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Reaeration at Low-Head Hydraulic Structures

by Steven C. Wilhelms
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Prepared for Headquarters, U.S. Army Corps of Engineers

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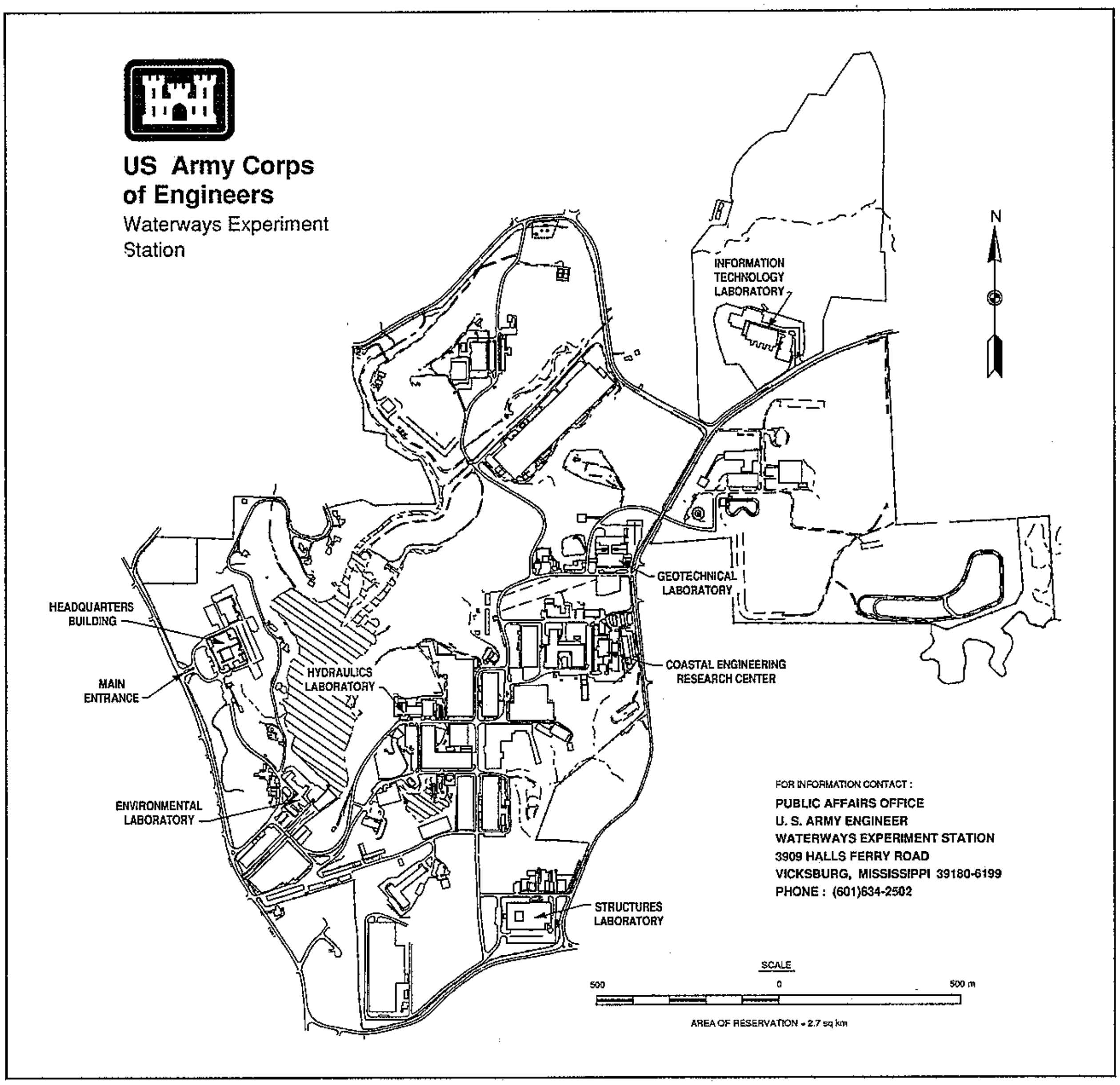

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PREFACE

The work reported herein was conducted as part of the Water Quality Research Program (WQRP). The WQRP is sponsored by the Headquarters, US Army Corps of Engineers (HQUSACE), and is assigned to the US Army Engineer Waterways Experiment Station (WES) under the purview of the Environmental Laboratory (EL). Funding was provided under Department of the Army Appropriation 96X3121, General Investigation. The WQRP is managed under the Environmental Resources Research and Assistance Programs (ERRAP), Mr. J. L. Decell, Manager. Mr. Robert C. Gunkel was Assistant Manager, ERRAP, for the WQRP. Technical Monitors during this study were Mr. David Buelow, Mr. Jim Gottesman, and Dr. John Bushman, HQUSACE.

This report was prepared by the Hydraulics Laboratory (HL), WES, and the St. Anthony Falls Hydraulic Laboratory (SAFHL) of the Department of Civil and Mineral Engineering, University of Minnesota, Minneapolis, as a summary of results from research into the area of gas transfer at low-head hydraulic structures. This effort was funded under WQRP Work Unit 32369 entitled "Reaeration at Low-Head Structures." The study was conducted under the general direction of Messrs. Frank A. Herrmann, Jr., Director, HL; Richard A. Sager, Assistant Director, HL; and Glenn A. Pickering, Chief, Hydraulic Structures Division (HSD), HL.

This report was prepared by Mr. Steven C. Wilhelms of the Reservoir Water Quality Branch (RWQB), HSD; Dr. John S. Gulliver, SAFHL, and Mr. Kenneth Parkhill, SAFHL, under the direct supervision of Dr. Jeffery P. Holland, former Chief, RWQB. Mr. Perry Johnson, US Bureau of Reclamation, and Dr. Charles Bohac, Tennessee Valley Authority, provided review comments. Field and literature studies were conducted by personnel from HL and SAFHL. Assistance in data analysis and report preparation was provided by Ms. Laurin I. Yates, RWQB, and Mrs. Barbara A. Parsons, HSD, and Mr. Benjamin Erickson, SAFHL. The report was edited by Mrs. Marsha C. Gay, Information Technology Laboratory, WES.

At the time of publication of this report, Director of WES was Dr. Robert W. Whalin. Commander and Deputy Director was COL Leonard G. Hassell, EN. Dr. Roger L. A. Arndt was Director, SAFHL.

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CONTENTS

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) unit as follows:

Multiply	By	To Obtain
degrees (angle)	0.01745329	radians
feet	0.3048	metres

REAERATION AT LOW-HEAD HYDRAULIC STRUCTURES

PART I: INTRODUCTION

Background

1. Presently, one of the most cited water quality parameters in the freshwater hydrosphere (rivers, lakes, and reservoirs) is dissolved oxygen (DO). The oxygen concentration in surface waters is a prime indicator of the quality of that water for human use as well as use by the aquatic biota. Many naturally occurring biological and chemical processes use oxygen, thereby diminishing the DO concentration in the water. The physical process of oxygen transfer or oxygen absorption from the atmosphere or air bubbles acts to replenish the used oxygen. This process is termed reaeration.

2. Low-head hydraulic structures within the US Army Corps of Engineers are generally associated with navigation projects. These structures are usually "run-of-the-river" and are used to maintain a constant upstream pool elevation. The oxygen transfer in these deeper, slower pools is lower than that of the open river. Biological and chemical oxygen demands may accumulate and concentrate in the impoundment and thereby degrade the DO concentration in the stored water because of the excess demand compared to reaeration potential. Without sufficient reaeration, release of this water may pose an environmental and water quality concern.

3. Some hydraulic structures exhibit remarkable reaeration, while others do very little to increase DO. If the DO in releases is lower than desired or required, operational or structural modifications or artificial aeration can be employed to improve the release DO concentration. The design engineer will be faced with the need to evaluate the oxygen transfer characteristics of existing conditions at a hydraulic structure for comparison with the characteristics of a proposed modification. As in any engineering application, it is imperative to possess a clear definition of the processes that affect water quality and the expected change as a result of implementing the particular improvement technique. Hence, the potential impacts of alternative techniques on the reaeration processes must be thoroughly understood and quantified.

4. In the past, the focus of interest in gas exchange at hydraulic

structures has been the transfer of atmospheric gases: absorption of oxygen to replace DO used by aquatic processes and the absorption of nitrogen, potentially resulting in nitrogen supersaturation. More recently, however, the desorption of volatile organics or toxics that may be dissolved in the water has become important. The air-water transfer of any chemical is called gas transfer because the chemical is a gas in one of the two phases. In general, the physical processes that influence oxygen absorption also affect the transfer of any dissolved volatile compound. Thus, it becomes even more important for the design engineer to possess a thorough understanding of (a) the physics of gas transfer, (b) the quantification of gas transfer, (c) the important physical processes, and (d) the hydraulic conditions that can enhance or degrade gas transfer.

Objective and Scope

5. The objective of this report is to familiarize field engineers with the oxygen transfer process and the oxygen transfer characteristics of various low-head hydraulic structures. The physics of gas transfer are conceptually explained and the mathematical description of the gas transfer process is developed. The important physical processes and their impact on the variables in the gas transfer equation are identified. The hydraulic conditions that contribute to these physical processes are described and applied to oxygen transfer, or reaeration.

6. It is hoped that field engineers, when familiar with the conceptual descriptions provided in this report, can qualitatively evaluate the oxygen transfer characteristics at a structure based solely on observed hydraulic conditions, e.g., they will be able to estimate whether the structure, upon testing, would exhibit a large or small degree of gas transfer. With the mathematical description of oxygen transfer at a "generic" type of structure and an understanding of how hydraulic conditions contribute to oxygen transfer, field engineers should be able to "bracket" or roughly estimate the transfer that is occurring at a specific structure.

7. The hydraulic structures at most low-head projects usually consist of a gated sill, gated low-head spillway, and a fixed- or adjustable-crest weir. The gas transfer analyses reported herein emphasize these "generic" types of structures. More complete descriptions of the geometry and flow

conditions at these structures are given in Part IV, "Hydraulics at Various Structures."

PART II: REAERATION THEORY

Physics of Gas Transfer

8. The transfer of any chemical across an air-water interface can be considered two processes acting together: (a) transport due to molecular diffusion and (b) transport due to turbulent mixing. These are characterized by the capture (or release) of gas molecules at the air-water interface and subsequent distribution of the gas throughout the water body. Molecular diffusion is caused by the inherent kinetic energy possessed by the gas molecules, whereas transport due to turbulent mixing results from the application of external forces on the water body.

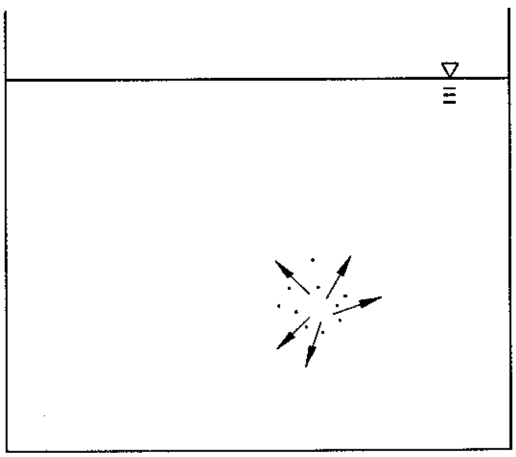

Figure 1. Transport due to diffusion in quiescent water (after Tsivoglou and Wallace 1972)

9. Consider the quiescent body of water shown in Figure 1. If a group of dissolved molecules could be placed in the water body without disturbing the water, the dissolved molecules would gradually spread throughout the water body and eventually achieve a uniform distribution. The molecules would accomplish this in totally still water because of their inherent kinetic energy associated with their surrounding temperature. The movement of the molecules, which is entirely random, causes the molecules to uniformly distribute throughout the water body. This process is termed molecular diffusion.

10. Molecular diffusion is usually described by Fick's first law*

$$J = -D_m \frac{dC}{ds} \qquad (1)$$

which states that J, the mass flux per unit area across an interface or a plane (in a direction away from the higher concentration, hence, the negative

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* For convenience, symbols and unusual abbreviations are listed and defined in the Notation, Appendix E.

sign), is proportional to the concentration gradient dC/ds across the interface, where dC is the change in concentration across the interface thickness ds . D_m is the molecular diffusion coefficient and is a measure of the ability of a medium to diffuse dissolved material. This relationship implies that even though the molecular diffusion coefficient may be small, the flux from one volume with a high dissolved concentration into a volume with zero dissolved concentration will initially be very high because of the large gradient across the interface between the volumes.

11. For an example application of these concepts to gas transfer, again consider a completely quiescent homogeneous body of water. Assume that the body of water is devoid of oxygen and that the surface is instantaneously exposed to the atmosphere. Initially, there would be a very high rate of oxygen absorption into a very thin layer at the water surface because of the large concentration gradient across the interface (Fick's law). However, since this is a quiescent water body, only molecular diffusion will cause oxygen molecules to move down into the water body. Since the diffusion downward is impeded by the molecular structure and viscosity of the water, oxygen molecules will tend to collect rapidly in the surface layers. Consequently, the rate of absorption would fall quickly because of the rapid decrease in the concentration gradient across the air-water interface. Thus, <u>the rate of gas transfer to the quiescent water would be very small without turbulent transport, or mixing.</u>

12. Consider that same body of water, devoid of oxygen and with its surface instantaneously exposed to the atmosphere. However, this time, the water is being agitated causing turbulent mixing (Figure 2). Turbulent mixing transports a water element to the water surface, which degrades the thin film at the surface for a short period. Although the period of exposure to the atmosphere may be very short and the molecular diffusion coefficient may be small, the element will absorb a significant amount of oxygen because of the large gradient across the interface (according to

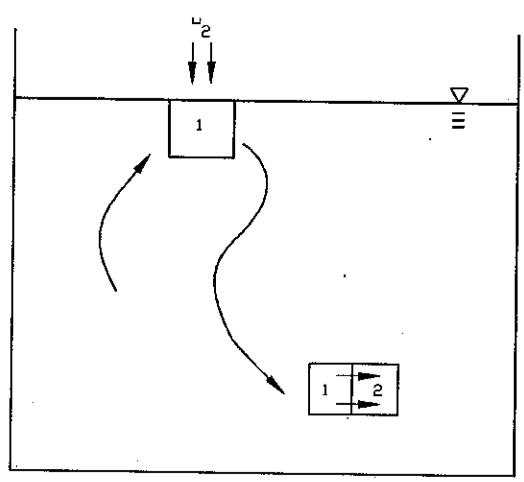

Figure 2. Transport due to diffusion and turbulent mixing

Fick's law, the rate of transfer would be large). Similarly, when that element is transported by turbulent mixing into the body of fluid, the rate of oxygen transfer to surrounding elements will be very high because of the relatively large gradient. It is easily deduced from this description that turbulent mixing greatly speeds oxygen transfer.

13. It is important to realize that gas transfer is governed by the same physical processes, turbulent mixing and molecular diffusion, regardless of the direction of transfer, i.e., gas absorption or desorption. This fact provided the basis for developing all tracer gas technologies. Tsivoglou et al. (1965) and Tsivoglou et al. (1968) reported this basis and the development of a technique that used radioactive krypton-85, injected into the water, as a tracer for oxygen absorption. The krypton-85 was being transferred (desorbed) to the atmosphere, while oxygen was being absorbed. Rathbun et al. (1978) modified the radioactive tracer technique with the injection of hydrocarbon gases, such as propane and ethylene, as tracers. Wilcock (1983) used methyl chloride as a tracer gas in this same fashion. McDonald, Gulliver, and Wilhelms (1990) reported preliminary measurements of gas transfer with in situ methane and sulfur hexafluoride. McDonald and Gulliver (1991) presented the development of methane as an alternative gas for measuring the gas transfer in a particular flow situation at hydraulic structures. The critical point upon which all this work was based is that measurements made with one gas can be used to calculate the transfer of another gas (Gulliver, Thene, and Rindels 1990).

Mathematical Description

14. Gas transfer is usually considered to be a first-order process in which the rate of change of the chemical's concentration in the water is linearly dependent on the ambient concentration. The driving force in the transfer process is the difference between the actual concentration of the dissolved gas in the water and the concentration that corresponds to equilibrium with the air. This equilibrium concentration or "saturation concentration" is defined using a Henry's law constant as follows:

$$C_{si} = H_i P_i \tag{2}$$

where

C_{si} = saturation concentration of gas i , g/m^3

H_i = Henry's law constant for gas i , g/m^3 – atm

P_i = partial pressure of gas i in the atmosphere, atm

Henry's law states that at a given temperature, a liquid can absorb an amount of gas that is proportional to the partial pressure of that gas in the overlying atmosphere. Thus, an "equilibrated" state exists at the saturation concentration. There is a rate coefficient on the air side (gas film coefficient) and on the water side (liquid film coefficient) of the interface. The inverse of these coefficients can be seen as "resistance" to gas transfer (Liss and Slater 1974), where the air- and water-side resistance act in series. For most compounds, such as O_2, CO_2, CH_4, N_2, etc., the water-side resistance is much greater than the air-side resistance, and only the liquid film coefficient is considered. The gas film coefficient can also be important in the transfer of H_2O (evaporation), NH_3, SO_2, and some herbicides and pesticides designed for this purpose. Because this report on reaeration concerns oxygen transfer, only the water-side resistance need be considered.

15. A measure of the driving force causing oxygen transfer can therefore be defined as the difference between the concentration in the water and the saturation concentration. This quantity is called the "saturation deficit" and is mathematically defined by

$$D = C_s - C \tag{3}$$

where

D = saturation deficit

C_s = saturation concentration

C = ambient concentration in water

At saturated conditions, the deficit is zero. If the deficit is positive, the water is "undersaturated." If the deficit is negative, then the water is "supersaturated." Oxygen would be absorbed for the former and desorbed for the latter to achieve the equilibrated saturated state.

16. To develop an expression that describes oxygen transfer as a

first-order process, the flux across the air-water interface is typically written as

$$J = k_L(C_s - C) = k_L D \tag{4}$$

where k_L is the liquid film coefficient for oxygen transfer. Examination of this equation and Equation 1 indicates that the liquid film coefficient is related to the concentration gradient near the air-water interface, and may be interpreted as the ease with which dissolved gases move across the air-water interface. The liquid film coefficient is dependent on the internal structure or ordering of water molecules and the breakdown of that structure by mixing.

17. To determine the rate of change in concentration for a well-mixed volume of water, the total rate of mass flux into a water body must be calculated by multiplying Equation 4 by the surface area and then dividing this quantity by the volume of water. Dividing the total mass flux by the volume results in the rate of change in concentration:

$$\frac{JA}{V} = \frac{dC}{dt} = -\frac{dD}{dt} = k_L\frac{A}{V}(C_s - C) \tag{5}$$

when C_s is constant and where

A = surface area associated with the volume V, over which transfer occurs

V = volume of the water body over which A is measured

dC/dt = rate of change of concentration

dD/dt = rate of change of the saturation deficit

18. Assuming that k_L, A, and V are constant over the time of flow, Equation 5 can be integrated to the following mathematical model of water-side controlled gas transfer:

$$\frac{D_f}{D_i} = \exp\left(-k_L \frac{A}{V} t\right) = \exp(-k_L a t) = \exp(-K_2 t) \tag{6}$$

where

D_f, D_i = final and initial oxygen deficits, respectively

t = elapsed time from initial to final deficits

a = specific surface area, A/V

K_2 = reaeration or oxygen transfer coefficient

From the perspective of flow in a stream reach or through a hydraulic structure, the initial and final deficits would be the upstream and downstream deficits, respectively, and the elapsed time would be the time of flow from the upstream to downstream locations. As presented later, some researchers have defined a quantity r as the deficit ratio, which is the inverse of Equation 6 such that

$$r = \frac{D_i}{D_f} = \exp\left(k_L a t \right) \tag{7}$$

However, the mathematical qualities of this formulation are less desirable than those of Equation 6 (Wilhelms and Smith 1981; Rindels 1990).

19. A convenient parameter called transfer efficiency can be defined as the fractional part of the incoming deficit that is satisfied as the water flows through the structure or stream reach. This efficiency E can be expressed mathematically with Equation 6 as

$$E = \frac{(C_f - C_i)}{(C_s - C_i)} = 1 - \frac{D_f}{D_i} = 1 - \exp\,(-k_L \frac{A}{V}\, t) \tag{8}$$

$$E = 1 - \exp\,(-K_2 t) \tag{9}$$

where C_f and C_i are the final and initial oxygen concentrations, respectively. If the transfer efficiency is zero, then there was no oxygen transfer and the downstream concentration equals the upstream concentration. If the transfer efficiency equals 1.0, then all of the upstream deficit was met by the oxygen transfer and the downstream concentration is at saturation. Furthermore, the transfer efficiency of oxygen can be transferred to other gases and vice versa through the indexing method developed by Gulliver, Thene, and Rindels (1990).

Important Physical Processes

20. Three processes occur at hydraulic structures that can significantly increase oxygen transfer. They can be related to the parameters given in Equations 8 and 9. The impacts of these processes are governed mainly by the fluid mechanics and flow conditions at the structure. The following provides a general description of these processes and their effect on gas transfer at hydraulic structures:

a. Turbulent mixing at the water surface and within the body of the flowing water. It would seem logical that the rate of turbulent mixing would significantly affect gas transfer because of the concept of water-surface renewal (water surface that is swept away from the surface and "renewed" with water from below) (Danckwerts 1951) causing increased gas transfer. A high degree of turbulent mixing, such as occurs on the face of a spillway or in a tailwater plunge pool, would increase k_L and likewise E .

b. Increased interfacial area resulting from air that has been entrained into the flow. When air is entrained into the flow either from the surface or at a plunge point, the surface area available for gas transfer can increase dramatically. Gulliver, Thene, and Rindels (1990) estimated that entrained air due to free surface aeration in a 1-ft*-deep flow on a 30-degree slope can increase the air-water surface area by a factor of nearly 500 compared to the unit area of surface exposed to the atmosphere. Thus, if air is entrained, gas transfer should increase significantly for a given flow condition. Although not defined, the amount of entrained air should also be a factor in establishing the oxygen transfer character of a hydraulic structure. The water-surface area A is greatly increased when air is entrained into the flow. This would also act to increase E .

c. Increased saturation concentration from the higher pressure experienced by bubbles in the plunge pool. In addition to the contribution that air bubbles make to the air-water surface area, absorption of atmospheric gases from the air bubbles can be increased because of the increased pressure that the bubbles experience as they are transported into the depth of the structure's stilling basin. Increased hydrostatic pressure on entrained air causes an increase in the saturation concentration (Equation 2) and thereby increases the saturation deficit (Buck, Miller, and Sheppard 1980; Wilhelms, Schneider, and Howington 1987; Wilhelms and Gulliver 1990; McDonald 1990). Thus, although the transfer efficiency does not change due to this effect, the downstream concentration would increase because of an increase in C_s .

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* A table of factors for converting non-SI units of measurement to SI (metric) units is found on page 5.

Each of these processes is included either directly or indirectly in Equations 8 and 9. Turbulent mixing is characterized by the liquid film coefficient k_L. The increase in oxygen transfer due to greater interfacial area because of entrained air bubbles is included in the interfacial area term A. The effects of pressure on oxygen transfer in a plunge pool result in an increased saturation concentration C_s. This increase in C_s can cause supersaturated oxygen and nitrogen concentrations, compared to the surface saturation concentration. By recognizing these processes and their impacts on oxygen transfer, flow conditions observed in a physical model or full-scale project can, at a very minimum, be qualitatively evaluated. If one objective in the operation of a hydraulic structure is to provide for oxygen transfer, this evaluation can identify means of meeting that objective.

PART III: MEASUREMENT TECHNIQUES

21. The efficiency, along with the variables in the exponents of Equations 8 and 9, defines the gas transfer characteristics of a structure. To calculate efficiency or estimate those variables, upstream and downstream dissolved gas concentrations are required. Essentially, two methods are available for collection of these data: (a) direct in situ measurements and (b) discrete sample collection with a subsequent analysis for gas concentrations. These two methods are discussed in the following paragraphs and an uncertainty analysis is presented that defines the minimum values required for an accurate assessment. Additionally, conditions for testing a hydraulic structure are discussed that will result in a complete picture of the oxygen transfer character of the structure.

In Situ Measurements

22. In situ measurements are usually made for DO with a portable electronic meter that uses a polarographic probe (Yellow Springs Instrument Company 1975; American Public Health Association 1975). The probe consists of two metal electrodes in contact with an electrolyte separated from the test water by a membrane that is permeable to oxygen. A polarizing voltage applied across the electrodes causes oxygen that has permeated the membrane to react at the cathode, resulting in an electrical current flow. Oxygen passes through the membrane at a rate proportional to its concentration in the water. If the concentration is high, more oxygen passes through the membrane, resulting in an increased current flow. The current flow is calibrated to correspond to a specific DO concentration.

23. These probes are usually very reliable, simple to operate and maintain, and relatively easy to calibrate. Failure to provide adequate measurements has occurred only under physical or biological fouling of the probe or extremely cold weather conditions. They are accurate to 1 percent of full scale or 0.1 mg/ℓ. They may be calibrated with an air contact or chemical titrametric method. Typically, these instruments incorporate a thermistor for temperature measurements and a stirrer for moving water past the permeable membrane.

Discrete Sampling and Analysis Techniques

24. Discrete sample collection with subsequent analysis is usually implemented when gases such as H_2S or CO_2 are of interest, although DO can also be determined with a discrete sampling method. A discrete sampling technique may also be used when oxygen concentrations are at saturation and a tracer gas must be used to determine the transfer characteristics. There are several methods for collection and analysis of discrete samples. The method used depends upon the dissolved gas of interest. A chemical titrametric technique can be used for accurate DO measurements. Gas chromatography can be used for DO, nitrogen, hydrogen sulfide, carbon dioxide, methane, and any other volatile gas of sufficient concentration. Chemical titrametric and colorimetric methods can also be used for some of these gases, with variable measurement accuracy.

Sampling techniques

25. The sampling instrument usually depends upon the gas of interest and the analysis technique. For example, samples for titrametric analysis for DO are usually collected in 300-ml biochemical oxygen demand (BOD) bottles in a sampler specifically designed for this purpose. Thene and Gulliver (1989) designed a similar sampler for use with 40-ml bottles in propane and methane tracer gas analyses. For any sampling technique, the critical issue is to collect and analyze a sample that is "undisturbed," relative to in situ measurements. Thus, care must be exercised in the collection and handling of samples to ensure that no loss or gain of gas occurs as a result of the sampling process. This may dictate that a special sampler be designed and constructed to meet the needs of the particular study.

Sample analysis techniques

26. As previously mentioned, a chemical titrametric technique is available for determining the DO concentration. For the azide modification of the Winkler titration technique, divalent manganese sulfate and alkaline iodide-azide are added to the 300-ml BOD bottles. They chemically react, producing a divalent manganous hydroxide precipitate. The oxygen in the water sample rapidly oxidizes an equivalent amount of the manganous hydroxide to hydroxides of higher valency states. Adding sulfuric acid to this solution causes the reversion of the oxidized manganese to the divalent state, while liberating iodine equal to the original DO content. The DO is determined by titrating

the iodine in the solution to its end point with standard solutions of sodium thiosulfate. More detail is given in the handbook *Standard Methods for the Examination of Water and Wastewater* (American Public Health Association 1975).

27. This technique is relatively simple, but is more time-consuming than the polarographic probe technique. Care must be exercised in sample handling to prevent gas transfer, but in general, this technique results in measurements that are reliable and accurate.

28. Gas chromatography is an analysis method useful for DO, nitrogen, hydrogen sulfide, carbon dioxide, propane, or methane. The technique is based on "sieving" component gases in a sample with a "molecular sieve" or absorbent polymer. The gas sample passes through a long tube (column) packed with a porous solid. The solid acts like a sieve, slowing the movement of larger diameter gas molecules or absorbing particular gases. This results in separation of the sample component gases of interest by the time the sample exits the column at the detector. Output from the detector is calibrated to correspond to a certain amount of gas. The type of detector used in the gas chromatograph (GC) depends upon the gas of interest. For DO and nitrogen, a thermal conductivity detector can be used in the GC. For propane and methane, a flame ionization detector is used in the GC. For sulfur hexafluoride, an electron capture device is used in the GC. These are examples of the sampled gases and tracer gases of interest for hydraulic structures.

29. In using the GC to determine the mass of a volatile compound, a headspace sample preparation technique has proven successful (Thene 1988). A headspace of inert gas is created in the sample bottle. By agitating the sample bottle, nearly all the gas in the water is stripped into the headspace. Samples from this headspace volume are extracted and injected into the GC for determination of compound mass. The actual gas concentration in the water sample is calculated based on the mass of the compound in the headspace, the relative volumes of the headspace and water sample, and the Henry's law constant of the particular gas.

30. Use of the headspace analysis technique with a GC is much more complex than the titrametric or colorimetric techniques. Experienced laboratory personnel are required to operate the GC and prepare the samples. However, for hydrocarbon tracer gases, this technique proves to be one of the best.

Uncertainty Analysis

31. Any adequate analysis requires that the accuracy and precision of the data be assessed. Uncertainty analysis techniques provide the tools and methodology to perform this evaluation. This technique can be used to evaluate existing data, or from a planning perspective, it can provide guidance on data requirements to assure that high-quality data are collected, which will provide a proper foundation for drawing conclusions. For in situ or discrete sampling methodologies, the uncertainty in the collected data must be evaluated for confidence to exist in any subsequent conclusions. Details of the analysis technique are presented in Appendix A for a typical data collection program.

32. If a first-order, second-moment uncertainty analysis (Rindels and Gulliver 1989) is performed upon the measurements that go into determining the transfer efficiency, the uncertainty in the efficiency can be expressed as:

$$U_E = \frac{\left\{ W_{c_f}^2 + \left[W_{c_i}(1 - E)\right]^2 + \left(B_c\, E\right)^2 + \left(B_{c_s}\, E\right)^2 \right\}^{1/2}}{C_s - C_i} \tag{10}$$

or

$$\frac{U_E}{E} = \frac{\left\{ \left(W_{c_f}/E\right]\right)^2 + \left[W_{c_i}(1 - E)/E\right]^2 + B_c^{\,2} + B_{c_s}^{\,2} \right\}^{1/2}}{C_s - C_i} \tag{11}$$

where

U_E = total uncertainty in E

W_{c_i}, W_{c_f} = precision uncertainties in C_i and C_f, respectively

B_c = bias uncertainty in the measurement of C_i and C_f

B_{c_s} = bias uncertainty in C_s

Equation 10 shows that the uncertainty in E is inversely related to the initial (upstream) deficit. The typical precision uncertainty for well-calibrated DO meters is 0.1 mg/ℓ; thus, it can be assumed that W_{c_i} and W_{c_f} are 0.1 mg/ℓ at the 95 percent confidence interval ($P = 0.95$). The variation in time of DO concentration at a given location can result in a W_{c_f} or W_{c_i} value that is somewhat larger. The last two terms of Equation 10 represent the bias uncertainties that need to be considered. These uncertainties represent potential error that would be identical in every measurement at that structure, or that day, in terms of DO meter calibration and the uncertainty in the value of saturation concentration. A typical bias uncertainty for DO probe calibration would be ±0.1 (P = 0.95). In experiments on river water sampled during midwinter in Minnesota, Rindels (1990) found that the DO saturation concentration was 98 percent of that for distilled water, with an uncertainty of ±2 percent. Thus, Rindels found that $B_{C_s} = 0.02C_s$. During summer, because of the decreased water quality, B_{C_s} may be higher, say $B_{C_s} = 0.03C_s$. This is the primary source of uncertainty in many measurements of transfer efficiency.

33. As mentioned at the beginning of this section, uncertainty analysis can be used in several types of measurements. Consider the following question, which would be posed prior to a field study: How much DO deficit must exist to accurately assess the oxygen transfer characteristics of a hydraulic structure? Uncertainty analysis provides the framework for establishing those minimum criteria for in situ measurements of DO. For example, if the oxygen saturation value C_s is 8.0 mg/ℓ, the expected efficiency is 50 percent ($D_f = 0.5D_i$) , a 3 percent bias uncertainty in C_s exists, and it is decided that the uncertainty in E must be less than 10 percent ($U_E < 0.30E$) to use in this analysis, then the results of applying Equation 10 are that D_i must be greater than 3.4 mg/ℓ. If these conditions exist for this study, e.g., $C_s = 8.0$ mg/ℓ , measured E = 0.50 , $D_i = 3.4$ mg/ℓ , then the measured efficiency would be 0.50 (50 percent) ±0.05 (or 10 percent of E) for a confidence interval of 95 percent. If the upstream deficit is less than 2.5 mg/ℓ, then the assessment of the gas transfer characteristics of the structure should not be based on the measurement of oxygen concentrations. An alternative method using a tracer gas should be considered.

34. After a field study, the uncertainty surrounding the observed data should be evaluated, as detailed in Appendix A, so that some measure of confidence can be defined about the study. For an upstream oxygen concentration of 3 mg/ℓ, a downstream concentration of 5.5 mg/ℓ, and saturation of 8 mg/ℓ, the efficiency determined with these data from Equation 10 is given as E = 0.5 ± 0.034 (P = 0.95). The uncertainty here is approximately 7 percent of the measured efficiency. Obviously, the uncertainty in E becomes smaller if the upstream deficit is larger. Hence, it is easy to conclude that there will be

less uncertainty if measurements are made when the magnitude of the upstream deficit is maximum.

35. The measurement uncertainty technique can be applied to any type of measurement, if the uncertainty of each part can be defined. In situ measurements of DO to determine efficiency were used as an illustration, but all the measurement techniques discussed may be analyzed in this fashion, using Equations A1, A2, and A3.

PART IV: HYDRAULICS AT VARIOUS STRUCTURES

36. Low-head structures range widely in purpose, size, and configuration. Consequently, the hydraulics that are encountered at the variety of structures are extremely varied. However, the processes identified in Part II are still the major influences on gas transfer. An examination of the hydraulic conditions at "generic" structures can reveal the impact of these processes and permit qualitative conclusions to be drawn regarding the relative gas transfer characteristics of these structures.

37. A committee evaluation of gas transfer at hydraulic structures (American Society of Civil Engineers (ASCE) 1991) categorized open-channel structures into four groups: (a) free-surface flows, such as flow in a channel or on a spillway or ogee crest without a tailwater plunge pool (Figure 3); (b) sub-merged flows, such as discharge under a submerged gate (Figure 4); (c) free jets, such as flow over a sharp-crested weir (Figure 5); and (d) transitional flows, where free-surface flows or jets interact with a pool of water resulting in plunging flow or a hydraulic jump (Figures 6 and 7). The hydraulics of each group differ greatly resulting in significantly different gas transfer characteristics. An understanding of the hydraulics and reaeration character of these groups of structures can permit extrapolation to more complex flow situations.

38. Free-surface flows, such as shown in Figure 3, generate a boundary layer along the spillway surface. If the length of flow is sufficient, then the turbulence of this boundary layer will intersect the free surface. Under these conditions, air is captured by the highly turbulent surface and sheared down to small bubbles, giving the flow a white appearance. As the entrained air is carried along the spillway, the bubbles are transported downward through the water column by turbulence. As a consequence of this entrained air, the surface area available for gas transfer greatly increases compared to spillway flows that do not experience free-surface aeration. Gulliver, Thene, and Rindels (1990) estimated that the surface area of the bubbles in a 1-ft-deep spillway flow can provide 500 times the area of air/water interface provided by the free surface. At overflow crests where air entrainment occurred in the free-surface flow, Rindels and Gulliver (1989) observed oxygen transfer efficiencies as high as 30 percent, which represents approximately half of the oxygen absorption occurring at the project.

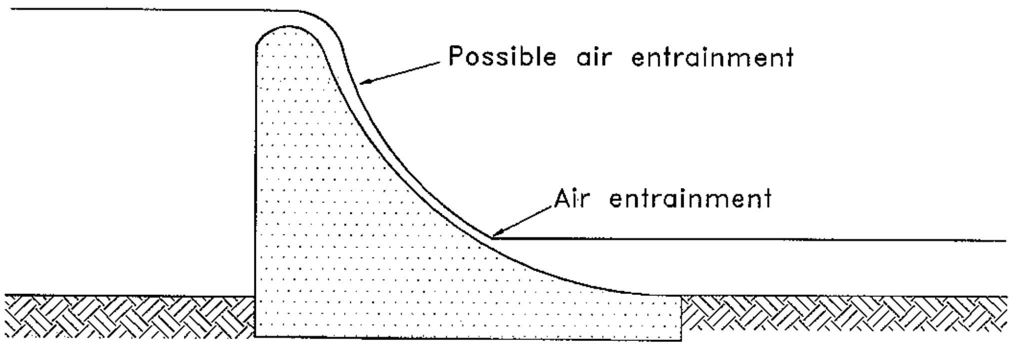

Figure 3. Open channel flow on a spillway (after ASCE 1991)

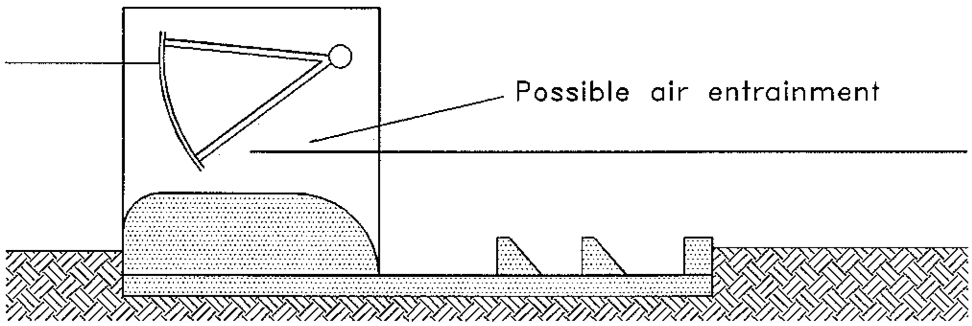

Figure 4. Open channel flow at gated structure (gated sill) (after ASCE 1991)

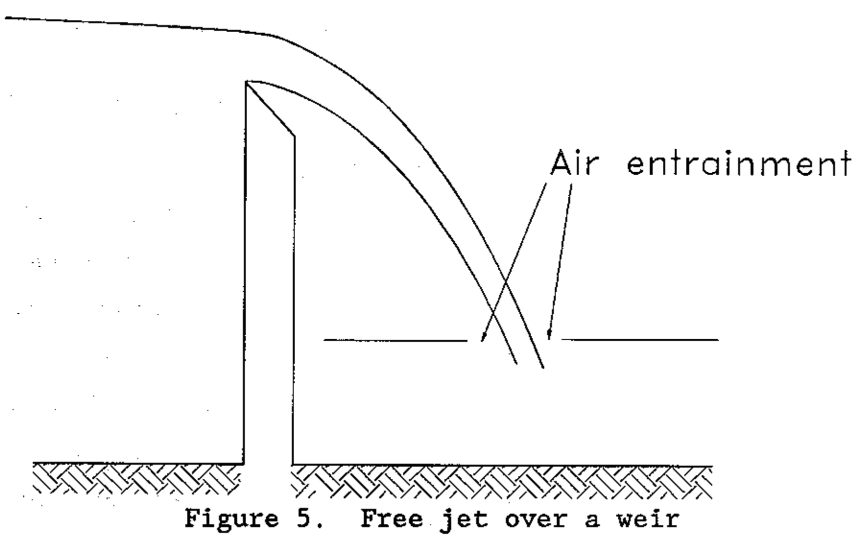

Figure 5. Free jet over a weir (after ASCE 1991)

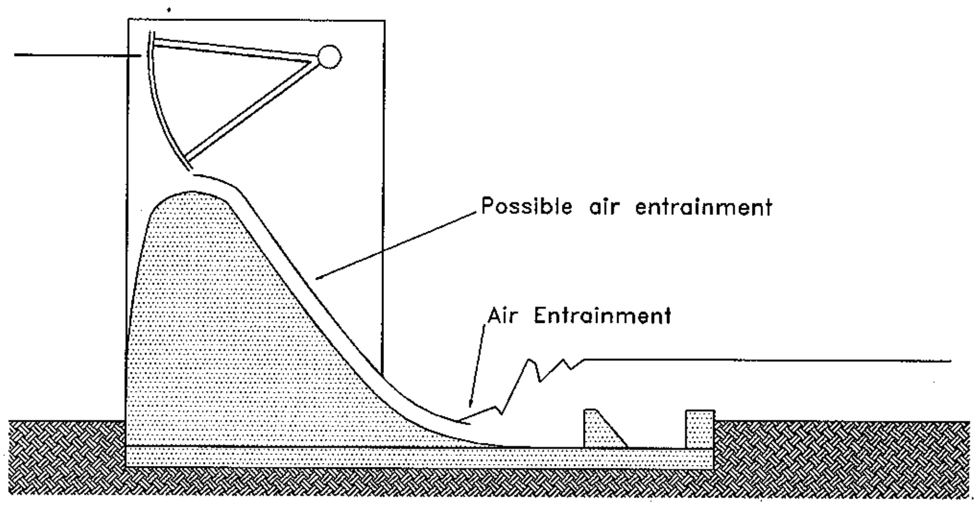

Figure 6. Transitional flows through a gated spillway (after ASCE 1991)

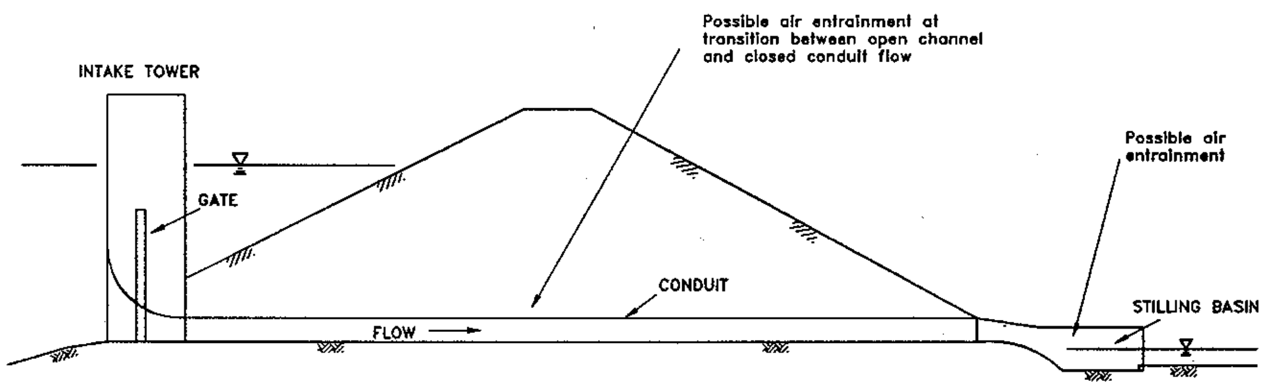

Figure 7. Transitional flows through a gated conduit

39. Submerged discharges (Figure 4) usually reaerate flow through oxygen absorption at the water surface of the turbulent "boil" in the stilling basin of the structure. Generally, there is little air entrainment as long as the discharge remains submerged. As a consequence, the reaeration that occurs in these hydraulic conditions is relatively small with oxygen absorption efficiencies usually below 10 percent (Wilhelms 1988; Thene, Daniil, and Stefan 1989). However, for many structures like the one shown in Figure 4, with higher discharges there is sufficient momentum in the jet issuing from under the gate to push the tailwater downstream and expose the jet to the atmosphere. This situation significantly alters the hydraulic characteristics of the structure. The flow conditions may more closely resemble those shown in Figure 6, where a high-velocity jet interacts with a pool of water causing a

hydraulic jump. When this occurs, large volumes of air are usually entrained at the velocity discontinuity between the jet and the static pool. With the entrained air bubbles and possibly hydrostatic pressure on those bubbles, oxygen absorption significantly increases to efficiencies as high as 40 percent (Wilhelms 1988; Thene, Daniil, and Stefan 1989).

40. Free jets, such as shown in Figure 5, appear to be similar to the free-surface flow on a spillway. These flows, however, do not demonstrate the free-surface air entrainment that results from the full development of the boundary layer at the spillway surface. Typically, reaeration is accomplished at this type of structure during the breakup of the jet when it collides with the bottom downstream. If the free jet plunges into a receiving pool, air entrainment and turbulent mixing contribute to increasing gas transfer. Further, the depth of the plunge pool can enhance the absorption because of the increased hydrostatic pressure on the entrained air bubbles. Avery and Novak's (1978) experiments indicate that the transfer efficiency increases with a tailwater depth up to 0.6 times the drop height. Oxygen absorption efficiencies vary widely, but for low-head overflow weirs, efficiencies of up to 70 percent have been measured.

41. The most complex situations occur in open channel transitional flows, which are typical of many hydraulic structures with stilling basins. Much of the time, these situations are combinations of several of the hydraulic conditions described in the preceding paragraphs. For example, this may include air-entrained flow over a crest that plunges into a tailwater; the hydraulic conditions when discharges from a gated sill sweep the tailwater downstream and expose the flow to the atmosphere, resulting in air entrainment; or the variety of flow conditions encountered in a gated conduit outlet. Obviously, the hydraulic action of these flow conditions is complex and extremely varied. Consequently, the reaeration characteristics of these flows are extremely varied with oxygen transfer efficiencies ranging up to nearly 100 percent.

PART V: FIELD DATA AND PREDICTION OF REAERATION

42. Qualitative comparisons can be made of the reaeration that occurs under various flow conditions by considering the significance of the major processes that affect gas transfer. Air entrainment usually results in greater gas transfer, as does plunging aerated flow. Generally, structures with greater differences between pool and tailwater elevations demonstrate greater gas transfer, i.e., there is more energy available to cause gas transfer. Through observations of oxygen absorption or tracer gas transfer at these structures, the reaeration characteristics of the structure may sometimes be parametrically described. This type of analysis results in mathematical relationships between gas transfer and the measurable parameters that are descriptive of the significant processes at work. The following sections review most of the efforts in this area, including data presentation, uncertainty analysis, and an evaluation of predictive capability of various mathematical descriptions.

Field Data

43. Parameter measurement and subsequent calculations with observed data constitute crucial initial steps in practically all engineering applications, and reaeration studies are no exception. While the amount of information needed to complete a thorough gas transfer study at a hydraulic structure is not large, measures of confidence are important and the collection of data is not trivial (Rindels and Gulliver 1989). Prediction equations also require special consideration since each uses a unique set of variables and special measurements are sometimes necessary. This section describes how data used to evaluate the predictive equations for gas transfer at hydraulic structures have been gathered and analyzed.

44. The entire raw data set used in the analyses reported herein is presented in Appendix B, which was created by reviewing current literature and includes numerous parameters, all in SI units. The following parameters are cited·

- a. The location and type of the hydraulic structure.
- b. The date the sampling was made.
- c. The flow per unit width over the structure.

d. The difference in elevation upstream and downstream of the structure (the head loss)

e. The depth of the tailwater.

f. The gate openings if applicable.

g. The concentration of dissolved oxygen both upstream and downstream of the structure.

h. The temperature of the water.

i. The saturation concentration of oxygen in the water.

j. The barometric pressure at or near the site.

45. When a report failed to include the saturation concentration of oxygen C_s the value was found using the equation of Hua (1990):

$$C_s = \exp\left\{-17.015355 + 0.0226297 * T + \left(\frac{3689.38}{T}\right) + \left[0.01166 - \left(\frac{6.544}{T}\right)\right] C_{Cl}\right\} \quad (12)$$

where T is temperature in degrees kelvin and C_{Cl} is concentration of chloride ions in grams per litre. Chloride concentration is normally not part of the data collected at hydraulic structures because for concentrations under 1 g/ℓ, the presence of chloride has little impact on saturation concentrations. However, most natural streams contain some chloride ions, and particularly near maritime environments, atmospheric deposition of chloride can cause error in estimations of oxygen saturation. For the purposes of this report, C_{Cl} was assumed in all cases to be 1 g/ℓ.

46. The equation of Hua expresses the saturation concentration of oxygen in distilled water at an atmospheric pressure of 760 mm mercury. The value was adjusted for the following:

a. Water quality by estimating the value of C_s to be 0.97 of the distilled water value (Rindels 1990).

b. The barometric pressure with

$$C_s\left(P_{atm}\right) = C_s\left(760 \text{ mm Hg}\right)\frac{P_{atm}}{760} \quad (13)$$

where P_{atm} is the reported pressure in millimetres of mercury. If barometric pressure was unavailable, then the saturation concentration was adjusted for the elevation of the structure above sea level by employing the following equation:

$$\text{Elevation Correction} = \left[\frac{1 - (0.0065M)}{298}\right]^{5.25} \tag{14}$$

where M is elevation above mean sea level in metres.

47. The processed data for comparison with the predictive equations are presented in Appendix C. This appendix lists several of the previous parameters that are used in predictive equations, in addition to values of transfer efficiency (Equation 8) and uncertainty (Equation 10). The efficiencies provided by the literature were commonly calculated at a variety of different temperatures. For comparison, these efficiencies were indexed to a common temperature (20 °C) using the equation of Gulliver, Thene, and Rindels (1990):

$$E_{20} = 1 - (1 - E_T)^{\frac{1}{f_T}} \tag{15}$$

where E_{20} and E_T are gas transfer efficiencies at 20 °C and at T °C and f_T is given by the equation:

$$f_T = 1.0 + 0.02103(T - 20) + 8.261 * 10^{-5}(T - 20)^2 \tag{16}$$

where T is temperature in degrees Celsius.

48. The uncertainty listed in Appendix C is the cumulative uncertainty to the 95 percent confidence interval associated with the gas transfer efficiency. It is calculated by Equation 10 or a similar equation, using the principles outlined in Appendix A. The bias uncertainty in saturation concentration accounts for the fact that the saturation for DO is not precisely established in field situations. Rindels (1990) found that during midwinter this bias was approximately ±2 percent. For the general data sets, however, and accounting for the degraded water quality of the growing season, the bias in C_s was taken to be ±3 percent. The calibration bias for DO meters was taken to be ±1 percent.

49. The maximum acceptable value of uncertainty U was set at 0.25 for values of transfer efficiency to be included in the final data set for comparison with the predictive equations. Most of the literature did not contain uncertainty analyses of the field data; therefore, appropriate approximations were made in calculating U .

Predictive Equations

50. As discussed at the beginning of this section, predictive equations that have been developed by various researchers for gas transfer at hydraulic structures generally use physical parameters of the structure or flow conditions, i.e., Froude number, depth of tailwater, discharge, etc., to estimate reaeration efficiency. These equations are all more or less empirical, with intuition being used to specify the independent variables, and regressions used to determine the constants in the equation. Each equation yields a unique oxygen transfer efficiency, and it is difficult to know the correct efficiency for a different site without performing a comprehensive field measurement investigation. The observed data discussed in the previous section and presented in Appendix B are used in a comparison with the predictive equations. Predicted and measured oxygen transfer efficiencies are compared to determine the performance of the various predictive equations. It should be recognized that predictive equations are reliable only as first approximations, and that they should be used carefully only when more accurate and detailed studies are not feasible.

51. The equations investigated and the references in which they may be found are listed in Table 1. A more thorough explanation on the development of these equations is presented in Appendix D.

52. Each of these equations is derived from a different data set, with coefficients adjusted to fit the specific data of the researcher. Because the size of most data sets is relatively small, large deviations between predicted values of gas transfer efficiency and measured values at different sites are common. Appendix D shows plots of predicted versus measured oxygen transfer efficiency, which illustrate this.

53. Each of the equations in Table 1 was used to predict an oxygen transfer efficiency for all of the acceptable measured data ($U < 0.25$) found in Appendix C. This was performed separately for each of the four categories

Table 1

Predictive Equations of Gas Transfer at Hydraulic Structures

Hydraulic Structure	Predictive Equation	Source
Sharp-Crested Weirs (Figure 5)	$E_{20} = 1 - \left(\frac{1}{1 + 0.64 \times 10^{-4} F_j^{1.787} R^{0.533}} \right)^{1.1149}$	Avery and Novak (1978)
	for $(h + 1.5H_c) > 1.2$ (m) and $q \le 235$ $E_{20} = 1 - \exp -[0.0861 (h + 1.5H_c)^{0.8.16} q^{0.428} H^{0.310}]$	Nakasone (1987)
	for $(h + 1.5H_c) \le 1.2$ (m) and $q > 235$ $E_{20} = 1 - \exp -[5.39 (h + 1.5H_c)^{1.31} q^{-0.363} H^{0.310}]$	
	for $(h + 1.5H_c) > 1.2$ (m) and $q > 235$ $E_{20} = 1 - \exp -[5.92 (h + 1.5H_c)^{0.816} q^{-0.363} H^{0.310}]$	
	for $(h + 1.5H_c) \le 1.2$ (m) and $q \le 235$ $E_{20} = 1 - \exp -[0.0785 (h + 1.5H_c)^{1.31} q^{0.428} H^{0.310}]$	
	$E_{20} = 1 - \exp -\left\{0.156\, N_f^{2.69} \frac{q}{2gh} \left[1 - \frac{1.1}{(2gh)^{0.5}}\right]^{-1}\right\}$	Thene (1988)

(Continued)

Table 1 (Continued)		
Hydraulic Structure	Predictive Equation	Source
	$E_{20} = \left\{\frac{1}{1 + 1.005 \times 10^{-5} F_j^{2.08} R^{0.63} \left[1 - 0.6 \exp\left(-3.7 \frac{H}{h}\right)\right]}\right\}^{f_t}$	Thene (1988)
Ogee Crest Spillways (Figure 3)	$E_{20} = \frac{0.21325h}{(0.21325h + 1)}$	Holler (1970)
	$E_{20} = 1 - \exp\left(\frac{-0.2625h}{1 + 0.2153q} - 0.2034H\right)$	Rindels and Gulliver (1991)
Gated Sills and General Hydraulic Structures (Figure 4)	$E_{20} = 1 - [\exp(-0.5249h)]^{0.9032}$	Foree (1976)
	$E_{20} = 1 - \exp(-0.1772h)$	Tsivoglou and Wallace (1972)
	$E_{20} = 1 - \frac{1}{\left(1 + 666 N_f^{-3.33}\right)}$	Preul and Holler (1969)
	$E_{20} = \exp\left[-0.00857884 \frac{hq}{S} - 0.188\right]$	Wilhelms (1988)
Gated Conduits (Figure 7)	$E_{20} = 1 - \exp(-0.1476h)$	Wilhelms and Smith (1981)
where F_j = Froude number of the jet		
(Continued)		

Table 1 (Concluded)		
Hydraulic Structure	Predictive Equation	Source
	$$F_j = \frac{(2g)^{0.25}h^{0.75}}{q^{0.5}}$$ g = acceleration due to gravity, m/sec^2 h = head loss, m q = specific discharge, $m^3/sec/m$ R = Reynolds number $$R = \frac{q}{2\nu}$$ ν = kinematic viscosity H_c = critical water depth on the weir, m H = tailwater depth, m N_f = Froude number at the point of impact $$N_f = \frac{(2gh)^{0.75}}{(gq)^{0.5}}$$ S = gate submergence, m	

of hydraulic structures (an ogee crest spillway, gated sill, weir, or gated conduit) regardless of which type of structure the predictive equation was developed for. The statistical results are given in Table 2. The standard error compares measured and predicted oxygen transfer efficiencies by the equation

$$SE = \sqrt{\frac{(E_m^2 - E_p)^2}{n}} \tag{17}$$

where E_m and E_p are measured and predicted oxygen transfer efficiency, respectively. The standard error is approximately equal to the uncertainty U to the 68 percent confidence interval. A 68 percent confidence interval may be interpreted as meaning that two out of three predicted values will be within the boundaries of ±U. An uncertainty to the 95 percent confidence

Table 2 Summary of Statistical Results				
	Standard Error			
Formula Title	Ogee	Gated Sill	Weir	Gated Conduit
Avery and Novak (1978)	0.282	0.458	0.166	0.340
Thene's Adjustment to Avery and Novak (1988)	0.297	0.451	0.170	—
Preul and Holler (1969)	0.647	0.141	0.615	0.690
Thene (1988)	0.302	0.324	0.174	0.420
Nakasone (1987)	0.267	0.487	0.172	—
Tsivoglou and Wallace (1972)	0.290	0.406	0.183	0.320
Foree (1976)	0.285	0.612	0.271	0.358
Rindels and Gulliver (1991)	0.160	0.463	0.210	—
Holler (1970)	0.327	0.296	0.205	0.339
Wilhelms (1988)	0.227	0.247	0.360	—
Wilhelms and Smith (1981)	0.322	0.355	0.212	0.312

interval (such as that used for the measurements) may be approximated as double the uncertainty of the 68 percent confidence interval, or:

$$U_{E(P=0.95)} = 2U_{E(P=0.68)} \tag{18}$$

where P is the probability of the confidence interval on efficiency.

54. Using the method described herein, the prediction of a transfer efficiency will result in:

$$E = \hat{E} \pm U_{E(p=0.68)} \tag{19}$$

where

E = true value of transfer efficiency that the given equation is designed to predict

$\hat{E}$ = the best estimate of the true value of transfer efficiency that is predicted by the equation

$U_{E(p=0.68)}$ = an uncertainty to the 68 percent confidence interval ($p = 0.68$) for the prediction

55. While specific equations may provide the best approximation for reaeration efficiencies at individual sites, the equation that minimizes predicted uncertainty provides the best estimate for gas transfer at a generic hydraulic structure. The results show that the equation of Avery and Novak (1978) best estimates the transfer efficiency for weirs. Rindels and Gulliver's (1989) equation is most accurate for gated and ungated ogee spillways. The equation of Preul and Holler (1969) predicts reaeration at gated sills most accurately. The equation of Wilhelms and Smith (1981) best describes the reaeration of gated conduits, although none of the equations predicted the process accurately. This is probably because of the transitional characteristics of the flow in the conduit (high-velocity open channel flow to plunging flow or hydraulic jump open channel versus closed conduit flow), and the lack of consistency between structures.

PART VI: SUMMARY AND CONCLUSIONS

56. Gas transfer or reaeration is considered a first-order reaction process and can be conveniently described by

$$E = \frac{(C_f - C_i)}{(C_s - C_i)} = 1 - \frac{D_f}{D_i} = 1 - \exp\left(-k_L \frac{A}{V} t\right) \qquad \text{(8 bis)}$$

The important physical processes that impact gas transfer are included in this formulation in the following manner: The effect of turbulent mixing is reflected by the liquid film coefficient k_L . The impact of surface area available for gas transfer, which must include the surface area of entrained air bubbles, is represented by the specific area term A/V . Enhanced gas absorption due to the effects of hydrostatic pressure on plunging aerated flow is included as a pressure modification of the saturation concentration C_s . The time of contact over which gas transfer can occur is t .

57. The unknowns in this equation often dictate that reaeration measurements be conducted to determine the gas transfer efficiency. The transfer efficiency can then be related to the physical processes through empirical relationships and regression analyses. Several alternatives are available for measurement of gas transfer. Direct in situ measurement of dissolved oxygen with polarographic probes is usually the most convenient. However, based on uncertainty analysis, a DO deficit of at least 2.5 mg/ℓ (when saturation is 8.0 mg/ℓ) is required for accurate analysis. Often, the DO is not sufficiently low to permit an accurate analysis. When this is the situation, alternative gas transfer tracers must be used in measurements for gas transfer. In situ methane gas shows the highest potential for general application in reaeration field studies.

58. Generally, low-head structures can be categorized into four groups: (a) free-surface flows, such as flow in a channel or on a spillway or ogee crest without a tailwater plunge pool; (b) submerged flows, such as discharge under a submerged gate; (c) free jets, such as flow over a sharp-crested weir; and (d) transitional flows, where free-surface flows or jets interact with a pool of water resulting in plunging flow or a hydraulic jump. The hydraulics of each group differ dramatically and, consequently, the gas transfer characteristics are significantly different. However, an understanding of the

hydraulics and the resulting gas transfer characteristics of each type of structure can permit limited extrapolation to other hydraulic structures. The observations reported herein show that gated and ungated ogee crests demonstrate gas transfer efficiencies up to nearly 100 percent. Submerged flows without air entrainment can result in efficiencies up to 40 percent. Sharp-crested overflow weirs or jets demonstrate efficiencies up to 70 percent. Transitional flow conditions, which include several or all of the other types of flow conditions, have shown reaeration efficiencies of up to nearly 100 percent.

59. Several sources of observed data were assembled and, after screening through an uncertainty analysis, are included in the appendices of this report. Predictions from eleven equations that describe gas transfer efficiency at various types of structures were compared to this comprehensive database. Although there are inadequacies in the models currently used for prediction of gas transfer, the equations listed in Table 3 were the most successful for given types of structures.

Table 3

Suggested Predictive Equations

Structure Type	Predictive Equation	Standard Error	Source
Ogee Crests	$E_{20} = 1 - \exp\left(\frac{-0.2625h}{1 + 0.2153q} - 0.2034H\right)$	0.16	Rindels and Gulliver (1991)
Gated Sills	$E_{20} = 1 - \frac{1}{\left(1 + 666N_f^{-3.33}\right)}$	0.14	Pruel and Holler (1969)
Sharp-crested Weirs	$E_{20} = 1 - \left(\frac{1}{1 + 0.64 \times 10^{-4} F_j^{1.787} R^{0.533}}\right)^{1.1149}$	0.17	Avery and Novak (1978)
Gated Conduit Outlets	$E_{20} = 1 - \exp(-0.1476h)$	0.31	Wilhelms and Smith (1981)

60. Standard error estimations for the equations of Preul and Holler (1969) and Rindels and Gulliver (1991) may be too low. Much of the sill and ogee spillway database included in this report was used in the regression of the two equations, and the paucity of accurate alternative data sources closely tailors these equations to this evaluation. While the compilation of accurate data is not simple, further field investigations are needed to define the performance of predictive equations.

61. In their efforts to predict the impact of hydraulic structures on levels of dissolved gases in river systems, many researchers sought to create equations that provide accurate estimates of transfer efficiency. Unfortunately, the development of each equation was limited by the size of the database that a researcher had available and the difficulty encountered in deriving a theory from first principles. As a result, many equations are useful only for specific types of structures under particular conditions, i.e., the domain of accuracy for the parameters used in the equation is relatively small. The relatively large deviations between the measured transfer efficiencies and the values computed from the predictive equations are understandable because of the size and diversity of the observed data. Consequently, a large uncertainty is associated with the prediction of an oxygen transfer efficiency from these equations. Field measurements are still the most consistent means of determining oxygen transfer characteristics.

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APPENDIX A: TYPICAL MEASUREMENT UNCERTAINTY ANALYSIS FOLLOWING THE EVALUATION TECHNIQUE OF RINDELS AND GULLIVER (1989)

1. Measurement uncertainty inadvertently occurs due to the inaccuracy of the instruments or of the measurement or because of operator error. In any measurement procedure, all significant uncertainties should be quantified, to provide a means of evaluating the quality of the procedure and resulting data. During the course of the measurements, sources of uncertainty should be identified. For example, in the chemical titrametric technique for determining dissolved oxygen (DO) concentration, the purity of the chemical titrant in the Winkler technique and the minimum gradations on the titration buret will contribute to the uncertainty of the measured DO concentrations and to the uncertainty in transfer efficiency. In the use of polarographic probes for measuring DO concentration, uncertainty occurs because of the accuracy and calibration of the instrument.

2. From Equations 8 and 9 in the main text, transfer efficiency E* was defined as

$$E = 1 - \frac{C_s - C_f}{C_s - C_i} = \frac{C_f - C_i}{C_s - C_i} \tag{A1}$$

where C_s , C_i , and C_f are the saturation and upstream (initial), and downstream (final) DO concentrations, respectively. The total uncertainty in the transfer efficiency U_E is inherent in each measurement. By definition, the total uncertainty of any measurement is a combination of precision (random) uncertainty introduced when measurements are repeated, and bias (systematic) uncertainty, or possible error that would affect each measurement in the same manner (Abernethy, Benedict, and Dowdell 1985).** The most common technique of analyzing measurement uncertainties is a first order-second moment analysis (Kline 1985). With this uncertainty technique, the total uncertainty of transfer efficiency U_E can be expressed as

$$U_E^2 = W_E^2 + B_E^2 \tag{A2}$$

* For convenience, symbols and unusual abbreviations used in this appendix are listed and defined in the Notation (Appendix E).

** References cited in this appendix can be found in the References at the end of the main text of the report.

where

W_E = precision uncertainty in the transfer efficiency

B_E = bias uncertainty in the transfer efficiency

3. The precision uncertainty W_E is a combination of the sampling uncertainties that result from determining C_i and C_f and is mathematically described by the following formula:

$$W_E^2 = \left(\frac{\partial E}{\partial C_f} W_{C_f}\right)^2 + \left(\frac{\partial E}{\partial C_i} W_{C_i}\right)^2 = \left(\frac{W_{c_f}}{C_s - C_i}\right)^2 + \left[\frac{(1 - E)\, W_{c_i}}{C_s - C_i}\right]^2 \qquad \text{(A3)}$$

where W_{C_i} and W_{C_f} are the uncertainties associated with the upstream and downstream DO measurements, respectively. According to Abernethy, Benedict, and Dowdell (1985), W_{C_i} and W_{C_f} can be calculated as follows:

$$W_{C_i} = \frac{t^* \, \sigma_{C_i}}{\sqrt{n_{C_i}}} \qquad \text{(A4)}$$

$$W_{C_f} = \frac{t^* \, \sigma_{C_f}}{\sqrt{n_{C_f}}} \qquad \text{(A5)}$$

where

t^* = Student's t-value corresponding to $n_{C_i} - 1$ or $n_{C_f} - 1$ degrees of freedom and a 95 percent confidence interval, as given in Table A1

$\sigma_{C_i}, \sigma_{C_f}$ = standard deviation of the upstream and downstream measurements, respectively

n_{C_i}, n_{C_f} = number of upstream and downstream measurements, respectively

For the given example of analysis with the titrametric method, W_{C_i} or W_{C_f} should be assigned a minimum value of 0.05 mg/ℓ, which is the uncertainty associated with reading the buret used for titrations. If W_{C_i} or W_{C_f}, as calculated with Equation A3, is greater than 0.05 mg/ℓ, then their respective calculated values would be used in determining the precision uncertainty W_E from Equation A3. For measurements with a DO meter, W_{C_i} or W_{C_f} should

Table A1
Student's t-Values at a 95 Percent Confidence Interval for Various Degrees of Freedom ($n_c - 1$)

Degree of Freedom	Student's t-Value
1	12.7
2	4.30
3	3.18
4	2.78
5	2.57
6	2.45
7	2.37
8	2.31
9	2.26
10	2.28
15	2.13
20	2.09
30	2.04

have, as minimum values, the stated accuracy of the DO meter, which is assumed to be the uncertainty at the 95 percent confidence interval. For most probes, this is usually 1 percent of full scale or 0.1 mg/ℓ, whichever is larger.

4. Two bias uncertainties should also be included in the uncertainty: (a) either the purity of the chemical titrant in the Winkler technique or the uncertainty in the calibration of the DO meter and (b) the uncertainty in the value of saturation concentration. The supplier of the sodium thiosulfate titrant guarantees the purity to within ±1 percent. This value could be assumed to be the uncertainty at the 95 percent confidence interval. Proper air calibration of a DO probe, considering air pressure and relative humidity, usually has a typical bias uncertainty of ±0.1 ($P = 0.95$). The saturation concentration of the water at a given hydraulic structure was estimated using the locally measured atmospheric pressure, before and after sampling, and the water temperature measured at the hydraulic structure. At several hydraulic structures, Rindels (1990) found the saturation concentrations to be less than the published values for a distilled water at a given temperature. He

estimated the actual saturation value to be 98 percent of the distilled water value with an uncertainty of ±2 percent for river water collected during winter. Summertime values of the same water have been assumed to be 97 percent of the distilled water value with an uncertainty of ±3 percent. In situ saturation measurement techniques, currently being developed, should reduce this uncertainty considerably.

5. These two bias uncertainties are added to give the total bias uncertainty in E as follows:

$$B_E^2 = \left(\frac{\partial E}{\partial C_s} B_{C_s}\right)^2 + \left(\frac{\partial E}{\partial C} B_C\right)^2 = \left(\frac{B_{C_s} E}{C_s - C_i}\right)^2 + \left(\frac{B_c\ E}{C_s - C_i}\right)^2 \tag{A6}$$

where B_{C_s}, the bias uncertainty in C_s, ranges from 0.02 C_s to 0.03 C_s; and B_C is the bias uncertainty in DO concentration due to the titrant or DO probe calibration

$$B_C = \alpha\ C_s \tag{A7}$$

where α equals 0.02 for the titrant and 0.01 for the probe.

6. Substituting the expressions for precision uncertainties (Equations A4 and A5) into Equation A3 gives a mathematical description of the total precision uncertainty in E. Substituting the expressions for bias uncertainty due to saturation concentration and titrant impurity or probe calibration into Equation A6 gives a mathematical description of the total bias uncertainty in E. Combining these in Equation A2, the uncertainty in the determination of transfer efficiency is described by

$$U_E^2 = \left(\frac{W_{C_f}}{C_s - C_i}\right)^2 + \left[\frac{W_{C_i}(1 - E)}{(C_s - C_i)}\right]^2 + \left(\frac{E}{C_s - C_i}\right)^2 + \left(B_{C_s}^2 + B_C^2\right) \tag{A8}$$

or

$$\frac{U_E}{E} = \frac{1}{C_s - C_i} \left\{ \left(\frac{W_{C_f}}{E} \right)^2 + \left[\frac{W_{C_i}(1 - E)}{E} \right]^2 + B^2_{C_s} + B^2_C \right\}^{1/2} \tag{A9}$$

7. An examination of Equations A8 and A9 clearly shows that the accuracy of the transfer efficiency is strongly dependent on the difference between saturation and the measured upstream DO concentration (the upstream oxygen deficit). Hence, the recommendation was made in paragraph 33 for minimum upstream deficits for acceptable uncertainty in transfer efficiency with measurements of DO. In fact, larger oxygen deficits result in a reduced measurement uncertainty.

APPENDIX B: OBSERVED DATA TAKEN FROM LITERATURE

1. For some references all the information that was needed to calculate important data was not included in the publications. In these cases allowances had to be made to extrapolate the information from different sources. For instance, if the barometric pressure was not included, then (on data taken within the United States) the nearest National Weather Service (NWS) recording station was found and barometric pressure was taken from NWS publications. The pressure was then converted to the elevation of the structure, because NWS pressure readings have all been converted to mean sea level before being published.

2. Barometric pressure was not reported for these structures and was gathered from NWS records.

Dresden Island Dam, Joliet, IL
Brandon Rock Dam, Joliet, IL
Starved Rock Dam, Peoria, IL
Arkabutla Dam, Arkabutla, MS
Crooked Creek Dam, Ford City, PA
East Branch Dam, Johnsonberg, PA
East Lynn Dam, Wayne, IN
Enid Dam, Enid, MS
Grayson Dam, Grayson, KY
Grenada Dam, Grenada, MS
Mississinewa Dam, Peru, IN
Salamonie Dam, Largo, IN
Sardis Dam, Sardis, MS
Dams listed in Butts and Evans (1983)*

3. Although pressure was not reported for some structures, the saturation concentration given in the references was assumed to be the best available estimate for the following structures:

Borgharen Weir, Netherlands
Lith Weir, Netherlands
Cascade, Netherlands
All the other structures in Nakasone (1979)

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* References cited in this appendix can be found in the References at the end of the main text of the report.

All of the structures listed by Foree (1976)

4. An "equivalent" specific discharge for all of the gated conduits was calculated from hydraulic radius considerations by using Fig. 6-5 of Chow (1959) for a fully developed velocity profile in a partially full circular conduit. Then:

$$q = \frac{Q}{R_h} \tag{B1}$$

where

q = specific discharge through the structure

Q = total discharge through a structure

R_h = the hydraulic radius of the flow in the conduit.

For the Japanese dams listed by Nakasone (1979), a mean elevation of 550 ft was assumed, and a typical air pressure at mean sea level was determined by inspecting NWS records. The measurement uncertainty associated with all of the structures was greater then $U_E = 0.25$, so none were included in the comparison with predicted equations.

Table B1

Data Available in the Literature That Were Suitable for Comparison with Predictive Equations

Location	Type	Date of Sample	Discharge m^3/sec/m	Head Loss, m	Tailwater Depth, m	Gate Opening	DO Measured Upstream of Dam	DO Measured Downstream of Dam	Temperature °C	Saturation Concentration mg/l	Barometric Pressure mm Hg	Publication
Kost Dam, Chisago County, MN	Ogee	2/02/85	0.13	4.01	0.24	N/A	7.36	10.17	0.20	14.21	745.20	Rindels and Gulliver 1989
		3/12/85	0.14	3.97	0.34	N/A	10.49	11.59	1.80	13.31	744.10	
		12/13/85	0.16	3.95	0.34	N/A	8.67	10.53	0.20	13.95	746.30	
		1/13/86	0.10	3.97	0.24	N/A	7.95	9.82	0.10	13.94	729.40	
		2/21/86	0.08	3.94	0.30	N/A	7.33	9.05	0.30	13.75	729.00	
		3/10/86	0.11	3.97	0.27	N/A	9.26	10.51	0.20	13.77	735.40	
St. Cloud Dam, St. Cloud, MN	Ogee	3/02/85	0.68	5.32	0.24	N/A	9.30	11.66	0.70	13.81	748.00	Rindels and Gulliver 1989
		3/14/85	0.83	5.35	0.27	N/A	9.46	11.75	1.00	13.61	744.40	
		12/19/85	1.10	4.98	0.24	N/A	10.83	12.27	0.10	14.03	747.30	
		1/17/86	0.98	5.23	0.46	N/A	10.65	12.21	0.10	13.78	729.00	
Elk River Dam, Elk River, MN	Ogee	1/16/85	0.14	4.17	1.68	N/A	5.89	11.51	0.50	13.76	740.60	Rindels and Gulliver 1989
		1/20/85	0.15	4.17	1.68	N/A	6.20	11.10	0.20	13.87	742.00	
		1/24/85	0.13	4.32	1.52	N/A	5.62	10.25	0.50	13.58	731.10	
		2/26/85	0.18	3.68	2.19	N/A	7.97	11.16	1.50	13.43	743.75	
		12/6/85	0.21	4.47	1.43	N/A	6.48	10.59	0.30	13.82	746.80	
		1/10/86	0.15	4.57	1.28	N/A	4.12	9.69	0.50	13.54	729.40	
		1/31/86	0.18	4.52	1.37	N/A	4.97	9.69	0.10	13.97	729.40	
Northfield Dam, Northfield, MN	Ogee	2/16/85	0.05	2.75	0.70	N/A	9.45	11.70	0.50	13.85	745.70	Rindels and Gulliver 1989
Byllesby Dam, Cannon Falls, MN	Ogee	2/23/85	102.19	16.50	0.00	N/A	10.17	13.30	1.10	13.57	742.10	Rindels and Gulliver 1989
Fairbault Woolen Mill, Fairbault, MN	Ogee	1/26/85	0.09	2.76	0.30	N/A	10.48	11.82	0.80	13.99	738.65	Rindels and Gulliver 1989
		3/7/85	0.13	2.62	0.49	N/A	11.19	11.88	1.70	13.25	737.75	
		12/17/85	0.10	2.73	0.37	N/A	10.10	10.69	0.20	13.91	744.20	
		2/28/86	0.08	2.75	0.30	N/A	8.82	9.73	0.20	13.95	744.30	

(Continued)

(Sheet 1 of 12)

Table B1 (Continued)

Location	Type	Date of Sample	Discharge $m^3/sec/m$	Head Loss, m	Tailwater Depth, m	Gate Opening	DO Measured Upstream of Dam	DO Measured Downstream of Dam	Temperature °C	Saturation Concentration mg/l	Barometric Pressure mm, Hg	Publication
Coon Rapids Dam, Minneapolis, MN	Ogee	1/22/85	1.29	3.78	0.94	N/A	11.15	12.68	0.50	13.69	737.10	Rindels and Gulliver 1989
			1.29	3.78	0.94	N/A	11.15	12.41	0.50	13.69	737.10	
		1/9/86	1.79	2.20	2.71	N/A	11.88	12.86	0.30	13.78	729.06	
Shady Lake Dam, Oronoco, MN	Ogee	1/22/86	0.13	5.15	1.22	N/A	11.74	13.30	0.10	14.05	729.50	Rindels and Gulliver 1989
		2/14/86	0.14	5.06	1.31	N/A	9.97	12.70	0.10	13.96	729.20	
Rum River Dam, Anoka, MN	Gated Ogee	2/2/84	0.76	3.99	3.60	0.13	6.00	11.10	0.10	14.15	736.80	Rindels and Gulliver 1989
			1.05	3.96	3.47	0.19	5.80	11.20	0.10	14.15		
			1.75	3.85	3.72	0.33	6.20	11.60	0.10	14.15		
			2.42	3.74	3.78	0.47	5.80	10.30	0.10	14.15		
			3.05	3.73	3.72	0.60	6.00	9.60	0.10	14.15		
			3.31	3.63	3.75	0.73	6.20	9.40	0.10	14.15		
			0.91	4.09	3.44	0.16	6.80	11.80	0.10	14.15		
		1/24/86	0.42	3.15	3.60				0.20	13.77	728.90	
		3/14/86	0.38	3.16	3.60				0.20	13.77	733.90	
	Weir	1/24/86	0.19	3.15	N/A		6.32	10.60	0.20	13.77	743.97	
		3/14/86	0.20	3.16	N/A		9.18	11.77	0.20	13.77	743.97	
New Richmond Dam, New Richmond, WI	Ogee	2/19/85	0.11	3.09	1.52	N/A	9.95	11.89	0.20	14.00	743.00	Rindels and Gulliver 1989
Dresden Island Dam, Joliet, IL	Gated Ogee	7/8/86	4.42	5.33	1.90	4@.61m	6.14	7.77	28.20	7.31	743.97	Butts, Adkins, and Schnepper 1989b
		7/15/86	3.95	4.78	1.90	6@.61m	6.18	8.56	25.60	7.50	743.97	
			3.94	5.05	1.90	5@.61m	6.08	7.72	28.10	7.17	743.97	
		7/22/86	5.83	5.89	1.90	2@.91m	7.15	8.03	29.70	7.01	748.79	
			5.82	5.81	1.90	2@.91m	7.59	8.15	29.60	7.03	748.79	
		7/28/86	5.76	5.88	1.90	2@.91m	6.72	8.01	28.90	7.03	740.41	
			5.87	5.88	1.90	2@.91m	7.74	8.29	29.30	6.98	740.41	
		8/12/86	2.00	6.12	1.90	4@.31m	5.90	7.85	26.40	7.45	749.55	
			2.00	6.19	1.90	4@.31m	7.34	8.41	27.50	7.30	749.55	
		8/13/86	2.00	6.10	1.90	5@.31m	6.26	7.99	26.40	7.42	747.01	
			1.99	6.26	1.90	3@.31m	6.29	7.99	26.90	7.36	747.01	

(Continued)

(Sheet 2 of 12)

Table B1 (Continued)

Location	Type	Date of Sample	Discharge $m^3/sec/m$	Head Loss, m	Tailwater Depth, m	Gate Opening	DO Measured Upstream of Dam	DO Measured Downstream of Dam	Temperature °C	Saturation Concentration mg/l	Barometric Pressure mm, Hg	Publication
Dresden Island Dam, Joliet, IL (Continued)		8/18/86	7.54	6.22	1.90	1@1.22m	6.53	7.70	27.50	7.30	749.05	
			7.52	6.24	1.90	1@1.22m	7.02	7.40	27.40	7.31	749.05	
		8/19/86	7.46	6.19	1.90	1@1.22m	6.67	7.82	26.90	7.37	748.28	
			7.53	6.48	1.90	1@1.22m	7.76	8.08	27.50	7.29	748.28	
		8/20/86	3.95	6.18	1.90	2@.61m	7.74	7.77	27.60	7.27	747.27	
		8/27/86	2.00	6.13	1.90	4@.31m	5.94	7.92	25.20	7.59	747.27	
Brandon Road Dam Joliet, IL	Gated	7/9/86	1.34	10.40	1.22		3.64	7.25	24.90	7.60	744.47	Butts, Adkins, and Schnepper 1989a
		7/16/86	1.51	10.28	1.22		4.41	6.83	25.20	7.55	743.97	
			1.19	10.43	1.22		2.54	7.60	25.20	7.55	743.97	
		7/23/86	1.14	10.37	1.22		2.48	7.34	25.60	7.54	748.79	
			1.24	10.40	1.22		3.08	7.15	26.20	7.46	748.97	
		7/29/86	1.18	10.37	1.22		3.15	6.87	27.30	7.27	744.73	
		8/7/86	1.22	10.35	1.22		2.39	7.08	24.90	7.57	741.93	
			1.19	10.37	1.22		2.58	7.12	24.90	7.57	741.93	
		8/14/86	1.28	10.38	1.22		3.38	7.05	25.40	7.48	740.41	
			1.34	10.40	1.22		3.58	7.11	25.20	7.51	740.41	
		8/20/86	1.21	10.35	1.22		3.25	7.01	25.60	7.52	747.27	
		8/27/86	1.18	10.35	1.22		3.14	7.32	24.70	7.65	747.27	
		9/5/86	1.32	10.38	1.22		3.42	6.99	25.60	7.49	744.22	
		9/11/86	1.22	10.37	1.22		3.57	7.74	22.40	7.85	733.55	
		9/17/86	0.86	10.27	1.22		3.83	8.64	21.40	8.16	747.27	
			1.19	10.33	1.22		2.76	6.68	20.90	8.24	747.27	
		9/24/86	0.70	10.34	1.22		5.28	8.43	22.20	7.95	739.90	
			0.86	10.38	1.22		3.38	8.05	22.10	7.97	739.90	
Starved Rock Dam, Between Joliet and Peoria, IL	Gated	6/13/85	4.49	5.04	2.99	2@2'	6.68	8.66	20.30	8.31	742.70	Butts and Adkins 1987
		6/18,19/85	4.50	4.94	2.99	3@2'	10.05	9.87	21.60	8.09	741.68	
		6/25/85	4.50	5.13	2.99	3@2'	11.28	10.54	23.70	7.81	745.74	
		7/2,3/85	4.47	4.89	2.99	2@2'	10.20	9.08	26.40	7.39	742.95	
		7/9/85	2.24	5.21	2.99	3@1'	9.08	9.19	26.50	7.35	740.16	
		7/16,17/85	2.25	5.08	2.99	4@1'	11.93	10.18	28.10	7.20	746.76	
		7/24/85	2.26	5.48	2.99	4@1'	12.63	10.17	26.40	7.38	741.68	
		7/29,30/85	2.24	5.21	2.99	3@1'	14.16	10.53	28.20	7.20	747.52	
		8/6/85	6.69	5.04	2.99	3@3'	8.25	9.07	25.20	7.54	741.43	

(Continued)

(Sheet 3 of 12)

Table B1 (Continued)

Location	Type	Date of Sample	Discharge $m^3/sec/m$	Head Loss, m	Tailwater Depth, m	Gate Opening	DO Measured Upstream of Dam	DO Measured Downstream of Dam	Temperature °C	Saturation Concentration mg/l	Barometric Pressure mm, Hg	Publication
Starved Rock Dam, Between Joliet and Peoria, IL (Continued)		8/13,14/85	6.66	5.28	2.99	1@3'	10.50	9.09	27.30	7.29	744.73	
		8/19/85	6.67	4.83	2.99	2@3'	7.76	8.57	25.40	7.57	746.76	
		8/26,27/85	6.68	5.40	2.99	1@3'	9.01	9.04	24.20	7.77	749.30	
		9/4/85	2.26	5.24	2.99	4@1'	7.14	8.23	25.80	7.47	742.44	
		9/9,10/85	8.68	5.11	2.99	1@4'	7.40	7.74	27.40	7.26	743.46	
		9/16/85	8.69	5.35	2.99	1@4'	11.84	9.69	20.60	8.31	747.27	
		9/22,23/85	4.48	4.82	2.99	3@2'	7.32	8.72	20.30	8.29	741.17	
		9/30/85	8.70	5.17	2.99	1@4'	7.81	8.95	18.40	8.65	743.46	
		10/14,15/85	8.69	5.22	2.99	1@4'	8.87	9.61	16.40	9.02	743.71	
		10/24/85	4.45	4.46	2.99	3@2'	9.02	10.20	16.80	8.95	743.71	
Borgharen Weir, Muese River, Netherlands	Weir		1.04	5.03	0.67		5.38	7.81	21.00	8.79		Nakasone 1987
			1.54	5.00	0.85		5.41	8.11	19.00	9.15		
			1.32	4.89	1.01		9.89	10.85	6.00	12.26		
			2.33	4.44	1.51		6.89	8.26	18.00	9.34		
			0.16	5.71	0.30		4.97	8.43	17.00	9.54		
			1.89	4.75	0.96		6.22	7.12	19.00	9.15		
			1.01	4.66	1.07		6.15	8.78	15.00	9.95		
			0.70	4.90	0.79		7.20	9.39	12.00	10.64		
			0.93	4.90	1.54		6.10	7.96	20.00	8.96		
			0.81	4.78	0.90		5.13	7.87	20.00	8.96		
			0.13	5.78	0.20		4.24	7.87	20.00	8.96		
			0.07	5.39	0.00		5.37	9.62	9.00	11.40		
			1.67	4.80	1.03		7.71	7.81	15.00	9.95		
			0.25	5.55	0.45		5.42	9.28	12.00	10.64		
			0.23	5.63	0.26		5.11	9.07	12.00	10.64		
			2.81	4.09	1.71		8.66	9.39	12.00	10.64		
			1.32	4.84	1.06		6.01	7.87	20.00	8.96		
			3.00	4.69	0.89		5.14	7.57	19.00	9.15		
			1.09	5.21	0.71		6.01	8.10	18.50	9.24		
			0.39	5.38	0.48		6.83	9.28	11.00	10.89		
			0.33	5.71	0.34		5.42	7.66	21.50	8.70		
			0.52	5.41	0.45		4.96	8.11	19.00	9.15		
			0.56	5.35	0.60		5.82	7.83	18.50	9.24		
			0.67	5.27	0.67		5.60	8.31	15.50	9.85		

(Continued)

(Sheet 4 of 12)

Table B1 (Continued)

Location	Type	Date of Sample	Discharge $m^3/sec/m$	Head Loss, m	Tailwater Depth, m	Gate Opening	DO Measured Upstream of Dam	DO Measured Downstream of Dam	Temperature °C	Saturation Concentration mg/l	Barometric Pressure mm, Hg	Publication
Borgharen Weir, Muese River, Netherlands (Continued)			1.04	4.97	0.92		5.03	7.81	21.00	8.79		
			0.19	5.62	0.32		5.91	7.40	23.50	8.37		
			0.68	5.25	0.68		6.94	9.50	11.00	10.89		
			1.17	4.78	0.95		7.26	9.40	11.00	10.89		
			1.48	4.66	1.11		7.94	9.85	9.00	11.40		
			0.83	4.96	0.75		5.53	8.34	17.00	9.54		
			1.55	4.46	1.14		6.28	8.43	17.00	9.54		
			0.52	5.29	0.56		4.37	7.18	24.00	8.29		
			0.57	5.19	0.59		5.38	7.38	21.00	8.79		
			0.51	5.17	0.55		6.05	8.55	12.00	10.64		
Lith Weir, Netherlands	Weir		7.83	1.52	7.48	8.84	9.06	9.00	11.40			Nakasone 1987
			6.01	1.73	6.90	6.22	6.63	13.00	10.40			
Cascade Hauge, Netherlands	Weir		0.03	1.00	0.66	3.10	8.70	10.10	11.11			Nakasone 1987
Nakasone	Weir		0.01	0.54	0.25		7.21	7.33	26.80	7.86		Nakasone 1987
			0.74	0.82	1.06		6.58	6.81	28.10	7.68		
			0.13	0.86	0.34		7.73	7.83	24.30	8.24		
			0.08	1.00	0.63		8.15	7.72	31.20	7.27		
			0.15	1.98	0.41		7.50	7.83	24.30	8.24		
			0.09	0.24	0.40		7.10	7.17	25.70	8.02		
Meldahl Dam, Cincinnati, OH	Gated Sill	8/30/67	4.37	9.14	4.57	10@.31m	5.52	6.58	25.80	7.76	749.30	Holler 1970
		3/17/69	4.46	9.14	4.57	11@.31m	10.40	11.00	6.00	11.65	742.70	
		4/16/68	7.62	9.14	4.57	12@.61m	9.30	10.03	14.00	9.67	741.17	
		9/1/67	8.66	9.14	4.57	3@.61m	5.43	7.45	25.50	7.90	758.70	
		8/28/67	5.11	9.14	4.57	9@.31m	5.96	7.06	25.50	7.80	749.30	
		12/16/68	6.78	9.14	4.57	4@.31m	10.41	11.81	4.50	12.95		
		8/29/67	7.00	9.14	4.57	4@.31m	5.72	6.88	25.50	7.83	751.84	
		8/9/67	4.82	9.14	4.57	9@.31m	5.79	6.96	27.00	7.60	747.78	
Markland Dam Cincinnati, OH	Gated Sill	10/1/68	2.79	10.67	4.57	5@.15m	3.05	3.45	23.00	8.09	739.90	Holler 1970
		9/28/67	5.57	10.67	4.57	4@.31m	3.94	5.04	22.00	8.40	759.97	

(Continued)

(Sheet 5 of 12)

Table B1 (Continued)

Location	Type	Date of Sample	Discharge $m^3/sec/m$	Head Loss, m	Tailwater Depth, m	Gate Opening	DO Measured Upstream of Dam	DO Measured Downstream of Dam	Temperature ^oC	Saturation Concentration mg/l	Barometric Pressure mm, Hg	Publication
Arkabutla Dam, Arkabutla, MS	Gated Conduit	8/17/70	29.73	14.29			N/A	N/A	26.30	7.88	754.88	Wilhelms and Smith 1981
		8/12/72	21.68	15.12			1.90	6.90	28.30	7.34	756.92	
C. J. Brown	Gated Conduit	6/4/74	0.91	10.61			8.70	7.80	20.80	8.27		Wilhelms and Smith 1981
		8/27/74	1.35	10.58			0.30	10.40	20.70	8.29		
		9/24/74	1.35	11.19			4.40	10.40	N/A	13.41		
		6/17/75	1.89	14.78			3.20	9.40	15.60	9.21		
		7/24/75	1.69	14.81			7.80	7.80	22.60	7.98		
		8/20/75	0.51	14.90			6.90	7.60	25.50	7.55		
Crooked Creek Dam, Ford City, PA	Gated Conduit	7/17/73	1.01	12.82			5.50	7.80	23.00	7.70	734.31	Wilhelms and Smith 1981
		10/10/73	4.92	12.21			8.00	9.00	17.10	8.68	735.07	
		6/12/74	0.86	12.74			7.60	9.00	17.60	8.52	728.98	
East Branch Dam, Johnsonberg, PA	Gated Conduit	9/16/70	1.82	39.21			8.70	9.00	17.20	8.47	737.10	Wilhelms and Smith 1981
		6/3/71	2.82	45.78			9.70	9.80	14.00	9.04	734.82	
		7/21/71	6.88	42.81			8.70	8.70	18.90	8.14	733.81	
		8/29/71	5.95	38.65			8.90	9.80	19.50	8.07	736.35	
		8/30/72	6.32	39.60			7.60	8.90	19.20	8.10	735.07	
		6/20/73	10.04	45.60			9.90	9.40	13.60	9.09	732.02	
		8/30/73	7.25	40.65			8.20	8.40	21.00	7.81	734.56	
		6/17/64	1.67	46.04			11.20	9.60	14.30	8.88	725.93	
		7/15/74	9.11	43.68			10.50	9.10	20.00	7.89	727.71	
		7/29/74	9.85	42.98			9.30	9.00	19.00	8.00	722.63	
		8/23/74	5.76	40.44			9.00	9.10	22.30	7.59	733.29	
East Lynn Dam, Wayne, IN		7/21/76	0.40	15.67			7.80	7.80	23.20	7.80	741.42	Wilhelms and Smith 1981
		8/26/76	0.49	15.64			7.70	7.80	22.50	7.92	742.96	
		10/6/76	3.40	15.28			7.40	8.40	19.30	8.63		
Enid Dam, Enid, MS	Gated Conduit	7/16/69	46.29	17.27			0.90	6.80	24.10	7.92	756.16	Wilhelm and Smith 1981
		8/18/71	23.11	18.17			2.00	6.80	25.70	7.68	755.14	

(Continued)

(Sheet 6 of 12)

Table B1 (Continued)

Location	Type	Date of Sample	Discharge $m^3/sec/m$	Head Loss, m	Tailwater Depth, m	Gate Opening	DO Measured Upstream of Dam	DO Measured Downstream of Dam	Temperature °C	Saturation Concentration mg/l	Barometric Pressure mm, Hg	Publication
Grayson, Grayson, KY	Gated Conduit	5/19/76	0.35	17.61			9.50	9.50	17.00	9.01	740.41	Wilhelms and Smith 1981
		6/18/76	0.32	17.62			9.70	8.40	21.80	8.17	739.90	
		7/20/76	0.33	18.22			6.50	8.00	22.40	8.12	744.47	
		8/23/76	1.46	17.96			0.50	7.90	20.00	8.49	741.68	
		9/5/76	1.08	16.82			7.90	8.60	18.50	8.74	740.54	
Grenada Dam Grenada, MS	Gated Conduit	7/15/69	52.25	16.03			3.10	7.70	28.20	7.21	752.86	Wilhelms and Smith 1981
		8/24/70	57.72	15.54			4.60	7.60	26.30	7.47	753.11	
		8/18/71	74.12	15.21			5.50	6.80	27.90	7.28	755.14	
		8/9/72	57.06	15.28			3.50	6.90	27.60	7.32	755.90	
Mississinewa Dam Peru, IN	Gated Conduit	6/30/71	3.02	22.82			1.00	9.40	17.00	8.79	738.89	Wilhelms and Smith 1981
		8/6/71	2.56	22.83			0.20		15.00	9.25	745.24	
		9/8/71	3.86	22.81			1.00	9.40	21.10	8.10	740.92	
		6/5/72	43.21	21.48			2.30	9.30	19.40	8.41	743.71	
		8/2/72	2.00	22.81			1.10		23.90	7.62	735.84	
		4/26/73	12.59	27.34			2.60	12.60	9.20	10.44	737.87	
		6/26/73	5.69	22.82			0.30	5.60	24.40	7.53	734.31	
		7/27/73	19.40	22.06			0.60	8.20	23.60	7.66	735.84	
		8/22/73	47.16	22.23			1.20	8.80	23.30	7.80	744.47	
		6/26/74	43.25	21.50			3.20	9.60	19.70	8.31	739.14	
		7/16/74	1.53	23.19			1.00	8.70	25.80	7.42	743.20	
		9/15/74	1.53	23.17			2.10	9.40	24.40	7.60	741.43	
		9/11/74	2.42	23.11			5.50	8.80	N/A	13.19	739.65	
		6/13/75	15.56	22.13			2.10	7.80	19.40	8.34	736.85	
Salamonie Dam, Largo, IN	Gated Conduit	6/8/71	0.60	21.25			2.90	11.00	17.20	8.77	741.17	Wilhelms and Smith 1981
		7/12/71	1.77	22.22			0.80	9.50	16.70	8.89	743.20	
		8/24/71	0.65	22.77			0.00	9.40	17.90	8.66	742.70	
		10/5/71	11.50	20.71			1.90	9.60	19.70	8.32	739.90	
		6/14/72	1.67	22.65			1.40	8.20	17.70	8.63	736.85	
		7/25/72	5.39	22.26			0.60	9.70	21.30	8.05	739.65	
		8/17/72	1.16	22.69			0.20	10.10	18.70	8.48	739.14	
		9/21/72	35.08	21.93			4.70	7.80	18.90	8.48	742.19	
		6/13/73	40.00	22.14			1.90	8.60	18.30	8.58	741.68	

(Continued)

(Sheet 7 of 12)

Table B1 (Continued)

Location	Type	Date of Sample	Discharge $m^3/sec/m$	Head Loss, m	Tailwater Depth, m	Gate Opening	DO Measured Upstream of Dam	DO Measured Downstream of Dam	Temperature °C	Saturation Concentration mg/l	Barometric Pressure mm, Hg	Publication
Salamonie Dam, Largo, IN (Continued)		8/9/73	0.65	22.73			0.40		22.40	7.85	736.85	Wilhelms and Smith 1981
		9/6/73	1.16	22.67			0.40	9.60	23.30	7.75	740.66	
		6/25/74	44.65	21.66			2.50	9.90	20.00	8.27	740.16	
		7/19/74	0.49	22.71			0.50	8.90	21.00	8.12	741.68	
		8/14/74	0.65	22.78			0.30		21.10	8.13	744.47	
		9/10/74	2.69	22.44			0.50	9.50	21.10	8.11	742.19	
		6/12/75	2.69	22.40			1.80		19.00	8.37	734.31	
		7/17/75	1.16	22.69			0.10	11.40	25.60	7.45	743.71	
		4/23/76	0.51	18.28					16.20	8.96	741.43	
Sardis Dam Sardis, MS		6/10/68	138.87	20.03			4.20	8.10	21.80	8.25	753.11	Wilhelms and Smith 1981
		7/25/68	117.92	17.98			1.70	7.60	25.20	7.77	757.17	
		8/20/68	67.58	17.60			4.50	7.70	26.70	7.56	757.94	
		5/28/69	120.62	20.40			6.90	7.90	21.90	8.27	756.92	
		6/12/69	126.37	19.33			5.30	8.70	20.80	8.40	752.09	
		7/17/69	122.31	17.68			3.50	7.90	26.50	7.55	754.38	
		8/14/69	109.81	16.12			5.50	7.80	25.90	7.64	754.38	
		5/25/70	137.52	21.96			5.80	8.20	21.20	8.33	751.33	
		6/18/70	154.41	21.05			3.90	8.10	21.60	8.30	755.14	
		7/27/70	136.50	19.54			4.90	7.60	23.80	7.96	755.65	
		5/19/71	34.80	19.90			7.50	9.00	19.00	8.73	753.87	
		6/9/71	98.32	18.51			5.90	8.40	22.00	8.23	754.38	
		7/16/71	82.78	17.00			4.10	7.70	25.30	7.72	753.62	
Kawasaki, Japan			16.73	1.00	0.74		8.04	8.15	22.80	8.23		Nakasone 1987
			16.27	1.00	0.73		7.92	8.01	22.40	8.29		
Sagamigawa Channel, Atsugi, Japan	Ogee		112.62	1.35	1.05		8.86	9.07	24.80	7.92		Nakasone 1987
Kamo River, Kyoto, Japan	Ogee		9.83	0.80	0.22		7.01	7.25	26.50	7.67		Nakasone 1987
			4.48	0.60	0.32		8.43	8.38	25.00	7.89		

(Continued)

Table B1 (Continued)

Location	Type	Date of Sample	Discharge $m^3/sec/m$	Head Loss, m	Tailwater Depth, m	Gate Opening	DO Measured Upstream of Dam	DO Measured Downstream of Dam	Temperature ^oC	Saturation Concentration mg/l	Barometric Pressure mm, Hg	Publication
Naganoseki River,	Ogee		18.13	1.15	0.86		6.24	6.55	28.30	7.42		Nakasone 1987
Takasaki, Japan			32.42	1.15	1.04		7.19	7.31	24.80	7.92		Nakasone 1987
Syakujii River,	Ogee		29.32	1.80	1.10		6.90	7.86	12.20	10.27		Nakasone 1987
Tokyo, Japan			6.50	1.42	1.10		7.07	7.74	12.50	10.20		
			8.62	1.42	1.14		6.19	6.88	13.00	10.09		
			7.97	1.42	1.13		5.41	6.27	12.90	10.11		
Onnabori River,	Ogee		4.62	1.50	0.63		8.13	7.72	31.20	7.06		Nakasone 1987
Honjyo, Japan			5.00	1.50	0.67		6.33	6.93	29.60	7.25		
Momonoki River,	Ogee		16.50	1.00	0.68		7.82	7.94	22.70	8.24		Nakasone 1987
Maebashi River			15.33	1.00	0.66		8.12	8.17	22.90	8.21		
			9.73	1.00	0.34		7.98	8.08	23.00	8.19		
			8.93	1.00	0.34		8.37	8.41	22.10	8.34		
Hanamuro River, Ibaragi	Ogee		0.68	0.75	0.25		7.21	7.33	26.80	7.63		Nakasone 1979
Naganoseki	Ogee		39.85	0.60	0.80		6.85	6.96	27.30	7.56		Nakasone 1979
Channel,			44.47	1.31	1.06		6.58	6.81	28.10	7.45		
Takasaki Chan.			43.63	1.35	1.03		6.68	6.95	27.80	7.49		
Momonoki River,	Ogee		9.15	1.00	0.34		7.73	7.83	24.30	7.99		Nakasone 1979
Maebashi Chan.	Sill		7.23	2.20	0.39		7.75	7.93	24.30	7.99		
	Ogee		14.35	1.00	0.64		8.09	8.15	22.50	8.27		
Onnabori River,	Ogee		4.62	1.50	0.63		8.15	7.72	31.20	7.06		Nakasone 1979
Honoyo Chan.			5.07	1.50	0.66		8.03	7.73	30.10	7.19		
			4.07	1.50	0.61		8.70	8.56	23.30	8.15		
Momonoki River,	Ogee		11.73	1.00	0.38		7.87	7.71	26.00	7.74		Nakasone 1979
Maebashi Chan.,			8.37	1.00	0.33		7.78	7.84	25.50	7.81		
Japan	Sill		8.68	2.20	0.41		7.50	7.83	24.30	7.99		

(Continued)

Table B1 (Continued)

Location	Type	Date of Sample	Discharge $m^3/sec/m$	Head Loss, m	Tailwater Depth, m	Gate Opening	DO Measured Upstream of Dam	DO Measured Downstream of Dam	Temperature °C	Saturation Concentration mg/l	Barometric Pressure mm, Hg	Publication
Ino River Takasaki Chan., Japan	Ogee		5.37	0.50	0.40		7.10	7.17	25.70	7.78		Nakasone 1979
			8.23	0.50	0.43		7.47	7.52	24.00	8.04		Nakasone 1979
			14.27	0.50	0.51		6.95	7.02	23.00	8.19		
Fugi River Maebashi Chan., Japan	Ogee		1.93	0.78	0.18		10.19	9.83	19.70	8.75		Nakasone 1979
Summary of Dam Aeration for the Des Plaines River Dam Reference Number												
15				N/A		9.25	9.30	4.20	12.82	0.02		Butts and Evans 1983
16				N/A		10.00	10.20	4.00	12.89	0.08		
17				N/A		11.40	11.60	4.70	12.66	0.23		
18				N/A		11.30	11.50	3.80	12.95	0.16		
19				1.13		6.14	7.29	12.31	10.57	0.28		
20				1.19		6.16	7.89	12.68	10.48	0.43		
21				0.48		2.92	3.62	20.49	8.88	0.12		
22				0.62		3.42	4.53	18.80	9.19	0.20		
23				0.94		5.00	5.94	18.99	9.15	0.24		
24				0.73		0.83	2.01	18.50	9.24	0.15		
25				2.38		4.15	6.20	24.62	8.19	0.54		
26				0.65		5.46	6.08	24.55	8.20	0.25		
Weisenberger Mill, Lexington		1975	1.06	3.02		3.00	7.10	23.00	8.44	0.79		Foree 1976
			1.26	3.05		4.00	7.50	17.50	9.37	0.69		
Forks of Frankfort, KY 1st Dam-Lawrenceburg, KY		1975	0.06	3.60		7.20	8.60	20.20	8.74	1.10		
		1975	0.06	2.90		2.80	6.90	22.50	8.54	0.75		

(Continued)

(Sheet 10 of 12)

Table B1 (Continued)

Location	Type	Date of Sample	Discharge m^3/sec/m	Head Loss, m	Tailwater Depth, m	Gate Opening	DO Measured Upstream of Dam	DO Measured Downstream of Dam	Temperature °C	Saturation Concentration mg/l	Barometric Pressure mm, Hg	Publication
2nd Dam-S Lawrenceburg, KY		1975	0.06	1.07		5.90	7.10	22.80	81.50	0.51		Foree 1976
3rd Dam-S Lawrenceburg, KY		1975	0.06	0.80		3.10	4.50	24.00	8.32	0.28		Foree 1976
Falls Russell Spring, KY		1975	0.06	4.30		9.20	8.60	22.50	8.53	0.65		Foree 1976
LeBus Dam Cynthiana, KY		1973	3.02	2.60		4.60	7.80	21.00	8.77	0.82		Foree 1976
Great Cro Georgetown, KY		1975	0.64	1.90		7.20	8.80	17.00	9.48	0.80		Foree 1976
			0.17	1.90		17.60	11.80	23.00	8.44	0.62		
Faribault Dam Faribault, MN	Ogee	9/20/89										McDonald 1990
Right Bank			0.03	4.19	0.15		1.68	3.68	23.00	8.33	741.10	
Center Bank			0.03	4.19	0.15		1.42	2.92	23.00	8.33	741.10	
Left Bank			0.03	4.19	0.15		1.99	4.71	23.00	8.33	741.10	
Coon Rapids Dam Minneapolis, MN	Gated Ogee											McDonald 1990
Left Bank		12/18/89		4.19			12.10	12.49	0.00	13.73	750.40	
Right Bank				4.19			12.09	13.49	0.00	13.73	750.40	
Left Bank		3/27/90		3.84			11.27	11.68	1.80	13.31	747.30	
Right Bank				3.84			12.05	12.87	1.80	13.31	747.30	
Elk River Dam Elk River, MN	Gated Ogee	1/19/90										McDonald 1990
Right Bank			0.10	4.63	1.12		5.12	10.86	1.40	13.40	740.50	
Center Bank			0.10	4.63	1.12		5.84	11.06	1.50	13.40	740.50	

(Continued)

(Sheet 11 of 12)

Table B1 (Concluded)

Location	Type	Date of Sample	Discharge $m^3/sec/m$	Head Loss, m	Tailwater Depth, m	Gate Opening	DO Measured Upstream of Dam	DO Measured Downstream of Dam	Temperature $^{\circ}C$	Saturation Concentration mg/l	Barometric Pressure mm, Hg	Publication
Left Bank			0.10	4.63	1.12		3.94	10.54	1.90	13.40	740.50	McDonald 1990
Kost Dam	Ogee											McDonald 1990
Chisago, MN												
Right Bank		1/26/90	0.05	4.19	0.20		8.70	10.59	0.32	13.73	739.60	McDonald 1990
Center Bank			0.05	4.19	0.20		8.58	10.53	0.27	13.73	739.60	
Left Bank			0.05	4.19	0.20		9.32	11.01	0.45	13.73	739.60	
Rum River Dam	Ogee	3/21/89	0.16	3.26	3.35		7.41	11.58	1.00	13.58	741.20	McDonald 1990
Anoka, MN		2/17/90	0.17	3.30	3.28		8.57	11.97	0.10	13.86	744.30	
		3/2/90	0.16	0.04	3.20				0.10	13.86		
			0.35	0.08	3.28				0.10	13.86		
			0.66	0.16	3.34				0.10	13.86		
			1.26	0.30	3.43				0.10	13.86		
			1.81	0.44	3.51				0.10	13.86		
			2.31	0.58	3.69				0.10	13.86		McDonald 1990
			2.58	0.66	3.70				0.10	13.86		
	Weir				0.61		8.05	11.63	0.10	13.86		
					0.30		11.63	12.00	0.10	13.86		
St Cloud Dam	Ogee	3/15/90	0.15	0.03	5.70				0.00	13.63		McDonald 1990
St Cloud, MN			0.31	0.06	5.70				0.00	13.63		
			0.63	0.12	5.76				0.00	13.63		
			1.27	0.24	5.79				0.00	13.63		
			2.09	0.40	5.79				0.00	13.63		
			2.93	0.55	5.79				0.00	13.63		
			3.61	0.67	5.79				0.00	13.63		

(Sheet 12 of 12)

APPENDIX C: PROCESSED DATA FOR COMPARISON WITH PREDICTIVE EQUATIONS

Table C1

Data Used for the Statistical Analysis of Predictive Equations

Location	Type	Date of Sample	Discharge $m^3/sec/m$	Head Loss, m	Tailwater Depth, m	Saturation Concentration mg/l	E	U	$E_{20°C}$	$U_{20°C}$
Kost Dam, Chisago County, MN	Ogee	2/02/85	0.13	4.01	0.24	14.21	0.41	0.04	0.58	0.05
		3/12/85	0.14	3.97	0.34	13.31	0.39	0.05	0.54	0.06
		12/13/85	0.16	3.95	0.34	13.95	0.41	0.02	0.58	0.02
		1/13/86	0.10	3.97	0.24	13.94	0.43	0.03	0.60	0.03
		2/21/86	0.08	3.94	0.30	13.75	0.45	0.03	0.62	0.03
		3/10/86	0.11	3.97	0.27	13.77	0.40	0.03	0.56	0.04
St. Cloud Dam, St. Cloud, MN	Ogee	3/02/85	0.68	5.32	0.24	13.81	0.53	0.04	0.70	0.04
		3/14/85	0.83	5.35	0.27	13.61	0.55	0.04	0.72	0.04
		12/19/85	1.10	4.98	0.24	14.03	0.45	0.05	0.62	0.06
		1/17/86	0.98	5.23	0.46	13.78	0.50	0.05	0.68	0.05
Elk River Dam, Elk River, MN	Ogee	1/16/85	0.14	4.17	1.68	13.76	0.67	0.03	0.83	0.02
		1/20/85	0.15	4.17	1.68	13.87	0.64	0.04	0.81	0.03
		1/24/85	0.13	4.32	1.52	13.58	0.58	0.03	0.75	0.03
		2/26/85	0.18	3.68	2.19	13.43	0.59	0.03	0.75	0.03
		12/6/85	0.21	4.47	1.43	13.82	0.56	0.03	0.74	0.03
		1/10/86	0.15	4.57	1.28	13.54	0.59	0.02	0.74	0.02
		1/31/86	0.18	4.52	1.37	13.97	0.56	0.02	0.74	0.02
Northfield Dam, Northfield, MN	Ogee	2/16/85	0.05	2.75	0.70	13.85	0.50	0.05	0.67	0.05
Byllesby Dam, Cannon Falls, MN	Ogee	2/23/85		16.50	0.31	13.57	0.92	0.09	0.98	0.03
Faribault Woolen Mill, Faribault, MN	Ogee	1/26/85	0.09	2.76	0.30	13.99	0.28	0.04	0.41	0.05
		3/7/85	0.13	2.62	0.49	13.25	0.33	0.07	0.46	0.09
		12/17/85	0.10	2.73	0.37	13.91	0.37	0.03	0.53	0.04
		2/28/86	0.08	2.75	0.30	13.95	0.43	0.03	0.60	0.03
Coon Rapids Dam, Minneapolis, MN	Ogee	1/22/85	1.29	3.78	0.94	13.69	0.50	0.11	0.67	0.12
			1.29	3.78	0.94	13.69	0.60	0.14	0.77	0.13
		1/9/86	1.79	2.20	2.71	13.78	0.51	0.09	0.68	0.09

(Continued)

Table C1 (Continued)

Location	Type	Date of Sample	Discharge $m^3/sec/m$	Head Loss, m	Tailwater Depth, m	Saturation Concentration mg/l	E	U	E_{20^oC}	U_{20^oC}
Shady Lake Dam,	Ogee	1/22/86	0.13	5.15	1.22	14.05	0.65	0.08	0.82	0.07
Oronoco, MN		2/14/86	0.14	5.06	1.31	13.96	0.68	0.05	0.84	0.04
Rum River Dam,	Gated	2/2/84	0.76	3.99	3.60	14.15	0.63	0.03	0.80	0.03
Anoka, MN	Ogee		1.05	3.96	3.47	14.15	0.65	0.03	0.82	0.03
			1.75	3.85	3.72	14.15	0.68	0.03	0.84	0.02
			2.42	3.74	3.78	14.15	0.54	0.02	0.72	0.02
			3.05	3.73	3.72	14.15	0.44	0.02	0.61	0.02
			3.31	3.63	3.75	14.15	0.40	0.02	0.56	0.02
			0.91	4.09	3.44	14.15	0.68	0.03	0.84	0.02
		1/24/86	0.42	3.15	3.60	13.77	0.62	0.03	0.79	0.03
		3/14/86	0.38	3.16	3.60	13.77	0.68	0.05	0.84	0.04
	Weir	1/24/86	0.19	3.15	N/A	13.77	0.57	0.04	0.75	0.04
New Richmond Dam, New Richmond, WI	Ogee	2/19/85	0.11	3.09	1.52	14.00	0.48	0.04	0.65	0.04
Dresden Island	Gated	8/12/86	2.00	6.12	1.90	7.45	1.26	0.23	1.30	0.23
Dam, Joliet, IL	Ogee	8/27/86	2.00	6.13	1.90	7.59	1.20	0.21	1.23	0.22
Brandon Road Dam,	Gated	7/9/86	1.34	10.40	1.22	7.60	0.91	0.07	0.89	0.08
Joliet, IL	Ogee	7/16/86	1.51	10.28	1.22	7.55	0.77	0.07	0.73	0.08
			1.19	10.43	1.22	7.55	1.01	0.06	1.02	0.08
		7/23/86	1.14	10.37	1.22	7.54	0.96	0.05	0.94	0.07
			1.24	10.40	1.22	7.46	0.93	0.06	0.90	0.07
		7/29/86	1.18	10.37	1.22	7.27	0.90	0.06	0.87	0.07
		8/7/86	1.22	10.35	1.22	7.57	0.91	0.05	0.88	0.06
			1.19	10.37	1.22	7.57	0.91	0.05	0.89	0.06
		8/14/86	1.28	10.38	1.22	7.48	0.89	0.06	0.87	0.07
			1.34	10.40	1.22	7.51	0.90	0.07	0.87	0.08
		8/20/86	1.21	10.35	1.22	7.52	0.88	0.06	0.85	0.07
		8/27/86	1.18	10.35	1.22	7.65	0.93	0.06	0.91	0.07
		9/5/86	1.32	10.38	1.22	7.49	0.88	0.06	0.85	0.07
		9/11/86	1.22	10.37	1.22	7.85	0.97	0.07	0.97	0.08
		9/17/86	0.86	10.27	1.22	8.16	1.11	0.08	1.12	0.08

(Continued)

(Sheet 2 of 8)

Appendix C (Continued)

Location	Type	Date of Sample	Discharge $m^3/sec/m$	Head Loss, m	Tailwater Depth, m	Saturation Concentration mg/l	E	U	E_{20^oC}	U_{20^oC}
Brandon Road Dam,			1.19	10.33	1.22	8.24	0.72	0.04	0.71	0.04
Joliet, IL		9/24/86	0.70	10.34	1.22	7.95	1.18	0.13	1.19	0.14
(Continued)			0.86	10.38	1.22	7.97	1.02	0.07	1.02	0.08
Borgharen Weir,	Weir		1.04	5.03	0.67	8.79	0.71	0.06	0.71	0.06
Muese River,			1.54	5.00	0.85	9.15	0.72	0.06	0.73	0.06
Netherlands			1.32	4.89	1.01	12.26	0.40	0.09	0.51	0.10
			2.33	4.44	1.51	9.34	0.56	0.08	0.57	0.08
			0.16	5.71	0.30	9.54	0.76	0.05	0.78	0.05
			1.89	4.75	0.96	9.15	0.31	0.05	0.31	0.05
			1.01	4.66	1.07	9.95	0.69	0.06	0.73	0.06
			0.70	4.90	0.79	10.64	0.64	0.07	0.70	0.07
			0.93	4.90	1.54	8.96	0.65	0.07	0.65	0.07
			0.81	4.78	0.90	8.96	0.71	0.06	0.71	0.06
			0.13	5.78	0.20	8.96	0.77	0.05	0.77	0.05
			0.07	5.39	0.00	11.40	0.70	0.04	0.79	0.04
			1.67	4.80	1.03	9.95	0.04	0.07	0.05	0.08
			0.25	5.55	0.45	10.64	0.74	0.05	0.80	0.04
			0.23	5.63	0.26	10.64	0.72	0.04	0.78	0.04
			2.81	4.09	1.71	10.64	0.37	0.09	0.42	0.10
			1.32	4.84	1.06	8.96	0.63	0.07	0.63	0.07
			3.00	4.69	0.89	9.15	0.61	0.05	0.61	0.05
			1.09	5.21	0.71	9.24	0.65	0.06	0.66	0.06
			0.39	5.38	0.48	10.89	0.60	0.06	0.68	0.05
			0.33	5.71	0.34	8.70	0.68	0.06	0.67	0.06
			0.52	5.41	0.45	9.15	0.75	0.05	0.76	0.05
			0.56	5.35	0.60	9.24	0.59	0.06	0.60	0.06
			0.67	5.27	0.67	9.85	0.64	0.05	0.67	0.05
			1.04	4.97	0.92	8.79	0.74	0.06	0.73	0.06
			0.19	5.62	0.32	8.37	0.61	0.08	0.58	0.08
			0.68	5.25	0.68	10.89	0.65	0.06	0.72	0.06
			1.17	4.78	0.95	10.89	0.59	0.06	0.66	0.06
			1.48	4.66	1.11	11.40	0.55	0.06	0.64	0.07
			0.83	4.96	0.75	9.54	0.70	0.06	0.72	0.05

(Continued)

(Sheet 3 of 8)

Table C1 (Continued)

Location	Type	Date of Sample	Discharge $m^3/sec/m$	Head Loss, m	Tailwater Depth, m	Saturation Concentration mg/l	E	U	$E_{20°C}$	$U_{20°C}$
Borgharen Weir, Muese River, Netherlands (Continued)			1.55	4.46	1.14	9.54	0.66	0.07	0.68	0.07
			0.52	5.29	0.56	8.29	0.72	0.05	0.69	0.05
			0.57	5.19	0.59	8.79	0.59	0.06	0.58	0.06
			0.51	5.17	0.55	10.64	0.54	0.04	0.61	0.05
Lith Wier, Netherlands	Weir		7.83	1.52	7.48	11.40	0.09	0.06	0.11	0.07
			6.01	1.73	6.90	10.40	0.10	0.03	0.11	0.04
Cascade Hauge, Netherlands	Weir		0.03	1.00	0.66	11.11	0.70	0.03	0.78	0.03
Nakasone	Weir		0.74	0.82	1.06	7.68	0.21	0.14	0.18	0.13
			0.08	1.00	0.63	7.27	0.49	0.14	0.42	0.13
			0.09	0.24	0.40	8.02	0.08	0.18	0.07	0.16
Meldahl Dam, Cincinnati, OH	Gated Sill	8/30/67	4.37	9.14	4.57	7.76	0.39	0.05	0.35	0.05
		3/17/69	4.46	9.14	4.57	11.65	0.29	0.08	0.37	0.09
		4/16/68	7.62	9.14	4.57	9.67	0.66	0.20	0.71	0.20
		9/1/67	8.66	9.14	4.57	7.90	0.70	0.07	0.66	0.07
		8/28/67	5.11	9.14	4.57	7.80	0.47	0.07	0.43	0.06
		12/16/68	6.78	9.14	4.57	12.95	0.55	0.09	0.68	0.09
		8/29/67	7.00	9.14	4.57	7.83	0.45	0.06	0.41	0.06
		8/9/67	4.82	9.14	4.57	7.60	0.51	0.07	0.46	0.07
Markland Dam, Cincinnati, OH	Gated Sill	10/1/68	2.79	10.67	4.57	8.09	0.07	0.02	0.07	0.02
		9/28/67	5.57	10.67	4.57	8.40	0.23	0.03	0.22	0.03
Arkabutla Dam Arkabutla, MS	Gated Conduit	8/17/70	29.73	14.29		7.88	0.00	0.02		
		8/12/72	21.68	15.12		7.34	0.92	0.05	0.89	0.06
C. J. Brown	Gated Conduit	8/27/74	1.35	10.58		8.29	1.26	0.05	1.27	0.05
		9/24/74	1.35	11.19		13.41	0.67	0.04	0.66	0.04
		6/17/75	1.89	14.78		9.21	1.03	0.06	1.00	0.01

(Continued)

(Sheet 4 of 8)

Table C1 (Continued)

Location	Type	Date of Sample	Discharge $m^3/sec/m$	Head Loss, m	Tailwater Depth, m	Saturation Concentration mg/l	E	U	$E_{20°C}$	$U_{20°C}$
Crooked Creek Dam, Ford City, PA	Gated Conduit	7/17/73	1.01	12.82		7.70	1.05	0.14	1.01	0.03
East Branch Dam, Johnsonberg, PA	Gated Conduit	6/17/64	1.67	46.04		8.88	0.69	0.10	0.68	0.10
		7/15/74	9.11	43.68		7.89	0.54	0.07	0.58	0.07
		7/29/74	9.85	42.98		8.00	0.23	0.11	0.23	0.11
East Lynn Dam, Wayne, IN		10/6/76	3.40	15.28		8.63	0.81	0.22	0.80	0.23
Enid Dam, Enid, MS	Gated Conduit	7/16/69	46.29	17.27		7.92	0.84	0.04	0.95	0.02
		8/18/71	23.11	18.17		7.68	0.85	0.04	0.82	0.05
Grayson, Grayson, KY	Gated Conduit	6/18/76	0.32	17.62		8.17	0.85	0.17	0.87	0.16
		7/20/76	0.33	18.22		8.12	0.93	0.18	0.92	0.19
		8/23/76	1.46	17.96		8.49	0.93	0.04	0.92	0.04
Grenada Dam, Grenada, MS	Gated	7/15/69	52.25	16.03		7.21	1.12	0.07	1.03	0.03
		8/24/70	57.72	15.54		7.47	1.05	0.10	1.07	0.14
		8/18/71	74.12	15.21		7.28	0.73	0.12	0.69	0.13
		8/9/72	57.06	15.28		7.32	0.89	0.07	0.85	0.08
Mississinewa Dam, Peru, IN	Gated Conduit	6/30/71	3.02	22.82		8.79	1.08	0.05	1.02	0.01
		9/8/71	3.86	22.81		8.10	1.18	0.05	1.15	0.05
		6/5/72	43.21	21.48		8.41	1.14	0.06	1.15	0.06
		4/26/73	12.59	27.34		10.44	1.27	0.06	1.30	0.06
		6/26/73	5.69	22.82		7.53	0.73	0.03	0.81	0.03
		7/27/73	19.40	22.06		7.66	1.08	0.04	1.10	0.05
		8/22/73	47.16	22.23		7.80	1.15	0.05	1.17	0.05
		6/26/74	43.25	21.50		8.31	1.25	0.08	1.28	0.08
		7/16/74	1.53	23.19		7.42	1.20	0.05	1.20	0.05
		9/15/74	1.53	23.17		7.60	1.33	0.07	1.37	0.07

(Continued)

(Sheet 5 of 8)

Table C1 (Continued)

Location	Type	Date of Sample	Discharge $m^3/sec/m$	Head Loss, m	Tailwater Depth, m	Saturation Concentration mg/l	E	U	$E_{20°C}$	$U_{20°C}$
Mississinewa Dam, Peru, IN (Continued)		9/11/74	2.42	23.11		13.19	0.43	0.03	0.40	0.03
		6/13/75	15.56	22.13		8.34	0.91	0.05	0.98	0.02
Salamonie Dam, Largo, IN	Gated Con-duit	6/8/71	0.60	21.25		8.77	1.38	0.08	1.21	0.07
		7/12/71	1.77	22.22		8.89	1.08	0.04	1.06	0.04
		8/24/71	0.65	22.77		8.66	1.09	0.04	1.07	0.04
		10/5/71	11.50	20.71		8.32	1.20	0.06	1.19	0.06
		6/14/72	1.67	22.65		8.63	0.94	0.04	0.94	0.04
		7/25/72	5.39	22.26		8.05	1.22	0.05	1.21	0.05
		8/17/72	1.16	22.69		8.48	1.20	0.05	1.20	0.05
		9/21/72	35.08	21.93		8.48	0.82	0.07	0.83	0.07
		6/13/73	40.00	22.14		8.58	1.00	0.05	1.00	0.04
		9/6/73	1.16	22.67		7.75	1.25	0.05	1.27	0.05
		6/25/74	44.65	21.66		8.27	1.28	0.07	1.31	0.07
		7/19/74	0.49	22.71		8.12	1.10	0.04	1.10	0.04
		9/10/74	2.69	22.44		8.11	1.18	0.05	1.19	0.05
		7/17/75	1.16	22.69		7.45	1.54	0.06	1.53	0.06
		4/23/76	0.51	18.28		8.96	0.00	0.02	0.00	0.01
Sardis Dam, Sardis, MS		6/10/68	138.87	20.03		8.25	0.96	0.06	1.00	0.01
		7/25/68	117.92	17.98		7.77	0.97	0.04	0.97	0.04
		8/20/68	67.58	17.60		7.56	1.04	0.08	1.06	0.09
		5/28/69	120.62	20.40		8.27	0.73	0.13	0.68	0.13
		6/12/69	126.37	19.33		8.40	1.10	0.08	1.11	0.09
		7/17/69	122.31	17.68		7.55	1.09	0.06	1.09	0.06
		8/14/69	109.81	16.12		7.64	1.08	0.11	1.10	0.13
		5/25/70	137.52	21.96		8.33	0.95	0.09	0.93	0.11
		6/18/70	154.41	21.05		8.30	0.95	0.05	0.95	0.06
		7/27/70	136.50	19.54		7.96	0.88	0.07	0.87	0.07
		5/19/71	34.80	19.90		8.73	1.21	0.22	1.24	0.22
		6/9/71	98.32	18.51		8.23	1.07	0.11	1.07	0.10
		7/16/71	82.78	17.00		7.72	1.00	0.06	0.99	0.08
Naganozeki River, Takasaki, Japan	Weir		18.13	1.15	0.86	7.42	0.26	0.11	0.39	0.15
			32.42	1.15	1.04	7.92	0.17	0.17	0.14	0.15

(Continued)

(Sheet 6 of 8)

Table C1 (Continued)

Location	Type	Date of Sample	Discharge $m^3/sec/m$	Head Loss, m	Tailwater Depth, m	Saturation Concentration mg/l	E	U	$E_{20°C}$	$U_{20°C}$
Syakujii River, Tokyo, Japan	Weir		29.32	1.80	1.10	10.27	0.28	0.04	0.42	0.06
			6.50	1.42	1.10	10.20	0.21	0.04	0.25	0.05
			8.62	1.42	1.14	10.09	0.18	0.03	0.21	0.04
			7.97	1.42	1.13	10.11	0.18	0.03	0.21	0.03
Onnabori River, Honjyo, Japan	Weir		4.62	1.50	0.63	7.06	0.38	0.14	0.54	0.17
			5.00	1.50	0.67	7.25	0.65	0.17	0.57	0.17
Naganoseki Channel, Takasaki Chan., Japan	Weir		44.47	1.31	1.06	7.45	0.26	0.14	0.23	0.13
			43.63	1.35	1.03	7.49	0.33	0.16	0.29	0.14
Onnabori River, Honjyo Chan., Japan	Weir		4.62	1.50	0.63	7.06	0.39	0.14	0.56	0.16
			5.07	1.50	0.66	7.19	0.36	0.18	0.30	0.16
Ino River, Takasaki Chan., Japan	Weir		8.23	0.50	0.43	8.04	0.09	0.21	0.08	0.19
			14.27	0.50	0.51	8.19	0.06	0.10	0.05	0.10
Fugi River, Maebashi Chan., Japan	Weir		1.93	0.78	0.18	8.75	0.25	0.10	0.37	0.14
Faribault Dam Faribault, MN	Ogee	9/20/89								
Right Bank			0.03	4.19	0.15	8.33	0.30	0.02	0.29	0.02
Center Bank			0.03	4.19	0.15	8.33	0.22	0.02	0.21	0.02
Left Bank			0.03	4.19	0.15	8.33	0.43	0.03	0.41	0.03
Elk River Dam Elk River, MN	Gated Ogee	1/19/90								
Right Bank			0.10	4.63	1.12	13.40	0.69	0.01	0.84	0.01
Center Bank			0.10	4.63	1.12	13.40	0.69	0.03	0.84	0.02

(Continued)

Table C1 (Concluded)

Location	Type	Date of Sample	Discharge $m^3/sec/m$	Head Loss, m	Tailwater Depth, m	Saturation Concentration mg/l	E	U	$E_{20°C}$	$U_{20°C}$
Left Bank			0.10	4.63	1.12	13.40	0.70	0.02	0.83	0.02
Kost Dam	Ogee	1/26/90								
Chisago, MN										
Right Bank			0.05	4.19	0.20	13.73	0.38	0.03	0.53	0.04
Center Bank			0.05	4.19	0.20	13.73	0.38	0.03	0.53	0.03
Left Bank			0.05	4.19	0.20	13.73	0.38	0.03	0.54	0.04
Rum River Dam	Ogee	3/21/89	0.16	3.26	3.35	13.58	0.68	0.03	0.83	0.03
Anoka, MN		2/17/90	0.17	3.30	3.28	13.86	0.64	0.02	0.80	0.03
		3/2/90	0.16	0.04	3.20	13.86			0.84	0.04
			0.35	0.08	3.28	13.86			0.80	0.04
			0.66	0.16	3.34	13.86			0.73	0.04
			1.26	0.30	3.43	13.86			0.69	0.04
			1.81	0.44	3.51	13.86			0.62	0.03
			2.31	0.58	3.69	13.86			0.59	0.03
			2.58	0.66	3.70	13.86			0.44	0.03
St Cloud Dam	Gated	3/15/90	0.15	6.40	5.70	13.63			0.68	0.10
St Cloud, MN	Ogee		0.31	6.40	5.70	13.63			0.63	0.08
			0.63	6.40	5.76	13.63			0.74	0.09
			1.27	6.40	5.79	13.63			0.83	0.11
			2.09	6.40	5.79	13.63			0.79	0.13
			2.93	6.40	5.79	13.63			0.24	0.17
			3.61	6.40	5.79	13.63			0.28	0.14

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APPENDIX D: DESCRIPTION OF PREDICTIVE EQUATIONS AND PLOTS OF PREDICTED VERSUS MEASURED OXYGEN TRANSFER EFFICIENCIES

Avery and Novak Equation

1. The Reynolds number and the Froude number of a flow are used by Avery and Novak (1978)* to predict the deficit ratio specifically for weirs as follows:

$$r_{15} - 1 = 0.64 \times 10^{-4}\, Fr_j^{1.787}\, R^{0.533} \qquad \text{(D1)}$$

where r_{15} = the deficit ratio at 15 °C**

Fr_j = the Froude number of the jet, given by the following equation:

$$Fr_j = \frac{(2g)^{0.25} h^{0.75}}{q^{0.5}}$$

g = the acceleration due to gravity

h = the head across the structure

q = the discharge over the structure

R = the Reynolds number of the jet:

$$R = \frac{q}{2\nu}$$

ν = the kinematic viscosity

The deficit ratio r can then be related to the efficiency E of the structure by:

$$E = 1 - \frac{1}{r} \qquad \text{(D2)}$$

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* References cited in this appendix can be found in the References at the end of the main text of the report.

** For convenience, symbols and unusual abbreviations used in this appendix are listed and defined in the Notation (Appendix E).

Also, using the relation of Gulliver, Thene, and Rindels (1990):

$$E_{20} = 1 - (1 - E_T)^{\frac{1}{f_T}} \tag{D3}$$

where E_T = gas transfer efficiency at a given temperature T

f_T = indexing coefficient given by:

$$f_T = 1.0 + 0.02103\ (T - 20) + 8.261 * 10^{-5}\ (T - 20)^2$$

which yields an efficiency indexed to 20 °C of:

$$E_{20} = 1 - \left(\frac{1}{1 + 0.64 \times 10^{-4}\ Fr_j^{1.787}\ R^{0.533}}\right)^{1.1149} \tag{D4}$$

The results of the Avery and Novak equation for the field data are presented in Figures D1–D4.

Adjustment of Thene to the Avery and Novak Equation

2. Thene (1988) felt that the accuracy of the Avery and Novak equation could be improved by considering the tailwater depth of the flow. Absorption is expected to increase as the depth and contact time of entrained bubbles increase. Thene created a function that varied with respect to the depth of the tailwater pool H, and the head across the structure h. The adjustment of Thene to Avery and Novak takes the following form:

$$r_{15} - 1 = 1.005 \times 10^{-5}\ Fr_j^{2.08}\ R^{0.63} \left[1 - 0.6 \exp\left(-3.7\ \frac{H}{h}\right)\right] \tag{D5}$$

which can then be converted to an efficiency at 20 °C by Equations D2 and D3 to yield:

$$E_{20} = \left\{\frac{1}{1 + 1.005 \times 10^{-5}\ Fr_j^{2.08}\ R^{0.63} \left[1 - 0.6 \exp\left(-3.7\ \frac{H}{h}\right)\right]}\right\}^{1.1149} \tag{D6}$$

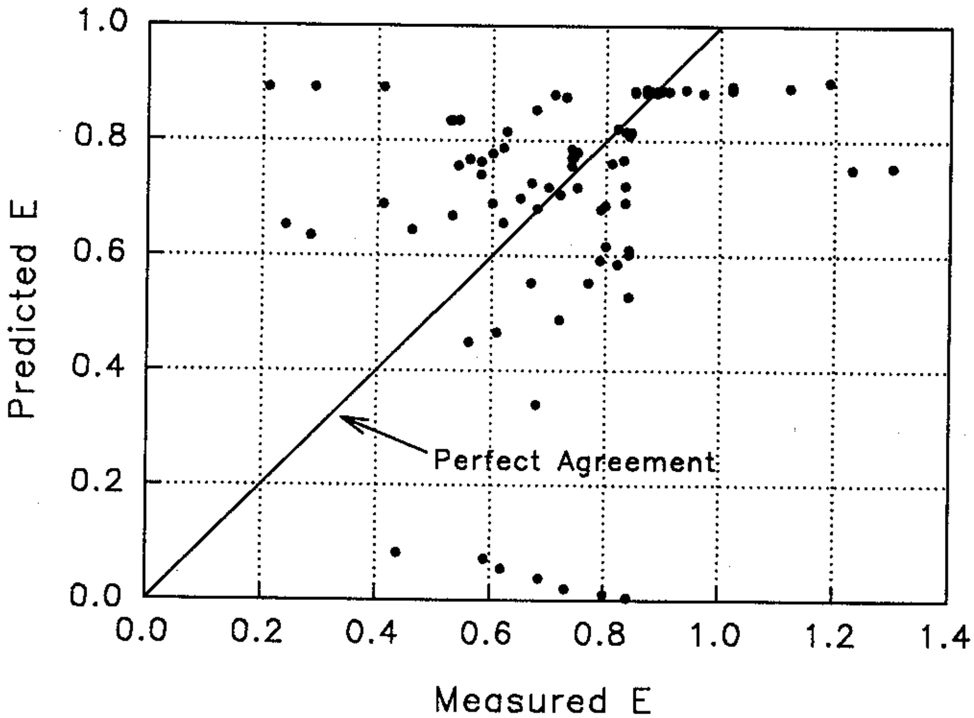

Figure D1. Avery and Novak equation for ogee spillways. Predicted values of gas transfer versus measured values

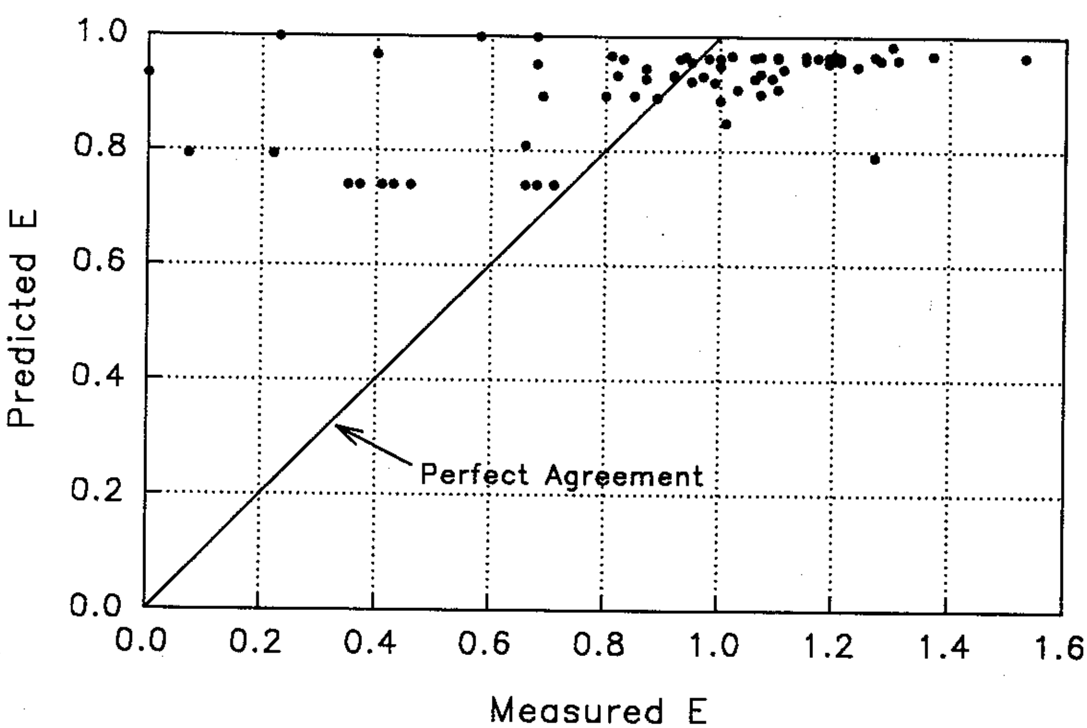

Figure D2. Avery and Novak equation for gated conduits. Predicted values of gas transfer versus measured values

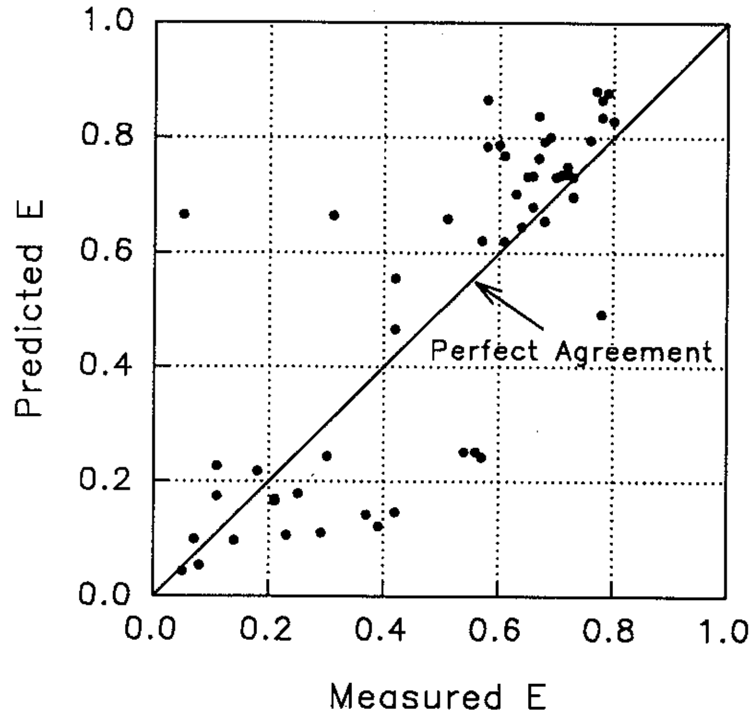

Figure D3. Avery and Novak equation for weirs. Predicted values of gas transfer versus measured values

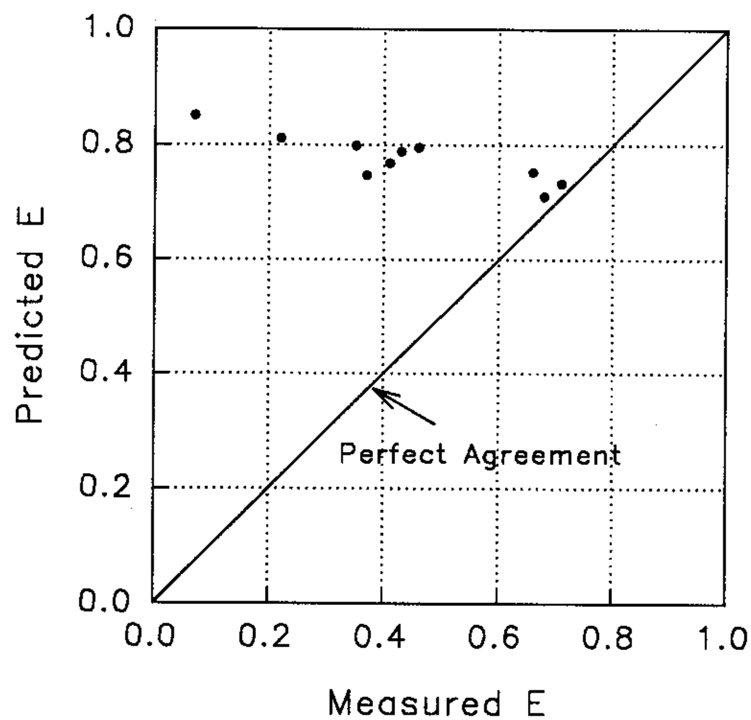

Figure D4. Avery and Novak equation for gated sills. Predicted values of gas transfer versus measured values

Thene reasoned that as the depth of the tailwater pool H increases, less flow reaches the bottom and the importance of H for gas transfer diminishes. Because of this, the fall height of the structure h becomes more crucial in estimating the amount of absorption that will occur. The coefficients of the roots in the original Avery and Novak equation were altered to improve the statistical accuracy of the adjusted formula. Figures D5–D7 show Thene's adjustment to the Avery and Novak equation.

Thene Equation

3. Thene (1988) extended the work of Elsawy and McKeogh (1977) into a predictive gas transfer equation for weirs. For a rectangular jet, Ervine and Elsawy (1975) found the air entrainment rate to predicted by:

$$\frac{Q_a}{Q_w} = 0.26 \, \frac{b}{p} \left(\frac{h}{t}\right)^{0.446} \left(1 - \frac{v_o}{v}\right) \tag{D7}$$

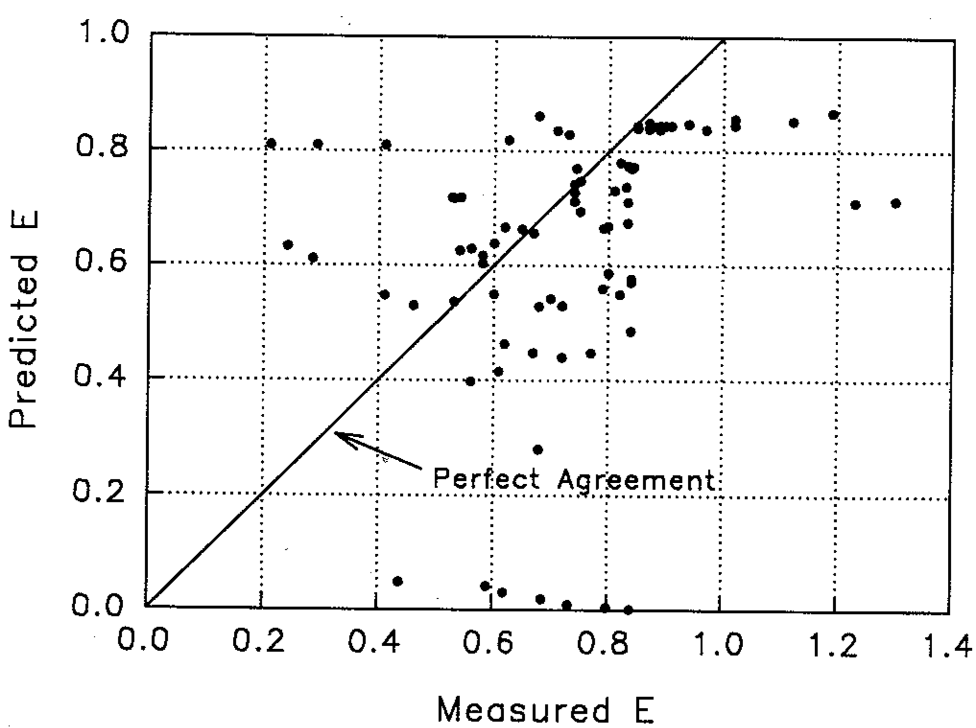

Figure D5. Thene's adjustment to the Avery and Novak equation for ogee spillways. Predicted values of gas transfer versus measured values

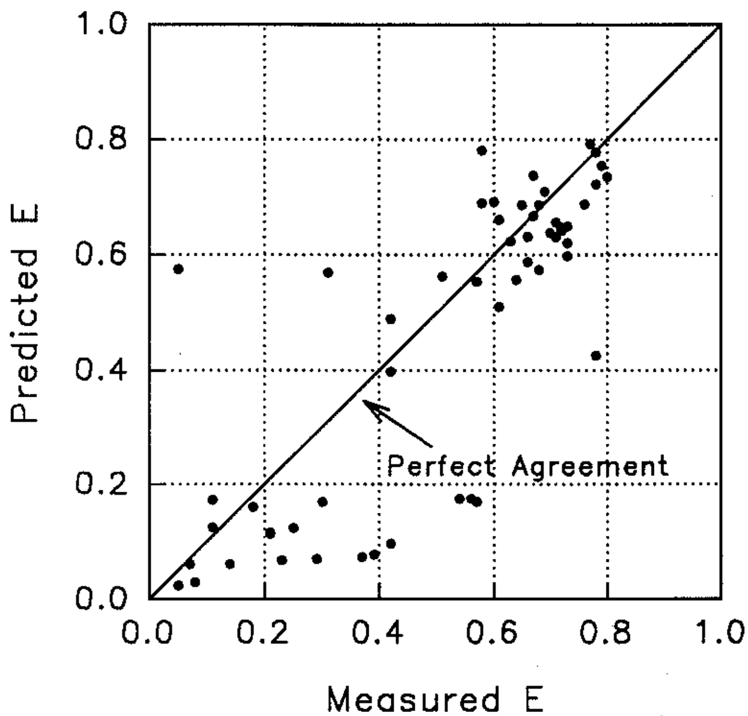

Figure D6. Thene's adjustment to the Avery and Novak equation for weirs. Predicted values of gas transfer versus measured values

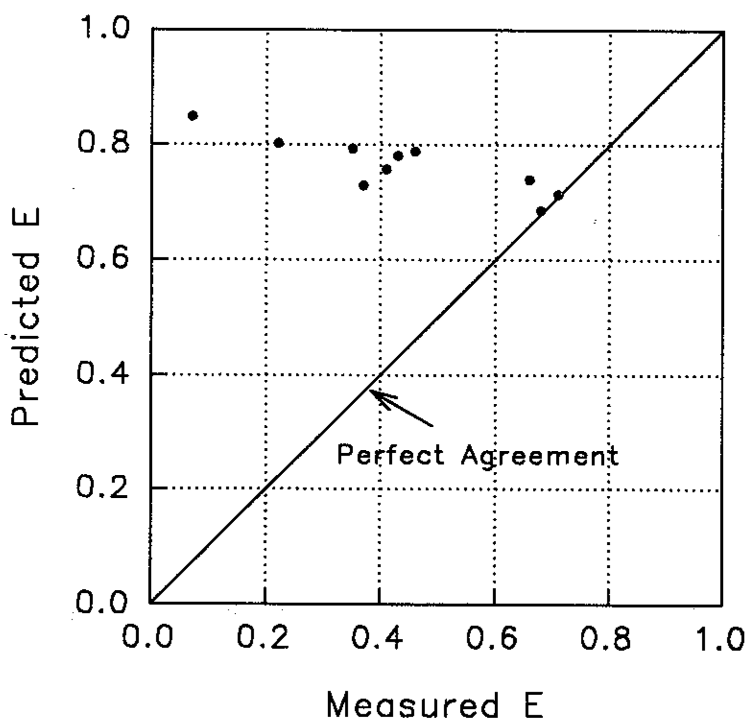

Figure D7. Thene's adjustment to the Avery and Novak equation for gated sills. Predicted values of gas transfer versus measured values

where

Q_a and Q_w = air and water discharges, respectively

b, p, t, and v = width, perimeter, thickness, and velocity of the jet, respectively

h = fall height

v_o = minimum velocity required to entrain air (1.1 m/sec).

Elsawy and McKeogh (1977) found for circular jets:

$$V_a = 1.2 \left(\frac{Fr_j}{0.5\ Fr_1 + 5.2} \right)^{3.7} d^3\ Fr_1^{1.66} \tag{D8}$$

where

V_a = volume of entrained air

Fr_j = Froude number of the jet

Fr_1 = Froude number of the jet as it exits the nozzle

d = diameter of the jet

4. Several slight modifications were made to the Elsway equations to make them applicable to weirs. Combining Equations D7 and D8, an equation for the deficit ratio r can be written as:

$$r = \exp\left(K_1 a \frac{V_a}{Q_a}\right) \tag{D9}$$

where k_1a is the liquid film coefficient. From this Thene developed a relation for the deficit ratio of:

$$\ln r = 0.156\ Fr_i^{2.69} \frac{t^2}{Q_w} \left(1 - \frac{v_o}{v}\right)^{-1} \tag{D10}$$

which can be transformed into an equation for transfer efficiency at 20 °C of:

$$E_{20} = 1 - \exp - \left[0.156\ Fr_i^{2.69} \frac{t^2}{Q_w} \left(1 - \frac{v_o}{v}\right)^{-1}\right] \tag{D11}$$

where Fr_1 is the Froude number of the flow at impact into the tailwater. Figures D8–D11 describe the performance of the equation of Thene (1988).

Nakasone Equations

5. Nakasone (1987) attempted to combine the variables of fall height, discharge, and tailwater depth to characterize the transfer efficiency at weirs. By quantifying the effects of each of these parameters on aeration efficiency, Nakasone identified zones where reaeration could be predicted, and derived the set of four equations.

a. For $(D + 1.5H_c) \leq 1.2$ m and $q \leq 235 m^3/h/m$

$$\ln r_{20} = 0.0785\,(D + 1.5H_c)^{1.31} q^{0.428} H^{0.310} \qquad \text{(D12)}$$

where

D = drop height, m

H_c = critical water depth on the weir, m

q = discharge per unit width of weir, $m^3/h/m$

r_{20} = deficit ratio at 20 °C

H = tailwater depth, m

or

$$E_{20} = 1 - \exp -\left[0.0785\,(D + 1.5H_c)^{1.31} q^{0.428} H^{0.310}\right] \qquad \text{(D13)}$$

b. For $(D + 1.5H_c) > 1.2$ m and $q \leq 235 m^3/h/m$

$$\ln r_{20} = 0.0861\,(D + 1.5H_c)^{0.816} q^{0.428} H^{0.310} \qquad \text{(D14)}$$

or

$$E_{20} = 1 - \exp -\left[0.0861\,(D + 1.5H_c)^{0.8.16} q^{0.428} H^{0.310}\right] \qquad \text{(D15)}$$

or

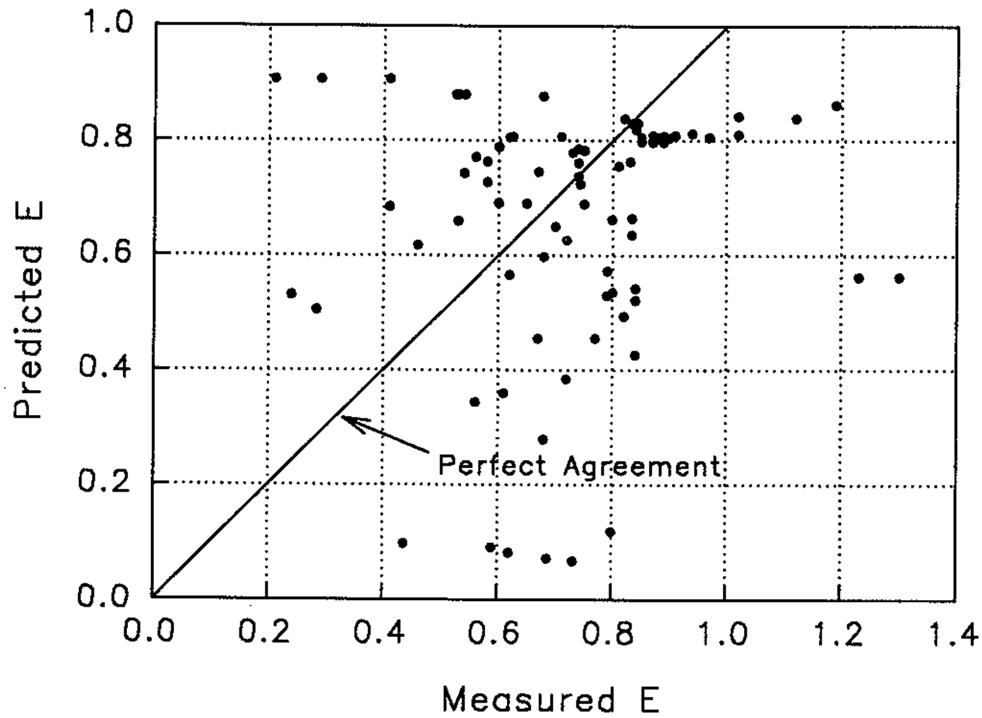

Figure D8. Thene equation for ogee spillways. Predicted values of gas transfer versus measured values

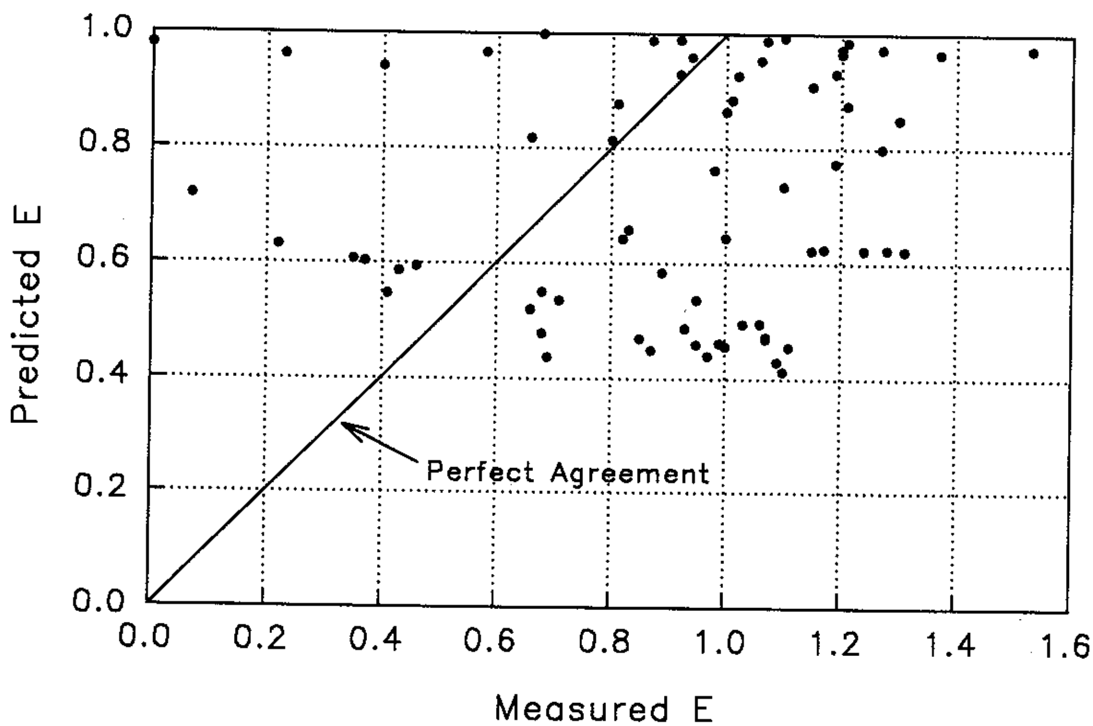

Figure D9. Thene equation for gated conduits. Predicted values of gas transfer versus measured values

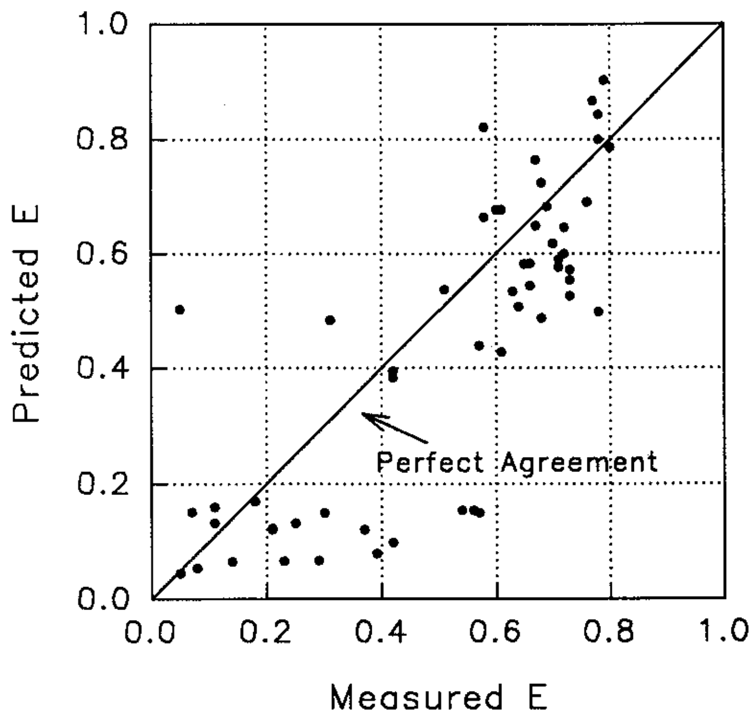

Figure D10. Thene equation for weirs. Predicted values of gas transfer versus measured values

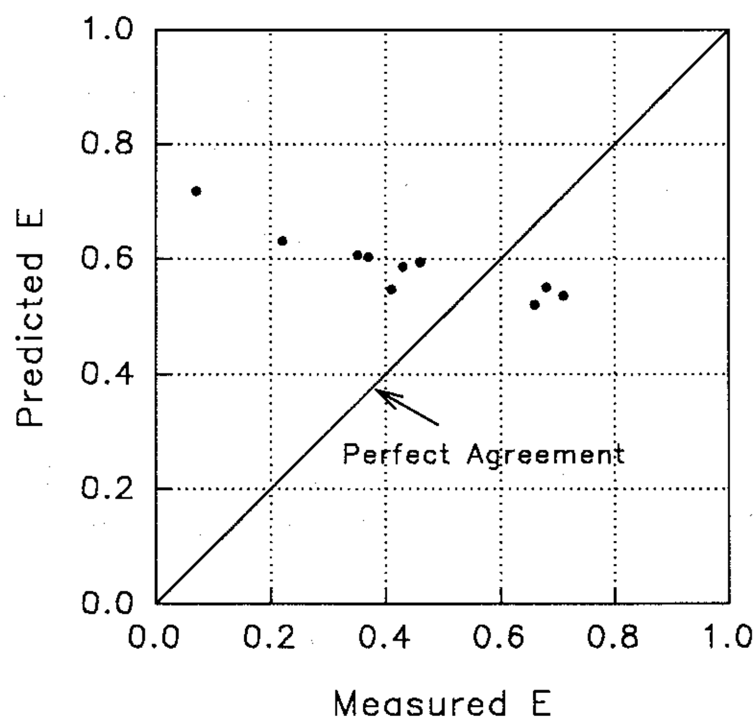

Figure D11. Thene equation for gated sills. Predicted values of gas transfer versus measured values

c. For $(D + 1.5H_c) \leq 1.2m$ and $q > 235 m^3/h/m$

$$\ln r_{20} = 5.39\,(D + 1.5H_c)^{1.31} q^{-0.363} H^{0.310} \tag{D16}$$

$$E_{20} = 1 - \exp -\left[5.39\,(D + 1.5H_c)^{1.31} q^{-0.363} H^{0.310}\right] \tag{D17}$$

d. For $(D + 1.5H_c) > 1.2$ m and $q > 235 m^3/h/m$

$$\ln r_{20} = 5.92\,(D + 1.5H_c)^{0.816} q^{-0.363} H^{0.310} \tag{D18}$$

or

$$E_{20} = 1 - \exp -\left[5.92\,(D + 1.5H_c)^{0.816} q^{-0.363} H^{0.310}\right] \tag{D19}$$

6. The results of these equations are described in Figures D12–D14.

Holler Equation

7. Holler (1970) used a form of Equation 7, main text, and proposed that the area of the air/water interface is a function of momentum change occurring in a hydraulic jump or in a jet impinging on a water surface. Using a series expansion for the exponential term in Equation 6 and reducing the variables, Holler approximated the deficit ratio by

$$r = 1 + \beta(\Delta v)^2 \tag{D20}$$

Δv is then related to the head by energy equilibrium, and for weirs and overfalls Holler presented the equation in the following form:

$$r = \beta H + 1 \tag{D21}$$

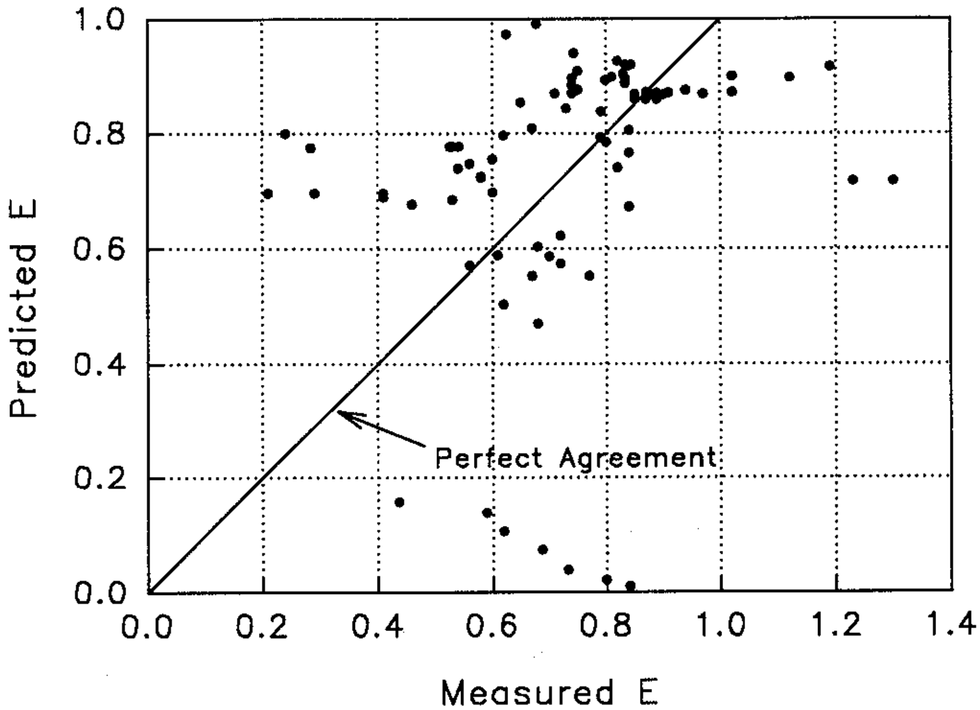

Figure D12. Nakasone equation for ogee spillways.
Predicted values of gas transfer versus measured values

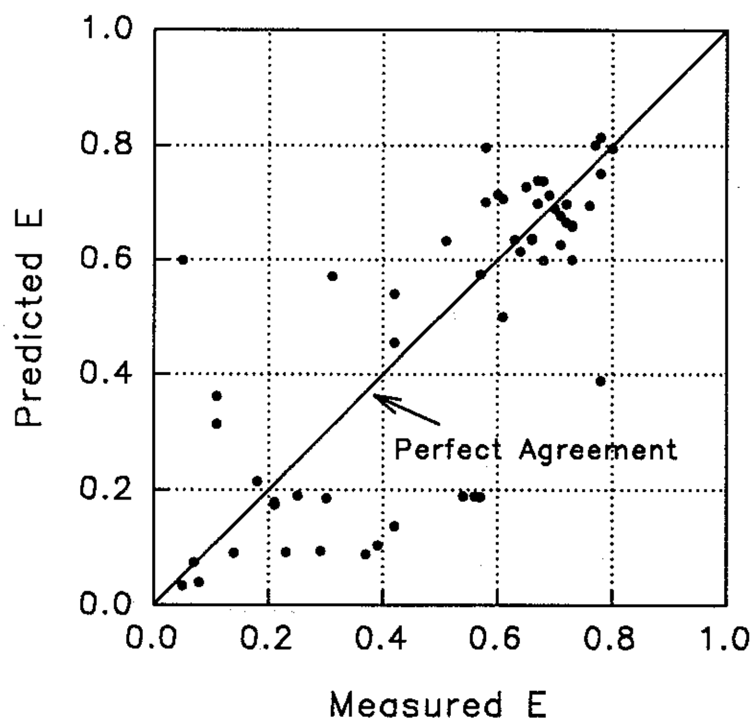

Figure D13. Nakasone equation for weirs.
Predicted values of gas transfer versus measured values

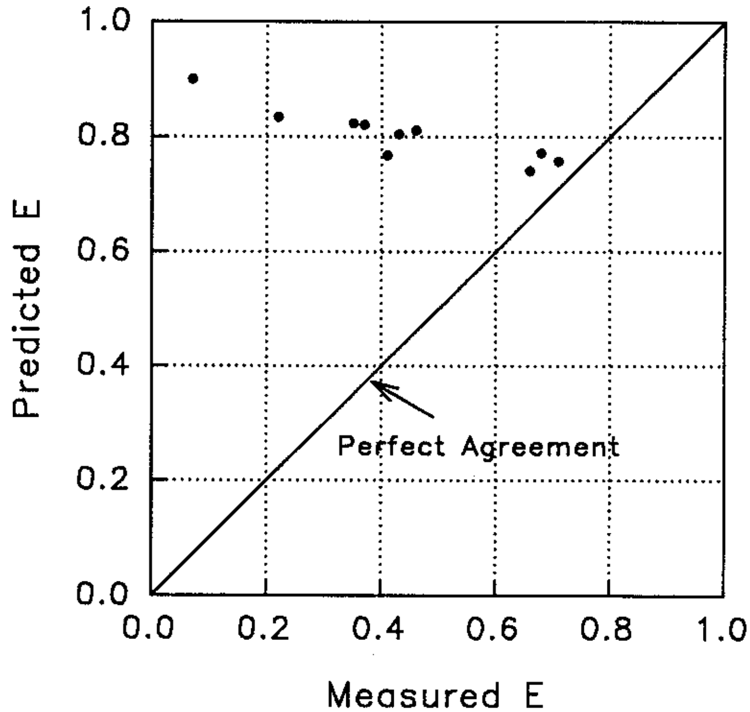

Figure D14. Nakasone equation for gated sills. Predicted values of gas transfer versus measured values

where β was equal to 0.065 ft^{-1}. This can be transformed into an equation for transfer efficiency at 20 °C:

$$E = \frac{0.21325H}{(0.21325H + 1)} \tag{D22}$$

where the head is described in metres. The results of the equation of Holler for the field data are presented in Figures D15–D18.

Tsivoglou and Wallace Equation

8. Tsivoglou and Wallace (1972) proposed a simple mathematical model that predicts gas transfer as a function of total energy dissipation. For streams the model takes the following general form:

$$K_2 = c\,\frac{\Delta E}{t_f} \tag{D23}$$

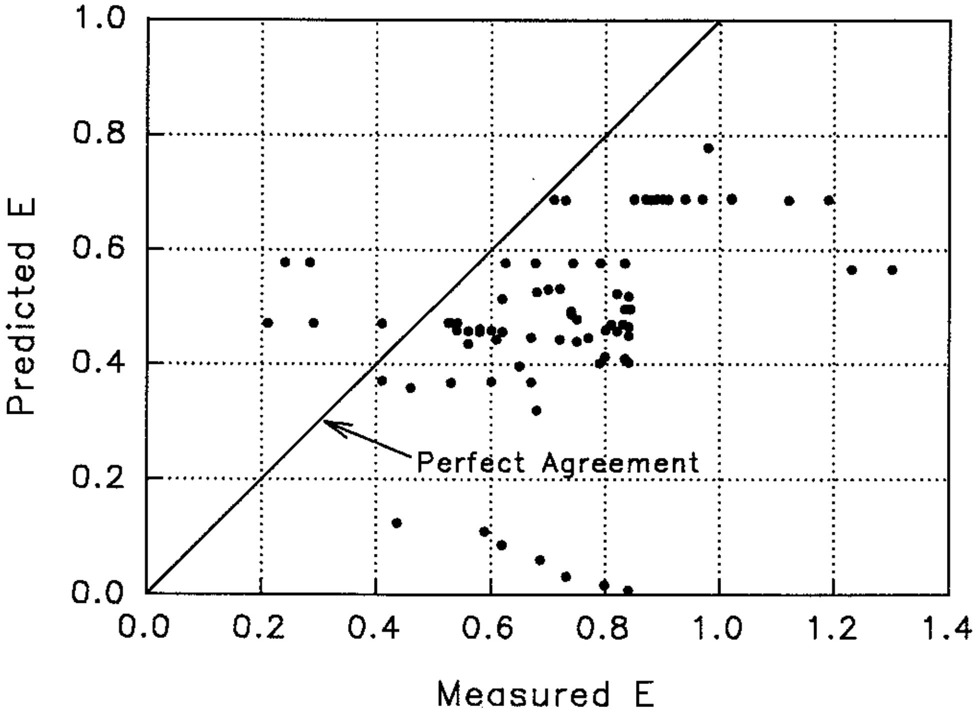

Figure D15. Holler equation for ogee spillways.
Predicted values of gas transfer versus measured values

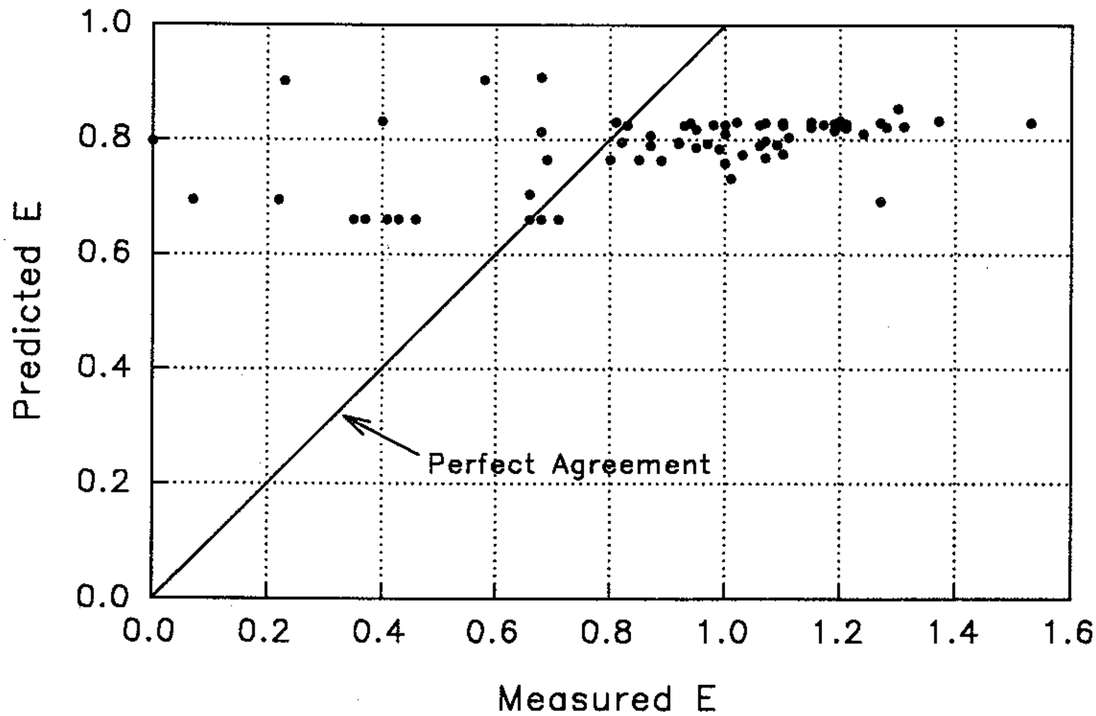

Figure D16. Holler equation for gated conduits.
Predicted values of gas transfer versus measured values

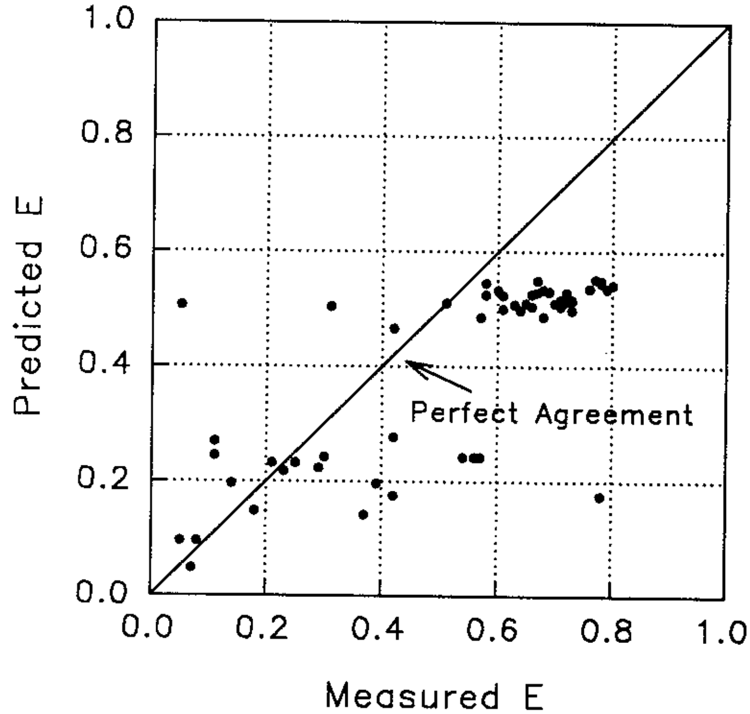

Figure D17. Holler equation for weirs. Predicted values of gas transfer versus measured values

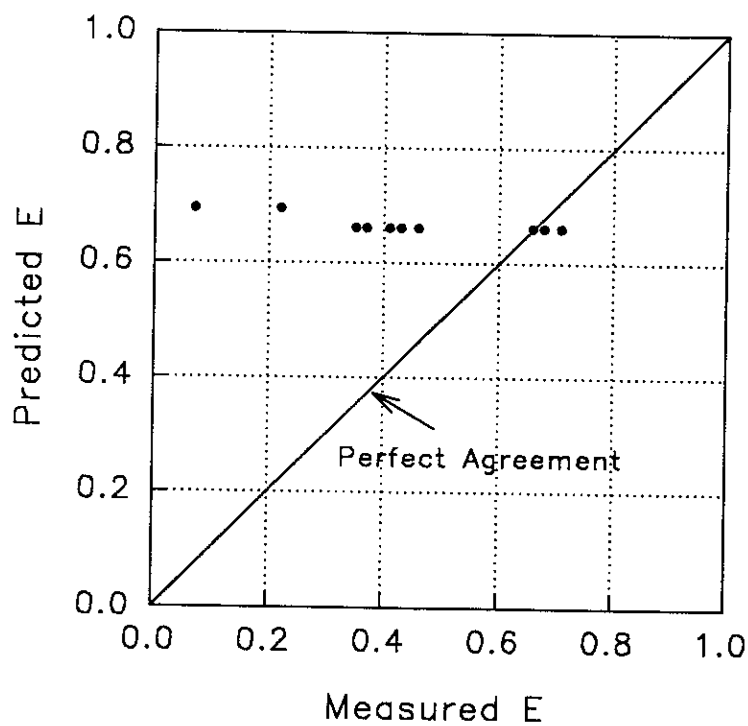

Figure D18. Holler equation for gated sills. Predicted values of gas transfer versus measured values

where

K_2 = reaeration coefficient

c = escape coefficient per foot of energy loss

ΔE = energy difference between upstream and downstream points

t_f = time of flow between the two points.

Combining this equation with the first-order reaeration equation of Streeter and Phelps (1925), Tsivoglou and Neal (1976) presented the equation in the following form:

$$E_{20} = 1 - \exp\left(-\beta h\right) \qquad \text{(D24)}$$

where β equals 0.054 ft^{-1} and h is the head in feet, or:

$$E_{20} = 1 - \exp\left(-0.1772h\right) \qquad \text{(D25)}$$

where h is described in metres. The results of the Tsivoglou and Wallace equation are presented in Figures D19–D22.

Foree Equation

9. The equation of Foree (1976) is intended to characterize the aeration that occurs over small falls and dams found in creeks. Foree took Tsivoglou and Neal's relationship and changed the coefficient β to fit his data from hydraulic structures. The efficiency at 20 °C can be written as:

$$E_{20} = 1 - \left[\exp\left(-0.5249h\right)\right]^{0.9032} \qquad \text{(D26)}$$

with head measured in metres. The applicability of the Foree equation to low-head structures on rivers is described by Figures D23–D26.

Preul and Holler Equation

10. The reaeration prediction equation developed by Preul and Holler (1969) was intended to be a generic gas transfer equation for low run of the

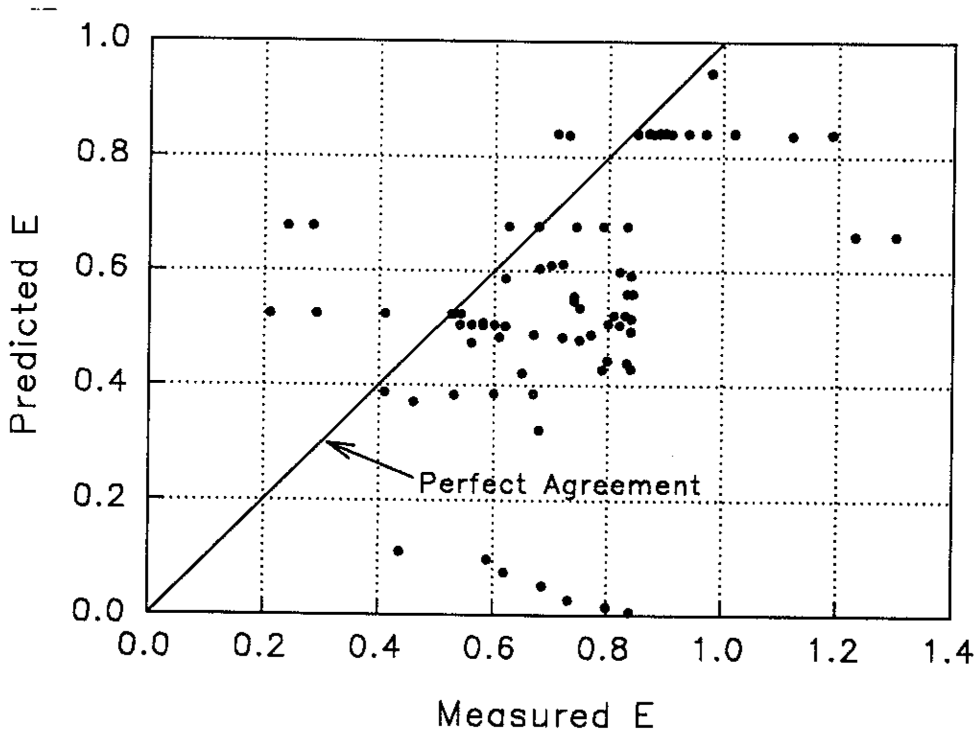

Figure D19. Tsivoglou and Wallace equation for ogee spillways. Predicted values of gas transfer versus measured values

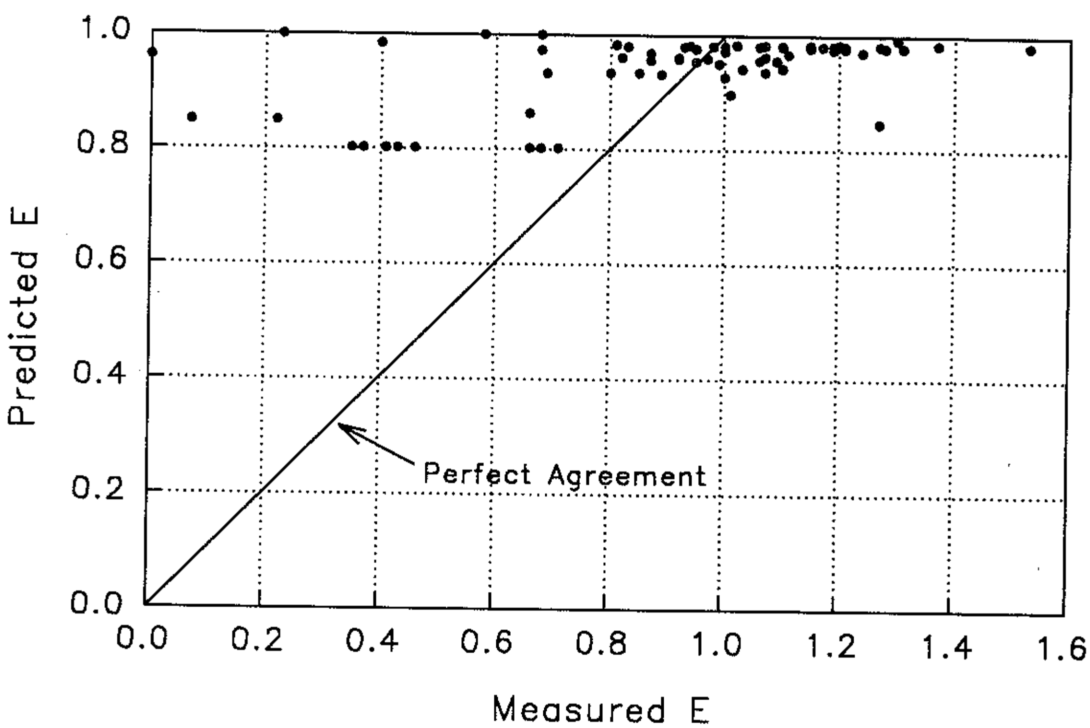

Figure D20. Tsivoglou and Wallace equation for gated conduits. Predicted values of gas transfer versus measured values

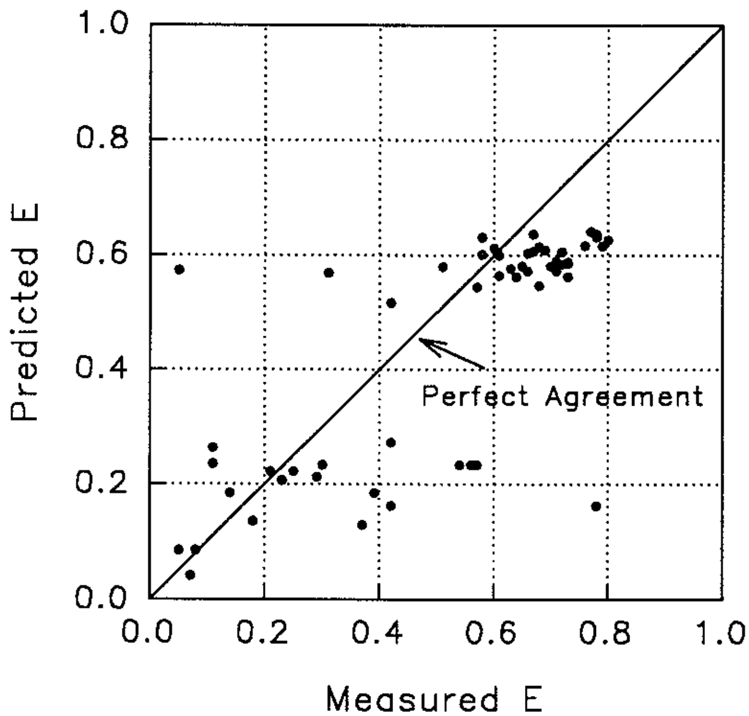

Figure D21. Tsivoglou and Wallace equation for weirs. Predicted values of gas transfer versus measured values

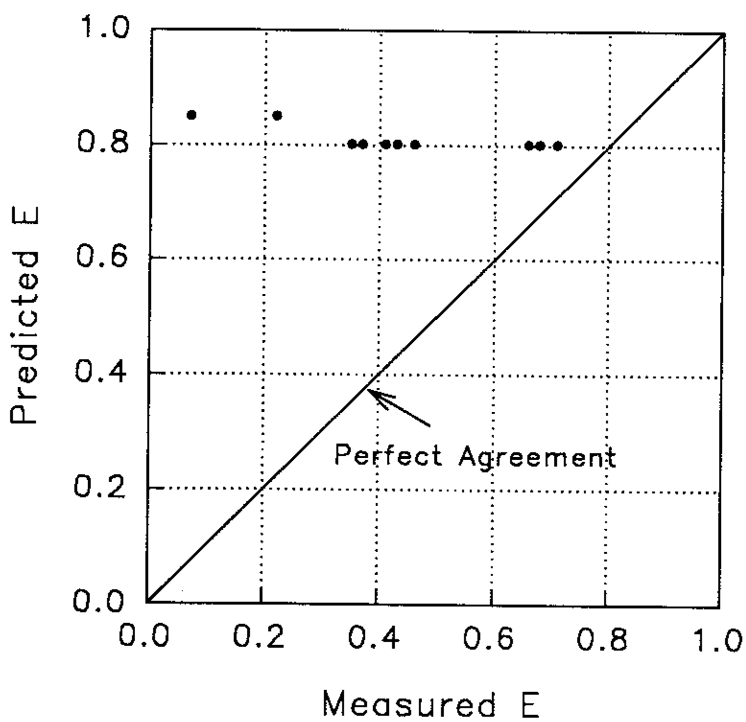

Figure D22. Tsivoglou and Wallace equation for gated sills. Predicted values of gas transfer versus measured values

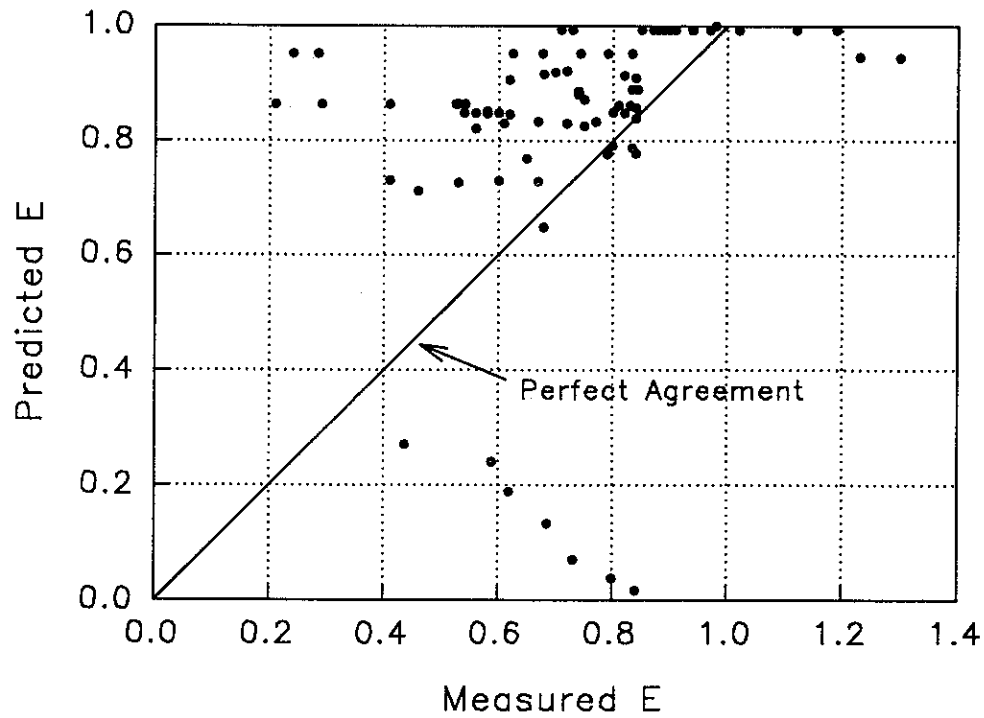

Figure D23. Foree equation for ogee spillways. Predicted values of gas transfer versus measured values

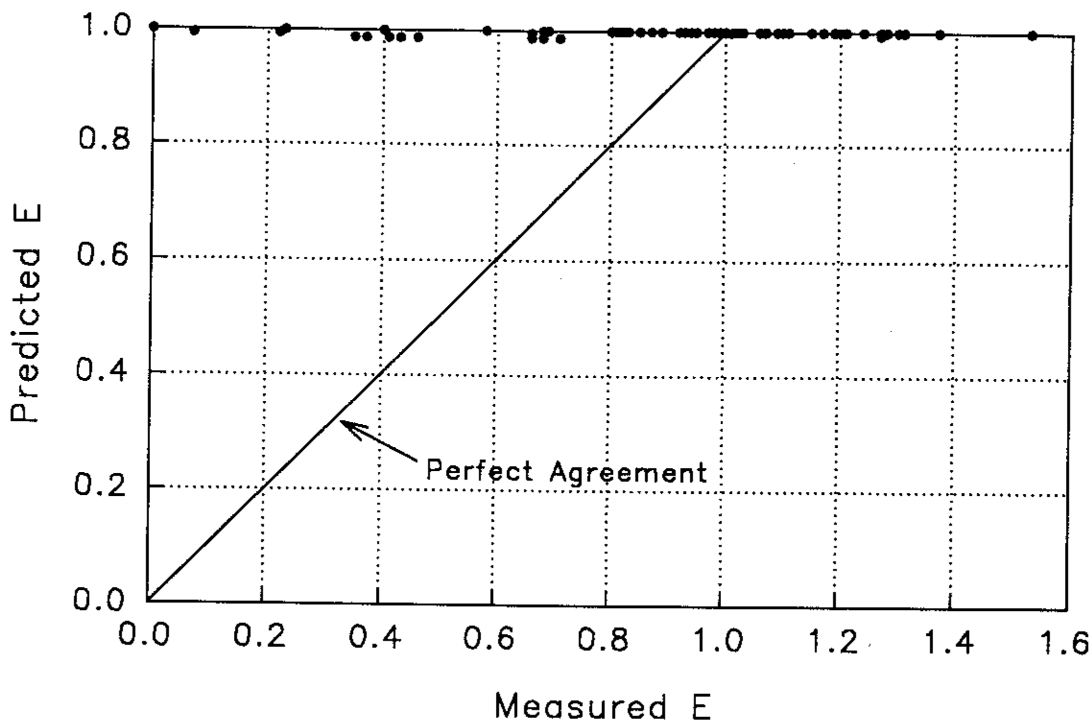

Figure D24. Foree equation for gated conduits. Predicted values of gas transfer versus measured values

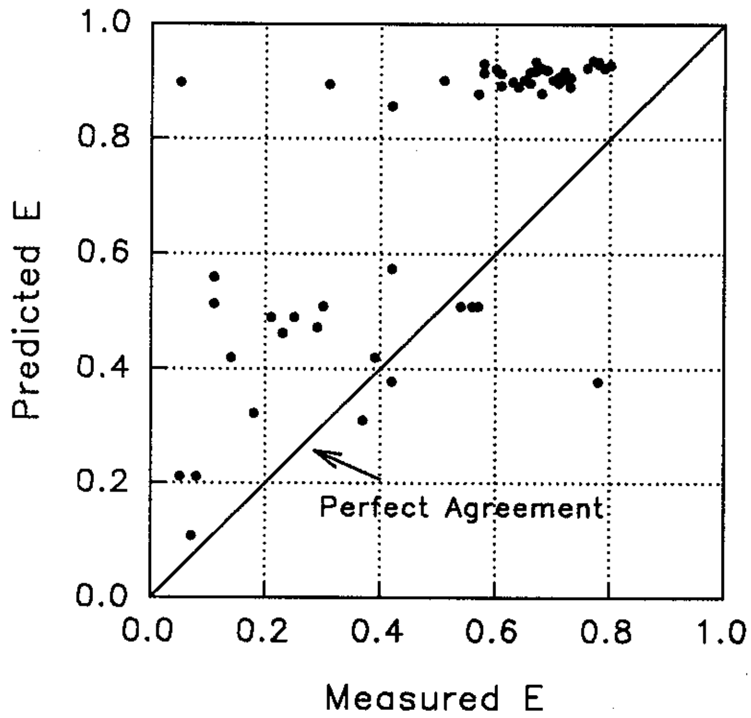

Figure D25. Foree equation for weirs. Predicted values of gas transfer versus measured values

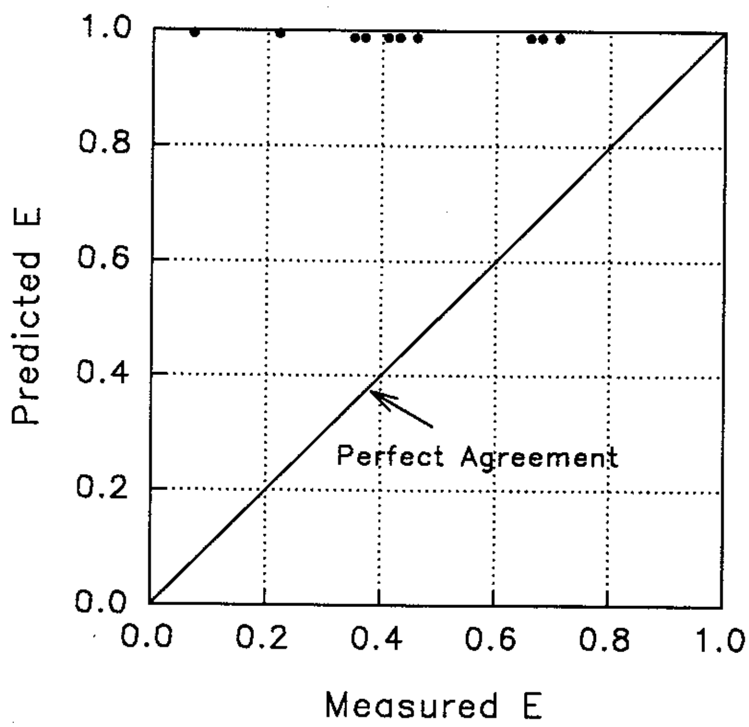

Figure D26. Foree equation for gated sills. Predicted values of gas transfer versus measured values river dams. Among the many parameters that they identified, Preul and Holler narrowed the scope of interest to just five terms, which in turn could be related to just one term. By using the Froude number of the flow before the hydraulic jump, a relation for the deficit ratio is presented as:

$$r_{20} = 1 + 666\ N_f^{-3.33} \tag{D27}$$

where N_f is the Froude number at impact. This is rewritten in terms of efficiency at 20 °C as:

$$E_{20} = 1 - \frac{1}{\left(1 + 666\ N_f^{-3.33}\right)} \tag{D28}$$

Figures D27–D30 present the comparison of the predicted values to actual measured values of transfer efficiency.

Rindels and Gulliver Equation

11. Rindels and Gulliver (1991) presented an equation of the form:

$$E_{20} = 1 - \exp\left(\frac{-0.08h}{1 + 0.02q} - 0.062Z_p\right) \tag{D29}$$

where the values are all in non-SI units, or:

$$E_{20} = 1 - \exp\left(\frac{-0.2625h}{1 + 0.2153q} - 0.2034Z_p\right) \tag{D30}$$

where Z_p is equal to the tailwater depth, h is the head across the structure, and q is the discharge per unit width (all values in SI units). The intention was to further refine the Tsivoglou and Wallace (1972) equation by incorporating the effects of tailwater pool depth on aeration with the effects of spillway height. The authors reasoned that transfer efficiency

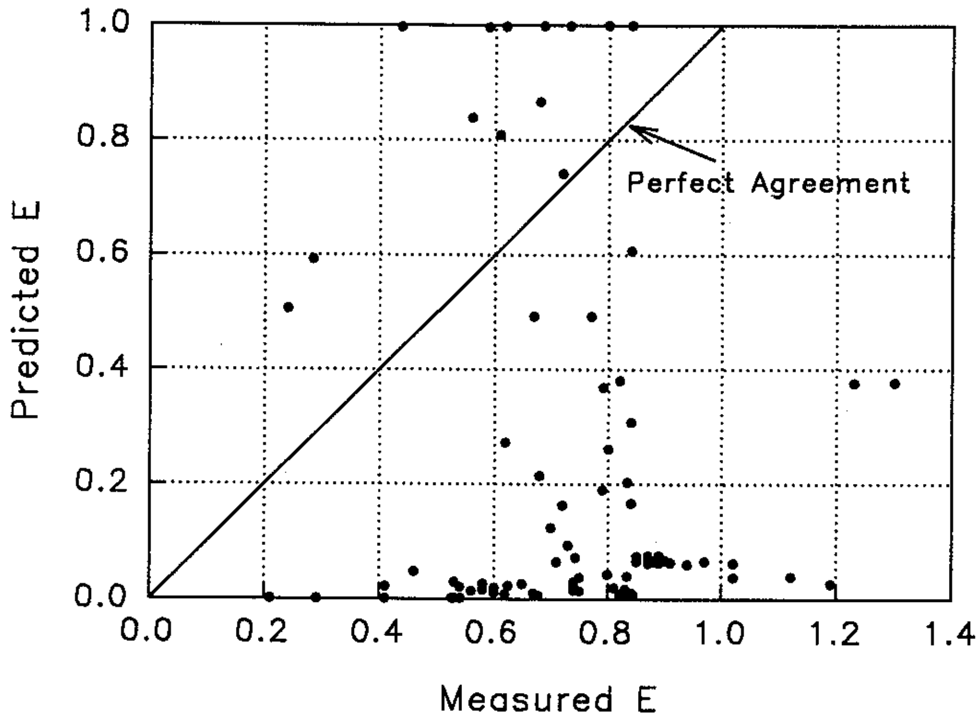

Figure D27. Preul and Holler equation for ogee spillways. Predicted values of gas transfer versus measured values

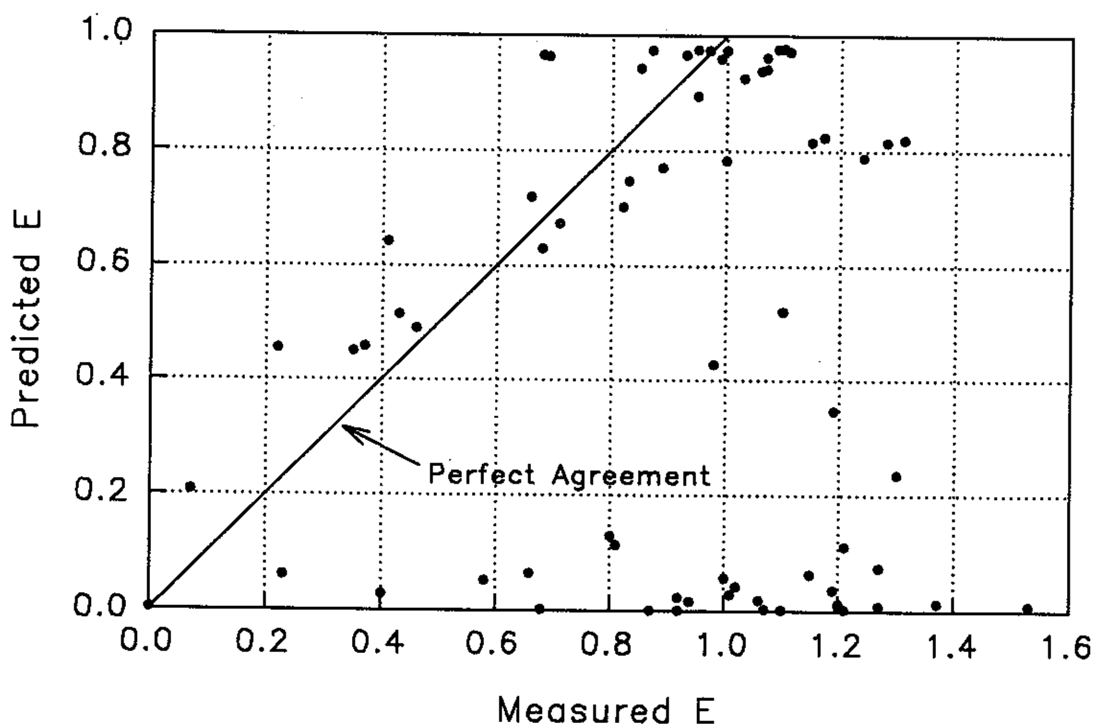

Figure D28. Preul and Holler equation for gated conduits. Predicted values of gas transfer versus measured values

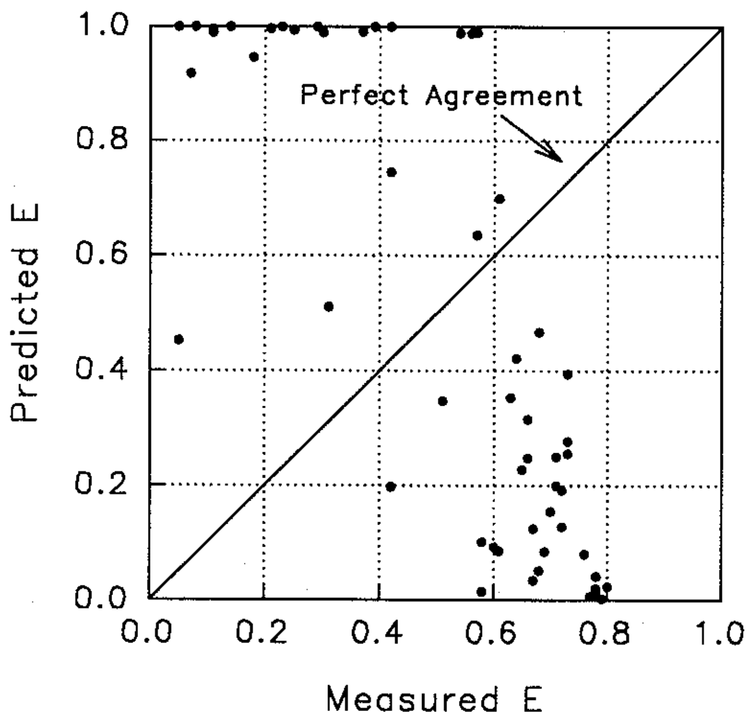

Figure D29. Preul and Holler equation for weirs. Predicted values of gas transfer versus measured values

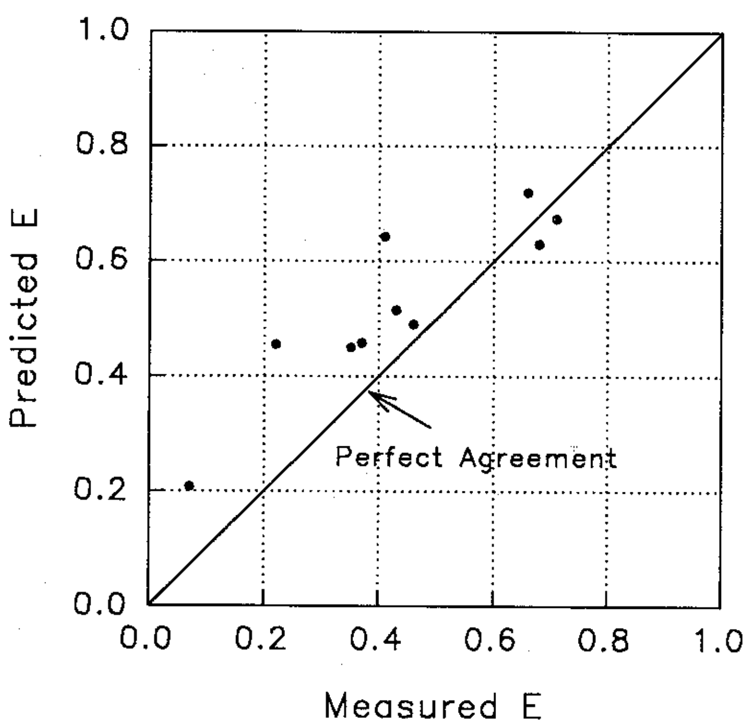

Figure D30. Preul and Holler equation for gated sills. Predicted values of gas transfer versus measured values would be proportional to the depth of tailwater and height of spillway and would be indirectly proportional to the unit discharge.

12. The results of the equation of Rindels and Gulliver are presented in Figures D31–D33.

Wilhelms (1988)

13. Wilhelms examined the hydraulic conditions at gated sill structures and, using an exponential form of equation, empirically related the oxygen transfer to the discharge, head loss across the structure, and submergence of the gate lip. The exponent in the gas transfer equation was directly related to the discharge and head loss and inversely related to submergence:

$$E_{20} = 1 - \exp\left[-0.000797\,\frac{hq}{s} - 0.188\right] \qquad \text{(D31)}$$

where

h = head across the structure, ft

q = unit discharge, ft^3/sec

s = submergence of gate lip, ft

Caution was advised in using the equation because of prediction errors for very small submergences or for large discharges that change the hydraulic action in the structure stilling basin. Figures D34–D36 show the results for the Wilhelms equation.

Wilhelms and Smith Adjustment to Tsivoglou Equation

14. Wilhelms and Smith (1981) modified the coefficient in the equation of Tsivoglou and Wallace (1972) for a data set that consists mostly of gated conduits. For their data set the following relation best predicted the transfer efficiency:

$$E = 1 - \exp(-0.1476h) \qquad \text{(D32)}$$

where h is the head across the structure. Figures D37–D40 present the results of the Wilhelms and Smith equation for several types of structures.

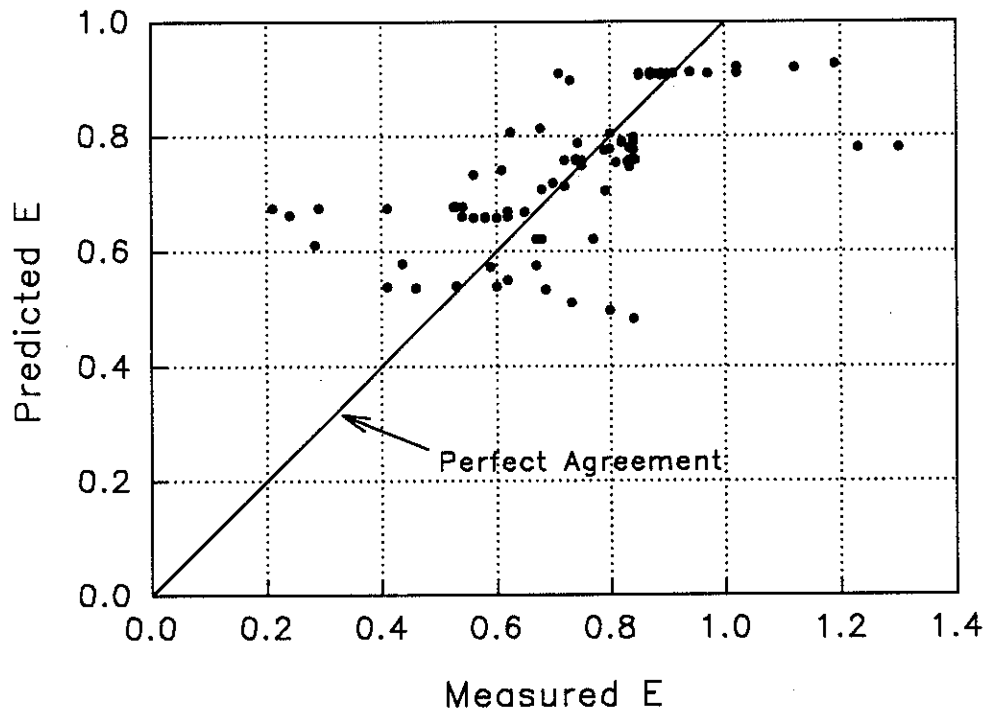

Figure D31. Rindels and Gulliver equation for ogee spillways. Predicted values of gas transfer versus measured values

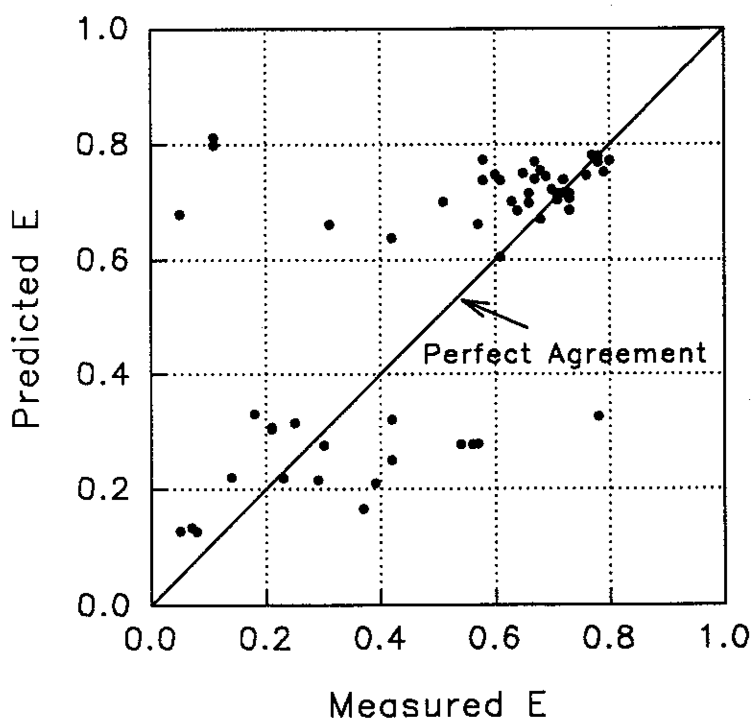

Figure D32. Rindels and Gulliver equation for weirs. Predicted values of gas transfer versus measured values

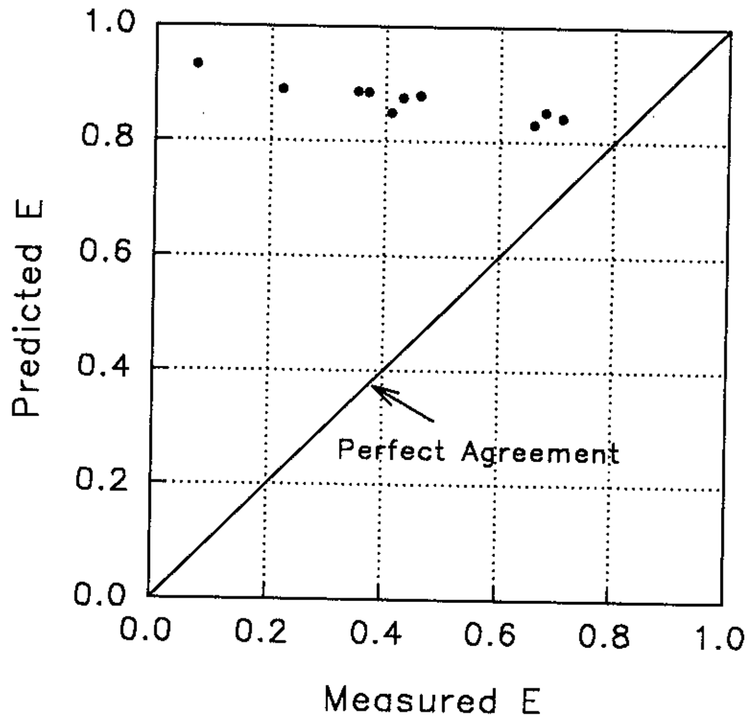

Figure D33. Rindels and Gulliver equation for gated sills. Predicted values of gas transfer versus measured values

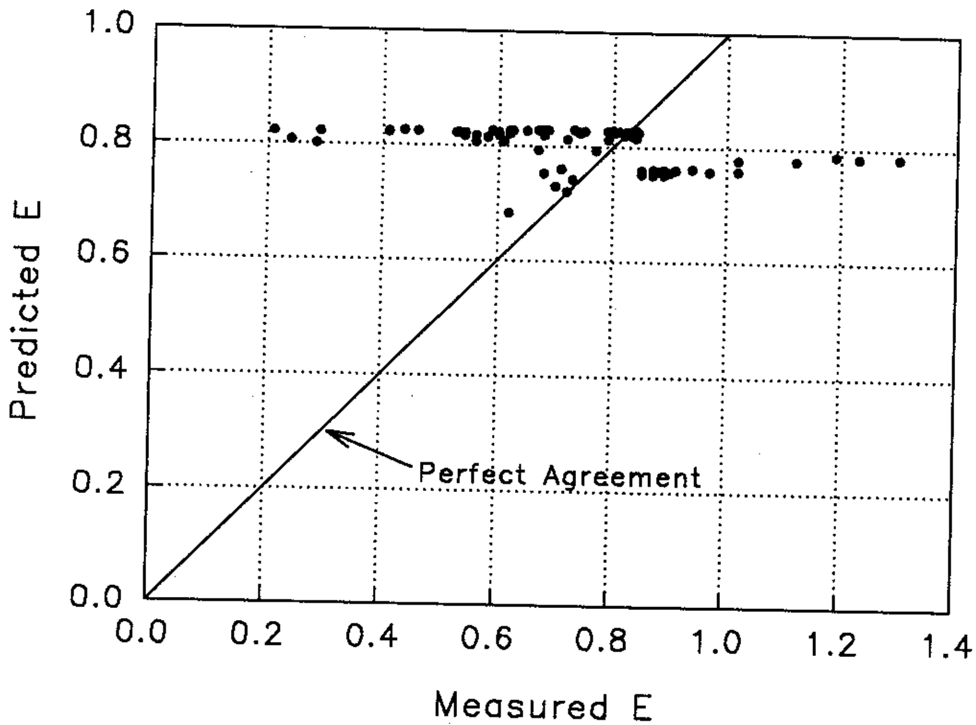

Figure D34. Wilhelms equation for ogee spillways. Predicted values of gas transfer versus measured values

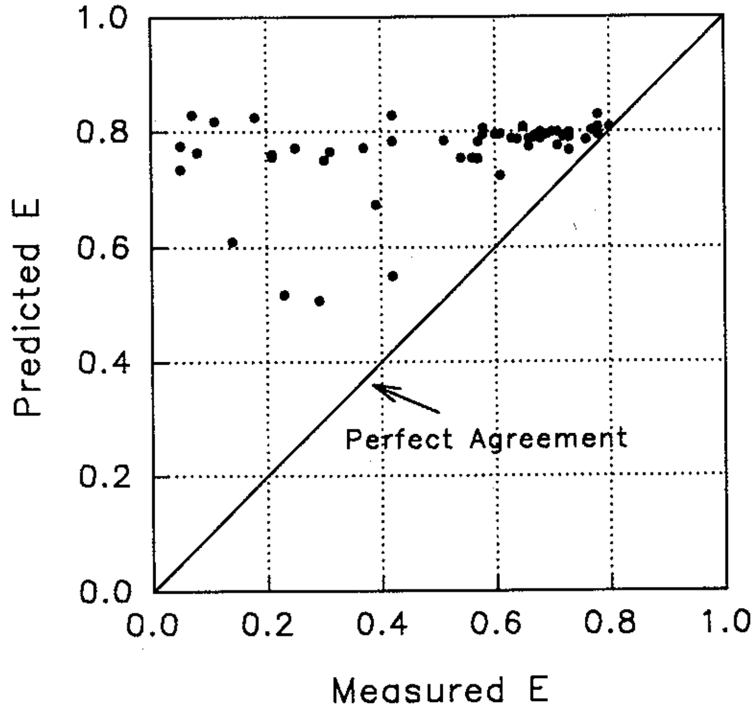

Figure D35. Wilhelms equation for weirs. Predicted values of gas transfer versus measured values

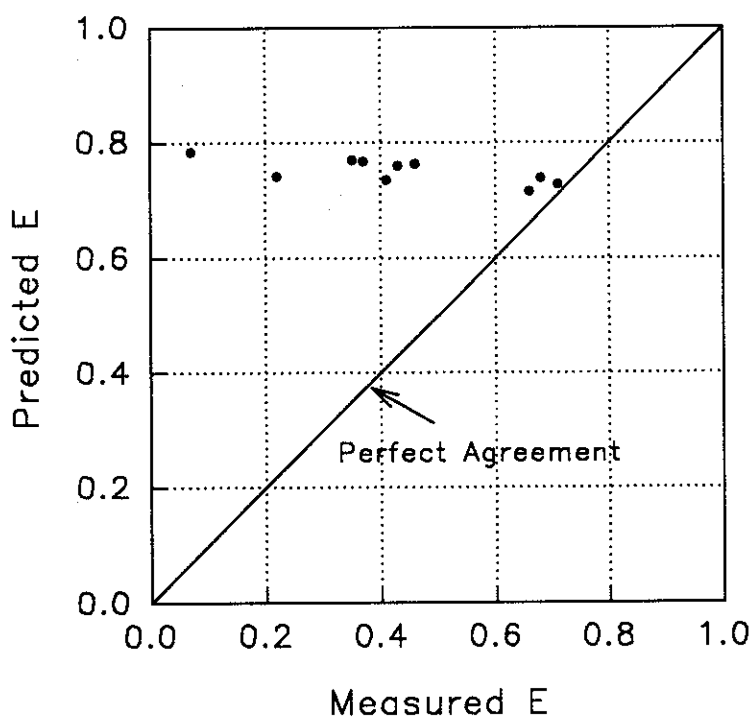

Figure D36. Wilhelms equation for gated sills. Predicted values of gas transfer versus measured values

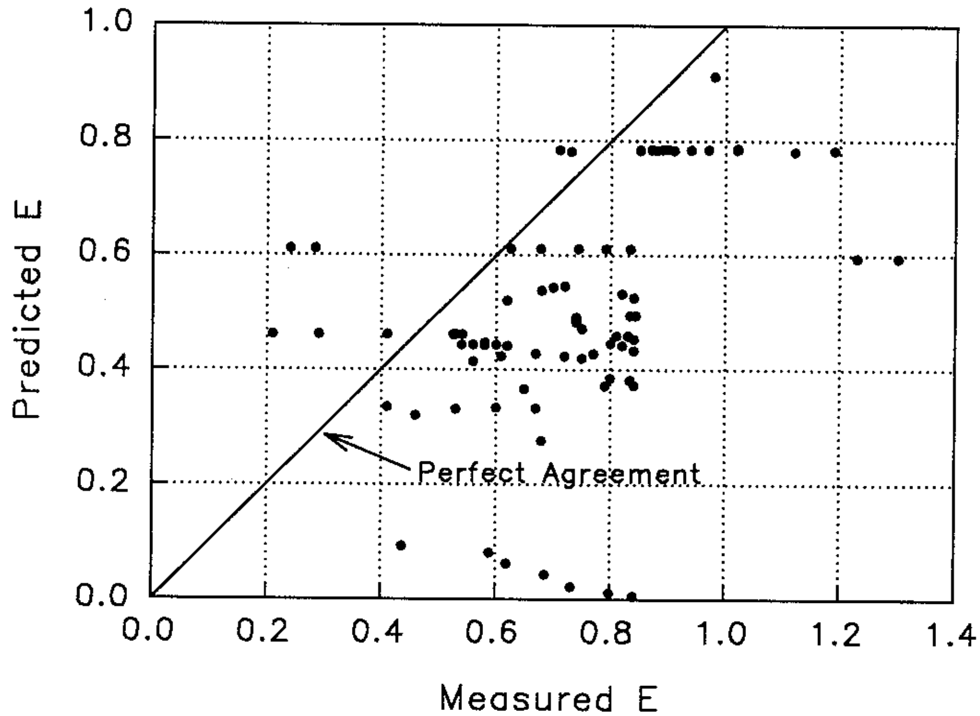

Figure D37. Wilhelms and Smith equation for ogee spillways. Predicted values of gas transfer versus measured values

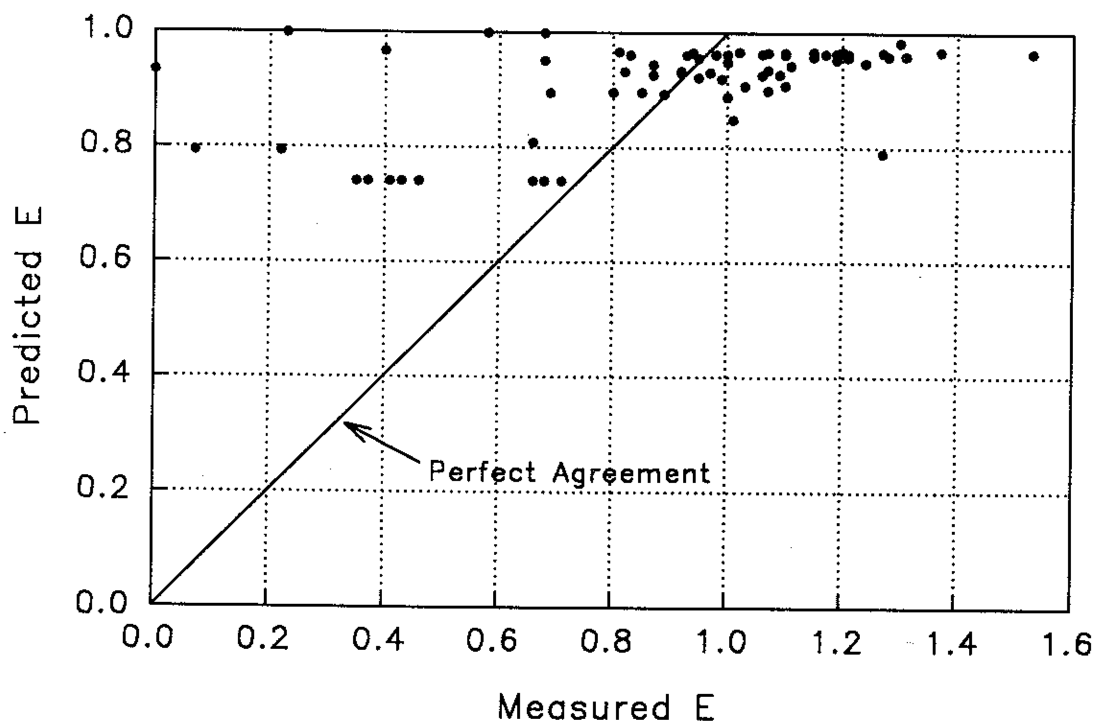

Figure D38. Wilhelms and Smith equation for gated conduits. Predicted values of gas transfer versus measured values

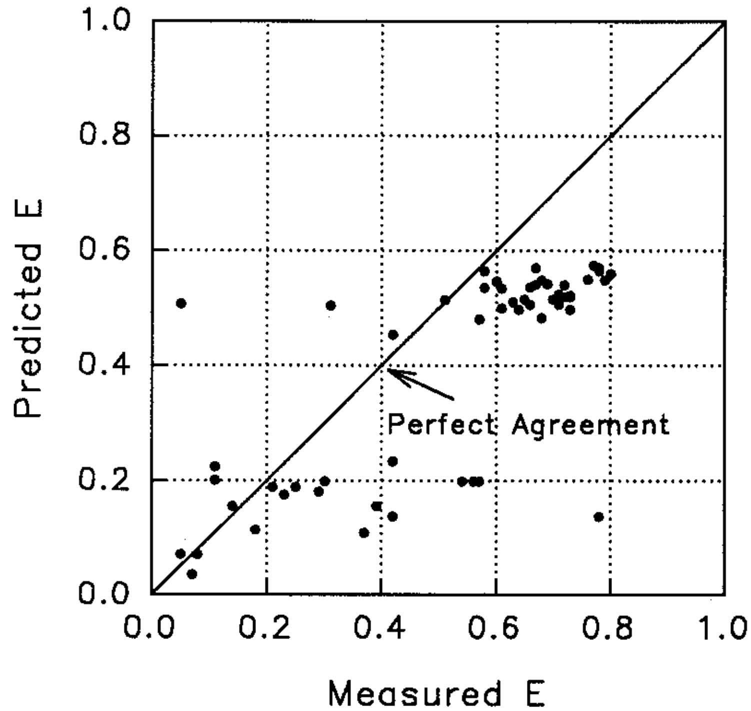

Figure D39. Wilhelms and Smith equation for weirs. Predicted values of gas transfer versus measured values

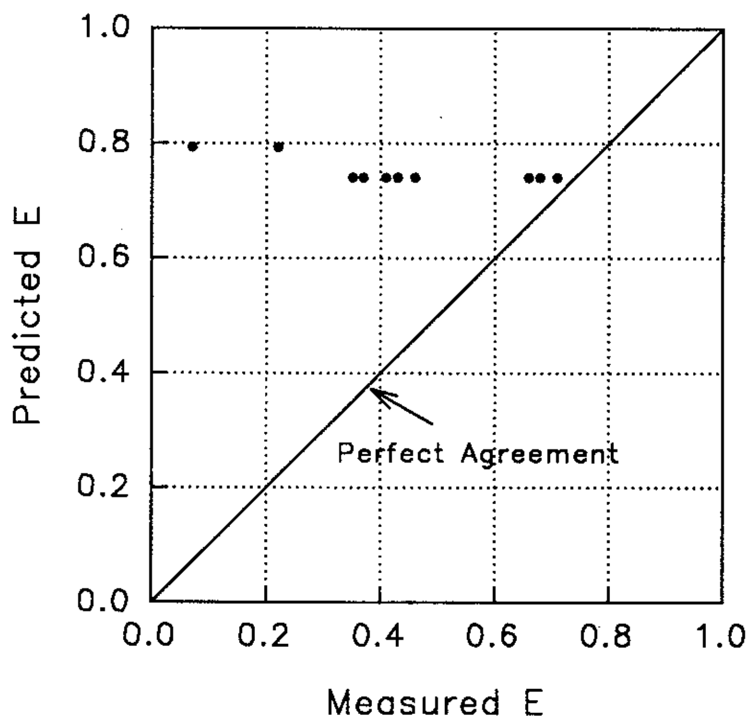

Figure D40. Wilhelms and Smith equation for gated sills. Predicted values of gas transfer versus measured values

APPENDIX E: NOTATION

Main Text

a	Specific surface area, A/V
A	Surface area associated with the volume V , over which transfer occurs
B_c	Bias uncertainty in the measurement of C_i and C_f ; bias uncertainty in DO concentration due to the titrant
B_{c_s}	Bias uncertainty in C_s
B_E	Bias uncertainty in the transfer efficiency
C	Ambient concentration in water
C_{Cl}	Concentration of chloride ions, g/ℓ
C_f	Final oxygen concentration
C_i	Initial oxygen concentration
C_s	Saturation concentration
C_{si}	Saturation concentration of gas i , g/m^3
dC	Change in concentration across the interface thickness
dC/dt	Rate of change of concentration
dD/dt	Rate of change of the saturation deficit
ds	Interface thickness
D	Saturation deficit
D_f, D_i	Final and initial oxygen deficits, respectively
D_m	Molecular diffusion coefficient
E	Transfer efficiency
$\hat{E}$	Best estimate of the true value of transfer efficiency that is predicted by the equation
E_m, E_p	Measured and predicted oxygen transfer efficiency, respectively
E_{20}, E_T	Gas transfer efficiencies at 20 °C and at T °C
F_j	Froude number of the jet
g	Acceleration due to gravity, m/sec^2
h	Head loss, m
H	Tailwater depth, m
H_c	Critical water depth on the weir, m
H_i	Henry's law constant for gas i , g/m^3 - atm
J	Mass flux per unit area across an interface or a plane
k_L	Liquid film coefficient for oxygen transfer
K_2	Reaeration or oxygen transfer coefficient
M	Elevation above mean sea level, m

n_{c_i}, n_{c_f} Number of upstream and downstream measurements, respectively

N_f Froude number at the point of impact

p Probability of the confidence interval on efficiency

P_{atm} Reported pressure, mm of mercury

P_i Partial pressure of gas i in the atmosphere, atm

q Specific discharge, volume/time/length; discharge over the structure

Q Total discharge through a structure

r Deficit ratio

R Reynolds number

R_h Hydraulic radius of the flow in the conduit

S Gate submergence, m

t Elapsed time from initial to final deficits

t^* Student's t-value corresponding to $n_{c_i} - 1$ or $n_{c_f} - 1$ degrees of freedom and a 95 percent confidence interval

T Temperature

U Uncertainty

U_E Total uncertainty in the transfer efficiency

V Volume of the water body over which A is measured

W_{c_i}, W_{c_f} Precision uncertainties in C_i and C_f, respectively

W_E Precision uncertainty in the transfer efficiency

ν Kinematic viscosity

$\sigma_{c_i}, \sigma_{c_f}$ Standard deviation of the upstream and downstream measurements, respectively

Appendix D

b, p, t, v Width, perimeter, thickness, and velocity of the jet, respectively

c Escape coefficient per foot of energy loss

d Diameter of the jet

D Drop height, m

E Efficiency

E_T Gas transfer efficiency at a given temperature T

f_T Indexing coefficient

Fr_i Froude number of the flow at impact into the tailwater

Fr_j Froude number of the jet

Fr_1	Froude number of the jet as it exits the nozzle
g	Acceleration due to gravity
h	Head across the structure; fall height
H	Depth of the tailwater pool
H_c	Critical water depth on the weir
K_1a	Liquid film coefficient
K_2	Reaeration coefficient
N_f	Froude number at impact
q	Discharge over the structure; discharge per unit width of weir, $m^3/h/m$
Q_a	Air discharge
Q_w	Water discharge
r	Deficit ratio
r_{15}	Deficit ratio at 15 °C
r_{20}	Deficit ratio at 20 °C
R	Reynolds number of the jet
s	Submergence of gate lip, ft
t_f	Time of flow between the two points
v_o	Minimum velocity required to entrain air (1.1 m/sec)
V_a	Volume of entrained air
Z_p	Tailwater depth
ΔE	Energy difference between upstream and downstream points
ν	Kinematic viscosity

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13. ABSTRACT *(Maximum 200 words)*

The physics of gas transfer is presented as a basis for describing the physical processes that contribute to the oxygen transfer (reaeration) characteristics of a hydraulic structure. A mathematical description of the gas transfer process is presented and the effects of the physical processes on the variables in the equation are described. Measurement techniques and uncertainties are discussed. Guidance is presented on the conduct of field studies. Based on the uncertainty in direct measurement of dissolved oxygen, guidance is presented to help determine the need for alternative measurement techniques, i.e., use of tracer gases. Oxygen transfer for several types of low-head structures is characterized on the basis of the theory and mathematical description with an empirical coefficient derived from field measurements. The uncertainty in predictions made with these descriptions is quantified. Enhancement techniques, including operational strategies, mechanical alternatives, and structural modifications, are discussed.

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