differentials and gate openings, to determine the range of valve opening that would result in satisfactory valve operation, will be conducted when the reservoir reaches conservation pool level. The San Francisco District and the U. S. Army Engineer Waterways Experiment Station developed the instrumentation and prototype testing program. Butterfly valves installed in each intake line normally are scheduled to be operated with at least one gate fully open with the other gates partially open to select the levels from which water is admitted to the wet well. The slide gate at the center pier regulates the quantity of water released. Further details are shown on Figure 1.

RECOMMENDATIONS. As more experience is gained with selective withdrawal systems, changes and improvements in the water quality outlet will be incorporated. These changes could include (a) Larger air vents for wet wells; (b) Improved gate valves for inlet pipe control; (c) Larger inlet and wet well pipes; (d) A transition section between the inlet and wet well; and (e) Provisions for blocking the low flow bulkhead slots between inlet pipes.

Table 1 - Summary of coefficients for head loss and discharge computation. Refer to Figure 2.

Inlet Pipe Elev.	Wet Well L_1	Wet Well K_1	K^1	Inlet Pipe L_2	Pool Elevation K_2	K^{11}
352	71.75	1.3613	48.12	272	1.7459	42.49
391	110.75	1.3713	47.94	391	1.7145	42.88
431	150.75	1.3818	47.77	136	1.6923	43.16

TABLE 2 - OUTLET WORKS M. & I. RELEASES, LOSS ASSUMPTION

Description		Formula	Loss Coefficient
5' Dia. Inlet Pipe			
1. Entrance		Ø.1	Ø.ØØ74
2. Trash Fenders		Ø.5	Ø.Ø37Ø
3. Bulkhead Slots		Ø.Ø1	Ø.ØØØ7
4. Friction in 5' dia Pipe			
n = Ø.Ø14	L=272'	$\frac{(n)^2 (2g)\ L}{2.2082\ R^{4/3}}$	1.1564-Inlet El. 352.Ø
	L=197'		Ø.8357-Inlet El. 391.Ø
	L=136'		Ø.5794-Inlet El. 431.Ø
5. Butterfly Valve in Line		Ø.417Ø	Ø.417Ø
6. Bend Loss		(4)(Ø.1Ø)	Ø.4ØØØ
7. Manifold Loss or Vel. Head		1.ØØØØ	1.ØØØØ
(in terms of velocity head for 5' dia. pipe)			3.Ø185-Inlet El. 352.Ø
			2.6978-Inlet El. 391.Ø
			2.4415-Inlet El. 431.Ø
5' Dia. Pipe Below 28Ø.25			
1. Friction Wet Well		$\frac{(n)^2 (2g) L}{2.2082\ R^{4/3}}$	Ø.Ø986
n = Ø.Ø14; L = 23.25'			
2. Bend Loss (2) (45^{o})		(2) (Ø.135)	Ø.27ØØ
3. Manifold Loss (2)		(2) (Ø.1ØØ)	Ø.2ØØØ
4. Butterfly Valve in Line		Ø.417Ø	Ø.417Ø
5. Bend Loss ($1Ø8^{o}$)		Ø.15	Ø.2Ø34
(in terms of velocity head for 5' dia. pipe)			1.189Ø
2' X 3.5' Gate Passage			
1. Gradual Contraction from 5' dia. to 2' X 3.5' gate		Ø.1	Ø.Ø873
2. Regulating Gate Slots		Ø.1Ø	Ø.1ØØØ
3. Velocity Head		1.ØØØØ	1.ØØØØ
(in terms of velocity head of 2' X 3.5' gate)			1.1873
6' Dia. Wet Well			
1. Friction 6' Dia. Wet Well L=Ø			
n = Ø.Ø14	L=71.75'	$\frac{(n)^2\ (2g)\ L}{2.2082\ R^{4/3}}$	Ø.12Ø8-Inlet El. 352
	L=11Ø.75'		Ø.1864-Inlet El. 391
	L=15Ø.75'		Ø.2537-Inlet El. 431
2. Gradual Contraction from 6' Dia. to 5' Dia.		Ø.1	Ø.Ø518
3. Manifold Inlet El. 352.Ø		(2)(Ø.1Ø)	Ø.2ØØØ
Inlet El. 391.Ø		(3)(Ø.1Ø)	Ø.3ØØØ
Inlet El. 431.Ø		(4)(Ø.1Ø)	Ø.4ØØØ
(in terms of velocity head for 6' dia. pipe)			Ø.3726-Inlet El. 352
			Ø.5382-Inlet El. 391
			Ø.7Ø56-Inlet El. 431

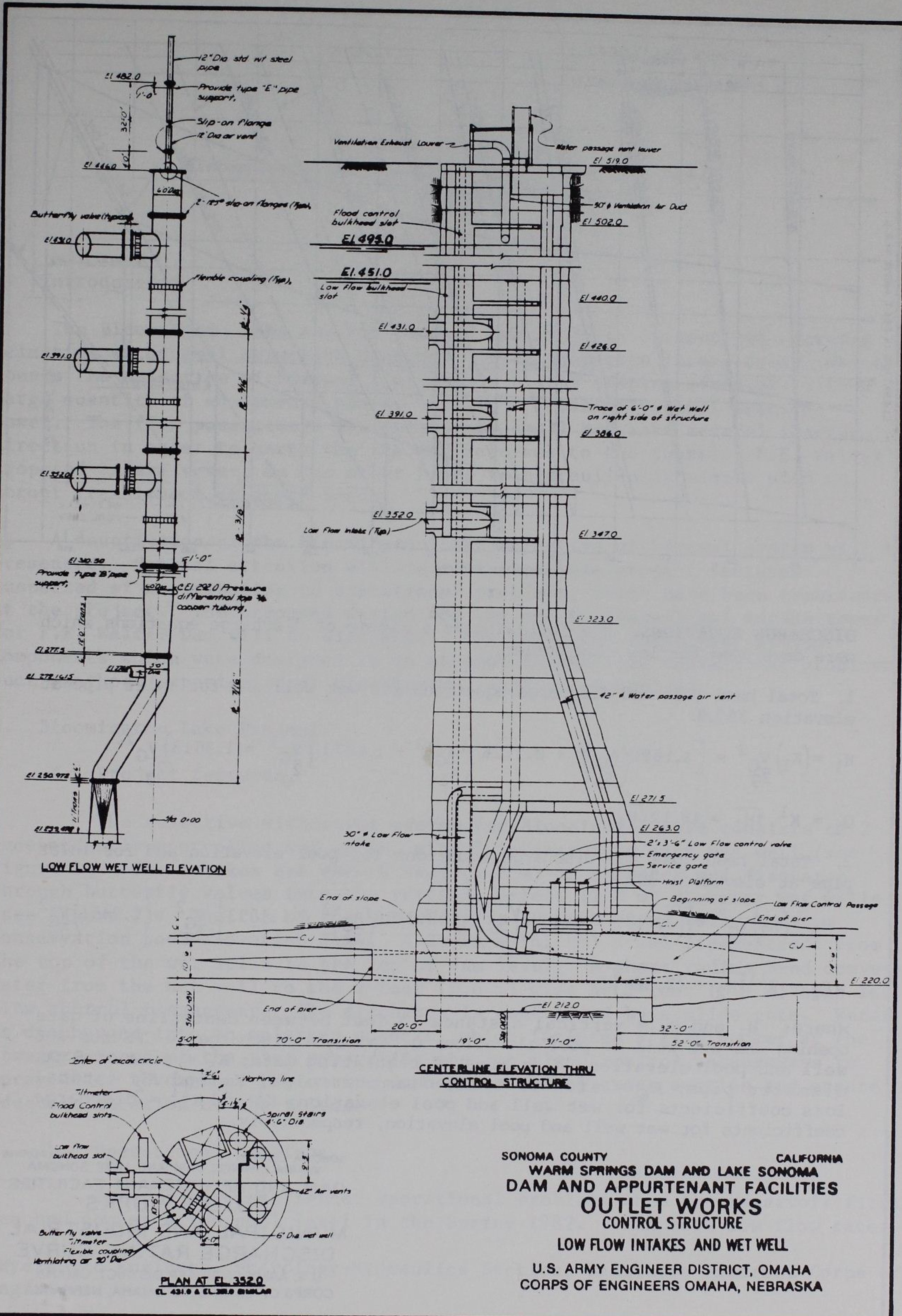
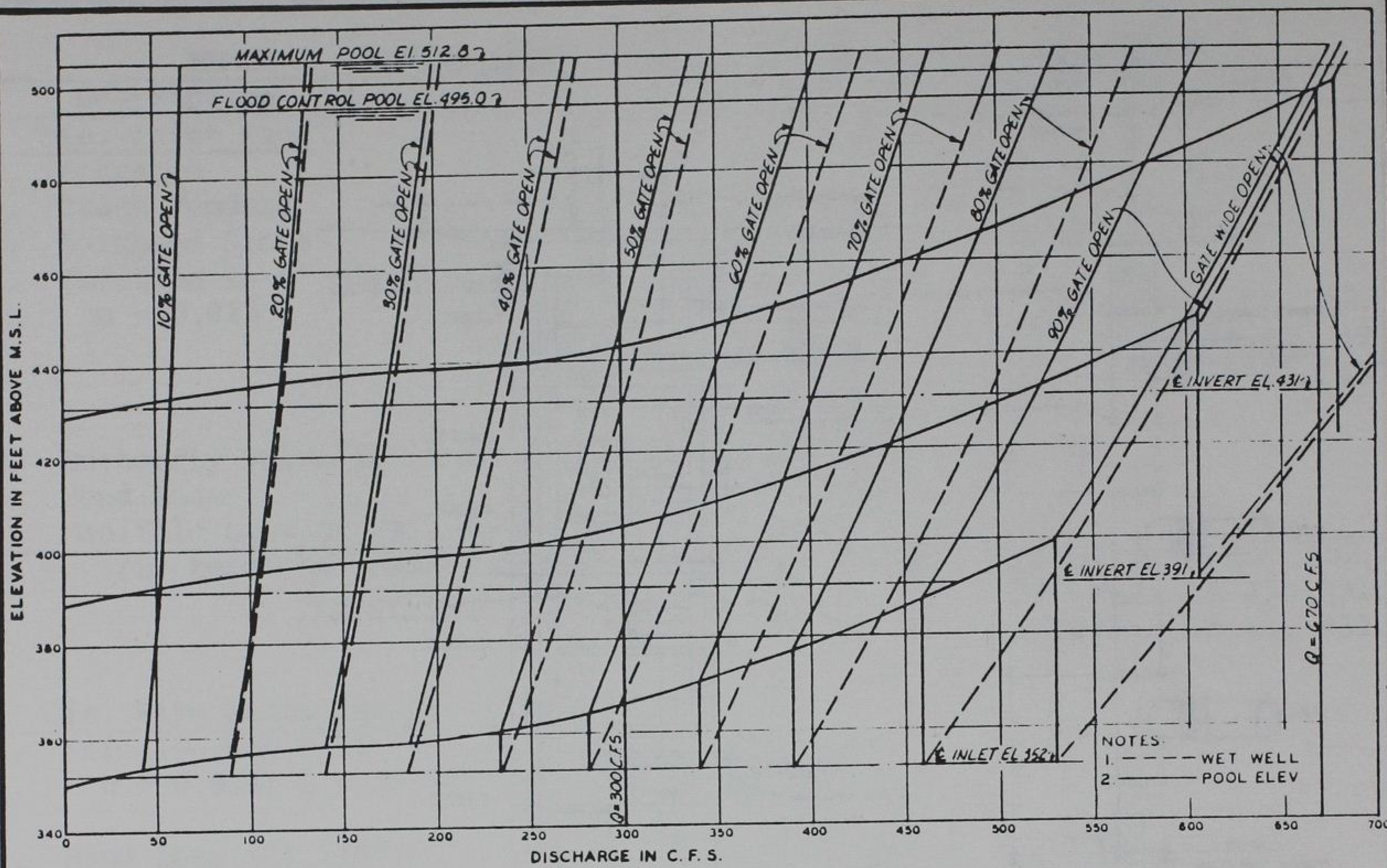

12" Dia std wt steel pipe
Provide type "E" pipe support,
Slip-on flange
12" Dia air vent
Butterfly valve (typical)
Flexible coupling (Typ.)
Provide type 'B' pipe support,
LOW FLOW WET WELL ELEVATION
Ventilation Exhaust Louver
Water passage vent louver
El 519.0
Flood control bulkhead slot
30" ø Ventilation Air Duct
El 502.0
El 451.0
Low flow bulkhead slot
El 440.0
El 431.0
El 426.0
Trace of 6'-0" ø Wet Well on right side of structure
El 391.0
El 386.0
El 352.0
Low Flow Intake (Typ.)
El 347.0
El 323.0
42" ø Water passage air vent
El 271.5
El 263.0
30" ø Low Flow intake
2'x 3'-6" Low Flow control valve
Emergency gate
Service gate
Hoist Platform
End of slope
Beginning of slope
Low Flow Control Passage
End of pier
El 220.0
El 212.0
70'-0" Transition
20'-0"
19'-0"
31'-0"
32'-0"
52'-0" Transition
CENTERLINE ELEVATION THRU CONTROL STRUCTURE
Center of excav circle
Northing line
Tiltmeter
Flood Control bulkhead slots
Spiral stairs
Low flow bulkhead slot
42" Air vents
Butterfly valve
Flexible coupling
6" Dia. wet well
PLAN AT EL 352.0
SONOMA COUNTY
CALIFORNIA
WARM SPRINGS DAM AND LAKE SONOMA
DAM AND APPURTENANT FACILITIES
OUTLET WORKS
CONTROL STRUCTURE
LOW FLOW INTAKES AND WET WELL
U.S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS OMAHA, NEBRASKA

FIGURE 1



DISCHARGE EQUATIONS. The following is a sample of discharge equations which were developed for low flow outlet.

1. Total head loss and discharge equations for wet well and for inlet pipe at elevation 352.0.

$$H_1 = (K_1)\frac{V_G^2}{2g} = \left[1.1890\left(\frac{A_G}{A_5}\right)^2 + 0.3726\left(\frac{A_G}{A_6}\right)^2 + 1.1873\right]\frac{V_G^2}{2g} = (1.3613)\frac{V_G^2}{2g}$$

$$Q_1 = K^1\sqrt{H_1} = 48.12\sqrt{H_1}$$

2. Total head loss and discharge equations for pool elevation and for inlet pipe at elevation 352.0.

$$H_2 = (K_2)\frac{V_G^2}{2g} = \left[(3.0185 + 1.1890)\left(\frac{A_G}{A_5}\right)^2 + 0.3726\left(\frac{A_G}{A_6}\right)^2 + 1.1873\right]\frac{V_G^2}{2g} = (1.7451)\frac{V_G^2}{2g}$$

$$Q_2 = K^{11}\sqrt{H_2} = 42.50\sqrt{H_2}$$

Where: H_1 and H_2 = vertical distance in feet between centerline of gate opening and wet well or pool elevation; Q_1 and Q_2 = Discharge in CFS for wet well and pool elevation; A_G = area of regulating gate; A_5 = area of 5 ft. diameter pipe; A_6 = area of 6-ft. diameter pipe; K_1 and K_2 = total loss coefficients for wet well and pool elevation; K^1 and K^{11} = discharge coefficients for wet well and pool elevation, respectively.

SONOMA COUNTY CALIFORNIA
WARM SPRINGS DAM AND LAKE SONOMA
DAM AND APPURTENANT FACILITIES
OUTLET WORKS
MUNICIPAL AND INDUSTRIAL
DISCHARGE RATING CURVE
U.S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS OMAHA, NEBRASKA

FIGURE 2

Hydraulic Design
Bloomington Lake and F.E. Walter Dam Projects'
Selective Withdrawal Structures

Dennis Seibel*

1. Introduction

The Bloomington Lake and F.E. Walter Dam Projects present two extremes of selective withdrawal structure designs. The Bloomington intake tower, which houses the selective withdrawal systems, is a very complex structure with a large quantity of equipment, pipes, etc. in a relatively small diameter tower. The flow passageways are relatively small and make several changes in direction in order to carry the desired releases to the tunnel. F.E. Walter's proposed intake tower, on the other hand, is hydraulically simple with no abrupt transitions or sharp bends.

A description of the Bloomington Lake selective withdrawal system will be presented. Special attention will be given to those project features suspected of contributing to operational problems, which have been experienced at the project. The proposed design for the new multiple-level intake tower for F.E. Walter Dam will be discussed. The selective withdrawal system components which were designed in an attempt to minimize operational problems, such as those experienced at Bloomington, will be emphasized.

2. Bloomington Lake Project

a. Project features

The selective withdrawal system for Bloomington Lake consists of 2 independently-controlled wet wells, with 5 intakes into each wet well (see figure 1). The intakes are each 6 feet (1.8 m) in diameter and discharge through butterfly valves into the vertical 6-feet (1.8 m) diameter wet wells (see figure 2). The top of the wet wells is 10 feet (3 m) below the conservation pool elevation 1466. A 24-inch (61 cm) steel pipe extends from the top of the wet wells to the top of dam level. A short radius bend conveys water from the wet well to the 2-feet (0.6 m) wide by 3-feet (0.9 m) high low flow control passageway. The discharge is controlled by a slide gate. Water is discharged into an enlarged conduit, which includes a flip bucket at the downstream end of the pier separating the two flood control passageways to spray water throughout the entire downstream tunnel cross section and ensure adequate aeration of the release.

b. Operational problems.

The project experienced operational problems when the reservoir first reached conservation pool level in the Spring 1982. With both low flow gates

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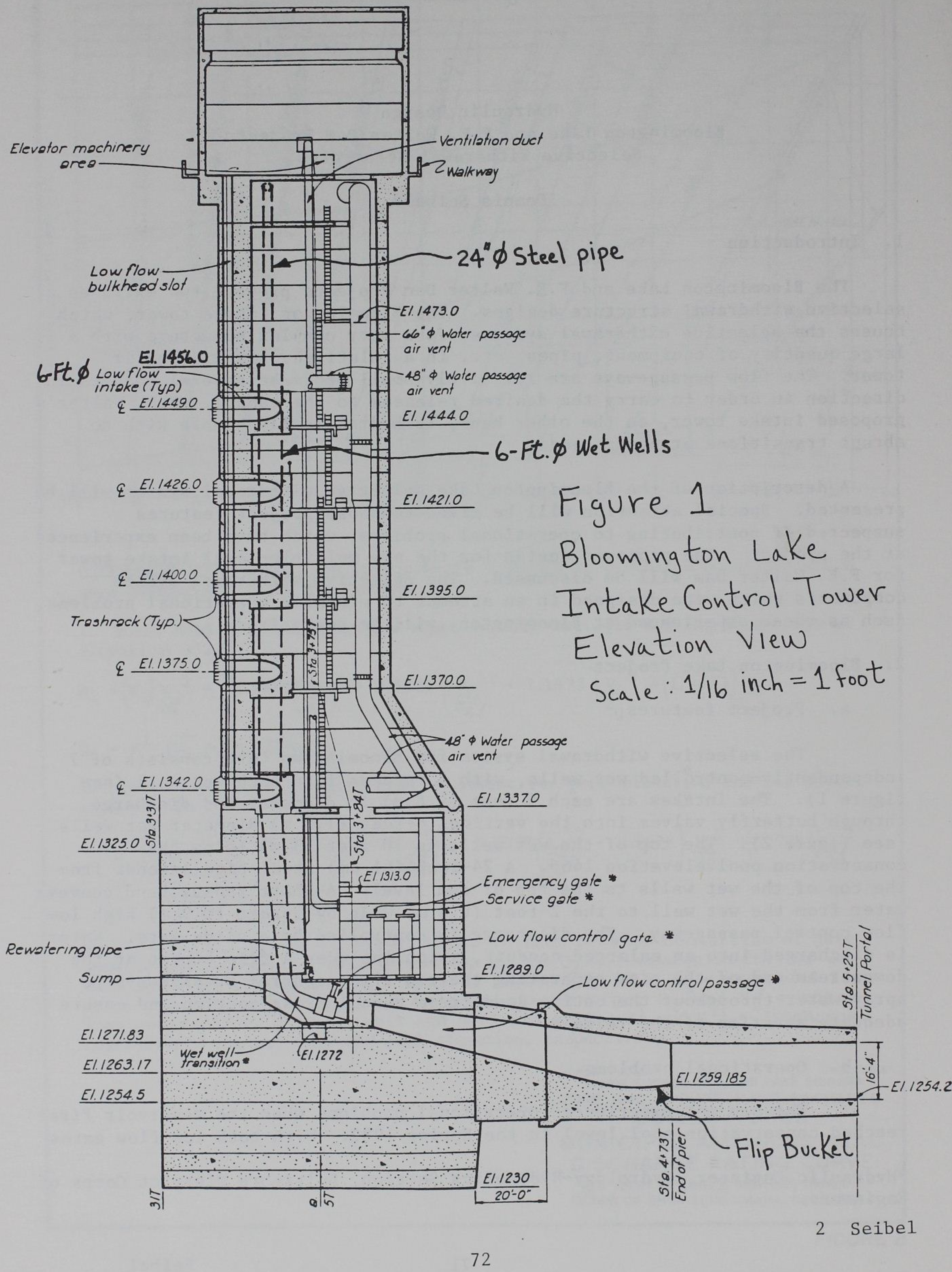


Figure 1
Bloomington Lake
Intake Control Tower
Elevation View
Scale: 1/16 inch = 1 foot

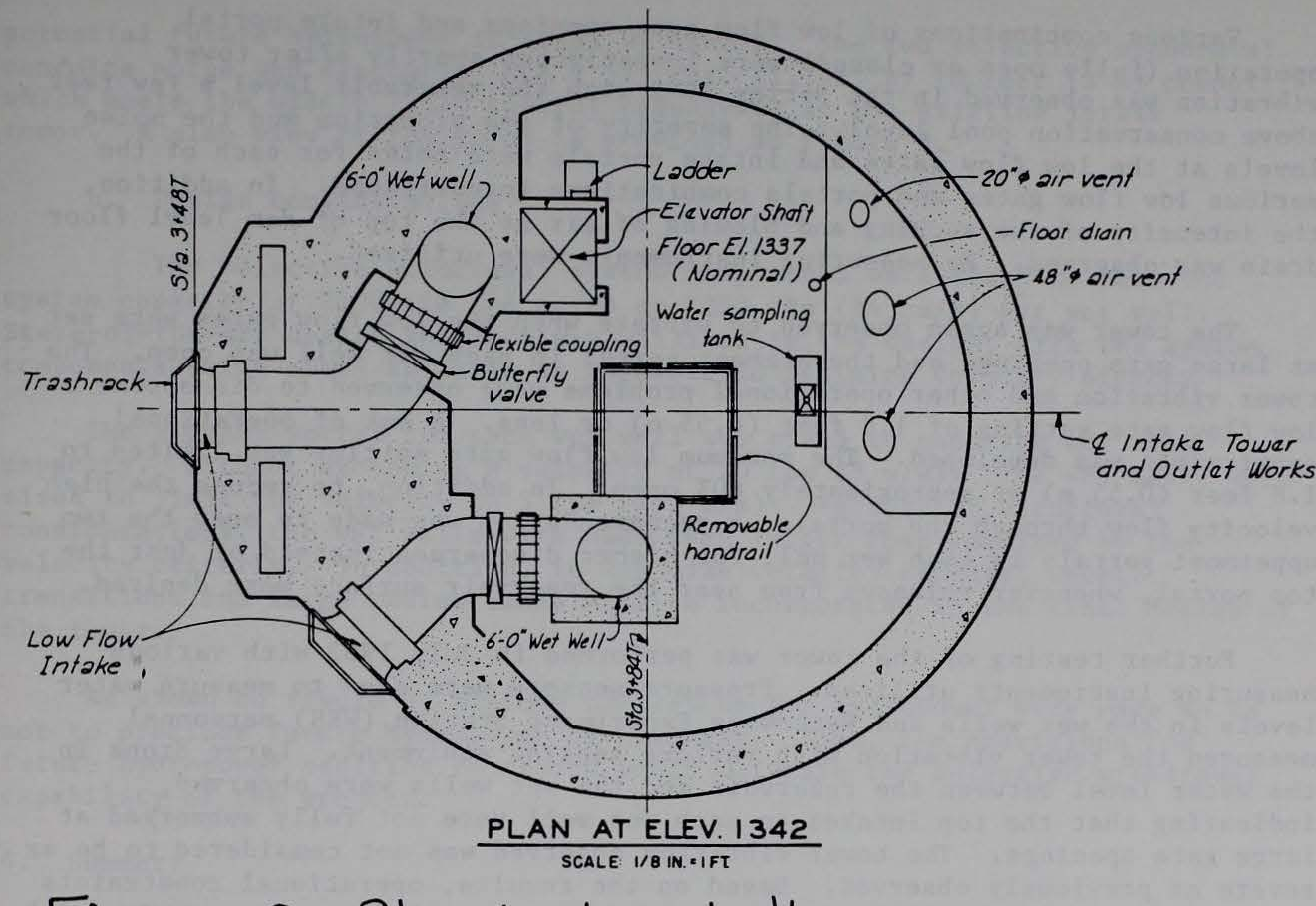


Figure 2. Bloomington Lake Intake Control Tower

open 100%, the tower was observed to vibrate and very loud noise was heard at the low flow gates. In addition, a floor drain at the top of dam level, which drains into one of the wet wells, was observed to alternately suck and blow air. At the time the problems were observed, the highest portal in each wet well was open. At the butterfly valves a sound similar to gravel flowing through a pipe was heard, indicating that cavitation was probably occurring.

c. Investigation of operational problems.

Among the potential causes suspected for the vibration and other operational problems was the high velocity flow through the portals striking the back wall of the wet wells with a considerable force. As seen on figure 2, the alignment of portal discharge into the wet wells is not symmetrical. This eccentricity of flow into the two wet wells is believed to have the potential for inducing vibration of the tower for high discharges through the portals. Other suspected causes are cavitation at the short radius bend at the bottom of the wet wells, operation of the low flow gates at openings greater than 80% open, drawdown of the wet well water level such that the portals are not fully submerged, and the top of the wet wells being 10 feet (3 m) below conservation pool level.

Various combinations of low flow gate openings and intake portal operation (fully open or closed) were investigated shortly after tower vibration was observed in the Spring 1982 with the reservoir level a few feet above conservation pool level. The severity of the vibration and the noise levels at the low flow gates and intake portals were noted for each of the various low flow gates and portals combinations investigated. In addition, the intensity of the sucking and blowing of air at the top of dam level floor drain was observed. No measuring instruments were utilized.

The tower was again observed to vibrate when the low flow gates were set at large gate openings and the highest portal in each wet well was open. The tower vibration and other operational problems were observed to dissappear at low flow gate setting of 1.8 feet (0.55 m) or less. A set of operational constraints was developed. The maximum low flow gate setting was limited to 1.8 feet (0.55 m) or approximately 60% open. In addition, to reduce the high velocity flow through the portals, a recommendation was made to open the two uppermost portals in each wet well for higher discharges instead of just the top portal, whenever releases from near the reservoir surface were desired.

Further testing of the tower was performed in July 1983 with various measuring instruments utilized. Pressure sensors were used to measure water levels in the wet wells and Waterways Experiment Station (WES) personnel measured the tower vibration with various sensing equipment. Large drops in the water level between the reservoir and the wet wells were observed, indicating that the top intakes in each wet well were not fully submerged at large gate openings. The tower vibration observed was not considered to be as severe as previously observed. Based on the results, operational constraints were somewhat relaxed. The maximum low flow gate setting was increased to 2.5 feet (0.75 m) or 80% open.

Additional tests were performed in the Spring 1984, with the intent of determining the source of the tower vibration and other operational problems. The results of the tests were inconclusive. In the Spring 1985, some tests were performed by WES personnel. No conclusions have been drawn from the latest tests at this time. Until the sources of the operational problems is determined and corrective measures taken, the project will continue to be operated with the constraints previously identified, to minimize operational problems.

3. F.E. Walter Dam -- New Intake Tower

a. Project Feature

A new intake tower was required for the project to accommodate the planned raising of the conservation pool level by 127 feet (38.7 m) and to obtain selective withdrawal capability from the deeper reservoir. The selective withdrawal system consists of two 18-feet (5.5 m) by 18-feet (5.5 m) wet wells with 4 portals discharging into one wet well and three discharging into the other. The highest portal for each wet well is 10 feet (3 m) high by 12.5 feet (3.8 m) wide. The remaining portals are circular in shape with a diameter of 10 feet (3 m). The portals discharge through butterfly valves into the wet wells. Discharge from each wet well is controlled by a slide gate in the 3.5 feet (1.1 m) wide by 10 feet (3 m) high selective withdrawal conduit. The maximum gate setting will be limited to 4 feet (1.2 m) until potential future hydropower is added by others. The two selective withdrawal conduits merge and discharge into a 5.5 feet (1.7 m) by 10 feet (3 m) conduit, which meets the middle flood control passageway at the existing intake tower. A plan view of the tower is provided as figure 3.

b. Design considerations

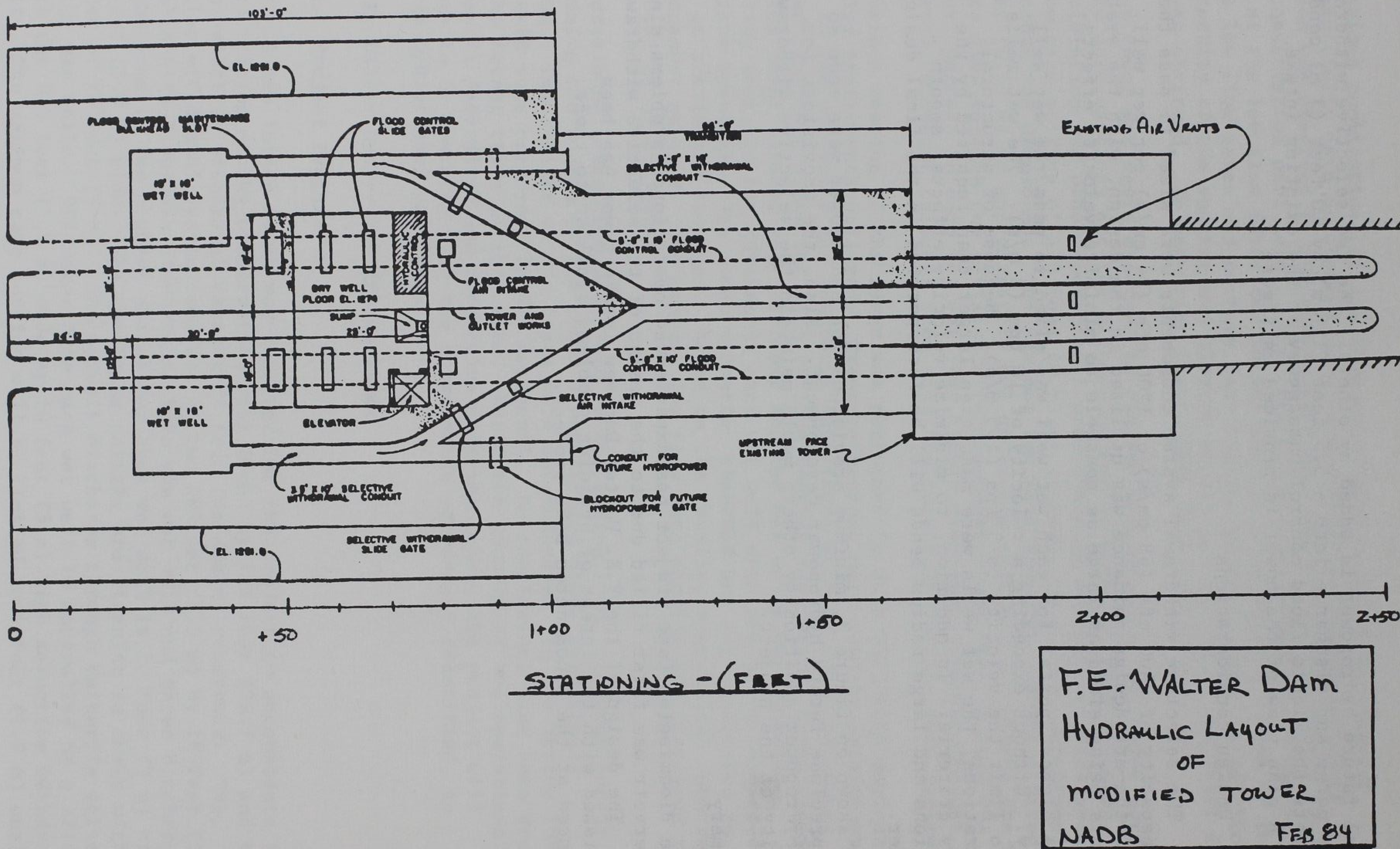
The selective withdrawal system components were sized to pass the system capacity of 2400 cfs (68 cm/s) or 1200 cfs (34 cm/s) per wet well. State-of-the-art design guidance was utilized in the design, with the system components being sized as large as possible to minimize velocity effects.

The highest portal for each wet well was sized to pass the wet well capacity. Without exceeding a velocity of 10 fps (3 m/s). The wet wells were sized to limit the velocity to 5 fps (1.5 m/s). Because of structural considerations, the wet wells were made even larger than required by the velocity criteria. In addition, to minimize velocity effects, smooth transitions and large radius bends will be incorporated in the final design of the tower.

As shown on figure 3, minimum provisions for hydropower were made so as not to preclude future hydropower development. Also, the provisions for future hydropower addition by others would maintain the selective withdrawal capability of the project.

4. Summary

The Bloomington Lake project has experienced operational problems since the reservoir was first filled due to the design of the selective withdrawal system. The design of the F.E. Walter Dam new intake tower has been accomplished with the intent of avoiding problems similiar to those experienced at the Bloomington Lake project.


103'-0"
EL. 1281.0
FLOOD CONTROL MAINTENANCE BULKHEAD SLOT
FLOOD CONTROL SLIDE GATES
TRANSITION
SELECTIVE WITHDRAWAL CONDUIT
EXISTING AIR VENTS
WET WELL
DRY WELL
HYDRAULIC CONTROL
FLOOD CONTROL CONDUIT
FLOOD CONTROL AIR INTAKE
SUMP
℄ TOWER AND OUTLET WORKS
ELEVATOR
SELECTIVE WITHDRAWAL AIR INTAKE
UPSTREAM FACE EXISTING TOWER
CONDUIT FOR FUTURE HYDROPOWER
BLOCKOUT FOR FUTURE HYDROPOWERE GATE
SELECTIVE WITHDRAWAL CONDUIT
SELECTIVE WITHDRAWAL SLIDE GATE
EL. 1281.0
0
+50
1+00
1+50
2+00
2+50
STATIONING - (FEET)
F.E. WALTER DAM
HYDRAULIC LAYOUT
OF
MODIFIED TOWER
NADB
FEB 84
Figure 3

DETERMINATION OF SELECTIVE WITHDRAWAL SYSTEM CAPACITY FOR INTAKE TOWER DESIGN

Kenneth S. Lee *

INTRODUCTION

The primary method for controlling water quality in and downstream of a reservoir is through a selective withdrawal system (SWS). This system provides the ability to selectively withdraw water from various levels in the lake and in some cases blend withdrawals from various levels. The effectiveness of the system depends upon the type of withdrawal structure, withdrawal capacity, portal size, portal location and portal number, and the ability of the water control manager to make effective use of the system and the available water resources in the reservoir. Engineering Manual 1110-2-1602 and Engineering Regulation 1110-2-1402 address the design criteria for the type of structure, portal size and portal location and number, but system capacity has not been addressed yet. The capacity is probably the single most important feature of a SWS. The purpose of this paper is to briefly discuss existing sizing methods and introduce a new approach for determining system capacity.

The original method for sizing SWS capacity was to arbitrarily choose a flow from a flow duration curve in either the annual or seasonal period, adding some engineering judgment, factoring cost, and hoping it works. This approach does not consider the particular SWS needs of the project under consideration. By this approach it is quite common to design a system that functions perfectly 98% of the time but each time the required discharge exceeds the SWS capacity, the downstream objectives are violated to such an extreme that severe long term damage is suffered by the downstream habitat. This situation can occur several times a year with quite disastrous consequences such as violations of stream standards, fish kills, or more likely, subtle and less obvious alteration of the invertebrate community upon which the whole system depends. To be effective, the SWS must function to prevent damaging deviations of physical and chemical parameters in the area influenced by the project. The more modern and conventional approach is to design a system that minimizes the sum of the square of the deviation for the downstream objective. The use of mathematical models to test the performance of a SWS is now a fairly standard procedure. By this approach, a system that maintains control most of the time and has only one or two major violations per year looks better from a least square standpoint, than a design that has frequent minor deviations but no major deviations. To overcome this obvious flaw in design approach, the U.S. Army, Corps of Engineers, Baltimore District, has developed a procedure to evaluate the impact of unavoidable system overload and

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thereby determine the required SWS capacity to prevent problems in the reach affected by the project. This approach considers all the facets of the old methodology such as annual and seasonal flow frequency, lake and downstream deviation, etc., but it carries the analysis one step further. It establishes a means of sizing the system based on the needs of the project.

FACTORS FOR DETERMINING SELECTIVE WITHDRAWAL CAPACITY

The impacts on the environment caused by the SWS capacity are great. However, as the system increases in size, so does the cost. Therefore, the selective withdrawal structure should be adequate to meet its objectives, but as small as possible. The function of capacity in controlling water quality by the SWS is the most important facet of a system. The role played by the portal number and location is similar but of generally lesser significance. Even if the portal number and location is correct, if the system capacity is undersized, release objectives may be severely and frequently violated. An undersized system is in most cases worse than no SWS at all. The undersized system allows control of release quality on an intermittent basis, while no system at least provides relative uniformity. The following are some of the factors to consider in determining system capacity:

1. State Water Quality Standards

State laws establish water uses and water quality criteria on streams and rivers. The SWS should be designed to meet the state water quality standards to the extent possible. It should be pointed out that a SWS is not a water treatment plant; it can be used only to manage and to some extent modify the resources of a project.

2. Project Water Quality Objectives Upstream and Downstream

Project water quality objectives must be determined. Each project should have upstream and downstream goals. These goals, such as 2 tier lake fisheries, cold water downstream fishery, pollution abatement, recreation releases, etc., must be considered in sizing the system.

3. Hydrologic Conditions

Inflow volume and its distribution are major factors in determining the SWS capacity. Proper SWS capacity is usually most important during periods of lake thermal stratification but there are exceptions. Lakes may stratify chemically or due to physically induced density gradients. Seasonal flow distribution is important for determining how well the system will work during extreme events. From a quality standpoint, the most extreme hydrologic events is not always associated with the most extreme capacity requirements of the SWS.

4. Physical Characteristics of a Project

Reservoir depth and hydraulic residence time affects withdrawal system design. Geometry of the lake and the approach to its outlet location are important factors.

5. Project Purposes

Project purposes and uses are important factors to consider in the determination of the system capacity. If the purposes of the project are water supply, low flow augmentation, white water sports, etc., the system capacity should be sized to accommodate these purposes without adverse water quality impacts.

6. Pool Fluctuation and Drawdown

Operational flexibility is a key factor for determining the system capacity. Maintaining a constant pool elevation requires a much larger system capacity than if the pool can accommodate some fluctuation. The fluctuating conservation pool is able to temporarily store high inflows and to then gradually release them. Artificial lowering of the conservation pool at critical times for the purpose of storing high inflow may also reduce the system capacity requirements. A fluctuating pool, however, will require a different port configuration than a constant pool.

7. Design for the Unexpected

Water quality problems such as iron, manganese, turbidity, etc. should be considered in determining the system capacity. However, unexpected problems do occur despite our best efforts to foresee them. Reasonable accommodation for the unexpected is important.

8. Evaluate the Consequence of Failure to Meet Objectives

A project should be evaluated for its impacts on water quality when its release requirements exceed the SWS capacity. Evaluation of an acceptable degree of violation and frequency should be accomplished. No SWS can meet all its objectives all the time but it can be designed and operated to come as close as possible. Violations of some parameters are so critical that it may take 2-3 years for the ecosystem to recover even though the violation may occur for only a few hours. The system capacity should be sized to avoid disastrous impacts downstream under all reasonably anticipated circumstances.

9. Future Development

If it is anticipated that a project may have future modifications such as hydropower, water supply or reallocation of storage that will affect pool levels or discharge requirements, the SWS capacity should be designed to accommodate these changes.

METHODOLOGY

The factors affecting the needs of SWS capacity differ from project to project. Each project must be evaluated separately. It is very dangerous to attempt to transfer a design form one project to another. What works at one location usually won't work at another. The following are step by step procedures on how to determine the SWS capacity:

Step 1. Establish System Objectives.

Each project has its own authorized purposes, objectives and water quality requirements. Determine those purposes and the standards, and establish the quality objectives. In some cases, the project may not meet the state standards. For these cases, establish reasonable goals which the project can accomplish. Make sure the goals are acceptable to all concerned parties.

Step 2. Prioritize the Water Quality Objectives.

No SWS meets all the objectives all the time. Prioritize the quality objectives and the project purposes. Prioritizing the quality parameters should be based on the magnitude of the impact of failure on water quality both in and downstream of the lake. Generally, temperature control is a primary objective. However pH, dissolved oxygen, manganese, iron, or any other parameter may be the primary objective.

Step 3. Determine Which Project Purpose Requires The Maximum Discharge.

The primary purpose of the project does not always require the maximum discharge. The second or third purpose may require the maximum discharge. For instance, if a project is authorized primarily for water supply and flood control, recreation and navigation are secondary purposes, the secondary purpose of recreation may require the maximum discharge.

Step 4. Make a Ballpark Estimate of System Capacity.

After prioritizing the purposes and objectives, a mathematical computer model needs to be developed to evaluate water quality conditions in the lake and downstream. Most numerical reservoir models need a SWS capacity to operate. For the initial model run, a ballpark estimate of system capacity is chosen from a seasonal duration curve. The flows equivalent to 5 and 10 percent exceedance from the seasonal duration curve will usually be adequate for shallow (less than 50 feet deep) or deep reservoirs, respectively.

Step 5. Numerical Model.

There are several numerical models available. Those developed by WES or HEC are recommended. The selection of the model depends upon the water quality objectives and expected water quality problems at the project. If temperature is the only primary concern, the model WESTEX may be adequate. Models are always being improved and it is wise to consult with WES or HEC before choosing a model.

The model simulates water quality conditions in the lake and downstream using the initial "ballpark" SWS capacity, portal location, portal number and an operational plan. The operational plan should as accurately as possible reflect how the project will operate. The model results will predict the expected conditions in the lake and downstream under given hydrological conditions (selected flow years).

Maximum adverse environmental impacts generally, but not always, occur when a project releases its maximum outflow. To assess the maximum adverse impacts, analyze the maximum reasonable discharge and its frequency from hydrological records.

Step 6. Determine a Maximum Reasonable Discharge.

Each project has its own operational limitations of maximum reasonable discharge. This maximum reasonable discharge may be determined by a downstream channel capacity, a limitation for downstream flood protection, or other rules that will govern the project's operation. For instance, the purposes of Cowanesque Lake, Pennsylvania are flood control, recreation and water supply. The channel capacity below the dam is 9,000 cfs. The maximum discharge capacity of the outlet structure is 9,000 cfs but the maximum reasonable discharge is only 4,000 cfs. This is because the project is operated to limit the downstream flow to no more than 4,000 cfs during storm events to prevent excessive surcharge to the downstream channel. The tunnel design of 9,000 cfs was for diversion during construction. 4,000 cfs is the maximum reasonable discharge. It will not be exceeded except in the most catastrophic flood event.

Step 7. Analysis of Outflow Magnitude and Its Frequency.

Determine when and how often the maximum reasonable discharge will occur at the project. The outflow magnitude and its frequency depends upon inflow volume and its distribution, pool fluctuation, and the operational plan. Allowable pool fluctuation and the operational plan influences outflow magnitude.

Table 1 is an example of the outflow magnitude and its frequency at Cowanesque Lake. The frequency table represents an event frequency not a daily flow frequency.

TABLE 1. OUTFLOW MAGNITUDE AND FREQUENCY FROM 1952 THROUGH 1978 BY CUMULATIVE NUMBER OF EVENTS PER MONTH AT COWANESQUE LAKE

	Apr	May	Jun	Jul	Aug	Sep	Oct
> 4,000 cfs	2		2			2	
3,500-4,000 cfs			1			1	
3,000-3,500 cfs		2	1				
2,500-2,500 cfs	1	1	2				
2,000-2,500 cfs	5	5	2				
1,500-2,000 cfs	2	2	1				2
1,000-1,500 cfs	4	9		2			2
850-1,000 cfs	2	2	4				3
650-850 cfs	8	8	7	1	2	1	1

Step 8. Develop a Realistic Release Scenario and Determine the Deviations from the Objectives.

A release scenario is developed to estimate the potential deviations from the objectives. The scenario includes various releases in the flow ranges from the initial SWS capcity to the maximum reasonable discharge. This scenario is applied each month from May through October and estimates of the deviations using the model results (profile data) are made. Table 2 through 4 exhibit the estimated resultant downstream temperature at Cowanesque Dam in May, June, July and August. This sample evaluation used temperature, but we can estimate the deviation of other parameters if they are of importance.

TABLE 2. DOWNSTREAM TEMPERATURE WITH DIFFERENT DISCHARGES AND SELECTIVE WITHDRAWAL SYSTEM CAPACITIES

MAY

Discharge (cfs)	Target Temperature	SWS Capacity 650 cfs Downstream Temperature	SWS Capacity 850 cfs Downstream Temperature	SWS Capacity 1,000 cfs Downstream Temperature
4,000	17.0°C	7.8°C	8.3°C	8.8°C
3,500	17.0°C	8.0°C	8.7°C	9.1°C
3,000	17.0°C	8.4°C	9.1°C	9.7°C
2,500	17.0°C	8.9°C	9.7°C	10.4°C
2,000	17.0°C	9.6°C	10.7°C	11.5°C
1,500	17.0°C	10.8°C	12.2°C	13.3°C
1,000	17.0°C	13.2°C	15.4°C	17.0°C
850	17.0°C	14.4°C	17.0°C	17.0°C
650	17.0°C	17.0°C	17.0°C	17.0°C

*ASSUMED: Surface Temperature 17.0°C
Bottom Temperature 6.0°C

Step 9. Evaluate Impacts on Water Quality With the Maximum Deviations.

It is very important to know the impact of the deviations. Literature reviews can usually provide some estimate of the impacts of various degrees of violations. For example, a sudden temperature drop of 12°C from acclimated temperature usually begins fish mortality; at a pH below 5.5 only a few organism can survive; a DO less than 4 mg/l is fatal for trout, etc. Compare these critical deviations or concentrations to the maximum deviation and evaluate the impacts.

TABLE 3. DOWNSTREAM TEMPERATURE WITH VARIOUS DISCHARGES AND SELECTIVE WITHDRAWAL CAPACITIES

JUNE

Discharge cfs	Target Temperature	SWS Capacity 650 cfs Downstream Temperature	SWS Capacity 850 cfs Downstream Temperature	SWS Capacity 1,000 cfs Downstream Temperature
4,000	22.0°C	9.4°C	10.2°C	10.8°C
3,500	22.0°C	9.8°C	10.6°C	11.3°C
3,000	22.0°C	10.3°C	11.3°C	12.0°C
2,500	22.0°C	10.9°C	12.1°C	13.0°C
2,000	22.0°C	10.9°C	12.1°C	13.0°C
1,500	22.0°C	13.5°C	15.5°C	17.0°C
1,000	22.0°C	16.8°C	19.8°C	22.0°C
850	22.0°C	18.5°C	22.0°C	22.0°C
650	22.0°C	22.0°C	22.0°C	22.0°C

*ASSUMED: Surface Temperature 22.0°C
Bottom Temperature 7.0°C

TABLE 4. DOWNSTREAM TEMPERATURE WITH VARIOUS DISCHARGES AND SELECTIVE WITHDRAWAL CAPACITIES

JULY AND AUGUST

Discharge (cfs)	Target Temperature	SWS Capacity 650 cfs Downstream Temperature	SWS Capacity 850 cfs Downstream Temperature	SWS Capacity 1,000 cfs Downstream Temperature
4,000	23.5°C	10.6°C	11.4°C	12.0°C
3,500	23.5°C	11.0°C	11.9°C	12.6°C
3,000	23.5°C	11.5°C	12.5°C	13.3°C
2,500	23.5°C	12.2°C	13.4°C	14.4°C
2,000	23.5°C	13.2°C	14.8°C	16.0°C
1,500	23.5°C	14.9°C	17.0°C	18.7°C
1,000	23.5°C	18.4°C	21.6°C	23.5°C
850	23.5°C	20.2°C	23.5°C	23.5°C
650	23.5°C	23.5°C	23.5°C	23.5°C

*ASSUMED: Surface Temperature 24.0°C
Bottom Temperature 8.0°C

Tables 2 thru 4 show the resultant downstream temperature at Cowanesque Lake from various SWS sizes under various temperature and flow conditions. The 650 c.f.s. system causes a maximum temperature deviation of 12.6°C in June and 12.9°C in July and August when the project releases the maximum reasonable discharge (4,000 cfs). A maximum deviation greater than 12°C starts fish mortality (L_T 50) according to the research. Consequently, the SWS capacity of 650 cfs is too small. This project needs at least a 1,000 c.f.s. or larger SWS capacity.

The question may be asked why we use the maximum temperature deviation of July and August for the evaluation when Table 1 shows that the maximum reasonable discharge has never occured in July and August at Cowanesque in the 26 year record. By looking at an adjacent project, F.E. Walter Dam, which is located only 100 miles East from the Cowanesque Lake site and has longer hydrological record (50 years). It is found that the maximum reasonable discharge can be experienced in July and August quite frequently (Table 5). That means that the Cowanesque Lake has a high probability to reach the maximum reasonable discharge in July and August at some future time. Therefore, the evaluation included the impacts due to the maximum deviations of July and August.

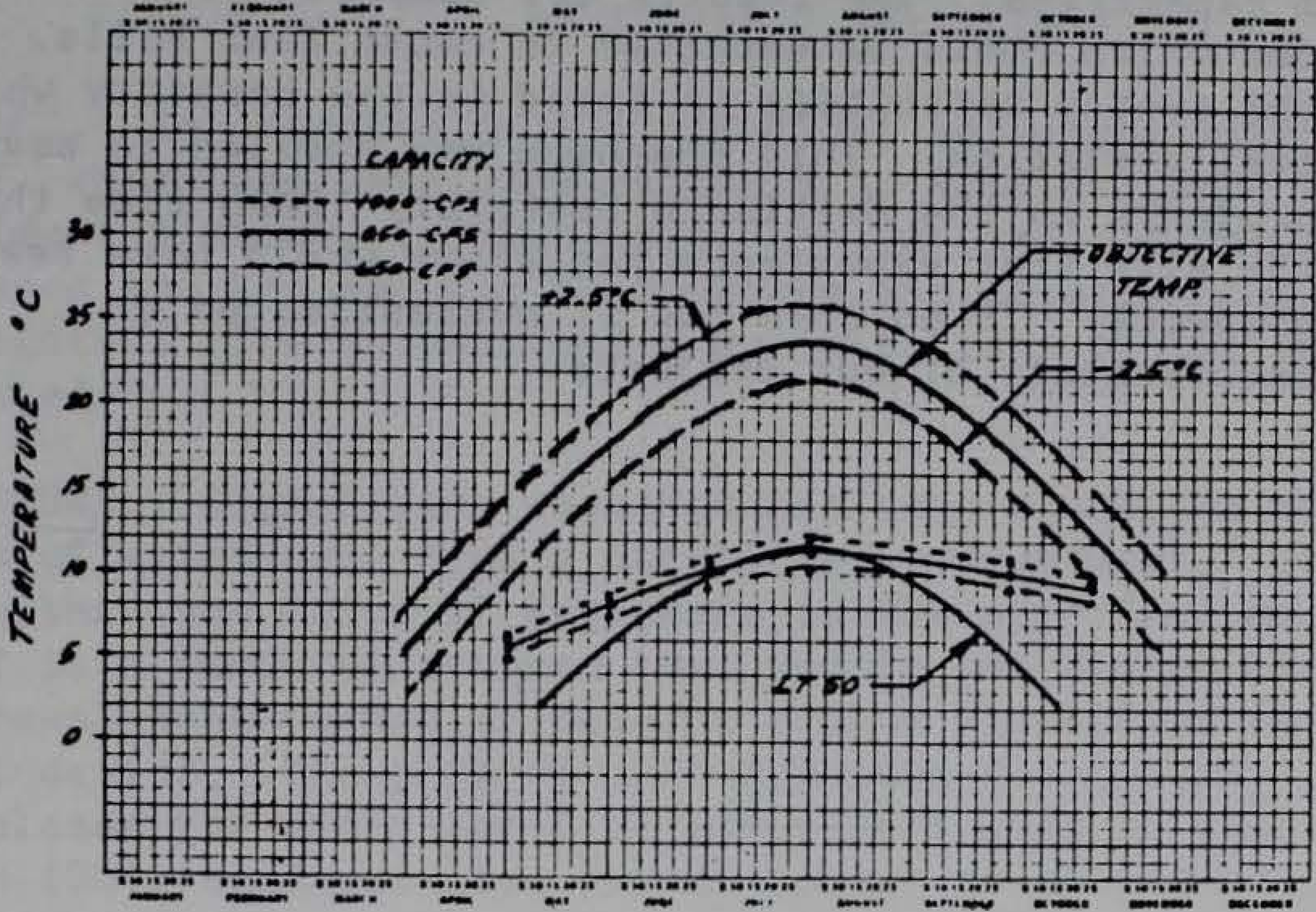
TABLE 5. OUTFLOW MAGNITUDE AND FREQUENCY FROM 1927 THROUGH 1977 BY CUMULATIVE NUMBER OF EVENTS PER MONTH AT F. E. WALTER DAM

CFS	15-30 April	May	June	July	August	September	October
> 8500				1	4		1
7001-8500			1	1			
6001-7000				1		1	
5001-6000			1				
4001-5000				1			2
3001-4000	7	2	1	2		1	2
2001-3000	11	12	4	3	4	3	2

Step 10. Estimate maximum deviations with different capacities and evaluate their impacts if necessary.

Additional system capacities are tested to find a proper capacity. For Cowanesque Lake, 850 cfs and 1,000 cfs were arbitrarily chosen to evaluate their impacts. Tables 2 through 4 show the results. The downstream temperature is significantly increased, especially in the flow range from 850 cfs to 1,500 cfs. The frequency of this flow range is very high (Table 1). The maximum temperature deviation is 11.8°C in June and 12.1°C in July and August with the 850 cfs capacity; 11.2°C in June and 11.5°C in July and August with the 1,000 cfs capacity. Figure 1 shows a summarized result of the capacities versus the maximum temperature deviations. The maximum temperature deviation with the capacity of 650 cfs is below the L_T 50 in June, July and August, which means a good chance of fish kill for each violation. The maximum temperature deviation with the capacity of 850 cfs is below the L_T 50 in July and August but the maximum temperature deviation with the capacity of 1,000 cfs is always above the L_T 50.

Figure 1. Expected downstream temperature with 3 capacities when the maximum reasonable discharge of 4000 cfs is released from Cowanesque Lake.



Step 11. Select a Final SWS Capacity.

A final SWS capacity is chosen from one of the capacities evaluated. The final capacity must be able to avoid critical impacts in the lake and downstream.

This selected capacity must also be evaluated for its impacts on other parameters such as DO, pH, iron and manganese, etc. If the capacity does not have critical impacts on other parameters, it can safely be adopted as the right capacity for the project. The numerical model should be rerun using this final capacity to verify the results.

CONCLUSION

A method for determining SWS capacity was recently developed by the Baltimore District. This method evaluates the expected impacts on water quality when the SWS ability to control release quality is exceeded. This can occur under normal flow operations as well as flood flow operations. The capacity needed to avoid disastrous consequences is evaluated by this method. The procedure for determining capacity is to analyze and to prioritize the water quality control objectives at the project. This includes an evaluation of the effects of the project on water quality in the reservoir and downstream under all anticipated or likely configurations of operation. A case study is used to illustrate the procedure step by step. The example considers downstream water temperature control as the primary concern. Downstream temperature objectives, monthly flow distribution and maximum reasonable discharge during flood flow and normal flows are analyzed. Water temperature profiles from a thermal model are used to estimate downstream temperature deviation extremes under various flow control operations using several system capacities. The results are compared to temperature deviations which could be expected to cause fish kills. The final selection of the system capacities is based on the capacity which can avoid this critical deviation. This approach was applied to several undersized SWSs in projects in the Baltimore District. Each time the approach predicted the size that experience has shown would have been adequate for those projects. In no case has it overestimated or underestimated the requirements of our projects.

SELECT: The Numerical Model

by Steven C. Wilhelms*

ABSTRACT. An overview of the numerical selective withdrawal model SELECT, which was developed at the U.S. Army Engineer Waterways Experiment Station, is presented. Its purpose and use are briefly discussed and the various subroutines and solution techniques are highlighted. The assumptions and limitations of the program are presented.

BACKGROUND. Significant research has been conducted on the characteristics of withdrawal from a stratified impoundment.[4] The results of this work within the Corps of Engineers is the numerical model SELECT[1,3]. SELECT is a computer program that models one-dimensionally the withdrawal zone formed by release from a stratified reservoir through an outlet device. The program also computes the quality characteristics of the release for user-specified parameters such as temperature, dissolved oxygen (DO), conductivity, and dissolved or suspended solids. The remainder of this paper is devoted to describing SELECT, its subroutines, capabilities, and limitations.

PROGRAM PURPOSE. SELECT, as stated, is a one-dimensional model of withdrawal from a stratified impoundment. It computes the vertical distribution of withdrawal based upon a user-specified density profile (usually input as temperature). SELECT will also compute, based on this withdrawal distribution, release water quality when given the vertical distribution of the quality parameter of interest. SELECT is not a water quality model nor a thermal simulation model. It does not model any of the chemical, biological, meteorological, or hydrological processes that are ongoing in a reservoir. Its purpose is to compute withdrawal characteristics.

PROGRAM METHODOLOGY. SELECT divides the reservoir pool into horizontal layers of user-specified thickness. Each layer is assigned a density (as well as temperature and quality(s)). The withdrawal distribution induced by the release varies only because of the density stratification. Since it is assumed that the density varies only in the vertical dimension, the withdrawal only varies vertically--hence, the one-dimensionality of the program.

SELECT computes the limits of withdrawal, which are defined as the vertical locations in the reservoir beyond which water is not withdrawn for release. The elevation of maximum velocity is then determined and

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the distribution of velocities within the withdrawal zone is computed. This velocity profile is "normalized," that is, the maximum velocity is set equal to 1.0 and the other velocities are less than 1.0. The normalized velocity profile is "scaled" to predict the reservoir withdrawal profile. Thus the velocities computed by SELECT are not actual withdrawal velocities, but the velocities required to withdraw the user-specified discharge.

Two concepts should be introduced that are used in SELECT: (a) the theoretical limit[3,4] of withdrawal and (b) the withdrawal angle[4,6]. The theoretical limit of withdrawal is computed in SELECT when boundary interference occurs. This is an analytically based technique that permits computation of limits when interference exists (Figure 1). Withdrawal angle is the angle in a horizontal plane through which fluid can be withdrawn if the outlet is located on the face of the dam with no lateral restrictions, then the withdrawal angle would be Π or 180° (Figure 2a). If the outlet is located at the abutment of the dam, then the withdrawal angle would be Π/2 or 90° (Figure 2b).

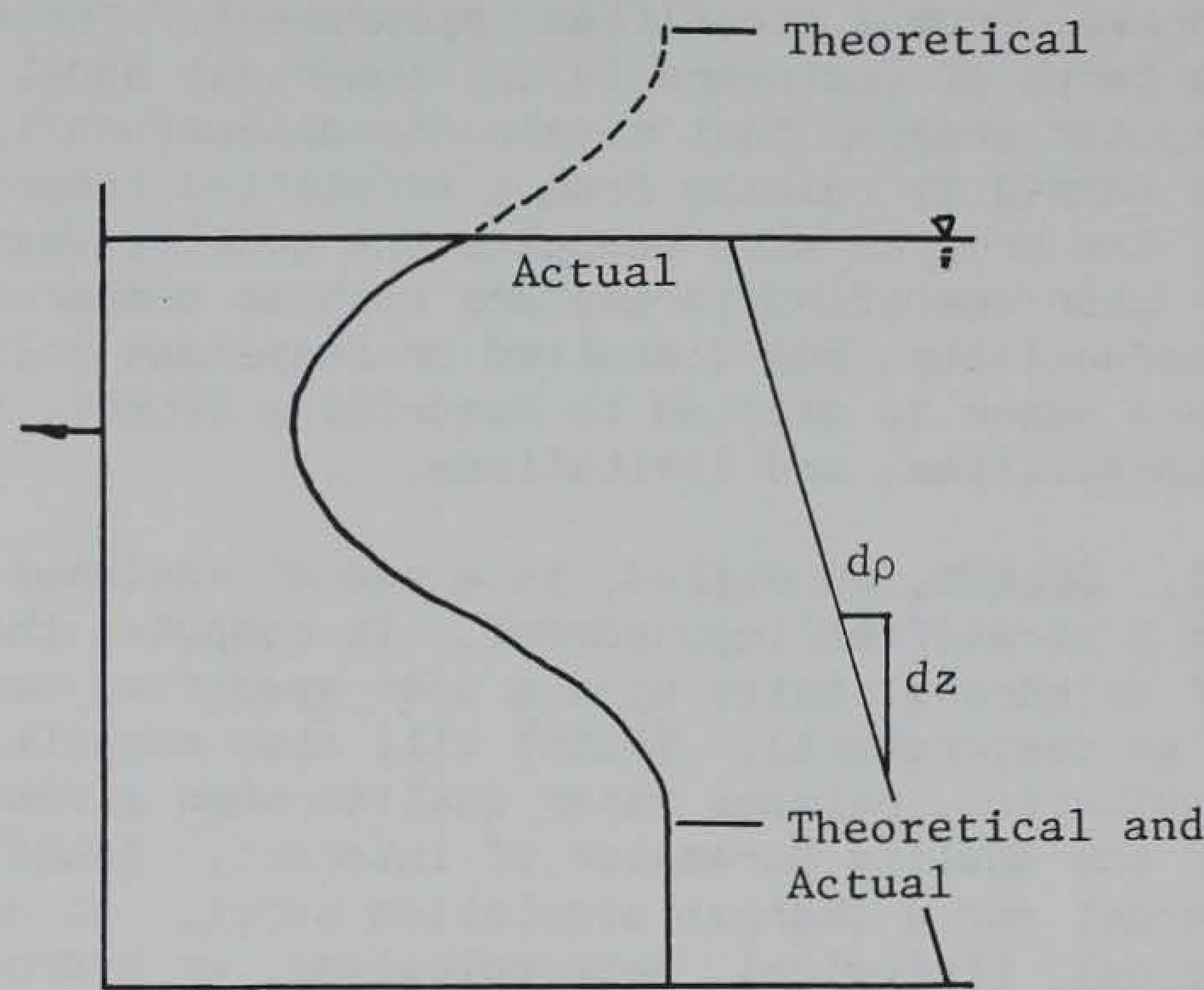

Figure 1. Theoretical and Actual Withdrawal Limits

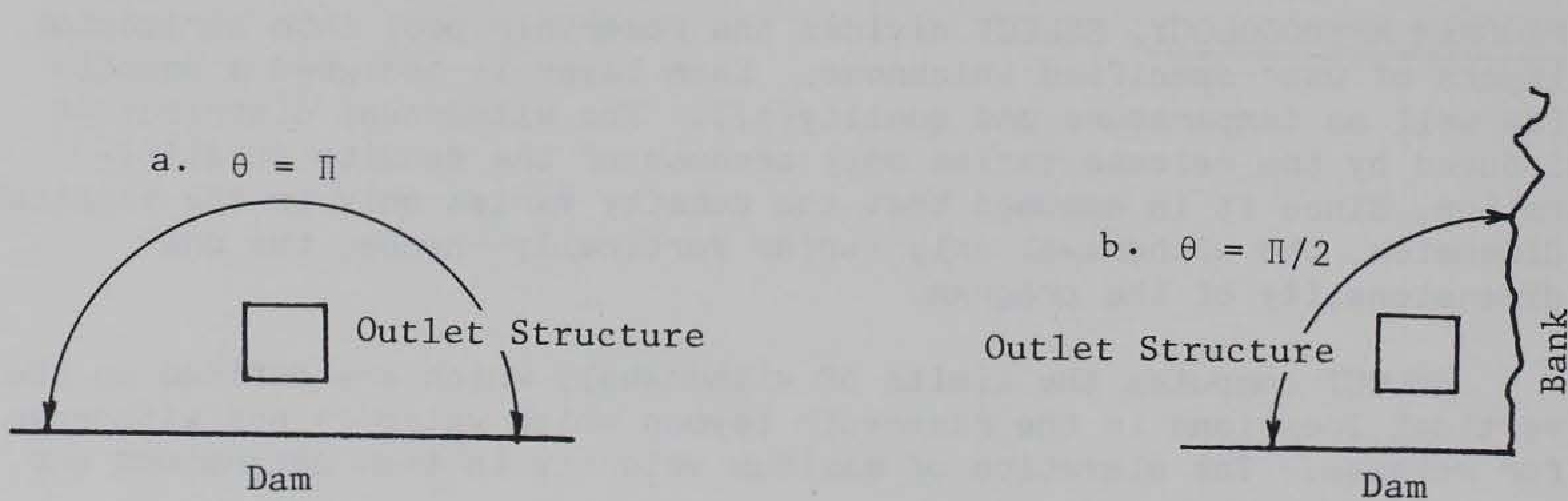

Figure 2. Withdrawal Angle

OVERVIEW OF PROGRAM. The equations, which are solved by SELECT to determine the withdrawal limits, incorporate the above concepts. They are transcendental in nature, which means they cannot be solved directly. Therefore an iterative technique must be employed to obtain their solution. Computation of the withdrawal profile and release temperature and qualities is accomplished with several subroutines. In addition to the computational subroutines, input and output routines assist the user. A brief description of the subroutines and their sequence of execution is provided in the following paragraphs.

SELECT's MAIN program controls the execution of the other routines. The subroutine XREAD is called first and program control data, such as the number of data sets, are read. XREAD is then called again and data describing the reservoir, outlet geometry, release rate, and vertical distribution of temperature (or density) and other water quality parameters are read.

The subroutine INTERP is called to calculate the value of temperature (or density) or other quality parameter for each layer of the reservoir. INTERP linearly interpolates between the vertically distributed values that were just input to obtain parameter values at the midpoint elevation of each layer. SELECT bases all of its computations on the profiles resulting from this interpolation.

The subroutine OUTVEL controls the routines that compute the withdrawal characteristics. Depending upon whether a weir or port is the outlet device, OUTVEL calls either VWEIR or VPORT, respectively. These two algorithms generate withdrawal zone limits and compute the withdrawal rate from each layer within the withdrawal zone. If two outlets are being operated and the withdrawal zones overlap[2,6] the subroutine SHIFT is called to modify the withdrawal zone appropriately. Using the information generated from these subroutines, OUTVEL calculates the release quality by applying flow-weighted averaging to the quality profiles.

OUTVEL assumes that the quality parameters are conservative, i.e., their release concentration or level is unaffected during release. For DO, this may not be the case, since reaeration may occur during flow passage through the outlet works. The subroutines AERATE and VENTING respectively account for oxygen uptake due to natural reaeration in the outlet works[5] or turbine venting[7] for hydropower releases. Implementation of these two algorithms is a user option.

The results of all the computations are presented to the user by subroutines XPRINT and DVPLOT. XPRINT provides a tabular display of the withdrawal limits, release temperature and density, total release rate, and profiles of temperature, density, withdrawal, and qualities. DVPLOT generates a graphical display of the withdrawal zone and density structure.

The computational sequence outlined in the previous paragraphs, and shown in Figure 3, is repeated for each data set of interest. Thus several stratification conditions with different release rates may be investigated with a single execution of SELECT. The release values of

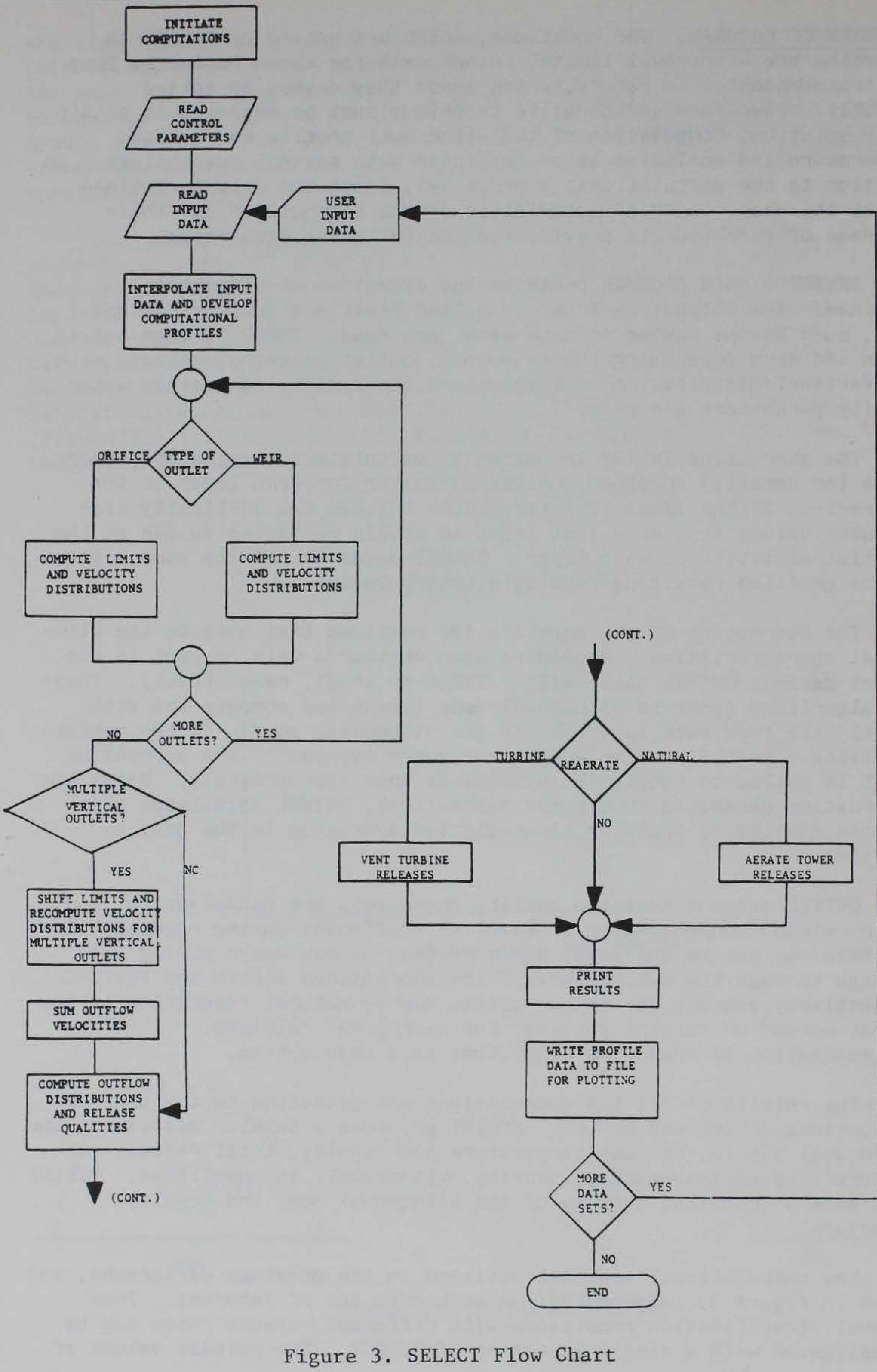

Figure 3. SELECT Flow Chart

various chemical stratifications (DO, pH, conductvity, etc.) may be predicted. There are, however, some assumptions and corresponding limitations on the application of SELECT. These are discussed briefly in the following paragraphs but you are referred to Davis, et al.[3] for further details.

ASSUMPTIONS AND LIMITATIONS. In general, if the outlet and approach geometry are simple, the results produced by SELECT will be accurate. However, if the outlet is complex, the assumptions inherent in the equations and theory of SELECT may be violated. The user of SELECT should be aware of these assumptions and how they impact the accuracy of the results. The following gives a brief listing of the assumptions and corresponding limitations on SELECT.

a. Geometry of Ports - Point sink outlet geometry is assumed, i.e. the orifice geometry has no effect on the withdrawal zone as long as the dimensions of the outlet are small relative to the withdrawal zone thickness.

b. Impoundment Width - The width of the reservoir approaching the outlet is assumed to be greater than the thickness of the withdrawal zone, Narrow approaches may cause lateral constrictions that force the withdrawal zone to thicken compared with the unrestricted situation. If this assumption is violated, the predicted withdrawal zone thickness will be less than actual.

c. Approach Path - The approach to the outlet is assumed free of obstruction. For example, topographic interference, such as a ridge just upstream of the outlet works, may interfere with the formation of the withdrawal zone. For some discharges, the ridge may control the withdrawal characteristics; for others, the outlet works may control.

d. Approach Curvature - The approach to the outlet is assumed relatively straight. If the approach is curved, the withdrawal zone prediction may be inaccurate because of the bending of streamlines (local acceleration of flow).

e. Multiple Horizontal Ports - User judgment is requisite for the application of SELECT when multiple horizontal ports are operated. If the ports are closely spaced, the point sink assumption may still be valid. Further, if the outlets are spaced far apart and do not hydrodynamically interact, the point sink assumptions may still be valid. In the transition between the closely spaced to widely spaced ports, the predictions may be inaccurate because withdrawal through one outlet interacts with and modifies the withdrawal zone formed by the other.

f. Weir Crest Elevation - It is explicitly assumed for weir withdrawal that the weir crest elevation is above the thermocline and that stratified flow does not occur over the weir and on downstream to the outlet.

g. Multilevel Port Operation - SELECT assumes independent flow control for each port when ports at different elevations are operated.

h. Simultaneous Port-Weir Operation - SELECT assumes separate and independent flow control for each device if a port and a weir are operated simultaneously.

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BLENDING IN A SINGLE WET WELL

Stacy Howington*

ABSTRACT. The concept of blending various qualities of water in a single wet well is presented. Potential application of the blending concept is discussed. The author presents a simplified theoretical approach to describe the mechanics of the blending phenomenon. A brief discussion on the limitations (because of the assumptions made in theory derivation) of practical application of the theory is presented. Examples of the current use of blending are presented highlighting these limitations and identifying areas that are in need of additional research.

INTRODUCTION. The technique of selective withdrawal has been used for years to control the quality of water released from a stratified body of water such as a reservoir in summer or early fall. Water is withdrawn from one or more levels in the stratified pool either to produce a desired release water quality or to conserve or remove a certain water quality resource in the reservoir. In many cases, sufficient quantities of the desired resource are available in the reservoir and only one level of withdrawal is needed to meet the water quality and flow rate objectives.

However, one level of withdrawal is not always adequate. Flow rate objectives might require that multiple port elevations be used. Additionally, if the port locations are not appropriate to withdraw the desired water or insufficient amounts of the desired resource are present in the reservoir, blending of the individual level withdrawal qualities may be required. This blending has traditionally been accomplished by employing a dual wet well system. Water is withdrawn from the desired levels in the pool (one level of withdrawal per wet well) and mixed downstream of the separate flow controls which are at the service gates. This way the quantity of flow from each level of withdrawal can be controlled and blending occurs readily in the highly turbulent flow in the outlet works.

Separate flow control for each of the levels of withdrawal is not always possible. For example, the addition of hydropower to a selective withdrawal structure often shifts the flow control downstream to the turbines, which places all the ports upstream of a single control point.

Without independent flow control on the wet wells, a multiple wet well system is effectively limited to a single-wet-well type operation. The withdrawal characteristics of the ports are the same for blending and for traditional, individually controlled withdrawal, but the flow rates from each level of open ports in the stratified pool are not known in the single-flow-control blending mode.

If the processes which occur during blending with downstream flow control

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could be adequately described and operational guidance developed, several benefits might be realized. Hydropower might be added to existing selective withdrawal structures while maintaining selective withdrawal capabilities. This would decrease the opportunity for environmental degradation. Also, future selective withdrawal structures and selective withdrawal add-ons might be single instead of dual wet well designs which might reduce their construction costs.

THEORY. With some simplifying assumptions, blending in a single wet well can be examined theoretically. Consider the idealized case given in Figure 2. It is a single wet well structure with two ports and a perfect two-layer stratification. One port resides in each of the homogeneous layers and both ports are open. The outlet from the wet well is at the bottom.

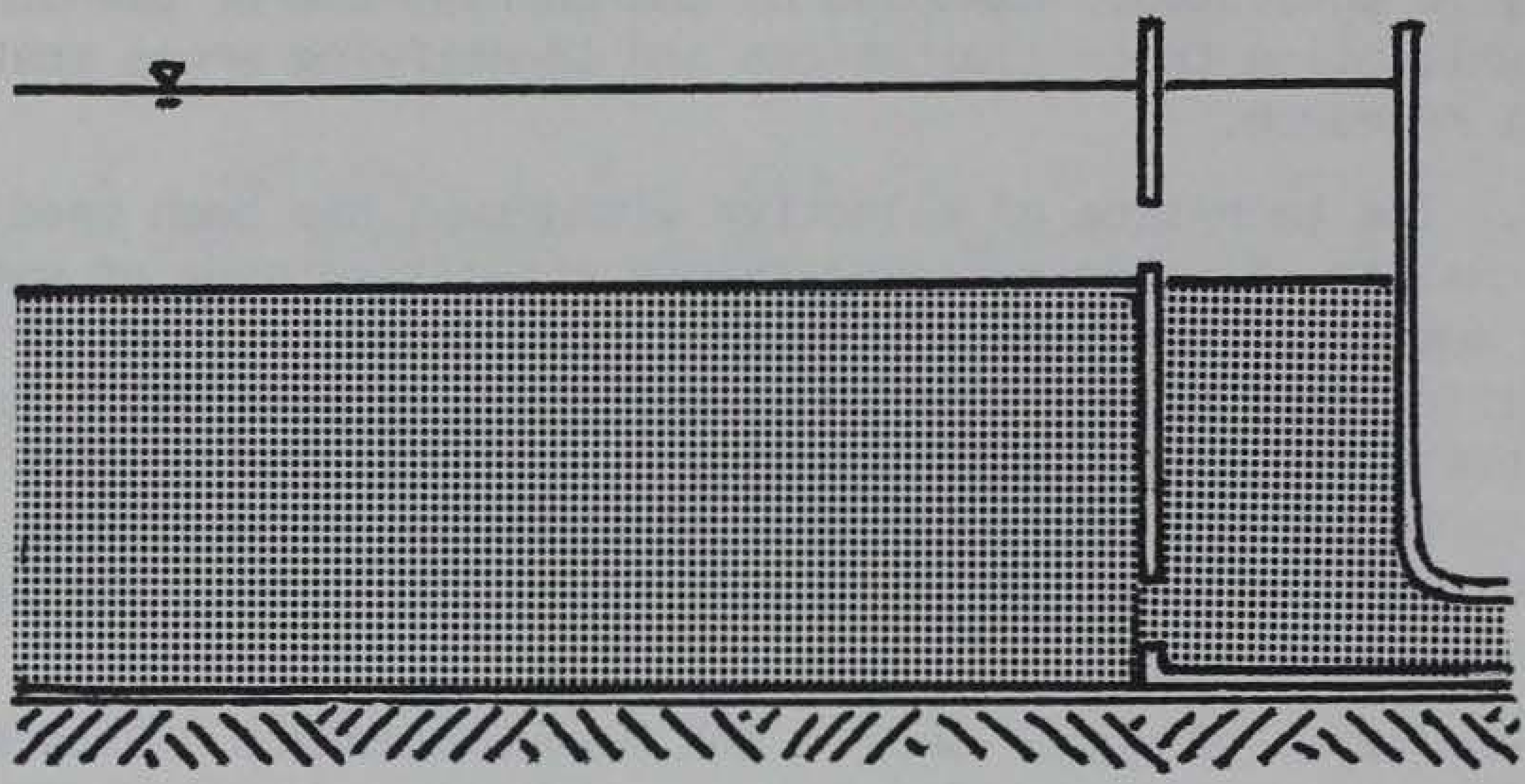

Figure 1. No Flow Condition

Under the "no flow" condition, the total discharge from the structure is zero. Stratification exists in the wet well as it does in the pool. As the intake tower service gate is opened slightly, flow begins through the lower port, but not through the upper port. This is due to the buoying up of the lighter water by the more dense water. This buoyant effect prevents withdrawal through the upper port at low discharges.

As flow enters the lower port, the energy loss across the port entrance is evidenced by a lowering of the thermocline in the wet well. Since the top port is open, the water surface elevation in the wet well will not change. As the thermocline is depressed in the wet well, water enters the top port to fill the void which is created by the dropping thermocline.

Once the thermocline in the wet well has been lowered to the top of the lower port, theoretically, a critical equilibrium has been reached. If the flow rate is increased, the head loss across the lower port will also increase and the thermocline in the wet well will be lowered into the flow entering the lower port. The buoyant forces will have been overcome and blending will occur. Therefore, the point at which this transition occurs is called the "critical" flow rate and this is the point at which "incipient blending" occurs. This is shown in Figure 2.

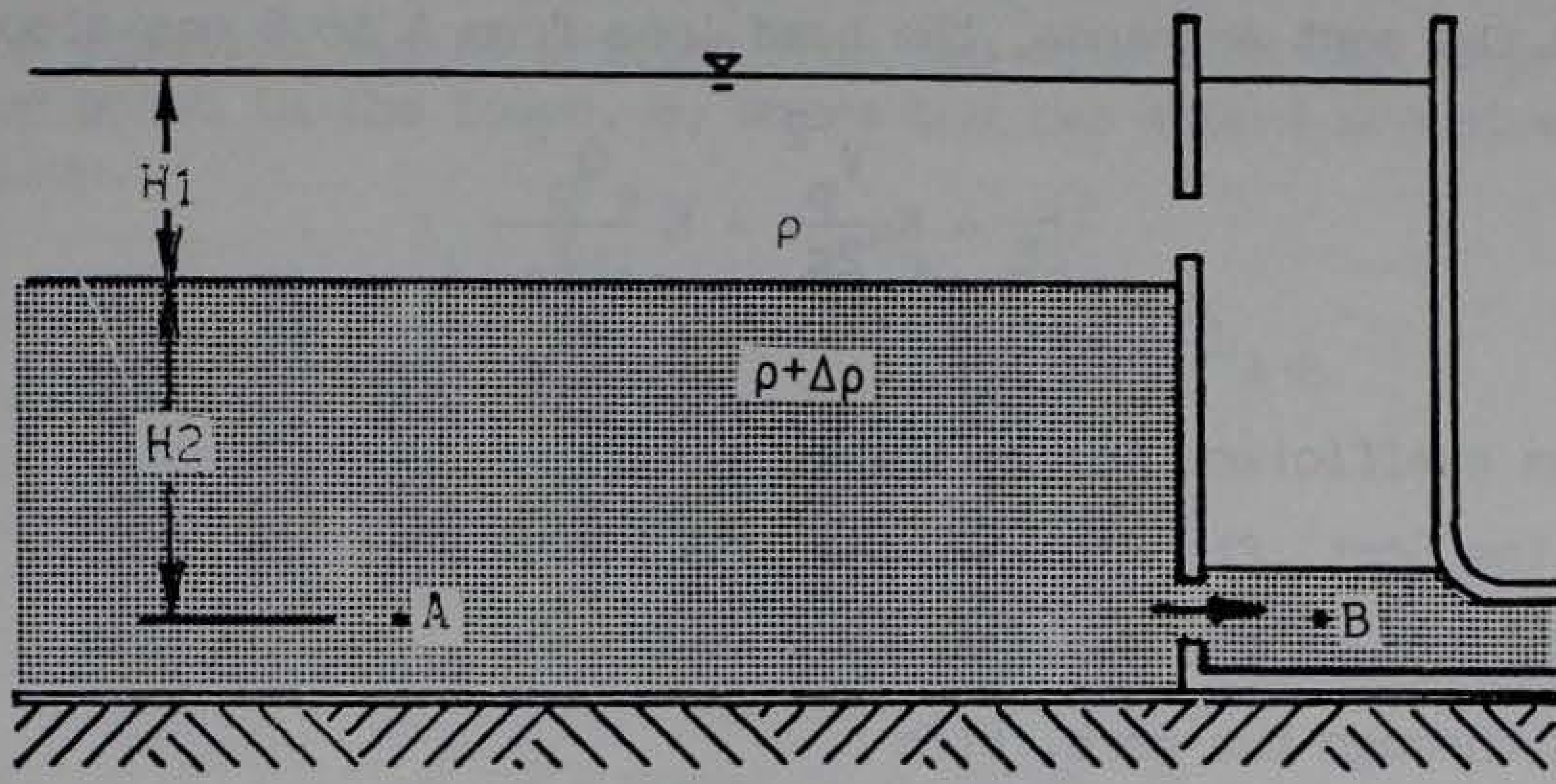

Figure 2. Incipient Blending

In order to analytically determine the theoretical critical flow rate, Bernoulli's equation is written from a point in the reservoir to a point inside the wet well at the lower port elevation. Bernoulli's equation, written from point A to point B states that

$$z_A + \frac{P_A}{\gamma} + \frac{V_A^2}{2g} = z_B + \frac{P_B}{\gamma} + \frac{V_B^2}{2g} + h_{L_{A-B}} \tag{1}$$

where

z_A, z_B = elevations of points A and B respectively referenced to a datum, ft

V_A, V_B = velocities at points A and B, respectively, ft/s

P_A, P_B = pressures at points A and B, respectively, lb/ft^2

g = gravitational acceleration, ft/s^2

γ = specific weight of the fluid, lb_f/ft^3

$h_{L_{A-B}}$ = head loss between points A and B, ft

The difference in the potential energy of the two points is zero and it is assumed that the difference in the kinetic energy of the two points is negligible. Therefore, the z and V terms in Equation 1 will drop out and the head loss from A to B is only a function of the pressure difference between points A and B. The following equation relates head loss to pressure differential.

$$h_{L_{A-B}} = \frac{P_A - P_B}{\gamma} = \frac{\rho g H1 + (\rho + \Delta\rho) g H2 - \rho g (H1 + H2)}{(\rho + \Delta\rho) g} = \frac{\Delta\rho H2}{\rho + \Delta\rho} \tag{2}$$

where

H1 = distance between the water surface and the thermocline, ft

H2 = distance between the thermocline and point A, ft

ρ = density of the upper layer, lb_m/ft^3

$\Delta\rho$ = density difference between the two layers, lb_m/ft^3

Assuming that the total energy lost between points A and B for this condition occurs across the port entrance, the head loss from A to B can also be written as

$$h_L = K \frac{V_p^2}{2g} = K \frac{Q_p^2}{A_p^2 2g} \qquad (3)$$

where

K = head loss coefficient of the intake

A_p = area of the port, ft^2

Q_p = flow rate through the port, ft^3/s

V_p = velocity through the port, ft/s

The critical flow rate for these conditions can then be calculated from Equations (2) and (3) by finding the head loss required to depress the thermocline in the wet well to the top of the lower port as follows:

$$Q_c = \sqrt{\frac{2gAp^2}{K} \cdot \frac{\Delta\rho H2}{\rho+\Delta\rho}} \qquad (4)$$

where

Q_c = critical flow rate, ft^3/s

The critical flow rate, from Equation 4, is directly related to port area. Therefore, if the lower port area is controllable by partially closing the gate at the port entrance, some control over the critical discharge may be gained.

From the equations, for all flow rates less than or equal to the critical flow rate, no flow contribution will be made by the upper port and the thermocline in the wet well will reach a stable elevation between the two port elevations. For all discharges greater than critical flow rate, flow will pass through both ports.

Once critical discharge is surpassed, the problem is no longer "Is flow coming from both port elevations?", but, "How much flow is coming from each port elevation?" This problem can also be approached from a theoretical standpoint.

The flow through the top port creates a head loss which is reflected by a water surface drop in the intake structure. This can be seen in Figure 3.

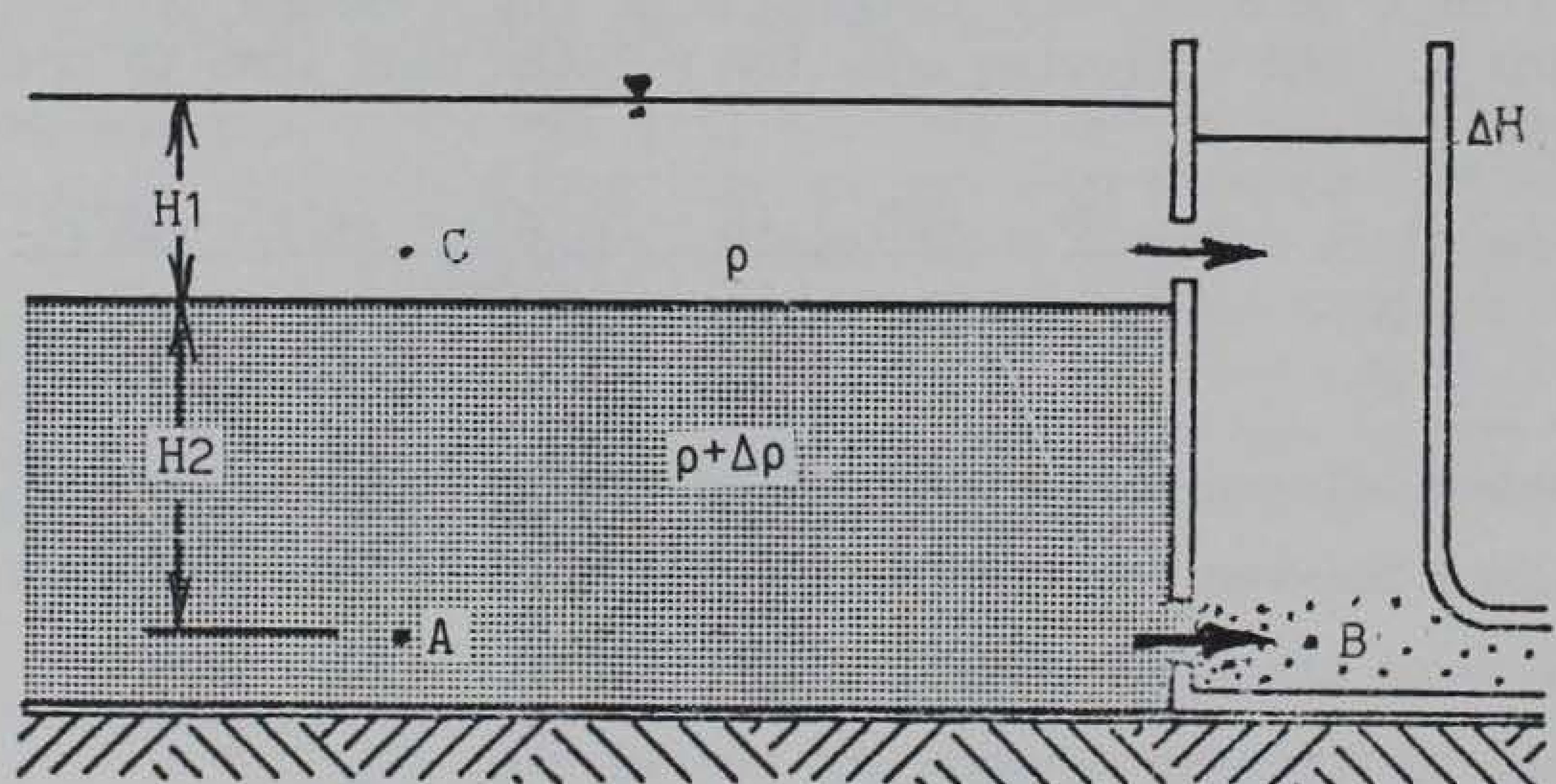

Figure 3. Blending

Bernoulli's equation can be written from two points in the reservoir, A and C, to a common point in the tower, B, where the two waters are mixed. These equations follow.

$$Z_A + \frac{P_A}{\gamma} + \frac{V_A^2}{2g} = Z_B + \frac{P_B}{\gamma} + \frac{V_B^2}{2g} + h_{L_{A-B}}$$
$$Z_C + \frac{P_C}{\gamma} + \frac{V_C^2}{2g} = Z_B + \frac{P_B}{\gamma} + \frac{V_B^2}{2g} + h_{L_{C-B}} \tag{5}$$

where

Z_C = elevation of point C referenced to a datum, ft

P_C = pressure at point C, lb/ft^2

V_C = velocity at point C, ft/s

$h_{L_{C-B}}$ = head loss between points A and C, ft

The flow through each individual port is directly related to the head loss experienced by a fluid particle as it travels from the reservoir to the point in the tower where mixing occurs. If it is assumed that the head loss through the ports is large compared to the other losses, the flow through each port can be easily estimated. This can be shown in the following equation.

$$\frac{kQ_U^2/A_U^2}{kQ_L^2/A_L^2} = \frac{h_{LU}}{h_{LL}} \approx \frac{h_{L_{C-B}}}{h_{L_{A-B}}} = \frac{\Delta H}{\Delta H + \frac{\Delta\rho H2}{\rho+\Delta\rho}} \tag{6}$$

where

Q_U, Q_L = flow rate through the upper and lower ports, respectively, ft^3/s

h_{LU}, h_{LL} = head loss through the upper and lower ports, respectively, ft

A_U, A_L = area of the upper and lower ports, respectively, ft^2

ΔH = water surface drop in the wet well, ft

Equation 6 indicates that the density stratification can impact the flow distribution between the port elevations. It can be seen that with large flow rates, the ΔH term will be large and the density impact will be decreased. Under weakly stratified conditions, the flow ratio will essentially be equal to a ratio of the port areas.

LIMITATIONS OF THEORY. The theory presented makes several simplifying assumptions which prevent its direct application to physical situations. These assumptions include: (a) the stratification consists of two, separate, homogeneous layers of water, (b) the water entering the lower port does not cause mixing as it deflects off the back wall of the tower, (c) the energy losses other than entrance losses are negligible, (d) the lower port velocity jet does not cause hydraulic blockage of the wet well.

Actual reservoir stratification is seldom close to and never reaches the perfect two-layer system used in the theoretical demonstration. The second assumption concerning the velocity jet impingement also serves to separate the theoretical predictions from physical results. In Equation 1, the velocity inside the tower was assumed to be small but in actuality, it may be significant. The jet from the lower port, as it deflects off the back wall of the tower, can cause mixing and thereby blending at flow rates much lower than the theoretically computed critical discharge.

The third assumption ignores friction losses in the tower which were assumed to be negligible. The last assumption was that the jet would not cause hydraulic blockage of the tower. This is a problem which could prevent flow from the upper port although the discharge may be higher than critical flow.

APPLICATION. The Lost Creek Dam in southern Oregon has a single wet well intake structure with a total of 12 ports at 5 levels over a vertical distance of 280 ft. It also has hydropower at the downstream end. Blending in this single wet well has been occurring for at least 6 years and the outflow data confirm that blending is occurring.

Blending has also been observed in two generic models and in a scale model of the Lost Creek structure. In these models, the critical discharge occurred at a much lower discharge than theory predicted. The mixing due to the velocity jet impinging on the back wall was plainly visible. Hydraulic blockage has not been observed in any of these models. An accurate description of the impacts of density stratification on multi-level, single wet well withdrawal will require much more investigation into the mixing characteristics in the wet well and the effects of density on flow distribution between port elevations.

DESIGN OF SELECTIVE WITHDRAWAL INTAKE STRUCTURES

by Jeffery P. Holland*

ABSTRACT. Presented herein is an overview of the general methodology used by the US Army Corps of Engineers in the design of selective withdrawal intake structures. Considered are the types of structures generally used by the Corps; the computation of the distribution of withdrawal for a given intake from a density-stratified reservoir; the optimum location of selective withdrawal intakes; and the hydraulic constraints that must be satisfied for effective structure flow control.

INTRODUCTION. As a result of increasing public awareness and State and Federal legislation, water resources projects are being operated with a greater priority on water quality considerations. The use of a reservoir outlet works incorporating multilevel selective withdrawal structures is a primary method for the control of reservoir release quality. These structures release water from various vertical strata in a density-stratified lake, thereby allowing, through blending or direct release, greater water quality control. It is therefore imperative that the selective withdrawal intakes be placed in such quantity and location as to maximize the control of reservoir release quality over a wide range of hydrological, meteorological, and operational conditions. However, selective withdrawal structures must also be designed to satisfy a number of hydraulic conditions that if unsatisfied by the design, could mitigate or negate selective withdrawal capability.

The purpose of this paper is to overview the general methodology used by the US Army Corps of Engineers (CE) in the design of selective withdrawal structures. This paper considers four basic questions concerning selective withdrawal structure design: (a) what general types of selective withdrawal intake structures have been built; (b) from what regions of a stratified impoundment will water be withdrawn for a given intake geometry, capacity, and location; (c) how can intakes be effectively placed to withdraw the quality of water desired for downstream release; and (d) what are the hydraulic constraints that must be satisfied to ensure the proper operation of the intakes. Other aspects of selective withdrawal intake structure design, such as the computation of optimum construction, will not be discussed. The reader is referred to other literature for discussions of these topics (Office, Chief of Engineers 1980; Fontane, Labadie, and Loftis 1981).

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TYPES. Selective withdrawal structures fall into three general types: (a) inclined intake tower on a sloping embankment; (b) freestanding intake tower, usually incorporated into the flood-control outlet facilities of embankment dams; and (c) face-of-dam intake tower, constructed as an integral part of the vertical upstream face of a concrete dam. The appropriate type of intake structure for a given project depends on the number of considerations, including project purposes, water quality objectives, and construction materials. Types (b) and (c) predominate at CE projects.

The most common type of selective withdrawal structure is (b), the freestanding intake tower. Three general types of freestanding intake tower predominate. The first consists of a flood-control system and a water quality control system with intakes on a single collection well. This type is generally appropriate for shallow reservoirs with minimum stratification where single intake operation is anticipated and the blending of flows between intakes at differing elevations is not required.

The second type of freestanding selective withdrawal structure is the dual wet-well structure that consists of a flood control system and two water quality collection wells. This type is generally appropriate for reservoirs that are expected to exhibit strong stratification and for which anticipated operations for water quality objectives indicate that the capability for blending between intakes is desirable. In both the single and dual collection well systems, the selective withdrawal capacity is generally equal to or less than the flood-control capacity. Additionally, the collection wells can be either separate from or integrated into the flood-control system. This distinction can be made by whether the flood-control and water quality control systems have separate or common flow-control gates.

The third type of freestanding selective withdrawal structure is one through which all or most discharges, except spillway, can be released (Bucci 1965). This type of system may need to have low- and high-flow capabilities. Many of the newer hydropower projects can selectively withdraw all power discharges.

WITHDRAWAL ZONE COMPUTATION. As just presented, there are a number of types of selective withdrawal structures, each with differing hydraulic, structural, and operational attributes. A common factor to each, however, is that each structure is designed to withdraw water of a specified quantity and quality from the reservoir in order to meet downstream and/or in-reservoir water quality requirements. Accomplishment of this objective, however, requires that a designer first be able to predict the vertical distribution of withdrawal that a given selective withdrawal intake will produce in a density-stratified impoundment for a particular set of hydrometeorological conditions. This distribution can tnen be used with a known reservoir quality profile to compute the quality of a reservoir release.

The work of Bohan and Grace (1973) at the US Army Engineer Waterways Experiment Station (WES) remains the basis upon which most of the selective withdrawal computations for density-stratified reservoirs are made in the CE. These investigators obtained generalized relationships that describe, for regular reservoir geometries, the vertical limits of the withdrawal zone and the subsequent normalized vertical velocity distribution produced by an intake (idealized as a point sink) for a given release flow, intake elevation, and reservoir density structure. Recently, at WES, Smith et al. (1985) have compared the work of Bohan and Grace (1973) with that of many other independent selective withdrawal investigators and have found a function form common to all that describes the vertical free limits of withdrawal from stratified impoundments for linear density stratification.

Two numerical procedures are available to model selective withdrawal for a fixed (steady-state) condition. The first code, SELECT, was developed from the work of Bohan and Grace (1973) and has received recent updates based on the work of Smith et al. (1985). SELECT predicts the vertical distribution of withdrawal from the reservoir and the outflow concentrations of specified water quality parameters (treated as conservative constituents) given the intake geometry, location, flow rate, and temperature and water quality profiles. SELECT is a general purpose code and can be accurately and easily applied for numerous cases. However, because of the fundamental assumptions upon which SELECT is based (i.e., simplified reservoir geometry; intake dimensions that are small compared with total pool depth and width), SELECT may not provide accurate predictions for complex outlet configurations.

For the more complex outlet configurations, a two-dimensional laterally averaged numerical hydrodynamic code, WESSEL, has been developed for analysis of stratified flow and selective withdrawal (Thompson and Bernard (1984)). Like SELECT, WESSEL could be used to evaluate selective withdrawal for a fixed condition. WESSEL, however, can more accurately determine the influence of geometry on selective withdrawal patterns through solution of the equations of motion with boundary-fitted coordinates. Still, three-dimensional physical models must often be used to study the stratified approach flow for highly site-specific complex withdrawal configurations. Physical models for selective withdrawal analyses are more accurate than numerical models but are more expensive to use. Like SELECT and WESSEL, each test for this type of physical model is run for a fixed condition. Such models have often been used to refine the withdrawal descriptions in SELECT (Loftis, Saunders, and Grace 1976; Dortch 1975; Smith et al. 1981) for water quality analyses, thereby incorporating the accuracy of physical modeling with the ease of a numerical approach.

INTAKE LOCATION. The most fundamental question that arises during the design of a selective withdrawal structure is where should the intakes be located, and how many intakes should be incorporated, so that operation of the structure best meets prescribed downstream water quality objectives. This objective is best accomplished through the use of numerical water quality models that simulate the effects of numerous hydrologic, meteorological, biological, chemical, and operational conditions on reservoir and release water quality over time. Within the CE, these models usually incorporate the SELECT algorithms for computation of withdrawal distributions. Until recently, these models were executed numerous times for various selective withdrawal intake configurations and input conditions until, based on judgment and experience, a satisfactory design was found. Recently, however, mathematical optimization techniques have been coupled with a reservoir simulation model to systematically obtain both the optimal number and location of selective withdrawal intakes for maintenance of prescribed downstream water quality objectives (Holland 1982; Dortch and Holland 1984). The utility of such a procedure is that the effectiveness of prospective selective withdrawal intake configurations with differing numbers of intakes can be compared over a wide range of hydrologic, meteorological, and operational conditions systematically rather than manually. Further, in certain instances, an optimized configuration of fewer intakes can meet downstream water quality objectives more effectively than a manually designed system with a greater number of intakes (Dortch and Holland 1984). Such an optimal design could, with fewer intakes needed for water quality maintenance, reduce operational complexity and construction costs.

HYDRAULIC DESIGN. Regardless of the methodology used to specify the location and number of selective withdrawal intakes required, various hydraulic design constraints must be satisfied in order to ensure that the structure will withdraw water as designed. It is possible to incorporate many of these constraints into reservoir water quality models (Dortch and Holland 1984). However, due to the site-specific nature of the purposes and objectives of CE projects, it would be difficult if not impossible to attempt to incorporate very specific hydraulic design guidance in such models. In general, the hydraulic design concepts that apply to outlet works will apply directly to selective withdrawal structures. General CE hydraulic design guidance for selective withdrawal structures and outlet works is given by the Office, Chief of Engineers (1980). Other sources of hydraulic information are reports of hydraulic model studies of specific selective withdrawal structures (i.e., Melsheimer and Oswalt 1969; Melsheimer 1969; Bucci 1965; and George, Dortch, and Tate 1980). For brevity, the reader is referred to the above references for information on the hydraulic design of selective withdrawal intake structures.

SUMMARY. Presented in this paper is an overview of the general methodology used by the US Army Corps of Engineers in the design of selective withdrawal intake structures. The design of these structures requires a multidisciplinary approach due to the inherent coupling of hydraulic, hydrologic, water quality, structural, and operational concerns. Although there are several different types of selective withdrawal structures within the CE, each of these has a common goal: the maintenance of some prescribed downstream release and in-reservoir water quality objectives. Effective maintenance of a prescribed water quality objective, however, requires that the selective withdrawal structure be designed so that an adequate number of intakes are appropriately located to allow flexibility of operation over a wide range of hydrometeorological conditions. Further, these structures must be designed to satisfy numerous hydraulic constraints to ensure that the quality and quantity of water desired can be physically withdrawn. The design of a structure that is environmentally and hydraulically efficient requires a thorough understanding of both the hydromechanics of density-stratified flow in reservoirs and the control of flow in hydraulic structures. The methodology presented herein makes direct use of engineering tools based on experience in these two areas for the design of selective withdrawal structures.

ACKNOWLEDGMENT. The information presented herein, unless otherwise noted, was obtained from research conducted under the US Army Corps of Engineers Civil Works Research and Development Program and from site-specific reimbursable studies. These investigations were performed by the US Army Engineer Waterways Experiment Station, Vicksburg, Miss. Permission was granted by the Office, Chief of Engineers, to publish this material.

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OPERATIONAL TOOLS: SELECTIVE WITHDRAWAL AND DAILY OPERATIONAL STRATEGY

Jeffery P. Holland[1] and Steven C. Wilhelms[2]

ABSTRACT The authors present the results of recent selective withdrawal research in the form of a general mathematical description of this stratified flow phenomenon. Results of past and present researchers were compared. Through symmetry arguments and the withdrawal angle concept, those results were reduced to a single expression. A new description for boundary interference, which often impacts the formation of the withdrawal zone, was explicitly included in the mathematical formulations. These results were incorporated into the computer code SELECT, a numerical model of withdrawal from a stratified impoundment. This model has been used extensively for long-term evaluation purposes in conjunction with reservoir simulation models. However, when coupled with a port-selection algorithm, the model has excellent potential as a tool for day-to-day decisions regarding hydraulic structure operation. The authors present an example of model application to provide guidance on outlet structure operation for maintenance of release water quality.

INTRODUCTION

As a concept, selective withdrawal is relatively simple. It is the capability to describe the vertical distribution of withdrawal from a density-stratified reservoir and then apply that capability at appropriate depths to be selective about the quality of water that is withdrawn. As an example, consider an outlet structure with a port relatively near the surface of the reservoir. Intuitively, it is reasonable to expect surface water to be withdrawn. However, the questions immediately arise: "Will release be all surface water? From how deep in the reservoir will water be withdrawn for release?" These questions are answered through an understanding of the hydrodynamics of selective withdrawal.

SELECT (1), a numerical selective withdrawal model, implements the analytical and experimental knowledge about selective withdrawal. It has been applied in several types of situations. One version of the numerical technique is a "stand-alone" computer code that has been used to estimate the quality of release water given the pool stratification conditions, discharge, and outlet configuration. The numerical technique has not, however, been widely used as an operational tool for guidance on hydraulic structure operation, especially regarding the operation of multilevel selective withdrawal structures. The SELECT model can be used to make day-to-day decisions regarding which outlets

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should be opened to achieve a desired quality downstream. It is the purpose of this paper to describe and demonstrate this application of the concept of selective withdrawal.

CONSOLIDATION OF RESULTS FROM PAST RESEARCH

The significance of selective withdrawal as a means for controlling water quality has grown with the increasing need for high quality water resources to meet demands for water supply, recreation, and wildlife. Selective withdrawal capabilities can often provide the operational flexibility to optimally respond to water quality demand. Because of these interests, research has continued into the processes that influence withdrawal and subsequent application of that knowledge.

In a survey of the literature, several expressions describing withdrawal are encountered that contain similar variables. However, there is considerable discrepancy among the analytical and experimental coefficients associated with these expressions. It is our contention that the effects of boundaries and the lack of symmetry consideration have contributed to this variability. These expressions characterize similar hydrodynamic situations differing only for boundary and stratification assumptions. Thus a relationship should exist that is common to several of the conditions and expressions.

Most descriptions of withdrawal are founded in the densimetric Froude number with a recommended coefficient. However, as mentioned, significant variability exists among these coefficients. Smith et al. (3) and Wilhelms et al. (4) show the development of a more generalized withdrawal description deduced by extending symmetry arguments and introducing the concept of "withdrawal angle" to obtain

$$F = K \frac{\Theta}{\pi} \tag{1}$$

where F is the densimetric Froude number, K is a coefficient dependent upon geometry of withdrawal, and Θ is the withdrawal angle in radians measured on a horizontal plane.

If a boundary (surface or bottom) interferes with the formation of the withdrawal zone established by releases from a stratified impoundment the generalized Equation 1 is inapplicable. Smith et al. (3) and Wilhelms et al. (4) show the development of an equation to describe withdrawal with arbitrary boundary interference. Few other attempts have been made to mathematically describe the withdrawal zone if boundary interference exists.

The improved withdrawal description that includes the withdrawal angle concept and the technique for determining the withdrawal zone when boundary interference occurs has been incorporated into the numerical selective withdrawal model SELECT. In addition, a subroutine was added to the model to simulate the operation of the selective withdrawal structure. The subroutine, named DECIDE, evaluates the thermal stratification of the reservoir, the downstream temperature objective, and the total flow to be released downstream. Based upon the operational constraints of the selective withdrawal structure, the subroutine determines the combination of selective withdrawal intakes to be operated and the flows to be released through those intakes such that the release temperature is as close as possible to the downstream temperature objective. The operational constraints of the selective withdrawal structure considered by the DECIDE subroutine include hydraulic constraints on the intake operations such as minimum and maximum allowable flows, intake geometry, number of wet wells, and floodgate capacity. Dortch and Holland (2) discuss this port selection routine in more detail.

In order to demonstrate the utility of the port-selection version of SELECT, hereafter referred to as SELCIDE, the operation of a hypothetical reservoir was predicted with the model. The remainder of this paper is devoted to discussing that application of SELCIDE to day-to-day decisions of structure operation.

CASE STUDY: LAKE FICTITIOUS, USA

Consider the hypothetical operation of the multilevel outlet tower on Lake Fictitious. The structure has two ports in each of two wet wells with a maximum combined release capability of 9 cms (318 cfs, 159 cfs per wet well). Only one port per wet well may be operated at one time. The Ports are located as shown in Fig. 1. The outlet tower also has a flood-control system with a minimum release of 0 cms. Given the thermal stratification illustrated in Fig. 2 and a release rate of 6 cms (212 cfs), which ports should be opened to achieve a desired downstream temperature of 23.5° C (74.3° F)?

This scenario and resulting question have probably been repeated many times at water resource projects. In most instances, the answer has been found through a trial-and-error opening of the ports and variation of the withdrawal rate through each wet well until the desired release temperature is achieved. The development of a numerical technique that systematically decides which ports should be opened and how much discharge should be released through each wet well eliminates the operational confusion and the waste of time and water resources required by the trial-and-error solution. Further, implementation of this technique on a microcomputer puts the guidance required for structure operation at the fingertips of the field personnel who need this information.

With the information given in the figures and previous paragraphs, SELCIDE was used to determine the ports that should be operated and the distribution of flow between the wet wells that would result in a release temperature of 23.5° C (74.3° F). Results indicated that ports Nos. 2 and 3 should be opened with 4.5 cms (159 cfs) and 1.5 cms (53 cfs) being released through wet wells A and B, respectively.

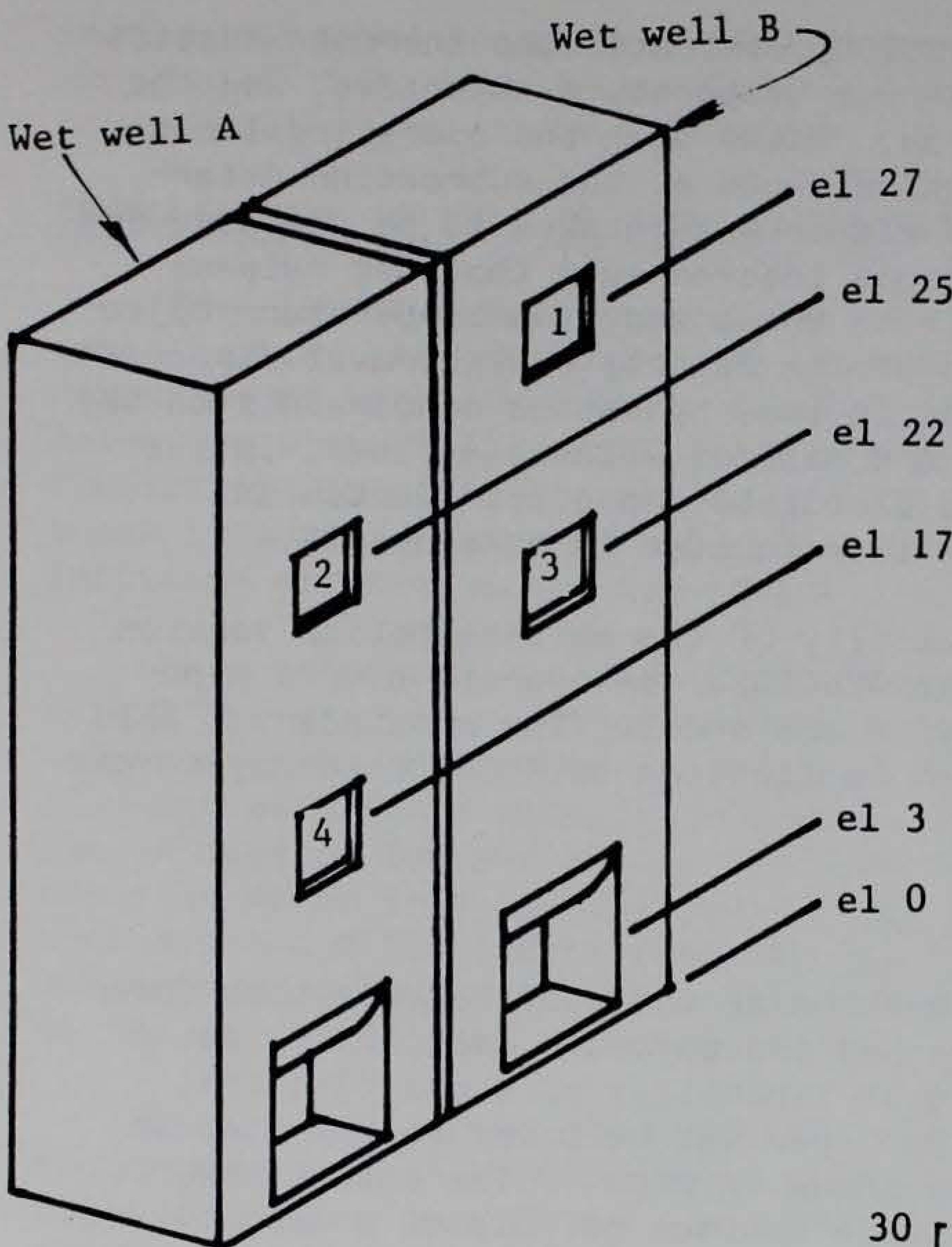

FIG. 1.--Structure schematic

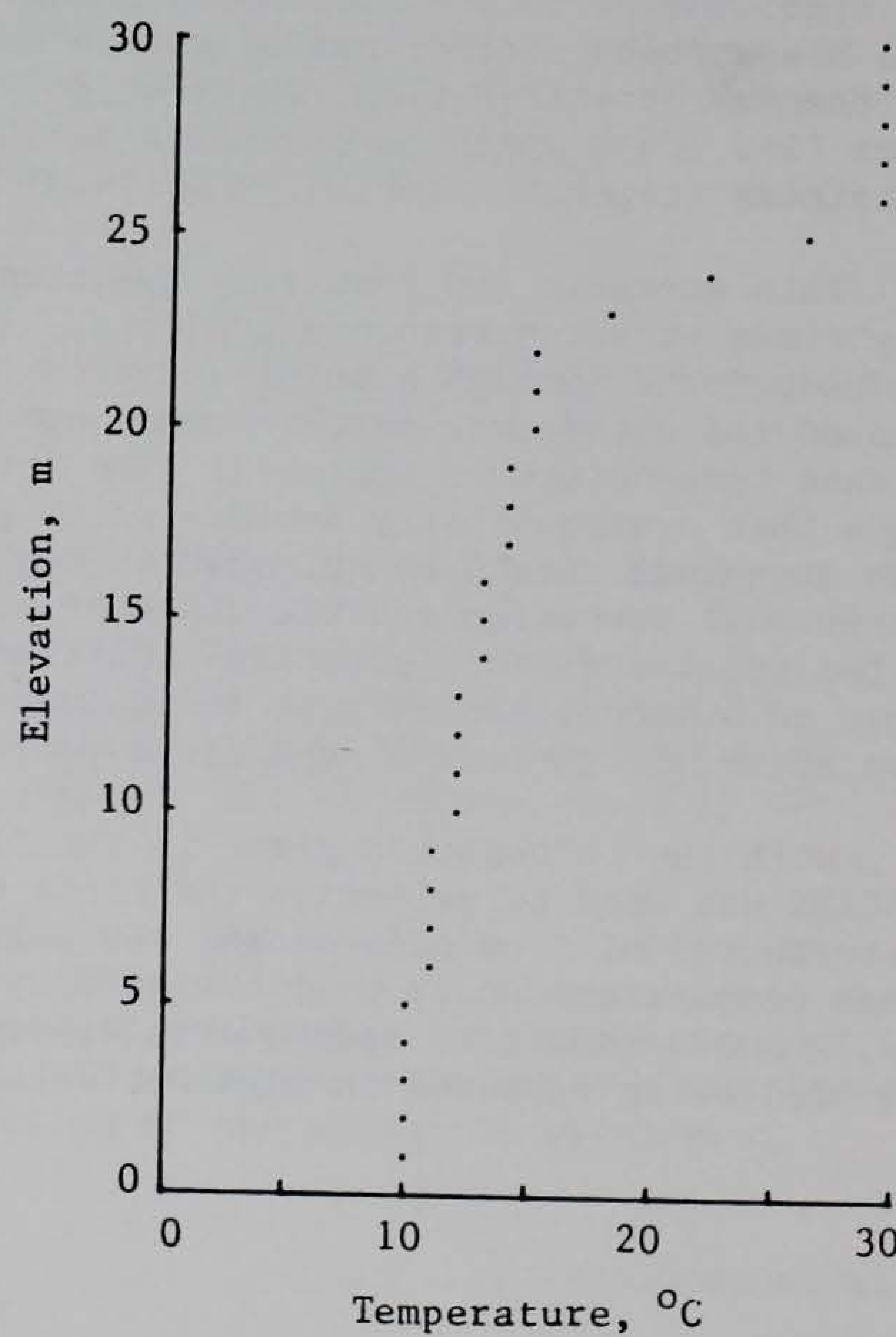

FIG. 2.--Temperature profile

Aided by this information, gate settings for flow control can quickly and efficiently be made to achieve the desired release temperature. After hydrological and meteorological effects change the thermal stratification Pattern of the impoundment or the release temperature objective changes, new data may be entered into the program and new gate settings determined. For example, if a new target temperature of 11.4° C (52.5° F) were desired with a release discharge of 9 cms (318 cfs), the flood system would have to discharge 7 cms (237 cfs) and port No. 4 would have to be opened and 2.3 cms (81 cfs) discharged through wet well A to accommodate this objective temperature.

CONCLUSIONS

Clearly, if control of water quality is a primary objective of a water resource project, and a multilevel withdrawal capability exists, then operational guidance is essential. By numerically predicting the appropriate operations of a structure to meet a release objective, the Problems that are often encountered in trial-and-error operations are avoided. Use of this technique on a microcomputer can place the necessary information quickly and efficiently in the hands of field personnel who actually operate the structure.

ACKNOWLEDGMENT

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Operational Tools: Optimal Control of Reservoir Water Quality

Steven C. Wilhelms and Michael L. Schneider*

BACKGROUND. There are many Corps of Engineer (CE) reservoirs that are operated to control the quality of release water. At most of these projects, the particular water quality parameter of interest is temperature. Control of release temperature may be important for several reasons. For example, the downstream temperature objective may be established to provide water within a particular temperature range for industrial uses, environmental concern may dictate that the temperature objective relate to the pre-reservoir in-stream temperatures, or a controlled, planned shift in the downstream environment (for example, from a warm- to a cold-water fishery) may determine the objective of controlling the release temperature.

From a classical limnological perspective, a reservoir stratifies in the summer months because of the input of thermal energy to its surface from inflow and solar heating. As a result of the thermal stratification, the lake is density-stratified with less dense water in the warmer surface layers (epilimnion) and heavier water in the colder lower levels (hypolimnion). Typically, release temperature control is achieved by withdrawing water from one of these strata in the reservoir. If an outlet near the surface of the impoundment is operated, the temperature of the release would be very similar to the temperature of the surface layers; if the outlet is near the bottom, then the temperature of the lower levels would characterize the release temperature. By selecting an outlet at an appropriate elevation, a desired release temperature can be withdrawn. If the capability exists, multiple outlets can be operated and their release water blended to achieve the desired temperature.

Two questions immediately arise, "How is the correct outlet (or outlets) selected? If two outlets are operated, what is the distribution of flow between them?" However, before addressing these two problems, another question must be asked, "What are the long-term and short-term objectives for operating the outlet structure?" The criticalness of violating the temperature objective must be examined. Is it more important to meet the temperature objective today or tomorrow? Can a small short-term violation be accepted in order to avoid a much larger long-term violation? This problem could develop if cold-water releases are the objective and the reservoir does not have a sufficient quantity of cold water to meet the objective for the entire year. This becomes a resource (cold water) management problem. However, included in any operational strategy, whether a short-term or long-term, are the hydrologic conditions and requirements, and the hydraulic constraints of the outlet structure. An example is discussed in later paragraphs.

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SHORT-TERM OR DAILY OPERATION STRATEGY. Consider the case in which there is no limit in the quantity of the desired resources. If such is true, then a day-to-day operation strategy will provide the needed operational guidance. Under this condition, the operation of the project today has no effect on the operation tomorrow or at any other time in the future. For this operational strategy, the only considerations for today's operation are the hydrologic requirements (how much flow must be released), today's target temperature, the hydraulic constraints of the outlet structure, and the initial question of which outlets to open.

To help CE field personnel make this type of operational decision, a numerical selective withdrawal model with a decision-making algorithm was developed under the Water Operational Support Technology (WOTS) program. The model, called SELCIDE (Holland and Wilhelms 1985), uses the numerical model of selective withdrawal, SELECT (Davis et al. 1985), combined with a decision-making routine that systematically selects the appropriate outlets for today's operation. The subroutine DECIDE makes outlet selections based on withdrawal from the in-lake temperature profile, release temperature criteria, required discharge, and hydraulic constraints of the outlet structure, i.e., minimum and maximum flows through the selective withdrawal system and flood-control system. The model has been adapted for use on a personal computer and can be easily used. However, we must emphasize that decisions made with this model are day-to-day decisions and do not consider limitations on resources or significant changes in operational objectives of the future.

LONG-TERM OR SEASONAL OPERATIONAL STRATEGY. For the situation where it is important to meet the objective or prevent large violations of the objective in the future, operational guidance must be based on more than the day-to-day decision strategy. As an example, consider the project that has a cold-water temperature objective. In the spring and through midsummer the release temperatures meet the desired objective. This, however, results in loss of the cold water in the hypolimnion of the reservoir since it is being released. In the late summer and early fall all the cold water in the hypolimnion has been released and only water warmer than the objective is left in the reservoir. Hence, a severe violation of the temperature objective occurs because warm water has to be released from the project. Figure 1 graphically shows the effect. The consequences of this violation could be quite severe if the maintenance of a cold-water fishery is the reason for temperature

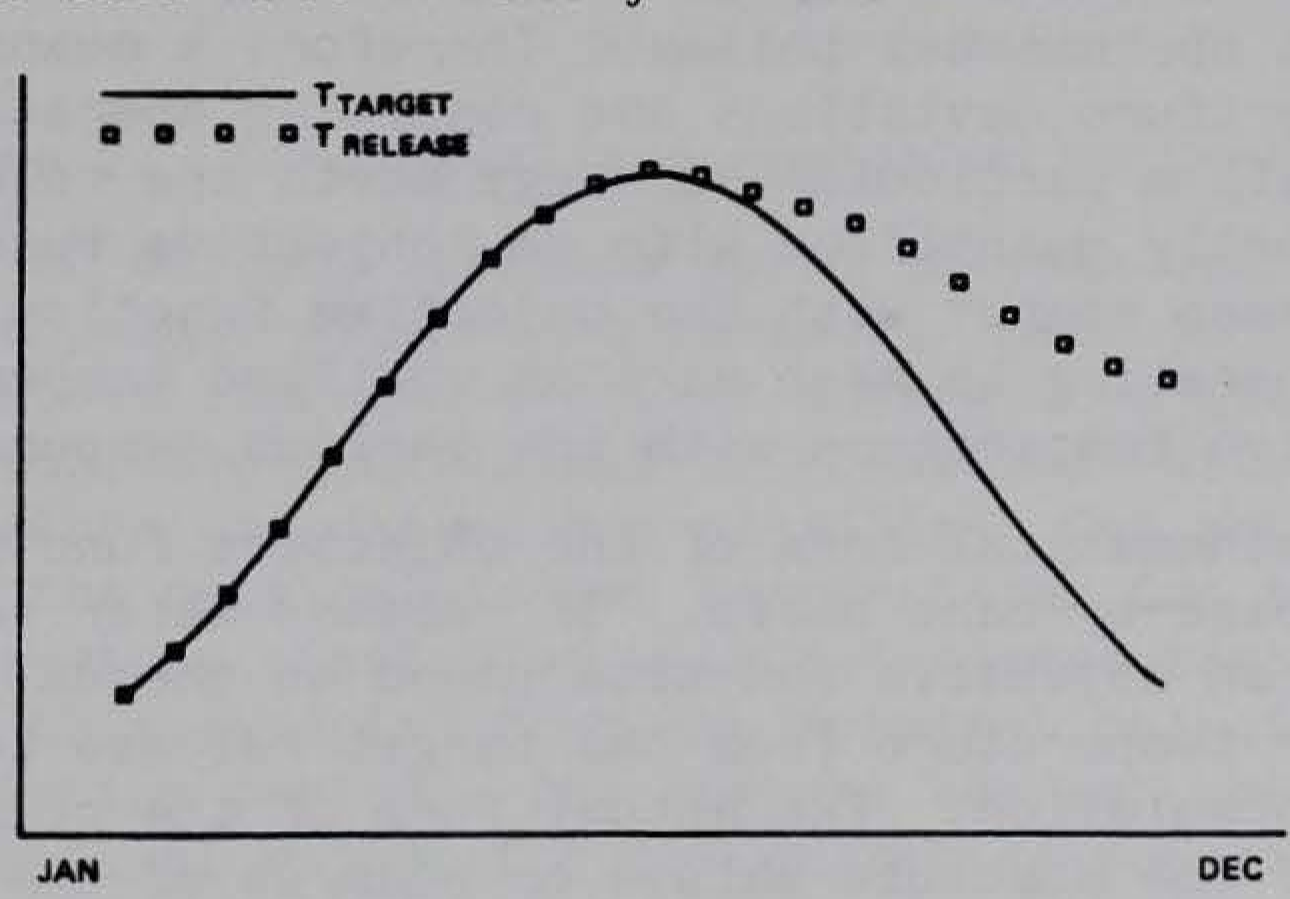

Figure 1. Release temperature with day-to-day operational strategy.

control. The day-to-day decision-making worked well in the early part of the season but caused significant deviation from the target temperatures late in the operational year.

This problem could possibly have been avoided if operational guidance had been developed that considered the effect of today's operation on operations in the future. Instead of exactly meeting the objective in the spring and early summer, a small deviation is allowed (warmer water than desired is released) and the cold hypolimnetic water is conserved for release later in the season. By conserving the cold water and allowing the small deviation from objective, the large violation late in the summer is avoided since cold water is available for release. How can decisions be made regarding today's operation that consider or "look ahead" to potential future conditions? How much of a small deviation will result in conserving enough cold water to assure the integrity of the objective in the late summer and fall? It is obvious that these questions coupled with the hydrologic requirements and hydraulic constraints of the project make the overall problem of operational guidance intractable unless a systematic computer-based solution technique is used. Under the Environmental and Water Quality Operational Studies, a computer code was developed to solve this problem that includes the port selection routine (SELCIDE) discussed in the previous section. The remainder of the paper briefly describes the code and its application.

OPTIMIZATION MODEL: CE-RES-OPT. To answer the questions posed in the previous paragraph, a systematic computer technique was developed (Labadie and Hampton 1979). This technique, which was called "objective space dynamic programming," was coupled with a reservoir simulation model to determine the best operational policy for meeting a desired release temperature objective. Fontane, Labadie, and Loftis (1982) give details of the optimization model that has subsequently been named CE-RES-OPT. CE-RES-OPT systematically evaluates the effects of a range of release temperature deviations from a desired objective and thereby determines how to operate today to more closely meet the release objective in the future.

An infinite number of operational scenarios can be formulated in an attempt to more closely adhere to the release objective late in the year. However, even with judicious selection of the temperature deviations to be evaluated, there may be too many alternatives to easily identify the best operational policy. Therefore a means of comparing the various temperature deviations and resulting operational policies is required. How well a particular strategy meets the release objectives can be mathematically quantified with an "objective function." Thus CE-RES-OPT can "keep score" with the objective function as it evaluates the impacts of operating to meet various modified temperature objectives (original objective temperature with the various temperature deviations).

The exact mathematical form of the objective function must be determined on a case-by-case basis. If release water temperature is the primary concern, an objective function based on the daily deviation of the release water temperature from the target release temperature may be an appropriate formulation. The actual form of the objective function may be the sum of the absolute values or squares of the deviations of release temperature from the target temperature. The operational policy that leads to a minimum value of this type of objective function would be the best or "optimal" strategy. It is extremely important to carefully formulate the objective function since the selection of the optimal operational policy is directly dependent upon the form of the objective function. In some cases, it may be important to include other water quality parameters. The importance of exactly meeting the objective at a certain time or over a certain period can also be included in the objective function.

The optimization model is composed of two major components: an optimization module and a one-dimensional reservoir simulation model. The optimization module provides data to the simulation model about initial reservoir conditions, the period of simulation, and the modified daily temperature objective. With this information and the hydrologic, meteorologic, and release quantity data, the reservoir model simulates the in-reservoir and release temperature characteristics. At the end of the simulation period, the in-reservoir conditions are saved for subsequent simulations. The daily release temperatures are returned to the optimization module to be included in the calculation of the objective function.

APPLICATION OF CE-RES-OPT. To demonstrate how the optimization model works, we will present a simple example representing the search for the optimum operational policy to conserve cold water. The search procedure can be conceptualized as shown in Figure 2.

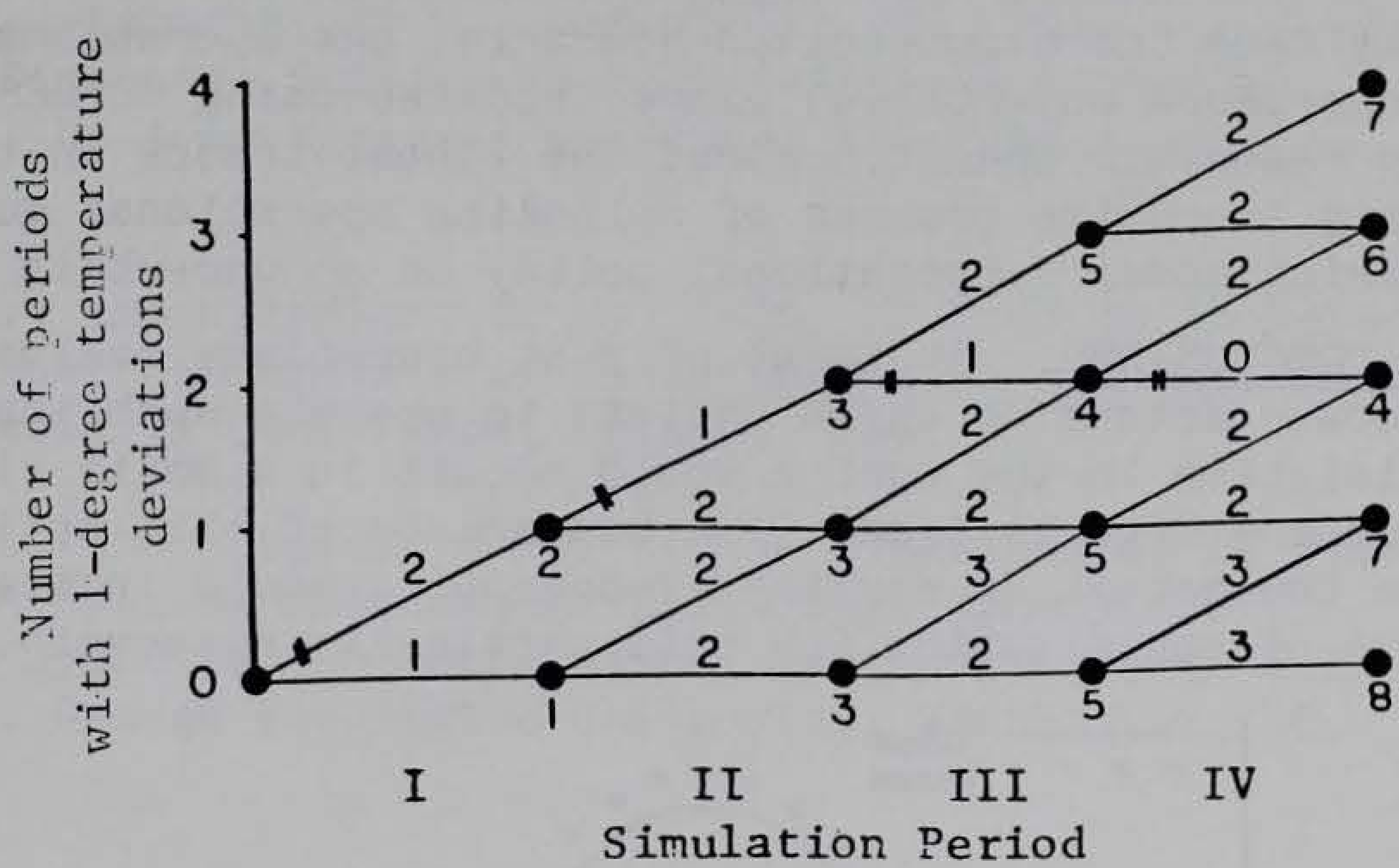

Figure 2. Conceptualized operation of CE-RES-OPT.

The time period of concern (perhaps April through October) has been divided into four equal simulation periods. Each junction in the network represents either the start or ending of a simulation period. From each junction, two operational alternatives (with and without a modified temperature objective) are possible. One path represents a simulation with the original target temperatures for that period, while the other represents a simulation with a 1-degree temperature deviation (an increase in this example) from the original objective for that particular simulation period. The numbers between the junctions represent hypothetical values for the objective function for that particular simulation. The numbers at the junctions represent the minimum cumulative objective function value that can be achieved through any of the preceding paths.

The first period would be simulated twice: once with the original target temperatures and once with a modified target temperature (1-degree increase). The objective function would be calculated for each simulation. The second period would be simulated four times with two simulations starting from each of the first period's two ending conditions. From each of the two starting conditions for period two, simulations would be made with the original target temperatures of period two and with a 1-degree increase in those target temperatures (modified temperature objective). Simulations of the third and fourth periods are conducted in a similar manner. The "path" of deviations from the original objective that leads to the minimum cumulative objective function value at the end of the fourth period represents the optimal deviations from the original target temperatures. In this example, the optimal operational policy dictates that the structure be operated to release water with a temperature 1-degree above the original target temperatures during periods I and II and then, in periods III and IV, operate for the original target temperatures (no deviation). This is a simplified example of the operation of the model CE-RES-OPT. In most cases, the number of simulation periods would be larger and more temperature modifications would be simulated over each discretized period.

The resultant operational policy is specific to given hydrologic, meteorologic, and release water quantity inputs. The model is generally applied to a wide range of hydrologic and meteorologic conditions to determine the sensitivity of the operational policy to these variables. If the actual hydrologic and meteorologic events vary significantly from the hypothesized scenario, the operational strategy (release temperature objectives) can be updated using CE-RES-OPT given the existing reservoir conditions and the latest trends in the weather. This iterative process of selecting operational guidance is a result of basing today's operational policy on an uncertain future.

SUMMARY AND CONCLUSIONS. In terms of past operations evaluation, CE-RES-OPT could determine which outlets to operate such that a small objective violation in the spring would result in a small violation in the fall (compare Figures 1 and 3). Comparison of this optimized operation to the actual or day-to-day operation would indicate the effectiveness of optimization for this particular reservoir. Hence, in

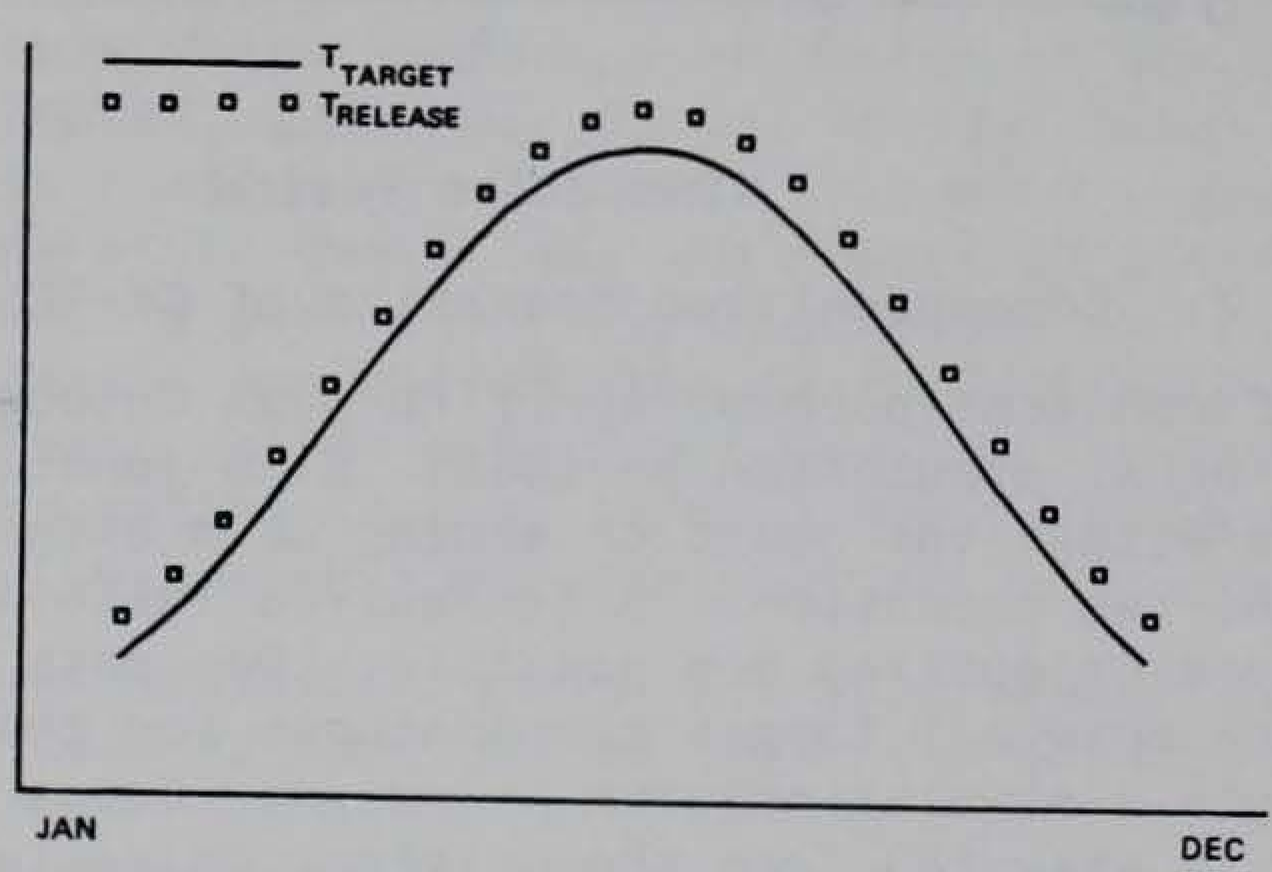

Figure 3. Release temperatures with long-term operational strategy

this manner, the potential can be determined for applying the model to real-time operational decisions. For this type of evaluation, past hydrologic, meteorologic, and hydraulic data are required input for the simulation model to perform the optimization.

In real-time operations, changes in outlets for temperature control are not usually necessary on a day-to-day basis. For many projects, meteorologic or hydrologic impacts significantly affect thermal stratification such that operational changes are only required weekly or biweekly. Therefore decisions regarding operations would have to be made every 10 to 14 days. Thus CE-RES-OPT would have to be executed when these decisions are required. The simulation period could be 2 weeks long. However, since CE-RES-OPT is not a "forecasting" model, i.e. it does not forecast weather, general operational guidance would be the result of model use. By using known real-time data (in-reservoir conditions and long-term weather forecasts), the applicability of the operational guidelines would be greatly enhanced.

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Field Measurements at Intake Structures

by Ellis Dale Hart*

Abstract

Prototype water-quality tests were conducted at Beltzville Dam in Pennsylvania. The purpose was to determine the location and degree of reaeration of flow that occurred as it passed through the outlet works. Temperature and dissolved oxygen data were collected in the reservoir, at seven stations within the outlet structure, and in the downstream channel. The tests involved various flow rates and intake levels.

Similar measurements are scheduled to be conducted at Taylorsville Dam, Kentucky in the summer of 1985. In addition, because of the unique intake tower trash rack design, inlet velocities will be measured for determining entering velocity profiles.

Beltzville Dam Study

The project is located in the Lehigh River Basin on Pohopoco Creek in northeastern Pennsylvania. Flow through the dam is regulated by a gated intake tower (Fig. 1) that contains two flood-control intakes (2.83 by 7.33 ft) located at the base of the structure (el. 503.39) and a water-quality control system, the intakes of which are located at various levels of the tower. The water-quality control system permits selective withdrawal through any one or a combination of the eight 2- by 4-ft multi-level intakes with invert elevations ranging from 545.5 to 615.0.

Flow passes through the multi-level intakes into a divided wet well that converges downward into a single vertical riser. From the vertical riser flow passes through a converging bend (or elbow), past a 2- by 3-ft control gate, and into the water-quality control conduit. The flow then exits the water-quality control conduit (which runs between the two flood-control conduits) through a portal in the structure's transition section. In the 70.17-ft-long transition section, the two flood-control conduits and the water-quality control conduit converge to form a single 1231-ft-long, 7-ft-diam conduit. Finally, the flow passes through a conventional hydraulic jump-type stilling basin and into the creek. A cross section of the dam is shown in Figure 2.

The primary purpose of the tests was to determine the locations and degree of reaeration that occurs as flow passes through the Beltzville Dam outlet works (2). It was generally accepted that reaeration did occur as flow passed through the outlet works. However, the locations

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and degree of reaeration had not been determined. This information was needed to evaluate the ability of proposed projects to meet release dissolved oxygen (DO) requirements and to determine appropriate design modifications, if needed, to increase reaeration characteristics of both proposed and existing structures.

Dissolved oxygen and temperature profiles were measured in the reservoir at the beginning of each day. These profiles were considered representative for the entire day's testing. Check measurements indicated that this assumption was reasonable. In addition, DO and temperature levels were measured at seven stations within the outlet works and at one station in the downstream channel for varying test conditions. The measurement stations are listed below and shown in Fig 2.

Measurement Station	Location and Description
1	Reservoir: every 5 ft (in some cases every foot through the metalimnion) from the surface to the bottom
2	Wet well: below the invert elevation of the intake when flow conditions permitted
3	Water-quality gate: (5 ft upstream)
4	Conduit: sta 5+09.0
5	Conduit: sta 9+29.0
6	Conduit: sta 11+91.0
7	Portal
8	Baffles
9	Channel: about 600 ft downstream of the stilling basin

The DO and temperature tests were conducted using a Yellow Springs Instrument Company Model 57 Oxygen Meter that includes a temperature sensor. The instrument had a DO measurement range of 0 to 20 ppm and a quoted accuracy of ±1 percent of full scale. It had a temperature measurement range of -5° to +45°C. The depths in the reservoir where DO and temperature were recorded were determined by attaching a weighted measurement tape to the sensor.

At measurement stations 4-6 (Fig. 2) in the conduit, samples were taken in 3-in. diam by 18-in.-deep canisters. The canisters were attached to 7-ft jacks that were wedged vertically in the conduit and were set just below the pre- determined water surface for each test condition. Discharge measurements were made at a gaging station one-half mile downstream of the dam.

Twelve tests were conducted at Beltzville Dam. The conditions for each test are listed below. Eleven of these were made while releasing flow through the water-quality system and one test was conducted with flow through the flood-control release system only. The reservoir pool was practically constant at el. 629.0 (+0.2 ft) throughout the testing program. Variables in the water-quality system tests were the number and location of intakes opened and the percentage opening of the water-quality control gate. For the flood-control test, both gates were open 3.25 ft.

Test No.	Intake No.* (100 Percent Open)	Control* Gate Opening Percent	ft	Discharge cfs	Pool el	Air Temperature °F
1	FC**	44	3.25	1234.0	629.0	74
2	5	10	0.3	25.4	628.8	50
3	5	50	1.5	159.0	619.1	68
4	5	100	3.0	331.0	629.0	82
5	3	10	0.3	23.0	629.0	71
6	3	50	1.5	156.0	629.0	80
7	3	100	3.0	255.0	629.0	57
8	7	10	0.3	26.6	629 0	62
9	7	50	1.5	155.6	629.0	82
10	7	100	3.0	328.5	629.0	75
11	4, 7	50	1.5	159.0	629.0	64
12	1	50	1.5	155.0	629.0	80

* See Figure 1. ** Flood Control Test.

The minimum allowable daily average DO level for the tributaries of the Lehigh River was 7.0 ppm (3). No discharge was allowed with a DO content below 6.0 ppm. During the period of the testing program the minimum level recorded in the downstream channel was 7.9 ppm (test 6). All other recordings were greater than 8.0 ppm indicating that the Beltzville Dam water-quality system effectively reaerates flow through the structure regardless of the level of withdrawal.

The reservoir metalimnion was found to lie approximately between the depths of 20 and 25 ft. The sharpest DO reduction occurred at depths between 23 and 25 ft. In most tests a major portion of the reaeration occurred downstream of the water-quality control gate (between Stations 3 and 4). The effectiveness of the water-quality control gate as a means for inducing reaeration is demonstrated in the tabulation below. The DO and temperature profile for a typical test (Test 5) is shown in Fig. 3.

Test No.	DO Above Water-Quality Gate ppm	Below Water-Quality Gate Sta 4 ppm	Change ppm	Percent
2	2.75	9.00	6.25	227
3	2.80	8.95	6.15	219
4	5.95	8.52	2.57	43
5	6.00	8.40	2.40	40
6	8.30	8.00	-0.30	-4
7	7.98	8.09	0.11	1
8	3.50	10.00	6.50	186
9	3.05	9.40	6.35	208
10	2.98	9.80	6.82	229
11	3.13	9.35	6.22	199
12	11.00	8.10	-2.90	-26

The following conclusions regarding the reaeration of flow through the Beltzville Dam outlet works were drawn:

a. The release DO level was above the State of Pennsylvania minimum requirement.

b. Most of the reaeration in the water-quality facilities occurred near the water-quality control gate due to the high air entrainment induced by the relatively shallow, turbulent, and supercritical flow in the water-quality conduit downstream of the control gate.

Taylorsville Dam Study

The project is located on the Salt River in North Central Kentucky. Reservoir releases are regulated by a gated intake tower consisting of two flood-control intakes at the base of the structure (el 474.0) and two wet wells with five 6- by 6-ft water-quality intakes in each at elevations ranging from 503.0 to 534.0. Both flood-control and water-qualtiy flows pass through the same 5.5- by 14.75-ft rectangular gate passages. During selective withdrawal operation, the conduit emergency gates are closed and flow is discharged through the multilevel intakes into the wet wells and through an opening located in the roof of the conduit between the emergency and service gates. The service gates are used to regulate the selective withdrawal releases. The locations of the multilevel intakes are shown in Fig. 4. The two gate passages transition into a single 11.5- by 14.75-ft oblong conduit. The last 20 ft of the oblong conduit contain a transition to a flat bottom conduit before discharging into an outlet transition and stilling basin.

Water-quality parameters will be measured in the reservoir and then monitored through the structure. These are temperature, dissolved oxygen, conductivity, and total gas. The water quality measurements will be made in the reservoir on a 160-ft radius from the intake tower, at 100-ft horizontal intervals and depth increments of 3 ft. Velocity measurements will also be made at these locations in the reservoir. Other measurement stations will be at the base of the wet well (just upstream of the service gate), downstream of the service gate, and in the down-stream channel.

The intake tower trashrack guides are located on the outside wall of the intake tower (Fig. 4). A special bracket (Fig. 5) was designed to fit these guides for positioning three velocity meters in front of the inlets. Velocity measurements will be made with Marsh-McBirney elec-tromagnetic current meters, model 511. The instrument measures the x and y components of velocity perpendicular to the meter probe. The meters will be positioned at three vertical locations for a total of nine velocity measurements per inlet.

The primary purposes of the prototype measurements are to:

a. Determine the locations and degree of water-quality blending occurring as flow passes through the outlet works.

b. Compare findings with those of the model study (1).

c. Determine velocity profiles of flow entering the multilevel intakes.

Summary

Water-quality measurements have been, and continue to be made by the Waterways Experiment Station at Corps structures. These measurements provide the locations and degree of blending of water quality parameters. The information can be used for corrective action (if necessary) and future design.

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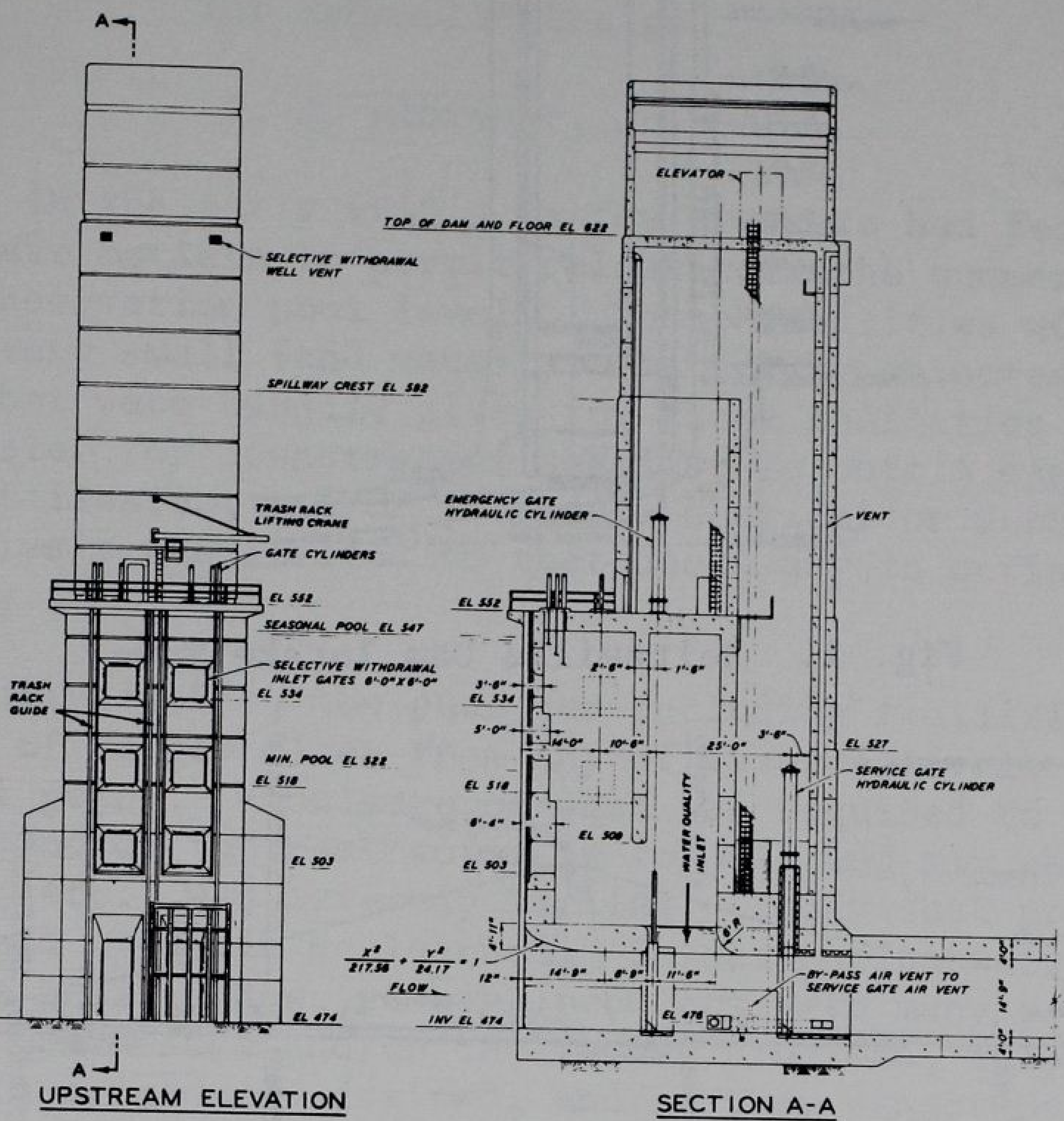

Fig. 4. Taylorsville Dam Intake Tower

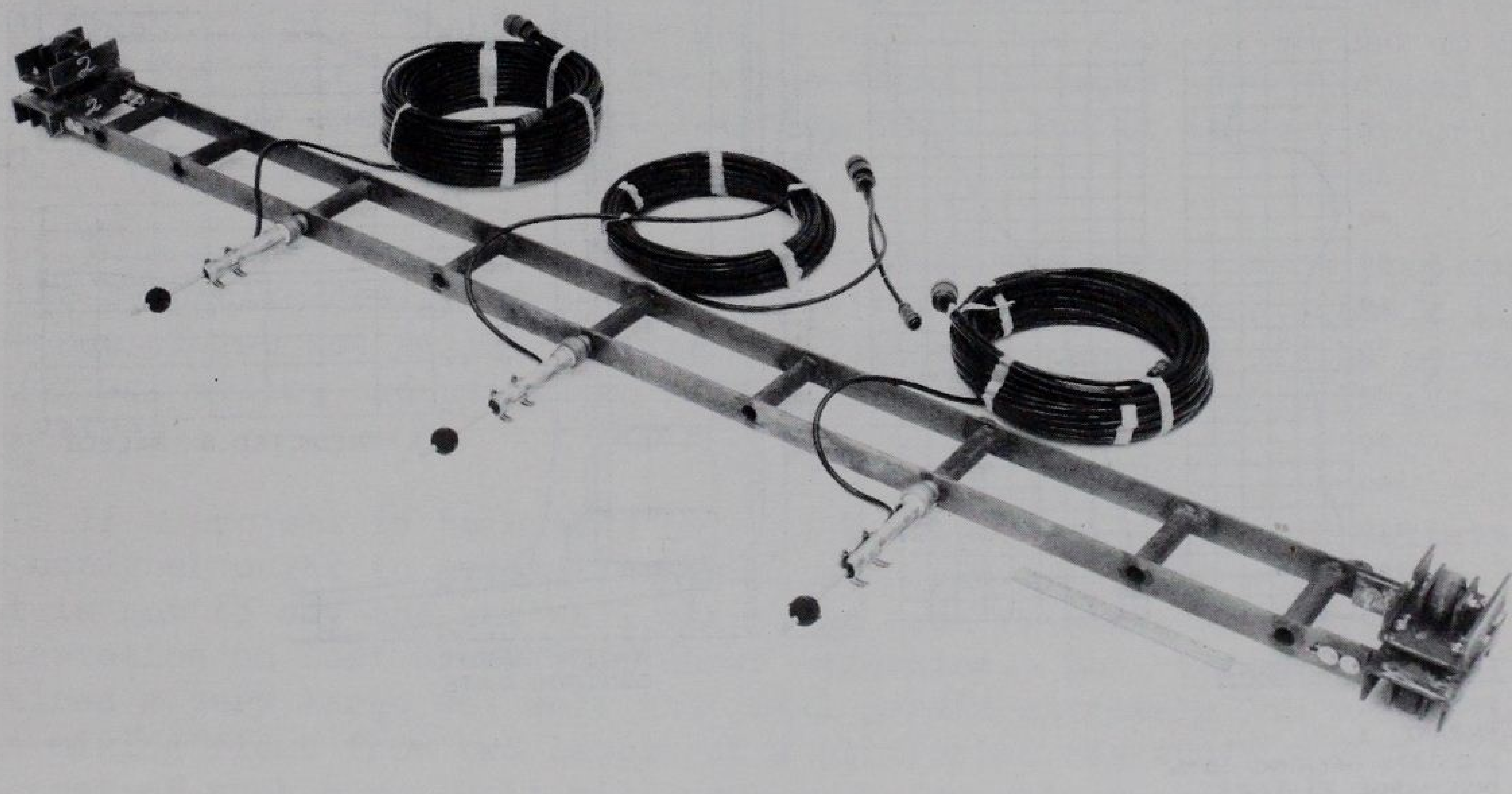

Fig. 5. Taylorsville Dam Intake Tower Transducers and Bracket

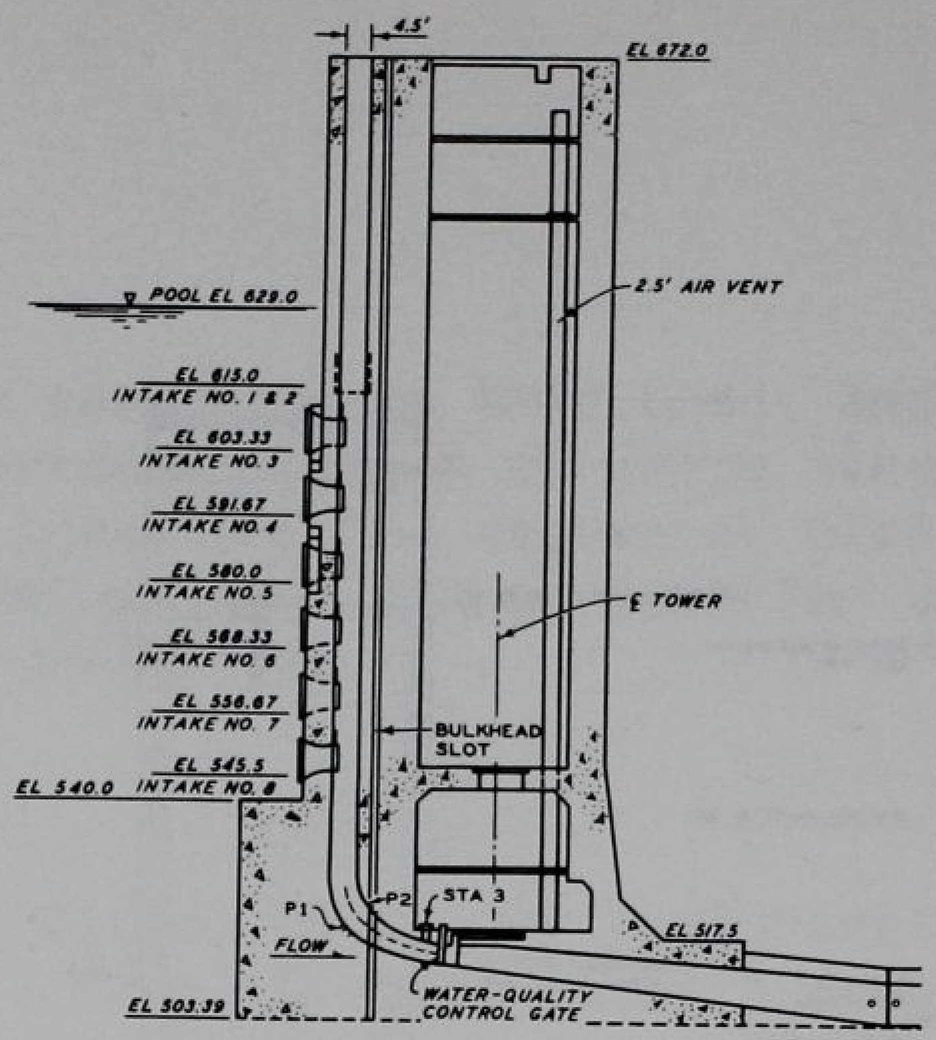

Fig. 1. Beltzville Dam Intake Tower

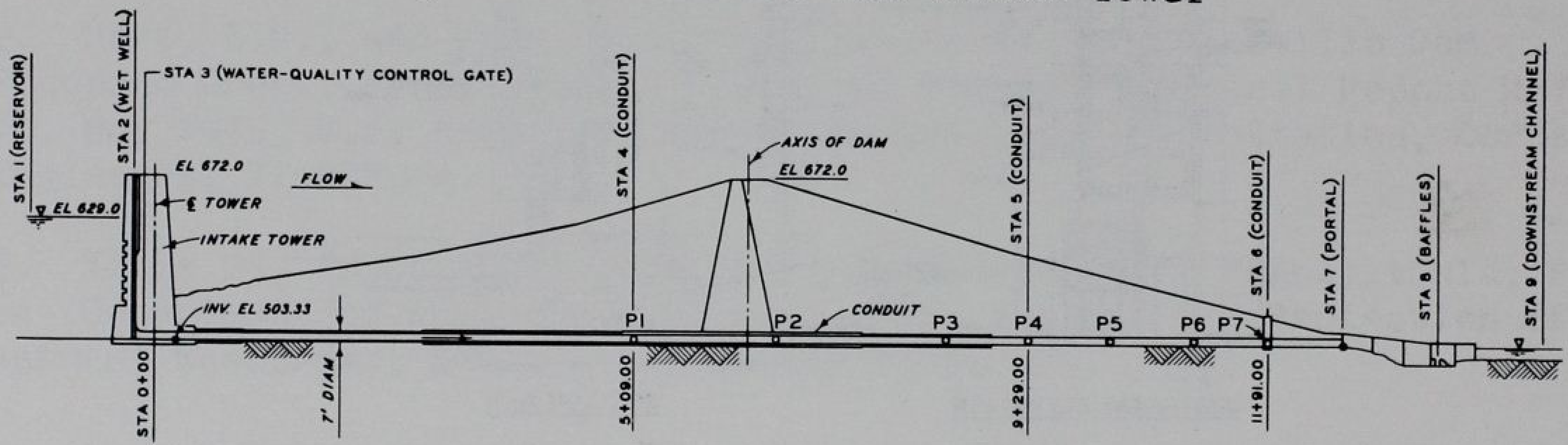

Fig. 2. Beltzville Dam Water Quality Measurement Stations

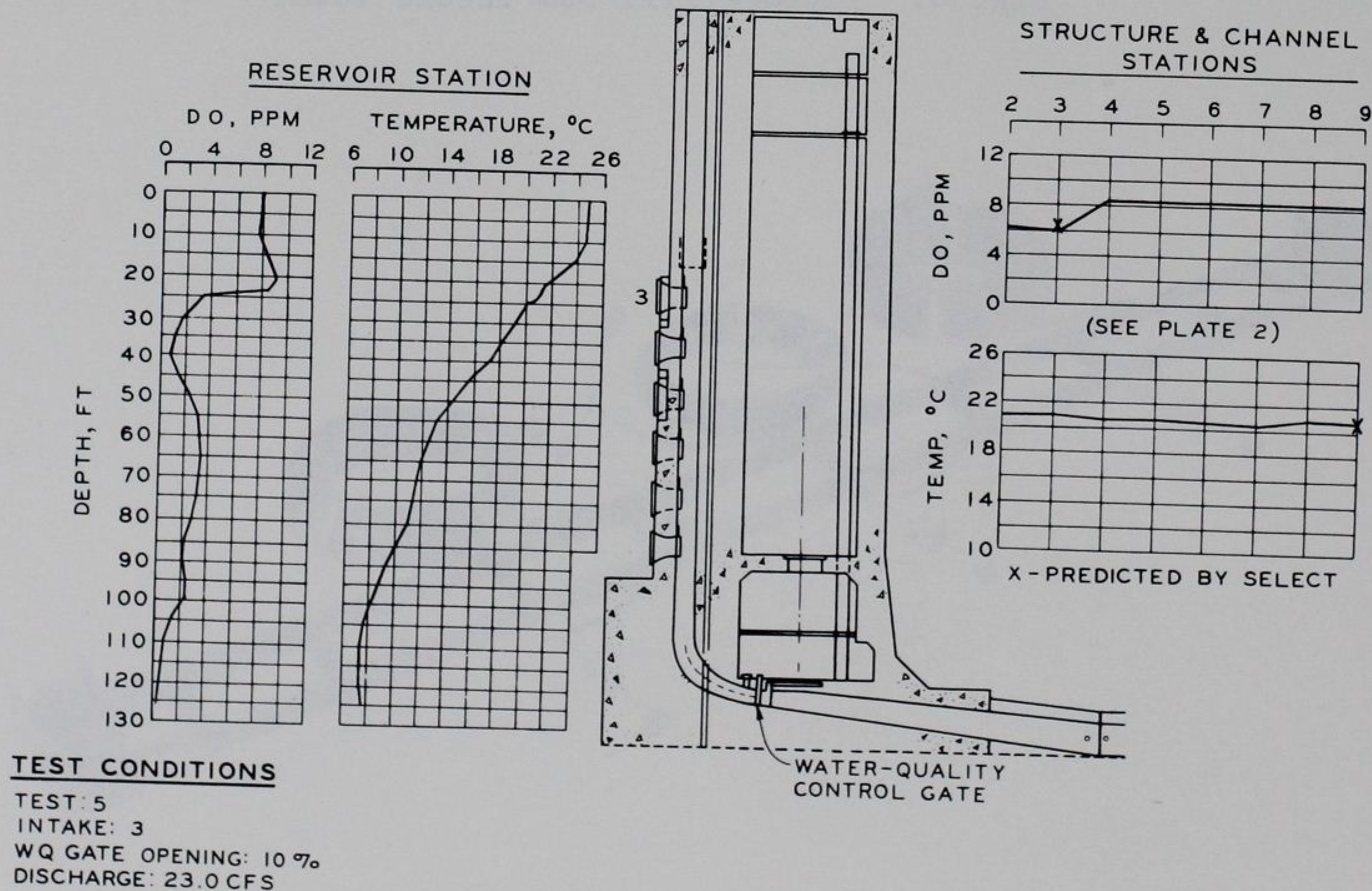

Fig. 3. Beltzville Dam Water Quality Measurements

Thoughts and Considerations for Hydraulic Design

T. J. Albrecht, Jr.*

Starting in the early 1950's, a few projects had facilities included in their outlets to permit releases in the summer months from their upper conservation pool levels. These facilities were generally sized to pass very small (and usually arbitrarily selected) flows. The purposes that were usually given for these facilities were: to release warm water for downstream fisheries, to obtain higher D.O. levels in the tailwater, and/or to reduce chemical or turbidity concentrations. Analysis was limited to that necessary to define the discharge capacity.

In the early 1960's, the push for including facilities that would permit release of larger flows from any of several levels in the reservoir really got going. Knowledge of what was required to pull water from a narrow band of the reservoir, or to pull and mix water from more than one level, etc., was severely lacking. Individual projects though, could not be held up while we learned. So assumptionswere made, and structures were built. Now, we have problems with many of them: they have poor flow conditions and/or cavitation problems, they cannot produce the results originally desired, and/or they cannot be operated in a manner that will satisfy today's needs.

However, in my view, today's problem is still the original problem - ignorance! Information gained from existing projects has not been quickly and freely passed around, and to the extent it has been passed around, it has been piecemeal, at best. In some instances, problems have been hidden at the project, and/or secreted away at the district or division level. There is a great reluctance to say we goofed; we built something that doesn't work. Then, others get a copy of the project DM and do a take-off for their design and the whole thing is repeated. Mistakes or problems can be one of our best learning tools, and it is very important that we do not repeat poor design ideas!

This presentation will focus on three of the more common problems in the hydraulic design of a selective withdrawal structure: should it have one or two wet wells, setting the discharge capacity of the ports and/or wet wells, and using and selecting commercially available gates and valves.

If a project is to have a selective withdrawal system, I believe the designer ought to automatically plan on providing two wet wells. That is not to say one wet well cannot be made to work; we just had a presentation on Lost Creek that shows otherwise. But, to make it work requires a very large wet well that will permit extremely low velocities when mixing flows from two levels of a stratified reservoir. However, the cost of such a wet well will usually be greatly in excess of the cost of a two well system.

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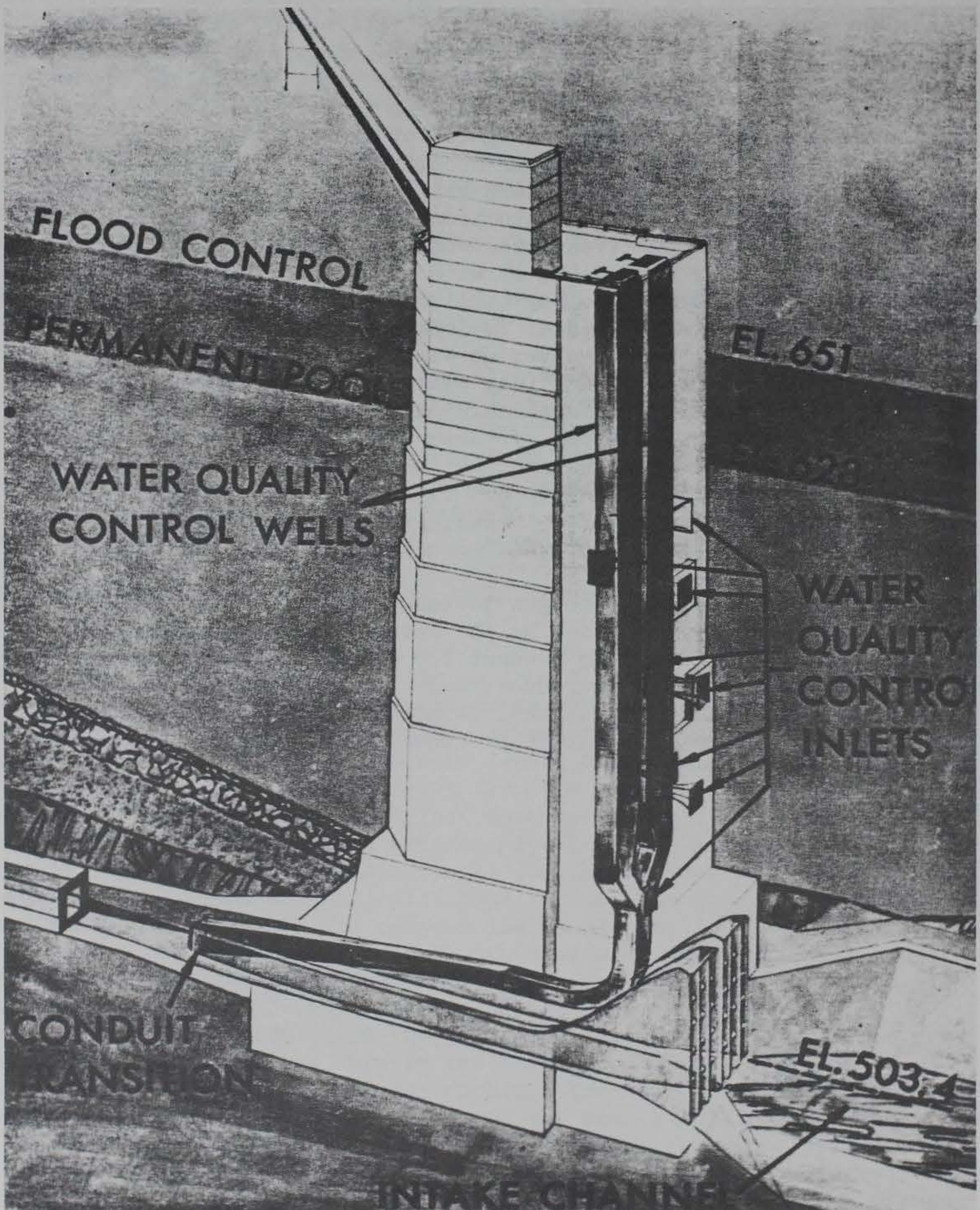
Beltzville and Gathright Dams both have what we used to refer to as a two wet well system(see plates). However, we now know that in such a system, we get ocsillations in the two vertical water passages and that the outflow is primarily first from one of these passages and then from the other. Warm Springs Dam, which you visited yesterday, originally was to have such a system (see plate). Each of the vertical passageshad the capability of passing water directly to the fish hatchery immediately downstream (at that time the hatchery was to use water directly from the reservoir). When it was decided to use well water for the hatchery, the second vertical passage was eliminated. Supplement No. 1 to DM 13 states, "Ordinary maintenance of the wet well can be accomplished by use of the service gates as a by-pass when water conditions are favorable. The few days over the life of the project when emergency maintenance might be required does not warrant the added cost of a standby wet well..." In truth, the added cost of a second wet well, a true second wet well, was never studied. Considering that the velocity in the vertical well can be as high as 7 to 8 feet per second, I personally hold no hopes for the system functioning as "designed", i.e., blending water from two levels of the reservoir.

For flexibility of operation with varying pool levels and/or conditions in the reservoir, the ports in the two wet wells ought to be at different levels. Exceptions might be the highest and/or lowest ports, which might be at the same level in both wells. With some designs, this can make the intake tower more costly, as more floors or landings may have to be built into the structure, but there will be a greater probability that one of the ports will be between the reservoir surface and the thermocline, and/or that one of the lower ports will not be at the level where the water too turbid or is loaded with an undesired chemical.

Port capacity is usually set after we have analyzed historical flows. From this analysis, we obtain the maximum flow that "will ever be required" from a high level port, and a similar maximum flow for a low level port. Along with varying the port levels between the two wet wells, I believe that every port should, at the very least, be capable of passing the highest flow obtained for any port from the analysis. For some flexibility for the future, I feel that each port should have even greater capacity, but I have no good argument for how much more; perhaps OCE should consider setting some criteria for this. Some wet wells have been designed where one well will pass the maximum high level port flow and the other will pass the maximum low level port flow, as obtained from the analysis. This has been done with a view to reducing first costs. However, they should have gone all the way and only provide low level ports in one well and only high level ports in the other, as some of the ports provided are (from the analysis) totally worthless. The preliminary designs for the dams in the Cottonwood Creek Project have wet wells designed for the two flows obtained from the analysis and both wells have ports from top to bottom at the same level in both wells (see plates). There is no flexibility what-so-ever in such a system for what the operator may find in the reservoir in any given instance; there is no flexibility for meeting a change in objectives based on new knowledgeof organisms in the river and what they need; and there is no way one wet well (particularly the smaller one) can meet the flow requirements, much less meet the flow requirements with water that is close to the desired quality.

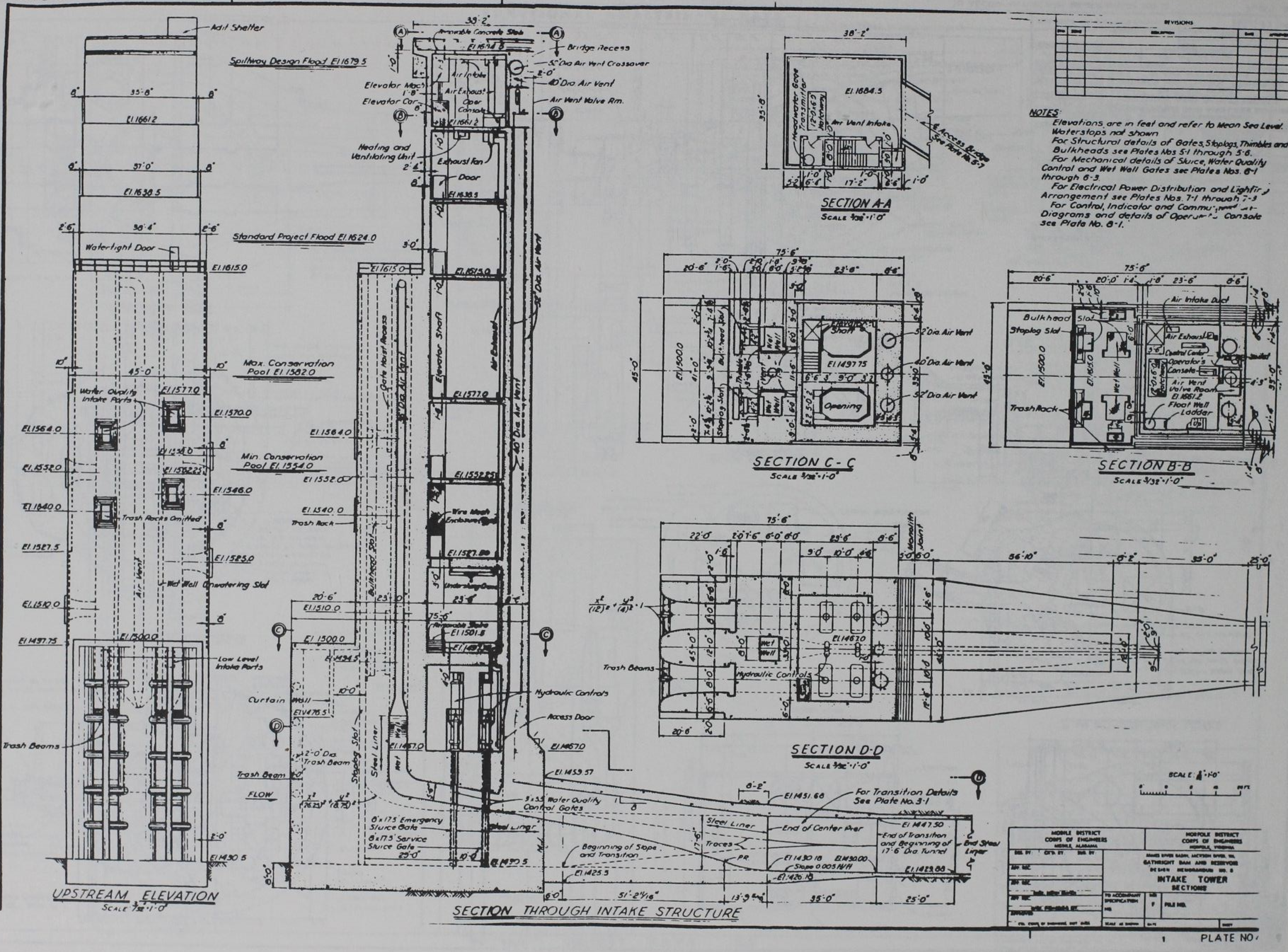
Frequently, commercially available gates and valves have been used. Such is not necessarily bad, as they are usually less costly than custom designed and fabricated units. However, all too often, the unit selected from the catalogs has not finctioned in a satisfactory manner. They are usually selected on the basis of being able to operate under a given head condition, pass a certain discharge (when full open), and their cost. Flow and/or pressure conditions for prt gate operations have rarely been considered. The contract specifications for gates and valves are written by a mechanical engineer or a specification writer, neither of whom have any knowledge of the hydraulic design or of the potential operating conditions. The hydraulic engineer should make the selection and write the specifications (or at least have veto power over what others may specify) after making his own evaluations of the units available. Even manufacturer's representatives recommendations should be suspect, as frequently their only experience is with water and sewage treatment plants, etc., which are in no way similar to conditions in an outlet works. The type of valve used at Taylorsville Dam and the type of seal specified for the butterfly valves at Warm Springs Dam are two good examples of what we do not want (see plates). In addition, if he is planning to use commercially available units, the hydraulic designer should allow for the dimensions of these units. Commercially available gates and valves have been designed by the manufacturers to match commercial pipe exactly. They do not fit custom made pipes or tunnels, dimensionedto the nearest three inches, etc. Warm Springs Dam again provides us with an example of what we can end up with (see Plate).

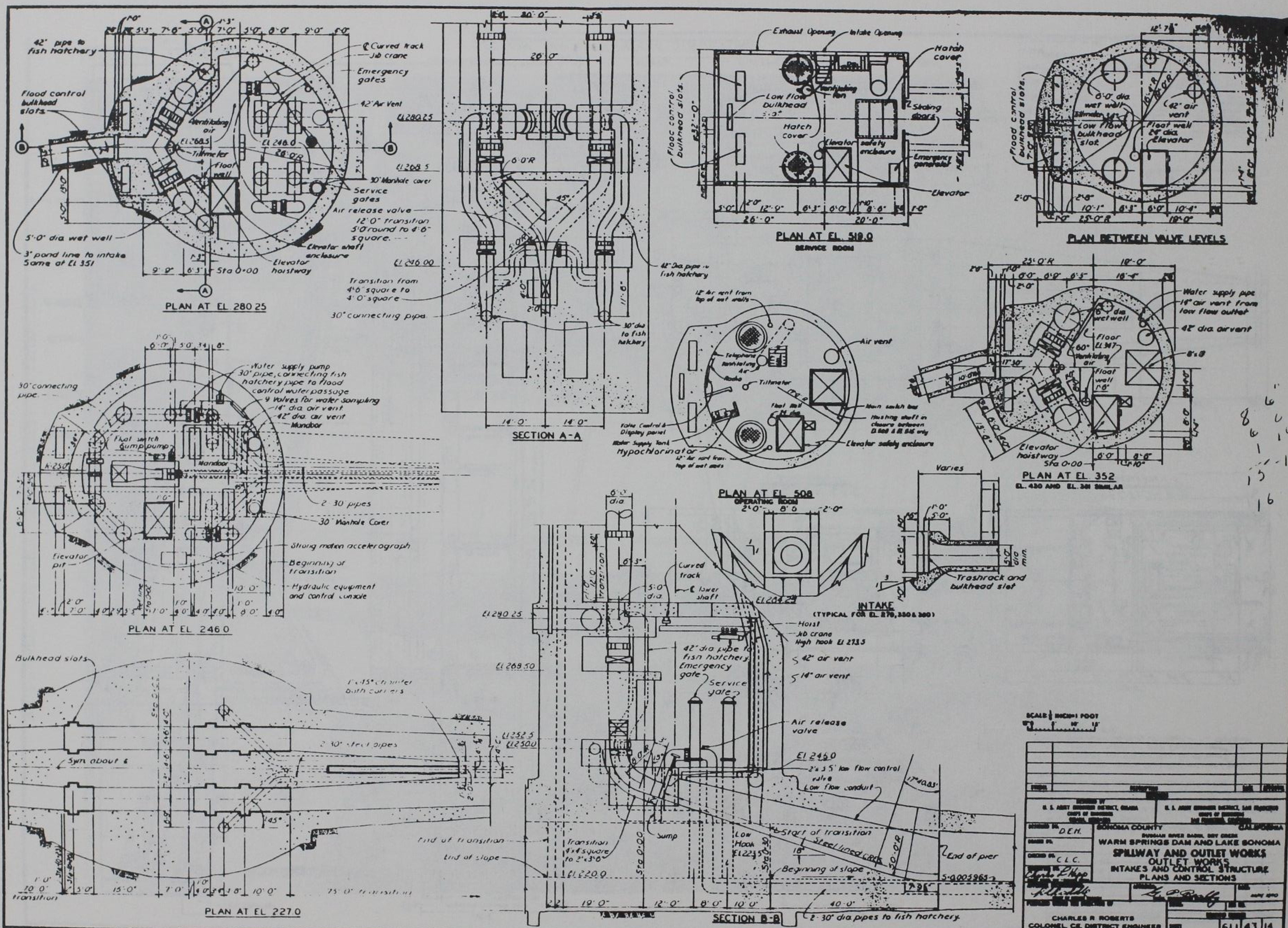
To be sure, I have not begun to touch on all of the issues and problems that must be addressed by the designer of a slective withdrawal system. I have only tried to present for your thought and considerations, those items that I feel have caused, and continue to cause, problems in meeting our original objectives, in meeting changes in project objectives, and give us our greatest maintenance headaches.



BELTZVILLE MULTILEVEL INTAKE STRUCTURE

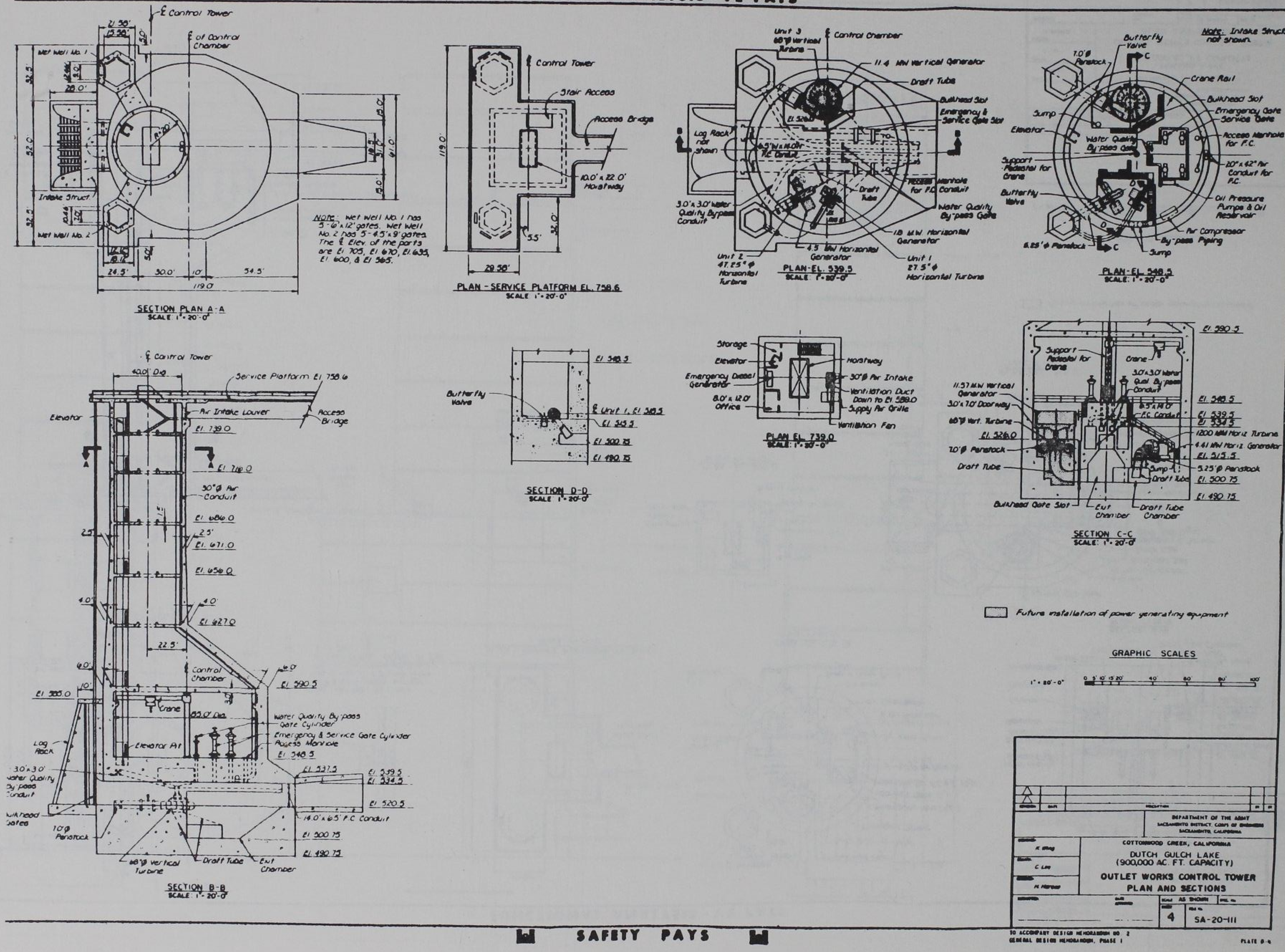
CORPS OF ENGINEERS





TO ACCOMPANY DESIGN MEMORANDUM NO. 13

PLATE 40


SECTION PLAN A-A
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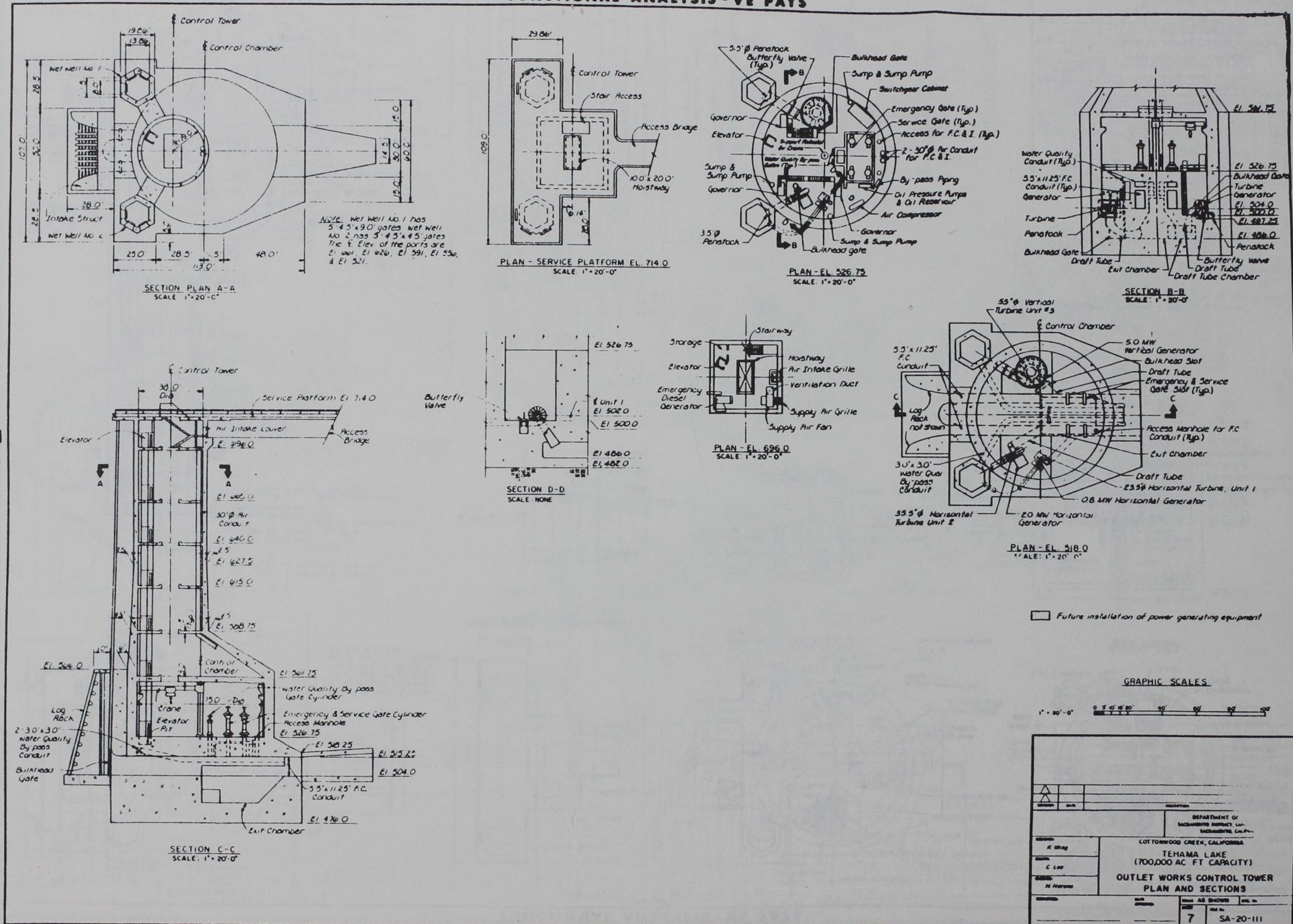
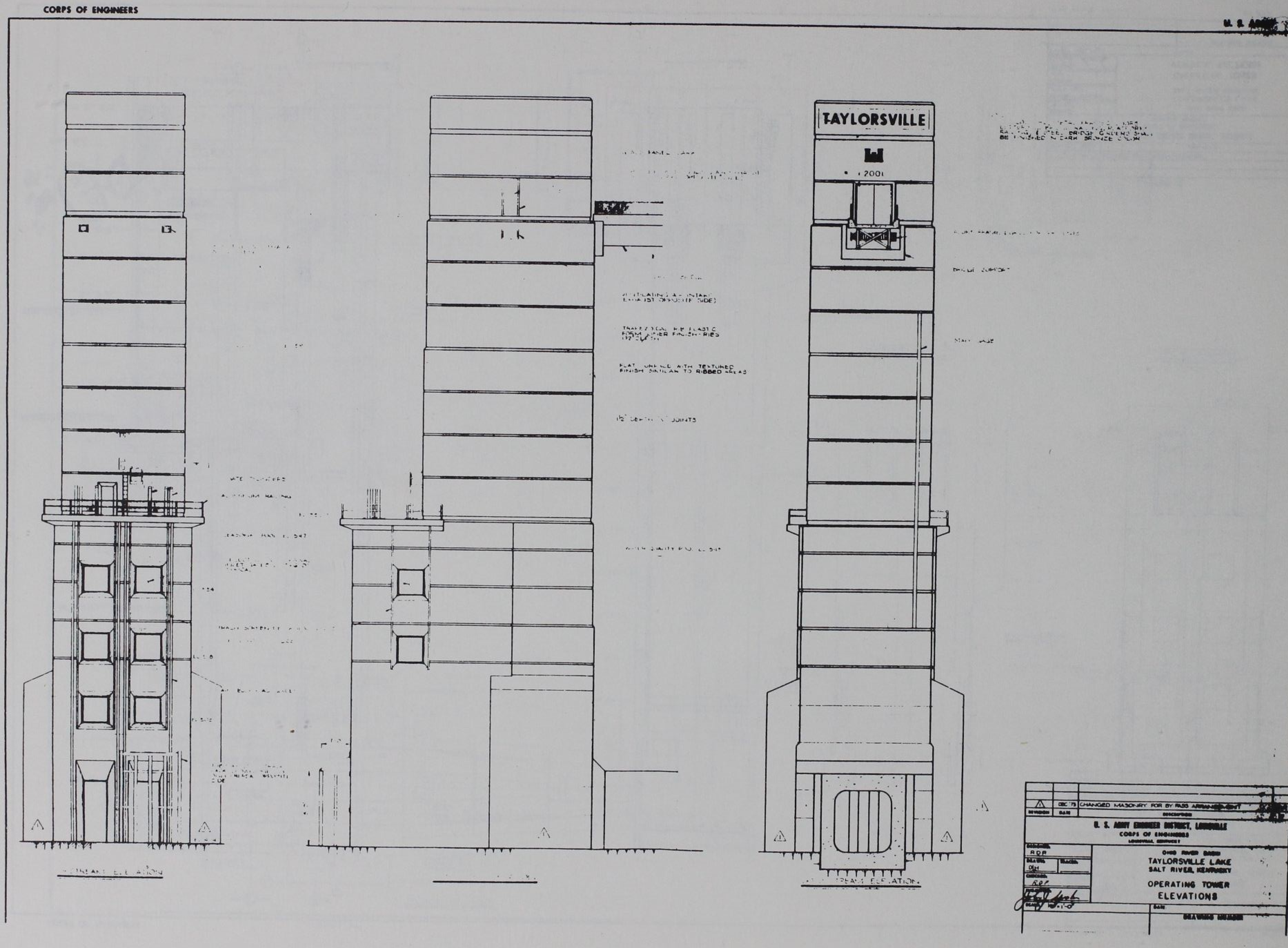
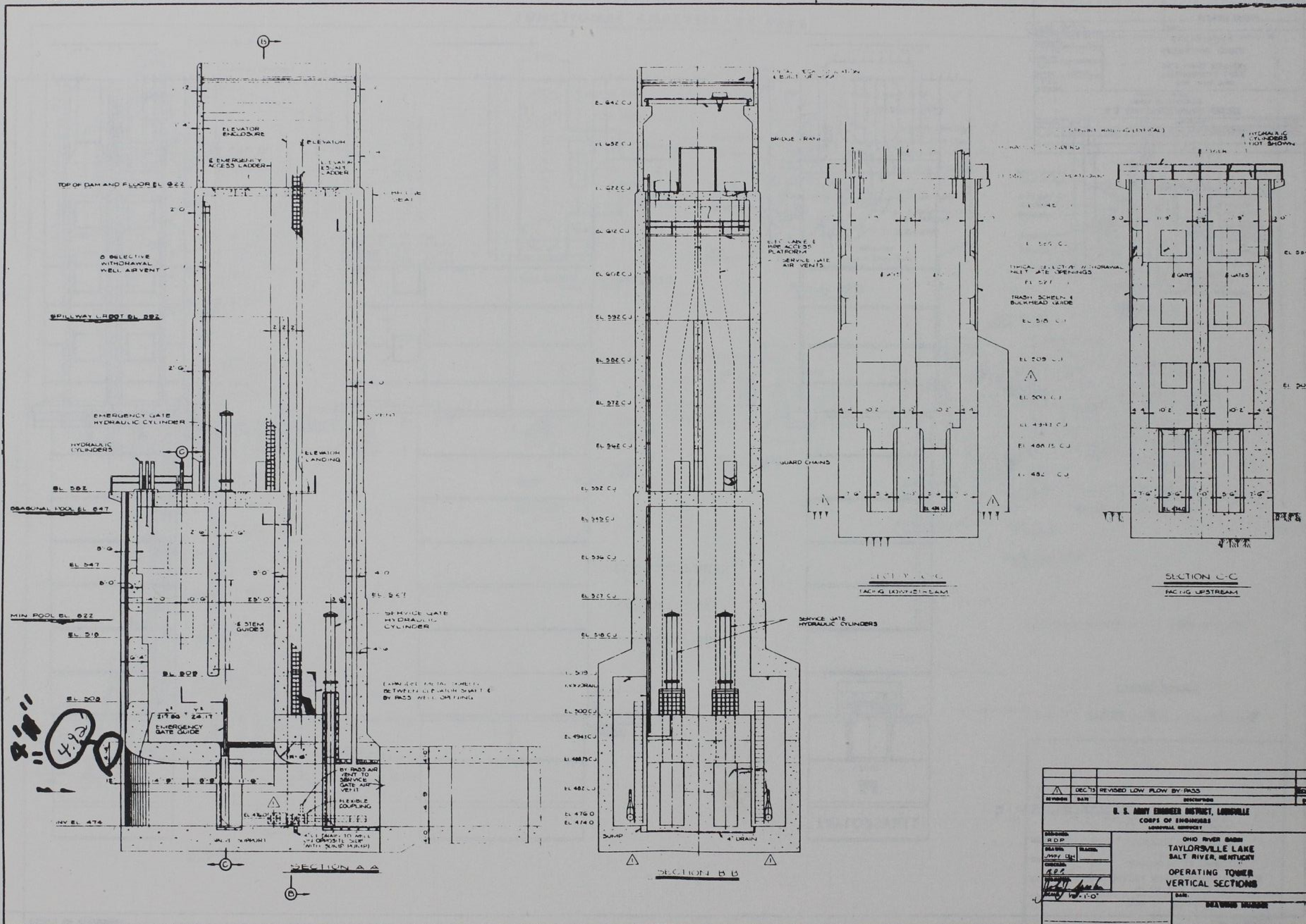
PLAN - SERVICE PLATFORM EL. 758.6
SCALE: 1" = 20'-0"
PLAN-EL. 539.5
SCALE: 1" = 20'-0"
PLAN-EL. 548.5
SCALE: 1" = 20'-0"
SECTION D-D
SCALE: 1" = 20'-0"
PLAN EL. 739.0
SCALE: 1" = 20'-0"
SECTION C-C
SCALE: 1" = 20'-0"
SECTION B-B
SCALE: 1" = 20'-0"
Note: Wet Well No. 1 has 5'-6"x12' gates. Wet Well No. 2 has 5'-4.5'x9' gates. The ℄ Elev. of the ports are El 705, El 670, El 635, El 600, & El 565.
Note: Intake Struct. not shown.
Future installation of power generating equipment
GRAPHIC SCALES
DEPARTMENT OF THE ARMY
SACRAMENTO DISTRICT, CORPS OF ENGINEERS
COTTONWOOD CREEK, CALIFORNIA
DUTCH GULCH LAKE
(900,000 AC. FT. CAPACITY)
OUTLET WORKS CONTROL TOWER
PLAN AND SECTIONS
AS SHOWN
4
SA-20-111

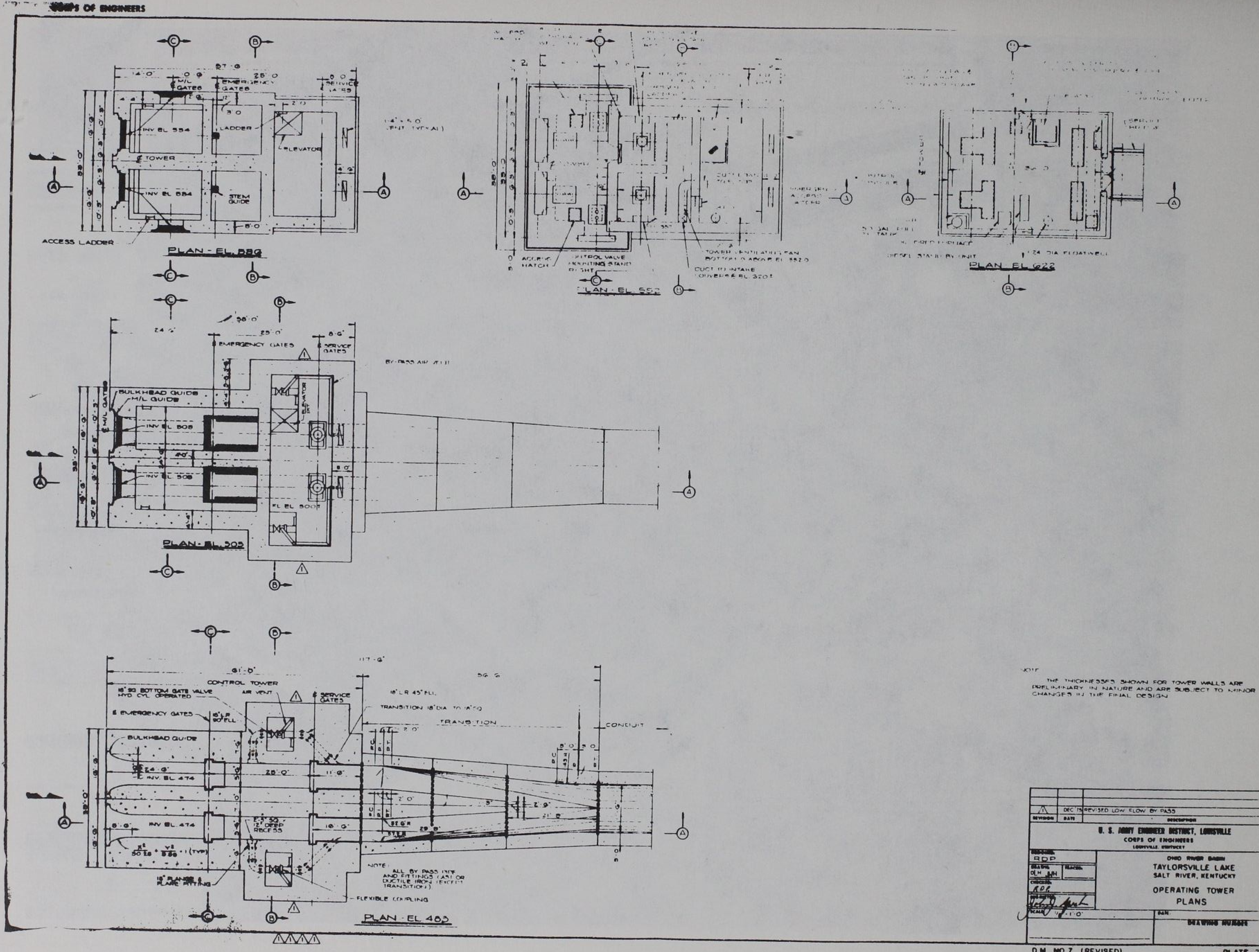

SECTION PLAN A-A
SCALE 1" = 20'-0"
Control Tower
Control Chamber
Wet Well No 1
Wet Well No 2
Intake Struct
Note: Wet Well No 1 has 5'-4.5'x9.0' gates. Wet Well No 2 has 5'-4.5'x4.5' gates. The ℄ Elev of the ports are El 661, El 626, El 591, El 556, & El 521.
PLAN - SERVICE PLATFORM EL. 714.0
SCALE 1" = 20'-0"
Stair Access
Access Bridge
10.0'x20.0' Hoistway
PLAN - EL. 526.75
SCALE: 1" = 20'-0"
5.5' Ø Penstock
Butterfly Valve (Typ.)
Bulkhead Gate
Sump & Sump Pump
Switchgear Cabinet
Emergency Gate (Typ)
Service Gate (Typ.)
Access for F.C & I. (Typ.)
2 - 30" Ø Air Conduit for F.C & I.
By-pass Piping
Oil Pressure Pumps & Oil Reservoir
Air Compressor
Governor
Elevator
Bulkhead gate
SECTION B-B
SCALE: 1" = 20'-0"
Water Quality Conduit (Typ.)
5.5'x11.25' F.C Conduit (Typ.)
Generator
Turbine
Penstock
Bulkhead Gate
Draft Tube
Exit Chamber
Butterfly Valve
Draft Tube Chamber
SECTION D-D
SCALE: NONE
Butterfly Valve
PLAN - EL. 696.0
SCALE: 1" = 20'-0"
Storage
Stairway
Hoistway
Air Intake Grille
Ventilation Duct
Emergency Diesel Generator
Supply Air Grille
Supply Air Fan
PLAN - EL. 518.0
5.5' Ø Vertical Turbine Unit #3
5.0 MW Vertical Generator
Bulkhead Slot
Emergency & Service Gate Slot (Typ.)
Log Rack not shown
Access Manhole for F.C Conduit (Typ.)
Exit Chamber
3.0'x3.0' Water Qual By-pass Conduit
2.5' Ø Horizontal Turbine, Unit 1
0.8 MW Horizontal Generator
3.5' Ø Horizontal Turbine Unit 2
2.0 MW Horizontal Generator
SECTION C-C
SCALE: 1" = 20'-0"
Service Platform El 714.0
Air Intake Louver
Access Bridge
Elevator
30" Ø Air Conduit
Control Chamber
Water Quality By pass Gate Cylinder
Emergency & Service Gate Cylinder
Access Manhole
Crane
Elevator Pit
Log Rack
2-3.0'x3.0' Water Quality By-pass Conduit
Bulkhead Gate
5.5'x11.25' F.C. Conduit
Exit Chamber
Future installation of power generating equipment
GRAPHIC SCALES
COTTONWOOD CREEK, CALIFORNIA
TEHAMA LAKE
(700,000 AC FT CAPACITY)
OUTLET WORKS CONTROL TOWER
PLAN AND SECTIONS
SA-20-111

131



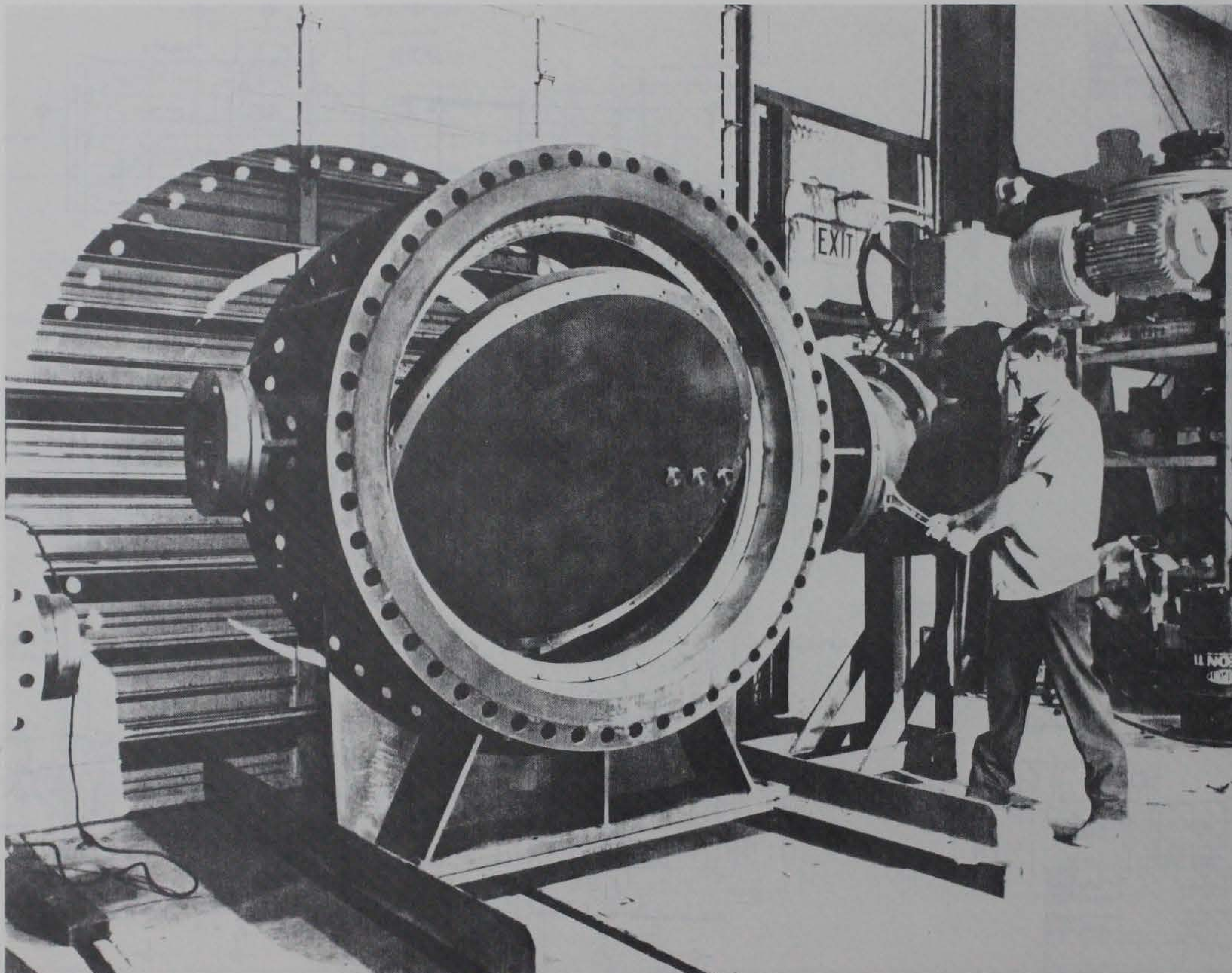
CORPS OF ENGINEERS
TAYLORSVILLE
FLAT CONCRETE WITH TEXTURED FINISH SIMILAR TO RIBBED AREAS
BRIDGE SUPPORT
CHANGED MASONRY FOR BY PASS ARRANGEMENT
CORPS OF ENGINEERS
TAYLORSVILLE LAKE
SALT RIVER, KENTUCKY
OPERATING TOWER
ELEVATIONS

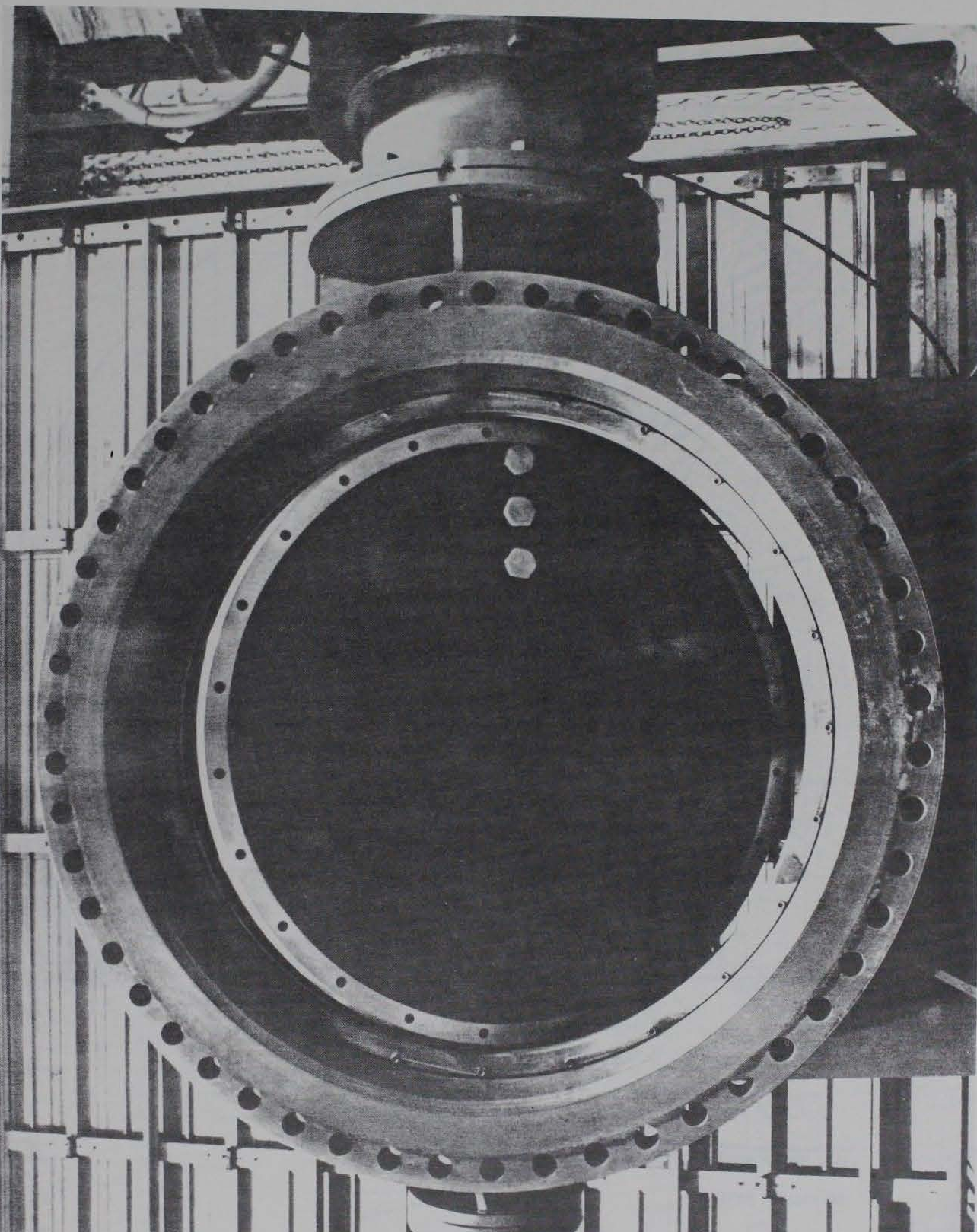

TOP OF DAM AND FLOOR EL 622
ELEVATOR ENCLOSURE
SELECTIVE WITHDRAWAL WELL AIR VENT
SPILLWAY CREST EL 592
EMERGENCY GATE HYDRAULIC CYLINDER
HYDRAULIC CYLINDERS
ELEVATOR LANDING
SEASONAL POOL EL 547
MIN. POOL EL 522
SERVICE GATE HYDRAULIC CYLINDER
EMERGENCY GATE GUIDE
FLEXIBLE COUPLING
INV EL 474
SECTION A-A
BRIDGE CRANE
SERVICE GATE AIR VENTS
GUARD CHAINS
SERVICE GATE HYDRAULIC CYLINDERS
SUMP
4" DRAIN
SECTION B-B
TRASH SCREEN & BUCKHEAD GUIDE
FACING DOWNSTREAM
SECTION C-C
FACING UPSTREAM
DEC 73 REVISED LOW FLOW BY-PASS
U. S. ARMY ENGINEER DISTRICT, LOUISVILLE
CORPS OF ENGINEERS
LOUISVILLE, KENTUCKY
OHIO RIVER BASIN
TAYLORSVILLE LAKE
SALT RIVER, KENTUCKY
OPERATING TOWER
VERTICAL SECTIONS

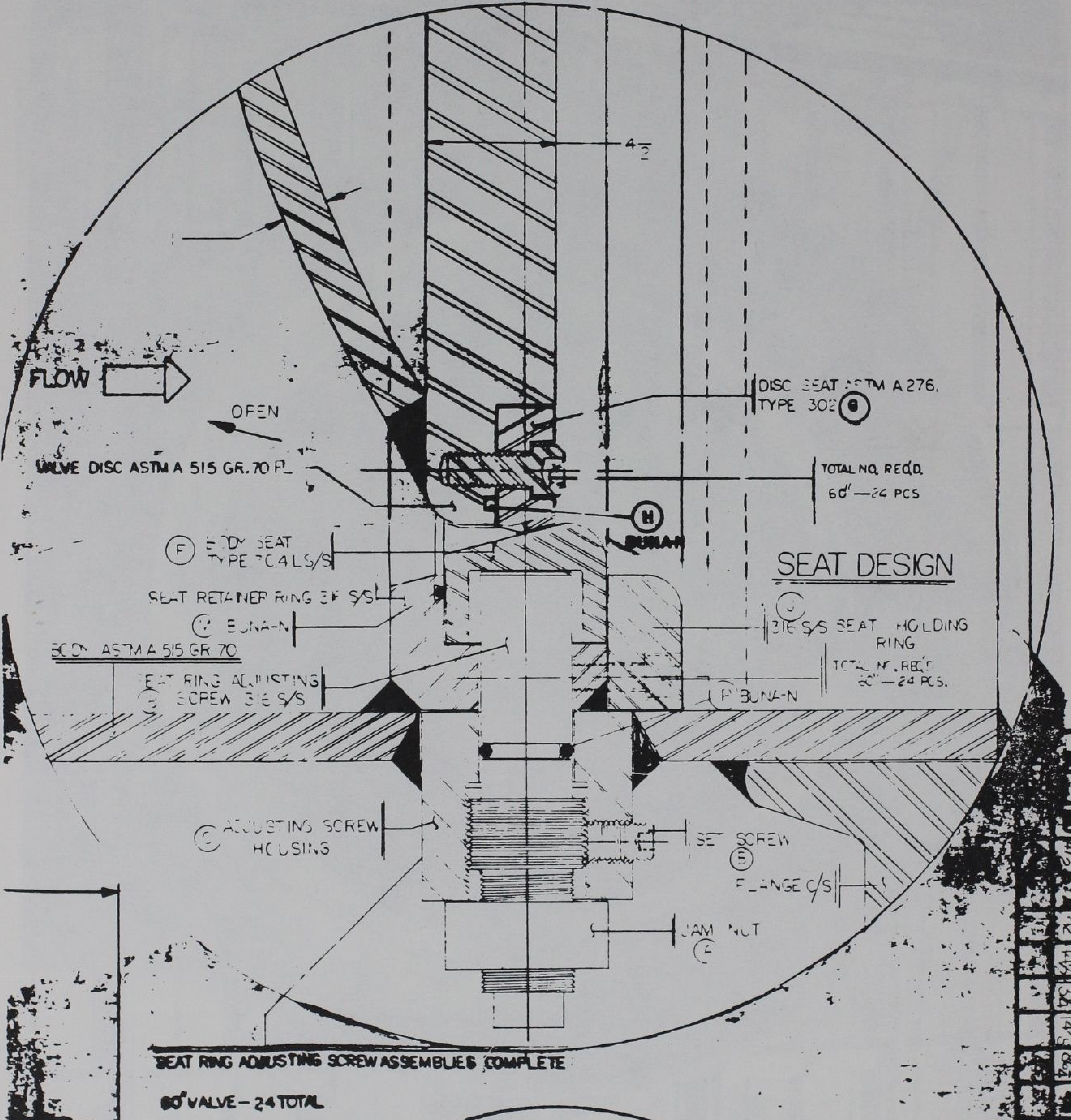


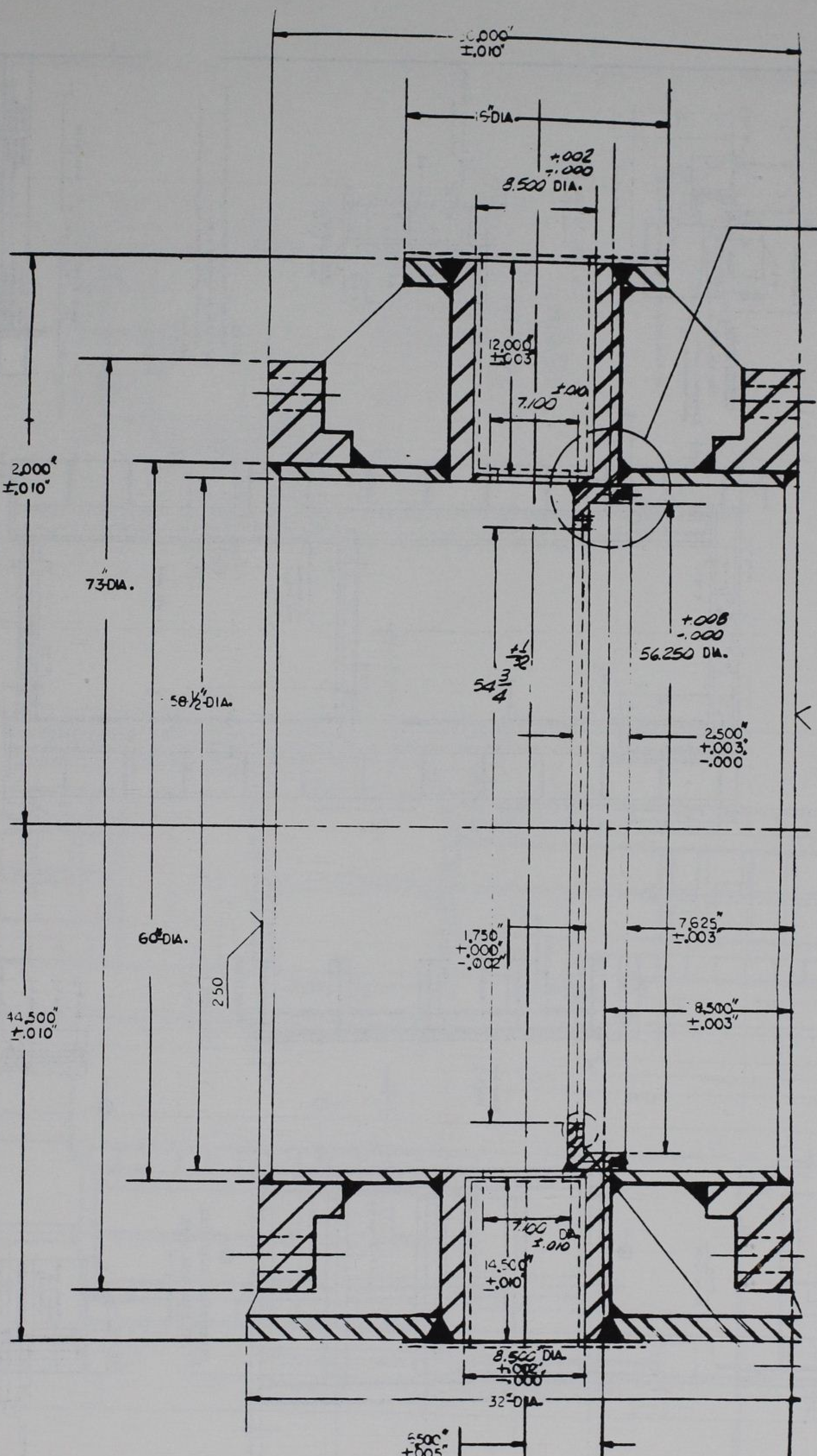
D M NO 7 (REVISED)

PLATE 2

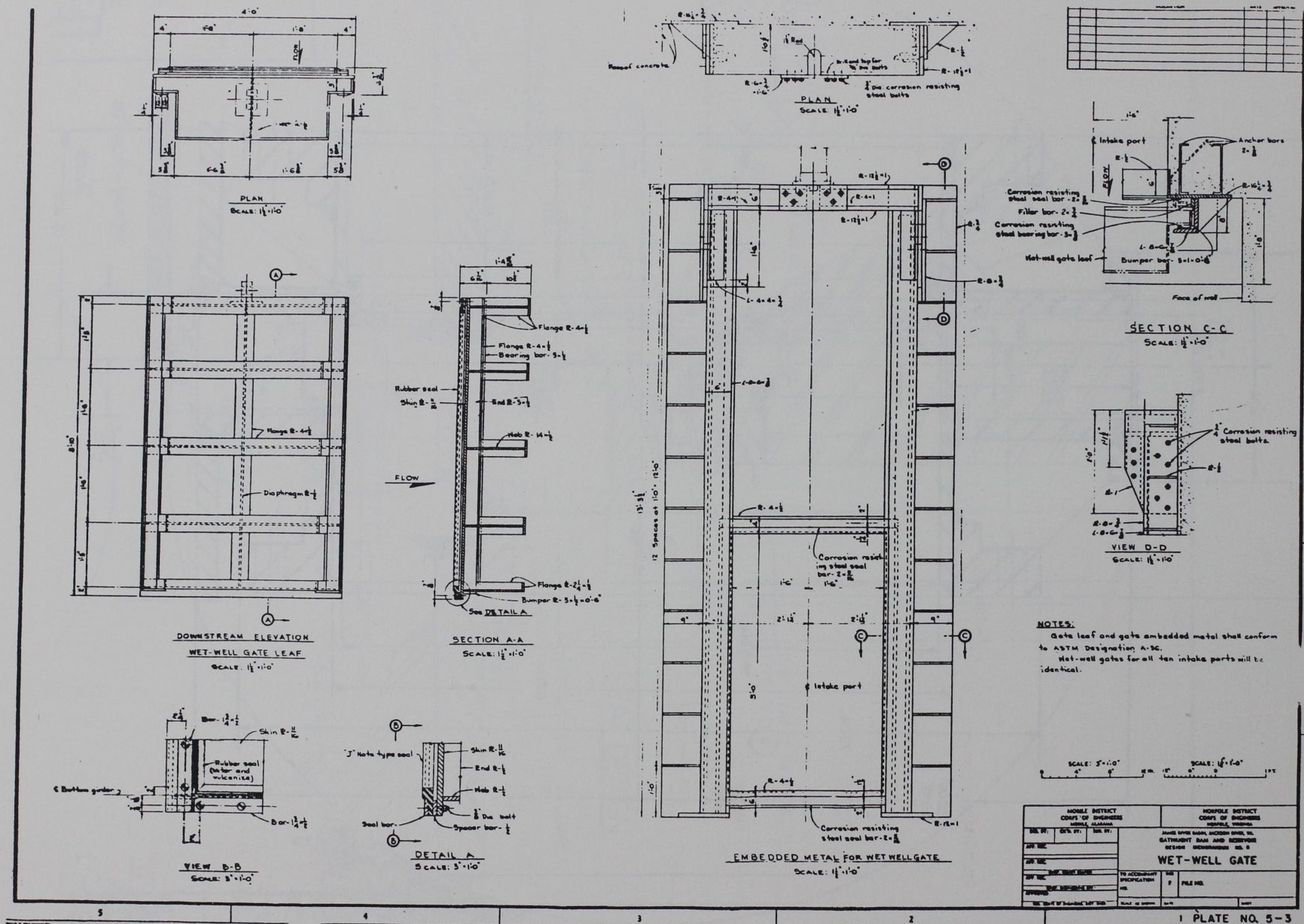

EXIT



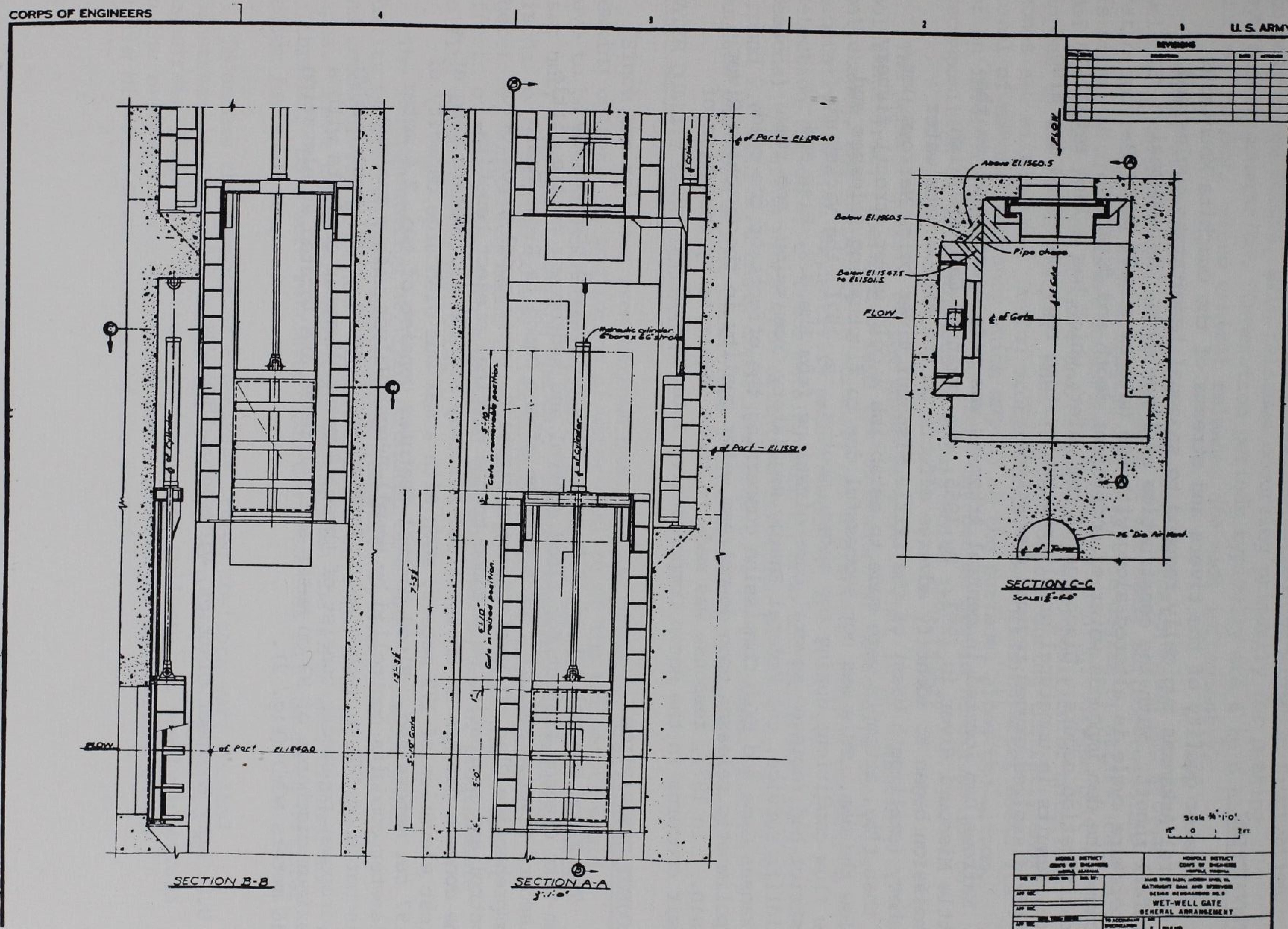

FLOW
OPEN
VALVE DISC ASTM A 515 GR. 70 PL
DISC SEAT ASTM A 276, TYPE 302
TOTAL NO. REQ'D. 60" — 24 PCS
H
BUNA-N
BODY SEAT TYPE 304 L S/S
SEAT DESIGN
SEAT RETAINER RING 316 S/S
BUNA-N
BODY ASTM A 515 GR 70
SEAT RING ADJUSTING SCREW 316 S/S
J
316 S/S SEAT HOLDING RING
TOTAL NO. REQ'D 60" — 24 PCS.
BUNA-N
ADJUSTING SCREW HOUSING
SET SCREW
FLANGE C/S
JAM NUT
SEAT RING ADJUSTING SCREW ASSEMBLIES COMPLETE
60" VALVE — 24 TOTAL



15" DIA.
+.002
-.000
8.500 DIA.
12.000"
±.003"
±.010
7.100
2.000"
±.010"
73" DIA.
58 1/2" DIA.
60" DIA.
250
44.500"
±.010"
+1/32
54 3/4
+.008
-.000
56.250 DIA.
2.500"
+.003"
-.000
1.750"
+.000"
-.002"
7.625"
±.003"
8.500"
±.003"
7.100 DIA.
±.010
14.500"
±.010"
8.500" DIA.
+.002"
-.000
32" DIA.
.500"
±.005"
±.005
15.000"


PLAN
SCALE: 1½" = 1'-0"
DOWNSTREAM ELEVATION
WET-WELL GATE LEAF
SCALE: 1½" = 1'-0"
SECTION A-A
SCALE: 1½" = 1'-0"
FLOW
Rubber seal
Flange
Bearing bar
Web
Diaphragm
See DETAIL A
PLAN
SCALE: 1½" = 1'-0"
Face of concrete
¾" Dia. corrosion resisting steel bolts
EMBEDDED METAL FOR WET WELL GATE
SCALE: 1½" = 1'-0"
12 Spaces at 1'-0" = 12'-0"
℄ Intake port
Corrosion resisting steel seal bar
SECTION C-C
SCALE: 1½" = 1'-0"
℄ Intake port
Anchor bars
Filler bar
Wet-well gate leaf
Face of wall
VIEW D-D
SCALE: 1½" = 1'-0"
¾" Corrosion resisting steel bolts
VIEW B-B
SCALE: 3" = 1'-0"
℄ Bottom girder
Skin
Rubber seal
DETAIL A
SCALE: 3" = 1'-0"
"J" Note type seal
Seal bar
Spacer bar
NOTES:
Gate leaf and gate embedded metal shall conform to ASTM Designation A-36.
Wet-well gates for all ten intake ports will be identical.
MOBILE DISTRICT
CORPS OF ENGINEERS
NORFOLK DISTRICT
CORPS OF ENGINEERS
WET-WELL GATE

1 PLATE NO. 5-3


CORPS OF ENGINEERS
U. S. ARMY
FLOW
Above El.1560.5
Below El.1560.5
Pipe chase
Below El.1547.5 to El.1501.5
℄ of Gate
36" Dia. Air Vent
SECTION C-C
℄ of Port - El.1564.0
Hydraulic cylinder 6" bore x 66" stroke
℄ of Cylinder
℄ of Port - El.1552.0
℄ of Port El.1540.0
SECTION B-B
SECTION A-A
Scale 3/4"=1'-0"
WET-WELL GATE
GENERAL ARRANGEMENT

SELECTIVE WITHDRAWAL NEEDS FOR LAKE GREESON ARKANSAS INTAKE STRUCTURE

BY R.E. PRICE AND D.R. JOHNSON*

INTRODUCTION

The water quality of the rivers and streams of the Ouachita Mountains in central Arkansas typically respond to seasonal temperature cycles and rainfall runoff. With the construction of large dams, the seasonal temperature cycle is altered along with the natural runoff pattern. Because the dam impounds water of sufficient depth and duration, stratification occurs. Discharge of the water, which has a long retention time, results in unnatural or altered temperature regimes. This may also modify dissolved oxygen regimes downstream.

Narrows Dam located in central Arkansas was made operational on the Little Missouri River in 1953. By 1955, the Arkansas Game and Fish Commission began to identify adverse effects on the native warm water fishery (smallmouth bass) of the Little Missouri River below Narrows Dam. At that time, attempts were made to establish a cold water (trout) fishery below the dam. This was not successful due to fluctuating releases and low flow conditions during the summer months. By 1971, the Vicksburg District had recieved several communications from the U.S. Fish and Wildlife Service, the Federal Energy Regulatory Commission, and the Arkansas Game and Fish Commission concerning the effects of the cold, hypolimmetic releases upon downstream water quality in the Ouachita River Basin. In 1977, response was made to these inquiries by inclusion of their concerns in the Basin Comprehensive Interim Study.

PROJECT DESCRIPTION

The Flood Control Act of 18 August 1941, authorized the Narrows Dam-Lake Greeson Project for flood control and hydropower on the Little Missouri River. (Fig. 1) As a portion of the Ouachita River Basin Comprehensive Plan of Development; navigation, water supply, pollution control and recreation were added to the original project benefits. The concete dam has an uncontrolled spillway section 45.7 meters long at a crest elevation of 172 meters, NGVD with a maximum dishcarge capacity of 1197 cu. meters sec. Flood control features consist of two, 2.6 meter diameter conduits controlled by Howell-Bunger Valves with a maximum discharge of 178 cu meters sec. Intakes are located at 131 meters NGVD. Hydropower penstocks consist of three, 3 meter diameter conduits with a maximum discharge of 92 cu meter sec. Intakes are located at elevation 146 meters NGVD (Fig. 2).

* U.S. Army Engineer District, Vicksburg, LMKED-HW

The rule curve for Lake Greeson allows for raising the pool from 161 meters, NGVD, on 1 January to 167 meters, NGVD on 15 June, followed by a gradual lowering to 161 meters, NGVD, on 25 November. Although the fluctuation is only 5.5 meters, actual operation can minimize fluctuation to only a few meters, with releases occurring primarily for peaking hydropower generation. Generation periods typically are 4 to 6 hours during the day. Flood control releases are made infrequently.

Thermal stratification of Lake Greeson begins in March and extends well into November each year. Because of the small surface-to-volume ratio and retention time averaging 395 days, the degree of stratification is strong with withdrawal occurring in the hypolimnion. Oxygen poor or deficit zones due to organic and/or bacterial decomposition of materials appear in the water column. When overturn occurs during the fall, anoxic zones are eliminated. During storm events, the inflowing water seeks its level of natural bouyancy which may be on the surface, along the bottom, or an interflow zone. These flows may become devoid of oxygen due to decomposition of allochthonous materials which they carry.

The general quality of the water is good in that it is relatively pollutant free. There have been few occurances of detectable levels of pollutants, and turbidity is normally not a problem. Mineral content is generally low with conductivity measurements at the lower limit of detection (5Ø umhos). Trace metals, such as iron and manganese, exhibit seasonal variation along with oxidation reduction potentials. pH ranges around 7.Ø with more basic conditions occurring in the summer near the surface and more acidic conditions in the bottom hypolimnial waters.

WATER QUALITY DATA COLLECTION

Since the releases from Narrows Dam are made primarily for hydropower peaking operations, monitoring of water quality of the tailwaters would have to be within discharge periods. Typically, generation periods will last only a few hours during the day. During nongeneration periods, discharge is limited to .28 - .42 cu meters sec. of gate leakage. Therefore, monitoring needs to be at least at hourly intervals to detect effects of discharges. This was best accomplished by use of programmable in situ water quality monitors.

The instrumentation utilized consisted of a four parameter water quality monitor which measured water temperature, dissolved oxygen, pH, and conductivity at regular intervals and stored the data on cassette tapes. These tapes were retrieved weekly when the units were cleaned and calibrated. Data were retrieved from the cassettes and stored on computer medium for analysis.

To assess the impacts of discharges, three monitors were located downstream: 1.2 km miles below the dam; 7.Ø km downstream; and 16.7 km downstream (Fig. 1). Vertical profiles were collected at weekly intervals in the reservoir upstream of the dam. One monitor was located in the Little Missouri River upstream of the reservoir.

RESULTS OF MONITORING

The results of monitoring are best analyzed by considering that Narrows Dam is a feature of the Little Missouri River, therefore analysis of existing conditions must include inflow water quality as well as reservoir and release quality. If the objective is to return the tailwaters of Lake Greeson to as near as possible to pre-1950 conditions, data collected on inflow will be as close to pre-reservoir conditions as possible.

The water quality of the Little Missouri River as it flows into Lake Greeson is affected by seasonal, synoptic, and diel changes of meterological conditions . As weather systems pass through the watershed, discharge increases with runoff. Temperatures will change somewhat as a result of ground temperature and dissolved oxygen levels will respond to increased aeration and oxygen demanding substances.

Lake Greeson is not impacted to the extent the Little Missouri River is by synoptic or diel events. However, extreme rainfall events may have signficant impacts.

As indicated earlier, the water releases are primarily through the hydropower penstocks. During the non-stratified periods (winter) elevation of intakes has no effect on discharge; however, as stratification sets in, the level becomes critical. The water being discharged comes from 12 to 20 meters below the surface, which would indicate it is cooler than surface water. In comparison to inflow termperature, it is cooler throughout the spring and summer but is relatively warmer in the fall. Figure 3 illustrates the extreme differences in inflow, reservoir and discharge temperatures that develop over the fishery spawning period.

As the discharge proceeds downstream, it warms during the summer months but is dependent on duration as well as volume of discharge. High discharges require greater distances downstream to reach equilibrium.

In regard to dissolved oxygen levels of the discharge, the 5 mg/l standard should be attained year round. However, during the late summer and early fall, the dissolved oxygen level drops below 5 and at some periods below 1 (Figure 4).

It is apparent that the discharge from Lake Greeson is vastly different from what would be if Narrows Dam was not there. Some method to provide warmer water during the summer as well as higher dissolved oxygen levels without significantly impacting the reservoir water quality is needed.

ALTERNATIVES:

There are a variety of techniques to prevent or mitigate the problem of cold water discharges from a reservoir. Since the flood control gates are at a lower elevation than the hydropower penstocks, operational changes would not improve the quality of the releases. Other alternatives such as destratification, surface pump down, mechanical pumping and submerged weirs, impact the reservoir water quality and/or are too costly. Therefore, some form of multilevel outlet appears to be the best means of improving released water quality.

Most multilevel intake structures consist of several vertically spaced intake ports controlled by gates and draining into a wet well. Since the intakes for the penstocks are located in the face of the dam, some modification could be made either by addition of an intake tower or directly to the intake penstocks. The simplest modification involves plating over the trash racks in front of the penstocks. This modification could be achieved at relatively minor costs and yet draw water from a higher elevation. In addition, the plates could be placed in segments and thus allow for operational changes to achieve a multilevel outlet capability. One alternative which was proposed by Stafford consisted of a fixed bulkhead or plate over the lower half of the trash rack with a smaller, movable bulkhead which could be raised with lower lake stages.

To similate water quality impacts, temperature and dissolved oxygen profiles for May through October 1982 were modeled using SELECT. The results which are plotted in Fig. 5, indicate release temperatures are increased by as much as 13 degrees C., but remain below the target temperatures. Dissolved oxygen is also increased, but not as dramatically. Dissolved oxygen remains above 5.0 mg/l for all months except September because release water is drawn from the epilimnion and the metalimnion instead of from the hypolimnion. Further modeling using a dynamic model is planned.

References

1 "Final Report, Arkansas Lakes Interim Study" Ouachita Baptist University, Arkadelphia, Arkansas, 1985.

2 Hulsey, A., Correspondence to U.S. Army Engineer District, Vicksburg, Mississippi from Arkansas Game and Fish Commission Little Rock, Arkansas, 1975.

3 "Lake Greeson, Narrows Dam, Little Missouri River, Arkansas, Periodic Inspection Report No. 2," U.S. Army Engineer District Vicksburg, Mississippi, 1975.

4 Stafford, J.P. III, Thermal Analysis of Lake Greeson, Arkansas and Sardis Lake, Mississippi, Masters Thesis, Mississippi State University, 1978.

5 Thornton, K.W., J.F. Nix and J.D. Bragg. Coliforms and Water Quality: Use of Data in Project Design and Operation. Water Resources Bulletin, Vol. 16, 1980, pp. 86-92.

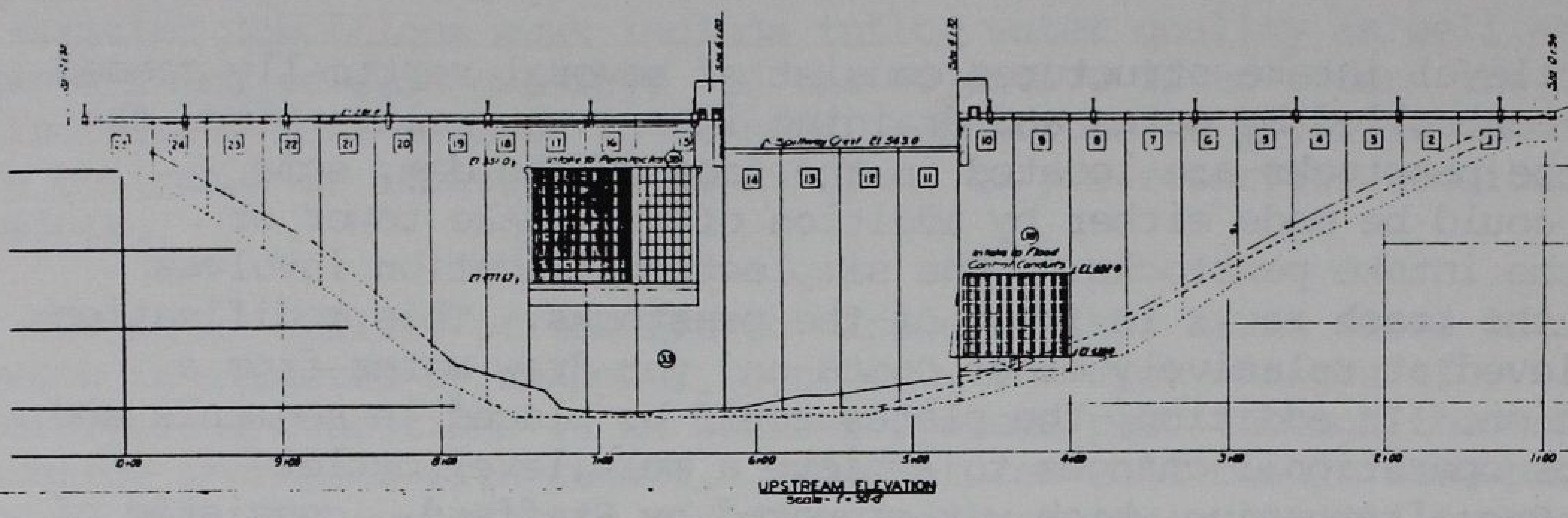
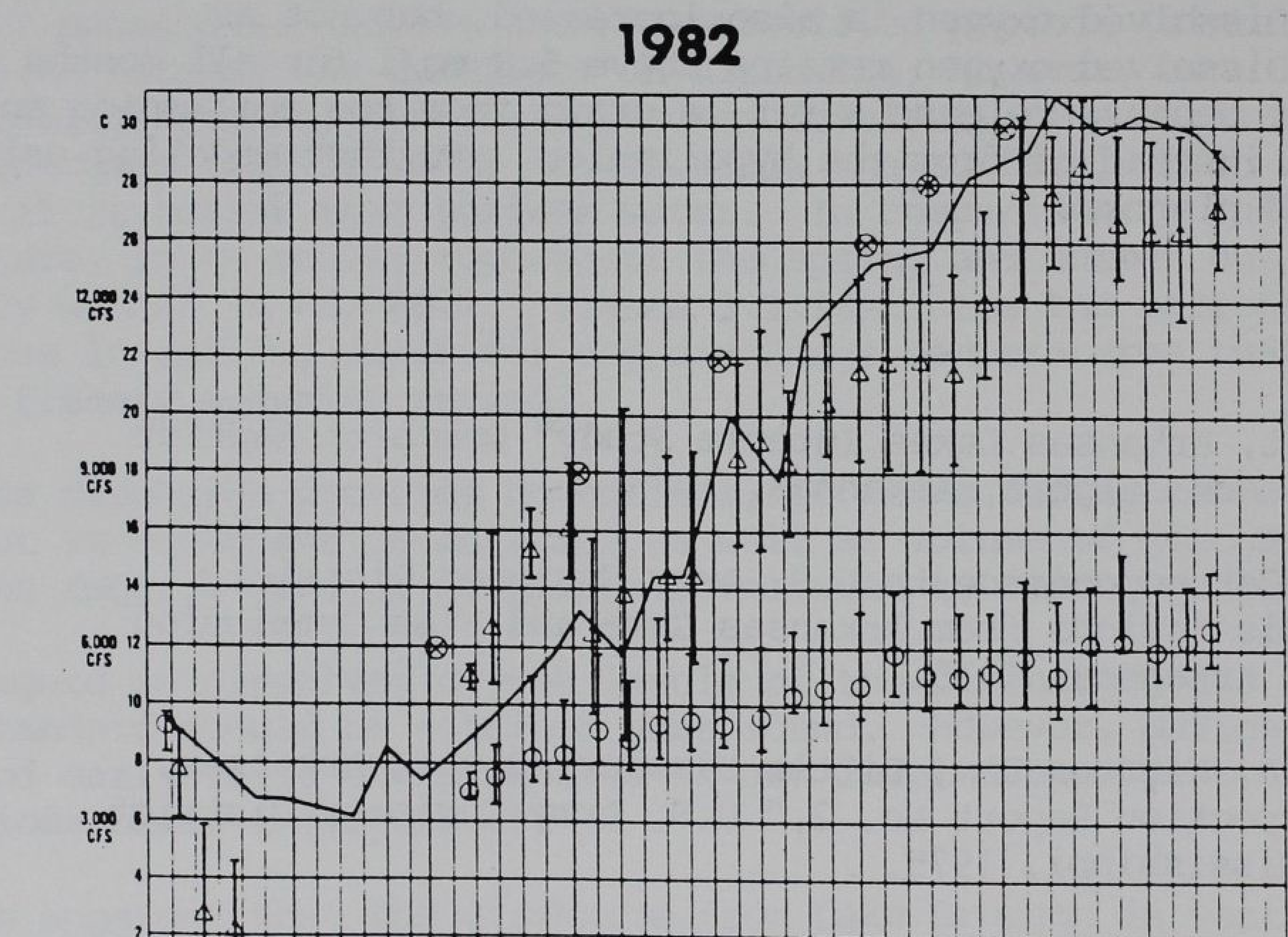


FIGURE 1. PLAN VIEW OF NARROWS DAM, LAKE GREESON, ARKANSAS



LEGEND

- NARROWS DISCHARGE TEMP
- LAKE GREESON SURFACE TEMP
- RECOMMENDED TEMP MINIMUMS, USFWS
- LANGLEY TEMP

NARROWS DAM
AND
LAKE GREESON

FIGURE 3. INFLOW, LAKE AND DISCHARGE TEMPERATURE TO LAKE GREESON

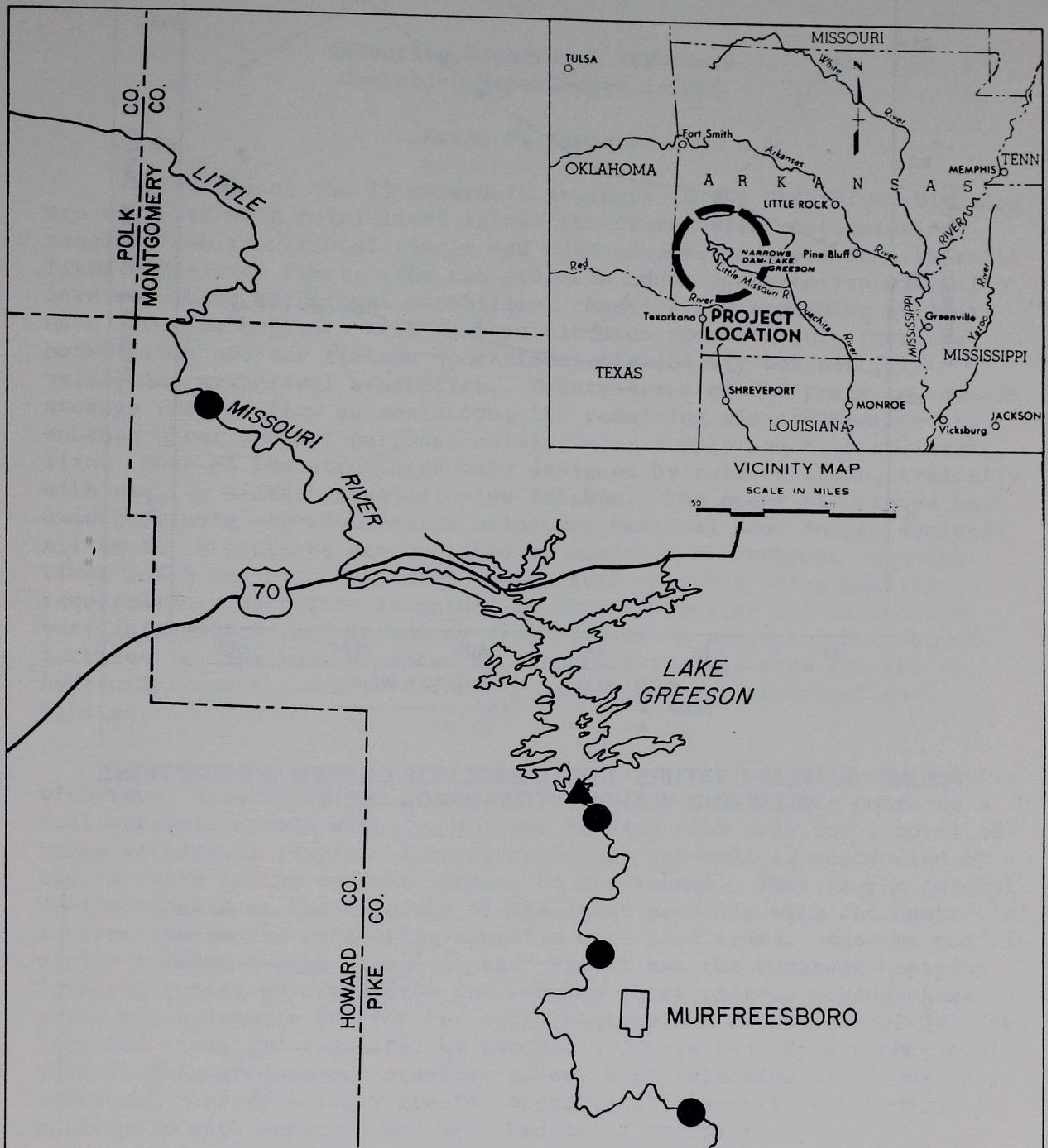


FIGURE 2. LOCATION OF WATER QUALITY MONITORS ON THE LITTLE MISSOURI RIVER

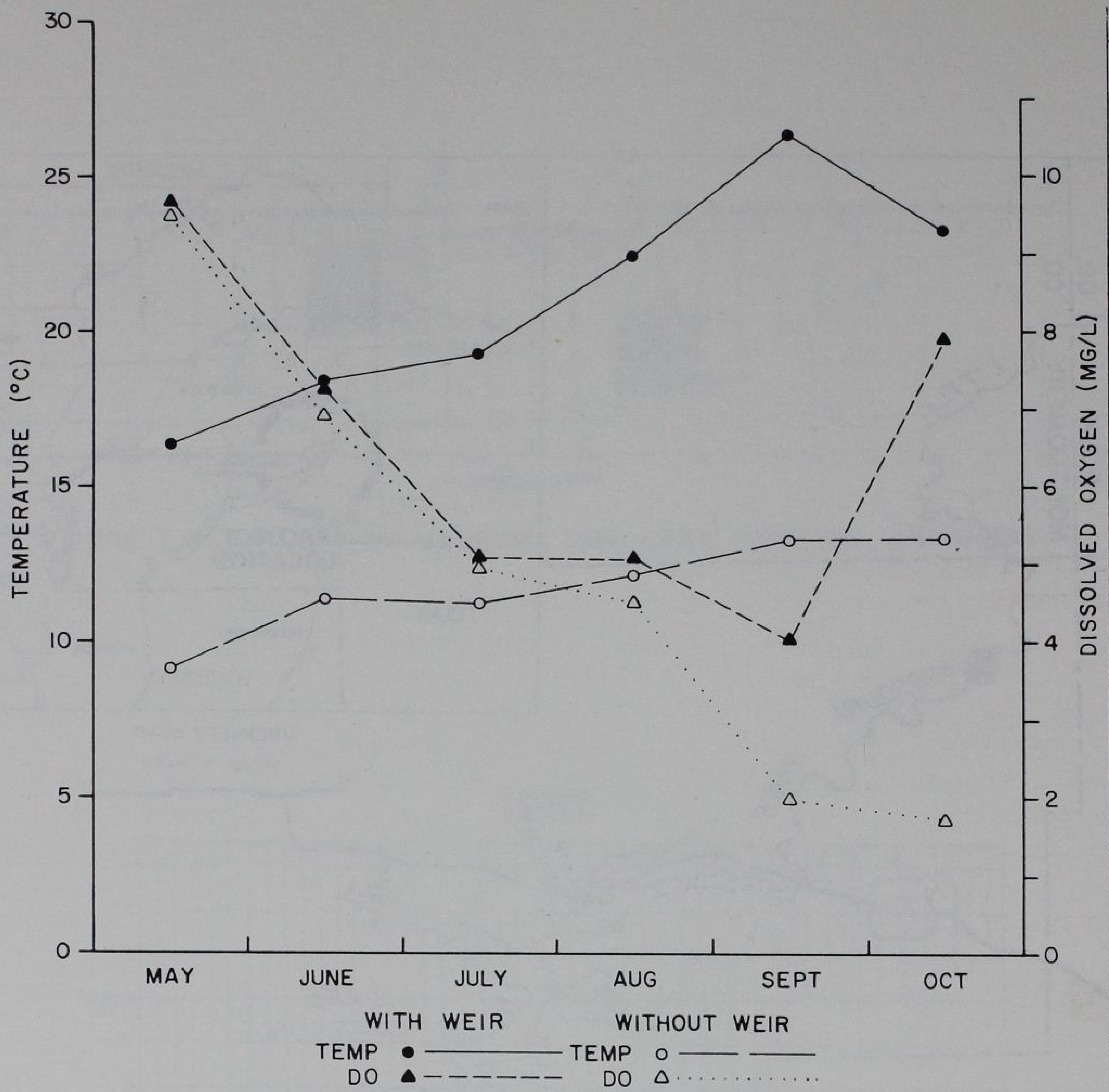


FIGURE 5. TEMPERATURE AND DISSOLVED OXYGEN PREDICTIONS WITH AND WITHOUT TRASHRACK WEIR

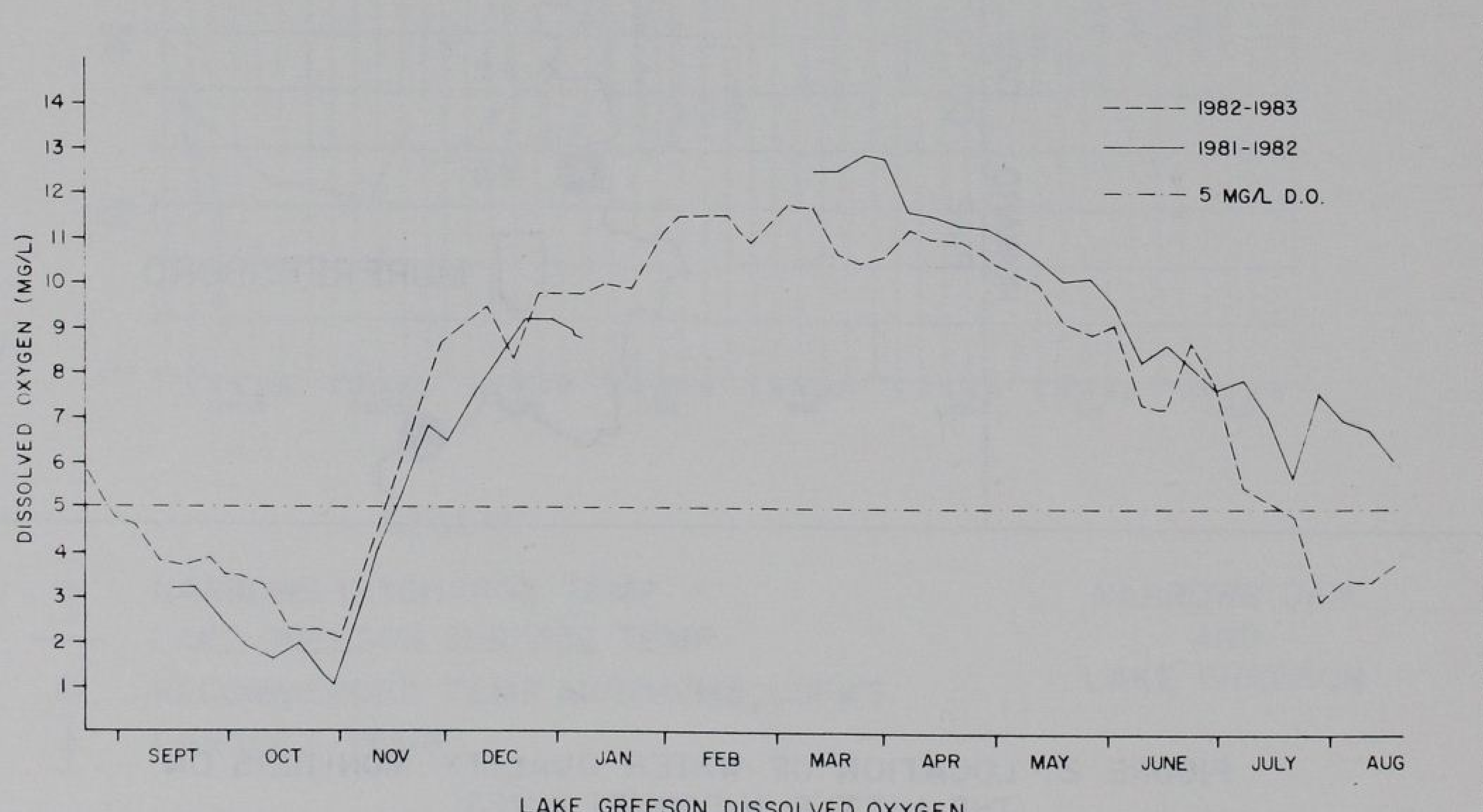


FIGURE 4. DISSOLVED OXYGEN CONCENTRATION OF LAKE GREESON TAILWATER FOR 1981 THROUGH 1983

Selective Withdrawal Structure
Operation Experiences in ORD

David P. Buelow*

Thirty-one of the 75 reservoir projects in the Ohio River Division are equipped with multi-level intake structures with configurations ranging from traditional single and dual wet-well systems to a retrofit fixed high-level riser. The two projects under construction will also have selective withdrawal capability. Many of the remaining projects have low-flow bypasses with inverts elevated some distance from the bottom that provide limited operational flexibility but not full sellective withdrawal capability. Twenty-seven of the projects contain storage for low-flow augmentation; the remaining six are operated to enhance other project purposes mainly water supply and fish and wildlife. Most of the structures were designed by rule-of-thumb, typically with equally sized high-middle-low intakes. The newer structures have undergone more rigorous design using mathematical heat budget analysis. All of the structures are operated to maximize achievement of objectives which may be a tailwater temperature or other water quality requirement. Operation is guided by data collection activities that vary in magnitude and intensity from project to project. Meeting objectives is sometimes hindered by structure inadequacies related to hydraulic capacity and port location and frequency of operational updates.

East Branch Clarion River Lake, the first project with a selective withdrawal structure, was placed in operation in 1952. It contains a dual wet-well system with two intakes feeding each well for a total of three withdrawal levels. Flow through each wet-well is controlled by a bypass valve in the conduit leading to the tunnel. This is the general design adopted at the majority of the other projects with the numbers of intakes and levels reflecting specific site conditions. Another modification to this design in use at one project has the bypasses emptying into the tunnel upstream from the service gates thereby making those gates the hydraulic control for both low-flow and flood control releases. Very low flows are regulated by two small valves that bypass the service gates. This arrangement provides a very high selective withdrawal capacity and correspondingly greater operational flexibility not normally achievable with seperate wet-well hydraulic controls.

Several single wet-well structures are in use in the Division. Inherent in their design is limited hydraulic capacity and inability to predictably blend, however, this does not hamper their operation to meet objectives as effectively as possible. Single wet-well blending is

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accomplished at some projects where warranted to meet downstream requirements. This is done by cracking open the lower intake gate in an attempt to restrict the inflow of cooler water to the tower. Success is measured by the response in the discharge quality.

Another type of selective withdrawal system in use consists of sluiceways with no common wet-well. This arrangement provides a great deal of flexibility since each sluice operates independently. A modification of this is the high-level riser retrofit at Sutton Dam that provides access to epilimnal waters by one of the low-level flood control sluices.

One project, Green River Lake, has a segmented, semi-circular gate arrangement that, through a series of manipulations, can be used to withdraw water from nine levels. A similar system will be used at Stonewall Jackson Dam now under construction. Such systems also offer great flexibility, however, unwieldiness of operation may be a problem. For instance, at Green River only three of the levels are routinely used.

Operational criteria vary for each of the projects and at some the specific objectives have changed to reflect new in-lake or downstream requirements. The ability to meet objectives is dependent on the flexibility allowed by the structure design and the amount of effort expended in regulating. A warm or cold water release is the typical objective. This sounds simple, however, other factors enter in and make the decision process more complicated. We're all familiar with the concept of knowingly violating the cold water objective early in the year in order to better meet it later in the season through conservation of the cold water supply. A more realistic case involves trade-offs between temperature and other water quality parameters when we have to draw from an anaerobic hypolimnion. The question then becomes do we violate the temperature objective or release excessively high levels of iron, manganese, etc. Coordination with responsible state agencies is absolutely necessary in this case. A similar situation arises with fall drawdown discharge and the service gates must be used. Restructuring the drawdown schedule should be considered if severe water quality problems downstream are likely. This situation exists at many of the older projects that were designed by traditional methods; contemporary design standards require a thorough evaluation of discharge capacity needed to meet all water management objectives.

Water quality data is a necessary ingredient for making intelligent operating decisions for selective withdrawal structures. The program of data collection is customized for each project to provide the needed information and always consists of lake profiles and tailwater data. Profiles are taken at least once per week, usually on Monday, and guide the selection of intake levels. Tailwater data is collected by project personnel at least twice per week and with every gate change. Huntington District has installed robot monitors and has incorporated them into the GOES network. That is the preferred course of action, providing an abduance of data that can be readily accessed to assess performance and aid in formulating operational changes. Data interpretation and determination of operating changes should be handled by the Water Quality Section and coordinated with the Reservoir Regulation Staff for dissemination. This doesn't always happen, however, and Res-Reg sometimes ends up being the only group involved. Lack of manpower is a major operational problem, and the amount of effort a district dedicates to this task is a measure of the importance assigned to the function.

Operational tools such as SELECT aren't used for day-to-day operation for two primary reasons; 1) the previously mentioned people shortage; and 2) lack of improvement in the end product. The fact is that we do a pretty good job now of meeting objectives given all the constraints on people, data and structure design. Failures can usually be explained within that framework. Another consideration is that at some projects our control of tailwater quality diminishes very rapidly after the water exits the outlet portal. For instance, very rapid heating in the tailwater under minimum flow conditions can negate all efforts to regulate effectively. The method used to regulate involves lining up intake elevations against the water quality profile and choosing which level or levels to withdraw from. If blending is required, the flows are proportioned accordingly. This technique provides an adequate level of performance.

Problems with our selective withdrawal structures can be categorized as structural/hydraulic and institutional. The former group includes factors such as insufficient bypass capacity, undersized and misplaced intakes. The end results are excessive head losses in some wet-wells, damage to wet-wells and bypass valves, cavitation at the bypass exit portal in the conduit and general lack of flexibility in operation. Damage to wet-wells and valves can present serious operational problems when the system has to be shut down for repair. This can be a critical situation for a single wet-well system and a challenging situation for a dual wet-well system, especially one that is undersized. Decisions then must be made as to how to operate the project without totally degrading the tailwater and options such as temporary storage of summer runoff or extending flood storage evacuation must be considered. Institutional problems can become important here. An operational change to benefit a selective withdrawal objective has to be evaluated against potentially conflicting project purposes such as recreation and flood control. A change may not be tolerable at a project with limited flood control storage or one with established recreation facilities on the lake.

A similar situation arises at some projects during Fall drawdown. In cases where the bypass capacity is much less than the regulated drawdown discharge, the flood control gates must be used to follow the rule curve. At a few projects, the additional problem of cavitation occurs when the service gate and bypass on the same side are operated. In either case, when the hypolimnion is anoxic, the risk of discharging water of less than desirable quality is great. A possible solution is to reschedule drawdown until after fall turnover. Another factor complicating this decision in the Ohio River Basin is that, although not authorized for navigation, the fall drawdown at the reservoirs does provide much needed water that enhances navigation during the normal fall low-flow period. Recommendations for variance from approved rule curve operations have to be evaluated against all potentially affected uses initially at the District level and finally at the Division level.

Little can be done to improve operational flexibility when all of the intakes are below the thermocline, as is the case at a few projects. This happened at one of them because, during construction, it was decided to raise the summer pool level significantly but the tower and intake levels were not redesigned. Here and at the other similar projects we've been remarkably lucky: the lakes don't go anoxic, a good quality coldwater release is provided and good lake conditions are maintained. Future design of new projects would not rely on luck as a design consideration, however.

The retrofit development of hydropower generation facilities at existing projects presents some interesting opportunities and challenges. Opportunities exist for increased project discharge capacity and operational flexibility with a new intake structure and conduit to service just the powerhouse and for upgrading of existing intake structures if they are to be used for hydropower regulation. Federal development proposals will address selective withdrawal aspects and incorporate features to minimize impacts and enhance operation. Non-federal development proposals, on the other hand, are likely to be simplistic and naive and lack any understanding of selective withdrawal. Dealing with such potential developers presents the challenge. Our approach is to not tolerate any reduction in our current operational capabilities and to encourage enhancement at least to the level that we would consider. This always means that the developer must demonstrate the capabilities of his proposed system, and this usually means that physical and/or mathematical modeling is required. We won't require more effort than we would expend, but we do require documented assurances without which we can withhold approval of start of construction and negotation of the Memorandum of Agreement covering project operation.

The real challenges with non-federal developers are to make it clear exactly what our requirements are, to judiciously evaluate proposals and, if necessary, to educate them about selective withdrawal, reservoir stratification and their interrelationship with reservoir regulation. Questions such as what happens to the operating characteristics of a dual wet-well system when it is pressurized and how should the project be operated in the summer when turbine capacity is much greater that selective withdrawal capacity are extremely complex and perhaps not totally solvable. Coordination, strict review and prudent compromise are needed to insure that the Corps' interests are not undermined.

Besides hydropower add-on development, the area of greatest application of selective withdrawal for the foreseeable future will be in improved operation and upgrading of existing projects. Focus of attention should be on projects where recurrent or serious problems exist. Measures that may be considered include modifying data collection programs, new techniques for determing gate operations, closer in-house coordination, educating project managers, temporary rule curve changes to avoid water quality problems, and physical modification of structures. The last option is the most costly and controversial, but, if a serious enough problem exists, a structural alteration may be justifiable and should be studied. The high-level riser at Sutton Dam was studied, justified and ultimately built, and upgrading the selective withdrawal capability at at least one other ORD project has been considered. This is an option that should not be overlooked.

In summary, ORD has constructed many selective withdrawal structures with varying degrees of success in design and resulting operation. Regardless of the shortcomings, a concerted effort is made to get the best performance possible from the structures in terms of meeting objectives while continually monitoring and noting deficiencies and evaluating structural or operational alternatives when warranted.

Overview of Pittsburgh District Selective Withdrawal Operation Experiences

Michael Koryak*

ABSTRACT

The Pittsburgh Engineer District is currently operating four structures with selective withdrawal intakes. One of the most important lessons that the District has learned in its more than three decades of experience with these structures is that operating objectives can change and operational flexibility is highly desirable.

The District has utilized selective withdrawal to maintain both cold and warm water outflow fisheries and other outflow temperature objectives; for the conservation of warm, cold, and very cold water strata within a reservoir to maintain a "three story" lake fishery; to control outflow water quality; for the control of reservoir stratification patterns to promote in-pool mixing and dilution of acid mine drainage pollution; and to control reservoir primary biological productivity.

Existing and potential problems in the operation of these intakes are related to vertical placement of the gates and insufficient withdrawal options; pump-back currents and stratification disruption from pumped-storage hydropower generation; conventional hydropower conversion; maintenance shutdowns; and periodic summer flood drawdowns where the required discharge exceeds the capacity of the selective withdrawal system.

INTRODUCTION

Four of the 15 reservoirs operated by the Pittsburgh Engineer District are equipped with selective withdrawal intakes. These projects are Kinzua, East Branch and Woodcock Creek Dams in northwest Pennsylvania, and Michael J. Kirwan Dam in northeast Ohio. A fifth structure with a highly innovative intake design, Stonewall Jackson Dam in northern West Virginia, is now under construction and will be discussed in a separate presentation. A brief overview of operating experiences will be presented for each individual project.

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KINZUA DAM

Kinzua Dam, completed in 1966, is a multi-purpose project, which includes pumped-storage hydropower (380 MW capacity) located on the Allegheny River in northern Pennsylvania. The project consists of an upper and lower reservoir. The lower reservoir, Allegheny Reservoir, has a surface area of 12,080 acres, a maximum depth of 128 feet and stores 572,000 acre-feet at its normal summer pool elevation of 1328 feet NGVD. The upper reservoir is much smaller (7,100 acre-feet capacity) and is located on top of a ridge 800 feet above and adjacent to Allegheny Reservoir.

Prior to the construction of Kinzua Dam, the Allegheny River supported an important smallmouth bass fishery. In order to maintain historical downstream water temperatures and perpetuate recreation, water quality, and the warm water fishery, Kinzua Dam was designed with two gated upper sluices to release warm water from the surface strata of the reservoir during the summer season. These two upper sluices (each 5.7'x10') are both located at an invert elevation of 1300 feet NGVD and have a combined discharge capacity of 3600 cfs at the normal summer pool elevation of 1328 feet NGVD. Winter releases and flood releases in excess of 3600 cfs involve utilization of six lower sluices (each 5.7'x10' at invert elevation 1205 feet with a combined discharge capacity of 25,000 cfs). Generation and pump-back flows of up to 7000 cfs pass to and from Allegheny Reservoir through a dual well inlet-outlet tower adjacent to the dam. Selective withdrawal is provided for each well through two gates, a 21'x41.5' gate at invert elevation 1289.5 feet and a 21'x31' gate at invert elevation 1226 feet. These gates were designed for use with water temperature control bulkheads. However, because of vibration and cavitation problems, they have been operated either fully open or closed.

Since construction of the project, cooler than anticipated water temperatures have occurred in the tailrace and the smallmouth bass fishery has declined. Prior to impoundment, the highest mean monthly water temperature of the river was about 22°C and this maximum occurred in July. At present, a mean monthly maximum of about 20°C now occurs in August. The cooling influence of the dam is significant for a distance of seven miles downstream of the project and is negligible beyond a distance of roughly 17 miles.

The very popular bass fishery, however, declined over more than a 100 mile long reach of the river and there are some unanswered questions about the exact role of water temperature in this problem. A mixed cold and cool water fishery (primarily rainbow and brown trout and walleye and muskellunge) with some bass and other warm water fish species developed naturally in the cooled seven mile long tailrace area. Farther downstream, walleye and muskellunge populations increased. In spite of the presence of quality cool water fisheries, however, there was an extreme adverse public reaction to the reduced bass fishery and considerable pressure was exerted to restore the warm water temperature regime.

The metalimnetic layer of the lower reservoir develops roughly between the invert elevations of the upper sluices and the upper power intake/discharge gates. Cooler than desired water is withdrawn by both, but because of their lower invert elevations and higher discharge rates, the upper power intakes generally withdraw a higher volume of significantly cooler water than the upper sluices. During the hydropower pumpback cycle, a strong current of cool water flows near the surface of the lake for a distance of about four miles. In addition, shearing from this current entrains cool hypolimnetic waters and mixes it up into the epilimnion.

Initially, the cool pump-back current and metalimnetic shearing were considered to be the principal reasons for the cooler than desired outflow water temperatures. However, a Waterways Experiment Station hybrid (mathematical and physical) model of the system which was completed in 1980 suggested that this was not the case. Counterintuitively, the model demonstrated that over the long run, the hydropower withdrawal and pump-back current actually cause the hypolimnion to warm without significant cooling of the epilimnion. The recommended solution to the problem was to attach risers to two of the lower sluices and to operate these throughout the period of summer stratification using the existing upper level sluices only when the discharge capacity of the riser modified sluices is exceeded.

The results of the model study were presented to the responsible fishery management agencies and to local sportsman groups. Because of a concern that warmer releases might jeopardize very fine cold and cool water sport fisheries that had developed below the dam without restoring the bass fishery, they recommended that the proposed structural modification not be made to Kinzua Dam.

EAST BRANCH DAM

East Branch Dam, completed in 1952, is a multi-purpose project located on the East Branch of the Clarion River in northwest Pennsylvania. The lake has a surface area of 1,160 acres, a maximum depth of 147 feet and stores 64,300 acre-feet at its normal summer pool elevation of 1,670 feet NGVD. Water is released from the lake through a control tower with intake gates located at four elevations (invert elevations 1641, 1620, 1552, and 1531 feet NGVD). Over the past three decades water quality and operational objectives at the project have changed a number of times and it has been necessary to utilize all of these available selective withdrawal options.

Extensive bituminous coal mining in the tributary basin of East Branch Lake was initiated nearly simultaneously with construction of the project. By the time the lake was filled, acid mine drainage had degraded the water quality of the lake to the point that it could not support fish and the lake became known locally as the "Dead Sea of Elk County." Because of the grossly acid degraded water quality and the absence of any reservoir or outflow fishery, fishery management was not considered in the operation of the dam. From 1952 until 1957, all water was withdrawn from the deepest, coldest strata of the lake (the invert elevation 1552 feet NGVD elevation gate) to provide cool, well aerated dilution waters to a downstream reach of the Clarion River that was septic and foul from a heavy load of paper mill wastes.

A series of pollution abatement measures at the paper mill allowed increasing operational flexibility at the dam. From 1958 until 1974, water was withdrawn from the warm surface strata of the lake (the invert elevation 1641 feet NGVD gate) during the summer to maintain warm water for a swimming beach in the river that had been developed at a downstream state park.

In 1974, this swimming area was replaced by a pool and the surface withdrawal operation was reconsidered. By that time, the state had completed extensive reclamation of the abandoned coal mines in the tributary drainage and mean pH values in the lake and outflow had climbed to levels that should have been able to support aquatic life. However, because of periodic and short-term but often extreme acid mine drainage events during the summer, a fishery still failed to thrive at the project.

Surface withdrawal was setting up a very high and well defined metalimnion. The periodic summer acid slugs rode high, above the elevated metalimnion and in the very biologically sensitive upper strata of the reservoir. Also they were passed relatively rapidly to the river downstream with little mixing or dilution. These pH extremes had a devastating effect on the aquatic life of both the lake and the outflow.

In 1975, the District tried to mitigate the impact of these acid slug events by withdrawing from a mid-level gate (invert elevation 1620 feet NGVD). This new operation warmed the epilimnion and increased the volume and depth of both the epilimnion and the metalimnion. The acid inflows were then drawn through the lake at a greater depth and considerably more mixing and dilution occurred in the expanded metalimnion. As a consequence, primary biological productivity and fish survival and growth rates all increased. Today the project supports a tailrace brook trout fishery and a three story lake sport fishery. During the summer there are bass, muskellunge, and pan fishes in the epilimnion, walleye and brown trout in the metalimnion, and lake trout and rainbow smelt in the very cold hypolimnion where maximum summer water temperatures range from 40-43°F.

A recent controversy at the project involves FERC licensed retrofit hydropower development with low elevation withdrawal. This low elevation withdrawal would evacuate the very cold hypolimnetic lake waters necessary to maintain the existing successful and popular lake trout fishery.

MICHAEL J. KIRWAN DAM

Michael J. Kirwan Dam is a multi-purpose project located in northeast Ohio. The project has been fully operational since 1966. The lake has a surface area of 2,650 acres, a maximum depth of 58 feet, and stores 56,700 acre-feet of water at its normal summer pool elevation of 985.5 feet NGVD. Water is withdrawn through a tower at three levels, invert elevations 972, 956, and 936 feet NGVD. Each of the three gates releases into a separate well and conduit barrel.

Warm water temperature release and water quality outflow objectives are achieved during the period of summer stratification by use of the invert elevation 956 feet NGVD gate. The metalimnion sets up near the invert of this gate and hypolimnetic iron and manganese concentrations both typically reach maximums of about 3 mg/l by late summer. The mid-level discharge operation results in somewhat elevated manganese concentrations in the outflow throughout the summer season. However, violations of the state standard of 1 mg/l manganese have never been documented during normal operations.

The most serious problem that has been experienced with this project occurred in 1978. Between 1 August and 16 November 1978, it was necessary to discharge from the invert elevation 936 feet NGVD gate because the invert elevation 956 feet NGVD gate was stuck and required bracket and stem maintenance. Attempts to utilize the invert elevation 972 foot gate were made, but had to be discontinued because of severe vibration problems. Since thermal stratification did not breakup in the reservoir until mid-October of 1978, it was unfortunately necessary to discharge much cooler than desired hypolimnetic waters with high iron and manganese concentrations for a period of two and one-half months.

WOODCOCK CREEK DAM

Woodcock Creek Lake is a small, eutrophic multi-purpose impoundment in northwest Pennsylvania. The project was completed in 1974. The lake has a surface area of 333 acres, a maximum depth of 44 feet and stores 4,930 acre-feet of water at its normal summer pool elevation of 1181 feet NGVD. Water is withdrawn through a dual well control tower at four levels, invert elevations 1167, 1157.5, 1139, and 1138 feet NGVD.

Very substantial quantities of iron and manganese (up to 20 mg/l and 15 mg/l, respectively), plus ammonia, hydrogen sulfide, and other potentially noxious compounds accumulate in the anoxic hypolimnion during the summer season and selective withdrawal is necessary to maintain acceptable downstream water quality. Modeling techniques were utilized in the design of the intake configuration and water quality and downstream warm water temperature objectives are achieved with summer releases from the invert elevation 1167 feet NGVD gate. Since the discharge capacity of the upper gate is relatively high, it is rarely necessary to augment its releases from the lower elevation gates during summer flood drawdown periods.

However, even with the very satisfactory predictive modeling, design and operational history of this structure for water quality and warm water release objectives, Woodcock Creek Dam can still serve as another example of the potential advantages of operationally flexible selective withdrawal intakes. This is demonstrated by the fact that after the dam became operational, local pressures were exerted to establish a spring and early summer put-and-take trout fishery in the tailrace. The mid-level intake could and most likely will eventually be utilized to improve and seasonally prolong this cold water tailrace fishery.

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SELECTIVE WITHDRAWAL FROM ANY LEVEL BETWEEN MINIMUM POOL AND SPILLWAY ELEVATION AT STONEWALL JACKSON DAM, WEST VIRGINIA

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I. Introduction

The benefits of selective withdrawal have been recognized for many years in the Pittsburgh District. The first of our 15 flood control dams (Tygart) was constructed nearly 50 years ago. At that time, incidental or indirect provision for selective withdrawal was provided in supplying municipal water from higher reservoir strata than was accessible by the conventional flood-control sluices. Our first direct capability (East Branch Dam) for selective withdrawal to the downstream channel was placed into operation 33 years ago. This, and subsequent projects having these capabilities, utilized either multi-level fixed ports or bi-level sluices to access different elevations in the impoundments.

The District has found that the conventional fixed withdrawal levels can sometimes impose undesirable limitations on outflow water quality. This was indicated to be the case for the Stonewall Jackson Dam which is presently under construction on the West Fork River, 202 stream miles south of and upstream from Pittsburgh, and located within the Monongahela River basin.

Stonewall Jackson is a multipurpose project having a drainage area of 102 square miles. Storage at spillway crest will be 75,000 acre-feet with maximum pool depth of 75 feet (ft). It is a 95-ft high concrete-gravity dam with uncontrolled spillway capacity of 28,000 cubic feet per second (cfs). Three 3.5-ft by 7-ft low-level flood-control sluices are provided in addition to the two 2.5-ft by 4-ft water-quality sluices.

The dam is constructed of mass concrete 620 feet in length. It is founded on rock with the lowest foundation elevation at 984.0. The top of the dam is located at elevation 1102.0 giving a maximum height above the foundation of 118 ft. The spillway is an ogee-type 117 ft long crossed by a 4-span concrete box beam bridge with a 15-ft 6-inch wide roadway. The right and left abutments are 184 and 319 ft in length, respectively. Both the flood-control sluices and the water-quality control sluices are controlled by tandem service and emergency slide gates operated by hydraulic cylinders which are controlled from the pylon building as well as from near the cylinders themselves.

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II. Hydraulic Aspects

A fixed-port intake system was considered initially to meet outflow requirements. This consisted of two wells on the upstream face of the dam, each with four fixed-level intakes. The port elevations for this scheme were positioned using a computerized thermal simulation of the lake.

Because of summer stratification conflicts between a rigid downstream water-temperature schedule and outflow water-quality objectives and restrictions, a more flexible withdrawal design was desired. Of primary concern was the problem of blending cool hypolimnetic waters into the discharge to support a downstream trout fishery during the summer when the deeper, colder strata of the lake are expected to have unacceptably high iron and manganese concentrations. An additional area of concern was a potential problem of turbid temperature-density currents penetrating the lake near the elevation of a fixed intake. With a fixed-port intake works, circumstances could develop where outflow-temperature goals would have to be sacrificed in the interest of water quality. Therefore, alternative withdrawal schemes were investigated for more versatility.

The adopted design is an innovative arrangement, consisting of two towers, one on each side of the spillway, projecting from the upstream face of the dam. Each tower has four movable-gate leaves to allow for withdrawal of waters from any level between spillway elevation 1082 and minimum pool elevation 1038. Each tower, in plan, will contain a 10-ft by 15-ft wide vertical shaft. The maximum discharge from each well will be 415 cfs. Flow from the reservoir into the tower will be controlled by three 15-ft wide regulating gate leaves and one hoist gate. These gates will be positioned to allow either weir flow over a gate leaf, or flow through a submerged opening between gate leaves. It will also be possible to withdraw water simultaneously from more than one level into the same tower. Debris will be prevented from entering by a trashrack in slots on the face of the tower ahead of the gates to protect the entire vertical opening. Although the regulating gate leaves and hoist gate have been designed to withstand full hydrostatic pressure, operation procedures will limit the head differential between reservoir pool and water-quality control tower pool to 1.5 ft. This is to assure that a free jet entering the shaft will not impinge against the concrete of the dam. The restraint on head differential will also prevent the average velocity through a submerged opening from exceeding about 6.5 ft per second. This is expected to minimize any tendency for vibration.

There will be instances when it is desirable to have flood-control sluice gates opened simultaneously with the water-quality sluice gates. The most obvious occasion would be during release of excess runoff from summer storms. This situation will call for both water-quality gates to be open full, with any additional required discharge coming from the flood-control sluices. No problems are anticipated as the water-quality outlets are downstream of and at a higher level than the flood-control outlets. Any intersection of the jets will be far removed from the outlets. Experimentation will determine the combinations of openings to produce the best performance.

The selective withdrawal system is designed to meet the release-temperature requirements of the West Virginia Department of Natural Resources. The proposed temperatures will enhance the Agency's fishery-management program for Stonewall Jackson tailwater. The desired objective is to raise the downstream water temperature to a peak of 70°F by the beginning of May and maintain that temperature until mid-October.

The movable gate-leaf system is more flexible than the system of fixed ports that was investigated because it will provide withdrawal from any level between 1038 and 1082. Simulation runs, using the Waterways Experiment Station's Selective Withdrawal Program, confirm the movable gate system's capabilities. The adopted design, for structural reasons, will not allow withdrawal below sill elevation 1038. It was judged that the absence of facilities for withdrawing between 1034, where one port was located in the fixed-port study, and 1038 is not significant. Colder water can be withdrawn through the flood-control sluices, if necessary.

The fixed-port simulation assumed a 40-ft long weir at elevation 1069.7. The proposed design will permit flow over a 15-ft bulkhead on each tower, for a total weir length of 30 ft. A shorter weir means that in some cases, it will have to be set lower than 1069.7 to pass the required flow, the consequences of which would be a 1°F cooler outflow during some spring releases.

The system will also be capable of removing water simultaneously from more than one level into the same tower. There is evidence that density differences in various levels of stratified impoundments can influence the quantities of water that would otherwise be expected to be withdrawn for multiple inlets. This phenomenon could conceivably result in warmer upper-level water being blocked from entering the shaft by the colder water entering through a lower intake. There is also a chance that an unsteady flow condition might develop in which the proportions of total discharge withdrawn from two levels changes with time. If thermal mixing cannot be accomplished satisfactorily inside the towers, there are two alternatives. First, a single precisely positioned intake might yield a steady discharge at the desired temperature. Secondly, each tower could withdraw from single but different levels with the mixing taking place in the stilling basin. Outflows from the two towers are directed toward each other into the stilling basin, a feature which would facilitate the blending of waters.

While enhancing operational water-quality flexibilities, an additional benefit of this innovative design will allow the withdrawal of desired temperature through only one tower without the need for basin blending. This feature is valuable in that it will afford full utilization of the station hydropower plant. The ability to meet the temperature schedule from either tower would also be advantageous during periods when one is out of service for inspection or repair or when the stilling basin is dewatered and its bypass is in operation. During high outflows, eddying can be minimized and symmetrical discharge can be provided with equal flow through the two water-quality sluices. This will be possible since the flows can be balanced from each tower and will not be governed by irregular flow requirements of temperature blending needed with a fixed-port system.

III. Mechanical Operation

Flow into the towers is regulated by four vertically stacked gate leaves 17 ft wide located in a single slot. Three regulating gate leaves are 12 ft in height while the bottom hoist gate is 12 ft 6 inches in height. The three regulating gate leaves normally rest on the bottom hoist gate which in turn is suspended from a twin stem interconnected electrically operated, floor stand hoist with a maximum lift of 18 ft. A pair of pivoting dogging devices, one on each side of the three upper regulating gate leaves, are operated by small low-pressure hydraulic cylinders powered by package-type hydraulic units located at the top of the towers. The systems are interconnected to permit operation if one should fail. These dogs engage lugs located at two-foot intervals along the sides of the regulating gate leaves and permit each higher leaf to be supported at any desired elevation. The lower regulating gate leaves can then be lowered to produce any opening. If desired, more than one leaf can be dogged off providing more than one opening. The dogs must be engaged when the selected lug on the gate leaf is about one foot above the desired gate level to avoid interference from the lower lugs. When the dogging devices are in the down position, the gate leaves are lowered until the lugs are sitting on top of the dogging devices. Hoisting speed of the gates is four feet per minute. When raising the gate leaves, the dogging devices are released when the gates have been raised about one foot. If inadvertently the gates are raised without retracting the dogging devices, the backside of the lugs would force the dogging devices out of the slot as hydraulic pressures in the cylinder are released by a relief valve. The dogging devices for the three upper gate leaves are located at elevations 1075.0, 1063.0, and 1051.0. The hydraulic cylinders for operating the dogs are located on the top deck of the water-quality control tower and operate the dogs by means of connecting operating stems. The dogs are made to be operated manually by removing the hydraulic cylinders and installing a mechanical lever system. Gate leaf elevation and position indicators are attached to each gate leaf to show their elevations during all periods of operation. Both the gate hoists and the hydraulic dogging cylinders are operated by remote control from the pylon building. Digital readouts in the pylon building and dial indicators at the water-quality control tower show the position of the gates so that the desired opening can be made in the proper location. Provisions are also made to operate the gates and dogs from the towers.

IV. Structural Aspects

Each water-quality control tower is constructed of two reinforced concrete walls projecting off the upstream face of the dam, from sluice invert elevation 1018.0 to platform elevation 1088.0. A reinforced concrete sill wall spans the 15 ft between the tower walls from elevation 1018.0 to 1038.0. A working deck is provided at elevation 1088.0. To assure smooth flow into the wet well, the regulating gates are located 10 feet from the dam face. Doors are provided on the outside face of the walls for access to the dogging devices. These doors are watertight and along with the tower and sill walls are designed for full hydrostatic pressure with pool at elevation 1082 and the tower wet well drained. The inlet for the 2.5 foot by 4.0 foot water-quality control sluice is located at the bottom of the well at invert elevation 1018.0. Entrance curves are provided for the inlet at the sides and top.

The three water-quality regulating gates consist of a skin plate welded to the upstream side of a framework of horizontal beams and vertical end plates and weigh 12,500 pounds each. Each of the three regulating gates contains dogging lugs and is 17'2-1/2" wide x 12'0" high with a J-type rubber seal attached along the bottom and sides of each gate. The hoist gate, lower most gate, is 17'2-1/2" wide x 12'6" high and also has rubber seals along the sides and bottom to reduce leakage. The gate weighs 13,000 pounds. The lugs attached to the end plates of the three regulating gates transfer vertical loads (gate weight) to the dogging devices, when engaged. The hoist gate supports the three regulating gates above it and the hoist assembly is capable of lifting and lowering the total weight of all four gates. The design for both sizes of gates is similar, using static pressure with pool at elevation 1082, and the tower wet well drained. Guides are incorporated in the end plates. Cathodic protection is provided on the gates.

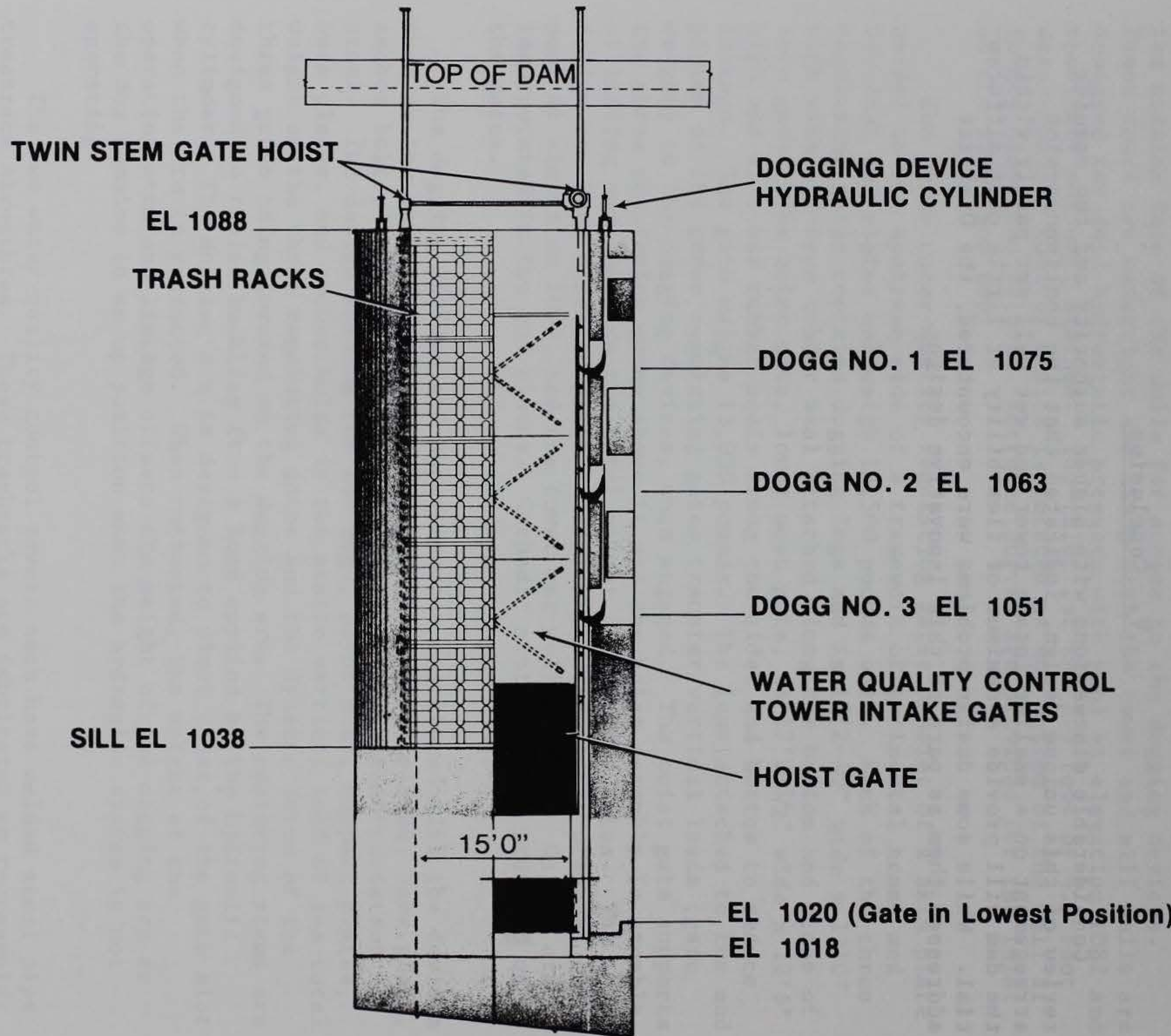
The dogging devices and appurtenances associated with the devices such as the operating stem, linkage, pins, support plates, base plate, anchor bolts, and access plate and frame, are corrosion-resistant steel. The design loading for the dogs, pivot pins, support plates, base plates, and anchor bolts is the static vertical load of the total weight of the three regulating gates and the dynamic force of the three gates being lowered on the dogging arm. The operating stems are designed to resist buckling from a load applied by the hydraulic cylinder. The dogging arm is designed to pivot clear of the gate slot when the arm is retracted. When retracted, the weight of the operating stem and linkage offsets the weight of the dogging arm so the dog remains in an up position when the hydraulic system is not operating.

The two water-quality control towers each have welded steel pipe trashrack structures. These trashracks are fabricated as rectangular panels 17'-1" wide x 9'-10" high. The eight-inch diameter steel pipes are spaced 30 inches on centers vertically and 24 inches on centers horizontally. The panels sit in slots on each side of the tower opening so that the panels can be removed. The slot can be used for a bulkhead or stoplogs to seal off flow and dewater the wet well for inspection and repairs. In the event of an emergency, the regulating gates from the operable (other) tower could be removed and used as bulkheads in the tower.

A maintenance bulkhead placed under no-flow conditions will close the water-quality control sluice during maintenance conditions. The bulkhead is designed for static pressures with reservoir pool at elevation 1098.2.

V. Conclusion

Considerable discussions with higher authority and the ensuing review of this unique design, indicated that the continuous-slot arrangement on a smaller intake tower and wet well set partly within the dam will provide a maximum of flexibility at little cost differential. While some design problems were encountered, the District addressed them as part of this innovative design.



WATER QUALITY CONTROL TOWER

SELECTIVE EXPERIENCES: ALIAS WITHDRAWAL PAINS

by

RICHARD E. PUNNETT, Ph D[1]

ABSTRACT: Eleven Corps of Engineer lakes, having various designs of outlet works, were evaluated for capability to meet the release objectives of the project purposes. Only two of the outlet works were designed with the aid of numerical lake models. The data collection and management, associated with the release regulation, was also discussed.

INTRODUCTION

Three of the projects were located in the central part of Ohio, five were located throughout West Virginia, and three were located in eastern Kentucky. All projects were within the Huntington District. For the evaluation, the release characteristics for the years 1982, 1983, and 1984 were used. Pertinent data for each project were provided in Table 1. For each project, a temperature objective curve was determined based on natural stream temperatures. Releases were managed so as to stay, when possible, within a temperature deviation of two degrees Fahrenheit either above or below the objective curve.

DATA COLLECTION AND MANAGEMENT

During the summer stratification period, lake profiles were determined at each project at the beginning of the week. At a minimum profiles included temperature and dissolved oxygen (DO); other parameters such as conductivity, pH, and turbidity were taken if the project

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1. Supervisory Hydraulic Engineer, Huntington District, Corps of Engineers.

had special concerns. Although the equipment was maintained by the Water Quality Section, the profiles were taken by project personnel. The profile data were phoned in directly to a Harris computer. The profiles were then plotted and used as a reference for determining withdrawal levels.

The outflow temperature, during 1982 and 1983, was determined at each morning at each project and sent in as part of the daily report required by the Reservoir Control Section. During 1983 and 1984, the outflow temperatures were collected hourly by data platforms and transmitted, via satellite, to computer files.

Chemistry samples were taken periodically at selected projects by field crews. The frequency of sampling, as well as the kinds of samples collected, were tailored to data needs of the projects.

PROJECT PECULIARITIES

Alum Creek, Deer Creek, and Paint Creek Lakes

These three projects are located in the plains of central Ohio and have similar basin characteristics. Typically, the releases from these lakes are below the temperature objective curves established from natural stream conditions. Paint Creek has the shallowest intake port and was the only lake that required some blending to meet objectives. The lower intake of Paint Creek has been used successfully to blend cooler water in a single wet well when hypolimnetic quality permitted. The method to determine gate openings was largely trial and error. The upper gate was fully opened while the lower gate was only opened from 10 to 20 percent.

Beech Fork and Burnsville Lakes

Both projects have dual wet well outlet works and are regulated for warm water releases. Formation of the thermocline in both lakes was typically shallow (5 to 10ft) and the uppermost gates were below the epilimnion. Both lakes were regulated successfully although the release temperature became highly variable when the intakes were located in the thermocline. The intake levels for

Burnsville Lake were determined with the aid of numerical modeling. The temperature objective curves were followed except during periods when hypolimnetic releases were needed for blending; during those periods, the temperature objective was abandoned so that poor quality water would not be released.

East Lynn, Grayson and Fishtrap Lakes

These lakes have single wet well outlet works and have had good success in meeting temperature objectives. Blending at Fishtrap was routine; blending at East Lynn has been successful but often obviated by the quality (high iron concentrations) of the hypolimnion. Typically, all three low flow gates are fully open at Fishtrap; closing of the upper gate was required during the warmest periods. Grayson Lake typically meets temperature objectives using a single intake; hypolimnetic releases are avoided when possible.

R. D. Bailey Lake

The outlet works have five intake locations; unfortunately, the upper gate was too deep to provide the warm water necessary to meet release objectives. The temperature objective curve for this project was warmer than all other District projects. Most likely, the curve used for any other project could be followed; maintenance of a cold water fishery may be possible.

Sutton Lake

Although the original project design did not include selective withdrawal, a "riser" was retro-fitted to one of the sluice gates which allowed for near-surface withdrawals. The release of turbid bottom water during non-flood periods was detrimental to the downstream fishery and resulted in the need for the riser. Because of design requirements, the riser intake was below the thermocline; this results in releases that are cooler than the normal objective curve but warmer than the historical releases. Preliminary studies indicate an improvement of the benthic community in the tailwaters since operation of the riser began.

Samples collected, by WV State agencies, below Sutton Dam have been evaluated for the presence of tri-halomethane (THM) precursors. Samples evaluated prior to the operation of the retro-fitted riser indicated a potential for THM formation; samples evaluated after riser operation did not have the potential for THM formation.

Paintsville Lake

This was the newest project in the District and has only been regulated for one year. The design includes dual wet well outlet works having intake levels determined by numerical modeling. A put-and-take trout fishery was managed below the project. The design provided the flexibility needed to meet the release objectives; although hypolimnetic quality did deteriorate, it was below the intakes ports needed for blending.

OBSERVATIONS

1. The location of the thermocline, relative to the intake gates, seems to be the most critical factor in meeting downstream temperature objectives. At least one intake gate should be located above the thermocline. Since accurate prediction of the thermocline is difficult, a safe design would have at least one gate within five to ten feet below the summer pool elevation. If all the gates are too low (below the thermocline), the temperature objective is seldom achieved and other water quality problems may occur due to hypolimnetic releases.

2. The most stable release, in terms of temperature objectives, occurred when two gates were used to blend and the thermocline was between the intake locations. This was true even when blending in a single wet well.

3. The most variable release, in terms of temperature objectives, occurred when a single gate was used which was located in the thermocline region.

4. Abandonment of the temperature objective curve occurred annually during and sometime after the fall turnover period. The fall isothermal temperature was often warmer than natural stream temperatures.

5. The "exact" temperature of the release water was somewhat nebulous; the recorded temperature had many inherent variables. The location of the temperature probe

(surface to bottom, side to side, or downstream distance), and the orientation of the sun were some of the sources of temperature variations.

6. Large diurnal temperature changes (up to 5 degrees Celsius) were noted. The fluctuation was a function of several variables such as flowrate, distance downstream, weather conditions, orientation of the sun relative to the release channel, and the water depth. When considering the fluctuations, what time of the day should be used to match the temperature objective? Perhaps the data used to derive the objective curve should be considered.

7. In projects where hypolimnetic deterioration occurred due to anoxia, the temperature curve was sometimes abandoned to avoid blending with putrid water. In the cases where high iron, manganese, sulfide, and/or ammonia nitrate concentrations existed, hypolimnetic releases would have been far more detrimental to downstream biota than releasing water warmer than the temperature objective.

8. At any given project, the temperature objective criteria were abandoned during periods of releases that exceeded the capacity of the low flow outlet works. Most of these releases occurred in the early summer stratification period and the temperature deviations were not severe. Late summer "flood" releases not only cause temperature deviations but can also cause water quality problems due to hypolimnetic releases.

9. The value of dual wet well outlet works is diminished at projects where hypolimnetic deterioration occurs. As indicated in the seventh observation, a temperature objective alone may not be a sufficient guideline to maintaining a viable downstream habitat.

SUMMARY

The temperature regulation program within the Huntington District was discussed. Some generalized observations for design guidance was given. Good engineering design begins with good design criteria; the price of good engineering is paid for, reguardless of whether or not it was received. The release temperature objective, while important, may not be a sufficient design criterion. The true objective should be to maintain a productive as well as viable downstream habitat.

TABLE 1

PERTINENT PROJECT DATA

Lake Name	State	Year Dam Completed	Lake Depth[3] (Ft)	Intake Depth(s)[3] (Ft)	Thermocline Depths (Ft)[3]			Temperature Difference (°C)		
					'82	'83	'84	'82	'83	'84
Alum Creek	OH	'74	68'	16,53	20	15-20	20-25	10-15	10	5-10
Beech Fork[2]	WV	'78	35'	10,10,19,28	5-10	5-10	5-10	15	15	10-15
Burnsville[1,2]	WV	'76	38'	11,11,21	5-10	10	10	10-15	10-15	10
Deer Creek	OH	'68	40'	24	10-20	20	15-12	10	10	5-10
East Lynn	WV	'71	50'	4,13	10	10-15	10-15	10-15	10-15	15
Fishtrap	KY	'69	84'	10,26,42	10-15	10-15	10-20	10	10	10
Grayson	KY	'68	60'	4,14	5-10	5-10	10	15	15	15
Paint Creek	OH	'73	50'	9,28	10-15	5-10	10-15	5-10	5-10	5-10
Paintsville[1,2]	KY	'83	102'	10,26,46,73,97	--	--	10	--	--	15
R.D. Bailey	WV		145'	15,35,49,69,125	10-15	20	10-15	5-10	10	10
Sutton	WV		115'	12	5-10	5-10	5-10	15	15	15

[1]Numerical lake modeling conducted during design phase.
[2]Dual wet well outlet works.
[3]Relative to summer pool.

DESIGN AND PERFORMANCE OF SKIMMING WEIRS IN THE KANSAS CITY DISTRICT

By Walter M. Linder, Chief, Hydrologic Engineering Branch,
U.S. Army Corps of Engineers, Kansas City District

INTRODUCTION

Skimming weirs are used by the Kansas City District as one method of achieving selective withdrawal from a stratified body of water. This paper describes the events that led to the construction of a skimming weir at two hydropower projects. Field data show these structures to be very effective in preventing the release of cold deoxygenated water over a significant range of pool elevations and discharge rates.

STOCKTON DAM AND LAKE

Stockton Dam, a multipurpose project located on the Sac River in southwestern Missouri, controls a drainage area of 1,160 square miles. The multipurpose pool volume of 875,000 acre feet (AF) has a surface area of 24,900 acres. The depth from the top of the multipurpose pool to the valley floor is slightly less than 90 feet (ft). The power facility consists of a single Kaplan turbine capable of discharging 11,000 cubic feet per second (cfs) at maximum drawdown of the power pool. However, downstream channel capacity limits the maximum release to 8,000 cfs.

The plant is operated as a peaking power facility with generation dependent upon power demands. Generation periods may vary from a few hours several times a day when the lake level is within the power pool to 24 hours a day during flood control releases. Discharges are normally between 5,000 and 8,000 cfs, with releases of 40 cfs during non-generation periods. Both the low flow and power intakes withdraw water from the bottom of the lake.

Stockton Water Quality Problems

Although preimpoundment water quality surveys were made, the studies concentrated on existing conditions and did not make an analysis of the probability of thermal stratification in the lake. Impoundment of the lake began in December 1969 and stratification was first documented in June of 1970. A significant downstream fish kill due to low water temperature and low dissolved oxygen (DO) concentration occurred during low flow releases in July 1970 prior to the lake reaching multipurpose level. A solution to this problem had to be found since power generation with much larger releases was scheduled to begin in 1973.

Skimming Weir

Among the various alternatives considered, the most cost effective solution was construction of a skimming weir across the approach channel to the powerhouse. Thermal modeling was used to evaluate various weir crest elevations. The optimum elevation was found to be 840 ft, m.s.l., or 27 ft below the multipurpose pool level of 867 ft, m.s.l. Thermal modeling for a range of pool elevations showed release temperature criterion would be met except during the fall when releases would be somewhat warmer than natural stream temperatures. There would also be a short period in the spring when releases would be colder than natural stream temperatures.

A 66-ft high rock filled weir was constructed in the spring of 1973 at a location approximately 1,000 ft upstream of the powerhouse and spillway structure. The 260-ft long, 5-ft wide crest ties to natural ground elevations on both sides of the approach channel.

Skimming Weir Performance

In order to operate with greatest efficiency, the weir crest should be located well above the thermocline in order to minimize withdrawal of hypolimnetic water over the weir. However, at Stockton the weir crest had to be low enough to provide for adequate drawdown of the power pool. With the weir crest at elevation 840 ft, m.s.l., drawdown of the power pool is limited to about 845 ft, m.s.l., instead of 838 ft, m.s.l., the original bottom of the power pool. This somewhat reduces the number of hours of generation available during critical drought years.

The skimming weir at Stockton has been reasonably successful in meeting the adopted release criteria. Figure 1 shows dissolved oxygen and temperature profiles obtained during generation in August 1973 shortly after the weir was completed. The water temperature between the weir and the powerhouse was essentially isothermal at about 25 degrees centigrade (^{o}C). The DO concentration was about 8 milligrams per liter (mg/l) in the upper 15 meters of the water column and then decreased to about 4 mg/l near the bottom.

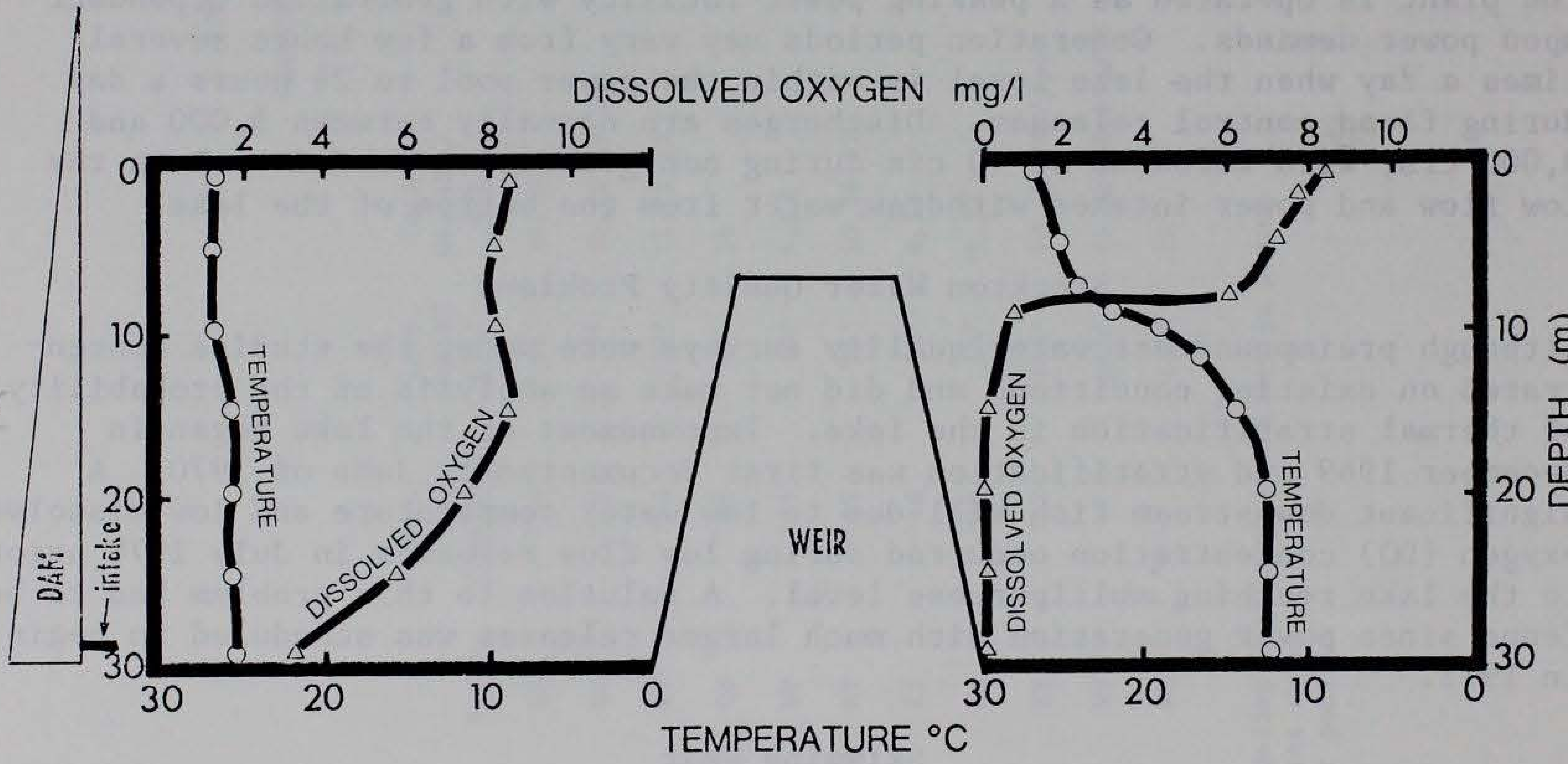


Figure 1 - Temperature and Dissolved Oxygen Profiles During Generation August 27, 1973

Restratification can develop between the weir and the powerhouse and result in brief periods of low downstream DO concentrations at the start of generation. If generation occurs on a frequent basis, there is insufficient time for restratification to develop between the weir and powerhouse. However, if a week or more passes between generation cycles, restratification will occur and results in depressed DO levels at the start of the power generation cycle. There are also occasions when the thermocline is slightly above the weir crest and some hypolimnetic water is drawn over the weir during generation. This also results in lowered downstream DO levels. Due to mixing, the downstream DO concentrations generally range from 5 to 6 mg/l. In any event, the lowest downstream DO levels that have been observed have been in the 4.0 to 4.5 mg/l range, indicating the weir is preventing the release of water with extremely low DO levels which would occur without the weir.

HARRY S. TRUMAN DAM AND RESERVOIR

Harry S. Truman Dam, a multipurpose project located in west-central Missouri on the Osage River, includes facilities for pump storage and power generation. The Lake of the Ozarks, which is created by Bagnell Dam and forms the tailwater of Truman Dam, is an extremely popular midwestern recreation area.

The project controls 11,500 square miles of drainage area, of which approximately 1,600 square miles are also controlled by five upstream reservoirs. The storage capacity at the multipurpose pool elevation of 706 ft, m.s.l., is 1,040,000 AF with a surface area of 55,600 acres. Depth to the average valley floor elevation, 660 ft, m.s.l., at the dam, is 46 ft. The depth to the invert of the power intakes is 103 ft. The power plant at Truman Dam consists of six reversible slant type pump turbines. Discharge from power generation can be as high as 65,000 c.f.s. and 27,500 c.f.s. can be discharged during pumping operations. Only a 2 ft increment of storage, elevation 706 to 704 ft, m.s.l., was provided specifically for power purposes since the intent was to rely on pumpback for maintaining the power pool. Severe problems with fish kills during pumpback testing have resulted in abandonment of pumpback until technology for adequate fish protection becomes available.

Harry S. Truman Water Quality Problems

Thermal simulations of the Harry S. Truman Reservoir were conducted early in 1972 to predict the degree of thermal stratification in the reservoir and the effect of power releases and pumpback on downstream water temperatures. It was assumed there would be no channel excavated between the river and the power plant intake and the natural overbank would function as a broad crested weir. These studies resulted in the following conclusions:

a. The reservoir would be essentially isothermal above elevation 660 ft, m.s.l., by the first week of August during most years.

b. The reservoir would not have a well-defined thermocline. Instead, there would be a more gradual change of temperature with respect to depth below the water surface.

c. During the spring months, releases would be cooler than natural stream temperatures. During most of the summer, release temperatures would be nearly the same as natural conditions and, during late summer and early fall, release temperatures would be warmer than natural stream temperatures.

d. Due to the shallow temperature profile above elevation 660 ft, m.s.l., and the hydraulics of flow over the natural weir, most of the outflow from the reservoir would come from above that elevation.

Several higher weir crest elevations were investigated, but it was found there would be no significant improvement in downstream water quality.

Temperature and DO profiles obtained at a location about 1 mile above the dam in 1978 and 1979, prior to filling the multipurpose pool, showed severely depleted DO levels well above elevation 660 ft, m.s.l. In July 1979, when the lake level was about 689 ft, m.s.l., the temperature gradient was nearly uniform from 28°C at the surface to 17°C at the bottom with anoxic conditions below elevation 676 ft, m.s.l.

Skimming Weir

The embankment closure section was located adjacent to the left abutment, while suitable borrow material was located upstream of the dam at the right side of the valley. The contractor constructed a haul road across the valley just upstream of the dam to transport fill material to the closure section. A bridge was used to span the approach to the uncompleted powerhouse and spillway. It was intended that after completion of the embankment, the contractor would remove the bridge and degrade the haul road as much as possible.

When it became apparent some means of selective withdrawal would be required, it was decided to only partially degrade the haul road and place a rock fill across the bridge opening. Evaluation of temperature and DO profile data indicated a weir crest should be about 20 ft below the multipurpose power pool, elevation 686 ft, m.s.l. However, delays in construction and a rapidly rising pool prevented lowering the haul road below elevation 693 ft, m.s.l. Operating experience has shown the higher weir crest elevation provides better control over the release of water with low DO concentrations. Figure 2 shows a plan view of the dam and the upstream haul road converted to a skimming weir.

Figure 2 - Plan View of Harry S. Truman Dam and Upstream Skimming Weir

Skimming Weir Performance

Data show the skimming weir is preventing the release of deoxygenated water during periods of lake stratification. Temperature, DO, and vertical velocity profiles have been collected in the vicinity of the weir and powerhouse intakes under a variety of flow conditions. Unfortunately, on several occasions flow conditions changed during the measurement period. However, even with changing flow conditions, the data show only minimal amounts of water drawn from below the oxycline.

The first unit in the powerhouse was just being placed in operation and was undergoing testing in August 1981. The lake was 1.6 ft above the multipurpose pool and was stratified, with the thermocline located about 20 ft below the surface or about 6 ft below the crest of the skimming weir. The top of the oxycline was very near the weir crest. Contradictory to the thermal modeling which indicated the thermocline would be below elevation 660 ft, m.s.l., it was nearly 30 ft above that elevation.

A DO profile at the weir crest showed the DO concentration varied from 7.1 mg/l at the surface to slightly less than 4 mg/l a few feet above the weir crest, indicating some withdrawal of water from below the oxycline. Midway between the weir and the powerhouse, mixing had occurred and the temperature and DO were nearly constant from top to bottom. DO profiles obtained in the downstream discharge channel earlier in the day showed levels varying from 5.6 mg/l at the surface to 5.3 mg/l at the bottom.

Measurements were made on May 20, 1985, with the intent to obtain data with three power units operating. However, flow conditions changed and the discharge varied from approximately 17,500 c.f.s. with two units generating to 26,700 c.f.s. with three power units in operation. The pool elevation was about 1 ft above the multipurpose pool level, with little in the way of a well developed thermocline. However, oxygen levels were severely depleted below elevation 670 ft, m.s.l.

Figure 3 shows temperature and dissolved oxygen profiles obtained in the vicinity of the weir and the powerhouse with two power units in operation. DO concentrations just upstream of the weir varied from 11 mg/l at the surface to between 1 and 2 mg/l at the bottom. DO concentrations at the weir crest varied from approximately 10 mg/l at the surface to slightly over 5 mg/l just above the weir crest. A short distance upstream of the power plant intakes, the DO varied from 10 mg/l at the surface to 6 mg/l in front of the power plant intakes. The DO level in the outlet channel during the time of the measurements was approximately 7.0 mg/l.

Heavy rains over the Osage basin during the first week of June 1985 resulted in a 13 ft rise in the Truman Reservoir. In order to obtain data with a higher pool level and high releases, temperature and DO profiles were obtained on June 20, 1985. Once again flow conditions varied during the measurements. DO levels were just under 4 mg/l at the elevation of the weir crest a short distance upstream of the weir. Below elevation 670 ft, m.s.l., DO levels were near zero. One would expect withdrawal over the weir of some water with reduced DO levels under these conditions. Figure 4 presents temperature and DO profiles obtained in the vicinity of the weir and between the weir and the powerhouse. The profiles are numbered in the sequence in which they were obtained and the associated number of power units and discharge are noted.

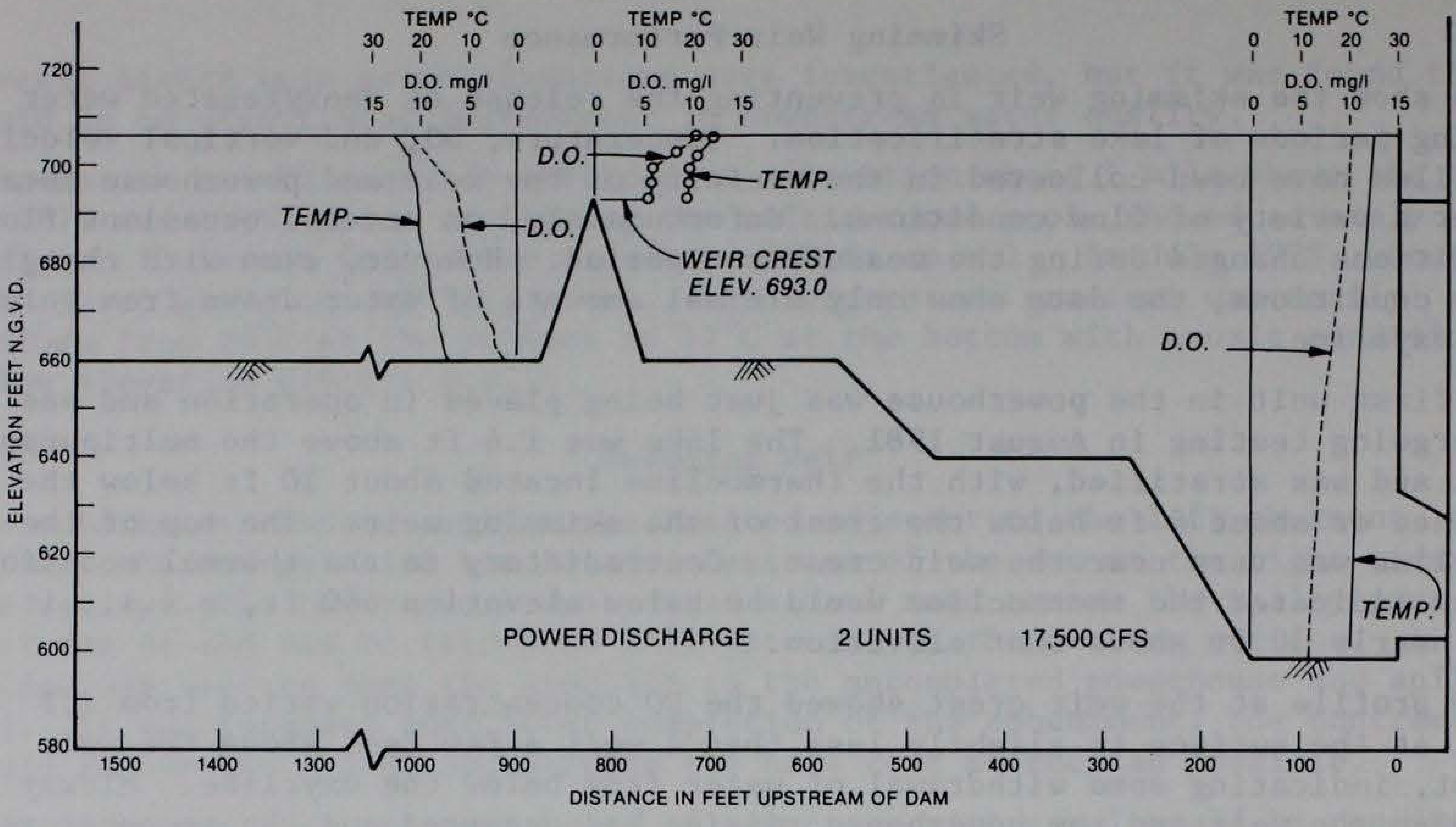
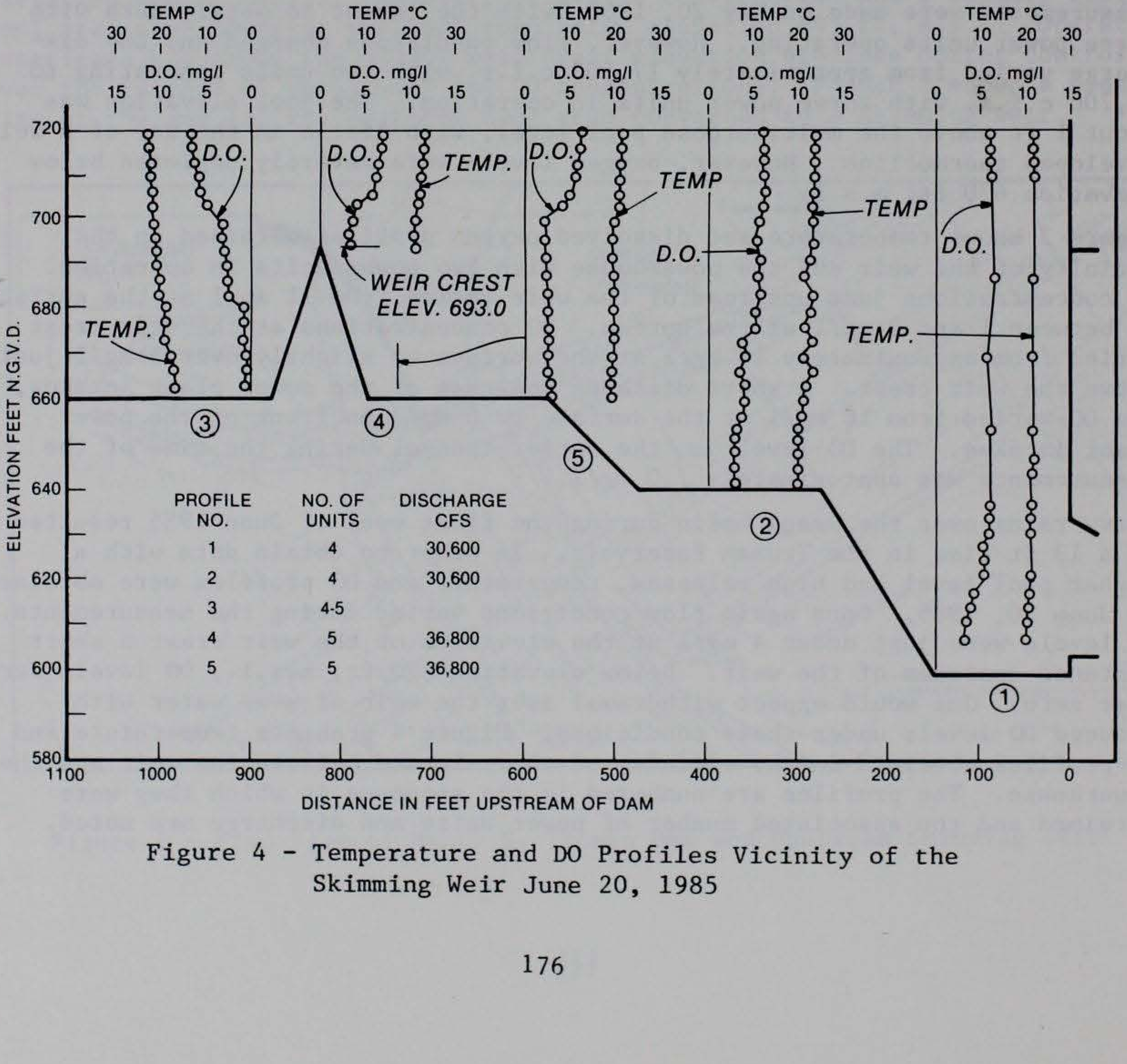


Figure 3 - Temperature and Dissolved Oxygen Profiles With Two Power Units in Operation



PROFILE NO.	NO. OF UNITS	DISCHARGE CFS
1	4	30,600
2	4	30,600
3	4-5	
4	5	36,800
5	5	36,800

Figure 4 - Temperature and DO Profiles Vicinity of the Skimming Weir June 20, 1985

At the upstream toe of the weir, DO levels were near zero below elevation 680 ft, m.s.l., between 2 and 3 mg/l between elevations 680 and 700 ft, m.s.l., and then increased to 6.5 mg/l at the surface. At the weir crest, the lower one-half to one-third of the water column had DO levels between 3 and 4 mg/l. Just downstream of the weir, the highest oxygen levels were near the surface with DO levels slightly below 3 mg/l below elevation 700 ft, m.s.l. Mixing occurred as the flow plunged toward the power intakes, resulting in DO levels of 5 mg/l or greater to near the bottom. Measurements downstream of the powerhouse showed DO levels varied from 5.9 to 5.1 mg/l along the right bank. DO levels were slightly lower and varied from 5.0 mg/l to 4.6 mg/l on the left side of the channel where velocities are lower.

SUMMARY

Operating experience at Stockton and Truman Dams has shown the skimming weirs to be effective in preventing the release of waters containing little or no dissolved oxygen. At times when the oxycline is near or above the crest elevation of the skimming weir, some water with reduced DO levels will be drawn over the weir. However, sufficient mixing occurs between the weir and power intakes to result in acceptable downstream water quality.

Skimming weirs cannot be expected to provide a release of well-oxygenated cold water such as might be desired for a downstream cold water fishery. They are very effective in providing withdrawal of the warmer, well-oxygenated surface waters. Selection of the proper crest elevation requires consideration of several factors. If the crest is placed too high, it can encroach on the ability to utilize the available storage. If placed too low, excessive amounts of water with low DO levels will be withdrawn, particularly if the lake is subject to significant flood inflows during the period of strongest stratification. Selective withdrawal modeling can be useful in selecting a crest elevation, particularly if the model also considers the oxygen balance. Thermal modeling alone may lead to incorrect conclusions as the oxycline can often be located well above the thermocline or there may be gradual thermal gradient from top to bottom with very low DO levels relatively close to the surface.

APPENDIX A: ABSTRACTS

ABSTRACT

Selective Withdrawal: Basic Concepts

Steve Wilhelms, WES

An intuitive approach is used to explain the stratified flow phenomenon known as selective withdrawal. Basic definitions and descriptions are given for the geometries, stratification conditions, and other processes that impact the selective withdrawal characteristics of ports and weirs. Equations that describe the establishment of a withdrawal zone are presented for various density stratifications. The basis for the numerical predictive model SELECT is presented. Application of SELECT to predict release concentrations of water quality constituents is discussed.

ABSTRACT

Hydraulic Design of the Selective Withdrawal Structures in the Rogue River Basin

Floyd Hall, NPP

The hydraulic design of the Lost Creek, Applegate, and proposed Elk Creek intake structures is discussed. Lost Creek and Elk Creek have single wet well designs while Applegate has a dual wet well design. A comparison of single and dual wet well designs is presented. The hydraulic design of the control systems and the sizing of the intake ports with regard to velocities are reviewed. A discussion of the techniques used to prevent possible nitrogen super-saturation at Lost Creek is also included.

ABSTRACT

RESERVOIR REGULATION AND

SELECTIVE WITHDRAWAL IN OREGON

U.S. Army Corps of Engineers
P.O. Box 2946, Portland, OR 97208

By: Richard A. Cassidy

Selective withdrawal structures have fused the real world considerations of water quantity with the previously esoteric issues of water quality to form dynamic water resources planning issues in the State of Oregon. Selective withdrawal capability in the Portland District's two Rogue River Basin projects not only has increased the day-to-day reservoir regulation considerations for the Corps of Engineers, but also has made profound changes in how the water resources agencies in the State of Oregon consider release changes.

Regulation of Lost Creek and Applegate Lakes represent another level of water resource management for the Portland District because, in addition to managing conflicting multiple purpose use of water, the control of water temperatures and turbidity has added another complex dimension to the project impact on fisheries. During the first few years following impoundment, the Portland District worked closely with the Oregon Department of Fish and Wildlife varying water temperature and outflows to determine the best combination for the fishery. Manipulation of releases from the two projects so inflammed the fishermen of the Rogue River Basin that the Portland District established a toll-free telephone service to inform fishermen of the latest regulation changes. Dissatisfaction ultimately lead Governor Victor Atiyeh to direct State water resources related agencies to no longer contact the Corps of Engineers directly concerning requests for unscheduled regulation changes. Since 1983, all State agency requests for the Corps of Engineers to make unscheduled regulation changes must be coordinated through the Oregon Water Resources Department.

ABSTRACT

Operation of Selective Withdrawal Facilities
Libby Dam, Montana

Jim Helms, NPS

During the late construction phase of the Libby Dam Project, located on the Kootenai River in Northwestern Montana, concerns were expressed over the effects of high nutrient loads upon the impoundment and downstream releases. Forecast methods indicated that growth and decay of large algal blooms might create periods when the dissolved oxygen in the lower reservoir levels were depleted. Releases from these levels might be detrimental to downstream fisheries either throught the absence of dissolved oxygen or the presence of hydrogen sulfide. Also, release temperatures from low reservoir levels would be too cold to promote a productive downstream fishery.

To alleviate these problems, release structures at the dam were modified to permit a selective withdrawal of reservoir water. Slotted tracks with baffles stacked one-on-top-of-another are used to control withdrawal of water from selected levels in the reservoir throughout nearly the entire depth of active storage.

Subsequent studies, after impoundment, allayed fears of entrophication effects and selective withdrawal operation has been directed towards enhancement of downstream fisheries via temperature modification. This mode of operation which has resulted in a significant improvement of the Kootenai River fishery will be discussed.

ABSTRACT

Alternatives for Improving Reservoir Water Quality

Margaret Morehead, SWL

Seasonal stratification in Table Rock Lake results in hydroelectric power generation releases which do not meet Missouri's State Water Quality Standard of 6 mg/l DO (dissolved oxygen) for downstream Lake Taneycomo. The Little Rock District studied the problem in response to a request by the State of Missouri (1976).

A successful solution of the problem is complicated by the magnitude of the flow rates of the turbine discharge. The 16,000 cfs upper design limit is equivalent to nearly 7.2 million gallons per minute. This is far in excess of the capacity of commercially available industrial equipment for mixing or adding oxygen for water treatment.

Twenty-five alternatives were evaluated for technical, environmental, economic, and social acceptability. Of these, only two plans which utilize selective withdrawal structures have the potential to meet the Missouri water quality standards of 6 mg/l DO. The nonstructural alternative which has been used on an interim basis was among the plans considered. The nonstructural plan involves a restricted operation which results in a substantial improvement in the quality of power generating releases, but it does not meet the State Water Quality Standard.

The use of selective withdrawal structures on two of the four penstock intakes has been identified as the most economically efficient solution. The application of selective withdrawal structures at Table Rock Dam is complicated by the need to meet both temperature and dissolved oxygen standards. Mathematical and physical model studies are needed to verify whether the structures can accomplish the objective and to develop additional engineering details for the selective withdrawal structures for Table Rock Dam.

ABSTRACT

A Review of Selective Withdrawal Performance in the Fort Worth District

Ronald Turner, SWF

The Fort Worth District has built or currently has under construction a total of 25 lakes. Of these, three are currently under construction. The construction of these lakes has been accomplished over a time span of 30 years. The District constructed 8 lakes in the early 1950's, and two of these had capability for selective withdrawal from three or four elevations in addition to the flood control conduit. A third lake constructed in this period had a single low flow intake or elevation which differed from the flood control conduit. The paper will describe the type of structures designed during that period and the basis for their design. The discussion will include available historic data on release water quality.

During the 1960's, eight lakes were constructed, none of which were provided with any selective withdrawal capability. The design and historic data on water quality for these lakes will be described.

An additional 3 lakes have been constructed in the 1980's, with two being provided with selective withdrawal facilities. Their design and any available historic data will be described.

Three lakes are currently under construction, all with selective withdrawal capability. Their design will be described. One of these is also designed for hydropower generation utilizing water from the selective withdrawal wet well.

Discussion about each period will include results of water quality modeling, historical problems encountered as a result of water quality, and efforts made at some locations with structural additions to reduce the impacts of poor water quality.

ABSTRACT

Modeling of Selective Withdrawal Intake Structures

by

CHANDRA ALLOJU

The Fort Worth District Corps of Engineers has several projects which can withdraw lake water from different selected levels. Two of those projects are Georgetown Lake and Granger Lake which were built recently. The two other projects Joe Pool and Ray Robert Lakes with selective withdrawal capability are under construction. A thermal simulation study has just been completed for the proposed Cooper Lake Project to determine if a selective withdrawal structure is needed. The workshop presentation will include models used in the design of selective withdrawal structures in the Fort Worth District its objective and experience gained in operating the structures.

ABSTRACT

Design of Bloomington and Warm Springs Towers

Frank Vovk and Laverne Horihan, MRO

The criteria available and development of rationale for the design of the multi-level outlet facility for selective withdrawal to achieve the desired water quality conditions for use downstream of Bloomington Dam are discussed. The justification for selection of the water quality system features are presented, and recommendations for future projects with similar requirements are furnished.

The analysis and design of the outlet works for Warm Springs Dam on Dry Creek, Sonoma County, California, are discussed. Water quality design considerations include both temperature and turbidity of discharged water which could pose problems to downstream fish and wildlife. A selective withdrawal system, discharging the water at several selected elevations, was considered to be needed to improve the downstream water quality. The adopted method of improving the quality of released water and final recommendations are presented.

ABSTRACT

Hydraulic Design
Bloomington Lake and F. E. Walter Dam Projects'
Selective Withdrawal Structures

Dennis Seibel

The Bloomington and F. E. Walter projects present two extremes of selective withdrawal structures (SWS) designs. Bloomington has small (6-ft-diameter) wet wells and Walter has very large (20 ft x 30 ft) wet wells. Bloomington design is similar to Warm Springs Dam's selective withdrawal structure. They both have inlets (or portals) and wet wells which are of similar size.

A brief description of the Bloomington design and its operating history will be presented. Bloomington's SWS was designed by the Omaha District, who also designed Warm Springs. The wet well is in effect a small standpipe with a short radius bend at its bottom to direct water towards the discharge control (or quality control (QC)) gate. The wet well has multiple inlets discharging into it at a 90 degree angle with no flare or transition to guide the flow into the wet well. Also, the wet well does not extend above the conservation pool level. A small diameter air vent is provided at the top of each wet well. At pool levels approaching the conservation pool level, water is forced up into the air vent. In addition, at large QC gate settings, (i.e., discharges approaching the capacity of the SWS) the intake tower was observed to vibrate. The program of testing currently underway to determine the cause of the vibration will be presented along with some preliminary findings. Also, a private interest is currently developing a proposal to add hydropower to the project. The hydropower proposal will be briefly described along with its impact on the Bloomington project.

The design of the new multi-level intake tower which was necessitated by a proposal to raise the reservoir (Bear Creek) behind F. E. Walter Dam 250 ft will be presented. The selective withdrawal system was designed using state-of-the-art design guidance. The system components were made as large as possible to minimize velocity effects. WES was consulted for their opinions on the final design.

In an attempt to avoid problems similar to those presently being experienced at the Bloomington project by the hydropower proposal, provisions for later hydropower addition were made in the design of the F. E. Walter intake structure.

ABSTRACT

Determination of Selective Withdrawal System Capacity for Intake Tower Design

Kenneth S. Lee

A method for determining selective withdrawal system (SWS) capacity was recently developed by Mr. Lee of the Baltimore District. This method evaluates the expected impacts on water quality when the SWS ability to control release quality is exceeded. This can occur under normal flow operations as well as flood flow operations. The capacity needed to avoid disastrous consequences are evaluated by this method. The procedure for determining capacity is to analyze and to prioritize the water quality control objectives at the project. This includes an evaluation of the effects of the project on water quality in the reservoir and downstream under all anticipated or likely configurations of operation. A case study is used to illustrate the procedure step by step. The example considers downstream water temperature control as the primary concern. Downstream temperature objectives, monthly flow distribution and maximum reasonable discharge during flood flow and normal flows are analyzed. Water temperature profiles from a thermal model are used to estimate downstream temperature deviation extremes under various flow control operations using several system capacities. The results are compared to temperature deviations which could be expected to cause fish kills. The final selection of the system capacities is based on the capacity which can avoid this critical deviation.

ABSTRACT

SELECT: The Numerical Model

Steve Wilhelms, WES

An overview of the numerical selective withdrawal model SELECT, which was developed at the U.S. Army Engineer Waterways Experiment Station, is presented. Its purpose and use are briefly discussed and the various subroutines and solution techniques are highlighted. The assumptions and limitations of the program are presented.

ABSTRACT

Blending in a Single Wet Well

Stacy Howington, WES

The concept of blending various qualities of water in a single wet well is presented. Potential application of the blending concept is discussed. The author presents a simplified theoretical approach to describe the mechanics of the blending phenomenon. A brief discussion of the theory is presented. Examples of the current use of blending are presented highlighting these limitations and identifying areas that are in need of additional research.

ABSTRACT

Design of Selective Withdrawal Intake Structures

Jeffery P. Holland

Presented herein is an overview of the general methodology used by the U.S. Army Corps of Engineers in the design of selective withdrawal intake structures. Considered are the types of structures generally used by the Corps; the computation of the distribution of withdrawal for a given intake from a density-stratified reservoir; the optimum location of selective withdrawal intakes; and the hydraulic constraints which must be satisfied for effective structure flow control.

ABSTRACT

Operational Tools: Selective Withdrawal and Daily Operational Strategy

Jeffery P. Holland and Steven C. Wilhelms

The authors present the results of recent selective withdrawal research in the form of a general mathematical description of this stratified flow phenomenon. Results of past and present researchers were compared. Through symmetry arguments and the withdrawal angle concept, those results were reduced to a single expression. A new description for boundary interference, which often impacts the formation of the withdrawal zone, was explicitly included in the mathematical formulations. These results were incorporated into the computer code SELECT, a numerical model of withdrawal from a stratified impoundment. This model has been used extensively for long-term evaluation purposes in conjunction with reservoir simulation models. However, when coupled with a port-selection algorithm, the model has excellent potential as a tool for day-to-day decisions regarding hydraulic structure operation. The authors present an example of model application to provide guidance on outlet structure operation for maintenance of release water quality.

ABSTRACT

Operational Tools: Optimal Control of Reservoir Water Quality

Steven C. Wilhelms and Michael L. Schneider

This paper presents a methodology that combines simulation and optimization techniques to determine guidelines for operating a selective withdrawal reservoir structure to meet downstream water temperature objectives. Optimal operation is achieved when current operations anticipate future critical temperature conditions.

A one-dimensional reservoir thermal simulation model developed by the U.S. Army Engineer Waterways Experiment Station was used to simulate the thermal stratification cycle of a reservoir. The model was interfaced with a formulation called objective-space dynamic programming (OSDP) to develop the optimal operation strategy for each decision period. The OSDP formulation retains the integrity of the simulation model and minimizes the deviations of predicted release temperature from downstream target temperature over the stratification cycle. Application to a case study shows the potential for using the dynamic programming technique, as compared to the normal period-by-period operation, to improve performance of the system.

ABSTRACT

Overview of Warm Springs

Harold Huff, SPK

An overview of the Warm Springs project is presented. A brief history from the project's inception to its current status is given. An outline of the field trip and points of interest at the dam are provided.

ABSTRACT

Field Measurements at Intake Structures

Ellis Dale Hart, WES

Prototype water-quality tests were conducted at Beltzville Dam in Pennsylvania. The purpose was to determine the location and degree of reaeration of flow that occurred as it passed through the outlet works. Temperature and dissolved oxygen data were collected in the reservoir, at seven stations within the outlet structure, and in the downstream channel. The tests involved various flow rates and intake levels.

Similar measurements are scheduled to be conducted at Taylorsville Dam, Kentucky in the summer of 1985. In addition, because of the unique intake tower trash rack design, inlet velocities will be measured for determining entering velocity profiles.

ABSTRACT

Thoughts and Considerations for Hydraulic Design

T. J. Albrecht, Jr., SPD (retired)

Experiences and reflections on the design of selective withdrawal systems. The author's thoughts on capacity, velocity within the system, selection of gates and valves, and control systems will be covered. Both existing and planned projects will be utilized to illustrate specific features, problems, and alternatives.

ABSTRACT

Selective Withdrawal Needs for Lake Greeson, Arkansas

R. E. Price and D. R. Johnson, LMK

Lake Greeson, a 7,000 acre reservoir located on the Little Missouri River in west central Arkansas, is an authorized combined power and flood control project. the 941-ft-long concrete dam, which was completed in 1953, provides up to 407,900 acre-feet of storage with 128,200 acre-feet allocated for flood protection, and 202,100 acre-feet for power generation. Normal releases from the three penstocks, each 8.5 feet in diameter, are from elevation 485 NGVD with flood control gates located at 444 NGVD.

Lake Greeson may be classified as a monomitic lake with typical thermal stratification and low dissolved oxygen levels in the hypolimnion. Because the penstocks are located in the hypolimnial region, discharges during the summer months are much cooler than inflows and much lower in dissolved oxygen and release flows are not stable enough to sustain a cold water fishery. This creates stress on the downstream environment and fishery, which the Arkansas Game and Fish would like for the Corps to improve. For a 3 year period between 1981 and 1984, temperature and dissolved oxygen monitors were placed on the inflow and discharge of Lake Greeson to identify the extent of the problems. Results of this investigation indicate that the temperatures of the releases are much colder than inflows during the late spring and summer and that dissolved oxygen levels may drop below state standards for extended periods of time.

To correct this problem, several alternatives are available and range from multilevel outlets to simply plating over the trash racks. Evaluation of these alternatives is underway.

Selective Withdrawal
Structure Operation Experiences in ORD

Dave Buelow, ORD

More than one-third of the 75 reservoir projects in ORD have multi-level intake structures ranging from fixed high level riser to single and dual wet-well systems. Many of the structures were designed and constructed without the benefit of reservoir heat budget/withdrawal zone analysis, the oldest one having been placed in operation in 1952. Operational objectives at these projects are primarily for temperature control; however, the flexibility inherent in the structure designs is frequently used to enhance other quality characteristics such as dissolved oxygen and to ameliorate problems such as high levels of iron, manganese and hydrogen sulfide. Some do not have a strict downstream objective and are operated in the best interests of the lake and tailwater. Data collection efforts in support of operation vary from minimal to adequate. Problems encountered include insufficient flexibility in withdrawal elevation, insufficient design discharge capacity and undersized hydraulic features.

This presentation will provide an overview of ORD experiences in the conceptualization and operation of selective withdrawal structures. Specific topics to be addressed are types and capabilities of selective withdrawal structures in use; operational criteria; data collection; overall performance; problems encountered; and future problems, specifically hydropower retrofit at existing dams. Specific projects will be used to demonstrate salient features and operational capabilities. Central to all discussions will be the integration of water quality considerations with res-reg activities to yield a comprehensive water management framework.

Overview of Pittsburgh District
Selective Withdrawal Operation Experiences
By
Michael Koryak
U.S. Army Engineer District, Pittsburgh

ABSTRACT

The Pittsburgh Engineer District is currently operating four structures with selective withdrawal intakes. These projects are East Branch Dam constructed in 1952, Kinzua and Michael J. Kirwan Dams which were both completed in 1966, and Woodcock Creek Dam which has been operational since 1974. A fifth structure with a highly innovative intake design, Stonewall Jackson Dam, is now under construction. Besides these five dams with selective withdrawal intake towers, a municipal water supply intake at Tygart Dam (1938) and a significant difference between the invert elevations of the sluice gates and low-flow discharge portal at Conemaugh Dam (1953) both have selective withdrawal implications.

One of the most important lessons that the District has learned in its more than three decades of experiences with these structures is that operating objectives can change and operational flexibility is highly desirable. The District has utilized selective withdrawal to maintain both cold and warm water outflow fisheries; for the conservation of warm, cold, and very cold water strata within a single reservoir to maintain a "three story" lake fishery; to control outflow water quality; and for the control of reservoir stratification patterns and subsequent in-pool mixing and dilution of acid mine drainage pollution and reservoir primary biological productivity.

Some obstacles encountered in the operation of these intake structures are related to vertical placement of the gates and insufficient withdrawal options. These design deficiencies have occurred at projects built before reliable predictive reservoir modeling methodologies were available. Modification of one of these older systems has been considered.

Other existing and potential problems involve pump-back currents and stratification disruption from pumped-storage hydropower generation, conventional hydropower conversion of existing projects, and periodic summer flood event drawdowns where the required discharge exceeds the capacity of the selective withdrawal system. During such events, cold hypolimnetic waters with high Fe, Mn, Al, H_2S and NH_3 concentrations have been discharged through sluice gates. Similar problems have occurred during maintenance shutdowns of systems and there have been a few learning curve misunderstanding and mistakes.

Selective Withdrawal From Any Level
Between Minimum Pool and Spillway Elevation
At Stonewall Jackson Dam, West Virginia

By

John C. Gribar and Robert W. Schmitt
U.S. Army Engineer District, Pittsburgh

ABSTRACT

A selective withdrawal intake system is presently under construction by the Pittsburgh District to meet water quality and temperature objectives downstream from the Stonewall Jackson Dam. Instead of conventional fixed elevation multiport intakes, a movable intake design was chosen which can access numerous levels in the impoundment.

Stonewall Jackson is a multi-purpose project located on the West Fork River and has a drainage area of 102 square miles. Storage at spillway crest will be 75,000 acre feet with a maximum depth of 75 feet.

A fixed-port intake system was considered intially to meet outflow requirements. This consisted of two wells at the upstream face of the dam, each with four fixed-level intakes. The port elevations for this scheme were positioned using a computerized thermal simulation of the lake.

Because of summer stratification conflicts between a rigid downstream water temperature schedule and outflow water quality objectives and restrictions, a more flexible withdrawal design was desired. Of primary concern was the problem of blending cool hypolimnetic waters into the discharge to support a downstream trout fishery during the summer when the deeper, colder strata of the lake are expected to have unacceptably high iron and manganese concentrations. An additional area of concern was a potential problem of turbid temperature-density currents penetrating the lake near the elevation of a fixed intake. With a fixed port intake works, circumstances could develop where outflow temperature goals would have to be sacrificed in the interest of water quality. Therefore, alternative withdrawal schemes were investigated.

The adopted design is an innovative arrangement consisting of two separate towers projecting from the upstream face of the dam, each with three movable gate leaves to allow for withdrawal of waters from any level between the spillway elevation of 1082 and the minimum pool elevation of 1038. As such, it will be possible to withdraw from the 20°C target water temperature level without having to use any of the deep, cold, high iron-content waters. It may also be possible to withdraw simultaneously from more than one level into the same tower.

While enhancing operational water quality flexibilities, an additional benefit of this design will allow the withdrawal of desired temperature through only one tower without the need for stilling basin blending. This feature will afford full utilization of the station hydropower plant. The ability to meet the temperature schedule from either tower would also be advantageous during periods when one is out of service for inspection or repair or when the stilling basin is dewatered and its bypass is operating. During high outflows, eddying can be minimized and symmetrical discharge can be provided with equal flow through the two water-quality sluices. This will be possible since the flows can be balanced from each tower and will not be governed by irregular flow requirements of temperature blending needed with a fixed-port system.

ABSTRACT

Selective Experiences: Alias Withdrawal Pains

Richard Punnett, ORH

Eleven Corps of Engineer lakes, having various designs of outlet works, were evaluated for capability to meet the release objectives of the project purposes. Only two of the outlet works were designed with the aid of numerical lake models. The data collection and management, associated with the release regulation, was also discussed.

ABSTRACT

Design and Performance of Skimming Weirs in the Kansas City District

by Walter M. Linder, Chief, Hydrologic Engineering Branch, Kansas City District

Two distinctly separate techniques have been used to meet water quality requirements downstream of Kansas City District lakes. These are (1) underwater skimming weirs at two hydroelectric power plants and (2) multilevel gated low flow intakes in conventional reservoir outlet structures. The three different types of multilevel intakes used include (1) multilevel gated inlets discharging into a wet well, (2) a wet well with the upstream face composed of stop logs with one or more openings that can be placed at any desired elevation, and (3) multilevel gated inlets discharging through individual pipes into the flood control conduit. Observed field data show the skimming weirs are performing their intended function very well. No specific field data have been collected on the performance of the multilevel gated intakes. However, no downstream water quality problems have been observed as a result of their operation.