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UNITED STATES ENVIRONMENTAL PROTECTION AGENCY
WASHINGTON, D.C. 20460

SEP 28 1984

OFFICE OF WATER

MEMORANDUM

SUBJECT: Technical Guidance Manual for Performing Waste Load Allocations Book VII, Permit Averaging Periods

TO: Regional Water Management Division Directors
Regional Environmental Services Division Directors
Regional Wasteload Allocation Coordinators

Attached, for national use, is the final version of the Technical Guidance Manual for Performing Waste Load Allocations, Book VII, Permit Averaging Periods. We are sending extra copies of this manual to the Regional Wasteload Allocation Coordinators for distribution to the States to use in conducting waste load allocations.

Modifications to the February 1984 draft include:

- The method to calculate the Reductions Factor in Chapter 2 has been elaborated to include the use of 95% cut-offs for frequency of permit violations.

- The example calculation in Chapter 3 has been expanded. Step 7 has been added to the step-procedure to show how permit limits can be specified using 95% cut-offs for frequency of permit violations.

- The document recommends that advanced treatment facilities should be built to meet the long-term average and the selected effluent variability.

- A flow diagram and an IBM PC-compatible program have been added to Appendix D.

If you have any questions or comments or desire additional information please contact Tim S. Stuart, Chief, Monitoring Branch, Monitoring and Data Support Division (WH-553) on (FTS) 382-7074.

Edwin L. Johnson

Edwin L. Johnson, Director
Office of Water Regulations and Standards (WH-551)

Attachment

Technical Guidance Manual for Performing Waste Load Allocations

Book VII: Permit Averaging Periods

September 1984
Final Report

for

Office of Water Regulations and Standards
Monitoring and Data Support Division,
Monitoring Branch
U.S. Environmental Protection Agency
401 M Street, S.W. Washington, D.C. 20460

TECHNICAL GUIDANCE MANUAL FOR PERFORMING WASTE LOAD ALLOCATIONS

Book VII

Permit Averaging Periods

Contract Number 68-03-3131-WA9

Project Officer

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FOREWORD

This guidance document is a product of several years of research on many complex water quality issues. Although much progress has been made, some issues still remain. User participation will be needed to develop answers to these unresolved issues and will be key to future revisions of this document.

Selection of permit averaging periods, as presented in this manual, is based on an assumed exceedance frequency of an acute violation in the stream no more than 1 day in 10 years. The EPA is currently considering the issue of allowable duration and frequency of exposure to acute as well as chronic toxicity. Based on this study, the choice of duration and frequency used in this document as examples may have to be changed.

CONTENTS

CONTENTS

(Continued)

LIST OF TABLES

LIST OF FIGURES

LIST OF FIGURES
(continued)

LIST OF ABBREVIATIONS AND SYMBOLS

BASIC	Computer language
BOD	Biochemical oxygen demand
BOD_5	The amount of dissolved oxygen consumed in five days by biological oxidation of organic matter
CE	Treatment plant effluent concentration
CFS	Cubic feet per second, unit of flow
CL	Concentration equal to a water quality standard
CO	Downstream concentration, after complete mixing
CRT	Cathode ray tube
C_{sat}	Saturation concentration of dissolved oxygen
CS	Stream concentration upstream of discharge
D	Flow ratio, equal to QS/QE
D_c	Critical (or maximum) dissolved oxygen deficit
DO	Dissolved oxygen
EL	Effluent limit. A maximum effluent concentration determined from a waste load allocation analysis, and specified by an NPDES permit
FAV	Final acute value
FCV	Final chronic value
K	Stream purification factor
Ka	Stream reaeration rate constant
Kd	BOD oxidation rate constant
MRI	Mean recurrence interval, expressed in years
NPDES	National pollutant discharge elimination system
P	Pollutant

LIST OF ABBREVIATIONS AND SYMBOLS

(Continued)

PDM-PS	Probabilistic dilution model: point source
POTW	Publicly-owned treatment works
Pr	Probability
7Q10	The lowest 7-day average stream flow with a recurrence interval of 10 years
QE	Treatment plant effluent flow
QS	Stream flow
QT	Total downstream flow, equal to QS + QE
R	Reduction factor, equal to the ratio of the mean CE for which a treatment plant is designed to the EL
TSS	Total suspended solids
WLA	Waste load allocation
WQ	Water quality
α	Exceedence probability
β	Dimensionless unit of concentration equal to CO/CL
μ_x	Mean value of x
Φ	Dilution factor
σ_x	Standard deviation of x
ν_x	Coefficient of variation of x
Z_α	Value of statistical parameter Z for a probability of α

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EXECUTIVE SUMMARY

Background

The conventional approach to developing Waste Load Allocations (WLAs) is based on a steady state analysis of stream conditions, using a design stream flow (usually the 7Q10) and a receiving water concentration (usually a water quality standard based on chronic criteria) for the pollutant to be allocated. An effluent concentration limit is computed for these conditions, and is used to establish the NPDES permit conditions.

The water quality based permit conditions apply, in addition to technology based requirements (e.g., BAT, BCT, and secondary treatment). This effluent requirement may be incorporated into the permit as the daily maximum limit, the average limit over a week (for POTWs) or the average limit over a month (for industrial as well as municipal source).[1] Typical practice for toxic pollutants is to incorporate the wasteload allocation result as the daily maximum permit limit. This document provides an innovative approach to determining which types of permit limits (daily maximum, weekly, or monthly average) should be specified for the steady-state model output based on the frequency of acute criteria violations.

Approach

The method used to evaluate the effect of permit averaging periods is based on a probabilistic dilution model (PDM) in which it is assumed that the stream flows, effluent flows and concentration are log-normally distributed

[1] See 40 C&R 122.45(d).

and uncorrelated. The log-normal distribution is known to be representative of effluent behavior and to almost always under-estimate the lowest stream flows somewhat. Thus, the analysis is generally conservative (overprotective) to some extent. However, a verification of the probabilistic dilution model indicates that, for the cases tested, it correctly estimates observed downstream concentration probability distributions to within the confidence limits of the data.

The method applied in using this model to evaluate permit averaging period choices is based on the following observation. If chronic criteria and 7-day, 10-year low flow, or any other state-specified low flow, are used in the WLA analysis to develop the maximum effluent concentration, the use of monthly or weekly permit limits for specifying this effluent requirement presents the possibility that simultaneous occurrences of high effluent concentrations and low stream flows may result in stream concentrations which exceed the acute criteria for a pollutant without violating maximum average discharge permit conditions.

The analysis consists of computing the level of treatment required for the three averaging period options for specifying the WLA results as permit limits. The analysis computes the frequency at which acute stream criteria concentrations are violated under each of the permit averaging period options, taking into account the likely range of stream and effluent variability. Computation results are normalized so that summary results can be applied to a variety of pollutants based on their ratio of acute-to-chronic criteria concentrations.

Uses

The primary use of this methodology will be specifying the required level of treatment and deriving permit limits based on water quality requirements. Care must be taken in the assumptions related to the permit limits and assumptions used in the methodology. For example, throughout this document, reference is made to 7-day and 30-day averages. These averages are equivalent to weekly and monthly permit limits where the assumption can be made that the monitoring data is adequate (i.e., that the data collected in a month adequately reflects the 30-day average). Where this requirement is not valid, alternative limits may be calculated which incorporate monitoring frequency, or monitoring frequency may be adjusted so that these conditions are met.

In addition to the usefulness of this method for permit writers in selecting the averaging period for discharge permits, the method has been used to calculate suitable averaging periods for the range of stream and effluent conditions typified in the U.S. The results have been summarized in convenient graphic and tabular displays, and can be used as a "screening tool" that provides a guide for water quality decisions. These summaries show, for instance, that for toxic pollutants with acute-to-chronic ratios of 10 or greater, 30-day permit averages will virtually always meet the criteria that have been adopted; that is, that acute criteria violations in the stream will recur with a frequency that averages less than 1 day in 10 years.[1]

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[1]The EPA is presently considering the issue of allowable duration and frequency of exposure to toxicity. Based upon this work, duration and frequencies used as the decision criteria may change. This guidance does not recommend any particular minimum acceptable duration or frequency.

For pollutants with acute-to-chronic ratios of between 5 and 10, monthly permit averages will be appropriate in most cases, although there will be some site-specific conditions that would call for the use of weekly averages. For pollutants with acute-to-chronic ratios of less than 5, site specific conditions must be considered, and no general rule is possible. In these cases, site-specific analyses of the effects of different permit averaging periods can be performed using the methods outlined in the text.

Limitations

Several technical refinements to the probabilistic model would be required to more accurately reflect the deviation of lowest stream flow from log-normality, and to account for serial and cross-correlation of stream flows[1] and effluent loads. For coupled reactions, such as BOD/DO, the procedures would have to be extended to provide a seasonal approach and results should be verified against field data. The analysis method would have to be extended to incorporate the variability of secondary water quality parameters such as pH, hardness and temperature, since these affect the toxicity of a number of pollutants. Finally, the chronic exposure event, as defined by the state design flow conditions, was used throughout the document to estimate the maximum effluent concentration. Further analyses to determine the possible underprotection or overprotection of chronic criteria based on the state design flow[1] were not done.

[1]The EPA is considering studying the impact of uncertainties involving the low flow estimating techniques on the selection of stream design flow.

CHAPTER 1

INTRODUCTION

1.1 Background

The conventional procedure for establishing a point source effluent limit using a waste load allocation (WLA) analysis begins by specifying a target concentration of the pollutant in the stream, such as a state water quality standard based on chronic criteria. This stream concentration is converted to a maximum effluent concentration using a mass balance calculation (for conservative substances) or a steady-state analysis (for reactive substances). The inputs to these analyses are a design stream flow (representing low stream-flow conditions)[1] and a measure of the effluent flow, typically the mean effluent flow. Although this technique is presumed to provide adequate protection for receiving water quality, it fails to account for random and other fluctuations in the flow rate and concentration that naturally occur in both the stream and effluent. Thus, the degree to which a given limit protects against exceedances of acutely toxic concentrations is not quantified.

Effluent permit limitations are currently specified as maximum concentrations for one day or averaged over a week or month. The number of observations from which the average is computed depends on the frequency of

[1] The design stream flow most commonly used is the 7Q10 flow, which represents the low-flow condition with a recurrence interval of 10 years based on a 7-day averaging period. Other flows, such as the 30Q10 or 30Q5 are occasionally used as the design stream flow. Wherever the use of stream design flow is called for, these or other stream design flows can be substituted throughout this document.

monitoring. Although there is no generally accepted rational basis for selecting permit averaging periods, the effluent requirement derived from a WLA is typically expressed as a monthly average for conventional pollutants and as the daily maximum for toxic pollutants. A set of conversion factors is then used to convert these concentrations to other averaging periods. In this document the maximum daily, weekly, and monthly permit limits are referred to as 1-day, 7-day, and 30-day permit levels, respectively.

The permit limit used to incorporate a WLA effluent requirement can have a substantial influence on the degree (and cost) of treatment required and on the quality of the receiving water. It is clear that a permit limit imposed as a daily maximum requirement is more restrictive than when the same permit limit is used as a 30-day average requirement, since in the latter case the effluent concentration can fluctuate above the effluent limit for days at a time and still meet the 30-day average requirement. Such fluctuations may or may not be significant in terms of receiving water quality. The appropriate choice of the averaging period, then, is one which ensures acceptable receiving water quality without imposing unnecessarily restrictive treatment requirements.

1.2 Objectives

This guidance document is intended to achieve the following:

(1) Present a rational method for selecting the level of treatment required based on considerations of water quality;

(2) Present a rational method to incorporate the water quality based treatment requirements as permit limits;

(3) Provide specific information, including detailed examples, so that the method can be applied to site-specific cases;

(4) Use the method to provide an overall analysis of a broad range of conditions likely to be encountered, so as to provide a screening tool for the rapid assessment of a wide variety of cases;

(5) Discuss the uses and limitations of the method.

1.3 Approach

The basis of the method is an evaluation of the extent and frequency of acute criteria violations to be expected in the stream receiving the discharge as a result of imposing the effluent concentration, computed from a steady state wasteload allocation, as a daily, weekly, or monthly average permit. A probabilistic framework is adopted to account for the inherent variability of flows and concentrations. Acute criteria violations are assumed to be associated with random simultaneous occurrences of high effluent loadings and low stream flows.[1] The analysis is based on an examination of the probability distributions involved and how they combine to influence the concentration downstream. The probabilistic dilution model provides the analysis framework.

The probabilistic dilution model is summarized in Figure 1-1. The inputs to the model include the flow and concentration histories (or projections) of both the effluent and the receiving stream. Each of these is

[1] While it is apparent that effluent loadings and stream flows experience both random and nonrandom (e.g., seasonal) variations, the problem is analyzed here in purely random terms to limit the complexity of the analysis.

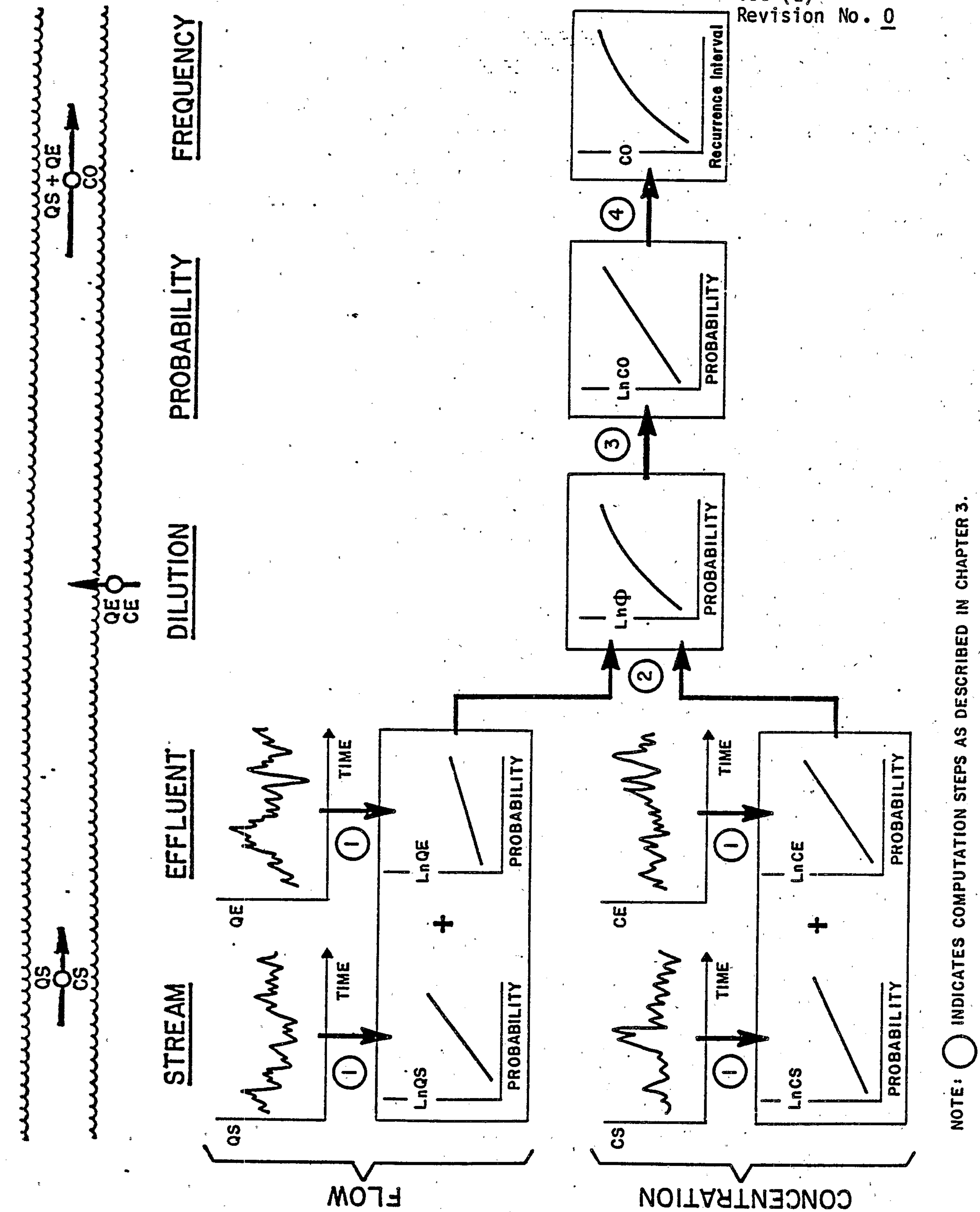


Figure 1-1 - Schematic outline of probabilistic method.

expressed as a probability distribution; that is, in terms of the probability that a given value is exceeded. Next, the effluent and stream flows are combined to yield the probability distribution of the dilution factor; then the dilution factor and concentrations are combined to provide the probability distribution for the resulting stream concentration. The stream concentration probability distribution is then converted to a plot showing the recurrence interval to be associated with each stream concentration so that the frequency of occurrence of a given (high) stream concentration can be compared to water quality objectives.

The probabilistic dilution model is used to guide the choice of the permit averaging period as follows. Given an effluent requirement from a WLA analysis, the mean effluent required to meet that WLA requirement is calculated for each of the three averaging periods, based on an assumed allowable frequency of effluent limit violation. This provides three levels of treatment for the plant in question. Each mean effluent concentration is then used, together with the parameters that characterize the stream variability, in the probabilistic dilution model. The result is a probability distribution of resulting stream concentration for each of the three treatment plant options, which can be compared to daily concentration/frequency water quality goals. The use of daily concentration frequencies allows the use of acute criteria in establishing water quality goals.

1.4 Organization

This document is organized as follows. Chapter 2 provides a detailed description of the methodology for finding an optimum averaging period based on a probabilistic dilution method. Chapter 3 presents an annotated example of the method performed first as a hand calculation and then using the computer program provided in Appendix D. Chapter 4 uses the model in several representative applications, and Chapter 5 discusses the uses of the method. Several appendices to this document provide detailed additional material, including a review of relationships for log-normal distributions (Appendix A) and a discussion of technical issues and assumptions employed in the analysis (Appendix B).

CHAPTER 2

METHOD OF ANALYSIS

This chapter lays the theoretical groundwork for the application of the probabilistic dilution model to the problem of permit averaging period selection. This discussion is presented in two parts. Section 2.1 describes the probabilistic dilution model. Section 2.2 develops the method whereby the probabilistic dilution model is employed to predict the water quality effects of the selection of different averaging periods.

2.1 Description of the Probabilistic Dilution Model

The probabilistic dilution model is based on a simple stream dilution calculation. The complexity of the model arises from the probabilistic framework that is superimposed upon the dilution equation. This section is intended to provide a description of the derivation of the model, and to reduce it to a manageable set of equations. While a strict mathematical derivation of the model is available [1], a rigorous treatment is considered beyond the scope of this manual.

Figure 2-1 illustrates a treatment plant discharge entering a stream. The effluent discharge flow (QE), having a concentration (CE) of the pollutant of interest, mixes with the stream flow (QS), which may have a background concentration (CS). The receiving water concentration (CO) is the concentration that results after complete mixing of the effluent and stream flows. It is the cross-sectional average concentration downstream of the discharge, and is given by:

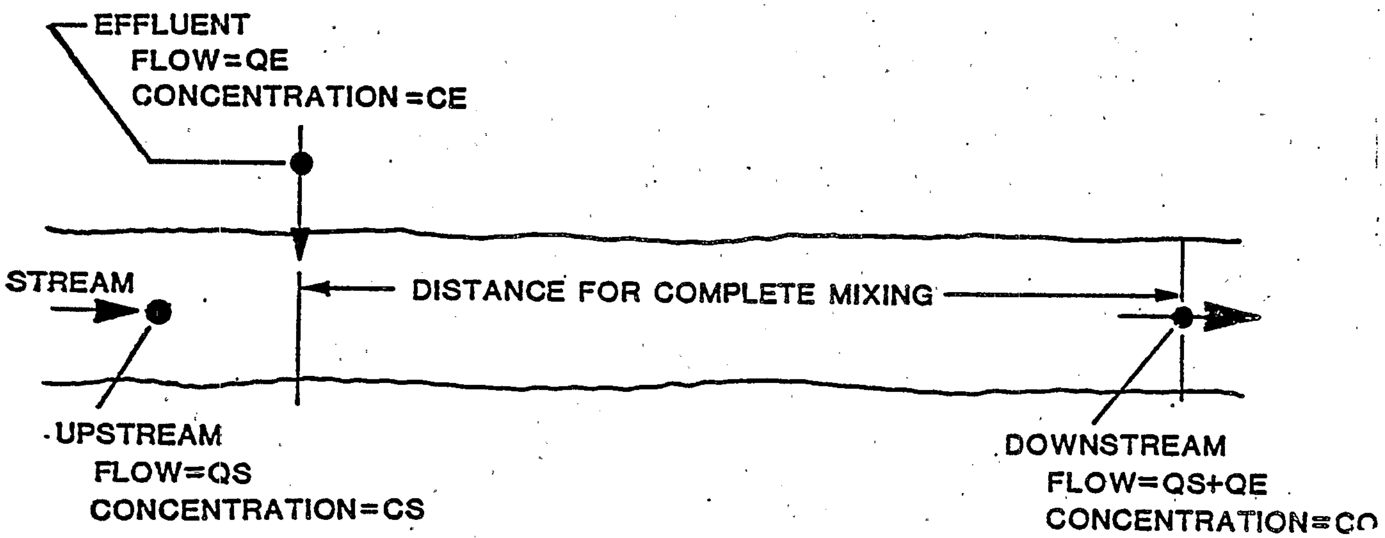


Figure 2-1 - Simple dilution model.

$$CO = \frac{(QE \cdot CE) + (QS \cdot CS)}{QE + QS} \qquad (2\text{-}1)$$

If the dilution factor, Φ, is defined as:

$$\Phi = \frac{QE}{QE + QS} = \frac{1}{1 + D} \qquad (2\text{-}2)$$

where

$D = QS/QE$, the ratio of stream flow to effluent flow.

CO may be calculated in terms of Φ by:

$$CO = [\Phi\, CE] + [(1 - \Phi)\, CS] \qquad (2\text{-}3)$$

The calculated value of CO for a given day could be compared to a water quality standard (CL) or to any other stream concentration which relates water quality to water use. This procedure could be repeated for a large number of days and the resulting set of values for CO could be subjected to standard statistical analysis procedures to obtain its probability distribution. If this were done, the total percentage of days on which the downstream concentration CO exceeded CL could be determined.

The ability to perform this direct computation depends upon the availability of long time series of upstream and treatment plant flows and concentrations of each pollutant of interest. Such long data records are usually only available for stream flow, but estimates based on more limited data sets may be available for the other elements. An important objective of any modeling framework is to cast the problem into a manageable form while at the same time preserving its essential features. Therefore, it is necessary to characterize the fluctuating behavior of the upstream and effluent flows and concentrations in a concise and realistic fashion.

The probabilistic dilution calculation procedure used in this report permits the probability distribution of downstream concentrations (CD) to be computed directly from the probability distributions of the flows and concentrations.

The first step in the use of the probabilistic dilution model is to develop the statistics of the concentration and flow of both the stream and effluent.[1] These statistics include both the arithmetic and logarithmic forms of the mean (μ), standard deviation (σ), and coefficient of variation (ν). The analysis is simplified here by specifying an upstream concentration of zero (CS = 0) so that the results reflect only those effects on the receiving water due to the effluent discharge, thus highlighting the comparative differences resulting from choice of permit averaging period.

The amount of dilution at any time is a variable quantity and the dilution ratio (D= QS/QE) has a log-normal distribution when both stream flow (QS) and effluent flow (QE) are log-normal. The log standard deviation of the flow ratio QS/QE is designated as $\sigma_{\ln D}$. This can be calculated from the log standard deviations of stream flow and effluent flow, assuming no cross-correlation between stream and effluent flows.

$$\sigma_{\ln D} = \sqrt{\sigma^2_{\ln QS} + \sigma^2_{\ln QE}} \qquad (2\text{-}4)$$

[1]Standard statistical procedures are used to compute the mean and standard deviation using the log transforms of the basic data. Conversion to the other statistical expressions used in the analysis is described in Appendix A.

The probability distribution of the dilution factor, $\Phi = 1/(1+D)$ is not truly log-normal, even with log-normal runoff and stream flows. It has an upper bound of 1 and a lower bound of 0, and where it approaches these values asymptotically, it deviates appreciably from a log-normal approximation. Deviations at values of Φ approaching 0 are of no practical significance to the calculations being performed since they occur at high dilutions.

For smaller streams relative to the size of the discharge, deviations from a log-normal approximation can be appreciable. They are large enough to introduce significant error into the calculated recurrence interval of higher stream concentrations. The error introduced is almost always conservative; that is, it projects high concentrations to recur more frequently than they actually would. The appropriateness of this assumption is discussed in detail in Appendix B.

A procedure is provided in this report for accurately calculating the probability distribution of the dilution factor (Φ) and stream concentration (CO). This numerical method uses quadratures and would be prohibitively tedious to perform manually. It has, therefore, been provided in the form of a computer program which can be utilized on a microcomputer (Appendix D).

For purposes of presenting the approach in a form which can be solved manually, and thereby better illustrate the basic procedure employed, the methodology description which follows in this section develops a log-normal approximation for the dilution function Φ and then proceeds with the calculations for stream concentration. Whether the log-normal approximation or the quadrature calculation is used, the subsequent steps in determining the appropriate averaging period are the same.

The manual procedure (moments method) estimates the mean and standard deviation of a log-normal approximation of dilution by first calculating, and then interpolating, between the 5% and 95% probability values. The value of the dilution factor (Φ) for any probability percentile (α) is given by:

$$\Phi_\alpha = \frac{\widetilde{QE}}{(\widetilde{QE} + \widetilde{QS}) \exp (Z_\alpha \sigma \ln D)} \tag{2-5}$$

where the value of Z_α is taken from a standard normal probability table for the corresponding value of α (see Appendix A).

For example, where $\alpha = 95\%$; $Z_{95} = 1.65$

$\alpha = 5\%$; $Z_5 = -1.65$

$\alpha = 50\%$; $Z_{50} = 0$

$\alpha = 84.13\%$; $Z_{84} = 1.0$

The log mean dilution factor is estimated by interpolating between the 5% and 95% values, calculated above.

$$\mu_{\ln\Phi} = 1/2 \, [\ln (\Phi_{95}) + \ln (\Phi_5)] \tag{2-6a}$$

The log standard deviation is determined by the following formula which, in effect, determines the slope of the straight line on the log-probability plot:

$$\sigma_{\ln\Phi} = \frac{1}{Z_{95}} \cdot \frac{[\ln (\Phi_5) - \ln (\Phi_{95})]}{2} \tag{2-6b}$$

From the log mean and log standard deviation of the dilution factor (Φ), the arithmetic statistics are computed using

$$\mu_{\Phi} = \exp\left(\mu_{\ln\Phi} + \frac{1}{2}\sigma^2_{\ln\Phi}\right) \tag{2-7}$$

$$\sigma_{\Phi} = \mu_{\Phi}\left[\exp\left(\sigma^2_{\ln\Phi}\right) - 1\right]^{1/2}$$

The arithmetic mean of the receiving water contaminant concentration (CO) downstream of the discharge after complete mixing, then, can be found by:

$$\mu_{CO} = [\mu_{CE}(\mu_{\Phi})] + [\mu_{CS}(1 - \mu_{\Phi})] \tag{2-8}$$

The arithmetic standard deviation of stream concentration is:

$$\sigma_{CO} = \sqrt{\sigma^2_{\Phi}(\mu_{CE} - \mu_{CS})^2 + \sigma^2_{CE}(\sigma_{\Phi}^2 + \mu_{\Phi}^2) + \sigma^2_{CS}(\sigma_{\Phi}^2 + [1 - \mu_{\Phi}]^2)} \tag{2-9}$$

The coefficient of variation of stream concentration (CO) is:

$$\nu_{CO} = \sigma_{CO}/\mu_{CO} \tag{2-10}$$

The arithmetic statistics used to derive the log statistics will be used to develop the desired probability of exceedence.

$$\text{log standard deviation} = \sigma_{\ln CO} = \sqrt{\ln(1 + \nu^2_{CO})} \tag{2-11}$$

$$\text{log mean} = \mu_{\ln CO} = \left(\ln \frac{\mu_{CO}}{\sqrt{1 + \nu^2_{CO}}}\right) \tag{2-12}$$

The probability (or expected frequency) at which a value of CO will occur is determined by constructing a probability distribution plot on log-probability paper. This is accomplished by computing the 50th percentile and 84th percentile concentrations and connecting them with a straight line:

$$50\%\ \text{concentration} = \widetilde{CO} = \exp(\mu_{\ln CO})$$

$$84\%\ \text{concentration} = \exp(\mu_{\ln CO} + \sigma_{\ln CO})$$

Using this procedure, any concentration of interest can be identified and its probability of occurrence scaled directly from the plot.

Alternatively, the concentration that will not be exceeded at some specific frequency (or probability) can be calculated from:

$$CO_{\alpha} = \exp(\mu_{\ln CO} + (Z_{\alpha}\sigma_{\ln CO})) \qquad (2\text{-}13)$$

where

Z_{α} = the value of Z from a standard normal table which corresponds to the selected percentile α.

To determine the probability of exceedence, $(1 - \alpha)$ is substituted in Equation 13.

One can also work in the reverse direction; that is, given some target stream concentration (CL), the probability of CO exceeding that level can be determined by:

$$Z = \frac{\ln(CL) - \mu_{\ln CO}}{\sigma_{\ln CO}} \qquad (2\text{-}14)$$

A standard normal table will provide the probability for the calculated value of Z.

Because of the way the standard normal table in Appendix A is organized, the probabilities calculated using this approach represent the fraction of time the target concentration (CL) is not exceeded. The probability that the concentration will be exceeded is obtained by subtracting the value obtained from 1.0.

2.2 Choice of the Permit Averaging Period

In order to examine the comparative effects of different choices of permit averaging periods on water quality, it is necessary to define the relationships between the established effluent limit (EL) from the steady state WLA, the permit averaging period, the treatment plant performance that results, in particular the mean effluent ($\overline{CE}$), the downstream concentration (CO), and a stream target concentration (CL).

The objective of this section is to examine the relationships among these parameters in order to be able to predict the probability of an (adverse) water quality outcome based on known or estimated stream and effluent characteristics and the choice of permit averaging period. The approach is based on the assumption that the EL will be violated with a particular frequency. The mean effluent required to meet this level of compliance with EL is then calculated for each of the three permit averaging periods, and the probabilistic dilution model is then used to develop a probability distribution of the downstream concentration (CO) for the three cases. A level of acceptable adverse water quality (a decision expressed in terms of the probability or frequency of experiencing a selected high value of CO, such as the acute criteria concentration) is then compared with the probability distributions to determine the longest permit averaging period that meets the water quality goals.

The first step in this sequence is to establish the relationship between the mean effluent ($\overline{CE}$), the effluent limit (EL), and the permit averaging period. In fact, what is required is the relationship between the treatment plant performance necessary to meet the effluent limit as either a daily, weekly, or monthly maximum permit. The reason for this is that the daily variation of stream quality is governed, not by the effluent limit which is a regulatory upper limit, but by the probability distribution of the daily effluent concentrations which results from the design of the treatment plant consistent with the effluent limit and the permit averaging period. For log-normally distributed random variables, this distribution is specified by the mean effluent concentration, $\overline{CE}$, and its coefficient of variation, ν_{CE}.

A particular effluent limit (say 30 mg/l) established by permit as a maximum daily value would require a higher level of plant performance (a lower mean effluent concentration) to avoid permit violations than would the same limit specified as a maximum monthly average. In the latter case, excursions above the effluent limit could be tolerated on individual days, without causing a violation of permit conditions. The reason for this is that a monthly average of 30 individual daily effluent concentrations is less variable than the daily concentrations themselves. Occasional high daily concentrations are averaged together with lower concentrations to produce a less variable monthly average. Hence, treatment plant performance is directly related to the averaging period specified in the permit.

In order to proceed with the analysis a quantification of this relationship is required. Daily treatment plant effluent concentration variations are well described by a log-normal distribution parameterized by a long term average concentration, $\overline{CE}$, and a coefficient of variation, ν_{CE}. Thus, a relationship between these parameters and the permit effluent limit and averaging period is required.

A method to be employed is based upon an interpretation of what is meant, in practice, by specifying permit effluent limits as maximum values which may never be exceeded for the specified averaging period without causing a violation. As Haugh, et al. [2] observe, fixed upper limits which are never to be exceeded are conceptually inconsistent with the stochastic nature of wastewater treatment processes and the effluent concentrations they produce. Realistically, some exceedence frequency must be acknowledged, regardless of the averaging period assigned. For the present analysis, it will be assumed that the effluent limit specified by a permit is not to be exceeded more frequently than 5 percent or 1 percent of the time. Of course, any other choice is possible.

Once a specific choice is made, say 1 percent, then the probability of compliance is $\alpha = 99$ percent and that establishes the fact that EL is the α-percentile effluent concentration: CE_α. This procedure, then, gives a specific probabilistic interpretation to the effluent limit. It is the effluent concentration that is exceeded with no greater frequency than $(1-\alpha)$ percent of the time. If the permit is specified as a daily maximum value, then EL is the α-percentile of daily effluent concentrations. If the permit is specified as a weekly (or monthly) maximum value, then EL is the α-percentile of 7-day (or 30-day) average effluent concentrations.

In order to compute the long term average effluent concentration, $\overline{CE}$, that would insure that $CE_\alpha = EL$ as a daily, weekly, or monthly permit the coefficients of variation are required for 1-day and 7-day or 30-day averages of effluent concentrations. Table C-2 presents representative values.

Thus, the requirement that:

$$CE_\alpha = EL \tag{2-15}$$

and for a coefficient of variation ν_{CE}, the average effluent concentration $\overline{CE}$ can be computed from

$$\overline{CE} = R_\alpha \cdot EL \tag{2-16}$$

where the reduction factor relating $CE_\alpha = EL$ to $\overline{CE}$, that is, $R_\alpha = \overline{CE}/CE_\alpha$ is

$$R_\alpha = \sqrt{1 + \nu_{CE}^2} \ \exp \left[- Z_\alpha \sqrt{\ln \ (1 + \nu_{CE}^2)}\right] \tag{2-17}$$

the ratio of the arithmetic average to the α-percentile of a log-normal random variable with coefficient of variation, ν_{CE}. Table 2-1 gives the values of R_α for various coefficients of variation.

The derivation of this formula follows from the expression for the α-percentile of a log-normal random variable:

$$CE_\alpha = \exp \ (\mu_{\ln CE} + Z_\alpha \ \sigma_{\ln CE}) \tag{2-18}$$

and the arithmetic average of a log-normal random variable:

TABLE 2-1 - Reduction factors for various coefficients of variation.

Coefficient of Variation v_{CE}	Reduction Factor R_α α = 95%	α = 99%
0.1	0.853	0.797
0.2	0.736	0.643
0.3	0.644	0.527
0.4	0.571	0.439
0.5	0.514	0.372
0.6	0.468	0.321
0.7	0.432	0.281
0.8	0.403	0.249
0.9	0.379	0.224
1.0	0.360	0.204
1.1	0.344	0.187
1.2	0.330	0.173
1.3	0.319	0.162
1.4	0.310	0.152
1.5	0.302	0.144

$$\overline{CE} = \exp\left(\mu_{\ln CE} + \frac{1}{2}\sigma^2_{\ln CE}\right) \tag{2-19}$$

Thus:

$$R_\alpha = \overline{CE}/CE_\alpha = \exp\left(\frac{1}{2}\sigma^2_{\ln CE} - Z_\alpha \sigma_{\ln CE}\right) \tag{2-20}$$

and since $\exp(1/2\sigma^2_{\ln CE}) = \sqrt{1 + \nu^2_{CE}}$ and $\sigma_{\ln CE} = \sqrt{\ln(1 + \nu^2_{CE})}$ (appendix A, page A-8) equation (2-17) follows.

At this point the effect of the choice of permit averaging period on treatment plant design can be illustrated. If the permit averaging period is 1-day, and the daily effluent coefficient of variation is $\nu_{CE} = 0.7$ (for example, extended aeration activated sludges, Table C-2), then for a 1 percent violation frequency $\alpha = 99$ percent, $R_\alpha = 0.281$, which indicates that the long term average effluent concentration must be 28.1 percent of the daily maximum permit limit.

However, if the permit averaging period is 7 days, then the coefficient of variation of 7-day averages is $\nu_{CE} = 0.6$ and $R_\alpha = 0.321$. Now the treatment plant can be designed to produce a long term average effluent concentration of 32.1 percent of the weekly permit limit. For a 30-day average permit limit $\nu_{CE} = 0.45$ and $R_\alpha = 0.404$. Hence, if EL = 10 mg/l, the treatment plant average effluent concentration must be 2.81, 3.21, or 4.04 mg/l for a daily, weekly, or monthly permit specification, respectively.

Hence the selection of the permit averaging period is related to the $\overline{CE}$ required for each of the three averaging periods in order to avoid exceeding the EL more often than the selected frequency. These average values are then used in the probabilistic dilution model (with other input parameters such as $\overline{QS}$ and $\overline{QE}$) to develop the probability distribution of CO for each of the three permit averaging periods.

The value of CO in the probability distribution can be normalized in terms of a stream target concentration (such as the chronic criteria concentration, CL) so that the calculation can be used for a wide variety of pollutants. Stream concentration is therefore expressed in terms of $\beta = CO/CL$, β being a dimensionless unit of concentration.

A convenient presentation of the resulting probability distribution makes use of the concept of return period. For daily stream concentrations the 1 percent exceedence value has an average recurrence rate of one day every 100 days so that its average return period is 100 days. Thus the return period for daily values is defined as:

$$\text{Return Period (days)} = 1/\text{Probability of Exceedence} \qquad (2\text{-}28)$$

The basic assumption in the use of return period as defined above is that the event whose probability is being examined has a characteristic time associated with it, in this case, one day for daily concentrations. Thus, it is assumed that daily stream concentrations are of concern, and each event corresponds to one day.

Figure 2-2 illustrates how the results of such an analysis can be expressed in a plot of concentration versus return period.

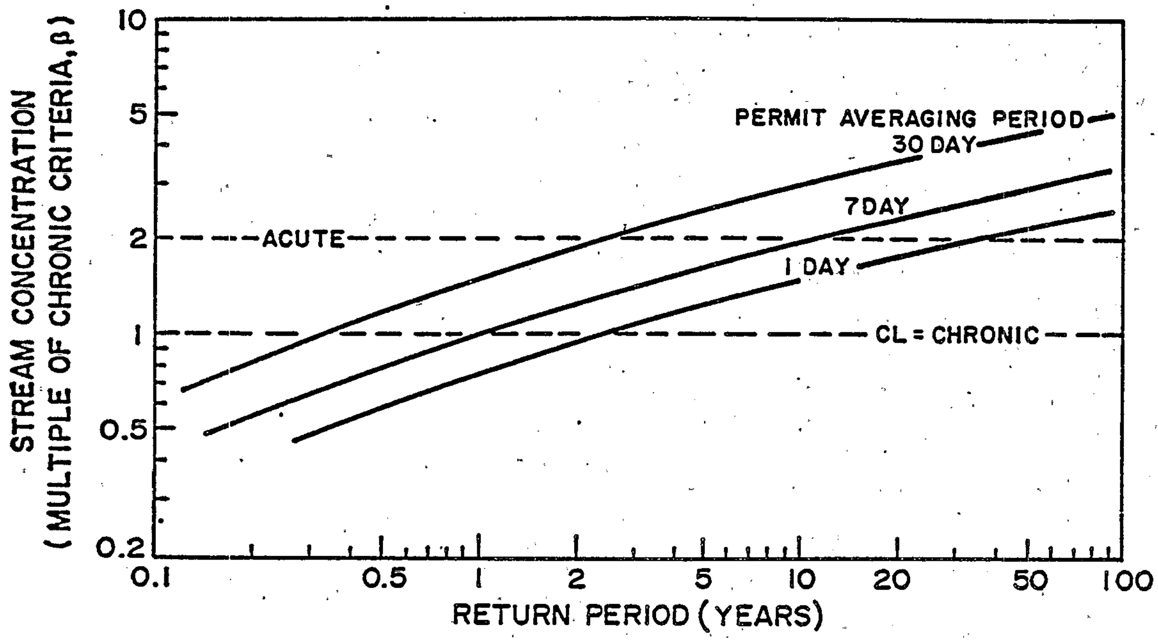


Figure 2-2 - Illustration of analysis results: stream concentration versus return period for three permit averaging periods.

The stream target concentration (CL) for a typical WLA is the chronic criteria concentration of the pollutant under consideration. The use of the chronic criteria as the stream target concentration is convenient for the comparison of permit averaging periods because it represents a specific and frequently used procedure. The analysis that follows does not attempt to quantify the frequency with which chronic criteria concentrations are met by either the conventional WLA procedure or the guidance provided for selecting permit averaging period. Instead, the analysis is designed to relate the choice of the permit averaging period to the frequency with which severe, short term water quality impacts are expected as a result of an effluent limit.

These short-term impacts are perhaps most effectively evaluated with respect to acute criteria concentrations. If the stream concentration exceeds the acute criteria as a result of an occasional high daily effluent loading, the result is presumed to be an undesirable impact. Hence, there is a direct connection between the permit averaging period and the probability of acute criteria violations. Specifying that the WLA requirement be met as a daily maximum permit limit significantly reduces the possibility of acute criteria violation since the effluent limit is specified using the chronic criteria, which is always a smaller concentration.

The frequency with which daily stream concentrations are allowed to exceed acute criteria is a regulatory decision.[1] The analyses presented herein employ a frequency that corresponds to a 1-day in 10-year recurrence, on average. The choice of 10 years is, of course, used for example purpose only but it is consistent with the 10 year return period that is conventionally used for the design stream flow.

The results of the permit averaging period analysis are presented in terms of CO/CL which is exceeded with a particular frequency, such as once in 10 years. This ratio can then be compared to the acute-to-chronic criteria concentration ratio for the pollutant of concern. For pollutants with large acute-to-chronic ratios, occasional large daily fluctuations can be tolerated; and a 30-day permit averaging period provides protection from acute criteria violations. Conversely, pollutants with small acute-to-chronic ratios are more likely to require shorter day permit averaging periods. Site specific

[1]This is currently under EPA study.

considerations, primarily the ratio of effluent to stream flow and stream flow variability, become significant in these cases.

The final translation of the selected averaging period option to permit limits requires consideration of the monitoring frequency. The method assumes either daily monitoring or other monitoring adequate to describe the performance of the plant on a monthly basis. If such conditions are not met, alternate limits may be calculated which incorporate monitoring frequency, or monitoring frequency may be adjusted so that these conditions are met.

CHAPTER 3

EXAMPLE COMPUTATION

This chapter presents an example problem, showing step by step computations using the methodology described in the previous chapter. A set of hypothetical conditions that apply to a site-specific situation is assumed, and an analysis is performed to determine the effect on receiving water quality resulting from the assignment of different permit averaging periods to the steady-state model output. The steps used to conduct this analysis are summarized below in Figure 3-1. The format used in this chapter presents data and computations on the left-hand page, and pertinent commentary and supporting discussion on the facing page immediately opposite those computations. The manual computation using the moments approximation is described first, followed by an analysis using the computer program (PDM-PS) in Appendix D. Both examples use the same set of hypothetical site-specific conditions.

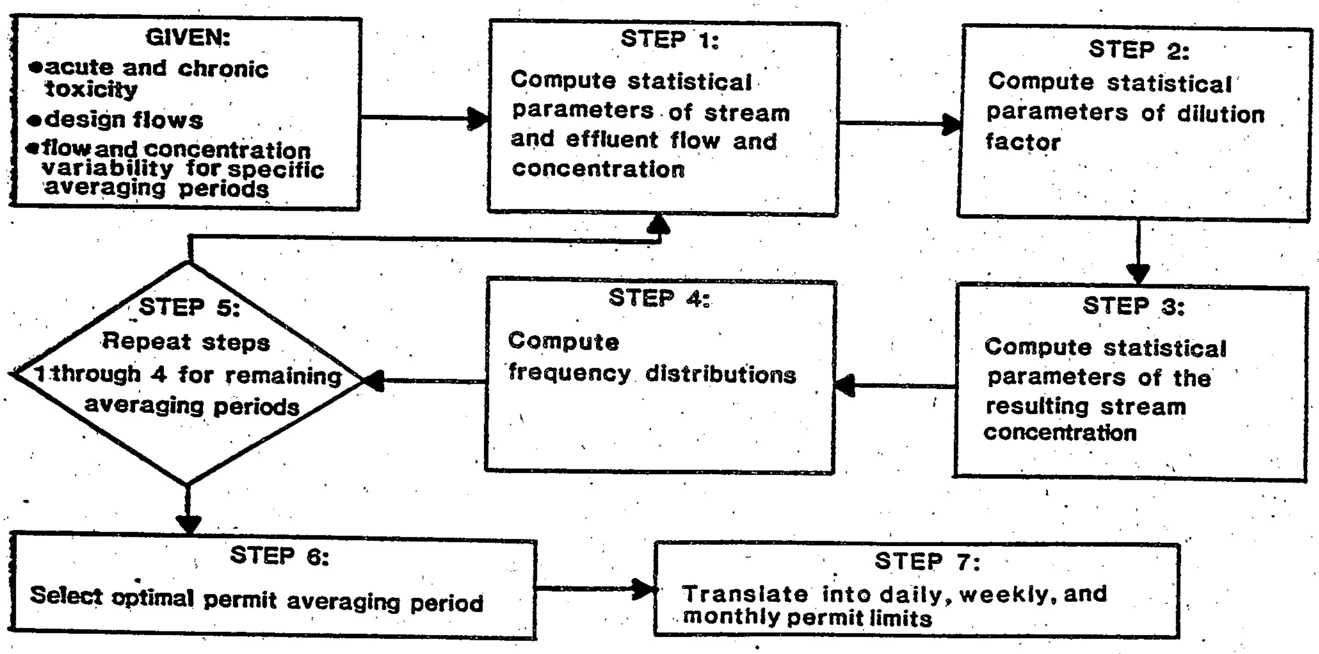


Figure 3-1 - Step procedure to select optimal permit averaging period.

HYPOTHETICAL SITE-SPECIFIC CONDITIONS

This section provides an example of the type and amount of information required to perform the analysis. It also establishes the basis for the example computations and assumes that pertinent site-specific conditions are as follows:

A. Site-Specific Waste Load Allocation (WLA) Results

The pollutant (P) to be allocated has a chronic toxicity concentration (CL) of 2.5, and an acute toxicity concentration of 6.25.

WLA policy for the agency performing the analysis is to use 7Q10 as stream design flow, to use the design capacity of the treatment plant as the effluent flow, and to compute (e.g., using a water quality model) the effluent concentration of pollutant (P) that will result in a stream concentration after dilution less than or equal to the chronic value (2.5 = the stream target concentration, CL). For this example, it was assumed that:

Design Effluent Flow (QE) = 5 MGD = 7.77 cfs

Design Stream Flow (7Q10) = 23.3 cfs

The stream target concentration (CL = 2.5) will be met under these design flow conditions, when the effluent concentration is CE = 10. Therefore, based on the WLA analysis, the effluent limit (EL) for pollutant (P) is specified by the permit as:

EL = 10

COMMENTARY

--- from EPA Criteria

State water quality standards do not usually specify both values; they are usually based on chronic values.

(Any concentration units may be assigned; stream concentrations will have to be in the same units.)

--- 7Q10 (the lowest 7-day average stream flow with a recurrence interval of 10 years) is the most common "design stream flow". Some states use other values (e.g., 30Q5). This analysis uses the numerical value of the "design flow". However, although the example terminology uses "7Q10", it should be interpreted as "design stream flow" and the appropriate value substituted, regardless of the averaging period or the recurrence interval on which it is based. (For example, if design flow in a state were 30Q5, assume that 30Q5 = 23.3 cfs).

NOTE: The only exception to this is in Figure C-1, in which the ratio of 7Q10 to average stream flow is used to estimate the variability of daily flows in the absence of a specific local analysis. The use of this figure is not requisite to either the analysis methodology or the computations.

--- $$CL = \frac{(QE \cdot CE) + (QS \cdot CS)}{QE + QS}$$

$$2.5 = \frac{(7.77 \cdot CE) + (23.3 \cdot 0)}{7.77 + 23.3}$$

$$CE = 10 = EL$$

HYPOTHETICAL SITE-SPECIFIC CONDITIONS
(continued)

B. Site-Specific Conditions

Stream Flow

Mean Flow ($\overline{QS}$) = 467 cfs

Coefficient of Variation (ν_{QS}) = 1.5

Upstream Concentration

Mean ($\overline{CS}$) = 0

Coefficient of Variation (ν_{CS}) = 0

Effluent Flow

Mean ($\overline{QE}$) = 7.77 cfs

Coefficient of Variation (ν_{QE}) = 0.20

COMMENTARY

--- Stream flow data are obtained from analysis of flow gaging records for the stream in question; where the stream reach is ungaged, it is obtained by extrapolation from an appropriate record.

At present, records are not normally analyzed for the coefficient of variation, although the computation is straight forward and can be readily incorporated into a routine statistical analysis of daily stream flows. In the absence of specific analysis results, the coefficient of variation of daily stream flows can be estimated using the material presented in Figure C-1.

--- Upstream concentration can be assumed to be zero if the stream concentration of the pollutant is very low compared to the discharge, or if the effect of the discharge only is to be examined. Site-specific values for upstream concentration statistics would be obtained from analysis of an appropriate STORET station, or from local monitoring records. If upstream concentrations are assigned, enter data here and in the equations when called for.

--- The design effluent flow is assumed to be the mean effluent flow. The variability of daily effluent flows for a new facility must be estimated on the basis of available data for existing treatment facilities (such as Table C-1). For an existing facility being expanded, or simply re-permitted, variability could be based on an analysis of past plant records. For many industrial dischargers, this data will be available in Book VI (Design Conditions) of the waste load allocation technical guidance document series (specifically, in Chapter 4: Effluent Design Conditions).

HYPOTHETICAL SITE-SPECIFIC CONDITIONS
(continued)

Effluent Concentration Mean ($\overline{CE}$) = (*)

Coefficient of Variation (ν_{CE}) = .7

* The mean concentration is a function of the permit averaging period and is that concentration required to avoid exceeding the effluent limit concentration (EL) more often than the compliance probability.

The coefficient of variation for the hypothetical treatment plant is not known because the plant has yet to be constructed. Assuming that the plant will produce an effluent with a variability similar to the values given in Table C-2, the following values are used:

Permit Averaging Period	Coeff. of Var. (ν_{CE})
Daily	0.70
7-Day	0.40
30-Day	0.20

Equation 2-17 is then used to determine the mean effluent concentration of (P) which is required to avoid a violation of EL more often than the compliance probability. For this example, assume that the exceedence probability is 1 percent. For α = 0.99 percent, Z_α = 2.327. For ν_{CE} = 0.70, $R_\alpha = \overline{CE}/EL$ is:

$$R_\alpha = \sqrt{1 + \nu_{CE}^2}\ \exp\left[- Z_\alpha \sqrt{\ln\left(1 + \nu_{CE}^2\right)}\right]$$

$$= \sqrt{1 + 0.49}\ \exp\left[- 2.327 \sqrt{\ln (1 + 0.49)}\right]$$

$$= 1.221 \exp [-2.327 \cdot 0.6315]$$

$$= 0.281$$

The reduction factor for 7-day and 30-day averages are computed similarly with ν_{CE} (7-day) = 0.40 and ν_{CE} (30-day) = 0.20. The results are:

Permit Averaging Period	Coeff. of Var. of Averaged Effluent Concentrations (ν_{CE})	Reduction Factor $R_\alpha = \overline{CE}/EL$	Required Mean Effluent Conc. ($\overline{CE} = R_\alpha$ EL)
Daily	0.70	0.281	0.281 · 10 = 2.81
7-Day	0.40	0.439	0.439 · 10 = 4.39
30-Day	0.20	0.643	0.643 · 10 = 6.43

COMMENTARY

--- The mean effluent concentration that a treatment facility is capable of producing is influenced significantly by process selection. For this example, it will be assumed that process selection will be made following the issuance of a permit, and influenced by its provisions.

The mean effluent concentration that a facility is required to produce is influenced by the permit averaging period and the variability of effluent concentrations of the pollutant in question.

The analysis employed here, which bases permit averaging period selection on receiving water impacts, is based on exceedance of the acute criteria on a daily basis. Therefore, all subsequent stream impact computations (Step 4) are based on the coefficient of variation of daily effluent concentrations, or 0.7, as shown.

The mean concentration is shown by (*), because a different value is used for each permit averaging period.

--- The recommended exceedence probability for the effluent limit is either 5 percent or 1 percent. For 5 percent, Z_α would be Z_{95} = 1.645.

--- Longer averaging periods reduce the variability of effluent concentrations, and allow permit exceedance limits to be met with higher effluent means. Computation of the required mean ($\overline{CE}$) uses the values of ν_{CE} for the corresponding permit averging period.

EXAMPLE COMPUTATION - HAND CALCULATION

This section illustrates the hand computation using the moments approximation to evaluate the stream concentration probability distribution.

STEP 1: Compute statistical parameters (arithmetic and logarithmic) of inputs using relationships for log-normal distributions (see notes on page 3-9 or Appendix A for equations).

- For the mean effluent concentration ($\overline{CE}$) for a 30-day permit averaging period with X = CE, that is for the variable CE:

ARITHMETIC

Mean (μ_X) = - - - - - - - - (page 3-6) - - - - - - - = 6.43

Coef. Var. (ν_X) = - - - - - - - - (page 3-6) - - - - - - - = 0.70

Std. Dev. $(\sigma_X) = \mu_X \cdot \nu_X = (6.43) \cdot (0.70)$ - - - - - - - = 4.50

Median $(\tilde{X}) = \mu_X / \sqrt{1 + \nu_X^2} = 6.43/\sqrt{1 + (0.7)^2}$ = 5.27

LOGARTHMIC

Log Mean $(\mu_{lnx}) = \ln(\tilde{x}) = \ln(5.27)$ = 1.662

Log Std. Dev. $(\sigma_{lnx}) = \sqrt{\ln(1 + \nu_x^2)} = \sqrt{\ln(1 + (0.7)^2)}$ = 0.6315

- These computations are repeated for each of the other input parameters. The results are tabulated below.

	Arithmetic				Logarithmic	
	Mean	Median	Std Dev	Coef Var	Mean	Std Dev
x	μ_X	$\tilde{X}$	σ_X	ν_X	μ_{lnx}	σ_{lnx}
Stream Flow: QS	467	259	701	1.50	5.5570	1.0857
Effluent Flow: QE	7.77	7.62	1.55	0.20	2.0307	0.1980
Upstream Concentration: CS	0	0	0	0	0	0
Effluent Concentration: CE	6.43	5.27	4.50	0.70	1.662	0.6315

COMMENTARY

The following parameters are used subsequently:

Input Parameter		Arithmetic: Median $\tilde{x}$	Mean μ_x	Std. Dev. σ_x	Coef. Var. ν_x	Logarithmic: Log Mean $\mu_{\ln x}$	Log S.D. $\sigma_{\ln x}$
Stream Flow	QS	$\widetilde{QS}$	μ_{QS}	σ_{QS}	ν_{QS}	$\mu_{\ln QS}$	$\sigma_{\ln QS}$
Stream Conc.	CS	$\widetilde{CS}$	μ_{CS}	σ_{CS}	ν_{CS}	$\mu_{\ln CS}$	$\sigma_{\ln CS}$
Effluent Flow	QE	$\widetilde{QE}$	μ_{QE}	σ_{QE}	ν_{QE}	$\mu_{\ln QE}$	$\sigma_{\ln QE}$
Effluent Conc.	CE	$\widetilde{CE}$	μ_{CE}	σ_{CE}	ν_{CE}	$\mu_{\ln CE}$	$\sigma_{\ln CE}$

The following definitions and equations summarize the relationships among the statistical parameters of log-normal random variables.

Arithmetic	Terms	Logarithmic
x	Random Variable	$\ln x$
μ_x	Mean	$\mu_{\ln x}$
σ_x^2	Variance	$\sigma_{\ln x}^2$
σ_x	Standard Deviation	$\sigma_{\ln x}$
ν_x	Coefficient of Variation	(not used)
$\tilde{x}$	Median	(not used)

$$\mu_x = \exp\left[\mu_{\ln x} + \frac{1}{2}\sigma_{\ln x}^2\right] \qquad \mu_{\ln x} = \ln\left(\frac{\mu_x}{\sqrt{1 + \nu_x^2}}\right)$$

$$\tilde{x} = \exp[\mu_{\ln x}]$$

$$\nu_x = \sqrt{\exp(\sigma_{\ln x}^2) - 1} \qquad \sigma_{\ln x} = \sqrt{\ln(1 + \nu_x^2)}$$

$$\sigma_x = \mu_x \nu_x$$

EXAMPLE COMPUTATION - HAND CALCULATION (continued)

STEP 2: (a) Compute the log standard deviation of the flow ratio QS/QE = D.

$$\sigma_{\ln D} = \sqrt{\sigma^2_{\ln QS} + \sigma^2_{\ln QE} + 2\rho \cdot \sigma_{\ln QS} \cdot \sigma_{\ln QE}}$$

The first two terms are taken from the table in Step 1 (and squared). Since, for this example, flows are not correlated ($\rho = 0$), the third term drops out. Therefore,

$$\sigma_{\ln D} = \sqrt{(1.0857)^2 + (0.1980)^2} = 1.1036$$

(b) Compute the 5th and 95th percentiles of the actual distribution of the dilution factor (Φ).

$$\Phi_\alpha = \frac{\widetilde{QE}}{(\widetilde{QE} + \widetilde{QS}) \cdot \exp(Z_\alpha \sigma_{\ln D})}$$

where:

$\widetilde{QE}$, $\widetilde{QS}$ = median values for effluent and stream flows (from table in Step 1)

Z_α = the standard normal Z score for selected percentiles(α)

$$Z_5 = -1.645 \quad ; \quad Z_{95} = 1.645$$

$\sigma_{\ln D}$ = 1.1036 (computed in Step 2 (a))

Substituting the appropriate values gives:

$\Phi_{95} = 0.004766$ $\quad\quad$ $\Phi_5 = 0.1531$

(c) Compute the log mean and log standard deviation of the log-normal approximation of the distribution of the dilution factor (Φ).

Log mean $\mu_{\ln\Phi} = 1/2\,[\ln(\Phi_{95}) + \ln(\Phi_5)] = -3.6115$

Log std dev $\sigma_{\ln\Phi} = \dfrac{1}{1.645} \cdot \dfrac{(\ln(\Phi_5) - \ln(\Phi_{95}))}{2} = 1.0546$

COMMENTARY

--- This equation accounts for any correlation that may exist between stream flow and effluent flow; e.g., where higher effluent flows tend to occur during periods of high stream flow.

--- Ordinarily, there is no reason to expect any such correlation; therefore $\rho = 0$, and the computation in step (a) is simplified as shown.

$$\Phi_{95} = \frac{\widetilde{QE}}{(\widetilde{QE} + \widetilde{QS})\exp\,(Z_\alpha \sigma_{\ln D})}$$

$$= \frac{7.62}{(7.62 + 259)\exp\,[(1.645)\,(1.1036)]}$$

$$= \frac{7.62}{7.62 + 1591}$$

$$= 0.004766$$

EXAMPLE COMPUTATION - HAND CALCULATION
(continued)

(d) Compute arithmetic statistical parameters (using equations on Page 3-9 and tabulate for convenience.

	Arithmetic				Logarithmic	
	Mean	Median	Std Dev	Coef Var	Mean	Std Dev
Dilution (Φ) Factor	0.0471	0.0270	0.0673	1.43	-3.6115	1.0546

STEP 3: Compute the statistical parameters of the resulting in-stream concentration (CO).

(a) Compute the arithmetic mean concentration using previously tabulated values, using Equation 2-8.

$$\mu_{CO} = [\mu_{CE} \cdot \mu_{\Phi}] + [\mu_{CS} \cdot (1 - \mu_{\Phi})]$$

$$= [6.43 \cdot 0.0471] + [0] = 0.303$$

(b) Compute the standard deviation, using Equation 2-9.

$$\sigma_{CO} = \sqrt{\begin{matrix} \sigma^2_{\Phi} \cdot (\mu_{CE} - \mu_{CS})^2 \\ + \sigma^2_{CE} \cdot (\sigma^2_{\Phi} + \mu^2_{\Phi}) \\ + \sigma^2_{CS} \cdot (\sigma^2_{\Phi} + (1 - \mu_{\Phi})^2) \end{matrix}} = \sqrt{\begin{matrix} (0.0673)^2 \cdot (6.43 - 0)^2 \\ + (4.50)^2 \cdot \begin{pmatrix} 0.0673^2 \\ +0.0471^2 \end{pmatrix} \\ + \quad 0 \end{matrix}} = \sqrt{\begin{matrix} 0.187 \\ + 0.137 \\ + \quad 0 \end{matrix}}$$

$$\sigma_{CO} = \sqrt{0.324} = 0.569$$

(c) Compute and tabulate for use in subsequent graphical or other summaries, the other statistical parameters of stream concentration.

	Arithmetic				Logarithmic	
	Mean	Median	Std Dev	Coef Var	Mean	Std Dev
Stream Concentration (CO)	0.303	0.142	0.569	1.88	-1.95	1.23

COMMENTARY

--- The equations are as follows:

$$\mu_\Phi = \exp\left[\mu_{\ln\Phi} + \frac{1}{2}\sigma^2_{\ln\Phi}\right]$$

$$= \exp\left[-3.6115 + \frac{1}{2}(1.0546)^2\right]$$

$$= 0.0471$$

$$\nu_\Phi = \sqrt{\exp(\sigma^2_{\ln\Phi}) - 1}$$

$$= \sqrt{\exp[(1.0546)^2] - 1}$$

$$= 1.429$$

$$\sigma_\Phi = \mu_\Phi \nu_\Phi$$

$$= (0.0471)(1.429)$$

$$= 0.06729$$

--- When the manual ("moments" approximation) analysis presented here is used, the stream concentrations computed are assumed to be log-normally distributed. That is, the log-normal distribution computed is an approximate representation of the actual distribution that results. The degree of approximation is examined subsequently.

--- The equations are:

$$\nu_{CO} = \sigma_{CO}/\mu_{CO} = 0.569/0.303$$

$$= 1.88$$

$$\mu_{\ln CO} = \ln\left(\frac{\mu_{CO}}{\sqrt{1 + \nu^2_{CO}}}\right)$$

$$= \ln\left(\frac{0.303}{\sqrt{1 + (1.88)^2}}\right)$$

$$= -1.95$$

$$\sigma_{\ln CO} = \sqrt{\ln(1 + \nu^2_{CO})}$$

$$= \sqrt{\ln[1 + (1.88)^2]}$$

$$= 1.23$$

EXAMPLE COMPUTATION - HAND CALCULATION
(continued)

STEP 4: Use the statistical parameters of stream concentration computed in the previous step to construct graphical or tabular displays summarizing the frequency distribution.

(a) To construct a probability plot using log-probability graph paper:

- The median concentration is plotted at the 50th percentile position.

$$\begin{aligned}\widetilde{CO} &= CO_{50\%} \\ &= \exp(\mu_{\ln CO}) \\ &= \exp(-1.95) \\ &= 0.142\end{aligned}$$

- Any other plotting position is determined as follows:

(1) From Table A-1, select a probability (α) and determine the corresponding value of Z_α. For example,

Probability = 0.841 (84%) - - - - - $Z_{84.1\%} = 1.00$

Probability = 0.159 (16%) - - - - - $Z_{15.9\%} = -1.00$

(2) Compute the concentration at probability (α) from the log mean and log standard deviation of stream concentration (CO).

$$CO_\alpha = \exp(\mu_{\ln CO} + Z_\alpha \cdot \sigma_{\ln CO})$$

84% plotting position

$$CO_{84\%} = \exp(-1.95 + 1.00 \cdot 1.23) = 0.487$$

16% plotting position

$$CO_{16\%} = \exp(-1.95 - 1.00 \cdot 1.23) = 0.0416$$

(3) Plot these concentrations on log-normal probability paper and connect with a straight line.

COMMENTARY

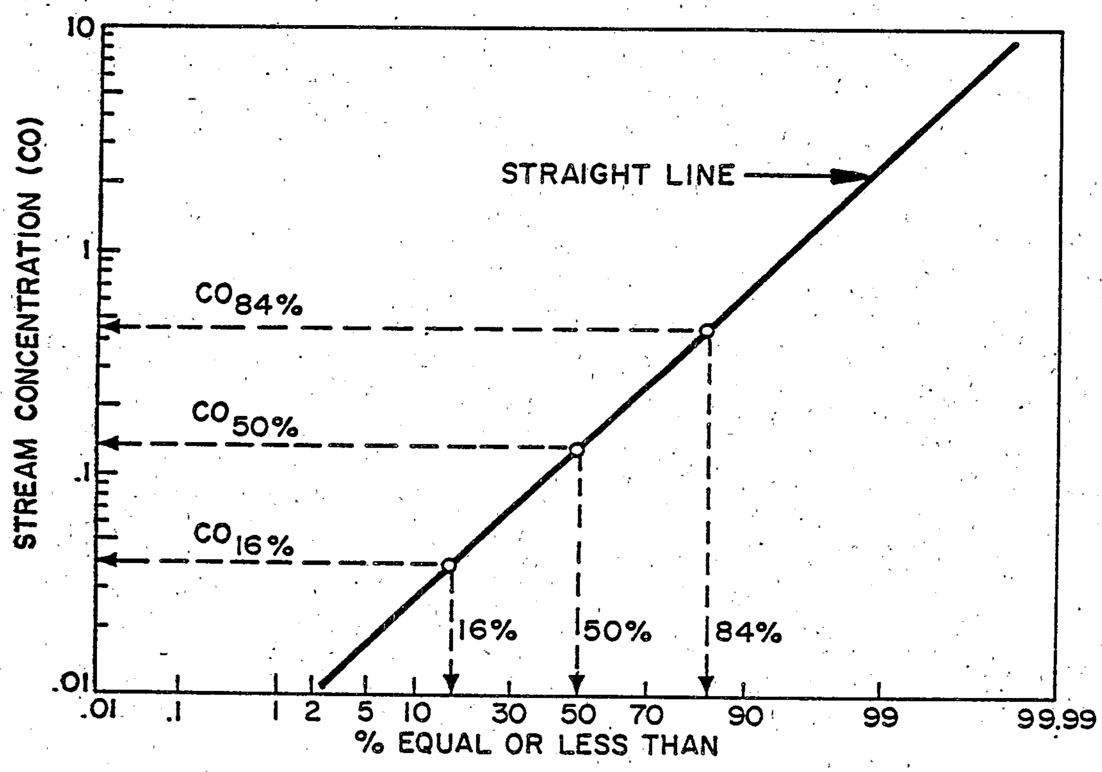


Figure 3-2 - Sample stream concentration versus probability plot for 30-day averaging period.

The probability plot indicates, for example, that the stream concentration of pollutant (P) will exceed a concentration of 1.0, at a frequency (probability) of about 5%. Since the analysis is based on daily values, this is interpreted as: 5% of all days will have stream concentrations greater than 1.

EXAMPLE COMPUTATION - HAND CALCULATION (continued)

STEP 4 (continued)

(b) To construct a recurrence interval (return period) plot using log-log graph paper:

- The formula used in the previous step

$$CO_\alpha = EXP(\mu_{lnCO} + Z_\alpha \cdot \sigma_{lnCO})$$

can be rearranged:

$$Z_\alpha = \frac{\ln(CO_\alpha) - \mu_{lnCO}}{\sigma_{lnCO}}$$

The log mean and log standard deviation were determined in Step 3:

$$\mu_{lnCO} = -1.95$$

$$\sigma_{lnCO} = 1.23$$

- Plotting positions are determined as follows:

(1) Select a series of values for stream concentration (CO) covering a range of interest, take the natural log (ln) and compute the value of Z.

(2) From Table A-1 identify the probability (Pr) associated with each Z.

(3) Compute the mean recurrence interval (MRI) for each of the selected concentrations:

$$\text{MRI (years)} = \frac{1}{Pr} \cdot \frac{1}{365 \text{ day/yr}}$$

For example:

Stream Concentration CO	Z	Probability Greater Than	Mean Recurrence Interval (years)
15	3.787	7.626×10^{-5}	35.9
10	3.457	2.732×10^{-4}	10.0
5	2.894	1.902×10^{-3}	1.44
1	1.585	5.648×10^{-2}	0.0485

Plot results. If necessary, compute additional values to assist in drawing a smooth curve.

COMMENTARY

--- Probability results can be misleading for the water quality issues being considered here, unless interpreted very carefully. For example, a 1% probability of exceeding a significant stream concentration means that this occurs nearly 4 times in 1 year, and for more than a month of individual days over a 10 year period. Expressing results as recurrence intervals is believed to provide a more useful expression of analysis results.

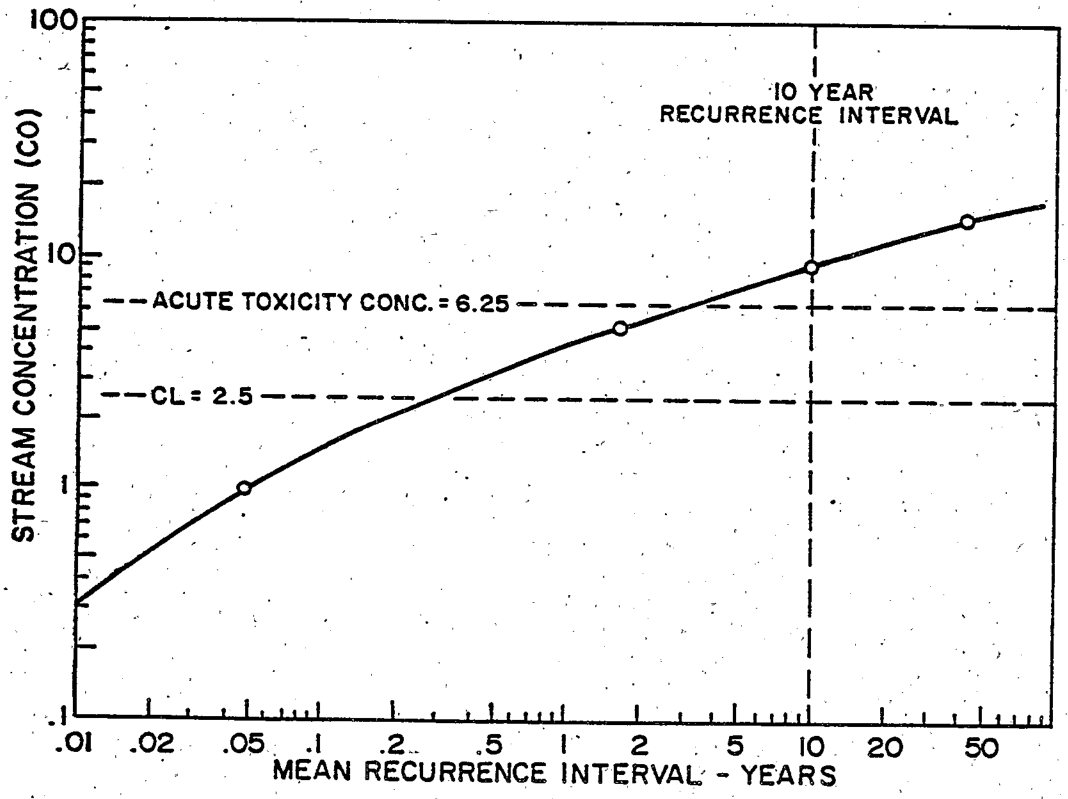


Figure 3-3 - Sample stream concentration versus mean recurrence interval for 30-day averaging period.

Note that the acute concentration assumed for the pollutant (6.25) is exceeded an average of once every 2.6 years. If the exceedance criteria to be met is an average of 1 acute toxicity exceedance every 10 years, then the assignment of a 30-day permit averaging period is insufficient; shorter averaging periods must be examined.

However, if the pollutant had an acute concentration of 12.5 (or an acute-to-chronic ratio of 5), the recurrence interval of 20 years would be sufficiently protective for acute events.

EXAMPLE COMPUTATION - HAND CALCULATION
(continued)

STEP 5: Compute the receiving water quality impact that would result from assigning other permit averaging periods.

Repeat Steps 1 - 4 using the values for $\overline{CE}$ that have been calculated for weekly and daily permit assignment.

7-day permit average - - - - - - - - $\overline{CE}$ = 4.39

Daily maximum permit average - - - $\overline{CE}$ = 2.81

All other imputs remain unchanged.

When the computations are repeated using these values, the statistical parameters for stream concentration (Step 3) that are developed are as follows:

STREAM CONCENTRATION (CO) STATISTICS

Permit Averaging Period	Mean μ_{CO}	Median $\widetilde{CO}$	Std. Dev. σ_{CO}	Coef. Var. ν_{CO}	Mean $\mu_{\ln CO}$	Std. Dev. $\sigma_{\ln CO}$
30-Day	0.303	0.142	0.570	1.88	-1.95	1.23
7-Day	0.207	0.0971	0.389	1.88	-2.33	1.23
1-Day	0.132	0.0622	0.248	1.88	-2.78	1.23

Probability and recurrence interval plots are then constructed as described in Step 4 to provide a graphical comparison of the influence of alternative choices for averaging period on the frequency of exceeding acutely toxic concentrations of pollutant (P) in the receiving system.

COMMENTARY

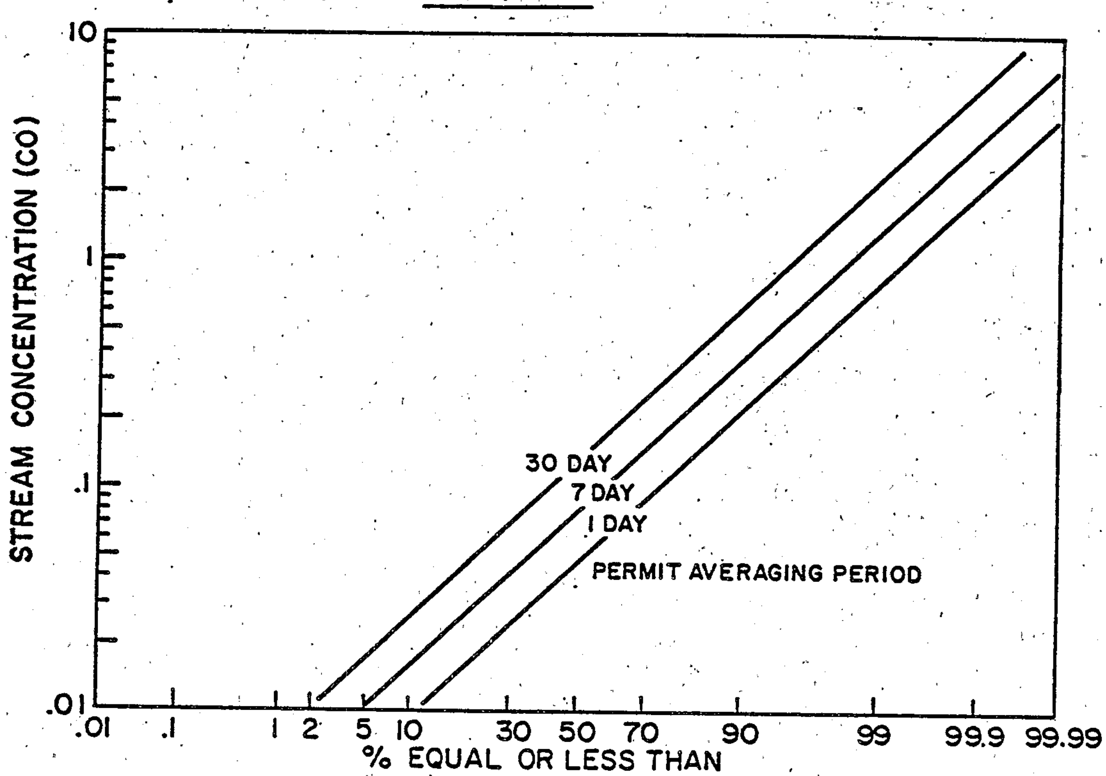


Figure 3-4 - Concentration versus probability plot for 1-, 7-, and 30-day averaging periods.

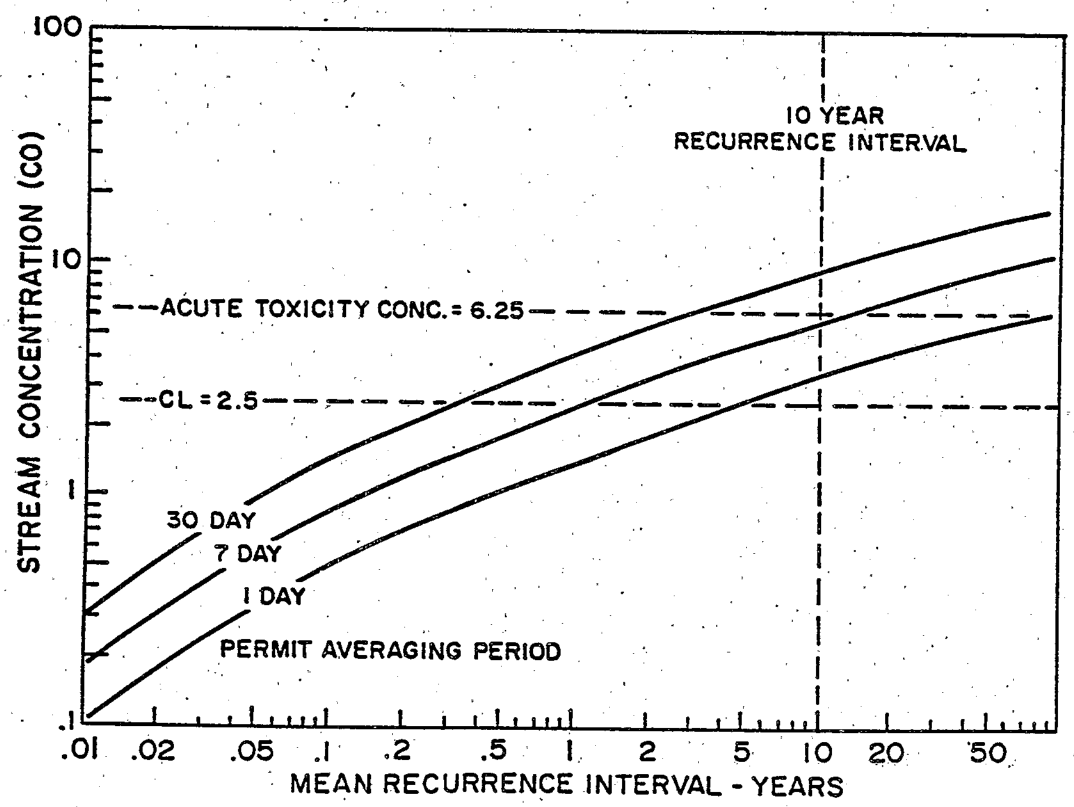


Figure 3-5 - Concentration versus mean recurrence interval plot for 1-, 7-, and 30-day averaging periods.

EXAMPLE COMPUTATION - HAND CALCULATION
(continued)

STEP 6: Select the appropriate permit averaging period.

The appropriate permit averaging period is chosen to provide an acceptable level of receiving water quality. The decision is based on the assumption that an unacceptable exceedence of the acute criteria in the receiving stream is more than once every 10 years, on average.

Therefore, the permit averaging period selected is the highest one that does not result in a mean recurrence interval for acute criteria violations that is less than 10 years. For this example, recurrence intervals for a stream concentration of 6.25 are approximately

30-Day Avg. Period = 2.6 years

7-Day Avg. Period = 7.7 years

1-Day Avg. Period = 31 years

For the site specific conditions assumed for this example, a 1-day permit averaging period could be assigned to the effluent limit of 10. However, as shown below using more exact calculations, a 7-day permit averaging period is sufficiently protective for acute events. Thus a 7-day permit averaging period is assigned to the effluent limit of 10.

COMMENTARY

--- For marginal cases, it should be recognized that the projections made using the moments approximation tend to be conservative. As shown below the more exact recurrence intervals are 6.4, 32, and 280 years.

--- The acceptable frequency of acute criteria violation is, of course, a policy decision. Alternate levels are evaluated directly from Figures 3-3 and 3-4.

--- The moments approximation used for the foregoing computations (because it approximates the distribution of dilution factor (ϕ) with a log-normal distribution) provides an approximation of the probability distribution and recurrence interval of the stream concentrations.

An exact computation that avoids the necessity of this approximation, is provided by use of the computer program detailed in the next section and in Appendix D. In this case, its use is warranted since a 7-day permit averaging period is sufficiently protective.

Based on the selection of the 7-day permit averaging period, the maximum 7-day average permit limits = EL = 10 mg/l. This permit limit is equivalent to a long-term average effluent concentration $\overline{CE} = R_\alpha \cdot EL = (0.439)(10) = 4.39$, with coefficient of variation daily effluent concentration (ν_{CE}) = 0.7. Thus, the design of the treatment facility and the selection of treatment process should be made to meet these specifications of $\overline{CE}$ = 4.39 mg/l with coefficient of variation of daily effluent concentrations ν_{CE} = 0.7.

EXAMPLE COMPUTATION - HAND CALCULATION (continued)

STEP 7: Compute permit limits for other averaging periods (daily maximum and monthly) and exceedence percentiles (1 percent and 5 percent) that are consistent with the treatment performance level established in Step 6.

At this point in the analysis, it has been determined that assigning the effluent limit of EL = 10 as a weekly permit, applicable to 7 day averages of the daily concentrations, is sufficiently protective. This choice is based upon an effluent limit violation frequency of one percent. The mean effluent concentration for these choices is $\overline{CE} = 4.39$.

If it is assumed that the same violation frequencies apply to the other permit concentrations, then they can be computed directly:

$$\text{Permit Limit} = \overline{CE}/R_\alpha$$

since $R_\alpha = \overline{CE}/CE_\alpha$ and the permit limits are assumed to be the α-percentile concentrations for each averaging period.

If other violation frequencies are desired, for example, 5 percent, then permit limits of this frequency can also be calculated using the appropriate R_α for $1-\alpha$ = 5 percent. The table below presents the results for the example considered above.

Permit Averaging Period	Coeff. of Var. of Avg.'ed Effluent Concentration[a] ν_{CE}	Reduction Factors[b] R_α 1%	Reduction Factors[b] R_α 5%	Permit Limits[c] 1%	Permit Limits[c] 5%
1-day	0.70	0.281	0.432	15.6	10.2
7-day	0.40	0.439	0.571	10.0	7.69
30-day	0.20	0.643	0.736	6.83	5.96

It should be pointed out that any or all of these permits are equivalent in the sense that a treatment plant meeting any of these requirements will also meet the desired water quality goal. Of course, this is true only if the actual coefficients of variation for daily values and 7 and 30 day average plant effluent concentrations are as specified.

[a] These are assumed to be representative of the treatment plant effluent behavior.
[b] Table 2-1, equation 2-17.
[c] Permit limit = $\overline{CE}/R_\alpha$; $\overline{CE}$ = 4.39.

COMMENTARY

Permit Limits	Daily Maximum	Weekly	Monthly
Reduction factors (see p. 3-6)	0.281	0.439	0.643
Choice of averaging period (from step 6)	no	yes	no
Value for the selected averaging period (from step 6 - steady state model output)	-	10	-
Permit limits using reduction factors, R_{α}'s	$\frac{10\,(0.439)}{0.281} = 15.6$	10.0	$\frac{10\,(0.439)}{0.643} = 6.8$
Long-term average effluent concentration, $\overline{CE}$ (see p. 3-6)	4.39	4.39	4.39
Coefficient of variation of daily, weekly, and monthly permit limits (see p. 3-6)	0.7	0.4	0.2

The long term average effluent concentration for the required level of treatment is equal to 4.39 mg/l with the coefficient of variation of daily effluent concentrations equal to 0.7. To meet the water quality standard at the state specified design flow and to meet the acute criteria at all times except for 1 day once in 10 years, the treatment facilities need to be built to meet the long term average concentration of 4.39 mg/l with coefficient of variation of daily effluent concentration $\nu_{CE} = 0.7$. The permit limits derived above are based on daily, weekly, and monthly reporting procedures. If less than adequate monitoring is required, the appropriate permit limits must be derived using the long term average and equivalent coefficient of variation.

EXAMPLE COMPUTATION - HAND CALCULATION (continued)

Recapitulation-

In order to aid in the understanding of the suggested procedure, the sequence is reviewed below in outline form.

1. Establish streamflow characteristics.

   $\overline{QS}$ ν_{QS}

2. Establish effluent flow characteristics.

   $\overline{QE}$ ν_{QE}

3. Establish effluent concentration variability characteristics (ν_{CE}) for daily values and 7 and 30 day averages.

Averaging Period	Coefficient of Variation ν_{CE}
1-day	0.7
7-day	0.4
30-day	0.2

4. Establish effluent limit from steady state wasteload allocation.

   EL = 10

5. Establish violation frequency of EL.

$$1-\alpha = 1\%$$
$$\alpha = 99\%$$

and assume $CE_\alpha = EL$

COMMENTARY

1. These should be site specific since the computation is usually sensitive to the values.

2. Mean effluent flow is important, but the coefficient of variation, since it is usually small, is usually not significant if $\nu_{QE} << \nu_{QS}$.

3. These coefficients of variations specify the behavior of the daily values and temporal averages of effluent concentrations. More detailed evaluations for industry specific or pollutant specific situations are required to be more definitive. The values used are not suggested as universal.

4. The analysis presented in this manual does not evaluate the degree of protection afforded by this choice. That is, the probability of violation of the chronic criteria is not calculated. It is assumed to be sufficiently protective.

5. The choice of violation frequency is necessary in order to give a specific probabilistic meaning to EL. Reasonable values appear to be one or five percent. However, a problem may arise if too frequent a violation frequency is chosen. It may turn out that even specifying the permit as a daily maximum does not insure that acute criteria violations are sufficiently rare. In this case, a lower probability of violation must be specified.

EXAMPLE COMPUTATION - HAND CALCULATION
(continued)

6. For α (step 5) and coefficients of variation (step 3) compute ratio of mean effluent to effluent limit, $R_\alpha = \overline{CE}/CE_\alpha$ and the resulting mean effluent concentration $\overline{CE}$ for each averaging period.

Averaging Period	Reduction Factor R_α	Mean Effluent Concentration $\overline{CE}$
1-day	0.281	2.81
7-day	0.439	4.39
30-day	0.643	6.43

7. Evaluate each mean effluent concentration using PDM to compute the return period of acute criteria violation. Choose the appropriate averaging period.

Averaging Period	$\overline{CE}$	Return Period (years) for CO = 6.25: Moments Approximation	Return Period (years) for CO = 6.25: Quadrature Method	
1-day	2.81	31	281	
7-day	4.39	7.7	31.8	> 10 years
30-day	6.43	2.6	6.44	

8. Establish appropriate permit limits for other averaging periods. $\overline{CE} = 4.39$, $1 - \alpha = 1\%$.

Averaging Period	ν_{CE}	R_α	Permit Limit[a]
1-day	0.70	0.281	15.6
7-day	0.40	0.439	10.0
30-day	0.20	0.643	6.83

[a] Permit Limit = $\overline{CE}/R_\alpha$; 1% violation frequency.

COMMENTARY

6. This calculation makes the connection between the effluent limit and the mean effluent concentration required to meet the effluent limit if it is assigned to daily values or 7 or 30 day averages. A treatment plant designed to produce $\overline{CE}$ and whose variability is as specified in (3) will meet the effluent limit with one percent violation frequency.

7. The three treatment plant designs (the three mean effluent concentrations) and the daily effluent variability are used in PDM to compute the return period of an acute criteria violation. The moments approximation is sufficient if the return periods are significantly less than or greater than the 10 year criteria violation frequency being examined. In this case, the 7-day averaging period result is close to 10 years and the more accurate computer method is used to improve the accuracy of the calculation. The calculation indicates that a mean effluent concentration of $\overline{CE}$ = 4.39 and a daily ν_{CE} = 0.7 is sufficiently protective for acute criteria violations. This, then, is the basis for the treatment plant design.

8. The permit limits for the other averaging periods are now calculated to be consistent with the treatment plant design. That is, these permit limits are consistent with effluent mean and coefficients of variation as indicated, and specify the same performance. Thus, they are equivalent requirements.

EXAMPLE COMPUTATION - COMPUTER PROGRAM

This section illustrates the use of the PDM-PS computer program (included and described in Appendix D) to the solution of the example presented in the previous section. The site-specific conditions used to define input values in the previous section are used in this section as well.

The PDM-PS is structured to accept inputs in the form of statistical parameters and ratios, determined readily from the data. The following ratios are entered for this example computation:

Stream Flow Ratio	$7Q10/\overline{QS} = 23.3/467 = 0.05$
Effluent Dilution Ratio	$7Q10/\overline{QE} = 23.3/7.77 = 3.0$
Effluent Concentration Reduction Factor	$\overline{CE}/EL$ = (*)

(*) Reduction factor assigned depends on permit averaging period. As determined earlier,

	30 Day - - - - R = 0.643
$\overline{CE}/EL$	7 Day - - - - R = 0.439
	1 Day - - - - R = 0.281

The only other inputs called for are the coefficients of variation of stream flow, effluent flow, and effluent concentration, which have already been determined.

The facing page illustrates the input prompts that are displayed when the program is run, and the values entered in response to the prompts, in this case for evaluating the 30-day permit averaging period.

COMMENTARY

DISPLAY AND PROMPTS	RESPONSE ENTRIES
POINT SOURCE - RECEIVING WATER CONCENTRATION ANALYSIS	
++++++++++++++++++++++++++++++ INPUTS: COEF VAR OF QS,QE,CE RATIO...7Q10/avgQS RATIO...7Q10/avgQE RATIO...avg CE/EL	
BACKGROUND STREAM CONC (CS) IS ASSUMED TO BE ZERO ++++++++++++++++++++++++++++++	
ENTER COEF VAR OF QS,QE,CE? - - - - - - - - - - -	1.5, 0.2, 0.7
ENTER FOLLOWING RATIOS: ....7Q10/avg QS ? - - - - - - - - - - - - - -	0.05
.......7Q10/avg QE ? - - - - - - - - - - - - - -	3.0
.......avg CE/EL? - - - - - - - - - - - - - - - -	0.643
ENTER LOWEST,HIGHEST,AND INCREMENT OF MULT OF TARGET FOR WHICH % EXCEED IS DESIRED ?	This prompt repeats after the selected range of values has been computed and displayed. It allows the user to be guided by output in selecting values and ranges for subsequent computations.
ENTER LOWEST,HIGHEST,AND INCREMENT OF MULT OF TARGET FOR WHICH % EXCEED IS DESIRED ?	
	0.01, 0.06, 0.01
	0.08, 0.36, 0.04
	0.40, 4.0, 0.2

NOTE: The manual analysis presented earlier, computed the exceedance probability and recurrence interval for specific stream concentration values. The computerized computation generates these results for stream concentrations expressed as multiples of the target concentration (CL) that is explicitly assumed to result when

Effluent Concentration	CE = EL	(the effluent limit)
Effluent Flow	$QE = \overline{QE}$	(average QE)
Stream Flow	QS = 7Q10	(the design stream flow)

EXAMPLE COMPUTATION - COMPUTER PROGRAM
(continued)

PROGRAM OUTPUT

```
********************************
   RECEIVING WATER CONC (CO)
   PROBABILITY DISTRIBUTION
      AND RETURN PERIOD
 FOR MULTIPLES OF TARGET CONC.
  DUE TO POINT SOURCE LOADS
********************************

   COEF VAR.....QS =   1.50
   COEF VAR.....QE =   0.20
   COEF VAR.....CE =   0.70

      7Q10/ave QS  =   0.05
      7Q10/ave QE  =   3.00
      ave CE/ EL   =   0.64

++++++++++++++++++++++++++++++++
```

STREAM CONCENTRATION (CO)

MULT OF TARGET (CO/CL)	PERCENT OF TIME EXCEEDED	RETURN PERIOD (YEARS)
0.01	92.699	0.003
0.02	80.916	0.003
0.03	71.039	0.004
0.04	62.788	0.004
0.05	55.862	0.005
0.08	40.808	0.007
0.12	28.659	0.010
0.16	21.170	0.013
0.20	16.201	0.017
0.24	12.728	0.022
0.28	10.206	0.027
0.32	8.320	0.033
0.36	6.875	0.040
➤ 0.40	5.746	0.048
0.60	2.650	0.103
0.80	1.399	0.196
1.00	0.804	0.341
1.20	0.490	0.559
1.40	0.312	0.878
1.60	0.206	1.331
1.80	0.140	1.961
2.00	0.097	2.821

COMMENTARY

--- This output is for a 30-day permit average period ($R_a = 0.643$)

The range of values selected here is broad enough to facilitate construction of probability and recurrence interval plots.

Stream concentrations listed are in terms of a ratio to the target concentration (CL). In this example, the target stream concentration is:

CL = 2.5

Actual stream concentration is this value multiplied by the listed value: e.g., the multiple of Target (CO/CL) = 0.4

Corresponding stream concentration is:

0.4 X 2.5 = 1.0

Since the acute-to-chronic ratio for pollutant (P) is 6.25/2.50 = 2.5, acute exceedences are reflected by multiple 2.5.

Probability or recurrence interval plots can be constructed, simply by plotting the values listed in the computer printout.

Note that the probability distribution of stream concentrations deviates from log-normal (a straight line) at the higher exceedance percentiles.

EXAMPLE COMPUTATION - COMPUTER PROGRAM
(continued)

STREAM CONCENTRATION (CO) (cont.)

MULT OF TARGET (CO/CL)	PERCENT OF TIME EXCEEDED	RETURN PERIOD (YEARS)
2.20	0.069	3.977
2.40	0.050	5.507
2.60	0.036	7.509
2.80	0.027	10.098
3.00	0.020	13.411
3.20	0.016	17.612
3.40	0.012	22.894
3.60	0.009	29.482
3.80	0.007	37.640
4.00	0.006	47.674

COMMENTARY

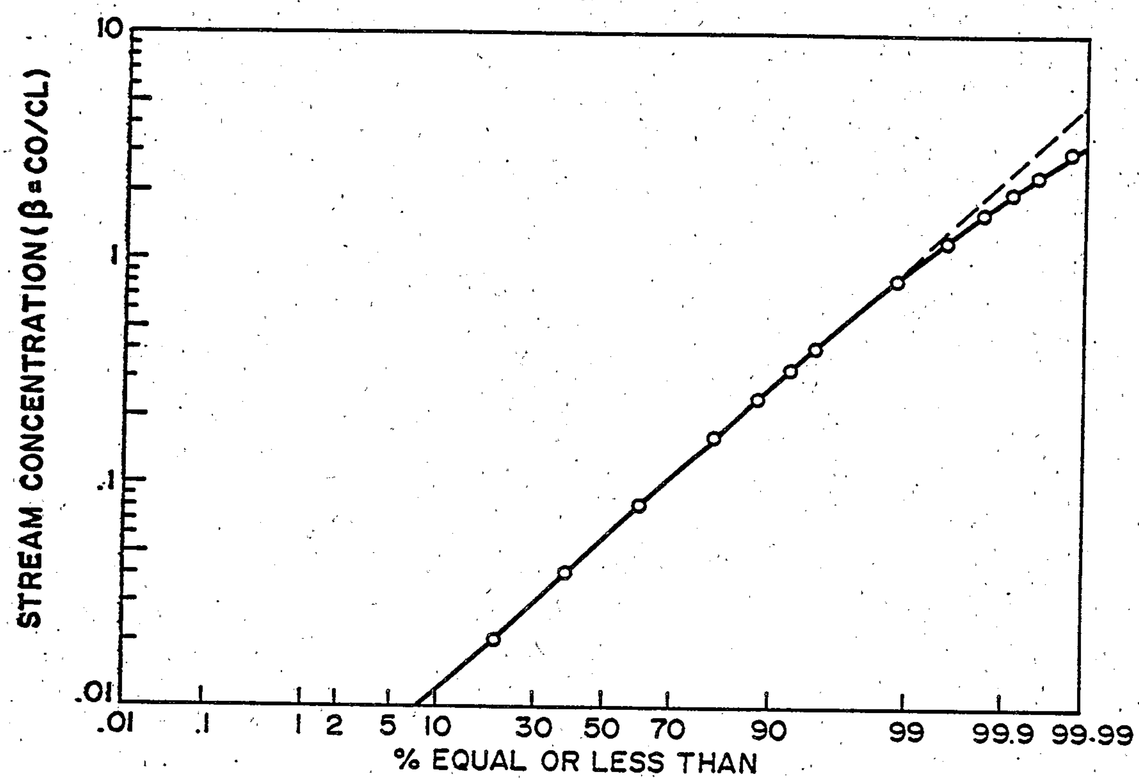


Figure 3-6 – Concentration versus probability for PDM-PS computation.

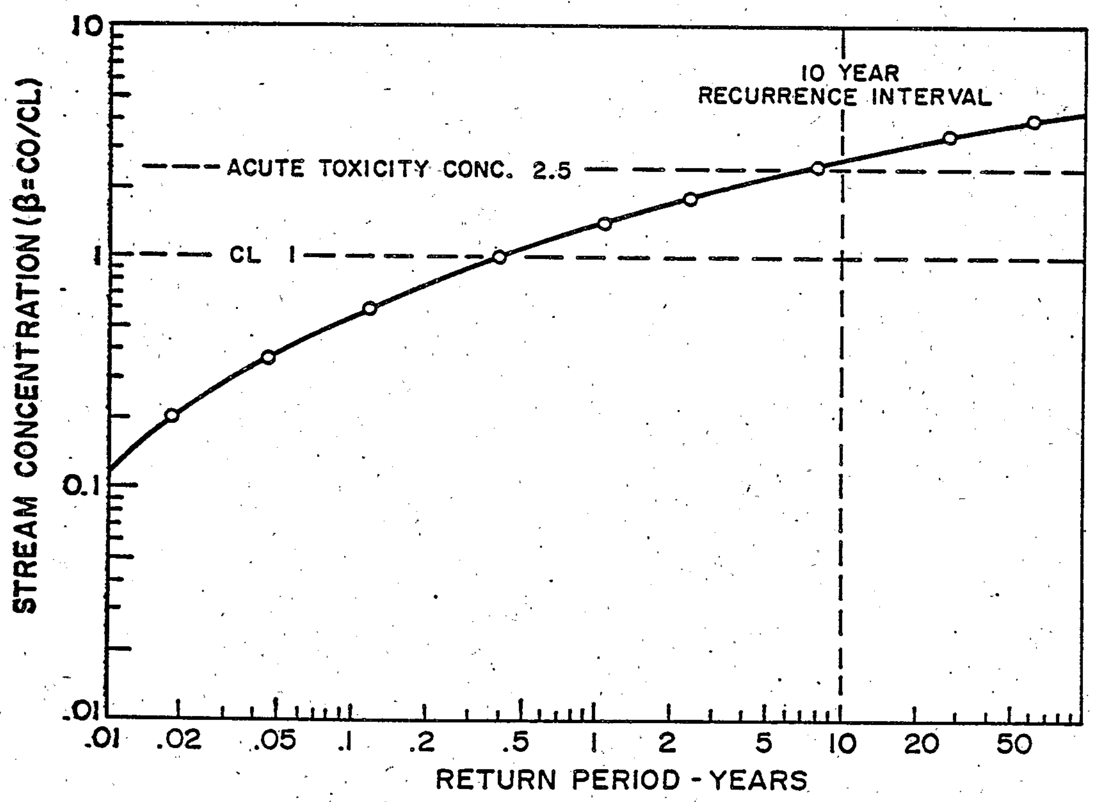


Figure 3-7 – Concentration versus mean recurrence interval for PDM-PS computation.

EXAMPLE COMPUTATION - COMPUTER PROGRAM
(continued)

To examine stream concentration effects for other permit averaging periods, repeat the analysis, substituting the appropriate value for the reduction factor ($R = \overline{CE}/EL$).

The return period curves provide a useful summary and perspective; however, the evaluation can be performed without constructing the graph. In this case, the range of concentrations specified might (as shown below) simply bracket those of principal interest. In this case, a range of CO/CL from 0.5 to 3 is selected, because the chronic limit (CL = 1), and the acute limit to be exceeded no more than once every 10 years is CO/CL = 2.5.

The relevant portions of the output for the three permit averaging periods are shown below:

STREAM CONCENTRATION (CO)

	MULT OF TARGET (CO/CL)	PERCENT OF TIME EXCEEDED	RETURN PERIOD (YEARS)
30-Day Average	0.50	3.818	0.072
	1.00	0.804	0.341
$\overline{CE}/EL = 0.643$	1.50	0.252	1.085
	2.00	0.097	2.821
	2.50	0.043	6.443
	3.00	0.020	13.411
7-Day Average	0.50	1.717	0.160
	1.00	0.272	1.008
$\overline{CE}/EL = 0.439$	1.50	0.069	3.957
	2.00	0.023	12.149
	2.50	0.009	31.819
	3.00	0.004	74.364
1-Day Average	0.50	0.560	0.489
	1.00	0.060	4.601
$\overline{CE}/EL = 0.281$	1.50	0.011	23.866
	2.00	0.003	90.571
	2.50	0.001	281.076
	3.00	0.000	756.249

COMMENTARY

In this case a different averaging period would be selected than that based upon the manual computation. Acute criteria exceedences have a mean recurrence interval shorter than 10 years for a 30-day permit average, so it would be rejected in favor of a 7-day average, which meets the guideline.

Note that the exact computation using the computer program indicates a 6.4 year return period for acute violations, compared with a 2.6 year return period estimated by the manual approximation. The manual approximation tends to give conservative projections for the longer return periods that are of interest, though differences vary depending on specific input conditions.

Hence, there will be marginal cases where the approximate computation may reject a 30-day average inappropriately.

On the other hand, wherever the manual approximation accepts a 30-day permit average as appropriate, it is safe to assume that the more exact computation will not modify the choice.

For the site specific conditions assumed for the example analysis:

- Any pollutant with an acute-to-chronic ratio of 9.5 or greater would, based on the manual approximation, always be assigned a 30-day permit average.
- The PDM-PS computation extends this to pollutants with acute-to-chronic ratios of 3 or more.

NOTE: EPA interprets any return period greater than 25 years as being highly improbable.

CHAPTER 4

RANGE OF EXPECTED VALUES FOR STREAMS IN U.S.

As illustrated in Chapter 3, the method can be applied to any site specific evaluation for which the relevant statistical parameters are available or can be estimated. The purpose of this section is to present a concise summary of the results of such computations for the range of site conditions that are likely to be encountered in practice. This chapter provides such a compilation along three lines. Section 4.1 describes the basis for the input values selected to provide a representative range of site conditions, and presents the results of an analysis using these typical ranges in the methodology described previously. The stream flow characteristics were determined from an analysis of 180 streams and rivers; treatment plant effluent characteristics are based on analysis of data from over 400 POTWs. The results in this section apply for conservative (nonreacting) pollutants. Section 4.2 describes how the information provided by such an analysis can be used as a screening tool for selecting permit averaging periods. Section 4.3 presents results of a similar analysis, except that it is specific to oxygen depletion by biochemical oxygen demand (BOD) loadings. Section 4.4 extends the analysis for conservative pollutants to the special case of streams that are highly effluent dominated, including those with significant zero-flow periods.

4.1 Analysis for Conservative Substances

The review of stream flow and effluent statistics presented in Appendix B indicates that the following ranges are reasonable. Effluent concentration variability, (ν_{CE}), is in the range of ν_{CE} = 0.3 - 1.1. Effluent flow variability, (ν_{QE}), is generally small relative to stream flow variability and, therefore, does not greatly influence the computation. ν_{QE} = 0.2 is consistently used. Stream flow variability follows from the empirical relationship of ν_{QS} and $7Q10/\overline{QS}$. For a specified ratio, the range of ν_{QS}, as indicated by the data discussed in Appendix B, is used. The ratio $7Q10/\overline{QS}$ varies considerably. A representative range is $7Q10/\overline{QS}$ = 0.01 - 0.25. Finally, the magnitude of the effluent flow relative to the stream flow is specified by the effluent dilution ratio: $7Q10/\overline{QE}$. A range from $7Q10/\overline{QE}$ = 1 - 50 is chosen to represent effluent dominated streams and large streams with small discharges. A 10 year return period has been selected as the acute criteria violation frequency.

In order to compute the ratio of the mean effluent concentration to the effluent limit $R_\alpha = \overline{CE}/EL$, it is assumed that the permit violation frequency is one percent. The final specification required is the relationship of 7 and 30 day average effluent concentrations to the daily effluent concentration coefficient of variation, ν_{CE}. Based upon the data presented in Table C-2, it appears reasonable to expect that the 7-day averages have a coefficient of variation that is 0.8 of the daily values, and that 30 day averages have a coefficient of variation of 0.6 of the daily values. Thus, the reduction factors used are:

Coefficient of Variation of Daily Values ν_{CE}	Reduction Factor, R_α α = 99 Percent		
	1-day	7-day	30-day
0.3	0.527	0.593	0.671
0.7	0.281	0.340	0.425
1.1	0.187	0.229	0.296

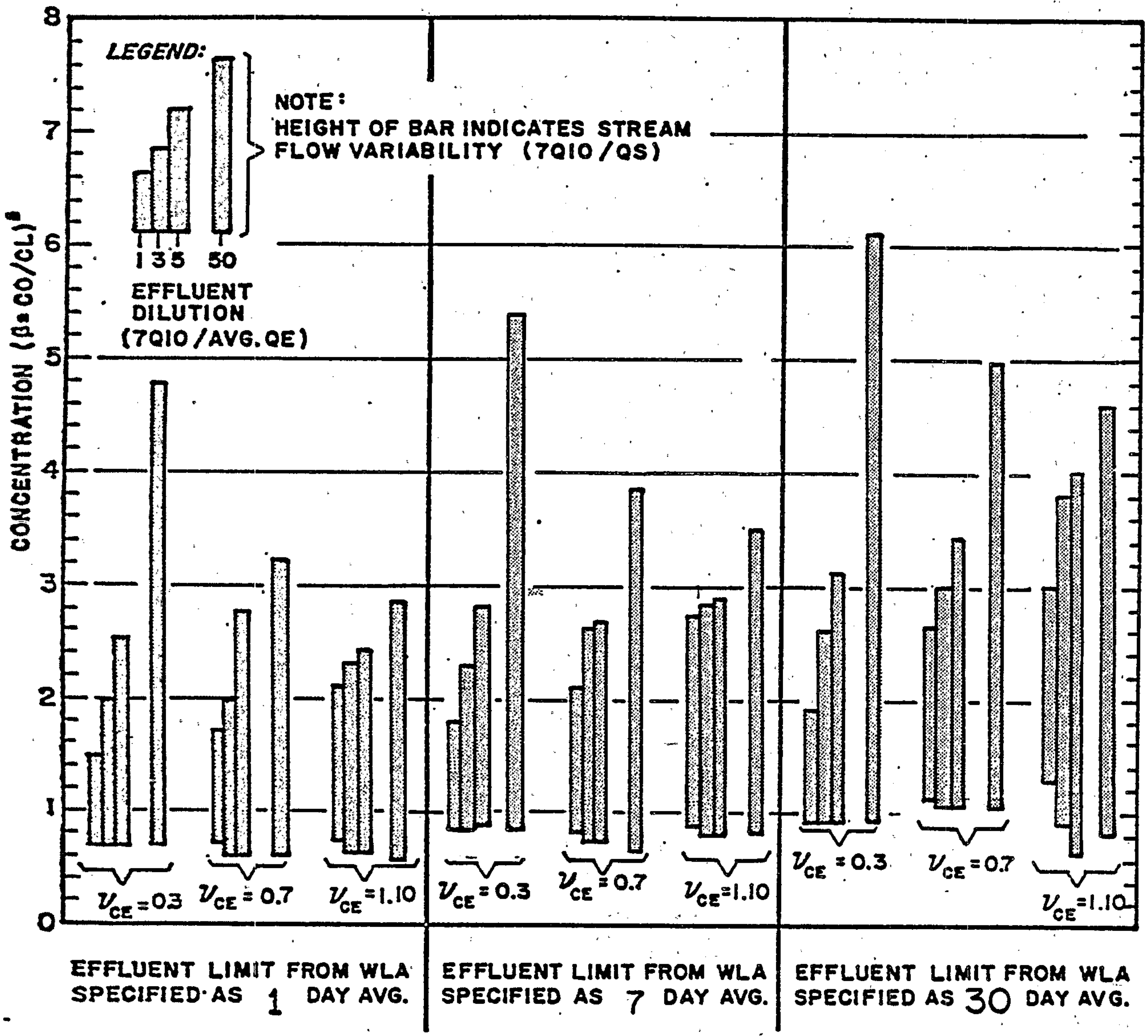
The results of these computations are summarized in Figure 4-1 and given in detail in Tables 4-1 to 4-4. The three choices for permit average are shown. Each group of bars represents the range in effluent concentration variability, ν_{CE}. Each individual bar represents a particular effluent dilution, $7Q10/\overline{QE}$. Finally, the length of each bar represents the range that results from the range of stream flow variability ($7Q10/\overline{QS} = 0.01 - 0.25$) and the associated coefficient of variation, ν_{QS}. The ordinate is the downstream concentration (in multiples of the chronic criteria) which has a 10 year return period.[1]

A number of features are immediately apparent. For pollutants with an acute to chronic ratio of greater than 10, no acute criteria violations are projected over the ranges investigated, and 30-day average permit specifications appear to be sufficiently protective. For acute-to-chronic ratios of less than 10, site specific considerations are important.

The results are most sensitive to the stream flow parameter $7Q10/\overline{QS}$, as can be seen from the range covered by each bar. For example, the last bar in the figure, 30-day permit averaging period, $7Q10/\overline{QE} = 50$, $\nu_{CE} = 1.1$, covers the range from $\beta = 0.9$ to 4.6, corresponding to $7Q10/\overline{QS} = 0.01$ and $\nu_{QS} = 2\text{-}4$.

Following, in order of decreasing sensitivity, is the effluent dilution ratio: $7Q10/\overline{QE}$. A significant distinction can be found between

[1] The EPA is presently considering the issue of allowable duration and frequency of exposure to toxicity. Based upon this work, duration and frequencies used as the decision criteria may change. This guidance does not recommend any particular minimum acceptable duration or frequency.



*INDICATES THE STREAM CONCENTRATION (CO) WHICH WILL BE EXCEEDED WITH A FREQUENCY OF ONCE IN TEN YEARS, EXPRESSED AS A MULTIPLE OF THE CHRONIC CRITERIA (CL).

Figure 4-1 - Effect of permit averaging period on stream concentrations for conservative substances: general analysis.

TABLE 4-1 - Averaging period selection matrix for conservative substances: effluent dilution ratio - $7Q10/\overline{QE}$ = 50.

Stream Flow $7Q10/\overline{QS}$ Avg. Q	Estimate of Variability Range ν_{QS}		Effluent ν_{CE} = 0.3 30-Day Avg.	7-Day Avg.	1-Daily Max.	Effluent ν_{CE} = 0.7 30-Day Avg.	7-Day Avg.	1-Daily Max.	Effluent ν_{CE} = 1.1 30-Day Avg.	7-Day Avg.	1-Daily Max.
0.01	LO	2.00	1.0	0.9	0.8	0.9	0.7	0.6	0.9	0.7	0.5
	PROB	3.00	3.0	2.7	2.4	2.5	2.0	1.7	2.3	1.8	1.5
	HI	4.00	6.1	5.4	4.8	4.9	3.9	3.2	4.4	3.4	2.8
0.05	LO	1.00	0.9	0.8	0.7	0.9	0.7	0.6	1.0	0.7	0.6
	PROB	1.50	2.3	2.0	1.8	2.2	1.7	1.4	2.2	1.7	1.4
	HI	2.00	4.7	4.2	3.7	4.2	3.4	2.8	4.1	3.2	2.6
0.10	LO	0.75	1.0	0.9	0.8	1.1	0.8	0.7	1.2	0.9	0.8
	PROB	1.00	1.7	1.5	1.4	1.8	1.4	1.2	1.9	1.5	1.2
	HI	1.50	4.4	3.9	3.4	4.2	3.3	2.7	4.2	3.2	2.7
0.15	LO	0.60	1.0	0.9	0.8	1.2	0.9	0.8	1.4	1.0	0.9
	PROB	0.90	2.0	1.8	1.6	2.2	1.7	1.4	2.4	1.8	1.5
	HI	1.25	4.1	3.7	3.3	4.1	3.3	2.7	4.3	3.3	2.7
0.25	LO	0.50	1.3	1.1	1.0	1.6	1.3	1.0	1.9	1.5	1.2
	PROB	0.75	2.4	2.1	1.0	2.6	2.1	1.7	2.9	2.3	1.9
	HI	1.00	4.1	3.6	3.2	4.2	3.4	2.8	4.6	3.5	2.9

TABLE 4-2 - Averaging period selection matrix for conservative substances: effluent dilution ratio - $7Q10/\overline{QE}$ = 5.

Stream Flow 7Q10/ Avg. Q	Estimate of Variability Range ν_{QS}		Effluent ν_{CE} = 0.3			Effluent ν_{CE} = 0.7			Effluent ν_{CE} = 1.1		
			30-Day Avg.	7-Day Avg.	1-Daily Max.	30-Day Avg.	7-Day Avg.	1-Daily Max.	30-Day Avg.	7-Day Avg.	1-Daily Max.
	LO	2.00	1.0	0.9	0.8	0.9	0.7	0.6	0.9	0.7	0.6
0.01	PROB	3.00	2.2	1.9	1.7	2.0	1.6	1.3	2.0	1.5	1.3
	HI	4.00	3.1	2.8	2.5	3.1	2.5	2.1	3.1	2.4	2.0
	LO	1.00	0.9	0.8	0.7	0.9	0.8	0.6	1.0	0.8	0.7
0.05	PROB	1.50	1.9	1.7	1.5	1.9	1.5	1.3	2.0	1.6	1.3
	HI	2.00	2.9	2.6	2.3	3.0	2.4	2.0	3.2	2.5	2.0
	LO	0.75	1.0	0.9	0.8	1.1	0.9	0.7	1.3	1.0	0.8
0.10	PROB	1.00	1.5	1.4	1.2	1.7	1.4	1.1	1.9	1.5	1.2
	HI	1.50	2.8	2.5	2.2	3.1	2.5	2.1	3.4	2.6	2.2
	LO	0.60	1.0	0.9	0.8	1.2	1.0	0.8	1.5	1.1	0.9
0.15	PROB	0.90	1.8	1.6	1.4	2.0	1.6	1.3	2.3	1.8	1.5
	HI	1.25	2.8	2.5	2.2	3.2	2.5	2.1	3.5	2.7	2.2
	LO	0.50	1.3	1.1	1.0	1.6	1.3	1.1	2.0	1.5	1.3
0.25	PROB	0.75	2.0	1.8	1.6	2.4	1.9	1.6	2.8	2.2	1.8
	HI	1.00	2.8	2.5	2.2	3.4	2.7	2.2	3.9	3.0	2.4

TABLE 4-3 - Averaging period selection matrix for conservative substances: effluent dilution ratio - $7Q10/\overline{QE} = 3$.

Stream Flow 7Q10/ Avg. Q	Estimate of Variability Range ν_{QS}		Effluent $\nu_{CE} = 0.3$			Effluent $\nu_{CE} = 0.7$			Effluent $\nu_{CE} = 1.1$		
			30-Day Avg.	7-Day Avg.	1-Daily Max.	30-Day Avg.	7-Day Avg.	1-Daily Max.	30-Day Avg.	7-Day Avg.	1-Daily Max.
0.01	LO	2.00	1.0	0.9	0.8	0.9	0.7	0.6	0.9	0.7	0.6
	PROB	3.00	1.9	1.7	1.5	1.9	1.5	1.2	1.9	1.5	1.2
	HI	4.00	2.6	2.3	2.0	2.7	2.2	1.8	2.8	2.2	1.8
0.05	LO	1.00	0.9	0.8	0.7	1.0	0.8	0.6	1.1	0.8	0.7
	PROB	1.50	1.7	1.5	1.3	1.8	1.5	1.2	2.0	1.5	1.3
	HI	2.00	2.5	2.2	1.9	2.7	2.2	1.8	3.0	2.3	1.9
0.10	LO	0.75	1.0	0.9	0.8	1.2	0.9	0.8	1.3	1.0	0.9
	PROB	1.00	1.5	1.3	1.2	1.7	1.3	1.1	1.9	1.5	1.2
	HI	1.50	2.4	2.2	1.9	2.8	2.3	1.9	3.2	2.5	2.0
0.15	LO	0.60	1.0	0.9	0.8	1.3	1.0	0.8	1.5	1.2	1.0
	PROB	0.90	1.7	1.5	1.3	2.0	1.6	1.3	2.3	1.8	1.5
	HI	1.25	2.4	2.2	1.9	2.9	2.3	1.9	3.3	2.6	2.1
0.25	LO	0.50	1.3	1.1	1.0	1.6	1.3	1.1	2.1	1.6	1.3
	PROB	0.75	1.9	1.6	1.5	2.3	1.9	1.5	2.8	2.2	1.8
	HI	1.00	2.5	2.2	1.9	3.1	2.5	2.0	3.7	2.8	2.3

TABLE 4-4 - Averaging period selection matrix for conservative substances: effluent dilution ratio - $7Q10/\overline{QE} = 1$.

Stream Flow 7Q10/ Avg. Q	Estimate of Variability Range ν_{QS}		Effluent $\nu_{CE} = 0.3$ 30-Day Avg.	7-Day Avg.	1-Daily Max.	Effluent $\nu_{CE} = 0.7$ 30-Day Avg.	7-Day Avg.	1-Daily Max.	Effluent $\nu_{CE} = 1.1$ 30-Day Avg.	7-Day Avg.	1-Daily Max.
0.01	LO	2.00	1.0	0.8	0.8	1.0	0.8	0.7	1.1	0.8	0.7
	PROB	3.00	1.5	1.3	1.2	1.7	1.3	1.1	1.8	1.4	1.1
	HI	4.00	1.8	1.6	1.4	2.1	1.7	1.4	2.4	1.8	1.5
0.05	LO	1.00	0.9	0.8	0.7	1.1	0.9	0.7	1.3	1.0	0.8
	PROB	1.50	1.4	1.3	1.1	1.7	1.4	1.2	2.0	1.6	1.3
	HI	2.00	1.8	1.6	1.4	2.2	1.8	1.5	2.6	2.0	1.7
0.10	LO	0.75	1.0	0.9	0.8	1.3	1.0	0.9	1.6	1.2	1.0
	PROB	1.00	1.3	1.2	1.1	1.7	1.4	1.1	2.0	1.6	1.3
	HI	1.50	1.8	1.6	1.4	2.4	1.9	1.6	2.9	2.2	1.8
0.15	LO	0.60	1.1	1.0	0.8	1.4	1.2	1.0	1.8	1.4	1.2
	PROB	0.90	1.5	1.3	1.2	1.9	1.6	1.3	2.4	1.9	1.5
	HI	1.25	1.8	1.6	1.4	2.5	2.0	1.6	3.0	2.3	1.9
0.25	LO	0.50	1.3	1.1	1.0	1.8	1.4	1.2	2.3	1.8	1.5
	PROB	0.75	1.6	1.4	1.3	2.2	1.8	1.5	2.9	2.2	1.8
	HI	1.00	1.9	1.7	1.5	2.6	2.1	1.7	3.3	2.6	2.1

the effluent dominated streams, $7Q10/\overline{QE} \leq 5$, and the large stream case, $7Q10/\overline{QE} = 50$. For the latter cases, the stream flow variability is a more important determinant of the normalized downstream concentration. Finally, the effluent variability, ν_{CE}, affects the results by approximately a factor of 2, all other things being equal.

4.2 Use As a Screening Tool

It is suggested that Figure 4-1 may be used as a screening tool to separate the cases which can be dealt with immediately from those for which more site specific information is required. For the latter cases, the flow ratios, $7Q10/\overline{QE}$ and $7Q10/\overline{QS}$ can usually be found quite easily so that a more specific answer can be found in Tables 4-1 to 4-4. The final determinant, ν_{QS}, requires a log-normal analysis of the stream flow record. Since this is reasonably straightforward, a more refined analysis is not excessively burdensome and would serve to reduce the range of possible values of β, from which the permit averaging decision can be made.

As an example of such a screening analysis, consider the hypothetical case of a state establishing permit averaging periods for phenol. Phenol has an acute-to-chronic ratio of 4, so that stream concentrations which exceed a multiple of 4 times the chronic concentration will not be accepted (assuming that the acute criteria is not to be exceeded on a daily basis more often than once every 10 years).

Comparing the bars on Figure 4-1 with the multiple of $\beta = 4$, the following conclusions relative to the permit averaging period can be drawn. For situations with an effluent dilution ratio of 5 or less ($7Q10/\overline{QE} \leq 5$):

a. A 30-day permit averaging period will be selected whenever the ν_{CE} is 0.7 or less.

b. Where $\nu_{CE} = 1.1$, a 7-day permit averaging period will meet requirements under all reasonable possibilities of stream flow variability (ν_{QS}). (The upper ends of the bars correspond to high values of ν_{QS}.)

c. Even for effluent variability as high as $\nu_{CE} = 1.1$, there will be many streams where it would be appropriate to select a 30-day permit average, since only the upper end of the bars exceeds a multiple of 4.

For an effluent dilution ratio $7Q10/\overline{QE} = 5$, the third column from the right ($\nu_{CE} = 1.1$; 30-day permit average) in Table 4-2 indicates that only the highly variable stream flows approach violations using a 30-day permit average. State records could be examined to determine if the set of streams under consideration (or a representative set from Appendix C) experiences ν_{QS} in this range.

A conservative decision, then, would be to select a 7-day permit averaging period, although a site-specific assessment of stream flow variability or a restriction of ν_{QS} values could be expected (in most cases) to support selection of a 30-day permit averaging period.

4.3 Preliminary Analysis for Dissolved Oxygen

The choice of permit averaging periods for effluent limits of oxygen-consuming pollutants, such as BOD or ammonia, is a more complex problem than that addressed in the previous sections. The variations of the minimum or critical DO are caused not only by effluent concentration and dilution fluctuations, which are addressed by the probabilistic dilution model, but also by fluctuations in reaction rates and other sources and sinks of DO, such as algal production, respiration, and sediment oxygen demand. Stream flow and temperature variations affect these parameters, the latter also determining the DO saturation. A comprehensive probabilistic analysis that would include these effects as well is beyond the scope of this report.

It is desirable, however, to provide at least a preliminary analysis for suitably restricted cases that are amenable to analysis using the probabilistic dilution model. The method to be employed makes use of the similarity of the formula for critical DO deficit for those streams for which the simple Streeter-Phelps formulation is adequate, and the dilution equation. The principal assumptions are (1) a single point source of BOD is the only DO sink; (2) the stream flow, geometry and reaction rates are spatially constant; and (3) the reaction rates are temporally constant. For this restrictive situation, the critical or maximum dissolved oxygen deficit (D_c) is a function of the reaeration rate (K_a), the BOD oxidation rate (K_d), and the ultimate-to-5-day BOD ratio.

The Streeter-Phelps equation can be solved for the critical or dissolved oxygen deficit (D_C):

$$D_C = CE \circ F \circ \Phi \circ P \tag{4-1}$$

where:

CE = treatment plant effluent BOD_5 concentration.

F = ratio of ultimate/5-day BOD. Stream calculations are based on ultimate BOD; effluent criteria on 5-day BOD.

Φ = stream dilution factor = QE/(QS + QE).

P = stream purification factor; for a BOD oxidation rate (K_d) and stream reaeration rate (K_a),

$P = (\lambda)^{\frac{\lambda}{1-\lambda}}$; where $\lambda = K_a/K_d$

(Note that if the purification factor were constant then Equation 4-1 would be formally equivalent to the dilution equation analyzed previously.) One remaining difficulty is that it is not the critical DO deficit (D_C) that is of concern but rather the critical dissolved oxygen (DO_C) itself:

$$DO_C = C_{sat} - D_C \tag{4-2}$$

which is a function of stream temperature through the DO saturation concentration, C_{sat}. Hence, the applicability of probabilistic dilution to the dissolved oxygen problem requires that the analysis be restricted to periods for which temperatures are essentially constant and fluctuations in the purification factor (P) are small.

An evaluation of this latter effect can be made as follows. A relationship between P and stream depth, H, which follows from K_a and K_d versus depth relationships is [3]:

$$P \simeq H^{0.8} \tag{4-3}$$

and for many streams, depth is proportional to total stream flow, Q_T, to a power $H \simeq Q_T^m$ with m = 0.4 - 0.6. Thus,

$$P \simeq Q_T^n \qquad n = 0.3 - 0.5 \tag{4-4}$$

Consider Equation 4-1 for critical deficit. Taking natural logs and applying the formula for the variance of a sum of independent random variables yields:

$$\sigma^2_{\ln DC} = \sigma^2_{\ln CE} + \sigma^2_{\ln \Phi} + n^2 \sigma^2_{\ln QT} \tag{4-5}$$

where $Q_T = QS + QE$. This equation, of course, ignores the fact that Φ and Q_T are correlated, but the point is that $n^2 = 0.09 - 0.25$ so that if the log variance of Q_T is comparable to the effluent concentration log variance, then the n^2 term is not a major contribution to critical deficit log variance; hence, it can be neglected. The fact that dilution (Φ) and total stream flow are negatively correlated would further reduce the effect.

Hence, the key observation is that if it were possible to restrict consideration to those flows for which $\nu_{QS} \simeq \nu_{CE}$, then purification factor fluctuations would not be very significant and probabilistic dilution can be applied. If these flows also correspond to periods of approximately constant temperature, then the two requirements for applying probabilistic dilution to critical dissolved oxygen have been met. For a site-specific analysis, the obvious solution is to seasonally analyze the stream flow and temperature data and apply probabilistic dilution, making any necessary corrections for purification factor variations. However, for the general case considered here, an alternate approach is required.

Consider, instead, restricting consideration to that period of the year during which flows are low. This period corresponds, presumably, to the period of time during which 7Q10 occurs, and includes the conditions for which the WLA was performed. Considering this period alone significantly reduces the variability of the stream flows to be considered. If, in addition, it can be argued that these low flows tend to occur during the same season each year, then the temperature variation is less than the annual variability and will be less significant as well. Hence, for these low flow periods, the assumption of constant P is much more realistic.

The technical problem to be solved is to compute the reduction in the average stream flow and coefficient of variation when flows are restricted to the low values for this restricted period. We restrict consideration to the lowest one-sixth of the total population. This corresponds to an average of 2 out of 12 months in each year, and the presumption is that this period recurs during the same months each year so that the temperature variation during this restricted period is small. This simplification also assumes that the lower one-sixth of the daily stream flows occur only in the two month period when temperature and reaction rates are assumed to be approximately constant.

As indicated earlier, a statistical analysis of actual stream data, stratified by month or critical season, could be performed to provide actual results and avoid the need for this type of estimate. However, data of this type are not presently available. The estimation described here is performed in order to allow a preliminary analysis for BOD/DO to be made.

The computation of the required statistical parameters, the stream flow average and coefficient of variation for flows restricted to the lower α-quantile of the total population, is straightforward. For log-normal random variables, it can be shown that these conditional moments, denoted by primes, are:

$$\frac{\overline{QS'}}{\overline{QS}} = Q(\sigma_{\ln QS} + Z_\alpha)/Q(Z_\alpha) \tag{4-6}$$

$$\nu^{2\,\prime}_{QS} = \frac{\exp(\sigma^2_{\ln QS})\, Q(2\sigma^2_{\ln QS} + Z_\alpha)\, Q(Z_\alpha)}{Q^2(\sigma_{\ln QS} + Z_\alpha)} - 1 \tag{4-7}$$

where $Q(Z^*) = \Pr\ Z > Z^*$ for Z, a standard normal random variable, and Z_α are the Z scores for the α-quantile which is the upper bound for the flows being considered. For $\alpha = 1/6$, $Z_\alpha = 0.967$. Table 4-5 presents the results. These corrections, when applied to $7Q10/\overline{QS}$ and ν_{QS} in the first two columns of Tables 4-1 to 4-4 adjust these parameters to represent the low flow periods. For highly variable streams, ν_{QS} and therefore $\sigma_{\ln QS}$ are large and the corrections are quite substantial.

Reduction factors for the mean range from 0.45 to 0.024 for the highly variable streams. The range in coefficient of variation is sharply

TABLE 4-5 - Conditional moments for the low flow subpopulation.

(α = 16.7%)

Coefficient of Variation for Entire Record ν_{QS}	Reduction in Mean $\overline{QS'}/\overline{QS}$	Reduced Coefficient of Variation $\nu_{QS'}$
0.50	0.450	0.188
0.60	0.384	0.216
0.75	0.306	0.254
0.90	0.247	0.287
1.00	0.216	0.306
1.25	0.158	0.348
1.50	0.120	0.381
2.00	0.0761	0.431
3.00	0.0389	0.500
4.00	0.0241	0.547

This table provides a basis for a preliminary estimate of the average stream flow and flow variability during critical low flow periods, relative to overall long-term characteristics. For site-specific cases, the actual values can be determined readily from a statistical analysis of stream flows during the selected critical period of the year.

compressed from ν_{QS} = 0.5 - 4.0 to ν_{QS}' = 0.19 - 0.55, so that the sub-population of low flows fluctuates much less violently than the entire population, which includes the annual cyclical variation as well.

A 10 year return period was selected for consistency with the general analysis, but since only one-sixth of the flow population is being considered, and we assume that no DO acute criteria violations occur during the remaining higher flows, the exceedence probability to be applied in the probabilistic dilution calculation is a 10/6 = 1.67 year return period. Figure 4-2 and Tables 4-6 to 4-8 present the results.

In order to properly evaluate the computations, it is necessary to realize that they apply to 10 year return period critical deficit ratios. To convert critical DO concentrations to the deficit ratio (β) shown by the tables, the DO standard (CL), the DO saturation (C_{sat}) used in the WLA, and the DO concentration taken to represent an acute criteria value are required. For most reasonable combinations of these values, the ratio will be between approximately 2.0 and 2.5. For example, if CS = 8, CL = 5, and acute DO = 2, then β = 2.0. Alternatively, if these concentrations are CS = 9.0, CL = 6.0, acute DO = 1.5, then (the acute-to-chronic deficit ratio) β = 2.5.

Appropriate permit averaging periods are seen in Tables 4-6 to 4-8 to be strongly influenced by local conditions of effluent load and stream flow variability. Because of this, a general statement on permit averaging period for effluent BOD/DO is not possible; it must be selected on the basis of site conditions.

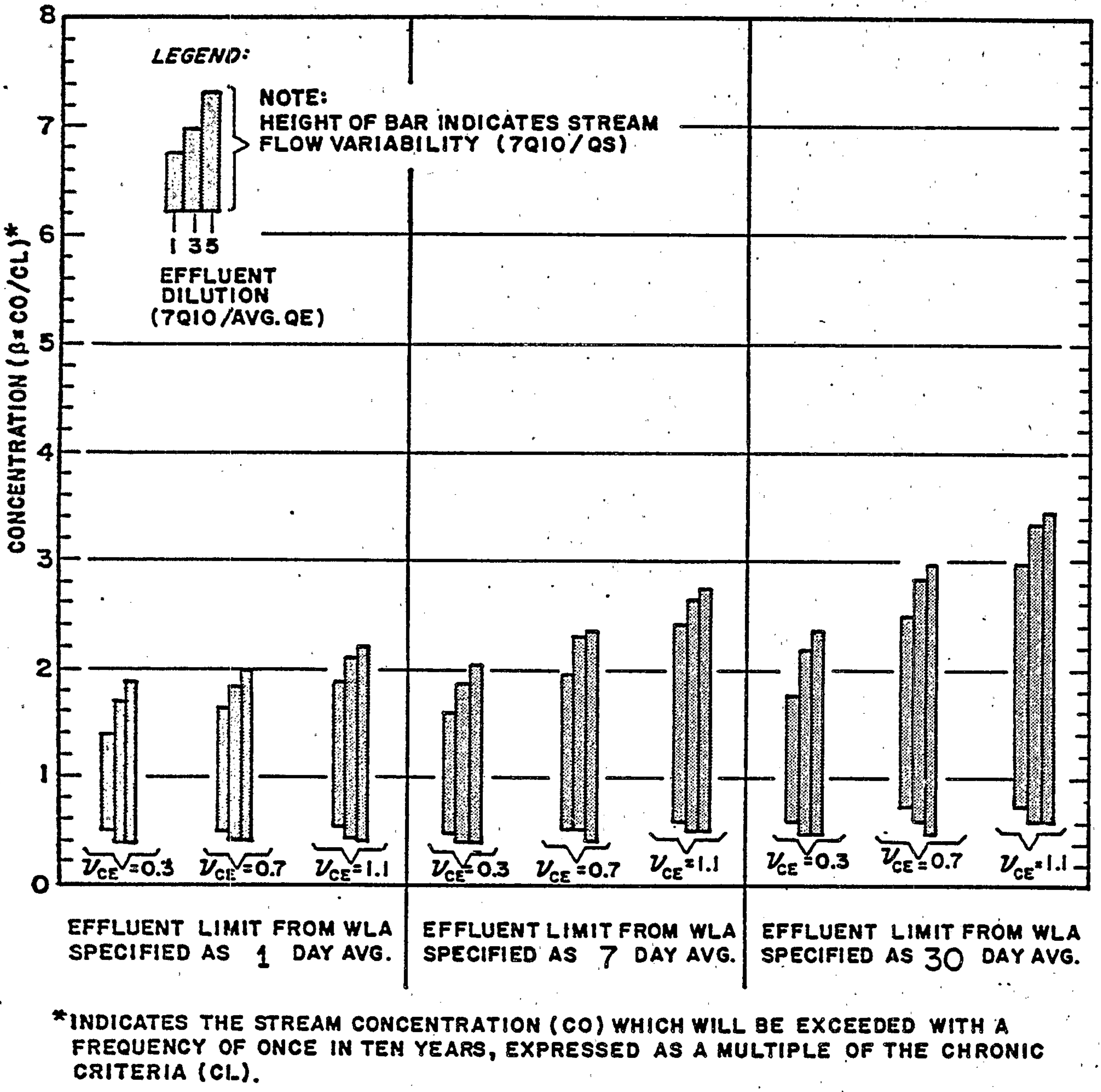


Figure 4-2 - Effect of permit averaging period on stream concentrations for BOD/DO.

TABLE 4-6 - Permit averaging period selection matrix for BOD/DO: effluent dilution ratio - $7Q10/\overline{QE} = 5$.

Stream Flow Characteristics				Effluent $\nu_{CE} = 0.3$			Effluent $\nu_{CE} = 0.7$			Effluent $\nu_{CE} = 1.1$		
All Periods		Low Flow Periods										
$7Q10/\overline{QS}$	ν_{QS}	$7Q10/\overline{QS'}$	$\nu_{QS'}$	30-Day Avg.	7-Day Avg.	1-Day Avg.	30-Day Avg.	7-Day Avg.	1-Day Avg.	30-Day Avg.	7-Day Avg.	1-Day Avg.
0.01	2.00	0.13	0.43	0.5	0.4	0.4	0.5	0.4	0.4	0.6	0.5	0.4
	3.00	0.26	0.50	1.0	0.9	0.8	1.1	0.9	0.7	1.2	1.0	0.8
	4.00	0.41	0.55	1.6	1.4	1.3	1.8	1.4	1.2	1.9	1.5	1.2
0.05	1.00	0.23	0.31	0.6	0.6	0.5	0.8	0.6	0.5	0.9	0.7	0.6
	1.50	0.42	0.38	1.2	1.1	1.0	1.4	1.2	1.0	1.6	1.3	1.0
	2.00	0.66	0.43	1.9	1.7	1.5	2.2	1.8	1.5	2.5	1.9	1.6
0.10	0.75	0.33	0.25	0.8	0.7	0.6	1.0	0.8	0.7	1.2	0.9	0.7
	1.00	0.46	0.31	1.2	1.0	0.9	1.4	1.2	1.0	1.7	1.3	1.0
	1.50	0.83	0.38	2.0	1.8	1.6	2.5	2.0	1.7	2.9	2.2	1.8
0.15	0.60	0.39	0.22	0.9	0.8	0.7	1.1	0.9	0.8	1.3	1.0	0.8
	0.90	0.61	0.29	1.4	1.3	1.1	1.8	1.4	1.2	2.1	1.6	1.3
	1.25	0.95	0.35	2.2	1.9	1.7	2.7	2.2	1.8	3.1	2.4	2.0
0.25	0.50	0.55	0.19	1.2	1.0	0.9	1.5	1.2	1.0	1.8	1.4	1.1
	0.75	0.82	0.25	1.7	1.5	1.3	2.2	1.8	1.5	2.6	2.0	1.6
	1.0	1.16	0.31	2.4	2.1	1.9	3.0	2.4	2.0	3.5	2.7	2.2

Critical DO deficit exceeded one day in 10 years as a multiple target deficit used in WLA.

TABLE 4-7 - Permit averaging period selection matrix for BOD/DO: effluent dilution ratio - $7Q10/\overline{QE} = 3$.

Stream Flow Characteristics				Effluent $\nu_{CE} = 0.3$			Effluent $\nu_{CE} = 0.7$			Effluent $\nu_{CE} = 1.1$		
All Periods		Low Flow Periods										
$7Q10/\overline{QS}$	ν_{QS}	$7Q10/\overline{QS}'$	ν_{QS}'	30-Day Avg.	7-Day Avg.	1-Day Avg.	30-Day Avg.	7-Day Avg.	1-Day Avg.	30-Day Avg.	7-Day Avg.	1-Day Avg.
	2.00	0.13	0.43	0.5	0.4	0.4	0.6	0.5	0.4	0.6	0.5	0.4
0.01	3.00	0.26	0.50	1.0	0.9	0.8	1.2	0.9	0.8	1.3	1.0	0.8
	4.00	0.41	0.55	1.5	1.3	1.2	1.7	1.4	1.2	1.9	1.5	1.2
	1.00	0.23	0.31	0.7	0.6	0.5	0.8	0.7	0.5	0.9	0.7	0.6
0.05	1.50	0.42	0.38	1.2	1.1	1.0	1.5	1.2	1.0	1.7	1.3	1.1
	2.00	0.66	0.43	1.8	1.5	1.4	2.2	1.7	1.4	2.5	1.9	1.6
	0.75	0.33	0.25	0.8	0.7	0.7	1.0	0.8	0.7	1.2	1.0	0.8
0.10	1.00	0.46	0.31	1.2	1.0	0.9	1.5	1.2	1.0	1.7	1.3	1.1
	1.50	0.83	0.38	1.9	1.7	1.5	2.4	1.9	1.6	2.8	2.2	1.8
	0.60	0.39	0.22	0.9	0.8	0.7	1.2	0.9	0.8	1.4	1.1	0.9
0.15	0.90	0.61	0.29	1.4	1.2	1.1	1.8	1.4	1.2	2.1	1.6	1.3
	1.25	0.95	0.35	2.0	1.8	1.6	2.6	2.1	1.7	3.0	2.3	1.9
	0.50	0.55	0.19	1.2	1.0	0.9	1.5	1.2	1.0	1.8	1.4	1.2
0.25	0.75	0.82	0.25	1.7	1.5	1.3	2.2	1.7	1.4	2.6	2.0	1.6
	1.0	1.16	0.31	2.2	1.9	1.7	2.9	2.3	1.9	3.4	2.6	2.1

Critical DO deficit exceeded one day in 10 years as a multiple target deficit used in WLA.

TABLE 4-8 - Permit averaging period selection matrix for BOD/DO: effluent dilution ratio - $7Q10/\overline{QE} = 1$.

Stream Flow Characteristics				Effluent $\nu_{CE} = 0.3$			Effluent $\nu_{CE} = 0.7$			Effluent $\nu_{CE} = 1.1$		
All Periods		Low Flow Periods										
$7Q10/\overline{QS}$	ν_{QS}	$7Q10/\overline{QS}'$	ν_{QS}'	30-Day Avg.	7-Day Avg.	1-Day Avg.	30-Day Avg.	7-Day Avg.	1-Day Avg.	30-Day Avg.	7-Day Avg.	1-Day Avg.
	2.00	0.13	0.43	0.6	0.5	0.5	0.7	0.6	0.5	0.8	0.6	0.5
0.01	3.00	0.26	0.50	1.0	0.9	0.8	1.3	1.0	0.8	1.5	1.1	0.9
	4.00	0.41	0.55	1.4	1.2	1.1	1.7	1.4	1.1	2.0	1.5	1.3
	1.00	0.23	0.31	0.8	0.7	0.6	1.0	0.8	0.7	1.2	0.9	0.7
0.05	1.50	0.42	0.38	1.2	1.1	1.0	1.6	1.3	1.0	1.8	1.4	1.2
	2.00	0.66	0.43	1.5	1.4	1.2	2.1	1.6	1.4	2.4	1.9	1.5
	0.75	0.33	0.25	0.9	0.8	0.7	1.2	1.0	0.8	1.5	1.1	0.9
0.10	1.00	0.46	0.31	1.2	1.1	0.9	1.6	1.3	1.0	1.9	1.5	1.2
	1.50	0.83	0.38	1.6	1.5	1.3	2.2	1.8	1.5	2.7	2.1	1.7
	0.60	0.39	0.22	1.0	0.9	0.8	1.4	1.1	0.9	1.6	1.3	1.0
0.15	0.90	0.61	0.29	1.4	1.2	1.1	1.8	1.5	1.2	2.2	1.7	1.4
	1.25	0.95	0.35	1.7	1.5	1.3	2.3	1.9	1.5	2.8	2.2	1.8
	0.50	0.55	0.19	1.2	1.1	1.0	1.7	1.3	1.1	2.0	1.6	1.3
0.25	0.75	0.82	0.25	1.5	1.4	1.2	2.1	1.7	1.4	2.6	2.0	1.6
	1.0	1.16	0.31	1.8	1.6	1.4	2.5	2.0	1.7	3.0	2.4	1.9

Critical DO deficit exceeded one day in 10 years as a multiple target deficit used in WLA.

A table for the effluent dilution ratio ($7Q10/\overline{QE}$) equal to 50 has not been prepared for BOD/DO. For small discharges entering larger streams, it is likely that an effluent BOD limit determined from a steady-state WLA analysis would be greater than the technology-based limit which would be used in the permit. The use of the standard matrix table, which would show a higher pattern of violations, would tend to be misleading, since the computations and the tables assume that the allowable effluent concentration determined from a WLA becomes the effluent limit (EL) specified by the permit.

It should be emphasized at this point that the dissolved oxygen analysis presented in this section is meant only as a preliminary application. There are, as yet, no verification examples that support the applicability of a probabilistic dilution/critical deficit analysis. It has not been shown that actual stream DO data conform to the probabilistic assumptions and simplifications used in this preliminary analysis. Further, it is well known that the DO distribution in streams cannot always be described by the simplest (Streeter-Phelps) model. Upstream sources of BOD and deficit are common, as are nitrification, algal effects, and sediment oxygen demand. A more comprehensive analysis would be required to incorporate these effects into a calculation of the effect of selecting a permit averaging period.

4.4 Analysis for Conservative Substances in Effluent-Dominated Streams

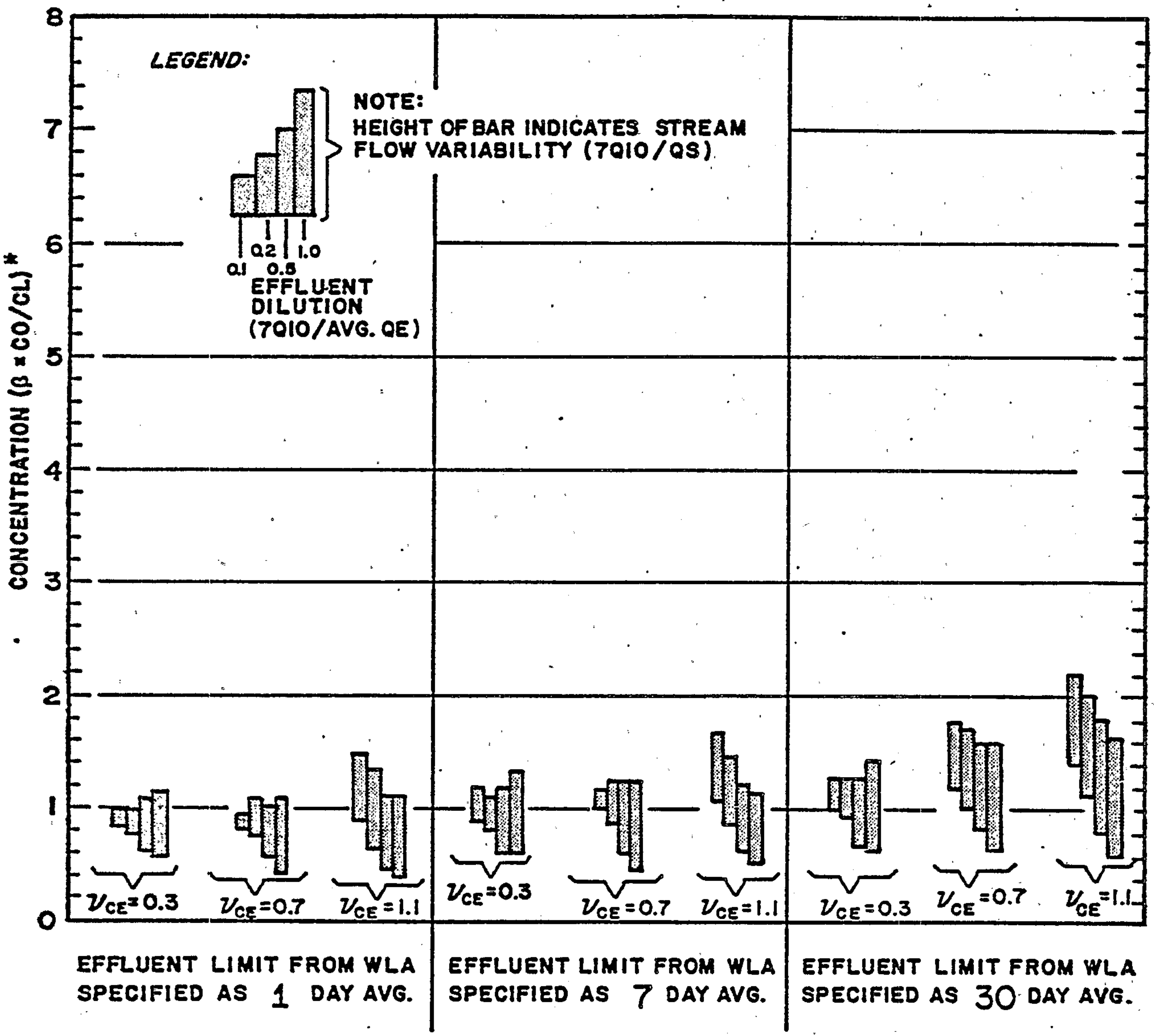
An effluent-dominated stream is defined, for the purpose of this analysis, as one in which the effluent flow exceeds the design stream flow (e.g., the 7Q10). There are then two bounds to this analysis. The upper bound is the effluent dilution ratio 7Q10/avg QE = 1, which was the lowest dilution ratio examined in Section 4.1. The lower bound is provided by the case where the design stream flow is zero (7Q10 = 0).

It should be recognized that as the degree of dilution decreases, a WLA-based EL becomes increasingly restrictive. When the design stream flow is zero, the effluent limit must equal the stream target concentration (CL).

While the degree of effluent domination has a subsequent influence on the magnitude of an EL assigned in a permit, the screening analysis results presented below suggest that in most situations, a 30-day permit averaging period will be adequate for effluent dominated streams.

The results of a broad hypothetical analysis of effluent dominated streams are summarized in Figure 4-3 and Table 4-9, using the format used earlier to illustrate the influence of permit averaging period, effluent variability and dilution ratio.

- o The bars on the right provide the upper bound; i.e., the condition where 7Q10/avg QE = 1 (these results were also shown in Figure 4-1).
- o The bars on the left represent an effluent dilution ratio of 7Q10/avg QE = 0.1, that is, where effluent flow is ten times greater than design stream flow. High variability of daily flow is expected for such streams, together with a very small ratio of stream design flow to average stream flow. The screening analysis assumes that the coefficient of variation ranges between ν_{QS} =



* INDICATES THE STREAM CONCENTRATION (CO) WHICH WILL BE EXCEEDED WITH A FREQUENCY OF ONCE IN TEN YEARS, EXPRESSED AS A MULTIPLE OF THE CHRONIC CRITERIA (CL).

Figure 4-3 - Effect of permit averaging period on stream concentrations for conservative substances in effluent-dominated streams.

TABLE 4-9 - Averaging period selection matrix for effluent-dominated streams.

Effluent Dilution Ratio $7Q10/\overline{QE}$	Estimate of Variability Range νQS		Effluent ν_{CE} = 0.3			Effluent ν_{CE} = 0.7			Effluent ν_{CE} = 1.1		
			30-Day Avg.	7-Day Avg.	1-Daily Max.	30-Day Avg.	7-Day Avg.	1-Daily Max.	30-Day Avg.	7-Day Avg.	1-Daily Max.
1.0	LO	2.00	0.6	0.6	0.5	0.6	0.5	0.4	0.6	0.5	0.4
	PROB	4.00	1.1	1.0	0.9	1.2	0.9	0.8	1.2	0.9	0.8
	HI	5.00	1.5	1.3	1.2	1.6	1.3	1.1	1.7	1.3	1.1
0.5	LO	2.00	0.7	0.6	0.6	0.8	0.6	0.5	0.8	0.6	0.5
	PROB	4.00	1.1	1.0	0.9	1.2	1.0	0.8	1.3	1.0	0.9
	HI	5.00	1.3	1.2	1.1	1.6	1.3	1.0	1.8	1.4	1.1
0.2	LO	2.00	0.9	0.8	0.7	1.0	0.8	0.7	1.1	0.9	0.7
	PROB	4.00	1.1	1.0	0.9	1.4	1.1	0.9	1.6	1.3	1.0
	HI	5.00	1.3	1.1	1.0	1.7	1.3	1.1	2.0	1.5	1.3
0.1	LO	2.00	1.0	0.9	0.8	1.2	1.0	0.8	1.4	1.1	0.9
	PROB	4.00	1.2	1.1	0.9	1.6	1.3	1.1	1.9	1.5	1.2
	HI	5.00	1.3	1.2	1.0	1.8	1.5	1.2	2.2	1.7	1.4

Applies for conservative pollutants. Assumes the ratio 7Q10/QS = 0.005 for all cases.

2 and v_{QS} = 5, and estimates a stream flow ratio 7Q10/avg QS = 0.005, for this condition near the lower bound for effluent-dominated streams.

The conditions under which the design stream flow is greater than zero are listed in more detail in Table 4-9. Results for several additional intermediate effluent dilution ratios ($7Q10/\overline{QE}$ = 0.2 and 0.5) are also presented. A comparison of results for an effluent ratio of 1.0 presented here as an upper bound, and previously (Table 4-4 and Figure 4-1) as a lower bound will indicate that results are similar but not exactly the same. The differences are due to different assumed values for 7Q10/QS and the range of coefficients of variations used as inputs for the PDM-PS model.

For the case where the design stream flow is zero, 7Q10 is zero and there appears to be a problem since $7Q10/\overline{QS}$ and $7Q10/\overline{QE}$ are both zero. However, what actually matters is $\overline{QS}$ and $\overline{QE}$. Thus, in order to evaluate these cases, the use of the actual $\overline{QS}$, $\overline{QE}$ and a small 7Q10 suffices since the computation depends only on $\overline{QS}/\overline{QE}$ and 7Q10 cancels out (Equation D-14). Finally, the use of a small $7Q10/\overline{QE}$ correctly indicates that the WLA is done with QS = 0 (Equation D-15). Thus, no problems arise.

Screening analysis results indicate that in the case of effluent-dominated streams, a 30-day permit averaging period provides adequate protection for pollutants with the acute-to-chronic ratios summarized below:

Acute-to-Chronic Ratio	When 30-Day Permit Average Is Adequate for Acute Protection
3 or more	Always
2 to 3	Effluent variability is relatively high, but less than $\nu_{CE} = 1.1$

APPENDIX A

Statistical Properties of Log-Normal Distributions

CHAPTER 5

USES AND LIMITATIONS

The probabilistic dilution model has been demonstrated to be useful in selecting the appropriate averaging period for discharge permits. The method is easily adaptable to situations which vary widely in terms of stream and effluent characteristics, data availability, and policy-level assumptions used in the analysis. Although the example in Chapter 3 of how to use the method is based on the typical WLA assumptions of 7Q10 as the design flow and chronic criteria as the effluent limit, the method is easily adjusted to accommodate other assumptions.

The method is intended to apply to pollutants for which the regulatory concern is at the point of complete mixing and for which the toxicity can be evaluated in terms of the total pollutant concentration. The method has been applied to a range of stream and effluent characteristics which typify the characteristics of streams and effluents in the United States. The results of this application are useful as a screening tool, by which the appropriate averaging period for many field situations can be readily identified. However, pollutants whose toxicity is a function of pH, temperature, and hardness require site-specific evaluations incorporating these parameters.

There are also several limitations on the use of the method. One of the technical limitations is that the level of chronic protection is based on state-specified design flow, e.g., 7Q10, 7Q2, etc., which may be overprotective or underprotective for many site-specific conditions. The EPA is presently considering the issue of allowable duration and frequency of exposure to acute as well as chronic toxicity. Users of this manual are advised to refer to Part A, Stream Design Flow, of Book VI, Selecting Design Conditions, when considering the choice of an appropriate chronic exposure event. Book VI is currently under peer review and will be issued by the Office of Water Regulations and Standards once the peer review process is completed.

Modifications are required to compute the probability distribution of 30-day average concentrations, as required for chronic criteria compliance; these would have to be investigated and verified in the field.

The major shortcoming of the log-normal probabilistic dilution model is its misrepresentation of the lowest stream flows, thus tending to overestimate the probability of high stream concentrations. The use of a seasonally segmented approach could be investigated.

The effect of serial correlation on the return period specification would also need to be investigated, particularly with regard to the duration of criteria violations. For example, a knowledge of the return period for n-day successive violations could be compared to the time scales of the criteria themselves. This would provide a direct link to the toxicity data. At a less sophisticated level of analysis, the tendency of criteria violations to cluster on successive days could be investigated to provide a basis for modifications to the method.

For pollutants whose toxicity is a function of such secondary variables as pH, temperature and hardness, probabilistic methods are essential in that it is not possible to rationally choose "critical" or "sufficiently protective" values for these variables. Arbitrary choices cannot be defended in terms of the probability of criteria violations. Methods for analyzing these situations could be developed, following the logic of probabilistic dilution and incorporating the additional random variations of the variable.

The application of this method to dissolved oxygen has indicated that the probabilistic method provides a useful approach to the problem of DO deficit. However this work has only been a first step. Probabilistic methods can be further developed to assess the effects of DO fluctuations on fishery resources and to provide a more rational approach to advanced waste treatment decisions.

CHAPTER 6

REFERENCES

1. DiToro, D. M., Probability Model of Stream Quality Due to Runoff, J. Environmental Engineering, American Society of Chemical Engineers, Vol. 110, No. 3, June 1984, p. 607-628.

2. DiToro, D. M. and Fitzpatrick, J. J., Verification Analysis of the Probabilistic Dilution Model, report prepared for EPA Contract No. 68-01-6275, U. S. Environmental Protection Agency, Washington, D.C., 1982.

3. Driscoll & Associates, Combined Sewer Overflow Analysis Handbook for Use in 201 Facility Planning, report prepared for EPA Contract No. 68-01-6148, U. S. Environmental Protection Agency, Washington, D.C. (1981).

4. Hazen and Sawyer, Review of Performance of Secondary Municipal Treatment Works, Draft Final Report for Contract 68-01-6275, Work Assignment No. 5, U. S. Environmental Protection Agency, Washington, D.C., December 1982.

5. Niku, Shroeder, and Samaniego, Performance of Activated Sludge Process and Reliability Related Design, JWPCF, Vol. 51, No. 12, December 1979.

6. Niku, et al., Performance of Activated Sludge Processes: Reliability, Stability and Variability, EPA 600/52-81-227, December 1981.

7. Haugh, et al., Performance of Trickling Filter Plants: Reliability Stability and Variability, EPA 600/52-81-228, December 1981.

8. Hydroscience, Inc., Simplified Mathematical Modeling of Water Quality, for U. S. Environmental Protection Agency, March 1971.

This appendix is intended to present a brief, simplified review of the statistical properties of log-normal distributions which characterize the important variables in the water quality analysis procedures used for this report. It is designed to help the user without a formal background in statistics to appreciate the physical significance of the statistical properties employed. It is not the intent of this appendix to present a theoretical discussion or to provide technical support for developing relationships or equations used in the development of the methods employed.

A-1. General Considerations

The factors which influence the concentration of a pollutant in a receiving water body are subject to a significant degree of variability. This variability results in fluctuations in the resulting stream concentration, which is compared with target concentrations such as criteria or standards, and which provides a basis for decisions on treatment requirements. The approach adopted in this report for examining the effects of different averaging periods on treatment plant discharges uses the concept "how much -- how often" as a basis for such decisions. It is, therefore, essential that statistical aspects be incorporated into the methodology even though they may add complexity.

The standard statistical parameters of a population of values for a random variable which are used as a concise means of describing central tendency and spread are:

Mean: (μ_x or $\overline{x}$) the arithmetic average. $\overline{x}$ defines the average of the available (usually limited) data set;

μ_X denotes the true mean of the total population of variable x. $\overline{x}$ will be an increasingly better approximation of μ_X as the size of the sample (the number of data points) increases.

Variance: (σ^2_X) by definition, the average of the square of the differences between individual values of x and the mean ($\overline{x}$). The greater the variation in the data, the higher the variance:

$$\sigma^2_X = \frac{(x_1-\overline{x})^2 + (x_2-\overline{x})^2 + \ldots\ldots (X_N-\overline{x})^2}{N}$$

Standard Deviation: (σ_X) another measure of the spread of a population of random variables; by definition, the square root of the variance:

$$\sigma_X = \sqrt{\sigma^2_X}$$

Coefficient of Variation: (ν_X) is defined as the ratio of the standard deviation (σ_X) to the mean (μ_X):

$$\nu_X = \sigma_X / \mu_X$$

It is the principal measure of variation used in the analyses described in this report. The coefficient of variation is a dimensionless quantity and is thus freed from any dependence on the specific dimensions used to describe the variable (e.g., flow rate, concentrations, etc.). High coefficients of variation reflect greater variability in the random variable x.

Median: ($\tilde{x}$) This is the value in a data set for which half the values are greater and half are lesser.

Mode: The "most probable value" -- more of the individual data points are at this value (or are within this interval) than at other values or ranges. On a frequency histogram, this is the highest point on the graph. The mode has no real significance in the calculations in the methodology employed.

Comparing the statistical properties of different data sets provides a convenient, concise way of recognizing similarities and differences. This could not be accomplished simply by "looking at the data" where reasonably large data sets are involved. These statistical properties convey no information concerning frequency, or the probability at which any particular value or range of values in the total population will occur. This essential item of information is provided by a knowledge of the type of distribution, technically, the probability distribution function (PDF).

A-2. Probability Distributions

There are several different patterns which characterize the distribution of individual values in a large population of variable events.

Most analysts are familiar with the normal distribution, in which a histogram of the frequency of occurrence of various values describes the familiar bell-shaped curve (Figure A-1(a)). When the cumulative frequency[1] is plotted on probability paper, a straight line is generated as in Figure A-1(b).

Many variables, particularly those which are important in water quality applications, have been shown by a rapidly accumulating body of data to be represented by or adequately approximated by a log-normal distribution. A log-normal distribution has a skewed frequency histogram (Figure A-1(c)) which indicates an asymmetrical distribution of values about an axis defining the central tendency of the data set. There is a constraining limit to lower values (sometimes zero) and a relatively small number of rather large values but no upper constraint. Point source effluent concentrations [1,2], and pollutant concentrations in combined sewer overflows and separate storm runoff [3,4], are parameters which are usually well characterized by log-normal distributions. In general, daily stream flows are satisfactorily approximated by log-normal distributions [5,6]. Scattered data from a number of unpublished sources suggest that receiving water concentrations are also log-normally distributed. Stream flows and concentrations are currently being examined from this perspective. A log-normal distribution appears as a straight line on log/probability paper (using cumulative frequency) as shown in Figure A-1(d). In this report natural (base "e") logs are used throughout.

[1]Cumulative frequency is the relative frequency (or probability) of values being less than or equal to a specific value.

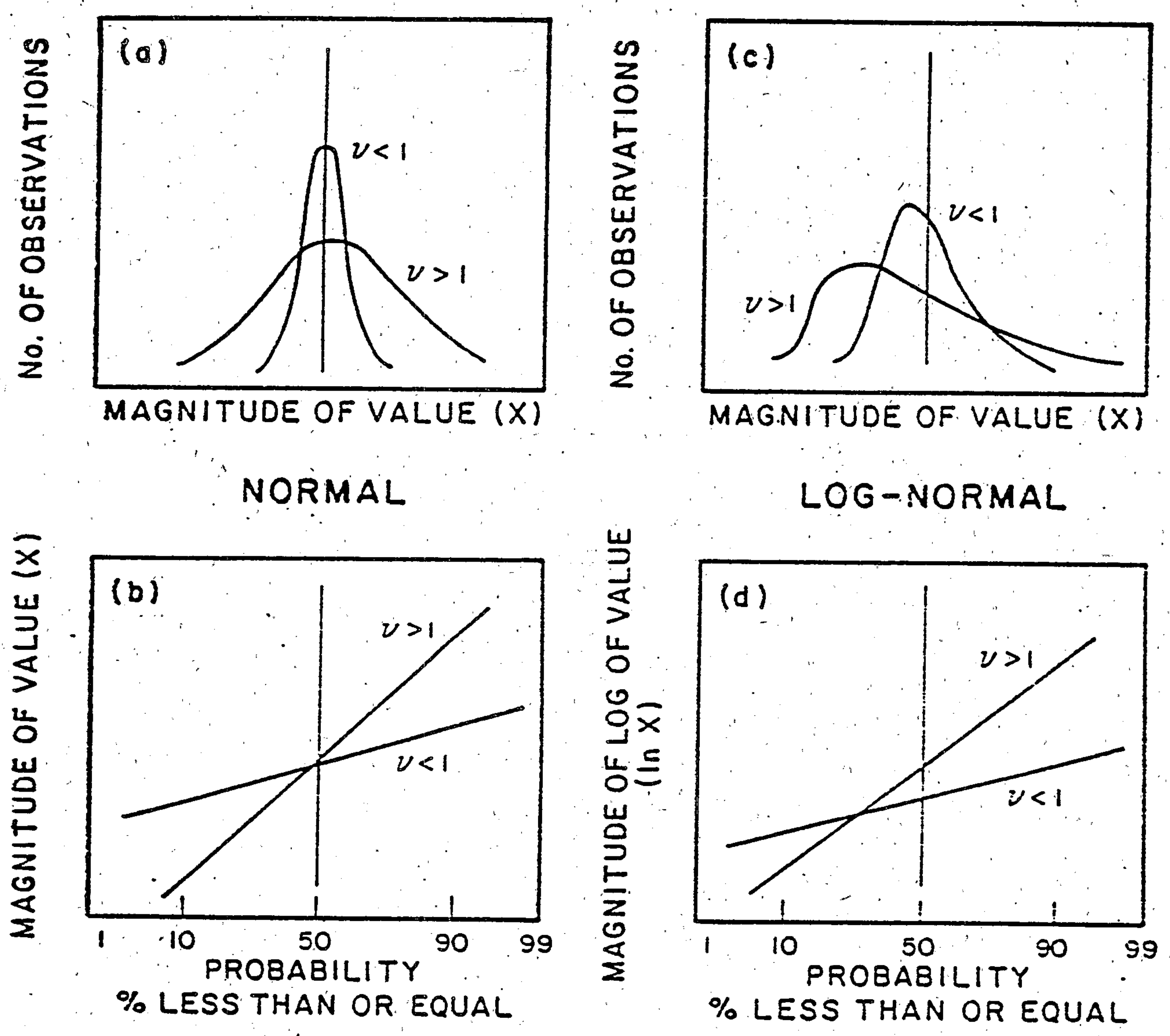


Figure A-1 - Probability distribution.

A-3. Relationship Between Distributions

There are circumstances when two different types of distribution can begin to look similar -- so that either one will provide a reasonably good approximation of the probability distribution of a particular data set. For example, as the coefficient of variation becomes smaller and smaller, approaching zero, log-normal distributions begin to look more and more like a normal distribution. Figure A-2 shows a series of histograms for log-normally distributed populations, all having (arithmetic) population means of 100, but with different coefficients of variation (ν) as shown. As discussed above, smaller values of ν approach a normal distribution.

A-4. Properties of Log-Normal Distributions

Figure A-3 summarizes the pertinent statistical relationships for log-normal probability distributions. The mathematical formulas shown are based on statistical theory, and permit back-and-forth conversions between arithetic properties (in which concentrations, flows, and loads are reported) and the log of the variable (in which probability and frequency characteristics are defined).

Normalized plots of probability versus the magnitude of a variable expressed as a multiple of the mean are presented in Figure A-4 for log-normal distributions. These plots present a family of curves reflecting the effect of coefficient of variation on probability of occurrence of events of specific magnitude. These plots can be used directly in the

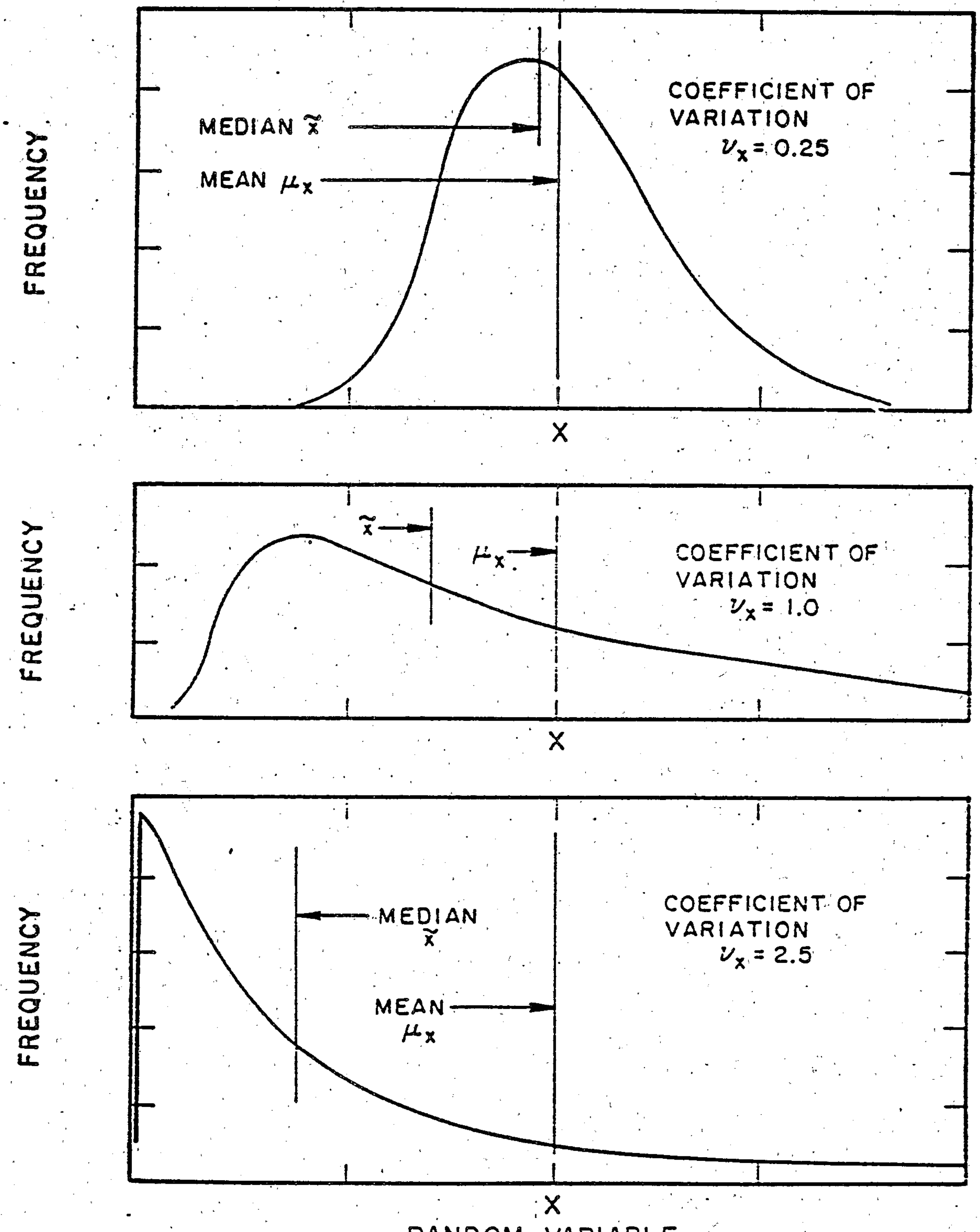
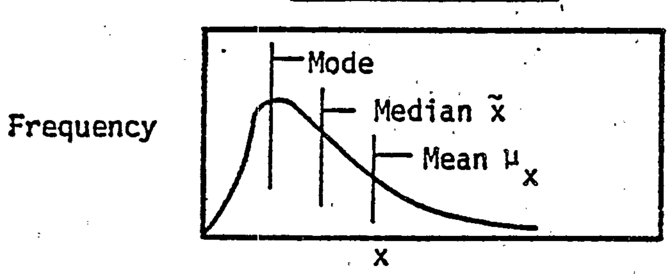


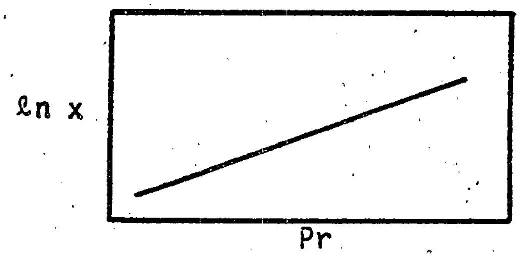
Figure A-2 - Effect of coefficient of variation on frequency distribution.

Arithmetic Space



Log Space

Natural Logs (base e)



x is a random variable

Definition of Terms

x	.... Random Variable	$\ell n\ x$
μ_x	.... Mean	$\mu_{\ell n\ x}$
σ_x^2	.... Variance	$\sigma^2_{\ell n\ x}$
σ_x	.... Standard Deviation	$\sigma_{\ell n\ x}$
ν_x	.... Coefficient of Variation ...	(not used)
$\tilde{x}$	.... Median	

Relationships Between Statistical Properties In Arithmetic and Log Space

$$\mu_x = \exp\left[\mu_{\ln x} + \frac{1}{2}\ \sigma^2_{\ln x}\right] \qquad \mu_{\ln x} = \ln\left(\frac{\mu_x}{\sqrt{1+\nu_x{}^2}}\right)$$

$$\tilde{x} = \exp\left[\mu_{\ln x}\right]$$

$$\nu_x = \sqrt{\exp\left(\sigma^2_{\ln x}\right) - 1} \qquad \sigma_{\ln x} = \sqrt{\ln\left(1+\nu_x{}^2\right)}$$

$$\sigma_x = \mu_x \nu_x$$

Figure A-3 - Pertinent relationships for log-normal distribution.

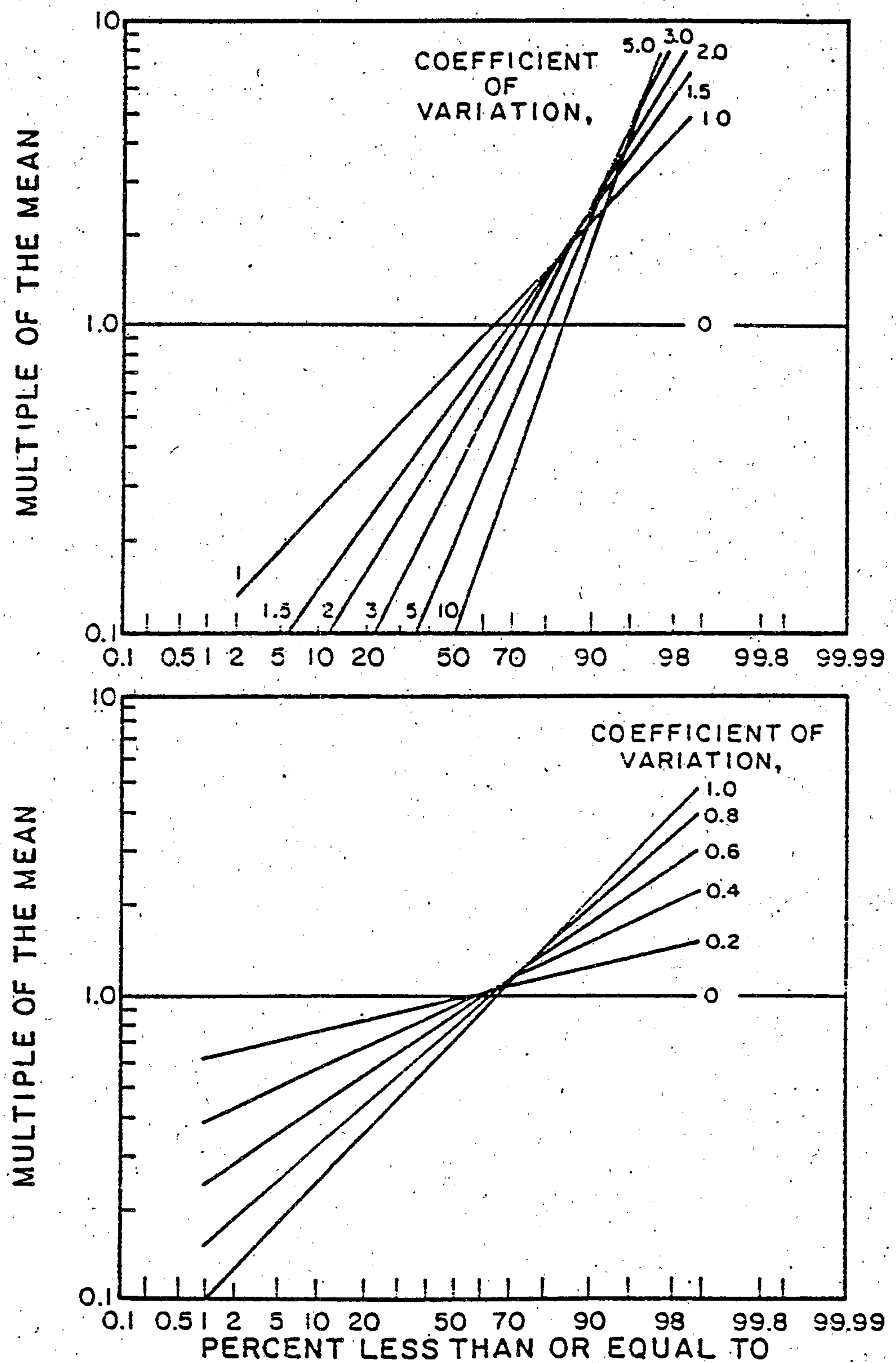


Figure A-4 - Cumulative log-normal distribution.

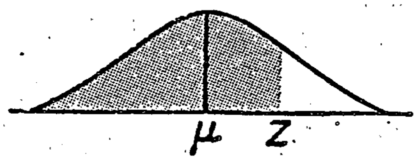
analysis methodology and permit direct determination of frequency for events of any specified magnitude with a known or estimated coefficient of variation.

A-5. Standard Normal Tables

For normal (or log-normal) distributions, probabilities can be defined in terms of the magnitude of a value, normalized by the standard deviation. This technique is used in the calculations of the probability of exceeding specified receiving water concentrations in this analysis. Standard normal tables can be obtained from any statistics textbook [8,9]. Table A-1 presents the standard normal table to provide a convenient source for the analyses used in this report. Table A-1 lists the probability for the interval between 0 and the value of Z listed. Thus, it represents the probability that a value will be less than or equal to the selected value of Z.

TABLE A-1 - Probabilities for the standard normal distribution.

Each entry in the table indicates the proportion of the total area under the normal curve to the left of a perpendicular raised at a distance of Z standard deviation units.



Example: 88.69 percent of the area under a normal curve lies to the left of a point 1.21 standard deviation units to the right of the mean.

Z	0.00	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09
0.0	0.5000	0.5040	0.5080	0.5120	0.5160	0.5199	0.5239	0.5279	0.5319	0.5359
0.1	0.5398	0.5438	0.5478	0.5517	0.5557	0.5596	0.5636	0.5675	0.5714	0.5753
0.2	0.5793	0.5832	0.5871	0.5910	0.5948	0.5987	0.6026	0.6064	0.6103	0.6141
0.3	0.6179	0.6217	0.6255	0.6293	0.6331	0.6368	0.6406	0.6443	0.6480	0.6517
0.4	0.6554	0.6591	0.6628	0.6664	0.6700	0.6736	0.6772	0.6808	0.6844	0.6879
0.5	0.6915	0.6950	0.6985	0.7019	0.7054	0.7088	0.7123	0.7157	0.7190	0.7224
0.6	0.7257	0.7291	0.7324	0.7357	0.7389	0.7422	0.7454	0.7486	0.7518	0.7549
0.7	0.7580	0.7612	0.7642	0.7673	0.7704	0.7734	0.7764	0.7794	0.7823	0.7852
0.8	0.7881	0.7910	0.7939	0.7967	0.7995	0.8023	0.8051	0.8078	0.8106	0.8133
0.9	0.8159	0.8186	0.8212	0.8238	0.8264	0.8289	0.8315	0.8340	0.8365	0.8389
1.0	0.8413	0.8438	0.8461	0.8485	0.8508	0.8531	0.8554	0.8577	0.8599	0.8621
1.1	0.8643	0.8665	0.8686	0.8708	0.8729	0.8749	0.8770	0.8790	0.8810	0.8830
1.2	0.8849	0.8869	0.8888	0.8907	0.8925	0.8944	0.8962	0.8980	0.8997	0.9015
1.3	0.9032	0.9049	0.9066	0.9082	0.9099	0.9115	0.9131	0.9147	0.9162	0.9177
1.4	0.9192	0.9207	0.9222	0.9236	0.9251	0.9265	0.9279	0.9292	0.9306	0.9319
1.5	0.9332	0.9345	0.9357	0.9370	0.9382	0.9394	0.9406	0.9418	0.9429	0.9441
1.6	0.9452	0.9463	0.9474	0.9484	0.9495	0.9505	0.9515	0.9525	0.9535	0.9545
1.7	0.9554	0.9564	0.9573	0.9582	0.9591	0.9599	0.9608	0.9616	0.9625	0.9633
1.8	0.9641	0.9649	0.9656	0.9664	0.9671	0.9678	0.9686	0.9693	0.9699	0.9706
1.9	0.9713	0.9719	0.9726	0.9732	0.9738	0.9744	0.9750	0.9756	0.9761	0.9767
2.0	0.9772	0.9778	0.9783	0.9788	0.9793	0.9798	0.9803	0.9808	0.9812	0.9817
2.1	0.9821	0.9826	0.9830	0.9834	0.9838	0.9842	0.9846	0.9850	0.9854	0.9857
2.2	0.9861	0.9864	0.9868	0.9871	0.9875	0.9878	0.9881	0.9884	0.9887	0.9890
2.3	0.9893	0.9896	0.9898	0.9901	0.9904	0.9906	0.9909	0.9911	0.9913	0.9916
2.4	0.9918	0.9920	0.9922	0.9925	0.9927	0.9929	0.9931	0.9932	0.9934	0.9936
2.5	0.9938	0.9940	0.9941	0.9943	0.9945	0.9946	0.9948	0.9949	0.9951	0.9952
2.6	0.9953	0.9955	0.9956	0.9957	0.9949	0.9960	0.9961	0.9962	0.9963	0.9964
2.7	0.9965	0.9966	0.9967	0.9968	0.9969	0.9970	0.9971	0.9972	0.9973	0.9974
2.8	0.9974	0.9975	0.9976	0.9977	0.9977	0.9978	0.9979	0.9979	0.9980	0.9981
2.9	0.9981	0.9982	0.9982	0.9983	0.9984	0.9984	0.9985	0.9985	0.9986	0.9986
3.0	0.9986	0.9987	0.9987	0.9988	0.9988	0.9989	0.9989	0.9989	0.9990	0.9990
3.1	0.9990	0.9991	0.9991	0.9991	0.9992	0.9992	0.9992	0.9992	0.9993	0.9993
3.2	0.9993	0.9993	0.9994	0.9994	0.9994	0.9994	0.9994	0.9995	0.9995	0.9995
3.3	0.9995	0.9995	0.9995	0.9996	0.9996	0.9996	0.9996	0.9996	0.9996	0.9997
3.4	0.9997	0.9997	0.9997	0.9997	0.9997	0.9997	0.9997	0.9997	0.9998	0.9998
3.5	0.9998	0.9998	0.9998	0.9998	0.9998	0.9998	0.9998	0.9998	0.9998	0.9998
3.6	0.9998	0.9998	0.9999	0.9999	0.9999	0.9999	0.9999	0.9999	0.9999	0.9999
3.7	0.9999	0.9999	0.9999	0.9999	0.9999	0.9999	0.9999	0.9999	0.9999	0.9999
3.8	0.9999	0.9999	0.9999	0.9999	0.9999	0.9999	0.9999	1.0000	1.0000	1.0000
3.9	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000

A-6. References

1. Niku, et al.,"Performance of Activated Sludge Processes and Reliability Based Design." Journal WPCF, Vol. 51, No. 12, (December, 1979).

2. McCarty, et al., "Reliability of Advanced Wastewater Treatment."

3. EPA Water Planning Division, "Preliminary Results of the Nationwide Urban Runoff Program," (March 1982).

4. Mancini, J. L., "Methods for Developing Wet Weather Water Quality Criteria." Progress Report, June 1981; EPA ORD Grant No. R806828010, Cincinnati.

5. Chow, V.T. "Handbook of Applied Hydrology." Mc-Graw Hill, New York (1964).

6. Linsley, et al., "Hydrology for Engineers." Mc-Graw Hill, 2nd Edition, (1975).

7. Hydroscience, In., "A Statistical Method for the Assessment of Urban Stormwater." USEPA, EPA 440/3-79-023, (May 1979).

8. Benjamin, J. R. and C. A. Cornell, "Probability, Statistics and Decision for Civil Engineers." McGraw-Hill, New York, (1970).

9. Johnson, R. R., "Elementary Statistics." Duxbury Press, North Scituate, Massachusetts, (1980).

APPENDIX B

Field Validation of Log-Normal Distribution and Related Assumptions

This appendix presents a discussion of several technical issues and assumptions which are necessary to the use of the probabilistic dilution model to guide selection of permit averaging periods. This discussion is organized in two sections: the first provides a justification for the use of the probabilistic dilution model in the method; the second provides a discussion of several key assumptions.

B-1. Use of the Log-Normal Distribution

A relatively simple and straightforward analysis is made possible by the assumption that each of the input variables is log-normally distributed and independent. The appropriateness of these assumptions and their implications are discussed below.

A basic feature of any random time series of numerical values is its probability distribution function, which specifies the distribution of values and their frequency of occurrence. More detailed characterizations which account for seasonal trends and day-to-day correlations are also possible, but at minimum the univariate probability density function is required. An examination of flow data from a number of streams indicates that the data can be reasonably well represented by a log-normal distribution. Figure B-1 summarizes an examination of the adequacy of a log-normal distribution for daily flows of 60 streams with long periods of record. The actually observed 10th and 1st percentile low flows are compared with the flow estimated by a log-normal distribution. The major important discrepancy occurs at the lowest flows where the predicted distribution is lower than that actually observed. The most likely cause

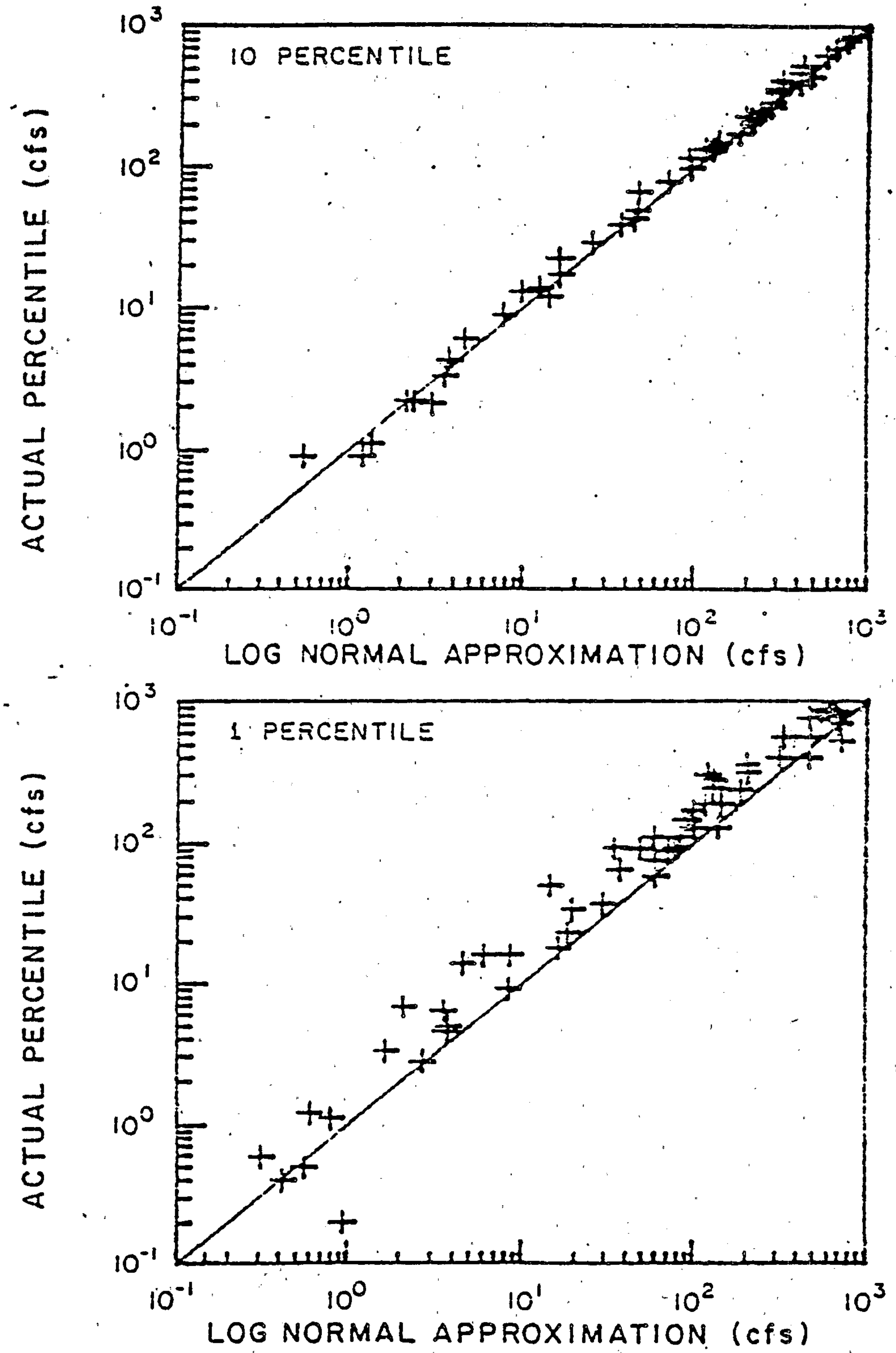


Figure B-1: Evaluations of log-normal distribution for stream flows.

is the presence of a base stream flow which does not vary appreciably. Thus, the log-normal representation is generally a lower bound characterization of this distribution of the very lowest flows, which will tend to provide upper bound estimates of stream concentrations if these misrepresented low flows are important. For the analysis results in this report, therefore, the calculations may be overprotective in some cases.

Log probability plots of treatment plant effluent flows and concentrations are illustrated in Figure B-2 for conventional pollutants and Figure B-3 for heavy metals. Essentially, all data examined to date indicate that a log-normal characterization is representative.

B-2. Verification of the Probabilistic Dilution Model

The probabilistic dilution model itself has been subjected to a number of tests in order to check its validity and realism. Detailed simulation studies using Monte Carlo methods [1] have verified the calculated downstream concentration probability distribution when the upstream and effluent flows and concentrations are exactly log-normal.

In addition, detailed analysis of actual discharges into streams, (11 data sets for 5 streams) has been performed [2]. Observed data were available for upstream and effluent flows and concentrations, as well as for downstream concentrations. The log-normal probability dilution model was used to predict the probability distribution of downstream concentrations. Table B-1 compares the observed and computed median and 95th percentiles values for selected water quality parameters. The 95% confidence limits of these observed quantities, computed from the known sampling

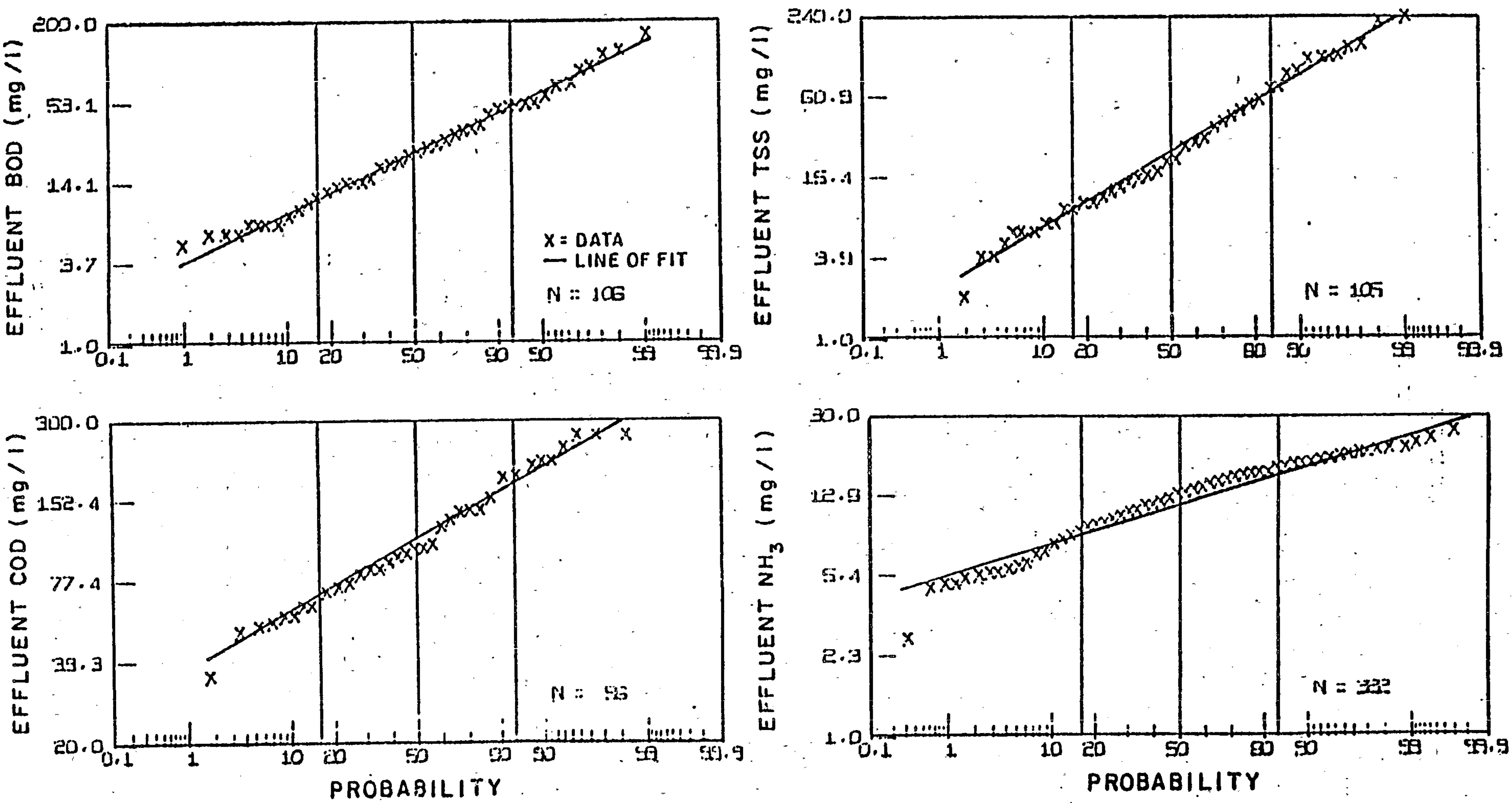


Figure B-2 - Probability distribution of treatment plant effluent concentrations - conventional pollutants.

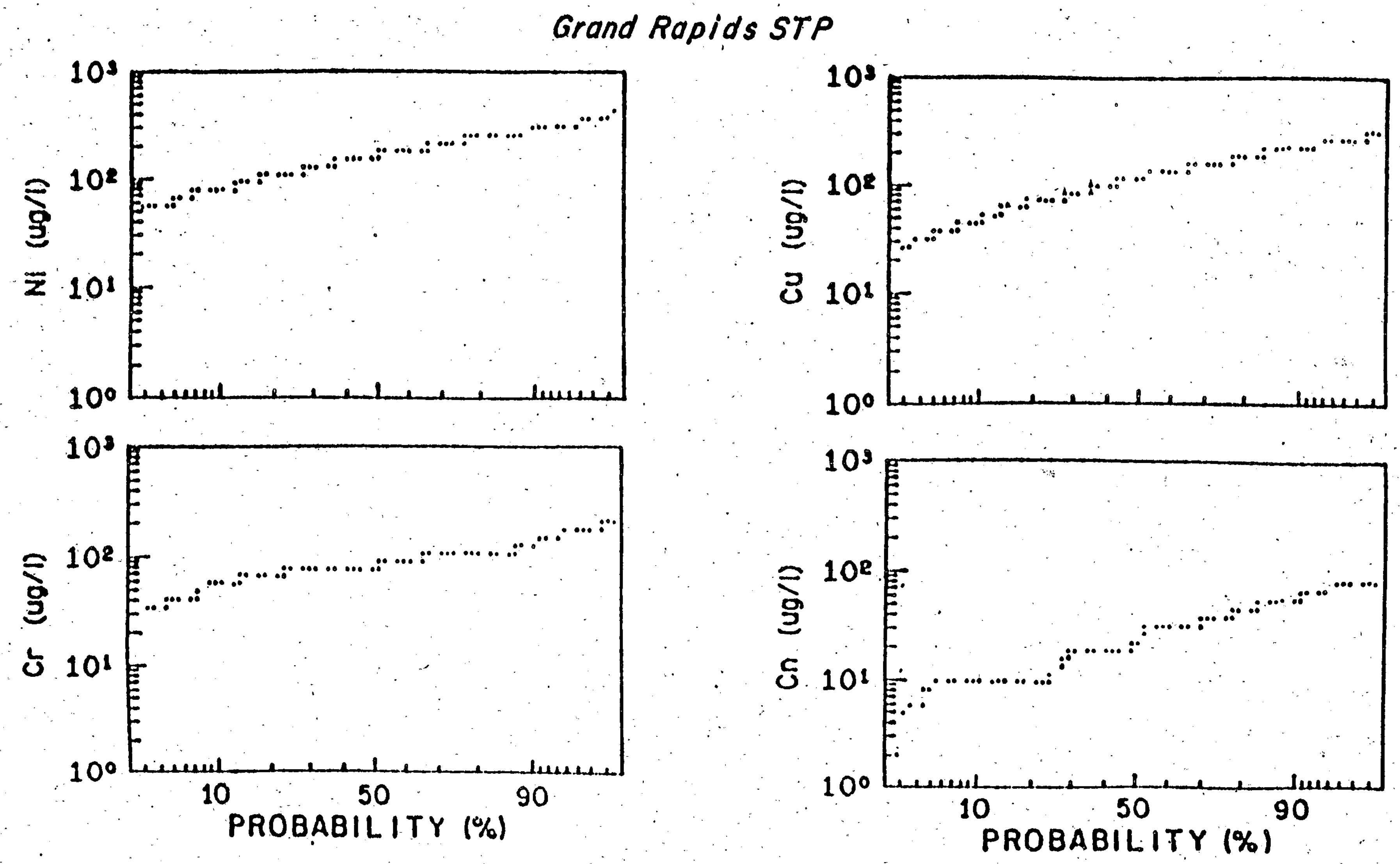


Figure B-3 -- Probability distribution of treatment plant effluent concentrations - heavy metals.

TABLE B-1 - Comparison of observed and computed downstream concentrations(2).

Median (50th Percentile) Concentrations

Location	Variable	Model Prediction	Observed Quantile	Confidence Limit of Observed Quantile
North Buffalo Creek, NC	BOD (mg/l)	9.7	10.0	8.5 - 11.0
	COD (mg/l)	51.0	59.0	47.0 - 66.0
	TSS (mg/l)	16.0	15.0	12.0 - 22.0
Jackson River, VA	BOD (mg/l)	6.0	5.3	4.2 - 6.0
	TSS (mg/l)	15.8	13.6	10.0 - 17.0
	Color (PCU)	110.0	100.0	90.0 - 130.0
Haw River, NC	BOD (mg/l)	2.0	1.7	1.5 - 1.7
	COD (mg/l)	23.8	22.0	19.0 - 26.0
Pigeon River, NC	BOD (mg/l)	3.7	3.8	3.0 - 5.1
	COD (mg/l)	85.0	78.0	65.0 - 87.0
Mississippi River, MN	NH_3 (mg/l)	1.0	1.1	1.0 - 1.2

95th Percentile Concentrations

Location	Variable	Model Prediction	Observed Quantile	Confidence Limit of Observed Quantile
North Buffalo Creek, NC	BOD (mg/l)	31.0	22.0	20.0 - 33.0
	COD (mg/l)	120.0	97.0	82.0 - 129.0
	TSS (mg/l)	15.8	13.6	10.0 - 17.0
Jackson River, VA	BOD (mg/l)	18.1	15.6	13.0 - 20.0
	TSS (mg/l)	41.6	32.0	30.0 - 40.0
	Color (PCU)	324.0	330.0	300.0 - 410.0
Haw River, NC	BOD (mg/l)	4.5	4.7	3.2 - 5.6
	COD (mg/l)	43.0	46.0	33.0 - 53.0
Pigeon River, NC	BOD (mg/l)	8.7	7.6	6.4 - 9.4
	COD (mg/l)	186.0	229.0	188.0 - 233.0
Mississippi River, MN	NH_3 (mg/l)	3.5	4.3	3.2 - 5.0

distribution of quantiles, are also listed. In all but one case, the computed quantiles are within the confidence limits.

Thus, there is no statistical evidence to reject the computed quantiles as not being the true quantiles of the observed concentration distribution. This is strong statistical evidence that indeed the log-normal probabilistic dilution model is representative of actually observed downstream concentration distributions for the 95th percentile at least.

The 11 data sets used in the verification analysis were examined for cross correlations between effluent flows and concentrations. The observed ranges in correlation coefficients have no significant impact on the computation. Correlations between stream flow and effluent load for a point source are not expected. Upstream concentrations are not employed in the comparison of permit averaging period effects, so that any correlation between stream flow and concentration is not relevant to this analysis. Modifications to the probabilistic dilution model computations are available for use in situations where cross correlations must be considered [1].

The influence of possible deviations from the assumed log-normality of the upstream and effluent flows and concentrations upon more extreme quantiles is unknown at present due to lack of larger data sets that encompass these extreme quantiles. However, the quality of the alternatives to and the simplicity of this model argue strongly for its use in the present context of describing comparative differences in water quality impacts.

B-3. Appropriateness of Assumptions

We have chosen to ignore the seasonal and day-to-day correlation structure of both stream flow and effluent behavior in order to simplify the characterization of each variable. The consequences of this simplification are discussed below in more detail, but it should be pointed out that trends and correlations do not invalidate the use of the log-normal probability distribution function to characterize the frequency of occurrence of flows and concentrations. Trends and day-to-day correlations affect the time sequences with which certain values occur, but not their long term frequency of occurrence. This is judged to be an acceptable penalty to be endured when compared to the simplification achieved. If a more refined, site specific analysis is required, then a seasonal breakdown of the data, with the appropriate means and standard deviations for each time period, can be generated and the analysis performed as described below.

The consequence of a possible serial correlation can be approximately quantified as follows. If, in fact, the serial correlation is such that 10 consecutive daily violations always occur when one violation occurs, then the proper percentile to consider is not 0.0274 (10 years) but rather 0.274 (1 year return period). The degree to which the 10 year return period concentration is overestimated can be estimated by comparing the ratio of the 10 year to the 1 year stream concentrations which are computed without regard to serial correlation.

The ratio of the 10 year return period concentration to that for some other return period can be computed for log-normally distributed random concentrations by:

$$\frac{C_{10\ yr}}{C_{x\ yr}} = \text{EXP}\ [(Z_{10\ yr} - Z_{x\ yr})\ \sigma\ \text{ln}C]$$

where

σ lnc = log standard deviation of stream concentrations (C)

$Z_{10\ yr}$, $C_{10\ yr}$ = Z score and concentration corresponding to a 10 year return period

$Z_{x\ yr}$, $C_{x\ yr}$ = Z score and concentration corresponding to an x year return period

Table B-2 summarizes results for a range of values for coefficient of variation of stream concentrations. Clustering tendencies of 5 and 10 are examined as approximations of the degree of serial correlation which might exist. If clusters of 10 occur, the comparison is between 10 and 1 year return periods as discussed above; for clusters of 5, the comparison is between 10 and 2 year return periods. On the basis of this analysis, the water quality effects presented in Chapter 4 for various permit averaging periods may overstate the 10 year stream concentrations by approximately a factor of 1.5 to 2.0.

Until stream and effluent data can be analyzed to define the serial correlation structure and the methodology modified to incorporate it, the results presented in Table B-2 should be interpreted to indicate with the following possibilities:

TABLE B-2.- Approximate overestimation of 10 year return period stream concentration by ignoring serial correlation.

Variability of Stream Concentration		Ratio of Stream Concentration At Indicated Average Return Periods	
Coefficient of Variation (ν_C)	Log Sigma (σ_{lnc})	10 Year to 1 Year (C_{10}/C_1)	10 Year to 2 Year (C_{10}/C_2)
0.5	0.4724	1.4	1.25
1.0	0.8326	1.8	1.50
1.5	1.0857	2.1	1.65
2.0	1.2686	2.4	1.80

$$\frac{C_{10}}{C_{1,2}} = \mathrm{EXP}\,[(Z_{10} - Z_{1,2})\ \sigma_{lnc}]$$

Z_{10} (10 year Return Period) = 3.456

Z_1 (1 year Return Period) = 2.778

Z_2 (2 year Return Period) = 2.996

- o Stream concentrations indicated by the methodology used in the report to recur on average for 1 day every 10 years would, if they actually never occur except in clusters of 5 to 10 days, have return periods of 50 to 100 years.

- o Conversely, for the same clustering assumptions, the stream concentrations that occur at 10-year intervals should be 50 to 70% (1/2 to 1/1.5) of the 10-year concentrations projected by the report methodology.

B-4. References

1. DiToro, D.M., "Probability Model of Stream Quality Due to Runoff." J. Environmental Engr. ASCE, Vol. 110, #3, June 1984 p. 607-628.

2. DiToro, D.M. and Fitzpatrick, J.J., "Verification Analysis of the Probabilistic Dilution Model" Report prepared for EPA Contract No. 68-01-6275, U.S. Environmental Protection Agency, Washington, D.C., (1982).

APPENDIX C

Characteristic Values for Input Parameters

The results reported here represent an attempt to develop characteristic values and ranges for stream flow and effluent variability. These values and ranges have been extracted from the results of published analyses, and are used in Chapter 4 to evaluate the influence of the permit averaging period on typical receiving water conditions. These values are provided for effluent flows (Section 1), effluent concentrations (Section 2), and stream flow (Section 3).

C-1. Treatment Plant Effluent Flows

A recent study [1] analyzed several years of performance data from approximately 400 secondary treatment plants in 8 different process categories. Average plant effluent flows ranged from 0.002 to 82 MGD. Table C-1 summarizes the coefficient of variation of treatment plant effluent flows.

C-2. Treatment Plant Effluent Concentrations

Data on the variability of effluent BOD_5 and total suspended solids (TSS) from municipal biological treatment plants are available from several sources. Niku, et al. [2] provide analysis results for 37 activated sludge plants which show the coefficient of variation of effluent BOD_5 concentrations to range between 0.34 and 1.11 for individual plants. The median of the individual plant values was 0.635. The EPA research report [3] on which the foregoing was based reported a mean coefficient of variation for 43 activated sludge plants using a variety of processes. Daily effluent concentrations were found to be well represented

TABLE C-1 - Coefficient of variation of daily effluent flows, ν_{QE}.

Process Category	Number of Plants	Range For Individual Plants	Median of All Plants
Trickling Filter Rock	64	0.06 - 0.97	0.27
Trickling Filter Plastic	17	0.16 - 0.88	0.38
Conventional Activated Sludge	66	0.04 - 1.04	0.24
Contact Stabilization Activated Sludge	57	0.06 - 1.35	0.34
Extended Aeration Activated Sludge	28	0.11 - 1.32	0.34
Rotating Biological Contact	27	0.12 - 1.19	0.31
Oxidation Ditch	28	0.09 - 1.16	0.31
Stabilization Pond	37	0.00 - 0.83	0.31

by a log-normal distribution. The mean of all plants analyzed had coefficients of variation of 0.7 for BOD_5 and 0.84 for TSS.

Two recent studies have extended the analysis of effluent concentration variability, and report coefficients of variation of BOD_5 and TSS for 7-and 30-day averages as well as for daily values. Results reported by Hazen and Sawyer [1] provide the basis for the summary presented in Table C-2 as well as the two other sources cited in the table. An analysis of the performance of 11 trickling filter plants by Haugh, et al. [4] produced the results summarized by Table C-3.

Based on available data, a single representative value for coefficient of variation of effluent concentrations cannot be defined. The most appropriate characteristic value will be influenced by process category, effluent concentration averaging period, and the pollutant in question (e.g., BOD, TSS, etc.), as well as individual plant differences. The computations in this report are performed using a range of values estimated to encompass most of the conditions of interest.

C-3. Stream Flow

Figure C-1 provides a basis for estimating the coefficient of variation of daily stream flows on the basis of the ratio of 7Q10 to average ($\overline{QS}$) stream flow. These flow values are usually readily available. The relationship shown is derived from a set of flow measurements and statistics which has been developed for a sample of 130 streams in various areas of the country [5] and is summarized in Table C-4, along with additional details on the location of the stream gages used. The ranges

TABLE C-2 - Summary of secondary treatment plant performance - median coefficients of variation, ν_{CE} (from reference 1).

Process Category	Number of Plants	Effluent BOD (mg/l) Mean	Effluent BOD Coefficient of Variation* Daily Values	Effluent BOD Coefficient of Variation* 7-Day Avgs.	Effluent BOD Coefficient of Variation* 30-Day Avgs.	Effluent TSS (mg/l) Mean	Effluent TSS Coefficient of Variation* Daily Values	Effluent TSS Coefficient of Variation* 7-Day Avgs.	Effluent TSS Coefficient of Variation* 30-Day Avgs.
Trickling Filter Rock	64	26.0	0.40	0.30	0.25	25.3	0.50	0.30	0.25
Trickling Filter Plastic	17	19.0	0.50	0.35	0.30	19.4	0.65	0.55	0.40
Conventional Activated Sludge	66	14.8	0.65	0.55	0.40	14.3	0.85	0.60	0.45
Contact Stabilization Activated Sludge	57	12.6	0.60	0.50	0.40	13.8	0.70	0.65	0.50
Extended Aeration Activated Sludge	28	7.2	0.70	0.60	0.45	9.8	0.65	0.45	0.30
Rotating Biological Contacter	27	17.0	0.60	0.45	0.35	15.2	0.70	0.50	0.35
Oxidation Ditch	28	8.4	0.60	0.55	0.40	12.3	0.70	--	0.50
Stabilization Pond	37	22.7	0.50	0.45	0.40	39.5	0.65	0.55	0.45

Values shown are rounded to nearest 0.05 for $\nu_{(CE)}$

*Basis: $\nu_{CE} = \frac{\text{Standard Deviation of Median Plant}}{\text{Mean of Median Plant}}$

Table C-2 (Cont.)

Chemical Precipitation/Settling[1]

Pollutant	Coefficient of Variation
Cr	.99
Cu	.60
Fe	.57
Mn	.84
Ni	.81
Zn	.84
Tss	.66

Pharmaceutical Industry[2]

Plant Number	Coefficient of Variation BOD	(n)	TSS	(n)
12015	1.01	46	.85	195
12072	.97	392	.63	395
12026	.95	44	.49	53
12036	.74	366	1.12	364
12097	1.08	222	1.21	249
12098	1.37	24	1.52	25
12117	.70	39	.81	51
12160	.92	34	1.11	32
12161	.55	249	.99	355
12186	.71	54	.50	54
12187	.21	12	.26	12
12136	1.02	110	1.16	111
12248	.58	50	.55	52
12257	.64	56	.92	56
12294	.93	56	1.25	50
12307	1.55	39	1.34	38

[1]From Table 3, page 14 of 10-18-83 memorandum from H. Kahn to E. Hall titled, "Revisions to Data and Analysis of the Combined Metals Data Base."

[2]From preliminary descriptive statistics generated on pharmaceutical data by SRI International, 11-12-82.

TABLE C-3 - Effluent concentration variability for trickling filters (from reference 4).

	BOD_5	TSS
Mean for 11 plants (mg/l)	29.6	29.3
Coefficient of Variation (median of individual plant values):		
Daily Values	0.39	0.55
7-Day Averages	0.35	0.31
30-Day Averages	0.31	0.26

shown reflect the bulk of the data in the sample of stream records which were used. However, a relatively small percentage of streams will have coefficients of variation which fall outside the indicated ranges. The statistical analysis was performed for the entire period of record. Results in some cases may be distorted, if flow regulation works were installed on the stream sometime during the period of record.

C.4. References

1. Hazen and Sawyer, "Review of Performance of Secondary Municipal Treatment Works." Draft Final Report for Contract 68-01-6275, Work Assignment No. 5, U.S. Environmental Protection Agency, Washington, D.C., (December 1982).

2. Niku, Shroeder, and Samaniego, "Performance of Activated Sludge Process and Reliability Related Design." JWPCF, Vol. 51, No. 12, (December 1979).

3. Niku, et al., "Performance of Activated Sludge Processes: Reliability, Stability and Variability." EPA 600/52-81-227, (December 1981).

4. Haugh, et al. "Performance of Trickling Filter Plants: Reliability, Stability and Variability." EPA 600/52-81-228. (December 1981).

5. Driscoll & Associates, "Combined Sewer Overflow Analysis Handbook for Use in 201 Facility Planning." Report prepared for EPA Contract No. 68-01-6148, U.S. Environmental Protection Agency, Washington, D.C. (1981).

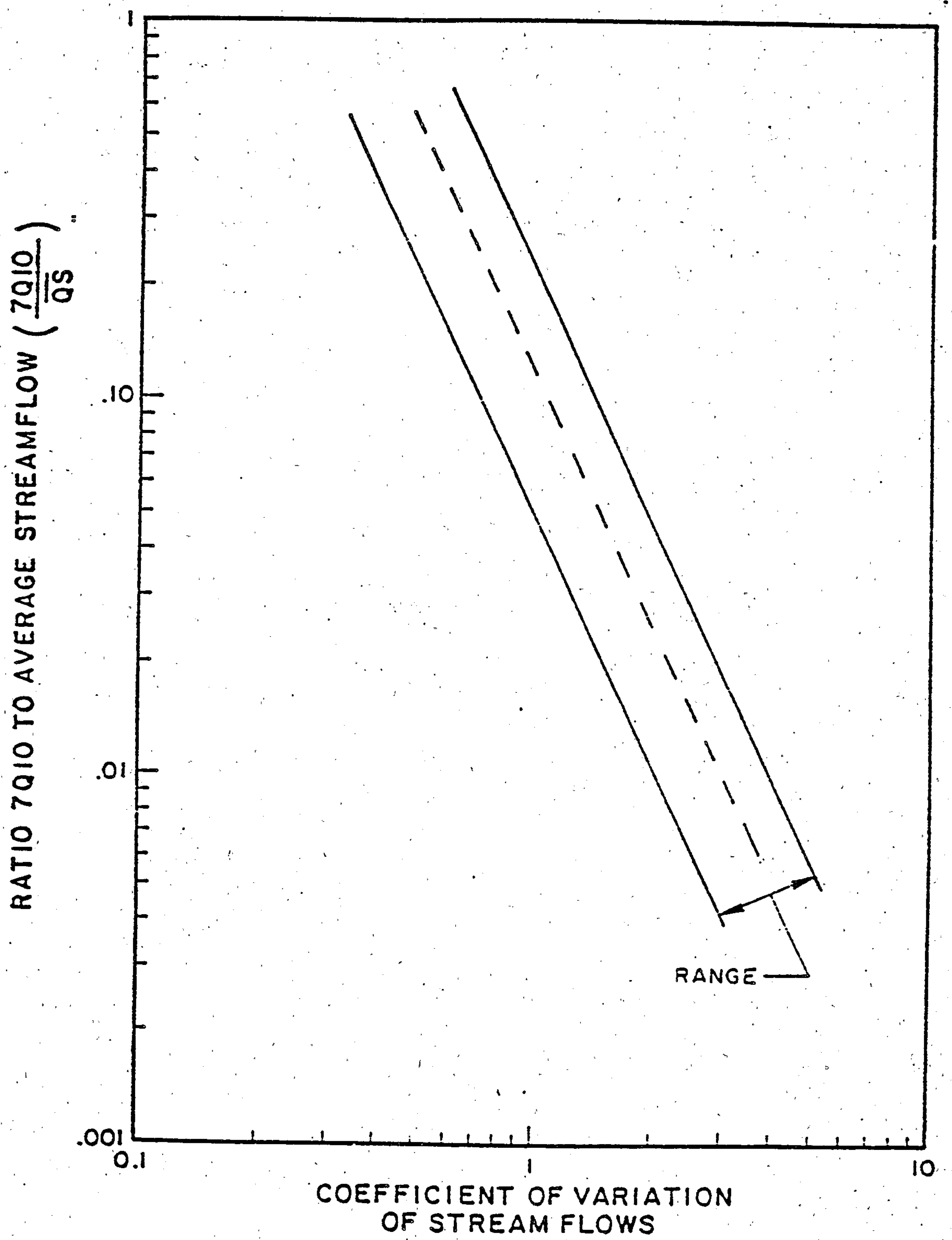


Figure C-1 - Typical low flow characteristics of U.S. streams.

TABLE C-4 - Summary of stream flow characteristics.

USGS Gage No.	Gage Location: State	River	(At or Near)	Drain Area (MI^2)	Stream Flow (cfs/MI^2): $\bar{Q}$	$\tilde{Q}$	7Q10	1Q2	ν_Q	$\frac{7Q10}{\bar{Q}}$	$\frac{7Q10}{1Q2}$
01 01 1000	ME	Alagash River	Alagash, ME	1250	1.49	0.84	.102	.034	1.46	.068	2.95
03 6500	ME	Kenduskeag Stream	Kenduskeag, ME	178	1.72	.62	.011	.008	2.58	.006	1.33
02 1500	ME	Machias River	Whitneyville, ME	457	2.00	1.30	.130	.081	1.17	.064	1.59
07 3000	ME	Oyster River	Durham, NH	12	1.49	.66	0	.016	2.02	0	0
09 1000	NH	S. Br. Piscataquag River	Goffstown, NH	104	1.58	.73	.029	.017	1.91	.018	1.67
09 4500	MA	N. Nashua River	Leominster, MA	110	1.75	1.19	.300	.086	1.07	.172	3.47
16 2500	MA	Priest Brook	Winchendon, MA	19	1.60	.77	0	.021	1.81	0	0
17 6000	MA	Quaboag River	W. Brimfield, MA	151	1.58	1.01	.093	.060	1.19	.06	1.56
18 1000	MA	W. Br. Westfield River	Huntington, MA	94	1.90	.96	.053	.030	1.70	.03	2.2
11 1500	RI	Branch River	Forestdale, RI	91	1.82	1.14	.132	.061	1.24	.07	2.1
12 4000	CT	Quinebaug River	..., CT	156	1.77	1.04	.103	.050	1.37	.06	2.1
12 7500	CT	Yantic River	..., CT	90	1.69	.91	.044	.042	1.56	.03	1.1
33 4500	NY	Hoosic River	Eagle Bridge, NY	510	1.75	1.15	.186	.076	1.14	.11	2.4
36 1500	NY	Catskill Creek	Oak Hill, NY	98	1.27	.35	0	.003	3.51	0	0
37 7000	NJ	Hackensack River	Rivervale, NJ	58	1.55	1.07	.121	.079	1.05	.08	1.5
39 8500	NJ	N. Br. Raritan River	Far Hills, NJ	26	1.72	1.20	.076	.095	1.03	.04	.8
42 0500	NY	Beaver Kill	Cook Falls, NY	241	2.26	1.34	.133	.068	1.35	.06	1.9
43 5000	NY	Neversink River	Claryville, NY	66	2.68	1.74	.152	.102	1.17	.06	1.5
44 9500	PA	Wild Creek	Hatchery, PA	17	2.02	1.49	.119	.149	0.91	.06	.8
48 1500	DE	Brandywine Creek	Wilmington, DE	314	1.38	1.11	.217	.175	0.74	.16	1.2

TABLE C-4 (Cont.)

USGS Gage No.	Gage Location			Drain Area (MI^2)	Stream Flow (cfs/MI^2)						
	State	River	(At or Near)		$\bar{Q}$	$\tilde{Q}$	7Q10	1Q2	ν_Q	$\frac{7Q10}{\bar{Q}}$	$\frac{7Q10}{1Q2}$
01 50 0500	NY	Susquehanna River	Unadilla, NY	982	1.57	.89	.081	.037	1.45	.05	2.2
51 1500	NY	Tionghnioga River	Itaska, NY	730	1.66	.78	.077	.018	1.87	.05	4.1
52 9500	NY	Cohocton River	Campbell, NY	470	.93	.45	.045	.012	1.79	.05	3.8
54 3000	PA	Driftwood Brook	Sterling, PA	272	1.63	.66	.011	.012	2.26	.01	.9
55 5500	PA	East Mahantango Creek	Dalmatia, PA	162	1.30	.69	.025	.025	1.58	.02	1.0
58 6000	MD	N. Br. Patapsco River	Ceda, MD	57	1.04	.82	.124	.106	0.78	.12	1.2
59 1000	MD	Patuxent River	Unity, MD	35	.98	.75	.086	.086	0.80	.09	1.0
59 7000	MD	Crabtree Creek	Swanton, MD	17	1.68	.75	0	.018	1.98	0	0
61 7000	WV	Tuscarora Creek	Martinsburg, WV	11	.80	.63	0	.071	.78	0	0
61 5000	VA	Opequon Creek	Berryville, VA	57	.64	.31	.017	.009	1.82	.03	2.0
64 5000	MD	Seneca Creek	Dawsonville, MD	101	.89	.66	.050	.066	.91	.06	.7
65 7000	VA	Bull Run	Manassas, VA	148	.88	.23	0	.001	3.67	0	0
66 3500	VA	Hazel River	Rixeyville, VA	287	1.15	.67	.014	.031	1.40	.01	0.4
02 01 2500	VA	Jackson River	Falling Sprg, VA	411	1.16	.70	.151	.036	1.32	.13	4.1
03 4000	VA	Rivanna River	Palmyra, VA	664	1.08	.62	.036	.027	1.42	.03	1.3
06 2500	VA	Roanoke (Staunton) River	Brookne, VA	2415	1.02	.69	.142	.046	1.10	.14	3.1
05 3500	NC	Ahoskie Creek	Ahoskie, NC	57	1.12	.31	0	.001	3.52	0	0
10 6500	NC	Black River	Tomahawk, NC	680	1.10	.71	.034	.044	1.19	.03	.8
09 9500	NC	Deep River	Randlenar, NC	124	.96	.45	.048	.010	1.89	.05	4.6
11 1000	NC	Yadkin River	Patterson, NC	29	1.59	1.33	.276	.231	.64	.17	1.2
13 8500	NC	Linville River	Nebo, NC	67	2.10	1.52	.223	.134	.96	.10	1.7
15 2500	NC	First Broad River	Lawndale, NC	198	1.41	1.02	.258	.091	.95	.18	2.8

TABLE C-4 (Cont.)

USGS Gage No.	Gage Location: State	Gage Location: River	Gage Location: (At or Near)	Drain Area (MI^2)	Stream Flow (cfs/MI^2): $\bar{Q}$	Stream Flow (cfs/MI^2): $\tilde{Q}$	Stream Flow (cfs/MI^2): 7Q10	Stream Flow (cfs/MI^2): 1Q2	ν_Q	$\frac{7Q10}{\bar{Q}}$	$\frac{7Q10}{1Q2}$
02 20 7500	GA	Yellow River	Covington, GA	378	1.13	.76	.061	.058	1.06	.05	1.0
21 6000	GA	Little Ocmulgee River	Towns, GA	329	.80	.20	.006	.001	3.84	.01	5.0
23 8000	FL	Haines Creek	Lisbon, FL	648	.45	.20	.154	.005	2.02	.34	33.0
29 7100	FL	Joshua Creek	Nocatee, FL	132	.89	.17	0	0	5.17	0	0
30 2500	FL	Blackwater Creek	Knights, FL	110	.93	.93	.018				
32 6900	FL	St. Marks River	Newport, FL	535	1.37	1.29	.600	.458	.36	.44	1.3
33 7000	GA	Sweetwater Creek	Austell, GA	246	1.35	.81	.057	.011	1.33	.04	5.2
34 3300	AL	Abbie Creek	Haleburg, AL	144	1.39	1.08	.208	.125	.82	.15	1.7
36 9000	FL	Shoal River	Crestview, FL	474	2.27	2.20	.635	.156	.24	.28	4.1
38 3500	GA	Coosawattee River	Pine Chapel, GA	856	1.70	1.26	.312	.116	.90	.18	2.7
39 2000	GA	Etowah River	Canton, GA	605	1.89	1.58	.405	.299	.66	.21	1.4
41 2000	AL	Tallapoosa River	Heflin, AL	444	1.41	.97	.065	.149	1.07	.05	0.4
42 2500	AL	Mulberry Creek	Jones, AL	208	1.50	.94	.226	.120	1.25	.15	1.9
43 4000	MS	Town Creek	Tupelo, MS	110	1.53	.14	0	.003	10.79	0	0
45 6000	AL	Turkey Creek	Morris, AL	82	1.53	.74	.123	.020	1.83	.08	6.25
47 6500	MS	Sowashee Creek	Meridian, MS	52	1.08	.32	0	.004	3.26	0	0
48 0500	MS	Tuxachanzie Creek	Biloxi, MS	92	1.98	.60	.032	.005	3.13	.02	6.0
48 4000	MS	Yockanookany River	Kosciusko, MS	484	1.25	.21	.017	.001	5.74	.01	20.0
03 02 5000	PA	Sugar Creek	Sugar Creek, PA	166	1.57	.86	.10	.03	1.52	.06	2.9
05 3500	WV	Buckhannon River	Hall, WV	277	2.12	.90	.007	.05	2.14	.003	.1
06 5000	WV	Dry Fork	Henricks, WV	345	2.13	1.05	.023	.03	1.77	.01	.8
10 9500	OH	L. Beaver Creek	Liperpool, OH	496	1.02	.48	.04	.01	1.85	.04	2.7

TABLE C-4 (Cont.)

USGS Gage No.	Gage Location: State	River	(At or Near)	Drain Area (MI^2)	Stream Flow (cfs/MI^2): $\bar{Q}$	$\tilde{Q}$	7Q10	1Q2	ν_Q	$\frac{7Q10}{\bar{Q}}$	$\frac{7Q10}{1Q2}$
03 14 6500	OH	Licking River	Newark, OH	537	.99	.50	.07	.01	1.71	.07	4.6
15 7500	OH	Hocking River	Enterprise, OH	459	.95	.49	.063	.01	1.69	.07	4.3
17 0000	VA	Little River	Graysonton, VA	300	1.20	.93	.223	.11	.81	.19	2.0
18 6500	WV	Williams River	Dyer, WV	128	2.50	1.12	.008	.03	1.95	.003	.3
21 3500	VA	Panther Creek	Panther, WV	31	1.17	.36	0	.003	3.09	0	0
22 4500	OH	Whetstone Creek	Ashley, OH	99	.89	.28	0	.003	3.04	0	0
24 0000	OH	L. Miami River	Oldtown, OH	129	.74	.46	.05	.02	1.26	.07	2.1
32 4000	IN	Little River	Huntington, IN	263	.84	.25	.01	.003	3.20	.01	4.3
35 2500	IN	Fall Creek	Millersville, IN	298	.78	.48	.04	.03	1.28	.06	1.6
35 7500	IN	Big Walnut Creek	Reelsville, IN	326	.98	.41	.01	.008	2.18	.01	1.5
42 1000	TN	Collins River	McMinnville, TN	640	1.78	.83	.096	.02	1.91	.05	4.8
42 7500	TN	E. Fork Stones River	Lascass, TN	262	1.58	.48	.01	.003	3.13	.01	3.8
04 02 7500	WI	White River	Ashland, WI	279	1.04	.85	.47	.13	.69	.45	3.6
04 6000	MI	Black River	Garnet, MI	28	.93	.75	.21	.09	.78	.23	2.4
06 4500	WI	Pine River	Pine R. Pwrplnt, WI	528	.79	.61	.13	.07	.85	.16	1.8
08 6500	WI	Cedar Creek	Cedarburg, WI	121	.51	.23	.008	.005	2.01	.02	1.7
11 4500	MI	Looking Glass River	Eagle, MI	281	.56	.34	.05	.02	1.34	.09	2.8
12 3000	MI	Big Salle River	Freesail, MI	127	1.09	1.05	.67	.43	.30	.61	1.5
15 5500	MI	Pine River	Midland, MI	390	.69	.51	.08	.05	.93	.12	1.6
15 9500	MI	Black River	Fargo, MI	480	.56	.14	.01	.001	3.90	.02	10.0
16 6500	MI	River Range	Detroit, MI	187	.56	.29	.02	.009	1.63	.04	2.2
18 0000	IN	Cedar Creek	Cedarville, IN	270	.85	.42	.07	.011	1.77	.08	6.3

TABLE C-4 (Cont.)

USGS Gage No.	Gage Location: State	River	(At or Near)	Drain Area (MI^2)	Stream Flow (cfs/MI^2): $\bar{Q}$	$\tilde{Q}$	7Q10	1Q2	ν_Q	$\frac{7Q10}{\bar{Q}}$	$\frac{7Q10}{1Q2}$
04 19 9000	OH	Huron River	Milan, OH	371	.72	.24	.008	.003	2.79	.01	2.7
22 7500	NY	Genesee River	Jones Bridge, NY	1417	1.12	.58	.05	.019	1.66	.05	2.7
05 29 3000	MN	Yellow Bank River	Odessa, MN	398	.14	.025	0	0	5.45	0	0
38 5500	MN	S. Fork Root River	Howton, MN	275	.45	.40	.196	.098	0.49	.44	2.0
41 3500	WI	Grant River	Burton, WI	269	.59	.42	.138	.035	.99	.23	3.9
41 7700	IA	Bear Creek	Monmouth, IA	61	.64	.34	.03	.011	1.59	.05	2.9
40 6500	WI	Black Earth Creek	Black Earth, WI	46	.61	.60	.26	.330	.19	.43	.8
43 2500	WI	Pecatonica River	Darlington, WI	273	.66	.44	.117	.030	1.11	.18	3.9
44 4000	IL	Elkhorn Creek	Penrose, IL	146	.56	.38	.10	.030	1.07	.17	3.4
45 7000	MN	Cedar River	Austin, MN	425	.41	.23	.05	.010	1.50	.12	5.1
45 5500	IA	English River	Kalona, IA	573	.57	.16	.003	.001	3.29	.01	2.2
48 6000	IA	North River	Norwalk, IA	349	.49	.09	0	.006	5.54	0	0
50 2000	MO	Bear Creek	Hannibal, MO	31	.48	.11	0	.001	4.43	0	0
51 5000	IN	Kankakee River	North Liberty, IN	174	.81	.76	.30	.260	.37	.38	1.2
52 8000	IL	Des Plaines River	Gurnee, IL	232	.52	.14	0	.001	3.64	0	0
55 4500	IL	Vermillion River	Pontiac, IL	579	.58	.15	0	.001	3.80	0	0
57 8500	IL	Salt Creek	Rowell, IL	335	.64	.24	.006	.003	2.43	.01	1.7
12 33 5000	MT	Blackfoot River	Helmville, MT	481	.73	.45	.146	.025	1.28	.20	5.7
37 0000	MT	Swan River	Bigfork, MT	671	1.70	1.21	.380	.109	.98	.22	3.5
32 1500	ID	Boundary Creek	Porthill, ID	97	1.98	.82	.124	.015	2.19	.06	8.0
45 5000	WA	Wenatchee River	Wentch. L., WA	273	4.82	2.97	.54	.147	1.28	.11	3.7
17 7500	OR	Stetattle Creek	Newhalem, WA	22	8.40	5.82	.82	.445	1.05	.10	1.8

TABLE C-4 (Cont.)

USGS Gage No.	Gage Location: State	River	(At or Near)	Drain Area (MI^2)	Stream Flow (cfs/MI^2): $\overline{Q}$	$\tilde{Q}$	7Q10	1Q2	ν_Q	$\frac{7Q10}{\overline{Q}}$	$\frac{7Q10}{1Q2}$
12 13 3000	WA	S. Fork Skyromish River	Index, WA	355	6.90	4.71	.80	.344	1.07	.12	2.3
14 8000	WA	S. Fork Tolt River	Carnation, WA	20	10.00	4.97	.76	.152	1.75	.08	5.0
10 4500	WA	Green River	Lester, WA	96	4.27	2.41	.29	.094	1.46	.07	3.1
08 2500	WA	Nisqually River	National, WA	133	5.92	4.90	1.25	.83	.68	.21	1.5
04 8000	WA	Dungeness River	Sequim, WA	156	2.45	1.94	.56	.26	.77	.23	2.2
01 3500	WA	Willapa River	Willapa, WA	130	5.04	2.02	.138	.038	2.29	.03	3.6
02 4000	WA	S. Fork Newaukum River	Onal..., WA	42	4.74	2.88	.49	.142	1.30	.11	3.5
13 04 7500	ID	Falls River	Squirrel, ID	326	2.44	1.87	.80	.205	.83	.33	3.9
18 5000	ID	Boise River	Twin Springs, ID	830	1.41	.87	.25	.048	1.28	.18	5.3
29 2000	OR	Imnaha River	Imnaha, OR	622	.80	.49	.10	.024	1.30	.13	4.3
31 3000	ID	Johnson Creek	Yellow Pine, ID	213	1.61	.75	.206	.019	1.90	.13	10.7
35 1000	WA	Palouse River	Hooper, WA	2500	.24	.07	.001	.001	3.03	.01	1.5
14 01 7000	WA	Tonchet River	Bolles, WA	361	.65	.35	.033	.014	1.55	.05	2.4
05 7500	OR	Fall River	LaPine, OR	45	3.41	3.27	2.18	1.33	.31	.64	1.6
14 5500	OR	M. Fork Willamette River	above Salt Cr., OR	392	2.90	1.97	.45	.14	1.09	.16	3.3
22 2500	WA	E. Fork Lewis River	Heisson, WA	125	6.12	3.08	.30	.09	1.72	.05	3.1
22 6500	WA	Cowlitz River	Packwood, WA	287	5.75	4.12	.832	.38	.97	.14	2.2
17 1000	OR	Mary's River	Philomath, Or	159	2.97	.86	.03	.006	3.31	.01	5.6
18 2500	OR	Little N. Santiam River	Meh..., OR	112	6.85	3.18	.18	.08	1.91	.03	2.2
20 3500	OR	Tualatin River	Dilley, OR	125	3.18	1.08	.016	.013	2.78	.01	1.25
31 2000	OR	S. Umpqua River	Brockway, OR	1670	1.74	0.56	.036	.006	2.96	.02	6.1
34 1500	OR	S. Fork Little Butte Cr.	Lakec..., OR	138	0.78	.39	.050	.011	1.70	.07	4.4
37 2500	OR	E. Fork Illinois River	Takilma, OR	42	4.38	1.62	.142	.02	2.52	.03	6.0

APPENDIX D

Computer Program for the Probabilistic Dilution Model - Point Source (PDM-PS)

This appendix describes a computer program (PDM-PS) which performs the computations of the Probabilistic Dilution Model for Point Source discharges using numerical methods based on quadratures. The program is written in BASIC for the HP-85 and the IBM-PC, and should be readily applicable to other personal computers with perhaps minor modifications to reflect individual machine characteristics.

The program is structured around a slightly different input format than that used for the manual calculation using the moments approximation. A series of normalizations (ratios) of certain of the input data items is used to provide a computation framework that provides a more generalized perspective.

The appendix is organized as follows. Section 1 describes the basis for the formulation and normalization of the input data, as used in the program. Section 2 provides an annotated description of the CRT and printer functions, as well as the nature of the user's response. Figures D-1 and D-2 provide the results of running the PDM-PS through the example described in Section 3.2 of this report. Finally, Figure D-3 provides a listing of the PDM-PS program for entry into a personal computer.

D-1. Formulation and Normalization

The analysis can be made more useful in a general way if the normalization described below is applied to reduce certain of the inputs to readily recognized ratios, and to express results (stream concentrations) as a multiple or fraction of the target stream concentration (CL).

The explicit assumptions in the normalization scheme that is used are that:

- The stream target concentration (CL) is produced when the discharge flow is the mean effluent flow ($\overline{QE}$), the discharge pollutant concentration is equal to the permit effluent limit (EL), and the stream flow is equal to the design value (here designated 7Q10 - though any other basis may be used for designating the numerical value of stream design flow, e.g., 30Q5, 30Q10, etc.).

- The reduction factor ($R = \overline{CE}/EL$) determines the mean effluent concentration of the pollutant being evaluated. It could be selected arbitrarily; however, as applied in this manual for evaluating the permit averaging period, the value selected will be dictated by the variability of effluent concentrations and the permit averaging period.

In the usual case, where the stream target concentration (CL) is set at the chronic toxicity level, the multiples of the target - in which stream concentrations are expressed (CO/CL) - correspond with the acute toxicity level. The basis for the normalization scheme adopted is as follows.

The downstream concentration, CO, is given by the dilution equation:

$$CO = \frac{CE\ QE}{QS + QE} = \Phi\, CE \qquad \text{(D-1)}$$

For a chronic criteria concentration, CL, the effluent limit concentration, EL, is computed using QS = 7Q10 and an average effluent flow, $\overline{QE}$:

$$CL = \frac{EL\ \overline{QE}}{7Q10 + \overline{QE}} = EL\Phi_{STD} \qquad \text{(D-2)}$$

where Φ_{STD} is the effluent dilution factor at the standard conditions, $\Phi_{STD} = \overline{QE}/(7Q10 + \overline{QE})$. Thus:

$$EL = CL/\Phi_{STD} \qquad \text{(D-3)}$$

However, the choice of permit averaging period forces a reduction of $\overline{CE}$ of magnitude, R, so that permit violations occur only 5 percent or 1 percent of the time. Thus the actual long term average effluent concentration is:

$$\overline{CE} = R\ EL = R\ CL/\Phi_{STD} \qquad \text{(D-4)}$$

The problem is to compute the probability that the downstream concentration exceeds a multiple, β, of the chronic concentration, CL. In particular, if the acute criteria concentration is selected, then β is the acute to chronic criteria ratio for the pollutant being regulated. Hence it is necessary to compute:

$$\Pr[CO > \beta CL] = \Pr[CO > \beta\Phi_{STD}\ \overline{CE}/R] \qquad \text{(D-5)}$$

where Equation D-4 has been substituted for CL. Dividing both sides of the inequality by $\overline{CE}$ provides the first normalization since

$$CO/\overline{CE} = (CE/\overline{CE})\ \frac{QE}{QS + QE} \qquad \text{(D-6)}$$

and $CE/\overline{CE}$ is the normalized effluent concentration. The probability distribution of this random variable no longer depends upon the mean effluent concentration, but only on the coefficient of variation, ν_{CE}. This is easily seen from the following representation of a log-normal random variable:

$$\ln CE = \ln \widetilde{CE} + Z\sigma_{\ln CE} \tag{D-7}$$

where $\widetilde{CE}$ is the median, $\sigma_{\ln CE}$ is the log standard deviation, and Z is a standard normal random variable with zero mean and unit standard deviation. For log-normal random variables,

$$\overline{CE} = \widetilde{CE} \sqrt{(1 + \nu_{CE}^2)} \tag{D-8}$$

and

$$\sigma_{\ln CE}^2 = \ln(1 + \nu_{CE}^2) \tag{D-9}$$

so that Equation D-7 becomes

$$\ln(CE/\overline{CE}) = -1/2 \ \sigma_{\ln CE}^2 + Z\sigma_{\ln CE} \tag{D-10}$$

Thus, it is seen that $CE/\overline{CE}$ is log-normal with log mean = $-1/2\,\sigma_{\ln CE}^2$ and only the coefficient of variation, which specifies $\sigma_{\ln CE}$ through equation D-9, is required to completely specify the behavior of $CE/\overline{CE}$.

The final normalization results from expressing Equation D-6 as

$$CO/\overline{CE} = \frac{CE/\overline{CE}}{1 + QS/QE} \tag{D-11}$$

Note that QS/QE is log-normally distributed since both QS and QE are assumed to be log-normal. Thus, only the ratio of the average flows, $\overline{QS}/\overline{QE}$, is required. A convenient normalization using ratios that are more readily available results if the average effluent and stream flows are standardized relative to design stream flow (here designated by 7Q10). Defining

$$F1 = 7Q10/\overline{QS} \tag{D-12}$$

$$F2 = 7Q10/\overline{QE} \tag{D-13}$$

Then

$$\overline{QS}/\overline{QE} = F2/F1 \tag{D-14}$$

and

$$\Phi STD = \frac{1}{1 + F2} \tag{D-15}$$

These ratios, F1 and F2, together with the coefficients of variation, νQS, νQE, and νCE, completely specify the characteristics of the random variables in the normalized dilution Equation D-11. R specifies the effect of permit averaging period and β, the acute to chronic criteria ratio, specifies the toxicity behavior of the substance being considered. This completes the normalization.

D-2. Description of Program Use

The program is easy to use. The values of the input variables are sequentially requested on the CRT. Once the input values are entered, a summary of the input data is printed out, as is a tabular listing of the results of the calculations. The user should be thoroughly familiar with the theoretical and practical bases for the PDM-PS as described in Chapters 2 and 3 before attempting to use the PDM-PS.

USER: Initiates program execution.

PRINTER: Writes title.

CRT: Displays title and general descriptive material shown in Figure D-1.

CRT: Question #1 is displayed: "Enter coefficient of variation of QS, QE, and CE:"

USER: Enters the values of ν_{QS}, ν_{QE} and ν_{CE}, separated by commas.

CRT: Question #2 is displayed: "7Q10/avg QS?"

USER: Enters the ratio of the 7Q10 flow to the average stream flow ($\overline{QS}$).

CRT: Question #3 is displayed: "7Q10/avg QE?"

USER: Enters the design dilution ratio, i.e., the ratio of 7Q10 flow rate to the average effluent flow rate ($\overline{QE}$).

CRT: Question #4 is displayed: "avg CE/EL?"

USER: Enters the ratio of the average effluent concentration which the treatment plant will be designed to produce (avg CE), to the effluent concentration derived from the

WLA analysis (EL). This latter value is that concentration in the effluent which will result in the stream target concentration being met, when the following flow conditions prevail:

Stream flow (QS) is at the 7Q10 flow rate.

Effluent flow (QE) is at the average discharge rate of flow.

PRINTER: Prints a tabular summary of the input data selected.

CRT: Question #5 is displayed: "Enter lowest, highest and increment of multiple of target for which % exceedence is desired."

USER: Decides on a range of stream concentrations (expressed as multiples of the target concentration, CL) for which the probability of occurrence and the recurrence interval are desired. The user enters (1) the lowest value, (2) the highest value and (3) the incremental step desired for values between the highest and lowest.

PRINTER: Prints tabular listing of results. For each multiple of CL, the exceedence frequency and return period are listed. When the printing is completed, a tone sounds and Question 5 is repeated.

USER: Enters a new set of values for multiples of CL, if desired. This allows the user to conveniently search out the ranges of interest and select the most appropriate levels of incremental detail. When the desired amount of output has been obtained, the program is interrupted, and begun again at Question #1 to examine another set of conditions. The user can formally "end" the program by entering 0,0,0 in response to Question 5.

```
POINT SOURCE - RECEIVING WATER
  CONCENTRATION ANALYSIS

+++++++++++++++++++++++++++++++
INPUTS:  COEF VAR OF QS,QE,CE
         RATIO...7Q10/avgQS
         RATIO...7Q10/avgQE
         RATIO...avg CE/EL

  BACKGROUND STREAM CONC (CS)
    IS ASSUMED TO BE ZERO
+++++++++++++++++++++++++++++++
```

GENERAL DESCRIPTIVE MATERIAL

```
ENTER COEF VAR OF QS,QE,CE?
1.5,.2,.7

ENTER FOLLOWING RATIOS:
 .......7Q10/avg QS ?
.05
 .......7Q10/avg QE ?
3
 .......avg CE/ EL?
.67
ENTER LOWEST,HIGHEST,AND INCREM-
ENT OF MULT OF TARGET FOR WHICH
 % EXCEED IS DESIRED
?
ENTER LOWEST,HIGHEST,AND INCREM-
ENT OF MULT OF TARGET FOR WHICH
 % EXCEED IS DESIRED
?
2.5,3,.05
```

QUESTION #1

QUESTION #2

QUESTION #3

QUESTION #4

QUESTION #5 (CONTINUES TO REPEAT AS NEEDED)

Figure D-1 - CRT displays.

```
*********************************
   RECEIVING WATER CONC (CO)            TITLE
    PROBABILITY DISTRIBUTION
       AND RETURN PERIOD
  FOR MULTIPLES OF TARGET CONC
    DUE TO POINT SOURCE LOADS
*********************************

   COEF VAR.....QS =   1.50
   COEF VAR.....QE =   0.20
   COEF VAR.....CE =   0.70

      7Q10/avg QS  =   0.05             SUMMARY OF INPUT DATA
      7Q10/avg QE  =   3.00
      avg CE/ EL   =   0.67

 VIOLATION    PERCENT     RETURN
  MULT OF     OF TIME     PERIOD
  TARGET      EXCEEDED    (YEARS)
 ---------    --------    -------
   1.00        0.894        0.3
   2.00        0.112        2.4
   3.00        0.024       11.3
   4.00        0.007       39.4
   5.00        0.002      114.4

   2.50        0.050        5.5
   2.55        0.046        5.9
   2.60        0.043        6.4             CALCULATED RESULTS
   2.65        0.040        6.9
   2.70        0.037        7.4
   2.75        0.034        8.0
   2.80        0.032        8.6
   2.85        0.030        9.2
   2.90        0.028        9.9
   2.95        0.026       10.6
   3.00        0.024       11.3
```

Figure D-2 - Example of printed output.

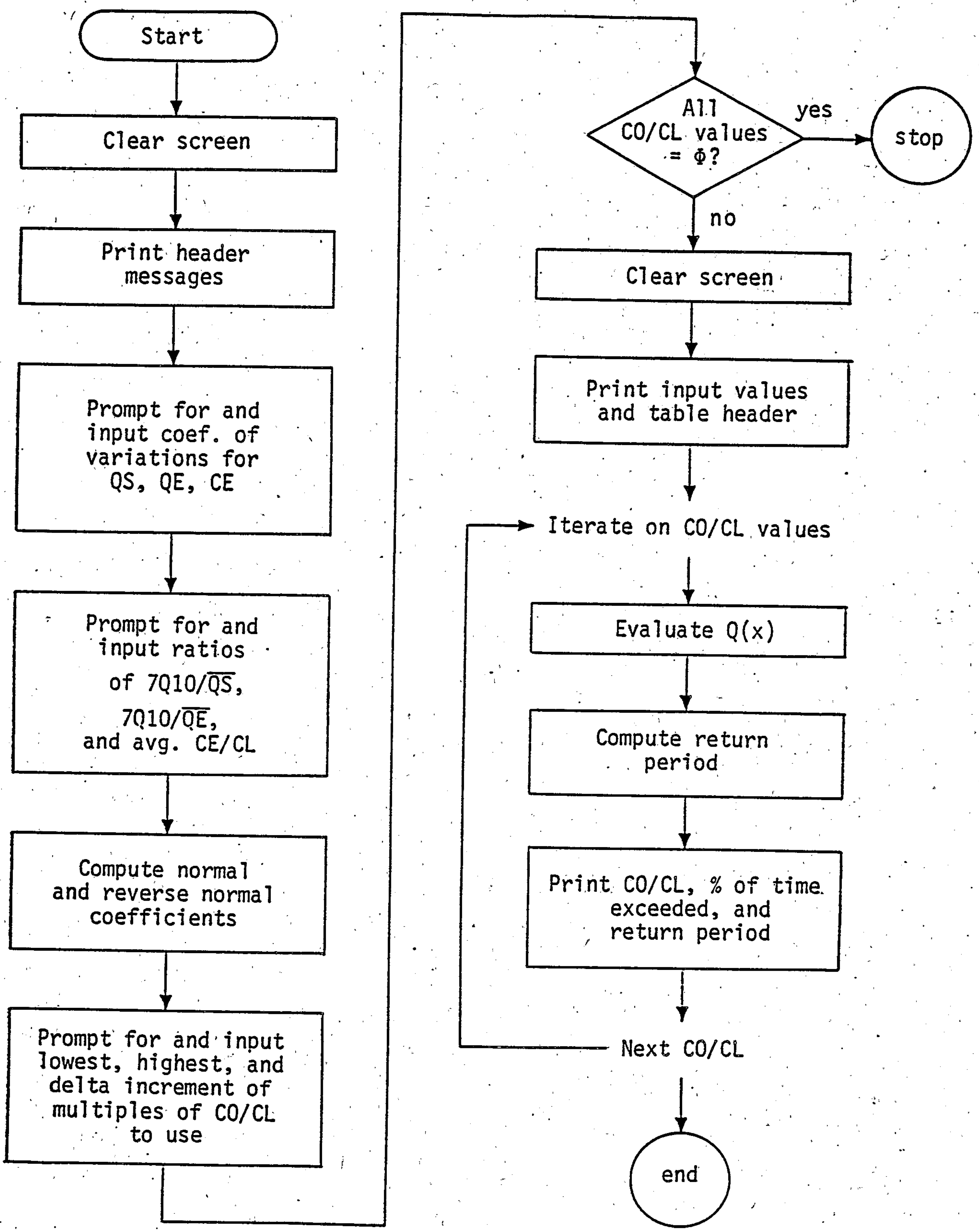


Figure D-3 - Flow chart for PDM-PS program.

```
10 ! ++++++   PDM-PS   ++++++
20 !
30 !      PROBABALISTIC
       DILUTION  MODEL
40 ! FOR POINT SOURCE DISCHARGE
50 !
60 !
70 !   DEFINITION - INPUT TERMS
80 !
90 !  QS = STREAM FLOW
100 !  QE = EFFLUENT FLOW
110 !  CE = EFFLUENT CONCENTR.
120 !
130 ! 7Q10/avgQS =       RATIO
140 !   SPECIFIED STREAM FLOWS
150 !
160 ! 7Q10/avgQE =      DESIGN
170 !  EFFLUENT DILUTION RATIO
180 !
190 ! avgCE/EL   =    RATIO OF
200 !  THE SPECIFIED AVERAGE
210 !  PLANT EFFLUENT CONCENTR.
220 !  -TO THE EFFLUENT LIMIT
230 !  (EL) CONCENTRATION.
240 !            EL IS THE EFFL
250 !  CONC THAT PRODUCES THE
260 !  STREAM TARGET CONC WHEN-
270 !     QS=7Q10 AND QE=avgQE
280 !
290 !
300 DIM R5(32),Z5(32)
310 DIM R(8),S8(8)
320 DIM P(16),Q(16)
330 PRINT "********************
    ***********"
340 PRINT "   RECEIVING WATER CO
    NC (CO)        PROBABILITY D
    ISTRIBUTION "
350 PRINT "          AND RETURN PER
    IOD"
360 PRINT " FOR MULTIPLES OF TAR
    GET CONC.    DUE TO POINT SO
    URCE LOADS"
370 PRINT "********************
    ***********"
380 DISP "POINT SOURCE - RECIEVI
    NG WATER"
390 DISP "  CONCENTRATION ANALYS
    IS"
400 DISP
410 DISP "+++++++++++++++++++++
    +++++++++"
420 DISP "INPUTS: COEF VAR OF QS
    ,QE,CE"
430 DISP "           RATIO...7Q10/a
    vgQS"
440 DISP "           RATIO...7Q10/a
    vgQE"
450 DISP "           RATIO...avg CE
    /EL "
460 DISP
470 DISP "  BACKROUND STREAM CON
    C (CS)        IS ASSUMED TO
    BE ZERO "
480 DISP "+++++++++++++++++++++
    +++++++++"
490 DISP
500 DISP "ENTER COEF VAR OF QS,Q
    E,CE";
510 INPUT V1,V2,V3
520 DISP "ENTER FOLLOWING RATIOS
    :"
530 DISP " .......7Q10/avg QS ",
540 INPUT F1
550 DISP " .......7Q10/avg QE ",
560 INPUT F2
570 DISP " ... ..avg CE/ EL";
580 INPUT F3
590 PRINT
600 IMAGE 21A,2DZ.2D
610 PRINT USING 600 ; "   COEF V
    AR.....QS = ";V1
620 PRINT USING 600 ; "   COEF V
    AR.....QE = ";V2
630 PRINT USING 600 ; "   COEF V
    AR.....CE = ";V3
640 PRINT
650 PRINT USING 600 ; "       7Q1
    0/avg QS   = ";F1
660 PRINT USING 600 ; "       7Q1
    0/avg QE   = ";F2
670 PRINT USING 600 ; "       .avg
     CE/ EL    = ",F3
680 PRINT
690 PRINT "+++++++++++++++++++++
    +++++++++++"
700 PRINT
710 PRINT "    STREAM CONCENTRAT
    ION (CO)  "
720 PRINT
730 PRINT " MULT OF ",TAB(13);"P
    ERCENT";TAB(25);"RETURN"
740 PRINT " TARGET ";TAB(13);"OF
     TIME";TAB(25);"PERIOD"
750 PRINT "(CO/CL) ";TAB(13);"EX
    CEEDED";TAB(25);"(YEARS)"
760 PRINT "--------   ---------
       --------"
770 W1=SQR(LOG(1+V1^2))
780 W2=SQR(LOG(1+V2^2))
790 W3=SQR(LOG(1+V3^2))
800 W9=SQR(W1^2+W2^2)
810 U9=LOG(F2/F1)+LOG(SQR(1+V2^2
    )/SQR(1+V1^2))
820 U3=LOG(F3*(1+F2)/SQR(1+V3^2)
    )
830 GOSUB 1230
840 DISP "ENTER LOWEST,HIGHEST,A
    ND INCREM-ENT OF MULT OF TAR
    GET  FOR WHICH % EXCEED IS D
    ESIRED"
```

Figure D-4 - PDM-PS program listing - HP-85 compatible.

```
850 INPUT B1,B2,B3
860 IF B1+B2+B3=0 THEN 1190
870 ! - LOAD QUAD. WGTS. & ROOTS
880 GOSUB 1480
890 ! - COMPUTE PORTION OF Q(X)
    ARGUMENT INDEP OF C0
900 DIM Z9(32)
910 FOR I=1 TO N0
920 ! - EVALUATE USING INV PROB
    TRANSFORMATION
930 P9=R5(I)
940 GOSUB 1380
950 Z9(I)=LOG(1+EXP(U9-W9*X9))-U
    3
960 NEXT I
970 ! - CONCENTRATION LOOP
980 FOR C0=B1 TO B2 STEP B3
990 I5=0
1000 ! --QUAD LOOP-- EVALUATE Q(
     X) = F AND SUM
1010 FOR I=1 TO N0
1020 X=(LOG(C0)+Z9(I))/W3
1030 X0=SGN(X)
1040 X=ABS(X)
1050 F=1+X*(D1+X*(D2+X*(D3+X*(D4
     +X*(D5+X*D6)))))
1060 F=.5*F^-16
1070 IF X0>0 THEN 1090
1080 F=1-F
1090 I5=I5+F*Z5(I)
1100 NEXT I
1110 ! --COMPUTE RETURN PERIOD
1120 I0=1/365/I5
1130 I5=100*I5
1140 PRINT USING 1150 ; C0,I5,I0
1150 IMAGE 2DZ.DD,5X,2DZ.3D,5X,3
     DZ.3D
1160 NEXT C0
1170 PRINT @ BEEP
1180 GOTO 840
1190 FOR L=1 TO 7
1200 PRINT
1210 NEXT L
1220 END
1230 ! -SUBROUTINE TO LOAD NORMA
     L AND REVERSE NORMAL COEFFI
     CIENTS
1240 D1=.049867347
1250 D2=.0211410061
1260 D3=.0032776263
1270 D4=.0000380036
1280 D5=.0000488906
1290 D6=.000005383
1300 ! ++++++++++++++++++++++
1310 E1=2.515517
1320 E2=.802853
1330 E3=.010328
1340 E4=1.432788
1350 E5=.189269
1360 E6=.001308
1370 RETURN
1380 ! -SUBROUTINE TO COMPUTE IN
     VERSE NORMAL TRANSF
1390 ! POLYNOMIAL APPROX TO INVE
     RSE NORMAL TABLE
1400 DEF FNC(X) = X-(E1+E2*X+E3*
     X^2)/(1+E4*X+E5*X^2+E6*X^3)
1410 S9=1
1420 IF P9<.5 THEN 1450
1430 P9=1-P9
1440 S9=-1
1450 P9=SQR(LOG(1/P9^2))
1460 X9=FNC(P9)*S9
1470 RETURN
1480 ! -QUADRATURE SUBROUTINE -
     COMPUTE ROOTS AND WEIGHTS
1490 !  I5 = INTEGRAL
1500 !  R5(N0) = N0 ROOTS (+- GA
     USSIAN ROOTS & N0/2 LAGRR R
     OOTS)
1510 !  Z5(N0) = N0 WEIGHTS
1520 !
1530 ! LOAD ROOTS AND WEIGHTS FO
     R 32nd ORDER QUADS
1540 ! FIRST THE GAUSSIAN AND TH
     EN THE LAGUERRE TERMS
1550 ! -QUAD ROOTS & WEIGHTS FOR
      16th ORDER GAUSSIAN
1560 R1=8
1570 R(1)=-.989400935
1580 R(2)=-.944575023
1590 R(3)=-.8656312024
1600 R(4)=-.7554044084
1610 R(5)=-.6178762444
1620 R(6)=-.4580167776
1630 R(7)=-.2816035508
1640 R(8)=-.09501250984
1650 S8(1)=.02715245942
1660 S8(2)=.06225352394
1670 S8(3)=.09515851168
1680 S8(4)=.1246289713
1690 S8(5)=.1495959888
1700 S8(6)=.1691565194
1710 S8(7)=.1826034154
1720 S8(8)=.1894506105
1730 N0=4*R1
1740 ! CONVERT GAUSSIAN ROOTS &
     WEIGHTS FOR (0,1) INTEGR  I
     NTVL
1750 ! AND DIVIDE BY TWO FOR COM
     POSITE FORMULA
1760 FOR K2=1 TO R1
1770 R5(K2)=.5+.5*R(K2)
1780 R5(K2+R1)=.5-.5*R(K2)
1790 Z5(K2)=S8(K2)/4
1800 Z5(K2+R1)=Z5(K2)
1810 NEXT K2
1820 ! -LOAD THE LAGUERRE ROOTS
     AND WEIGHTS, PROPERLY CONVE
     RTED
```

Figure D-4 (cont'd.)

```
1830 ! -16th ORDER LAGUERRE ROOT
     S & WEIGHTS
1840 P(1)=51.7011603395
1850 P(2)=41.9404526477
1860 P(3)=34.5833987023
1870 P(4)=28.5787297429
1880 P(5)=23.515905694
1890 P(6)=19.1801568568
1900 P(7)=15.4415273688
1910 P(8)=12.2142233689
1920 P(9)=9.43831433639
1930 P(10)=7.07033853505
1940 P(11)=5.07801861455
1950 P(12)=3.43708663389
1960 P(13)=2.1292836451
1970 P(14)=1.14105777483
1980 P(15)=.462696328915
1990 P(16)=8.76494104789E-2
2000 Q(1)=4.16146237E-22
2010 Q(2)=5.0504737E-18
2020 Q(3)=6.297967003E-15
2030 Q(4)=2.127079033E-12
2040 Q(5)=2.862350243E-10
2050 Q(6)=1.881024841E-8
2060 Q(7)=6.828319331E-7
2070 Q(8)=1.484458687E-5
2080 Q(9)=2.042719153E-4
2090 Q(10)=1.84907094353E-3
2100 Q(11)=1.12999000803E-2
2110 Q(12)=4.73289286941E-2
2120 Q(13)=.136296934296
2130 Q(14)=.265795777644
2140 Q(15)=.331057854951
2150 Q(16)=.206151714958
2160 FOR K2=1 TO N0/2
2170 R5(K2+N0/2)=EXP(-P(K2))
2180 Z5(K2+N0/2)=Q(K2)/2
2190 NEXT K2
2200 RETURN
```

Figure D-4 (cont'd.)

```
A>TYPE B:DILMOD.BAS
10 REM  +++++++    PDM-PS    ++++++++++
20 REM         PROBABALISTIC
30 REM         DILUTION  MODEL
40 REM FOR POINT SOURCE DISCHARGE
50 REM
55 REM    AUGUST, 1984
60 REM    IBM-PC AND MS-DOS COMPATIBLE VERSION
70 REM    HORIZON SYSTEMS CORPORATION
80 REM          (703) 471-0480
85 REM
90 REM  ++++++++++++++++++++++++++++++
300 DIM R5#(32),Z5#(32)
310 DIM R#(8),S8#(8)
320 DIM P#(16),Q#(16),Z9#(32)
321 CLS
322 KEY OFF
330 PRINT "*********************************************************"
340 PRINT " RECEIVING WATER CONC (CO) PROBABILITY DISTRIBUTION "
350 PRINT "                    AND RETURN PERIOD"
360 PRINT "              FOR MULTIPLES OF TARGET CONC"
370 PRINT "               DUE TO POINT SOURCE LOADS"
380 PRINT "*********************************************************"
390 PRINT "POINT SOURCE - RECEIVING WATER"
400 PRINT "CONCENTRATION ANALYSIS"
410 PRINT
420 PRINT "+++++++++++++++++++++++++++++++++++++++++++++++++++++++++"
430 PRINT "INPUT COEF OF VAR OF QS,QE,CE"
440 PRINT "        RATIO...7Q10/AVGQS"
450 PRINT "        RATIO...7Q10/AVGQE"
460 PRINT "        RATIO...AVG CE/CL"
470 PRINT " BACKGROUND STREAM CONC (CS) IS ASSUMED TO BE ZERO"
480 PRINT "+++++++++++++++++++++++++++++++++++++++++++++++++++++++++"
490 PRINT
500 PRINT "ENTER COEF OF VAR OF QS,QE,CE"
510 INPUT V1,V2,V3
520 PRINT "ENTER THE FOLLOWING RATIOS:"
530 INPUT " .......7Q10/AVG QS ";F1
540 INPUT " .......7Q10/AVG QE ";F2
550 INPUT " .........AVG CE/EL ";F3
560 PRINT
565 CLS
570 PRINT " COEF OF VAR.....QS = ";V1
580 PRINT " COEF OF VAR.....QE = ";V2
581 PRINT " COEF OF VAR.....CE = ";V3
590 PRINT
600 PRINT "         7Q10/AVG QS = ";F1
610 PRINT "         7Q10/AVG QE = ";F2
620 PRINT "           AVG CE/EL = ";F3
630 PRINT
640 PRINT "+++++++++++++++++++++++++++++++++++++++++++++++++++++++++"
```

Figure D-5 - PDM-PS program listing - IBM-PC and MS-DOS compatible.

```
720 W1=SQR(LOG(1+V1^2))
730 W2=SQR(LOG(1+V2^2))
740 W3=SQR(LOG(1+V3^2))
750 W9=SQR(W1^2+W2^2)
760 U9=LOG(F2/F1)+LOG(SQR(1+V2^2)/SQR(1+V1^2))
770 U3=LOG(F3*(1+F2)/SQR(1+V3^2))
780 GOSUB 1160
790 PRINT "ENTER LOWEST, HIGHEST, AND INCREMENT OF MULT OF TARGET FOR"
795 INPUT "WHICH % EXCEED IS DESIRED";B1,B2,B3
796 IF B1+B2+B3=0 THEN GOTO 1120
797 CLS
803 PRINT " COEF OF VAR.....QS = ";V1
804 PRINT " COEF OF VAR.....QE = ";V2
805 PRINT " COEF OF VAR.....CE = ";V3
806 PRINT
807 PRINT "        7Q10/AVG QS = ";F1
808 PRINT "        7Q10/AVG QE = ";F2
809 PRINT "          AVG CE/EL = ";F3
810 PRINT
811 PRINT "++++++++++++++++++++++++++++++++++++++++++++++++++++++"
812 PRINT
813 PRINT "     STREAM CONC (CO)"
814 PRINT
815 PRINT " MULT OF";TAB(13);"PERCENT";TAB(25);"RETURN"
816 PRINT " TARGET ";TAB(13);"OF TIME";TAB(25);"PERIOD"
817 PRINT "(CO/CL) ";TAB(13);"EXCEEDED";TAB(25);"(YEARS)"
818 PRINT "--------";TAB(13);"--------";TAB(25);"-------"
820 REM - LOAD QUAD. WGTS & ROOTS
830 GOSUB 1410
840 REM COMPUT PORTION OF Q(X) ARGUMENT INDEP OF CO
850 FOR I=1 TO NO
860 REM - EVALUATE USING INV PROB TRANSFORMATION
870 P9#=R5#(I)
880 GOSUB 1310
890 Z9#(I)=LOG(1+EXP(U9-W9*X9))-U3
900 NEXT I
910 REM - CONC LOOP
920 FOR CO=B1 TO B2 STEP B3
930 I5=0
940 REM - QUAD LOOP - EVALUATE Q(X) = F AND SUM
950 FOR I=1 TO NO
960 X=(LOG(CO)+Z9#(I))/W3
970 XO=SGN(X)
980 X=ABS(X)
990 F=1+X*(D1+X*(D2+X*(D3+X*(D4+X*(D5+X*D6)))))
1000 F=.5*F^(-16)
1010 IF XO>0 THEN GOTO 1030
1020 F=1-F
1030 I5=I5+F*Z5#(I)
1040 NEXT I
1050 REM COMPUTE RETURN PERIOD
1060 IO=1/365/I5
```

Figure D-5 (cont'd.)

```
1070 I5=100*I5
1080 PRINT USING "###.###      ";CO,I5,IO
1090 NEXT CO
1100 PRINT CHR$(7)
1101 INPUT "ENTER <CR> TO CONTINUE, OR 'STOP' ";A$
1102 IF A$<>"STOP" THEN GOTO 560
1110 REM GOTO 790
1120 FOR L=1 TO 7
1130 PRINT
1140 NEXT L
1145 KEY ON
1150 END
1160 REM SUBROUTINE TO LOAD NORMAL AND REVERSE NORMAL COEFFICIENTS
1170 D1=.049867347#
1180 D2=.0211410061#
1190 D3=.0032776263#
1200 D4=3.80036E-05
1210 D5=4.88906E-05
1220 D6=5.383E-06
1230 REM ++++++++++++++
1240 E1=2.515517
1250 E2=.802853
1260 E3=.010328
1270 E4=1.432788
1280 E5=.189269
1290 E6=.001308
1300 RETURN
1310 REM SUBROUTINE TO COMPUTE INVERSE NORMAL TRANSFORMATION
1320 REM POLYNOMIAL APPROX TO INVERSE TABLE
1330 DEF FNC(X#)= X#-(E1+E2*X#+E3*X#^2)/(1+E4*X#+E5*X#^2+E6*X#^3)
1340 S9=1
1349 IF P9#<1E-18 THEN P9#=1E-18
1350 IF P9#<.5 THEN GOTO 1380
1360 P9#=1-P9#
1370 S9=-1
1380 P9#=SQR(LOG(P9#^-2))
1390 X9=FNC(P9#)*S9
1400 RETURN
1410 REM QUADRATURE SUBROUTINE - COMPUTE ROOTS AND WEIGHTS
1420 REM I5=INTEGRAL
1430 REM R5(NO)= NO ROOTS
1440 REM Z5(NO)= NO WEIGHTS
1450 REM LOAD ROOTS AND WEIGHTS FOR 32ND ORDER QUADS
1460 REM FIRST THE GAUSSIAN, THEN THE LAGUERRE TERMS
1470 REM QUAD ROOTS & WEIGHTS FOR 16TH ORDER GAUSSIAN
1480 R1=8
1490 R#(1)=-.989400935#
1500 R#(2)=-.944575023#
1510 R#(3)=-.8656312024#
1520 R#(4)=-.7554044084#
1530 R#(5)=-.6178762444#
1540 R#(6)=-.4580167776#
1550 R#(7)=-.2816035508#
```

Figure D-5 (cont'd.)

```
1560 R#(8)=-.09501250984#
1570 S8#(1)=.02715245942#
1580 S8#(2)=.06225352394#
1590 S8#(3)=.09515851168#
1600 S8#(4)=.1246289713#
1610 S8#(5)=.1495959888#
1620 S8#(6)=.1691565194#
1630 S8#(7)=.1826034154#
1640 S8#(8)=.1894506105#
1650 NO=4*R1
1660 REM CONVERT GAUSSIAN ROOTS & WEIGHTS FOR (0,1) INTEGR. INTERVAL
1670 REM AND DIVIDE BY TWO FOR COMPOSITE FORMULA
1680 FOR K2=1 TO R1
1690 R5#(K2)=.5+.5*R#(K2)
1700 R5#(K2+R1)=.5-.5*R#(K2)
1710 Z5#(K2)=S8#(K2)/4
1720 Z5#(K2+R1)=Z5#(K2)
1730 NEXT K2
1740 REM LOAD THE LAGUERRE ROOTS AND WEIGHTS, PROPERLY CONVERTED
1750 REM 16TH ORDER LAGUERRE ROOTS AND WEIGHTS
1760 P#(1)=51.7011603395#
1770 P#(2)=41.9404526477#
1780 P#(3)=34.5833987023#
1790 P#(4)=28.5787297429#
1800 P#(5)=23.5159056940#
1810 P#(6)=19.1801568568#
1820 P#(7)=15.4415273688#
1830 P#(8)=12.2142233689#
1840 P#(9)=9.43831433639#
1850 P#(10)=7.07033853505#
1860 P#(11)=5.07801861455#
1870 P#(12)=3.43708663389#
1880 P#(13)=2.1292836451#
1890 P#(14)=1.14105777483#
1900 P#(15)=.462696328915#
1910 P#(16)=.0876494104789#
1920 Q#(1)=4.16146237D-22
1930 Q#(2)=5.0504737D-18
1940 Q#(3)=6.297967003D-15
1950 Q#(4)=2.127079033D-12
1960 Q#(5)=2.862350243D-10
1970 Q#(6)=1.881024841D-08
1980 Q#(7)=.000000682819331#
1990 Q#(8)=.00001484458687#
2000 Q#(9)=.0002042719153#
2010 Q#(10)=.00184907094353#
2020 Q#(11)=.0112999000803#
2030 Q#(12)=.0473289286941#
2040 Q#(13)=.136296934296#
2050 Q#(14)=.265795777644#
2060 Q#(15)=.331057854951#
2070 Q#(16)=.206151714958#
```

Figure D-5 (cont'd.)

```
2080 FOR K2=1 TO NO/2
2090 R5#(K2+NO/2)=EXP(-P#(K2))
2100 Z5#(K2+NO/2)=Q#(K2)/2
2110 NEXT K2
2120 RETURN

A>
```

```
        ***************************************************
         RECEIVING WATER CONC (CO) PROBABILITY DISTRIBUTION
                      AND RETURN PERIOD
                  FOR MULTIPLES OF TARGET CONC
                   DUE TO POINT SOURCE LOADS
        ***************************************************
        POINT SOURCE - RECEIVING WATER
        CONCENTRATION ANALYSIS

        +++++++++++++++++++++++++++++++++++++++++++++++++++
        INPUT COEF OF VAR OF QS,QE,CE
               RATIO...7Q10/AVGQS
               RATIO...7Q10/AVGQE
               RATIO...AVG CE/CL
         BACKGROUND STREAM CONC (CS) IS ASSUMED TO BE ZERO
        +++++++++++++++++++++++++++++++++++++++++++++++++++

        ENTER COEF OF VAR OF QS,QE,CE
        ? 1.5,.2,.7
        ENTER THE FOLLOWING RATIOS:
         .......7Q10/AVG QS ? .05
         .......7Q10/AVG QE ? 3.0
         .........AVG CE/EL ? .67


        COEF OF VAR.....QS =  1.5
        COEF OF VAR.....QE =  .2
        COEF OF VAR.....CE =  .7

               7Q10/AVG QS =  .05
               7Q10/AVG QE =  3
                 AVG CE/EL =  .67

        +++++++++++++++++++++++++++++++++++++++++++++++++++
        ENTER LOWEST, HIGHEST, AND INCREMENT OF MULT OF TARGET FOR
        WHICH % EXCEED IS DESIRED? 1,5,1


        COEF OF VAR.....QS =  1.5
        COEF OF VAR.....QE =  .2
        COEF OF VAR.....CE =  .7

               7Q10/AVG QS =  .05
               7Q10/AVG QE =  3
                 AVG CE/EL =  .67
```

Figure D-5 (cont'd.)

++

STREAM CONC (CO)

MULT OF TARGET (CO/CL)	PERCENT OF TIME EXCEEDED	RETURN PERIOD (YEARS)
1.000	0.894	0.306
2.000	0.112	2.443
3.000	0.024	11.313
4.000	0.007	39.429
5.000	0.002	114.356

ENTER <CR> TO CONTINUE, OR 'STOP' ?

```
COEF OF VAR.....QS =  1.5
COEF OF VAR.....QE =  .2
COEF OF VAR.....CE =  .7

      7Q10/AVG QS =  .05
      7Q10/AVG QE =  3
        AVG CE/EL =  .67
```

++

ENTER LOWEST, HIGHEST, AND INCREMENT OF MULT OF TARGET FOR WHICH % EXCEED IS DESIRED? 2.5,3,.1

```
COEF OF VAR.....QS =  1.5
COEF OF VAR.....QE =  .2
COEF OF VAR.....CE =  .7

      7Q10/AVG QS =  .05
      7Q10/AVG QE =  3
        AVG CE/EL =  .67
```

++

STREAM CONC (CO)

MULT OF TARGET (CO/CL)	PERCENT OF TIME EXCEEDED	RETURN PERIOD (YEARS)
2.500	0.050	5.501
2.600	0.043	6.395
2.700	0.037	7.410
2.800	0.032	8.558
2.900	0.028	9.854
3.000	0.024	11.313

ENTER <CR> TO CONTINUE, OR 'STOP' ? STOP

Figure D-5 (cont'd.)