Technical Report

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STATISTICAL ANALYSIS OF WATER QUALITY EXCURSIONS

by

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Variable	Definition
A	Drainage basin area, in acres
C	Upstream concentration
CV	Coefficient of variation
d	Duration of rainfall
D	Ratio of upstream flow to the discharge flow
e	Discharge concentration
f	Fraction of time runoff occurs
F	Upstream flow
I	Rainfall intensity (in/hr)
n	Number of values
N_h	Number of hours per year for which excursions occur due to non-point source discharges
N_p	Number of days per year for which excursions occur due to point source discharges
N_s	Number of storms per year for which excursions occur due to non-point source discharges
P	Percentile
Pr()	Probability of indicated condition
q	Discharge flow
R	Runoff coefficient
S	Water quality standard
S()	Sample standard deviation of associated variable
T	Downstream concentration
T_r	Downstream concentration only during periods when removal occurs
V	Volume of runoff
Z_x	Alpha percentile of a standard normal variable
α,β,γ	Parameters for distribution
δ	Time between midpoints of successive storms
Δ	Incremental excursion frequency
$\mu()$	Arithmetic mean of associated variable
$\mu(\ln)$	Logarithmic mean of associated variable
$\rho()$	Correlation between associated variable
$\sigma()$	Arithmetic standard deviation of associated variable
$\sigma^2()$	Arithmetic variance of associated variable
$\sigma^2(\ln)$	Logarithmic variance of associated variable
Φ	Discharge flow ratio
7Q10	7-day, 10-year low flow

PREFACE

The work was supported by the Massachusetts Division of Water Pollution Control, Research and Demonstration Program, Contract Number 87-01-01. The authors would like to acknowledge the contributions of Warren Kimball of the Division of Water Pollution Control.

This report presents results of research performed by Lawrence R. Soucie, a Research Assistant under the direction of James W. Male. The findings presented in this report are summarized from Mr. Soucie's M.S. Project Report (Soucie, 1989).

I. INTRODUCTION

Overview

As outlined in the Clean Water Act, the ultimate objectives of water pollution control policies are to: "restore and maintain the chemical, physical, and biological integrity of the Nation's waters." The principal means of achieving these goals is through the use of water quality standards.

Water quality standards are numerical or narrative criteria which the water body must attain to support its designated use (EPA, 1983a). To ensure that water quality standards are met, the Clean Water Act established the National Pollutant Discharge Elimination System (NPDES), which required all dischargers to obtain a permit. The effluent limits specified in the permit can be of two types: technology based or water quality based.

Technology based permits require minimum levels of treatment for different types of discharges. For example, municipal wastewater treatment plants (MWTP) are required to practice secondary treatment as a minimum. This level of treatment is defined as processes which produce an effluent with BOD and TSS concentrations of 30 mg/l or less, or which result in 85 % removal (whichever results in the cleaner water). In many situations, particularly when there are few discharges and when the upstream water quality is good, technology based permits are adequate to maintain water quality standards.

However, in situations where the minimum treatment levels specified by technology based permits fail to meet the required water quality standard in the receiving water, more stringent water quality based permits are required. Water quality based permits use what is known as a "wasteload allocation" to set the level of treatment necessary to meet the water quality standard.

Wasteload allocations are calculated using design conditions which are thought to provide a reasonable level of safety. The stream design flow most commonly used in wasteload allocations is the seven-day, ten-year (7Q10) low flow (Lamb and Hull, 1985). The 7Q10 is defined as the lowest 7-day average flow which would occur, on average, once every 10 years. It is determined by a statistical analysis of streamflow data from USGS gaging stations; or, at ungaged sites, it can be estimated from basin characteristics (Male and Ogawa, 1982).

Based on years of experience using the 7Q10 low flow for regulating point source discharges, the Massachusetts Division of Water Pollution Control (MDWPC) has determined that it provides a high level of protection for designated uses of the receiving waters (Haas and Kimball, 1988). Therefore, the MDWPC hopes to adopt the same level of protection afforded by the 7Q10 in developing standards for combined sewer overflows (Haas and Kimball, 1988). The difficulty in accomplishing this task lies in quantifying the protection given by point source wasteload allocation procedures and then transferring that level of protection to standards for non-point sources of pollution.

One way of relating the effect of point source standards to those for non-point sources is by quantifying the excursion frequency of pollutants in the receiving water body. An excursion is simply an instance when the concentration of a pollutant is above (or below in the case of dissolved oxygen) the concentration specified by the water quality standard. An excursion may result from a violation, from a case when the streamflow is below the 7Q10 (not a violation), or a combination of the two.

Research has shown that the 7Q10 is a fairly conservative flow which is exceeded on the order of 99% of the time (Ray and Walker, 1968; Male and Ogawa, 1982). Therefore, if violations of water quality standards depended only on streamflow, the excursion frequency would be about one percent. However, since variations in background concentrations and effluent loading will also affect the excursion frequency, the actual excursion frequency may be more or less.

Objectives

The goal of this research is to study the statistical nature of water quality excursions. In particular, the effect of water quality standards for point source discharges are analyzed to provide the basis for development of standards for non-point sources of pollution. This overall objective can be broken into three sub-objectives:

1. To assess the effect of current regulations on point source discharges, in terms of water quality excursions.

2. To assess the statistical nature of water quality and related excursions due to non-point sources of pollution.

3. To compare the excursion frequencies of both types of discharges with the ultimate intent of developing regulations for non-point source discharges that have an impact on the receiving stream similar, in a statistical sense, to those of point source discharges.

II. BACKGROUND

Previous research efforts concerning wasteload allocations, methods of statistical analysis, and non-point source pollution are relevant to the current study. These topics are reviewed below.

Wasteload Allocation Procedures

Conventional wasteload allocation procedures are based, in part, on the mass balance equation:

$$T = \frac{FC + qe}{F + q} \tag{2-1}$$

where:

F = upstream flow

C = upstream concentration

q = discharge flow

e = discharge concentration

T = downstream concentration

This equation assumes complete mixing at the point of discharge and a conservative pollutant.

In a steady state wasteload allocation, a critical low flow is used for the upstream flow (F) and average values are used for upstream concentration (C) and discharge flow (q). Equation 2-1 is then solved to find the discharge concentration (e) so that the downstream concentration (T) just meets the water quality standard. As long as the streamflow is greater than the critical low flow, the standard will be met. When the streamflow is below the critical low flow, instream pollutant concentrations are likely to be below the standard, resulting in an excursion.

The use of low flows in wasteload allocation procedures results from the recognition that the most severe impacts often occur during periods of low flow. A majority of states have adopted the 7Q10 low flow as their critical stream flow (Biswas, 1983). The 7Q10 is relatively conservative criterion since it is exceeded, on a daily basis, around 99 percent of time (Ray and Walker, 1968, and Male and Ogawa, 1982).

Other low flows have been proposed for use as the critical low flow, including the 1Q10 and the 30Q2. The Environmental Protection Agency has studied various low flow criteria (1985, 1986a, 1986b) and recommended a two tier approach, based on acute and chronic effects. In addition, Biswas and Bell (1984) and McKeown (1984) have studied various aspects of different low flows.

Because the 7Q10 streamflow is exceeded a large percentage of the time, it has been thought that pollutant concentrations are frequently well below the water quality standard. This belief has led researchers to propose variable discharge permits based on streamflow and/or season (Hendrick 1983, Noss and Gladstone, 1987, Stein et al., 1985).

Sykes (1984) proposed a method that uses the cumulative density functions of streamflow and effluent loading to derive the probability of a violation in a water quality standard. The proposed method determines the probability of one or more violations during periods of drought.

Great Britain used to use a procedure for wasteload allocations based on the 95th percentile low flow and average values for discharge flow and upstream water quality (Crabtree et al., 1986). However, in Great Britain, Warn and Brew (1980) and Warn (1982) realized that the substitution of statistics, such as averages or percentiles, in the mass balance equation does not result in corresponding averages or percentiles for the calculated downstream concentration, but in an unknown statistic. The problem was solved in Great Britain by implementing procedures which account for the variability of all the variables in the mass balance equation (F, C, q, e). These procedures, as well as procedures which have been developed in the United States, will be discussed further.

Probability Distributions

To incorporate the variability of all of the input parameters to the mass balance equation (eq. 2-1), characteristics of the parameters in terms of their probability distributions must be known. Table 2-1 summarizes the characteristics of several distributions that are currently in use in water quality modeling. Crabtree et al. (1986) note that probability

Table 2-1: Characteristics of Some Common Probability Distributions

Distribution	Number of Parameters	General Comments
Normal	2	Symmetrical distribution Plots as straight line on normal probability paper
2-Parameter Lognormal	2	Skewed distribution (fixed) Plots as straight line on log probability paper
3-Parameter Lognormal	3	Skewed distribution (variable)
Uniform	2	Symmetrical distribution
Pearson Type III	3	Skewed distribution (variable)
Log-Pearson Type III	3	Skewed distribution (variable)
2-Parameter Gamma	2	Skewed distribution (fixed) Special case of Pearson Type III
Exponential	1	Special case of Gamma distribution

distributions of flow and concentration typically have the following characteristics: (1) a fixed non-negative lower bound, (2) a positive skew, and (3) an extended tail of higher magnitude outlier values. They evaluated the goodness of fit of 212 water quality data sets (BOD_5, SS, NH_3-N, and NO_3-N) to normal, lognormal, and Pearson type III distributions using the Kolmogorov-Smirnoff statistic at the 95% significance level. The authors found that 13 of the data sets were best fit to the normal distribution, 48 to the lognormal, and 62 to the Pearson type III, but concluded that since 89 of the data sets could not be fit to a parametric distribution, it is incorrect to assume that any data set follows a particular distribution without first analyzing the data statistically.

Irrespective of the conclusions cited in the previous paragraph, the lognormal distribution frequently fits data representing natural processes and has been used numerous times (eg., Driscoll 1986), and is often accepted as a valid representation of the stochastic nature of natural events. When addressing the pertinent characteristics of pollutant discharge to receiving waters, the lognormal distribution has been widely accepted. Examples include: the National Urban Runoff Program (NURP), EPA (1983b) which analyzed the pollutant concentrations in urban runoff; analysis of data sets from highway stormwater runoff, combined sewer overflows, urban runoff, point source discharges from publically owned treatment works (POTW), and agricultural runoff (Driscoll 1986); daily BOD and TSS concentrations from activated sludge treatment plants (Niku et al., 1979, 1982) and treated effluent (EPA, 1985).

The lognormal distribution results when the logarithm of the real parameter values are used, and fit a normal distribution. (In the following discussion, capital letters denote random variables and small case letters denote specific values that the random variable may assume. Also, for the sake of clarity, the random variable X and all variables subscripted with x shall be used to represent real variables, while the random variable Y shall be used to denote log-transformed variables).

Chow (1954) shows that if a random variable X is made up of the sum of many small effects (x1,x2,..xn), then according to the Central Limit Theorem, X would be normally distributed. Likewise, if a random variable X is made up of the product of many small effects, then the log of X would be normally distributed. Since it is reasonable to assume that the processes which cause change in some hydrologic variables act multiplicatively, the logarithms of these processes would act additively:

$$\ln(x1*x2*..xn) = \ln(x1) + \ln(x2) + ..\ln(xn)$$

Letting Y = ln(X), the distribution of Y is normally distributed:

$$\Pr(y) = \left(2\pi\sigma^2(y)\right)^{-.5}\exp\left[-.5\left(y-\mu(y)\right)^2/\sigma^2(y)\right] \qquad (2\text{-}2)$$

where:

$\sigma^2(y)$ = the variance of logarithms of X, and

$\mu(y)$ = the mean of logarithms of X

The parameters $\mu(y)$ and $\sigma^2(y)$ are the mean and variance of the population of Y. Since these parameters can never be known, they must be estimated from the sample data. Two methods are commonly used for estimating the parameters of a probability distribution: method of moments and maximum likelihood.

The method of moments estimates for the 2-parameter lognormal distribution are simply the arithmetic average and variance of the log-transformed data. An alternative procedure was developed by Chow (1954), which allows calculation of the parameters $\mu(y)$ and $\sigma^2(y)$ without taking the logarithms of all the data.

$$\mu(y) = .5(\ln(\mu^2(x) \ / \ (CV^2 + 1))) \tag{2-3}$$

$$\sigma^2(y) = \ln(CV^2 + 1) \tag{2-4}$$

$$CV = S(x) / \mu(x) \tag{2-5}$$

where:

CV = the coefficient of variation,

$\sigma(x)$ = the standard deviation of un-logged data, and

S(x) = the arithmetic mean of un-logged data

The method of moment estimates for the parameters of the 2-parameter lognormal distribution are conceptually easy to understand, and easy to apply. This is because the tails have long moment arms and therefore have a disproportionate effect on the estimates of the parameters. This problem is minimized by the maximum likelihood method.

Matalas and Wallis (1973) note that it is generally accepted that maximum likelihood estimates are more efficient than moment estimates, and therefore should be used whenever possible. The basic principle behind maximum likelihood estimates is that the values of the parameters maximize the likelihood that a random sample was obtained from the population. For the 2-parameter lognormal distribution, Kendal and Stuart (1963) show that:

$$\mu(y) = \Sigma_{1}^{n} \left(\frac{y}{n}\right) = \text{mean} \tag{2-6}$$

$$\sigma^2(y) = \frac{(n-1)S^2(y)}{n} \tag{2-7}$$

where:

Sy = standard deviation of the log transformed data set.

Stochastic Methods

There are essentially three ways of incorporating the variability of all the variables in the mass balance equation: continuous simulation, Monte Carlo simulation, and a lognormal probabilistic model (LimnoTech, 1985).

Continuous simulation combines daily time series data for the four variables in the mass balance equation to determine the downstream concentration. The downstream concentrations calculated with continuous simulation have the same time sequence as the input variables. A probability plot can be constructed from the calculated downstream concentrations, and this in turn can be used to determine the excursion frequency. An advantage of continuous simulation is that any averaging period may be used (LimnoTech, 1985). However, a major disadvantage is that extensive time series data are required for all the variables, and these data are seldom available (Freedman et al., 1988). In addition, continuous simulation will reproduce only those conditions which have historically occurred, and not the entire range of possible conditions.

Monte Carlo simulation is a stochastic modeling technique in which random values are repetitively drawn from the distributions of model inputs, and then combined in the model to determine a probability distribution for the model output. With Monte Carlo simulation, a wide variety of probability distributions can be used, and depending on the distributions, correlations between variables can be incorporated.

Figure 2-1 shows a schematic of Monte Carlo simulation as it applies to the mass balance equation. Random values are drawn from probability distributions of F, C, q, and e, and then combined in the mass balance equation to determine the downstream concentration. The procedure is

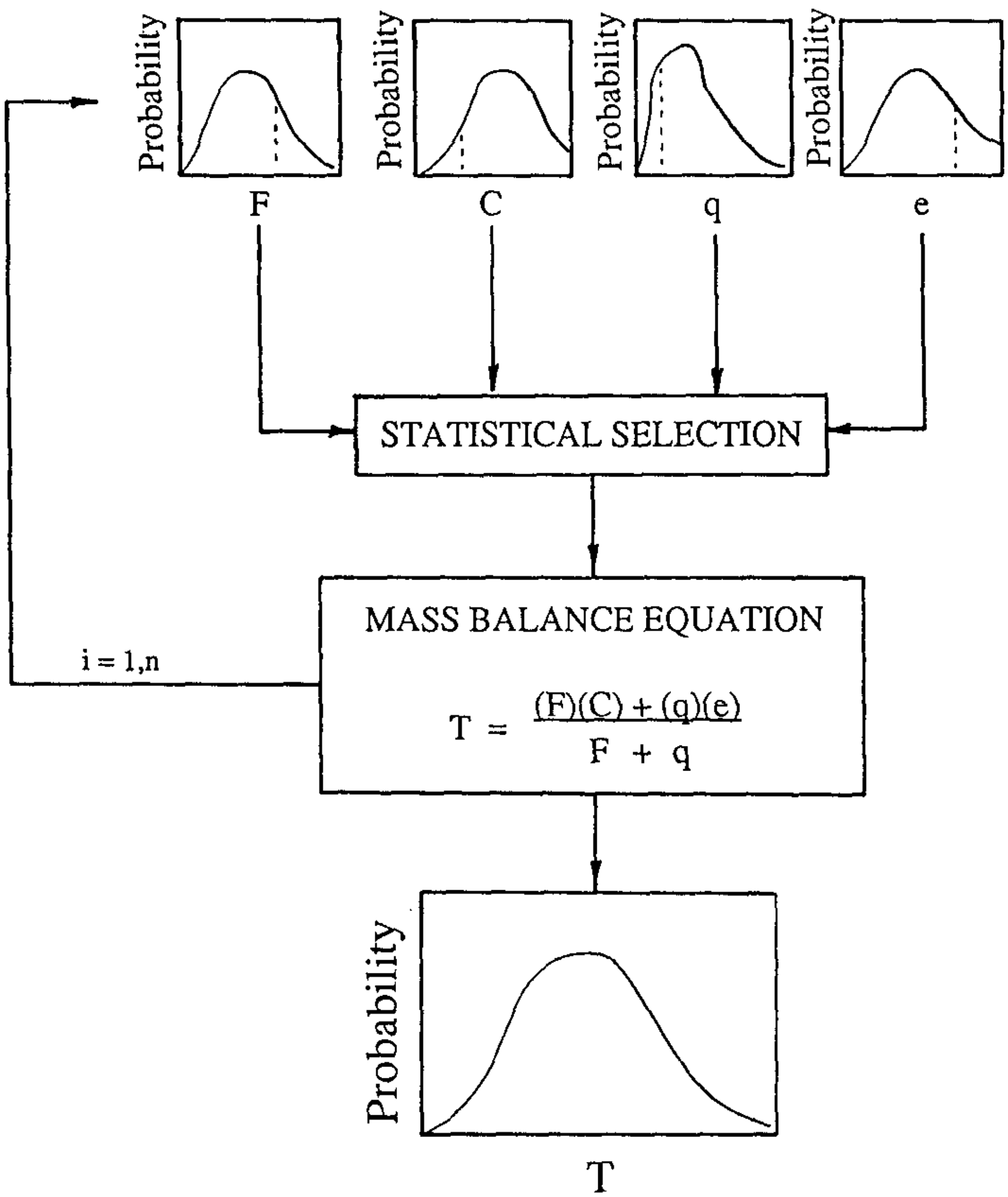


Figure 2-1. Schematic Diagram of the Monte Carlo Simulation Process

repeated until sufficient values are available to determine the probability distribution of the downstream concentration.

Monte Carlo simulation techniques have been used by Freedman et al. (1988) to determine seasonal discharge permits, and by Marr and Canale (1988) to determine wasteload allocations for toxics which have a specified excursion frequency. In Great Britain, the procedure used for wasteload allocations iteratively adjusts effluent concentrations in successive simulation runs until the 95th percentile of the downstream concentration is equal to the water quality standard.

If all the input variables in the mass balance equation (F, C, q and e) are lognormally distributed, then the probability distribution of downstream concentrations (T) can be calculated directly from the distributions of the input variables without resorting to Monte Carlo simulation (Warn and Brew, 1980; DiToro, 1984). There are three methods of accomplishing this: an approximation which uses a numerical integration (Warn and Brew, 1980), an approximation which uses simultaneous equations (DiToro, 1984), and an exact procedure in which the probability distribution of downstream concentrations is evaluated as a multiple integral of the joint probability of the variables in the mass balance equation (DiToro, 1984). The following discussion is based on procedures developed by Warn and Brew (1980) and DiToro (1984). In the discussion, the notation μ and σ denote the mean and standard standard deviation of the associated variable, and ln signifies the logarithm of the variable.

If the discharge flow fraction (Φ) is defined as the ratio of the discharge flow to the total flow:

$$\Phi = \frac{q}{F + q} \tag{2-8}$$

Then the mass balance equation (equation 2-1) can be re-written as:

$$T = e\Phi + C(1 - \Phi) \tag{2-9}$$

Warn and Brew (1980) show that T is approximately lognormal and that the arithmetic mean and variance of T are given by:

$$\mu(T) = \mu(e)\mu(\Phi) + \mu(C)[1 - \mu(\Phi)] \tag{2-10}$$

$$\sigma^2(T) = \sigma^2(\Phi)[\mu(e) - \mu(c)]^2 + \sigma^2(e)[\sigma^2(\Phi) - \mu^2(\Phi)]$$
$$+ \sigma^2(C)[\sigma^2(\Phi) + (1 - \mu(\Phi))^2] \tag{2-11}$$

The arithmetic mean μ, and variance σ^2, can be converted to their lognormal equivalents $\mu(\ln x)$ and $\sigma^2(\ln x)$ by equations 2-3 to 2-5. Therefore, the percentiles of T are:

$$T_\alpha = \exp(\mu(\ln T) + z_\alpha \sigma(\ln T)) \tag{2-12}$$

where:

z_α = the α percentile of a standard normal random variable.

Warn and Brew (1980) assume that Φ is lognormally distributed, and use a numerical procedure to calculate the moments. DiToro (1984) avoids the integrations by solving two simultaneous equations to obtain the following equations for the moments of Φ:

$$\mu(\ln\Phi) = .5(\ln\Phi_\alpha + \ln\Phi_{1-\alpha}) \tag{2-13}$$

$$\sigma(\ln\Phi) = \frac{1}{2z_\alpha}\left(\ln\Phi_\alpha - .5\ln\Phi_{1-\alpha}\right) \tag{2-14}$$

where:

$$\Phi_\alpha = \frac{1}{1+\exp[\mu(\ln D) - z_\alpha \sigma(\ln D)]} \tag{2-15}$$

$$\mu(\ln D) = \mu(\ln F) - \mu(\ln q) \tag{2-16}$$

$$\sigma^2(\ln D) = \sigma^2(\ln F) + \sigma^2(\ln q) - 2\sigma(\ln F)\sigma(\ln q)\rho(Fq) \tag{2-17}$$

$\rho(Fq)$ = the cross correlation between F and q.

DiToro refers to this procedure as the "moment approximation method."

Both of these procedures are approximations because they assume that Φ is lognormally distributed, which is only true when F (upstream flow) is large relative to q (effluent flow). DiToro (1984) removes the need for this assumption by integrating over the joint probability distribution of F, C, q, and e. However, the method is very complex and requires advanced numerical procedures to carry out the required integrations.

Continuous simulation, Monte Carlo simulation, and the lognormal probability model can readily be applied to determine the excursion frequency due to point source discharges. However, their application to non-point source discharges is more complex and is discussed subsequently.

Non-Point Sources

Analysis of non-point source pollution is more complicated than that for point sources because there are periods when the discharge is zero (i.e., when it is not raining). Also, because the quantity and quality of pollutant concentrations depend on both climatic and basin factors their values are more variable and less predictable.

The driving force behind urban runoff is rainfall. The rainfall process can have variability within events, in that rain may be heavy during part of a storm and light during the rest. In addition, there is variability between events since storms do not occur on a regular basis. Variation between events can be described by three parameters: (1) the time between the midpoints of successive storms (δ), (2) the duration of the storm (d), and (3) the runoff volume (V). The EPA (1979) analyzed long term rainfall records from 11 cities and found that, with the proper definition of when a storm begins and ends, δ is exponentially distributed.

DiToro (1980) studied within-event variability by comparing the results of an analytical solution, which did not consider within-event variations, with an hourly simulation which did. He concluded that within-event variation is not significant if the dispersion (the extent to which a pulse of a pollutant discharged into a receiving water spreads out as it travels downstream) is larger. Since hourly rainfall records are extensive, a good estimate of runoff variability can be made by transforming the hourly rainfall records to runoff. The rational formula is the simplest method of doing this:

$$q = RIA \tag{2-18}$$

where:

q = runoff (CFS),

R = runoff coefficient,

I = rainfall intensity (in/hr), and

A = area (acres).

A straight application of the rational formula will result in q having the same probability distribution as I, since multiplying by a constant will not change the underlying distribution. Metcalf and Eddy (1983) analyzed rainfall intensities from 546 storms in Vermont and concluded that the underlying distribution was approximately exponential. However, very small rainfall intensities do not contribute to runoff, due to infiltration and storage. Metcalf and Eddy (1983) suggest that after accounting for infiltration and storage, the distribution of runoff flow would shift to a lognormal shape. In addition, since many rainfall events do not contribute to runoff, the EPA (1983b) concluded, based on the research of Goforth et al. (1983), that the coefficient of variation for rainfall intensity should be reduced by 15 percent to provide an estimate of the coefficient of variation for runoff flow.

The EPA (1983b) notes that there are three types of water quality impacts associated with urban runoff: (1) rapid, short term impacts which occur during and shortly after storm events, (2) long-term impacts, such as bio-accumulation of toxics in aquatic organisms, and (3) short term impacts due to scour and resuspension of pollutants. The EPA concluded that the first type of impact, rapid short-term effects, were of primary concern. Therefore, the time scale of analysis should be that of rainfall events, which is on the order of hours. In addition, the event mean concentration (EMC), which is defined as the total mass of a pollutant discharged to a stream during a storm event divided by the total volume, was selected by the EPA as a measure of the average pollutant concentration. Analysis of EMCs for individual sites by the EPA (1983b) has found that they are lognormally distributed and therefore can be described by two values: a mean and a coefficient of variation. In addition, the EPA found that the EMCs were not correlated with runoff volume, and that the geographic location or land use are not important factors in explaining the inherent variability. As a result, runoff concentration data collected at one site can be transferred to an unmonitored site (EPA, 1983b). This is the approach followed in Vermont (Metcalf and Eddy, 1983; Moore and Langseth, 1985).

DiToro (1984) presented an approach of calculating the combined distribution of runoff and non-runoff events that included the fraction of hours, f, that runoff occurs:

$$f = \frac{\mu(d)}{\mu(\delta)} \tag{2-19}$$

where:

f = the fraction of hours runoff is occurring,

$\mu(d)$ = the mean rainfall duration, and

$\mu(\delta)$ = mean time between midpoints of successive storms.

The probability of a downstream concentration being greater than the water quality standard is equal to the fraction of time it is raining times the probability of the downstream concentration during runoff events being greater than the standard, plus the fraction of time it is not raining times the probability of the upstream concentration being greater the standard. Mathematically:

$$\Pr(T > S) = f[\Pr(T_r > S)] + (1-f)[\Pr(C > S)] \qquad (2\text{-}20)$$

where:

S = the water quality standard,

T = the concentration of pollutant downstream during both runoff and non-runoff periods,

T_r = the concentration of pollutant downstream during runoff events, and

C = the concentration of pollutant upstream.

III. DATA SOURCES AND PRELIMINARY ANALYSES

Analysis of downstream water quality excursions requires knowledge of the four variables used in the mass balance equation: upstream flow (F) and concentration (C) and effluent flow (q) and concentration (e). Values for F and C were obtained from USGS sources and those for q and e were obtained for both point and non-point source discharges. Sources of these data and analysis of their characteristics are described in this chapter. The characteristics were assessed by fitting a probability distribution to the input data. The distributions listed in Table 2-1 were fit to data sets for the four input variables using the method of moments and maximum likelihood procedures. The goodness of fit was assessed using the Kolmogorov-Smirnoff test at the 95% significance level.

The algorithms to fit the lognormal and Pearson distributions were adapted from Kite (1977), and the algorithms for the Kolmogorov-Smirnoff test are part of the International Math and Science Library (IMSL), which is installed on the College of Engineering's VAX computer. If an acceptable fit could not be obtained to one of the parametric distributions, then a non-parametric technique was used in which the data is re-sampled with replacement. This non-parametric technique is an adaptation of the Bootstrap method, which has been described by Efron and Gong (1983), and Diaconis and Efron (1983).

Upstream Flow

Five rivers, representing a range of average streamflows, were selected for study. The selection of these streams was based on their average flow rate, and the availability of streamflow and water quality data. Table 3-1

Table 3-1: Case Study Rivers and Characteristics

River	USGS Gauge No. City, State	Flowrate (CFS) Average	7Q10	Constituents for which data are available[4]
Blackstone	01111230 Millville, MA	598[1]	77[3]	Cu, BOD_5 Coliform
Charles	01103500 Dover, MA	324	14.1	Coliform, Cu
Connecticut	01184000 Thompsonville, CT	17,668	2,081	TSS, Cu, Coliform,
Merrimack	01096550 Lowell, MA	7,999[2]	824[3]	Coliform, Cu
Quinnipiac	01196500 Wallingford, CT	234	34.8	Coliform, Cu, TSS, NH_3-N

Notes:
1. Instantaneous flows only.
2. Computed from 01099550 and 01100000.
3. 7Q10 values estimated from Male and Ogawa (1982).
4. Total and fecal coliform, and total and dissolved copper were available. Only fecal coliform and total copper were used in the analyses.

lists the rivers along with flow rates and constituents for which data is available for the selected streams.

For the three rivers for which daily streamflow data was available (the Charles, Connecticut, and the Quinnipiac) the 7Q10 values were calculated using the lognormal distribution. The 7Q10 values for the Merrimack and Blackstone Rivers were estimated from Male and Ogawa (1982). The results of these determinations are also listed in Table 3-1. The percent of time that the streamflow is less than the 7Q10 was determined by constructing flow duration curves for each river for which daily streamflow data was available, and then reading the percentage of flows less than the 7Q10 from the graph. For the Charles, Connecticut and Quinnipiac Rivers daily average streamflow was less than the 7Q10 approximately 0.7, 1.0 and 1.9 percent of the time, respectively.

Mean streamflows are also listed, along with coefficient of variation, in summary form in Table 3-2. Table 3-3 lists the best fit distribution and associated parameter values for all five rivers.

Table 3-2. Arithmetic Means and Coefficients of Variation (in parentheses) for Case Study Study Rivers

	Parameter					
River	River Flow (cfs)	Total Copper (μg/l)	Fecal Coliform (#/100 ml)	BOD_5 (mg/l)	TSS (mg/l)	NH_3- N (mg/l)
Blackstone	598 (0.964)	22.348 (0.341)	1033 (1.736)	2.36 (0.531)	NA	NA
Charles	324 (1.020)	4.79 (0.444)	214 (2.758)	NA	NA	NA
Connecticut	17668 (0.940)	8.9 (0.755)	2789 (2.78)	NA	15.5 (0.8735)	NA
Merrimack	7999 (0.908)	8.74 (0.770)	1326 (1.464)	NA	NA	NA
Quannipiac	233.8 (0.929)	14.13 (0.392)	5695 (1.583)	NA	23.6 (0.6675)	0.3156 (0.733)

Note: NA = Data not available.

Table 3-3. Best Fitting Distributions and Relevant Parameters for Case Study Rivers.

River	Parameter					
	River Flow	Total Copper	Fecal Coliform	BOD_5	TSS	$NH_3 - N$
Blackstone	Log Pearson III α = 0.16507 β = 26.211 γ = 1.6975	Gamma α = 8.0717 β = 2.7687	Log Pearson III α = 0.47767 β = 7.483 γ = 2.4155	2-Para. Lognormal μln = 0.71363 σln = 0.5636	NA	NA
Charles	2 Para. Lognormal μln = 5.3244 σln = 0.9940	Uniform A = 1.1093 B = 8.474	Log Pearson III α = 0.24477 β = 37.714 γ = -5.1815	NA	NA	NA
Connecticut	2 Para. Lognormal μln = 9.4462 σln = 0.8162	2 Para. Lognormal μln = 1.9797 σln = 0.6265	Gamma α = 0.4818 β = 5788.2	NA	3-Para. Lognormal μln = 2.8261 σln = 0.6068 A = -4.789	NA
Merrimack	Pearson III α = 8347.7 β = 0.75633 γ = 1685.6	Log Pearson III α = 0.1936 β = 8.750 γ = 0.29413	2 Para. Lognormal μln = 6.3933 σln = 1.448	NA	NA	NA
Quinnipiac	Log Pearson II α = 0.12794 β = 33.453 γ = 0.8834	Log Pearson III α = 0.868 β = 18.289 γ = 0.99136	2 Para. Lognormal μln = 7.4525 σln = 1.7407	NA	3-Para. Lognormal μln = 3.4564 σln = 0.43106 A = -11.21	Gamma α = 1.8591 β = 0.1697

Note: NA = data not available

Upstream Water Quality

Selection of the specific constituents used in the analysis was dictated, to a certain extent, by the availability of data. Three types of pollutants were selected: (1) biological (fecal coliform), (2) conventional (BOD_5 and TSS), and (3) metals (total copper). Copper was selected as the metal to analyze because the EPA (1983b) has identified it as the key toxicant in urban runoff. For conventional pollutants, BOD was analyzed for the Blackstone, and TSS for the Connecticut and Quinnipiac rivers. USGS (1985) notes that TSS is usually obtained by subtracting dissolved from total solids. Unfortunately, the large errors caused by subtracting two large numbers may limit the usefulness of this portion of the analysis. Also, since data was available, NH_3-N was analyzed for the Quinnipiac River. Table 3-2 lists means and coefficients of variation for the constituents.

Preliminary analyses of the water quality data for the five rivers were conducted to determine: (1) whether the data has changed over time, (2) if correlations exist between instantaneous streamflow and constituent concentration, (3) the basic shape of the histogram for each variable, and (4) estimates of parameters and goodness of fit for selected probability distributions.

Instream pollutant concentrations may have changed over the years, primarily resulting from water quality improvement due to the construction of wastewater treatment plants. If the data has changed significantly over time, then use of the complete data set would result in biased estimates of the instream pollutant concentrations.

To determine if the water quality data for recent years were significantly different than earlier years, the data was separated (by visual examination) into two sets: before and after a year which seemed to delineate higher and lower concentrations, respectively. The Chow test was employed at the five percent significance level (Johnston, 1984) to determine if the two data sets were different. Based on the results of the tests, there are significant changes over time in the Quinnipiac River total copper, dissolved copper and ammonia concentrations, and in the Connecticut River fecal coliform concentrations. Therefore, only those data from the Quinnipiac and Connecticut rivers which are representative of current conditions were used in the analysis.

Correlations between each of the constituents listed in Table 3-1 and instantaneous streamflow were calculated to assess the relationship between concentrations and streamflow. The calculation of relationships between concentrations and flows, and log-transformed concentrations and log-transformed flows shows that there are significant correlations for: total coliform in the Merrimack; total coliform, fecal coliform, and TSS in the Connecticut; fecal coliform in the Blackstone; and fecal coliform in the Quinnipiac.

Since some of the correlations are positive and some are negative, it is difficult to predict, aprioi, whether the concentration of a constituent will have a positive or negative correlation with streamflow. Warn and Brew (1980) note that a constant pollutant load can be expected to have a negative correlation with streamflow due to dilution effects. On the other hand, runoff could cause both the pollutant concentration and streamflow to increase, and thus lead to positive correlation. Positive correlations could also be due to the scouring action of greater than normal streamflows which may resuspend contaminated sediments.

The probability distributions listed in Table 2-1 were fit to water quality data from the five rivers, and the goodness of fit was compared using the Kolmogorov-Smirnoff test. Of the 28 data sets which could be fit to a parmametric distribution (including several not shown in Table 3-3), 12 were best fit by the 2-parameter lognormal, two by the 3-parameter lognormal, one by the uniform, one by the Pearson Type III, eight by the log-Pearson Type III, and four by the gamma. Table 3-3 lists the distributions and parameter values for the constituents used in the analysis. These results are consistent with the results reported by Driscoll (1986), who concluded that the underlying population of pollutant concentrations is either lognormally distributed, or can at least be approximated by it.

Wastewater Treatment Plant Effluents

Based on a thorough search of treatment plants that are close to USGS water quality monitoring stations, the wastewater treatment plant at Wallingford, CT. was selected as a representative POTW. This plant, which discharges into the Quinnipiac river, was selected because it is located on a small, stressed river, has data on ammonia, and is located approximately 2.5 miles downstream from a USGS water quality monitoring and gaging station. The most recent two years of BOD_5, TSS, NH_3-N, and flow data from the Wallingford POTW were obtained. Because the Wallingford data set did not contain fecal coliform data, this data was obtained from the Amherst POTW.

The average effluent concentrations (e) used in the simulations are listed in Table 3-4, along with the coefficients of variation (CV) for each of the constituents. The effluent values for TSS and NH_3-N were obtained from the Wallingford POTW and are thought to be representative of typical

Table 3-4: Summary of Mean Effluent Concentrations, Coefficients of Variation, and Water Quality Standards Used in Wasteload Allocation Simulations

Constituent	Mean	Coefficient of Variation	Water Quality Standard
Total Copper	118 μg/l	1.14	20 μg/l
TSS	34.8 mg/l	0.332	25 mg/l
BOD_5	30 mg/l	0.65	5 mg/l
NH_3-N	18.8 mg/l	0.189	0.5 mg/l

POTWs. However, the average BOD_5 for the Wallingford POTW was 55 mg/l, which is considered high. Therefore, as a more representative value, 30mg/l was used. The estimate of the coefficient of variation for BOD_5 was obtained from typical values reported by the EPA (1985). The mean and coefficient of variation for total copper were obtained from an analysis of pooled data from secondary POTWs in Massachusetts (Hanley, 1985).

Preliminary analyses of this effluent data were conducted to determine correlations between flow and concentrations, and estimates of the parameters and goodness of fit for selected probability distributions. Determination of correlations between effluent flow and the four pollutants revealed that correlations between TSS and flow in the Wallingford data set, and between fecal coliform and flow in the Amherst data set, are statistically significant at the five percent level (Fisher and Yates, 1963).

The probability distributions listed in Table 2-1 were fit to flow, BOD_5, TSS, NH_3-N, and fecal coliform data, and the goodness of fit compared using the Kolmogorov-Smirnoff statistic. The best fitting distribution and estimates of appropriate parameters are provided in Table 3-5. A significant fit could not be achieved for daily flow.

Non-point Source Discharges

Rainfall is the driving force behind runoff, and since rainfall records are extensive, runoff flow was calculated from rainfall intensity (equation 2-18). Use of rainfall records allows calculation of intensity for each storm, average storm duration and average time between storms. Average yearly values, based on hourly rainfall records from Worcester Airport (from 1957 to 1985), were used for this portion of the analysis. Analysis of rainfall events was conducted using the Synoptic Rainfall Data Analysis Program (SYNOP) (Woodward-Clyde Consultants, 1988).

Pertinent values are:

Storm intensity (in/hr.)	
Average	0.05
Coef. of Variation	1.171
Ave. storm duration (hrs.)	7.40
Ave. time between storms (hrs.)	71.01

To account for the fact that small rainfall intensities do not contribute to runoff (due to interception), the coefficient of variation of rainfall intensities was reduced by 15 percent (to 0.995), as recommended by the EPA (1983b). The minimum time between rainfall recordings which result in independent rainfall events was calculated by SYNOP to be four hours. The EPA proposed that, since rainfall is a Poisson process, the time between rainfall events is exponentially distributed, and therefore has a coefficient of variation equal to one (EPA, 1979).

Representative pollutant concentrations for both storm sewers and combined sewers were used in the analysis and were obtained from the

Table 3-5: Parameters of Best Fitting Distributions Used in Monte Carlo Simulations for Point Source Flow and Constituent Concentrations

Parameter	Arithmetic Mean and CV	Best Fitting Distribution
Daily Flow	μ = 7.0 CV = 0.167	log Pearson III α = 0.03285 β = 23.887 γ = 1.1462
Fecal Coliform	μ = 46.1 CV = 1.026	Gamma α = 0.72738 β = 63.366
TSS	μ = 34.8 CV = 0.332	3-Para Lognormal μln = 3.3877 σln = 0.3550 A = -11.21
NH_3-N	μ = 18.8 CV = 0.189	Uniform A = 12.642 B = 24.967
BOD_5	μ = 53.3 CV = 0.423	Pearson III α = 7.0964 β = 10.092 γ = -18.305

National Urban Runoff Program (EPA, 1983b) and Metcalf and Eddy (1986), respectively. These values are listed in Table 3-6.

Water Quality Standards

The water quality standards (S) used in the simulations are summarized in Table 3-4. The standards for total copper, and NH_3-N were obtained from the Massachusetts Division of Water Pollution Control (Kimball, 1988). There are no formal water quality criteria for TSS and BOD_5. However, for screening level analyses similar to the present research, the EPA (1982) used a 24 hour value of 25 mg/l for TSS and 5 mg/l for BOD_5. The 5 mg/l BOD_5 standard is similar to the 4 mg/l BOD_5 which is currently used in Great Britain for the most sensitive river use (Anglian Water Authority, 1981).

Table 3-6: Summary of Urban Runoff Non-Point Source Discharge Characteristics

	Storm Sewer Discharges[1]		Combined Sewer Overflows[2]	
Constituent	Median Value	Coefficient of Variation	Median Value	Coefficient of Variation
TSS (mg/l)	100	1.0 - 2.0	131	1.1
BOD_5 (mg/l)	9	0.5 - 1.0	38	1.1
Tot. Cu (μg/l)	34	0.5 - 1.0	135	1.5
Fecal Coliform (per 100 ml)	21,000	0.8	100,000	2.8

Notes: 1. Source: Nationwide Urban Runoff Program (EPA, 1983b)
2. Source: Metcalf & Eddy (1986)

Determination of a more exact BOD_5 standard would be possible by modeling the river and determining a BOD_5 value based on minimum dissolved oxygen levels. However, for such an analysis to be an improvement over the use of a single BOD_5 standard, a large amount of data on flow velocity, stream geometry, re-aeration rates, decay rates, etc. would have to be obtained. Given the preliminary nature of this study, such detail is not warranted and a single number BOD_5 water quality standard is adequate.

Point source discharge permits are based on: (1) the best available technology (BAT) which is available for treating a particular waste and (2) downstream water quality. Limitation on downstream water quality may require that treatment be in excess of BAT, and conversely BAT may result in downstream concentrations better than water quality criteria. The governing regulation depends on values of all of the input variables, but primarily on the relative magnitude of the effluent discharge and river flow rates. Since the concentrations listed in Table 3-4 were obtained from an analysis of the effluent concentrations in wastewater treatment plants, these values were used to approximate what technology based standards would require. For water quality constrained effluent concentrations, the required values were obtained by solving equation 2-1 for the effluent concentration (e), and substituting the water quality standard (S) for the downstream concentration (T) and the 7Q10 flow for river flow (F):

$$e = \frac{7Q10*S + S*q - 7Q10*C}{q} \qquad (3\text{-}1)$$

The water quality standard for fecal coliform in Class B rivers states that it:

> Shall not exceed a log mean for a set of samples of 200 per 100 ml, nor shall more than 10% of the total samples exceed 400 per 100 ml during any monthly sampling period (314 CMR, 1986).

The fecal coliform standard for Class C rivers is similar to the Class B standard, except that the standard is based on a log mean of 1000 per 100 ml. Since the Monte Carlo simulation computer code was not developed to accommodate such a standard, a fecal coliform standard of 1000 per 100 ml was chosen for use in the simulation because it lies between the Class B and Class C standards.

IV. ANALYSES OF POINT SOURCE DISCHARGES

The results of determining the excursion frequencies for point source discharges are presented in this chapter. The analyses were conducted to gain insight into: (1) the excursion frequencies resulting from the use of the 7Q10 low flow as the critical streamflow in wasteload allocations for point source discharges, and (2) the affect of variations in the values of the variables in the mass balance equation (F, C, q, e) on excursion frequencies.

Description of Excursion Frequency Simulation Procedure

Monte Carlo simulations were performed to determine the probability distributions of downstream constituent concentrations. For each constituent a wasteload allocation was performed, using the 7Q10 streamflow as the critical low flow, to determine the maximum effluent concentration of the constituent. Excursion frequencies for the downstream concentrations were obtained by allowing the four input variables (F, C, q, & e) to vary according to the probability distributions of each variable. To gain a broader understanding of the results (beyond those available for the given data), a range of values for average effluent flowrates was used.

A series of excursion frequency simulations was performed by varying the effluent flow rate (q) over a range of possible values from 1 to 250 MGD. The effluent concentrations (e) used in the excursion frequency simulations were either calculated from equation 3-1, or set equal to the values listed in Table 3-4, whichever was the lower value.

At each increment in the simulation (new mean value for q), 5000 iterations were performed in which random values were drawn from the probability distributions of F, C, q & e, and then combined in the mass balance equation to determine the resulting probability distribution for the downstream concentrations (T). The probability distribution of T was analyzed to determine the excursion frequency, and then the cycle was repeated for a new value of q (and recalculated value of e).

In the excursion frequency simulations, effluent flow and concentration (q & e) are assumed to be lognormally distributed. The parameters of the lognormal distribution ($\mu(y)$ and $\sigma^2(y)$ of equation 2-2) were obtained from the arithmetic mean and CV using equations 2-3 to 2-5. For river flow (F) and river concentration (C), the distributions determined in the preliminary analysis (as summarized in Table 3-3) were used.

To allow an easier comparison of the results for the different rivers, and also between point and non-point sources of pollution, the difference between the total excursion frequency and the excursion frequency due solely to background concentrations was used. This value is referred to as the incremental excursion frequency (Δ):

$$\Delta = Pr(T > S) - Pr(C > S) \tag{4-1}$$

where:

S = water quality standard

T = concentration of pollutant downstream

C = concentration of pollutant upstream

Effect of 7Q10 Low Flow on Excursion Frequency

Simulations were performed, as described in the previous section, to gain insight into the magnitude of excursion frequencies resulting from the use of the 7Q10 streamflow as the critical low flow in regulating point source discharges. Simulations were performed for most of the rivers and constituents shown in Table 3-1. It was not possible to perform wasteload simulations for total copper in the Blackstone River or for fecal coliform in any other the rivers because the average background concentrations for these constituents are greater than the water quality standard.

The downstream concentrations resulting from wasteload allocation simulations for total copper at discharge rates of 2, 10, and 25 MGD into the Quinnipiac River are plotted in Figure 4-1. For ease of viewing, only the highest 30 percent of the values were plotted. From this figure it can be seen that, for a water quality standard for total copper of 20 μg/l, the excursion frequency is approximately 19 percent for a 2 MGD discharge, 21 percent for a 10 MGD discharge, and 23 percent for a 25 MGD discharge. Also shown is the excursion frequency for the background concentration of total copper (approximately 13 percent), which would result when no discharge is present.

Figure 4-2 presents the results in a different way, showing the incremental excursion frequencies for 2, 10, and 25 MGD discharges of total copper to the Quinnipiac River. Here, the background excursion frequency of 13 percent has been subtracted from the excursion frequencies taken from Figure 4-1.

The results of the wasteload allocation simulations for TSS, total copper, NH_3-N, and BOD_5 are plotted in Figures 4-3 to 4-6, respectively. Generally, the effluent limitations at the lower flows were technology based, while at the higher flows they were water quality limited. These two limitations are represented in different ways in the figures. A dashed line represents excursions resulting from technology based effluent limitations, while a solid line represents excursions resulting from water quality based effluent limitations.

An examination of these figures shows that larger mean discharge flows result in higher incremental excursion frequencies for both technology based

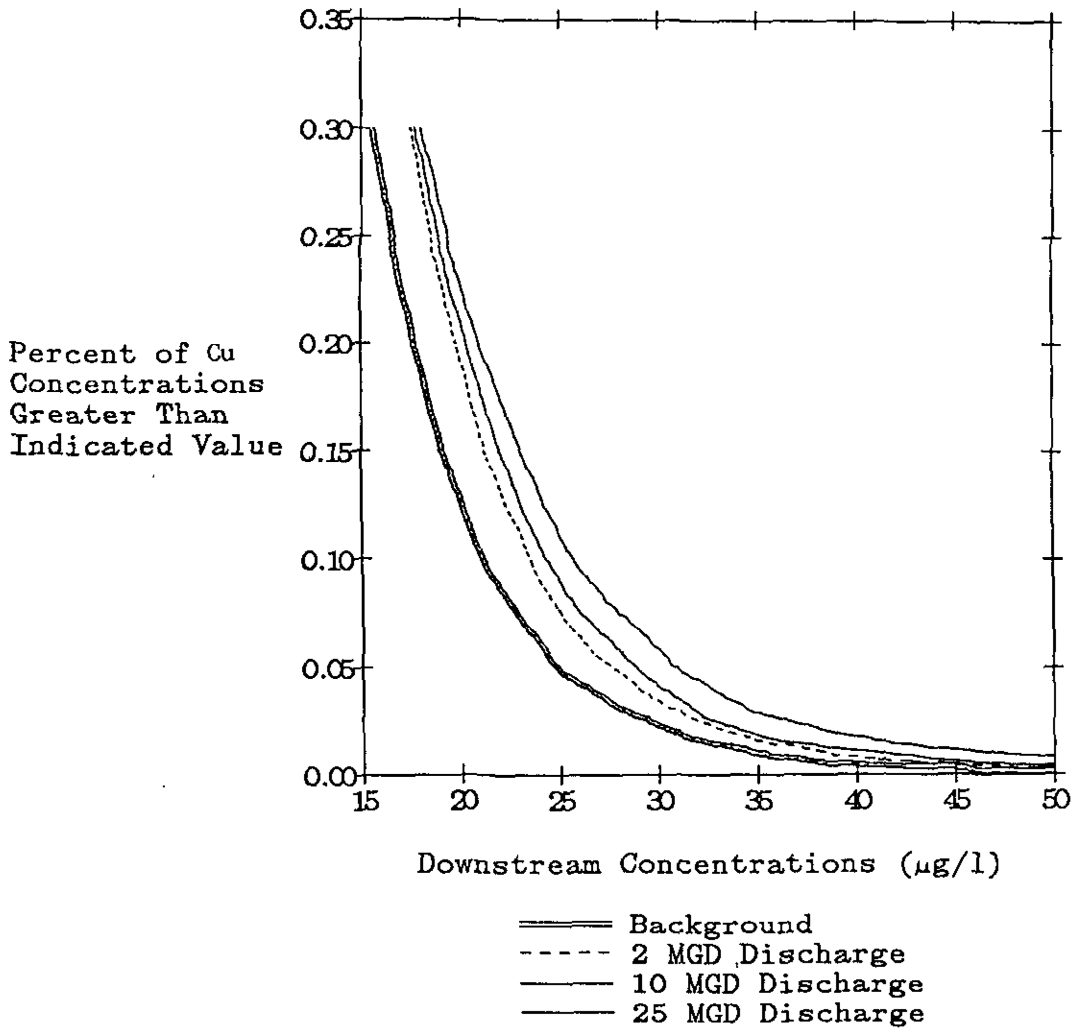


Figure 4-1: Cumulative Probability Distributions of Downstream Total Copper Concentrations Which Result From Wasteload Allocations for Hypothetical Discharges of 2, 10, and 25 MGD into the Quinnipiac River

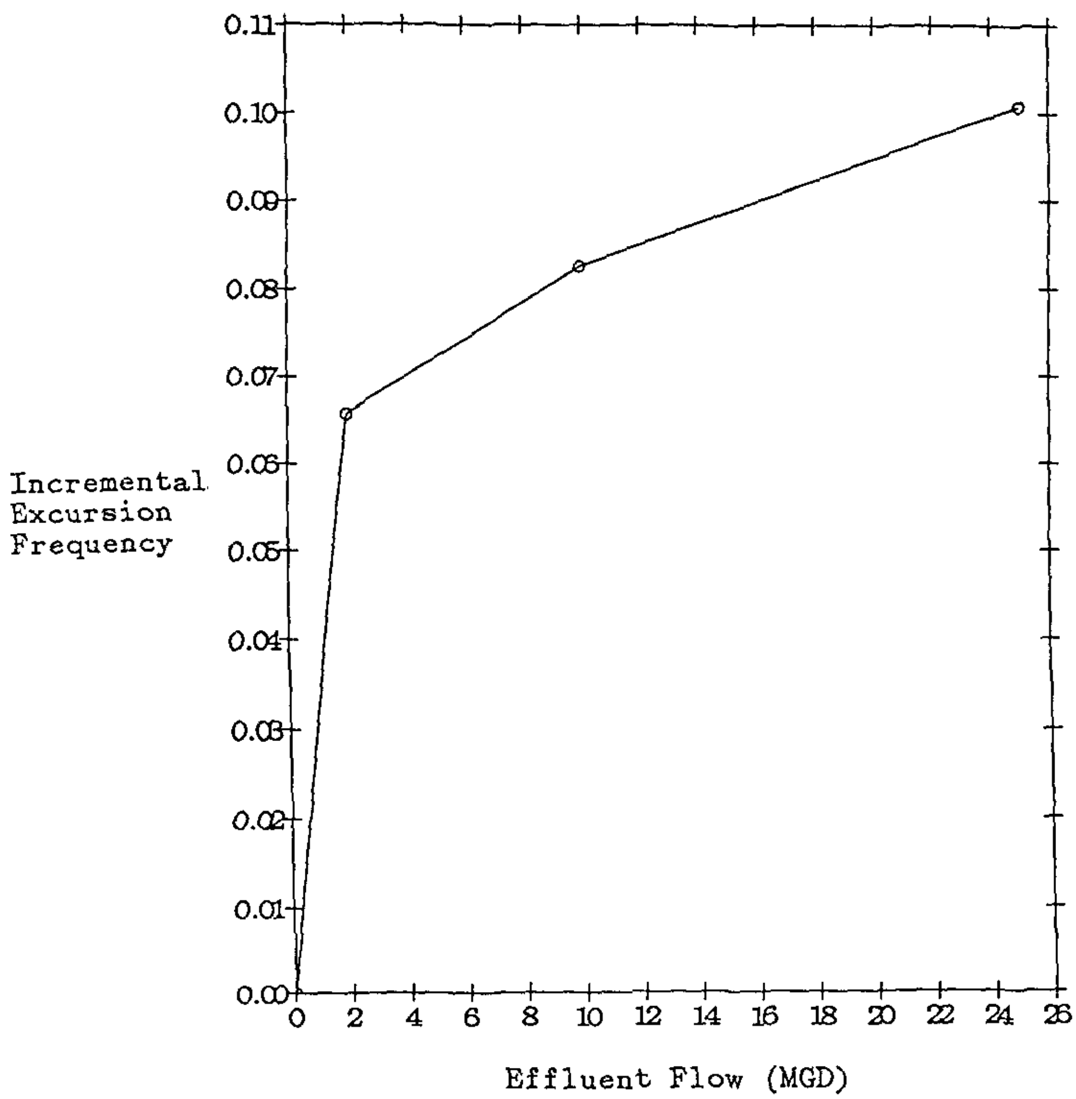


Figure 4-2: Incremental Excursion Frequencies Which Result From Wasteload Allocations for Total Copper Discharges for Different Effluent Flows into the Quinnipiac River

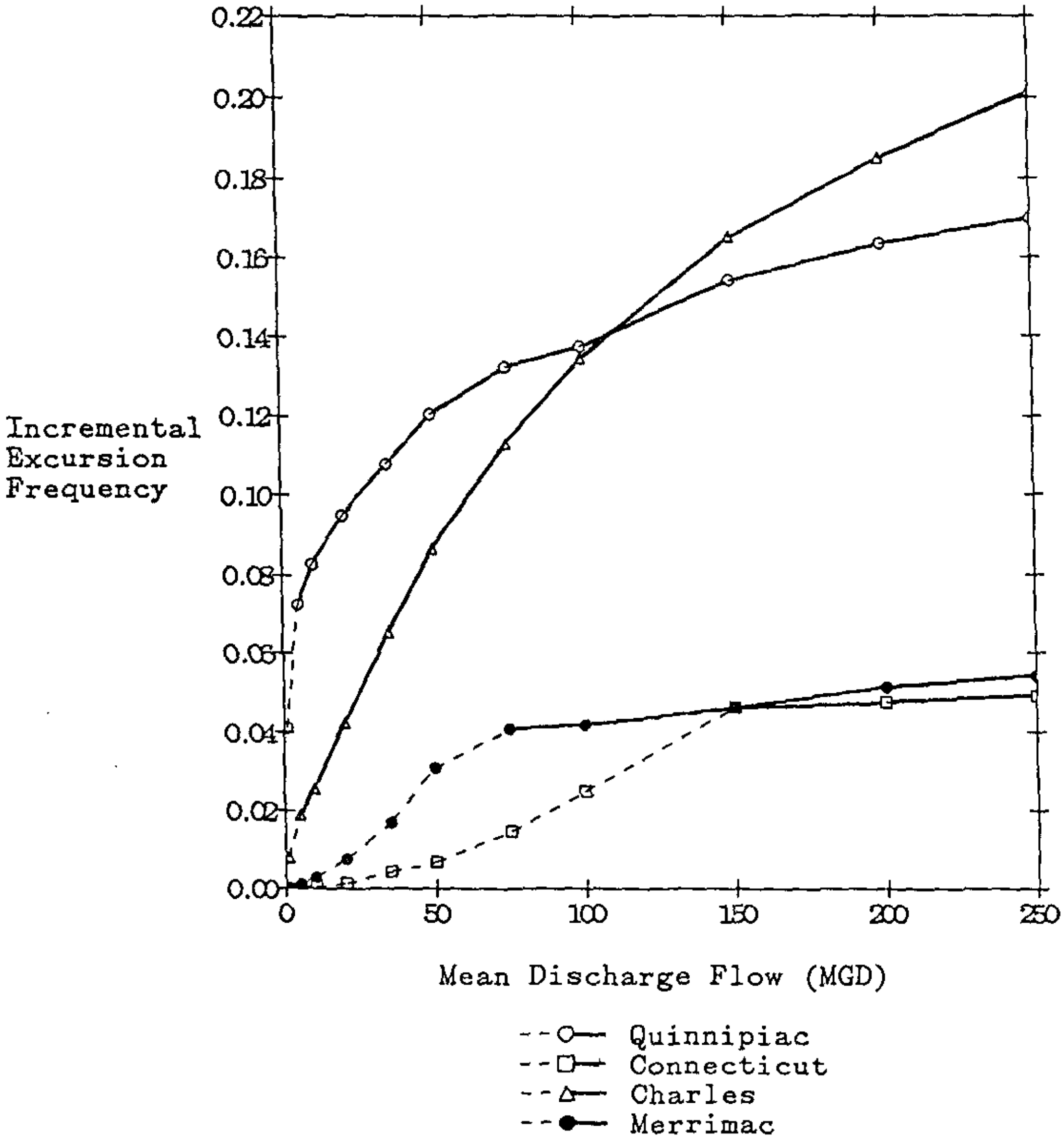


Figure 4-3: Incremental Excursion Frequencies Resulting From Wasteload Allocations for Total Copper Discharges for Different Effluent Flows

Note: Dashed line represents technology based limitation and solid line represents water quality based effluent limitation.

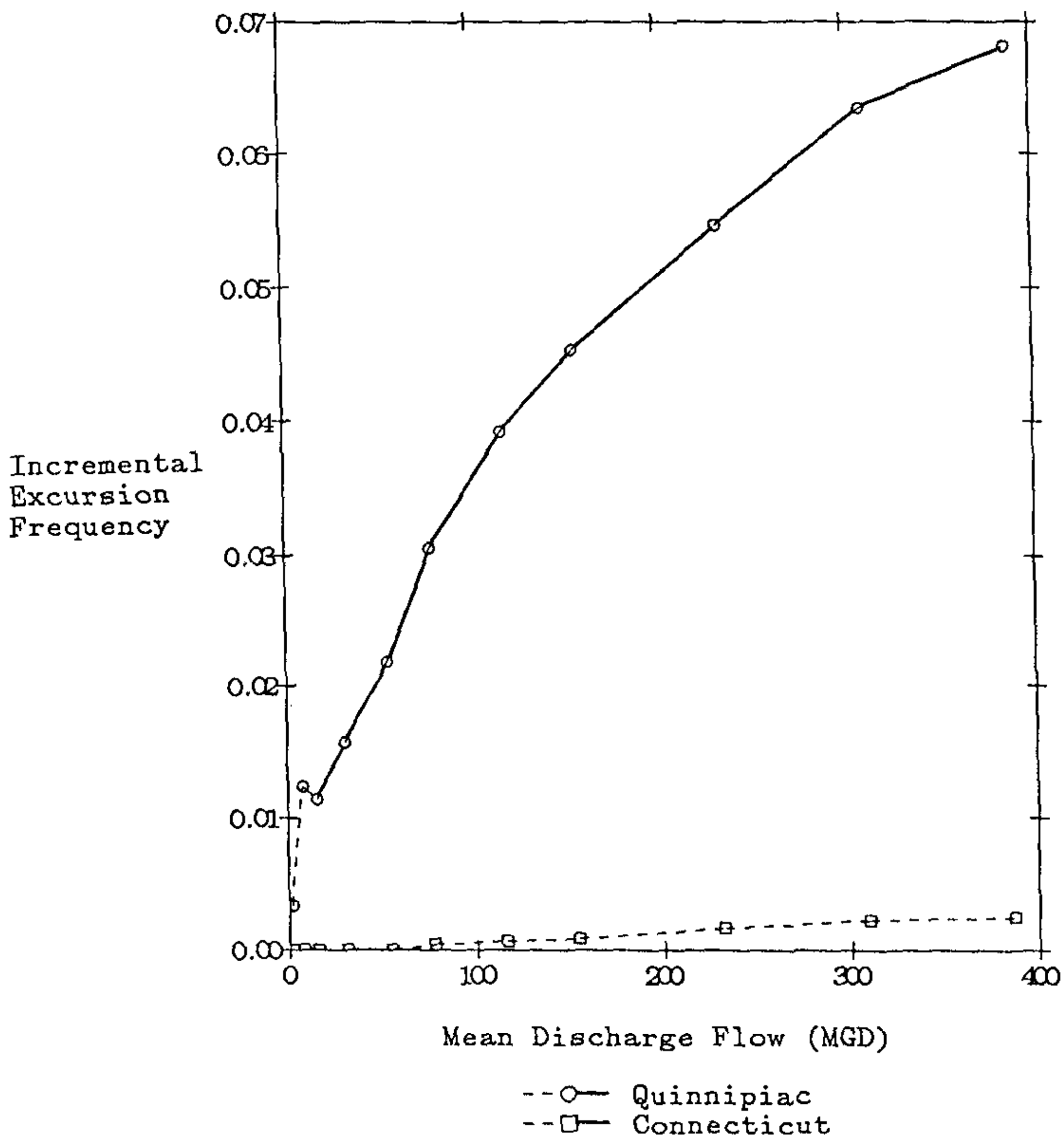


Figure 4-4: Incremental Excursion Frequencies Resulting From Wasteload Allocations for TSS Discharges for Different Effluent Flows

Note: Dashed line represents technology based limitation and solid line represents water quality based effluent limitation.

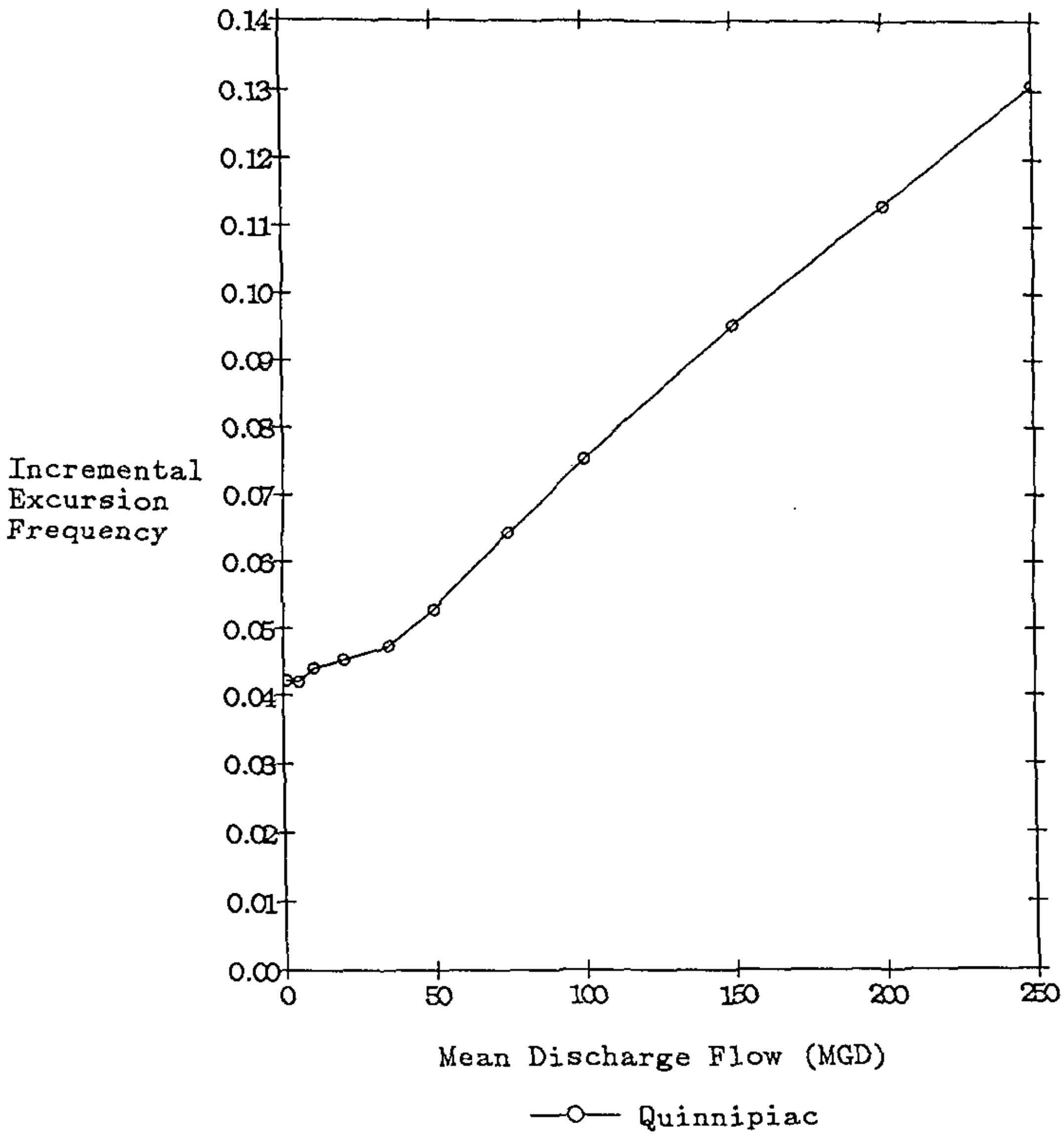


Figure 4-5: Incremental Excursion Frequencies Resulting From Wasteload Allocations for NH_3-N Discharges for Different Effluent Flows

Note: Dashed line represents technology based limitation and solid line represents water quality based effluent limitation.

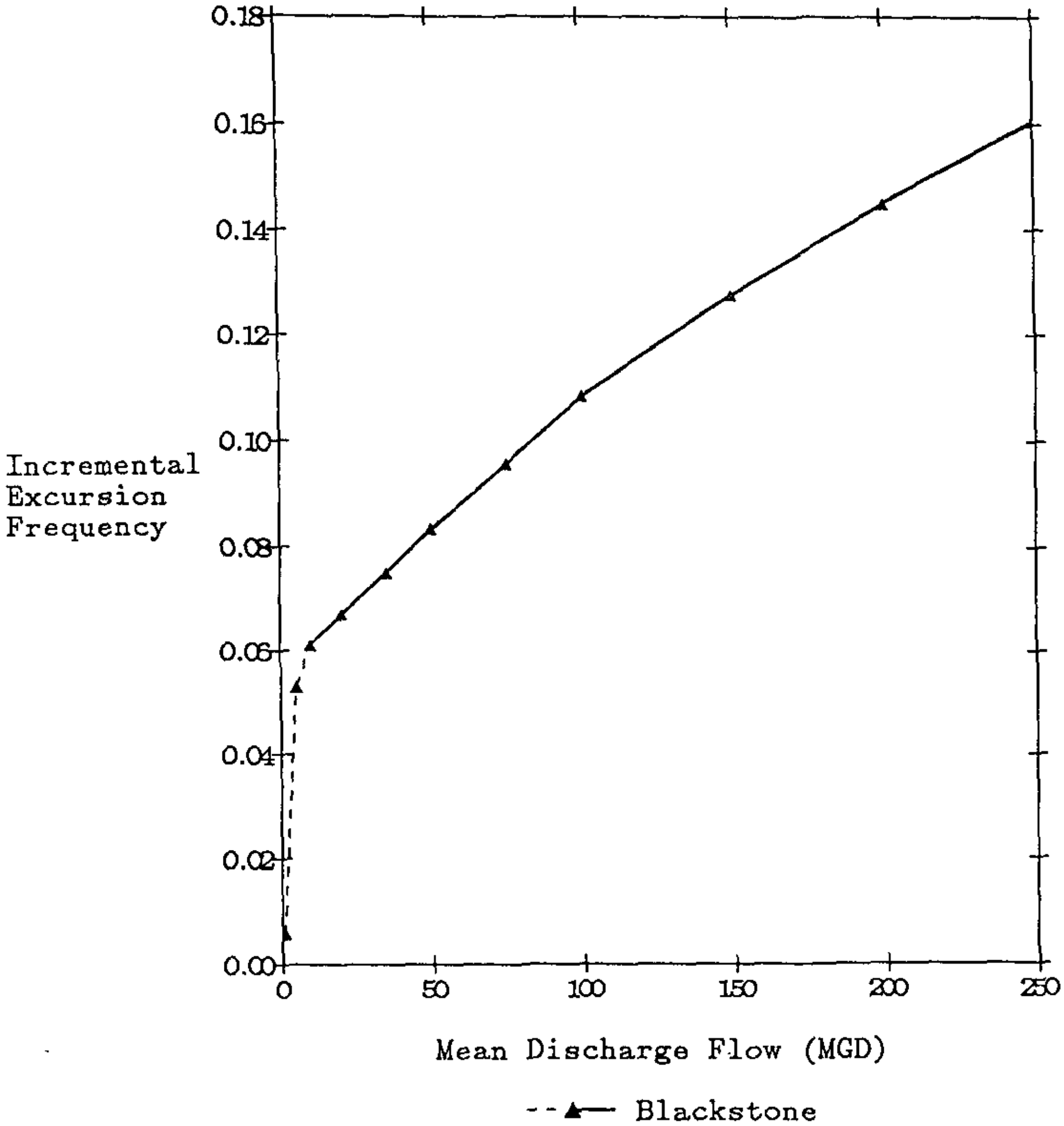


Figure 4-6: Incremental Excursion Frequencies Resulting From Wasteload Allocations for BOD_5 Discharges for Different Effluent Flows

Note: Dashed line represents technology based limitation and solid line represents water quality based effluent limitation.

and water quality based effluent limitations. This relationship can be demonstrated by relating the average downstream concentration (T) resulting from a wasteload allocation to the ratio of the discharge flow to the total flow, Φ, which was defined earlier as:

$$\Phi = \frac{q}{F + q} \tag{4-2}$$

Substituting equations 4-2 and 3-1 into the mass balance equation (equation 2-1), and then rearranging yields:

$$T = \left(S - C - \frac{7Q10(S - C)}{F}\right)\Phi + C + \frac{7Q10(S - C)}{F} \tag{4-3}$$

As can be seen from this equation, a plot of T versus Φ results in a straight line with a slope equal to $\left(S - C - \frac{7Q10(S - C)}{F}\right)$, and an intercept equal to $C + \frac{7Q10(S - C)}{F}$. Since the mean streamflow (F) is a constant for a particular, as q increases, Φ increases.

A caution should be raised concerning equation 4-2. The equation was derived based on the assumption that a non-zero value for q was used in a wasteload allocation to calculate the concentration of e. In addition, at very small values of q, a wasteload allocation would permit a very large concentration of a pollutant to be discharged. Such a situation would not exist in an actual wasteload allocation because the river would cease being water quality limited and technology based standards, which specify minimum treatment levels, would apply.

The results showing higher excursion frequencies for larger discharges were based on representative values for constituents from the Wallingford wastewater treatment plant. The coefficient of variation for flow and for the constituents were assumed to be the same for all size dischargers. The assumption may be overly simplistic since larger treatment plants would tend to have less variability to their operations, and therefore the respective coefficients of variation would be lower than smaller plants. With lower coefficients of variation, the resulting excursion frequencies would be lower (for the higher discharge flows) than are shown in Figures 4-3 through 4-6.

Even though two average downstream concentrations may be below the standard, the higher of the two would likely result in a higher excursion frequency. This fact can be seen from Figure 4-7, which depicts three hypothetical probability distributions with different means (all below the standard of 30) but with the same coefficient of variation. The shaded areas in the plots represent the excursion frequencies. Thus it can be seen that a higher effluent flow will result in a higher average downstream concentration, which in turn results in a higher excursion frequency.

Effect of Mass Balance Variables on Excursion Frequency

To gain insight into the effect that each variable in the mass balance equation (F, C, q & e) has on the excursion frequency, the wasteload allocation simulations described in the first section of this chapter were repeated allowing the variable of interest to assume stochastic properties, while the other variables were held constant. Three cases were considered:

1. The upstream river flow, F, was allowed to assume stochastic properties, while the remaining input variables (C, q, and e) were held constant at their mean values.
2. The upstream river concentration, C, was allowed to assume stochastic properties, while the remaining input variables (F, q, and e) were held constant at their mean values.
3. The effluent flow and concentration, q and e, were allowed to assume stochastic properties, while the remaining variables (F and C) were held constant at their mean values.

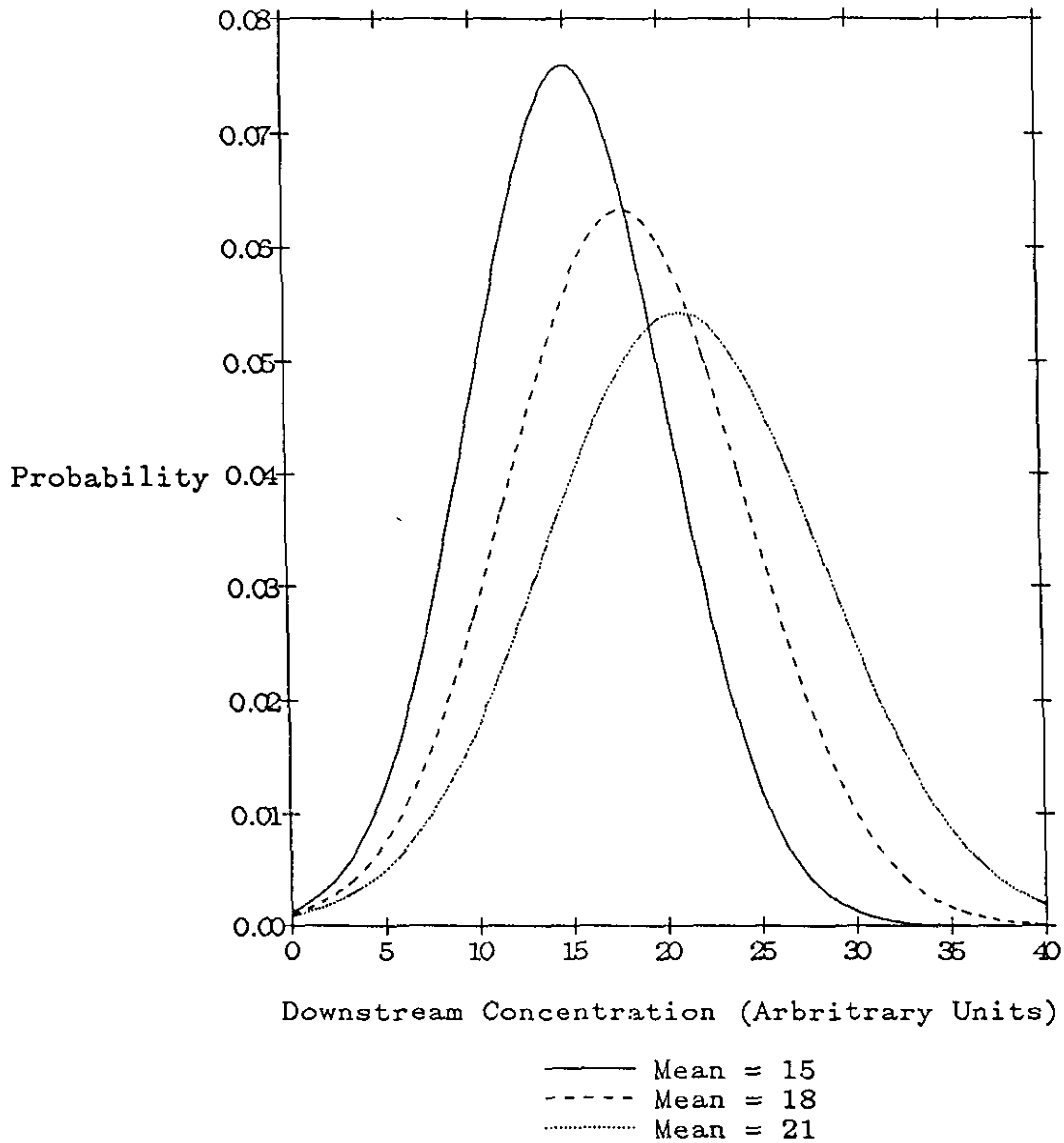


Figure 4-7: Effect of Different Mean Downstream Concentrations (All with the same coefficient of variation) on the excursion frequency

In addition to the above, the case where all four input variables are allowed to have stochastic properties (the results of the previous section) was included for comparison.

The excursion frequencies for total copper discharges into the Quinnipiac River are shown in Figure 4-8, as an illustration of the results of these analyses. Analyses for total copper discharges in the Charles, Connecticut, and Merrimack Rivers, TSS discharges into the Connecticut and Quinnipiac Rivers, NH_3-N into the Quinnipiac, and BOD_5 into the Blackstone River were performed and are detailed by Soucie (1989). In this figure, the total excursion frequency was plotted as opposed to the incremental excursion frequency, because one of the variables being studied is the effect of background concentrations on the excursion frequency.

Several interesting observations can be made from the results presented in Figure 4-8 and by Soucie, including: (1) the relative magnitude of excursion frequencies resulting in variations in different input parameters, (2) the changes (and relative changes) in excursion frequencies with increasing discharge flow, and (3) the differences in results for the cases in which only F or C are allowed to have stochastic properties, as opposed to the case where only q and e are allowed to vary.

For all the constituents tested, the excursion frequency resulting from the case where all of the variables (F, C, q, and e) are allowed to have stochastic properties (presented in previous subsection) is greater than for any of the other cases in which one or more of the variables are held constant. The reason for this result can be seen from equation 2-11, which is reprinted below:

$$\sigma^2(T) = \sigma^2(\Phi)[\mu(e) - \mu(c)]^2 + \sigma^2(e)[\sigma^2(\Phi) - \mu^2(\Phi)]$$
$$+ \sigma^2(C)[\sigma^2(\Phi) + (1 - \mu(\Phi))^2] \qquad (4\text{-}4)$$

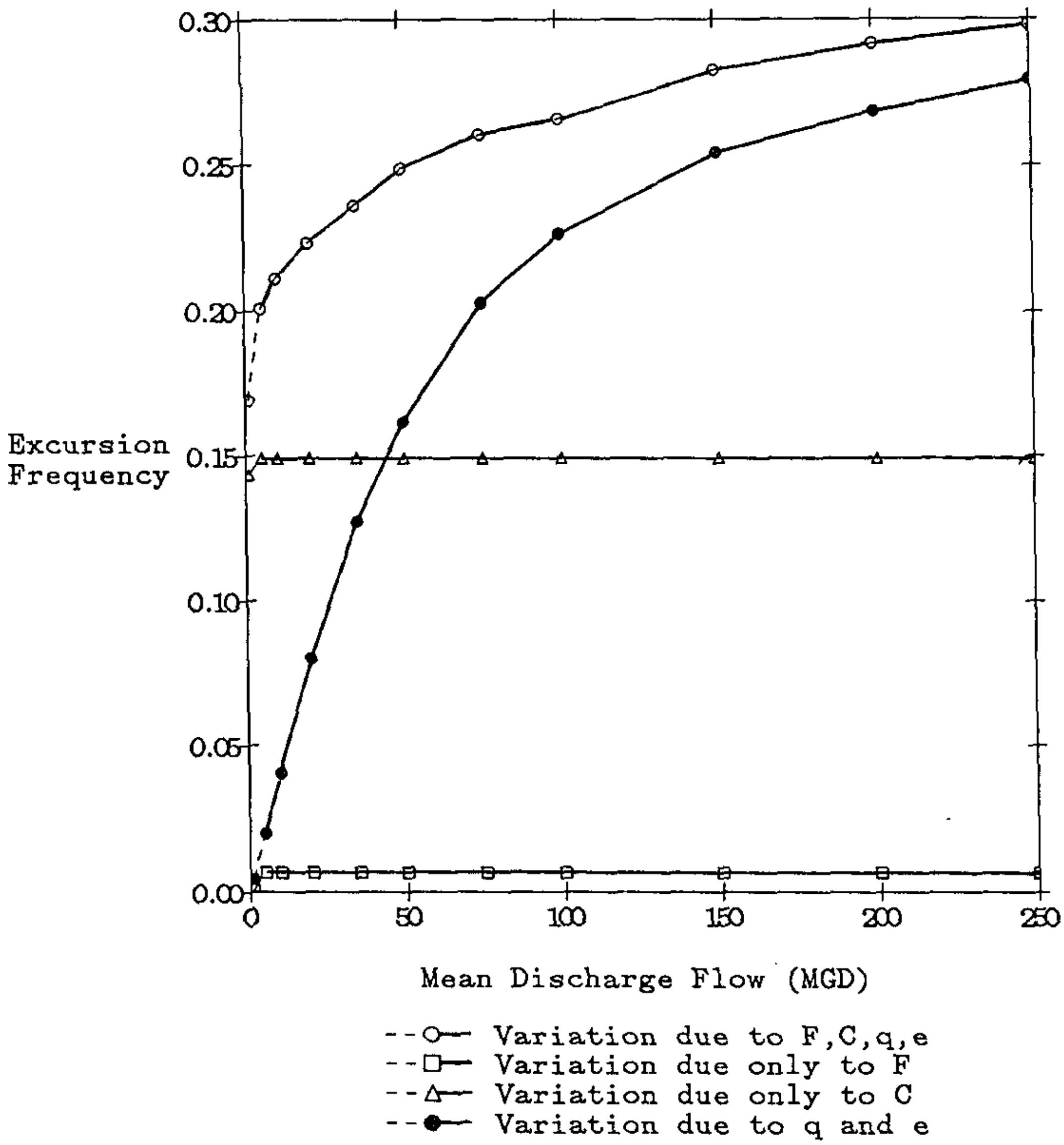


Figure 4-8: Excursion Frequencies Resulting From Allowing Indicated Variables to Have Stochastic Properties for Different Size Discharges of Total Copper Into the Quinnipiac River

From this equation, it can be seen that the variance of the downstream concentration ($\sigma^2(T)$) is directly related to the variances of e, C, and Φ. When any of the values of e, C, or Φ are held constant, its variance is zero, thereby reducing the value of $\sigma^2(T)$. Even though the means of two downstream concentrations are equal, a higher variance of one will result in an increase in the excursion frequency. This fact is illustrated in Figure 4-9, which depicts three hypothetical probability distributions with the same mean, but with different values for variance. The shaded areas in the plot are the excursion frequencies. Thus it can be seen that a probability distribution with the same mean but with a higher variance will result in a higher excursion frequency than one with a lower variance. It is interesting to note that the opposite conclusion is true if the mean is above the standard. In this case the higher value of CV would result in fewer excursions.

Because of the complex nature of the factors which influence the excursion frequency, it can not be predicted with certainty which individual variables (F, C, q, or e) at a given discharge flow rate exert the greatest influence on the excursion frequency. For example, referring to Figure 4-8, which shows the excursion frequencies resulting from total copper discharges into the Quinnipiac River, it can be seen that below a discharge flow rate of about 45 MGD, the relative magnitude of the excursion frequency when only C is allowed to vary is greater than the case in which q and e are allowed to vary. However, above a discharge flow rate of 45 MGD, the opposite is true. In another river or for another constituent, the situation is different.

An interesting feature of Figure 4-8 is the observation that, for the cases in which either F or C is the only variable to have stochastic

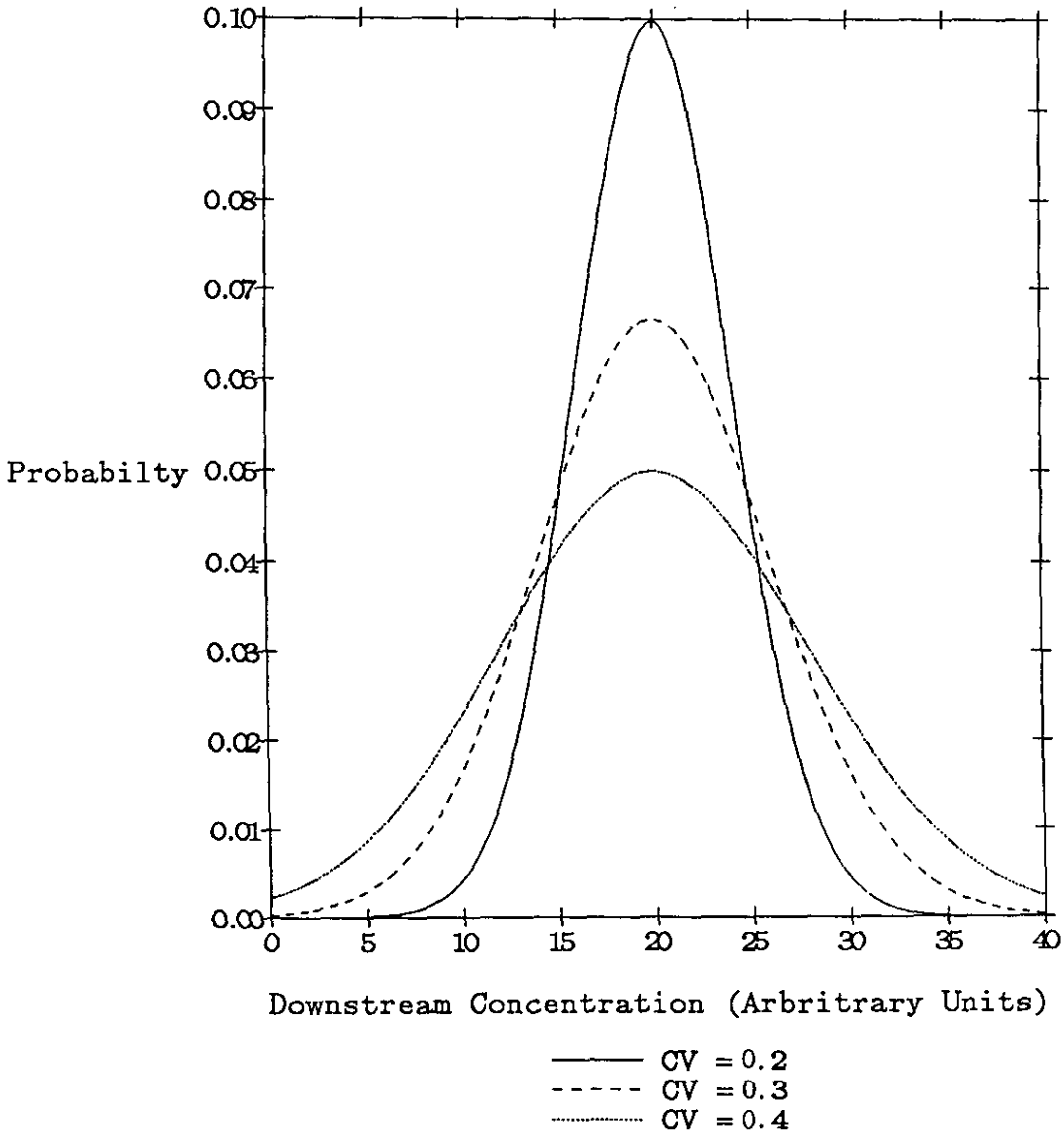


Figure 4-9: Effect of a Larger Variance for Equal Mean Concentrations on the Excursion Frequency

properties, and when the effluent concentrations are water quality limited, the excursion frequencies do not change substantially with an increase in discharge flow. This observation is true for all of the other simulations of this type. This result occurs because the parameters of the probability distributions of F and C do not change as the discharge flow is increased. For the cases in which the effluent limits are water quality based, and F is the only variable allowed to have stochastic properties, the only possibility of an excursion occurs when the flow is less than the 7Q10 low flow. Since the probability of having a flow less than the 7Q10 does not change as q is increased, the excursion frequency does not increase. A similar situation occurs under water quality based effluent limits when only C is allowed to have stochastic properties. When technology based effluent limitations apply, there is an added buffer because the effluent is treated to a level beyond that which is specified by water quality based standards. This buffer allows flows less than the 7Q10 to occur without an excursion occurring.

The above results are in contrast to the case where only e and q are allowed to have stochastic properties, in which the excursion frequency increases as the discharge flow rate is increased whether or not the effluent limits are technology based or water quality based. This result occurs because the parameters of the probability distributions of q and e change as q is increased. As noted previously, an increase in the discharge flow rate q results in a decrease in e (due to the wasteload allocation). If the increase in q was balanced by a linear decrease in e, then there would be no net increase in the excursion frequency as the discharge flow rate is increased. However, the relationship between q and e is not linear.

This can be seen by differentiating the wasteload allocation equation with respect to q:

$$\frac{de}{dq} = \frac{7Q10}{q^2}\left(C - S\right) \tag{4-5}$$

From equation 4-5 it can be seen that the rate of change of e with respect to q is greatest at low values of q. As q is increased, the corresponding decrease in the effluent concentration (e), due to the wasteload allocation, becomes less and less.

Another interesting feature of Figure 4-8 is that the excursion frequency which results when only the upstream flowrate (F) is allowed to vary is very close to the percent of time the streamflow is less than the 7Q10 (on the order of one percent). This follows logically since the 7Q10 was used in the wasteload allocations to set the average effluent concentration.

Soucie (1989) showed that the excursion frequencies which result when only background concentrations (C) are allowed to vary are quite different for different pollutants, ranging from a high of about 35% for TSS in the Quinnipiac to almost 0% for total copper in the Charles. This large difference in the impact of background concentrations on the excursion frequency illustrates why it is advantageous to use the incremental excursion frequency (equation 4-1) to assess the impacts of point and non-point sources of pollution.

An important conclusion from this aspect of the research is that the excursion frequency resulting from a wasteload allocation that uses the 7Q10 low flow as the critical flow is much higher than the previously estimated value of one to two percent based on the variation of the 7Q10 flow alone. This result occurs for two reasons:

1. The variance of the downstream concentration depends on the stochastic nature of all the variables in the mass balance equation, and not just streamflow. This is described mathematically by equation 2-11. As illustrated in Figure 4-9, an increase in the variance of downstream concentrations results in an increase in the excursion frequency.

2. The average downstream concentration which results after a wasteload allocation has been performed depends on the relative size of the discharge. This is described mathematically by equation 4-2. For example, assuming all other factors are equal, the average downstream concentration which results after a wasteload allocation has been performed for a 25 MGD discharge will be greater than that for a 10 MGD discharge. As illustrated in Figure 4-7, an increase in the average downstream concentration will result in a higher excursion frequency.

V. ANALYSES OF NON-POINT SOURCE DISCHARGES

Analyses were conducted for non-point sources of pollution to determine which constituents, and under what conditions, non-point source discharges result in excursions.

Description of Excursion Frequency Simulations

The Monte Carlo simulations for non-point source discharges are similar those which were performed for point source discharges. The primary difference is that, for the non-point source simulations, the probability distributions of pollutant concentrations in the runoff did not change as the runoff flow was increased; whereas, in the point source simulations, the probability distribution of effluent concentrations changed as the effluent flow was increased, due to the determination of a wasteload allocation. In addition, for the non-point source studies, the runoff flow rate was varied as a function of the drainage area using the rational formula.

The probability distribution of rainfall intensities (I) were used in the rational formula, with the coefficient of variation reduced by 15 percent to account for the fact that not all rainfall contributes to runoff. A value of 0.35 was used for the runoff coefficient (R) since this was the median value of runoff coefficients determined as part of the National Urban Runoff Program (EPA, 1983b). Sensitivity of the results to changes in this value are discussed later in the chapter.

Simulations were performed for the rivers and constituents shown in Table 3-1, with the exception of ammonia in the Quinnipiac River. Two broad categories of non-point source discharges were considered: urban runoff with combined sewers (CSO), and urban runoff with separate (storm) sewers. The median and coefficient of variation of the pollutants in storm sewers and combined sewer overflows which were used in the simulations are listed Table 3-6. These concentrations were assumed to be lognormally distributed (EPA, 1983b; Driscoll, 1986). The parameters of the lognormal distributions ($\mu(y)$ and $\sigma(y)$ of equation 2-2) were obtained from equations 2-3 to 2-5 using the values listed in Table 3-6.

To make comparison with the results of the point source analyses easier, the urban area sizes (in acres) for which non-point source simulations were run were converted to mean discharge flows (in MGD) using the rational formula (equation 2-18) and multiplying by an appropriate conversion factor. For the upcoming results, the nomograph shown in Figure 5-1 provides an easy comparison between area and discharge if the runoff coefficient is held constant at 0.35.

The water quality standards used in the non-point source excursion frequency analyses are the same as those used in the point source analyses for total copper, TSS, and BOD_5. A fecal coliform standard of 1000 per 100 ml was used in the following analyses.

For point source discharges, the incremental excursion frequency (Δ) was defined as:

$$\Delta = Pr(T > S) - Pr(C > S) \tag{4-1}$$

where:

Pr( ) = probability of

S = water quality standard

T = concentration of pollutant downstream

C = concentration of pollutant upstream

Rearranging equation 4-1 and substituting into equation 2-20 yields an equation for the incremental excursion frequency for non-point source discharges:

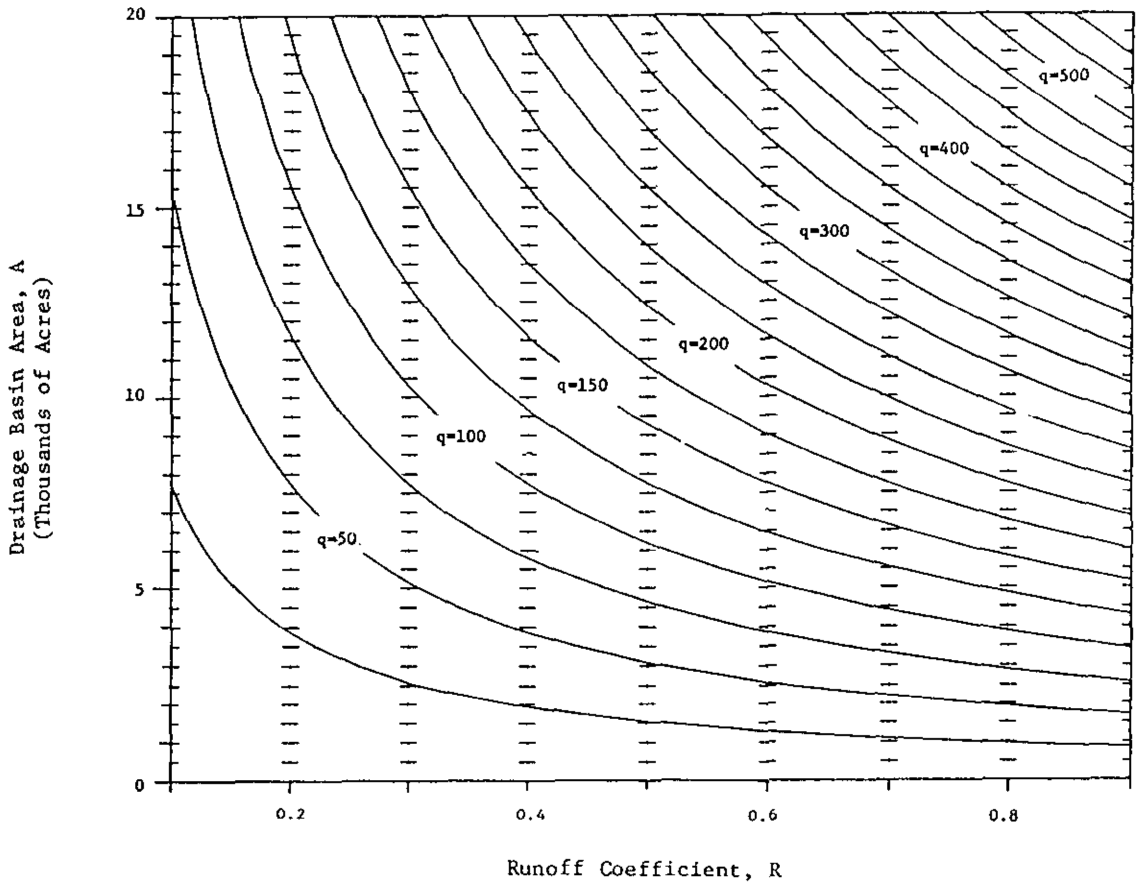


Figure 5-1: Nomograph Showing Calculated Values of Mean Runoff Flow (q, in MGD) for Values of Runoff Coefficients (R) and Drainage Basin Areas (A, in thousands of acres)

$$\Delta = f[\Pr(T_r > S) - \Pr(C > S)] \tag{5-1}$$

where:

f = fraction of time discharge occurs (equation 2-19)

T_r = concentration of pollutant downstream during runoff events

Equation 5-1 represents the contribution of the discharge to the total excursion frequency, which includes both wet and dry periods.

Another framework for analyzing the excursion frequency is as the number of events which result in an excursion during a specified time period (i.e., per year). For intermittent, non-point source discharges an event is a storm, while for continuous, point source discharges, an event is most appropriately defined as a day. The incremental excursion frequencies resulting from point and non-point sources can be converted to the average number of excursions per year. The average number of days per year for which excursions occur due to point source discharges is obtained by multiplying the number of days in a year by the incremental excursion frequency:

$$N_p = (365 \text{ days/year})(\Delta) \tag{5-2}$$

Similarly, the average number of hours per year for which excursions occur due to non-point source discharges is obtained by multiplying the number of hours per year by the incremental excursion frequency:

$$N_h = (8760 \text{ hrs/year})(\Delta) \tag{5-3}$$

The average number of storms for which excursions occur can be obtained by dividing the average number of hours per year for which excursions occur (N_h) by the mean storm duration ($\mu(d)$):

$$N_s = N_h \;/\; \mu(d) \tag{5-4}$$

Combining equations 5-3 and 5-4 yields the average number of storms resulting in excursions:

$$N_s = \left(\frac{8760 \text{ storms/year}}{\mu(d)}\right)(\Delta) \tag{5-5}$$

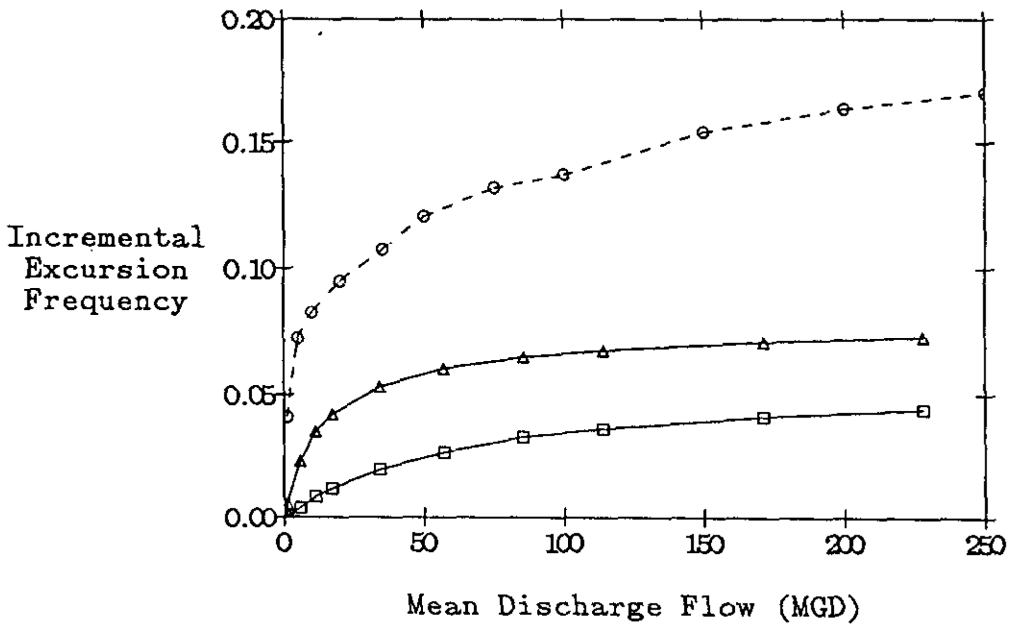
Equations 5-2 and 5-5 can be used for point and non-point source discharges, respectively.

<u>Non-Point Source Excursion Frequency Analyses</u>

The results of the combined sewer overflow and storm sewer excursion frequency simulations of total copper discharges into the Quinnipiac River are plotted in Figures 5-2, for illustrative purposes. The results of total copper in the Blackstone, Charles, Connecticut, and Merrimack; TSS in the Connecticut and Quinnipiac Rivers; Fecal coliform in the Blackstone, Charles, Connecticut, Merrimack, and Quinnipiac; and BOD_5 into the Blackstone are shown in the Appendix. Where applicable, the results of the point source analyses are plotted on the same graphs for comparison. In these figures, the incremental excursion frequencies are plotted on the upper panel, and the number of excursion events per year are plotted on the lower panel.

Several observations can be made from the results presented graphically in Figure 5-2 and those in the Appendix, including: (1) the differences between the point and non-point source discharge excursion frequencies, (2) the difference in the magnitude of the excursions resulting from storm sewer and combined sewer overflow discharges, and (3) the relative magnitudes of excursion frequencies for different mean discharge flows.

An examination of the incremental excursion frequencies resulting from point source discharges, combined sewer overflows, and storm sewer



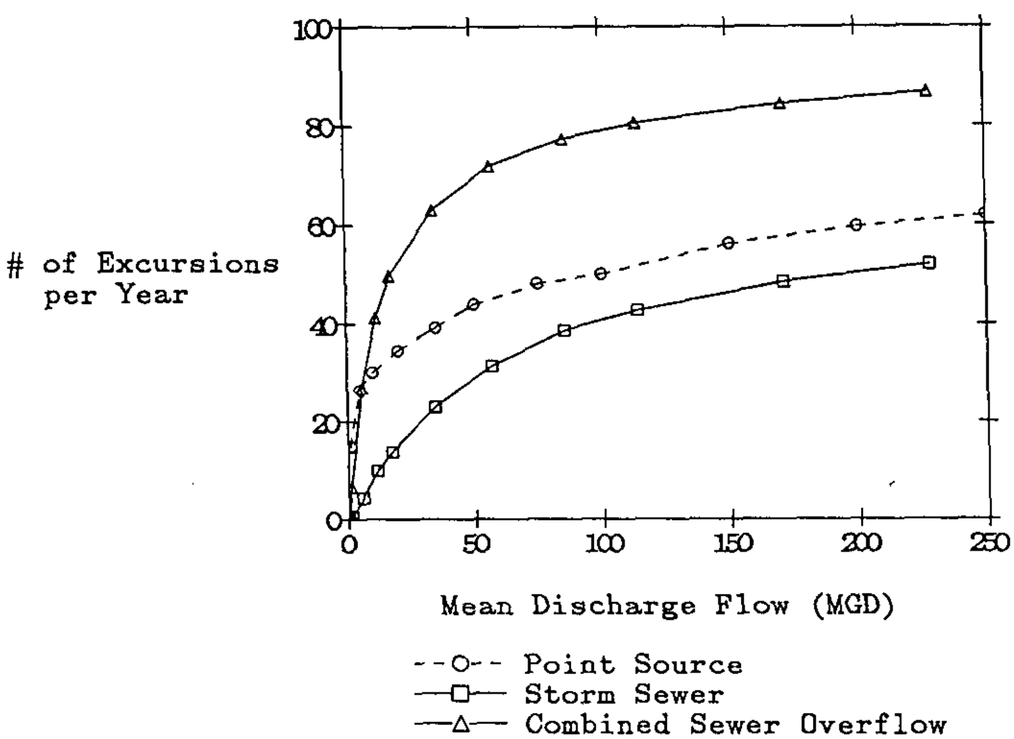


Figure 5-2: Excursion Frequencies for Total Copper in The Quinnipiac

discharges shows that the point source discharges result in a higher incremental excursion frequency than the non-point source discharges for copper and BOD_5. The opposite is true for TSS for all discharge flows in the Connecticut river (upper panel, Figure A-5) and for the lower discharge flows in the Quinnipiac river (upper panel, Figure A-6).

The incremental excursion frequencies for BOD_5 and total copper were higher for the point source discharges than for the non-point source discharges because point source discharges occur all of the time, while the non-point source discharges occur only about ten percent of the time (the fraction of time it rains). A similar result was not observed for discharges of TSS because the coefficient of variation for TSS which was used in the point source simulations was 0.332, which is much less than the values of 1.1 and 1.5 which were used in the simulations of storm sewers and combined sewer overflows, respectively. As discussed in the previous chapter, a higher variance results in a higher excursion frequency. Thus, even though the non-point source discharges of TSS occur only ten percent of the time, the incremental excursion frequencies due to non-point source discharges are higher than those for point source discharges.

An examination of the number of excursions per year (lower panel, Figure 5-2 and those in the appendix) shows that for discharges of BOD_5 in the Blackstone, total copper in the Charles, and total copper in the Quinnipiac, the number of excursion which occur due to combined sewer overflow events exceeds the number of excursions for point source events. This situation occurs because an event for a point source discharge is a day; whereas, an event for a non-point source discharge is a storm.

Mathematically, this can be seen by solving equations 5-2 and 5-5 for the incremental excursion frequency (Δ), setting them equal, and then solving for N_s:

$$N_s = \frac{N_p\ (8760)}{\mu(d)\ (365)} \qquad (5\text{-}6)$$

where:

N_s = the number of excursion events (storms) which result due to non-point source discharges,

N_p = the number of excursion events (days) which result due to point source discharges, and

$\mu(d)$ = the mean storm duration in hours.

From the analysis of rainfall data the average storm duration ($\mu(d)$) was found to be equal to 7.4 hours. Therefore, equation 5-6 reduces to:

$$N_s = 3.25(N_p) \qquad (5\text{-}7)$$

Thus, for the same incremental excursion frequency, a non-point source discharge will result in approximately three times as many excursion events as a point source discharge. This result should be viewed with caution, however, since the use of a one day interval is somewhat arbitrary. Point source discharges are usually continuous, and, if data were available the number of excursions could be presented as the number of one-hour (or even one-minute) excursions.

The differences in the number of excursion events per year resulting from combined sewer overflows and storm sewer discharges is an important result because it gives an estimate of the relative benefits of eliminating combined sewer overflows and assuming that storm sewers were in place instead. Table 5-1 is a summary of the percent improvement for different

Table 5-1: Percent Improvement in the Number of Excursion Events Which Would Result From Changing Combined Sewer Overflows to Storm Sewers at Discharge Flows of 1 and 228 MGD

River	Constituent	Discharge Flow Rate (MGD) 1	228
Blackstone	BOD_5	97	59
Quinnipiac	Total Copper	89	40
Charles	Total Copper	98	49
Blackstone	Total Copper	84	59
Merrimack	Total Copper	[1]	88
Connecticut	Total Copper	[1]	92
Quinnipiac	Fecal Coliform	52	0[2]
Charles	Fecal Coliform	69	0[2]
Blackstone	Fecal Coliform	72	0[2]
Merrimack	Fecal Coliform	73	19
Connecticut	Fecal Coliform	83	38
Quinnipiac	TSS	23	28
Connecticut	TSS	[1]	32

1. The number of excursion events are insignificant at this flow
2. There was actually a slight decrease in the percent improvement, which was taken as zero

flow rates resulting from such a change. The rivers listed in the table are in order of increasing mean flow rate. The values in the table were calculated by taking the difference between the number of CSO and storm sewer excursion events per year, and dividing that difference by the number of excursion events which result due to CSOs, and then multiplying by one hundred. It should be noted that the percent improvement is based on the relative difference in the number of excursions resulting from CSOs and storm sewers. The actual number of excursion events which would be eliminated would be much different. For example, a 75 percent improvement from 4 excursion events is equal to 1, while a 50 percent improvement from 40 excursion events is equal to 20. An examination of this Table shows that:

1. The percent improvement is generally greater at lower discharge flows (1 MGD) than at higher discharge flows (228 MGD). The only exception to this trend was TSS in the Quinnipiac. This aspect is discussed further below.
2. The percent improvement is generally greater in rivers which have larger mean streamflows. The only exception to this trend was total copper in the Blackstone. This result is also discussed below.
3. The three rivers which have the lower mean streamflow (Quinnipiac, Charles, and Blackstone), all showed no improvement for fecal coliform at discharge flows of 228 MGD. This occurred because the incremental excursion frequency due to storm sewers reached its maximum theoretical value. From equation 4-1, the maximum incremental excursion frequency is equal to one minus the background excursion frequency. Since the maximum value is the same for both CSOs, and storm sewers, there was no difference between the two. In general, at very high mean discharge flows, the incremental excursion frequency is close to its maximum value.

An examination of Figures 5-2, and A-1 to A-12 shows that the basic shape of the curves resulting from plotting the non-point source incremental excursion frequencies against the magnitude of the mean discharge size is the same for each of the constituents tested. It should be noted that the analyses were run to determine what effects different size discharges would have on the incremental excursion frequency. Thus, although the results are plotted against the mean discharge flow, the mean discharge flow is not a variable which would change under ordinary circumstances, although increased urbanization could have this effect. In these curves, there are relatively large differences in the incremental excursion frequencies in the lower range of discharge flow rates, while at higher discharge sizes, there are relatively little differences in the incremental excursion frequencies. For example, referring to lower panel of Figure 5-2 (total copper in the Quinnipiac), a mean CSO discharge flow of one MGD results in about seven excursions per year, while a mean CSO discharge of five MGD results in about 27 excursions per year, a difference of about 20. However, a mean discharge size of 150 MGD results in about 80 excursions per year and a mean discharge size of 200 MGD results in about 82, a difference of only two. Thus, these curves can be thought of as having two distinct portions: an initial portion in which the incremental excursion frequency is highly sensitive to the mean discharge flow, and a latter portion, in which the incremental excursion frequency is not sensitive to the mean discharge flow.

The division of the curves into two sections (sensitive and insensitive) can be used to help predict the possible result of storage/treatment schemes for non-point source discharges. If temporary storage were provided for non-point source discharges, and the accumulated volume was released gradually, the effluent flowrate would be reduced considerably. This reduction in flowrate would result in a corresponding decrease in associated excursion frequencies. The resulting decrease would be far greater for low-flowrate discharges than for large discharges. Therefore, many small storage basins may be more effective, in terms of the number of excursions, than a few large ones. This suggestion however, does not consider the treatment (sedimentation) associated with storage, nor the economic economies of scale.

Additional observations regarding the incremental excursion frequency curves are noted below:

1. At low mean discharge flows, the change in the incremental excursion frequencies resulting from an increase in the mean discharge flow is greater for discharges with higher pollutant concentrations.
2. At high mean discharge flows, the change in the incremental excursion frequency with increasing mean discharge flow is not highly dependent on concentration. This result also explains why the differences between CSOs and storm sewers noted previously were greatest at low mean discharge flows.
3. For the two rivers which have the highest mean streamflow (Merrimack and Connecticut), all of the mean discharge flows are in the first portion of the excursion frequency curves, in which an increase in the mean discharge flow results in a large increase in the incremental excursion frequency. This is why the relative differences between CSOs and storm sewers noted previously were generally greater for rivers with higher mean flowrates.

Illustrating the excursion frequencies on the basis of mean discharge flow allows an easy comparison between point and non-point source discharges. However, the mean discharge flow from non-point source discharges is computed on the basis of the rational formula and includes the runoff coefficient, drainage basin area, and rainfall intensity. The stochastic nature of runoff is incorporated using the distributions for rainfall intensity, while the other two parameters are constant. Selection of different values for either the drainage basin area or the runoff coefficient will result in different values of runoff flow (q). Plots could be developed showing variation of either of the parameters along the horizontal axis. An example is shown in Figure 5-3, where the drainage area is held constant (at 10,000 acres) and results are shown for different values of the runoff coefficient. The plot displays the impact that different runoff coefficients would have on the excursion frequency.

Rather than develop plots for other drainage area sizes, the nomograph shown in Figure 5-1 could be used in conjunction with Figures 5-2 and A-1 to A-12 to determine the incremental excursion frequency which would result from any combination of drainage basin area and runoff coefficient. By holding either the drainage basin area (A) or the runoff coefficient (R) constant, the other variable can be varied to determine the value of the mean runoff flow (q). This value in turn can then be used with any of the figures for non-point source runoff that have q on the horizontal axis. For example, holding the area constant at 1500 acres, a coefficient of 0.1 results in a discharge of about 4.8 MGD, while a coefficient of about 0.6 results in a discharge of about 29 MGD. With these discharge rates, the excursion frequencies can be determined for a particular river. For example, referring to Figure 5-2, storm sewer discharges of total copper in the Quinnipiac at mean discharge flows of 5 and 29 MGD result in incremental excursion frequencies of about 0.005 and 0.02, respectively.

Sensitivity of Results

The sensitivity of the results to changes in the values of crucial parameters used in the analyses are discussed in this section. Since methods other than the Monte Carlo simulation are more easily used to study the sensitivity of various assumptions, the moment approximation method was used in the analyses. The following discussion provides a comparison between the Monte Carlo simulation approach and the moment approximation method for solving the lognormal probability model.

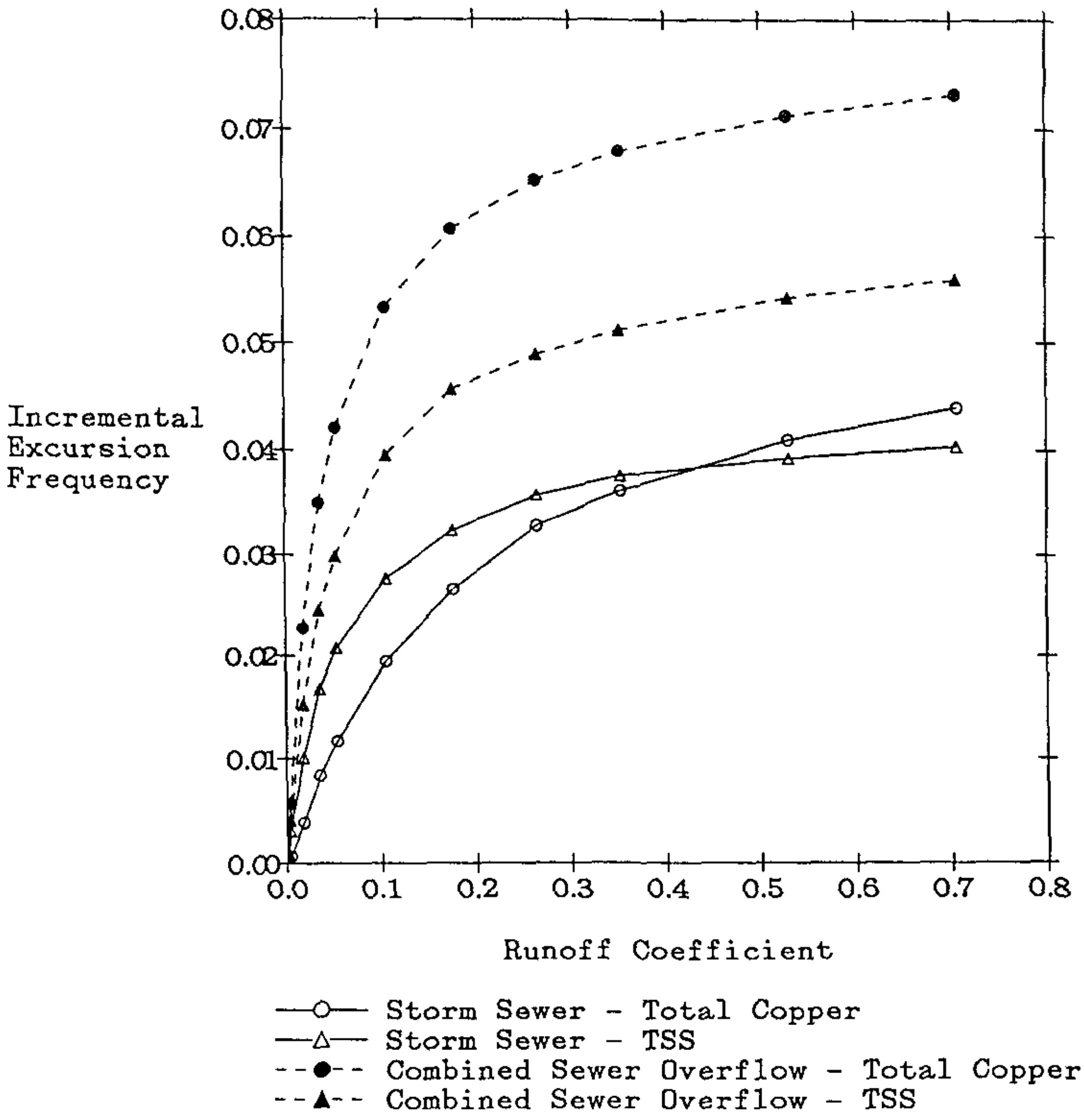


Figure 5-3: Excursion Frequencies for Different Runoff Coefficients on a 10000 Acre Drainage Basin for Storm Sewer and CSO Discharges of Total Copper to the Quinnipiac River

All of the previous results are based on Monte Carlo analyses, and used distributions for input parameters that best fit the raw data. For effluent flow and concentrations these were lognormal distributions, but for upstream flow and concentrations the distributions differed for different rivers and constituents. The moment approximation method is based on the assumptions that the probability distributions of the input variables (F, C, q, and e) are lognormally distributed and that there is no correlation between flow and concentration.

The moment approximation method was used to check the results of the Monte Carlo simulations. This check was done for at least one constituent in each of the rivers analyzed. The results presented in Figure 5-4 for storm sewer discharges of total copper to the Quinnipiac are typical of the results obtained for all of the analyses. In this figure, the incremental excursion frequencies resulting from the moment approximation method are plotted with those using Monte Carlo simulation (results from previous section). As can be seen from this figure, even though the Monte Carlo simulations used different distributions for upstream flow and concentration, the results were not significantly different than those determined using the moment approximation method at low mean discharge flows. Based on the comparisons which were done for the Quinnipiac river and other rivers, it appears that the results are not overly sensitive to the distributions which are used for the input variables. In addition, the moment approximation method can be used with confidence to study the sensitivity of certain input parameters.

As an example of how sensitivity analyses can be performed, the moment approximation method was used to analyze the case of storm sewer discharges

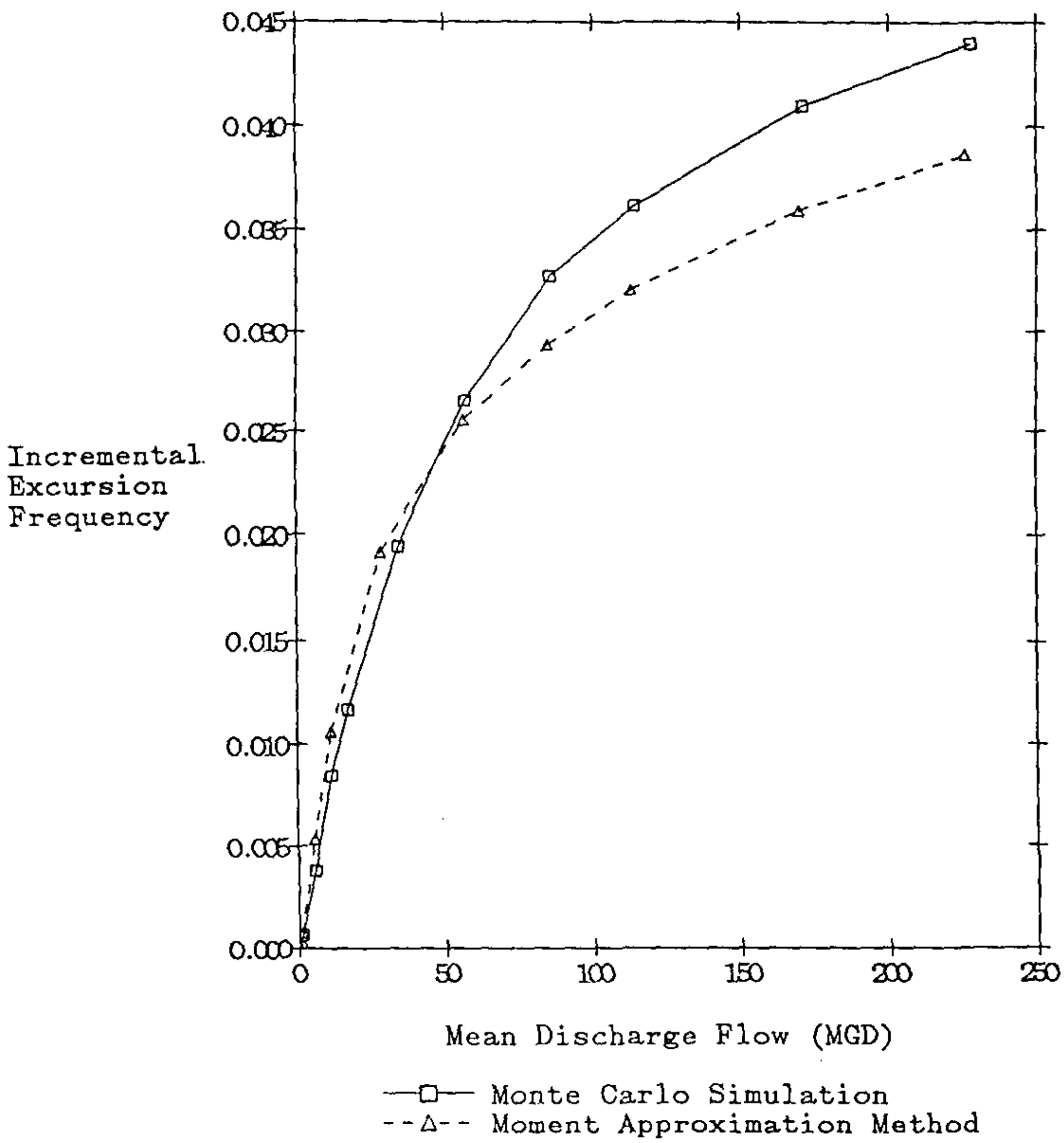


Figure 5-4: Comparison of Results from the Moment Approximation Method and Monte Carlo Simulation for Storm Sewer Discharges of Total Copper to the Quinnipiac

of total copper to the Quinnipiac river. The parameters which were used in the moment approximation method were either increased or decreased by 15 percent, whichever resulted in the better and worse cases. The average values of the parameters, along with the values that were increased and decreased by 15 percent, listed in Table 5-2. The incremental excursion frequencies resulting from the better, average, and worse cases are plotted in Figure 5-5. As can be seen from this figure, the largest difference between the better and worse cases was 0.015 (1.5%). Therefore, the results for this case do not appear to be overly sensitive to small errors in the parameters used in the analyses.

Table 5-2: Input Parameter Values Used to Analyze Better and Worse Cases for Storm Sewer Discharges of Total Copper in the Quinnipiac River Using the Moment Approximation Method

Parameter	Average Case	Better Case	Worse Case
Runoff Coefficient (R)	0.35	0.30	0.40
Mean Rainfall Intensity (μ(I))	0.050	0.042	0.058
CV of Rainfall Intensity	1.171	0.995	1.347
% to Reduce CV of Rainfall Intensity By	15.0	17.2	12.8
Average Storm Duration (μ(d))	7.4	6.3	8.5
Average Time Between Storms ($\mu(\delta)$)	71.0	81.7	60.4
Mean Upstream Concentration (μ(C))	14.13	12.01	13.81
CV of Upstream Concentration	0.392	0.333	0.451
Mean Upstream Flow (μ(F))	234	269	199
CV of Upstream Flow	0.929	0.790	1.068
Mean Effluent Concentration (μ(e))	34.0	28.9	39.1
CV of Effluent Concentration	0.750	0.638	0.862

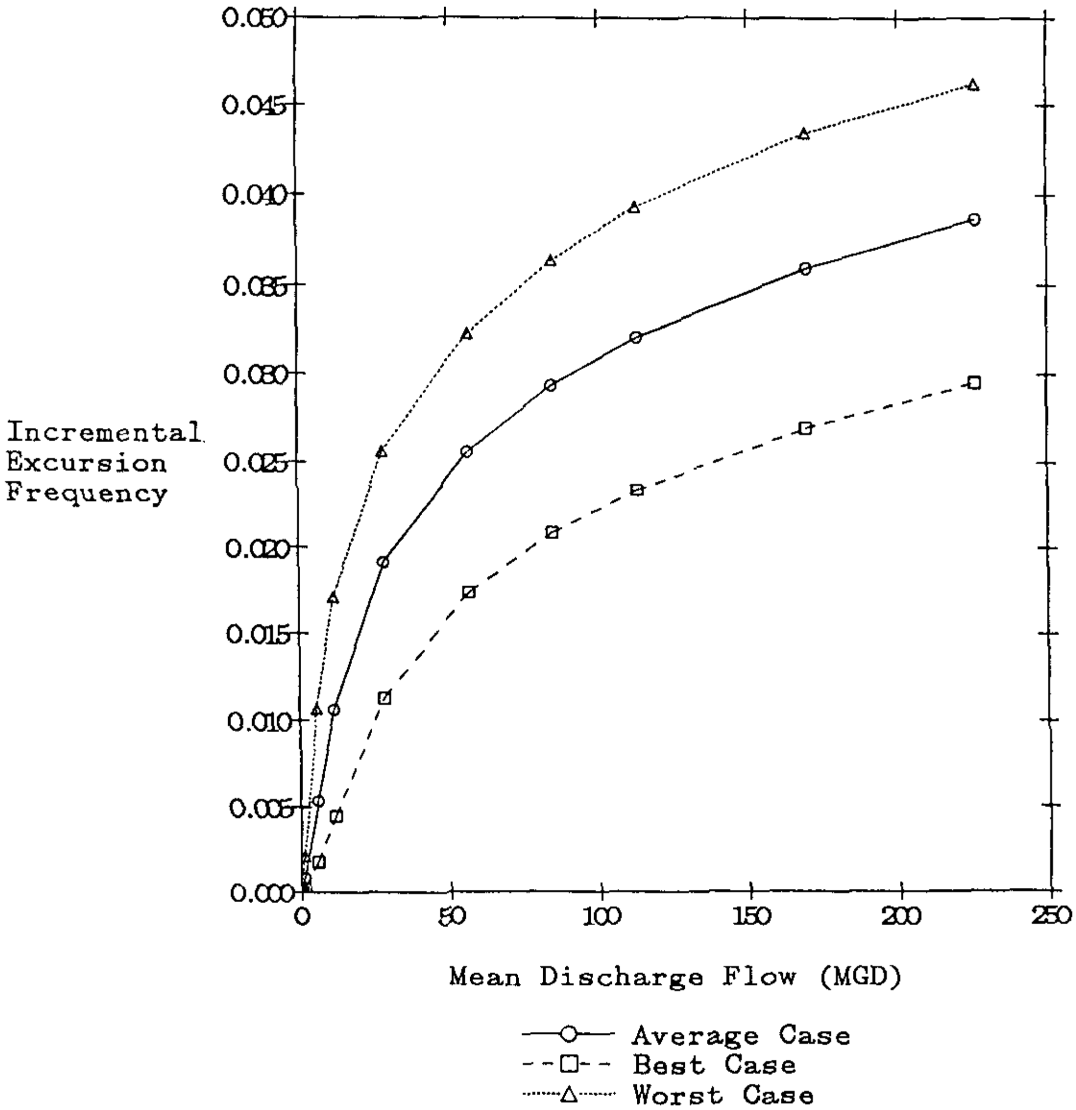


Figure 5-5: Average, Better, and Worse Cases of Storm Sewer Discharges of Total Copper in Quinnipiac (Computed Using the Moment Approximation Method)

VI. SUMMARY AND CONCLUSIONS

The overall objective of this research was to provide the basis for uniform administration of water quality statutes for point and non-point sources of pollution. Water quality standards for point source discharges have traditionally been enforced using the 7Q10 low flow as the critical streamflow. It is generally agreed that this procedure provides a high degree of protection for the designated uses of receiving waters, and that a similar level of protection is desirable for non-point source discharges.

Summary

The approach taken in the research was to assess the excursion frequency resulting from using the 7Q10 low flow to regulate point source discharges, and then compare that excursion frequency to the excursion frequency resulting from non-point source discharges. To accomplish this comparison, Monte Carlo simulations were employed, utilizing probability distributions for the four input parameters to the traditional mass balance equation. The resulting probability distribution for downstream concentration allowed determination of excursion frequencies. Excursion frequencies for point source discharges, storm sewer discharges and combined sewer overflows were determined using input values representative of Massachusetts conditions.

The analysis considered only excursion frequency and number of excursions. Other considerations, such as the severity and duration of excursions, are equally important, particularly when considering the viability of biota.

Conclusions

Several interesting conclusions can be drawn from this research which have bearing on the possible regulation of non-point discharges. One of the most interesting results is the assessment that excursion frequencies for point source discharges are considerably higher than previously thought. Based solely on variation of streamflows, water quality excursions are in the range of one to two percent of the time. However, when all of the variables in the mass balance are allowed to have stochastic properties, the excursion frequency will almost always be higher than for the cases in which one or more of the variables are held constant. This result occurs because the variance of the downstream concentration is directly related to the variance of all the variables in the mass balance equation. An increase in the variance of the downstream concentrations results in an increase in the excursion frequency, even though there may not be any change in the mean value of the downstream concentration. Because the excursion frequency depends on both the mean and variance of all the variables in the mass balance equation, the excursion frequency can be quite different for different pollutants in same river, or for the same pollutant in different rivers.

Although the protection offered by the current regulation of point source dischargers is not as high as previously thought, it is still sufficient to insure maintenance of adequate water quality. However, the "margin of safety" provided by the approach may not be as great as previously thought. As such, if variable discharge permits (based on flowrate, season, etc.) were to be considered, caution should be exercised in their adoption to insure an adequate margin of safety is maintained.

The results of this research show that point source discharges have higher incremental excursion frequencies than either storm sewer discharges or combined sewer overflows, when the coefficients of variation of the pollutant concentrations are similar. This result is, in large part, due to the fact that non-point source discharges occur only about 10 percent of the time.

This conclusion raises questions about the emphasis given the rapid, short term impacts, of non-point source discharges as opposed to long term effects (eg., bio-accumulation of toxics) and short term impacts due to scour and resuspension of pollutants. Given the recognized adverse impact of non-point sources of pollution, these other effects should not be minimized.

Other, more specific results of the research, include the following aspects:

1. A large number of the water quality data sets were best fit by the log-normal distribution. Therefore, it is concluded that in-stream water quality data can, in most cases, be adequately modeled using the 2-parameter log-normal distribution.
2. The background pollutant concentrations in the five case study rivers are highly variable. In some cases, such as for fecal coliform, the average background concentration actually exceeds the in-stream water quality standard. Therefore, background concentrations exert a large effect on the excursion frequency.
3. Larger mean effluent flows from point sources result in higher incremental excursion frequencies when considering both technology and water quality based effluent limitations. This situation occurs because the average downstream concentration resulting from a wasteload

allocation, is linearly related to the discharge flow ratio. Thus, if wasteload allocations were performed for two treatment plants which have different mean effluent flows, with all other factors being equal the excursion frequency will be higher for the larger plant, even though the larger treatment plant would be required to treat its waste to a higher degree.

4. For the smaller rivers analyzed (Quinnipiac, Charles, and Blackstone), the curves which result when the incremental excursion frequency is plotted against the mean runoff flow generally have two distinct portions: an initial portion in which the incremental excursion frequency is sensitive to the value of the mean runoff flow, and a latter portion in which it is not. This observation could be used to help design the best control scheme for non-point source discharges.
5. The relative benefits (as measured by percent improvement) of installing storm sewers, and thereby eliminating combined sewer overflows, is generally greater at lower mean runoff flows than at higher. This results from the fact that the incremental excursion frequency is more sensitive to differences in the discharge concentration at lower mean runoff flows. Since the pollutant concentrations are higher in combined sewer overflows than in storm sewers, eliminating combined sewer overflows (and having storm sewer discharges instead) will have a greater effect on the incremental excursion frequency at lower mean runoff flows.
6. The excursion frequencies calculated using the moment approximation method show good agreement with the results obtained using Monte Carlo simulation. Since the moment approximation method is much easier to use than Monte Carlo simulation, it is advantageous for use as a screening

tool. However, for actually calculating discharge permits, Monte Carlo simulation or continuous simulation should be used because of its greater accuracy.

Further Research

This study was intended to be preliminary in nature, providing the necessary basis for development of regulations for non-point source discharges. Further research would both augment what has been presented here and provide a more detailed look at possible regulatory approaches.

Possible research areas are outlined below:

1. Assessment of a wider range of discharges in Massachusetts could be undertaken to confirm underlying distributions for flow and constituent concentrations. This study should address a range of discharge sizes, carefully addressing coefficients of variation.
2. Excursion frequencies should be addressed in a more comprehensive manner, rather than just at the point of mixing. A comprehensive approach could include: (1) modeling of constituent behavior downstream, particularly for non-conservative pollutants, (2) inclusion of several discharges to a stream, and analyzing the combined impact, and (3) addressing the possible interactive nature of pollutants and/or treatment processes resulting in pollutant reduction.
3. Other effects of non-point source pollution should be considered in any regulatory approach. These impacts would include severity and duration of excursions, bioaccumulation and resuspension of pollutants.
4. Practical aspects of non-point source control (eg., feasibility, cost, etc.) should be addressed and linked to potential excursion frequencies

once potential storage/treatment schemes have been identified. If possible, generic relationships between flow and pollutant concentration reduction and resulting excursion frequencies should be developed.

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APPENDIX

Excursion Frequencies for

Selected Constituents in

Case Study Rivers

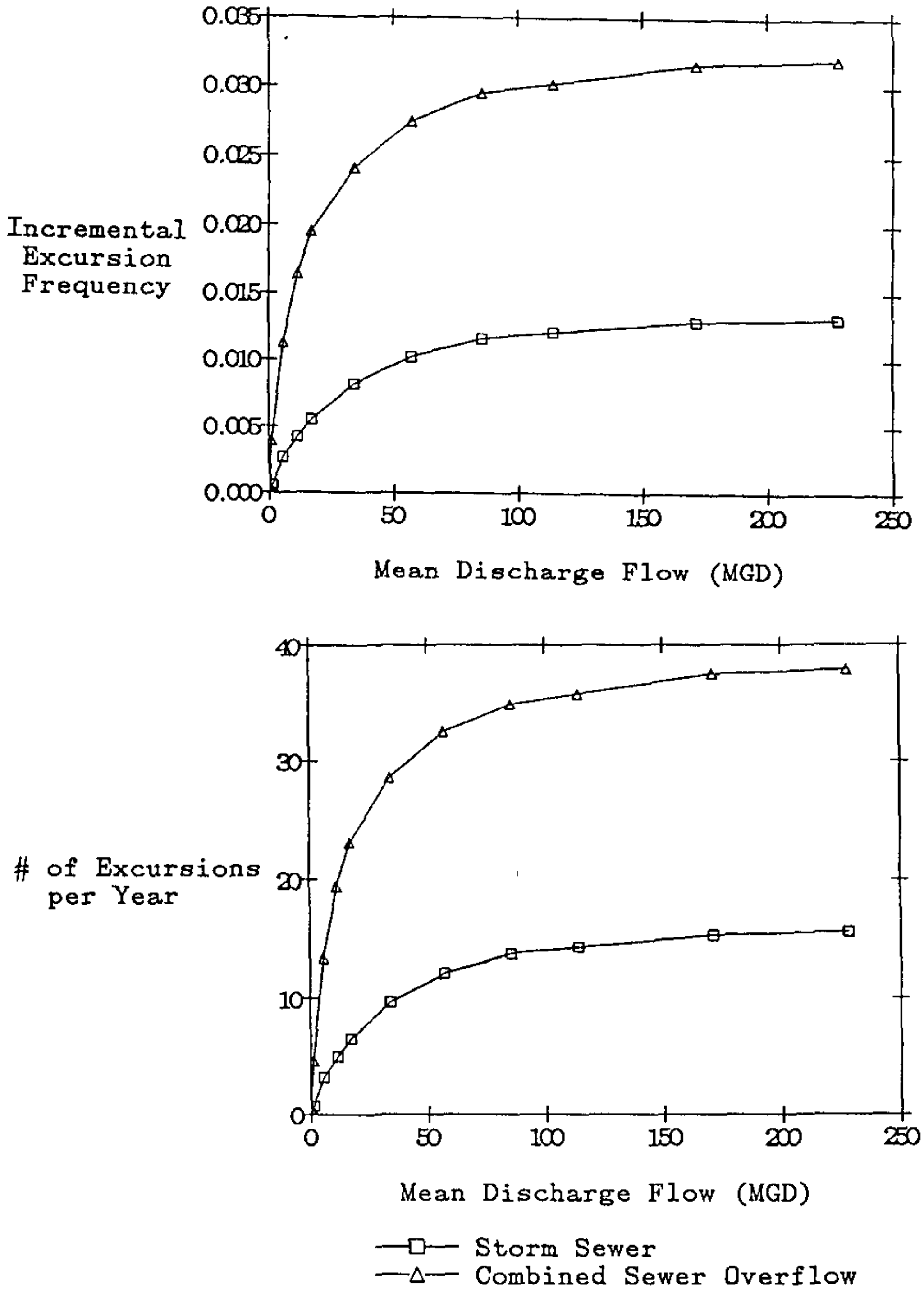


Figure A-1: Excursion Frequencies for Total Copper in the Blackstone River

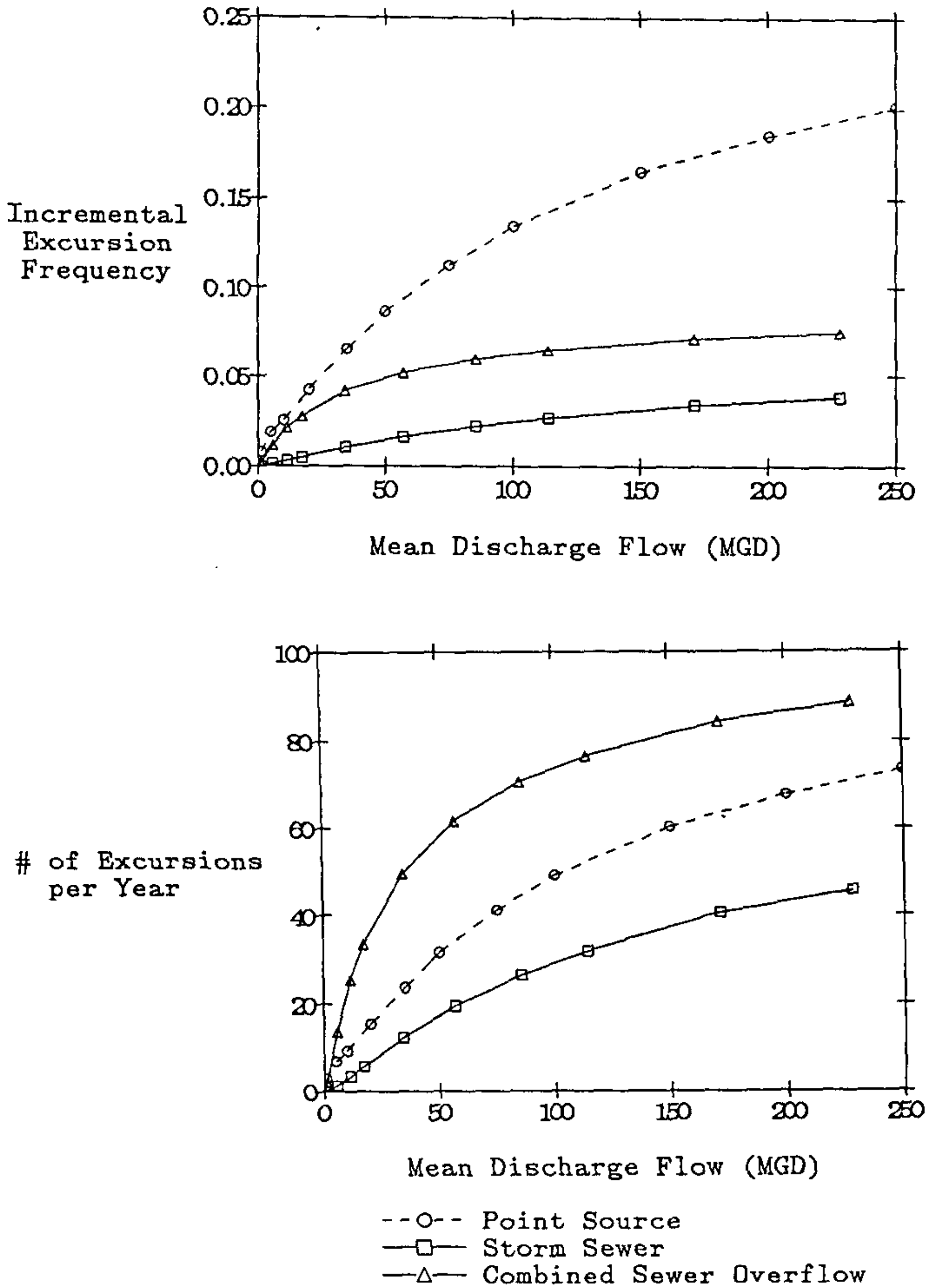


Figure A-2: Excursion Frequencies for Total Copper in the Charles River

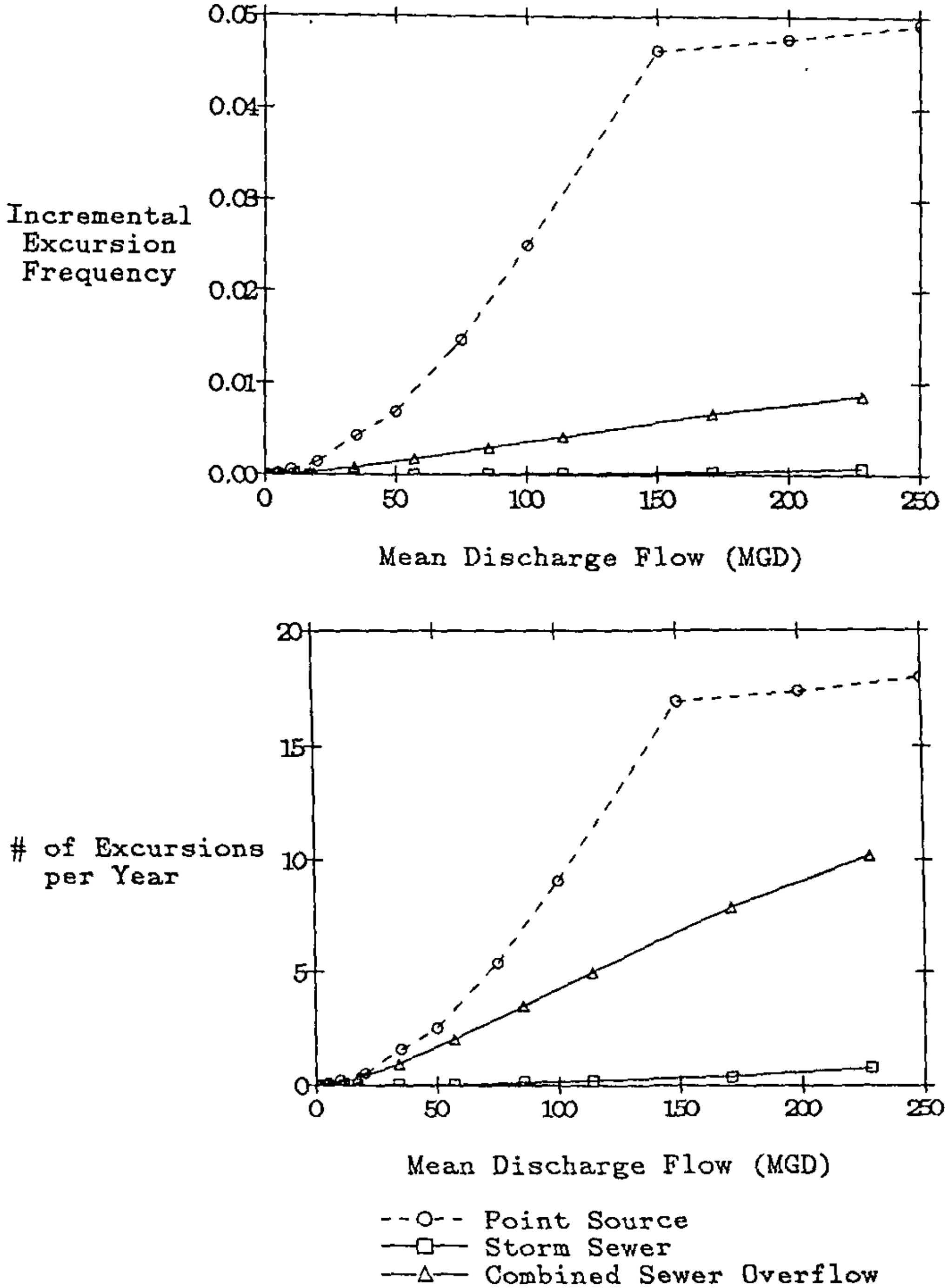


Figure A-3: Excursion Frequencies for Total Copper in the Connecticut River

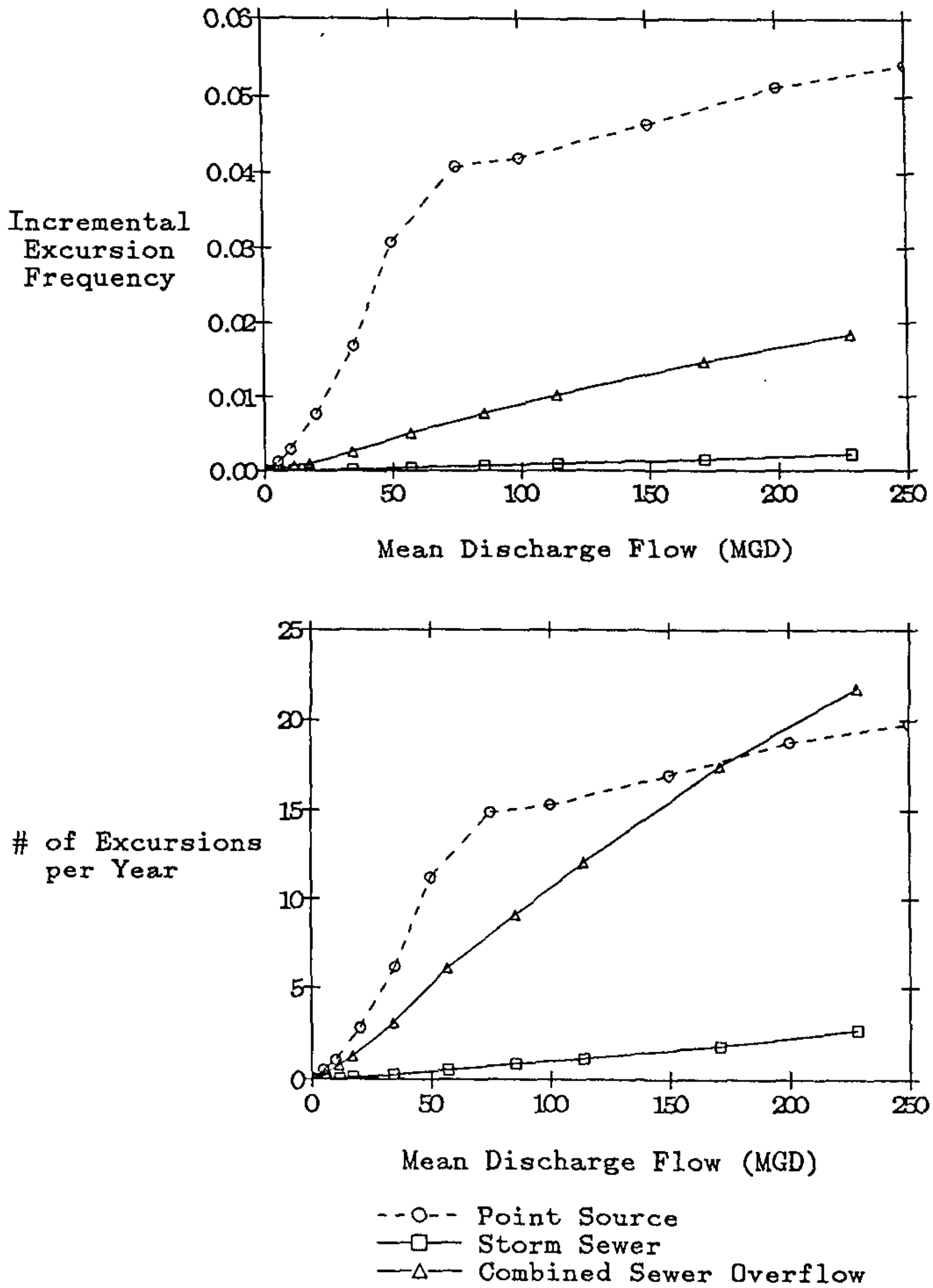


Figure A-4: Excursion Frequencies for Total Copper in the Merrimack River

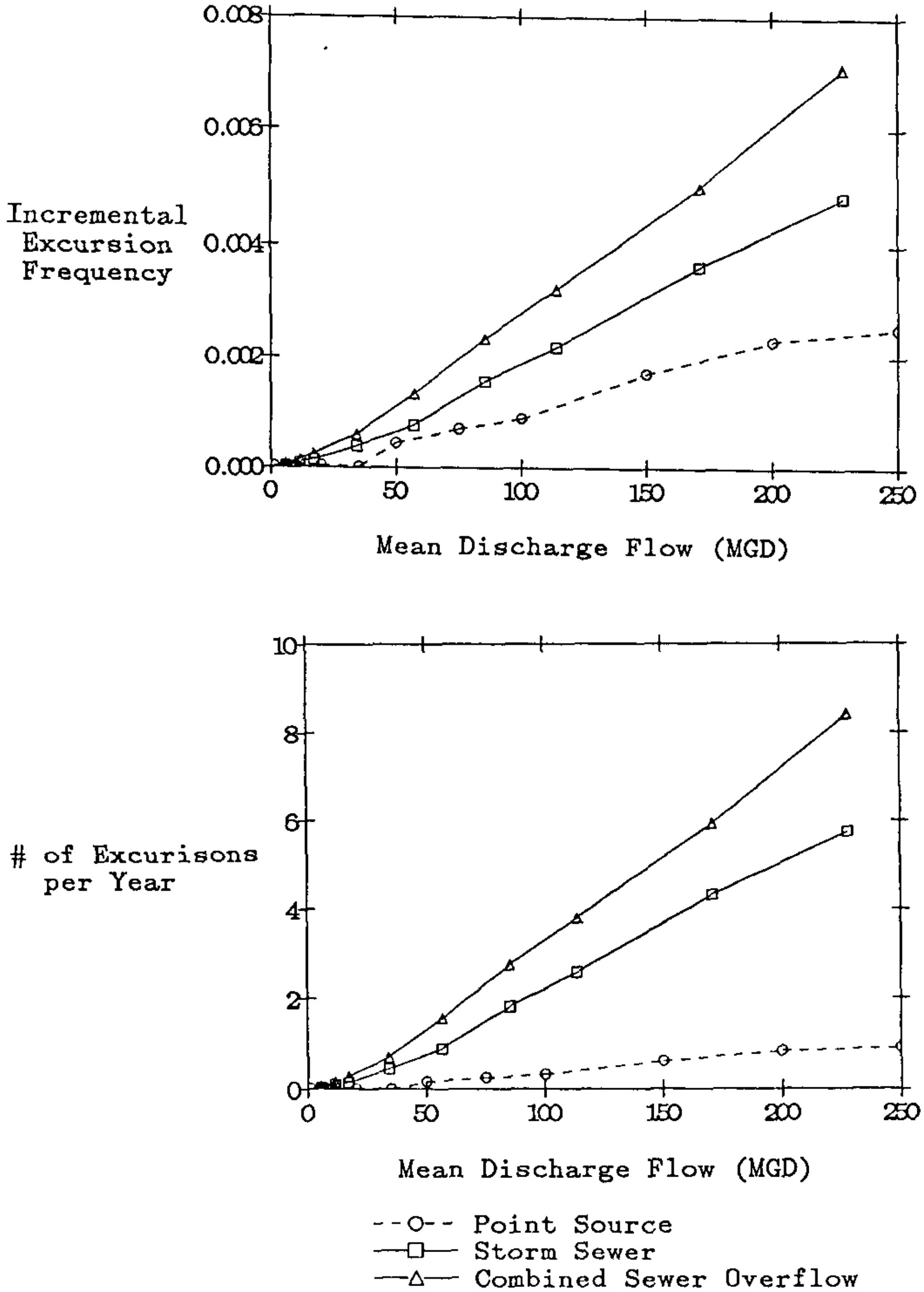


Figure A-5: Excursion Frequencies for TSS in the Connecticut River

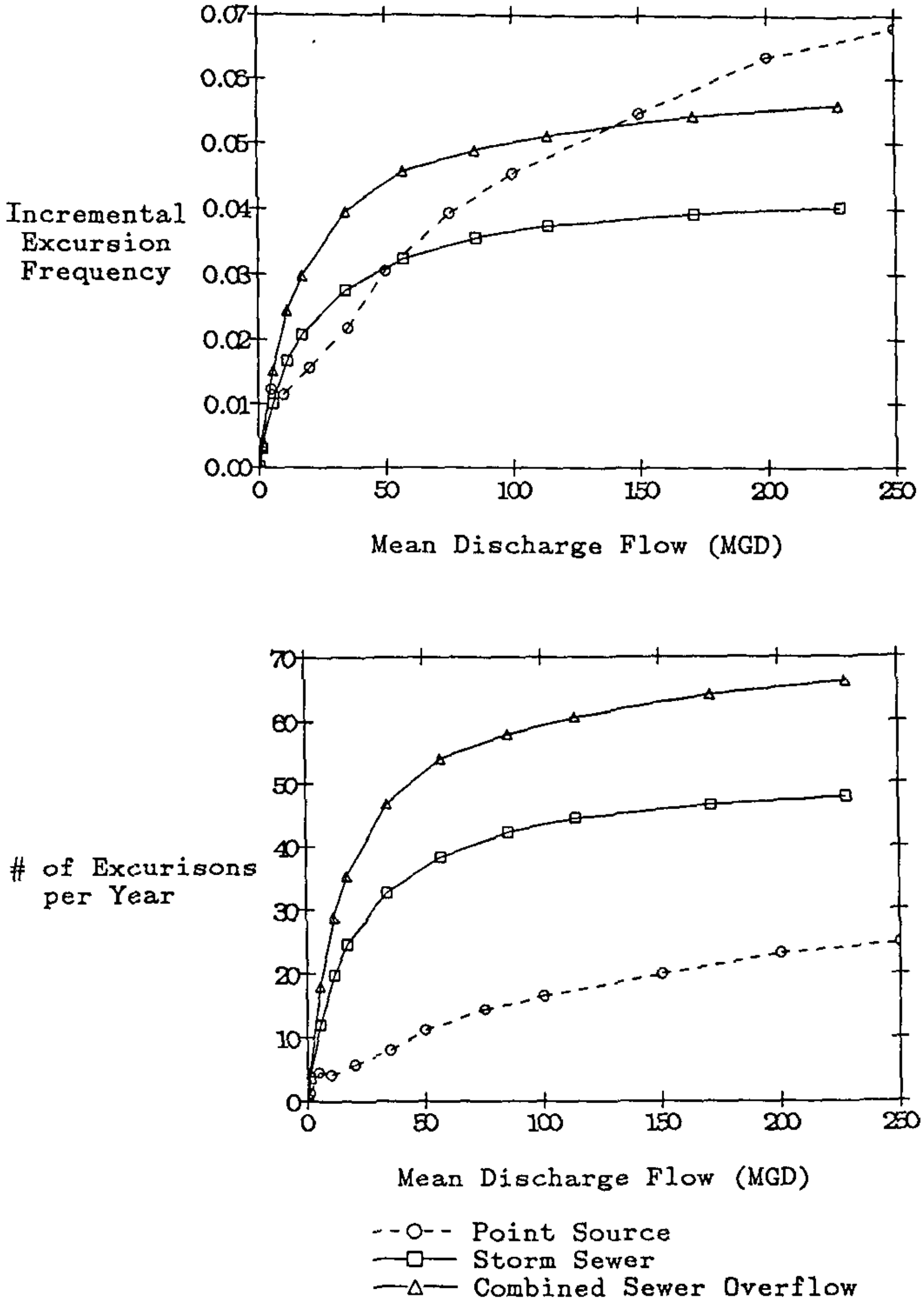
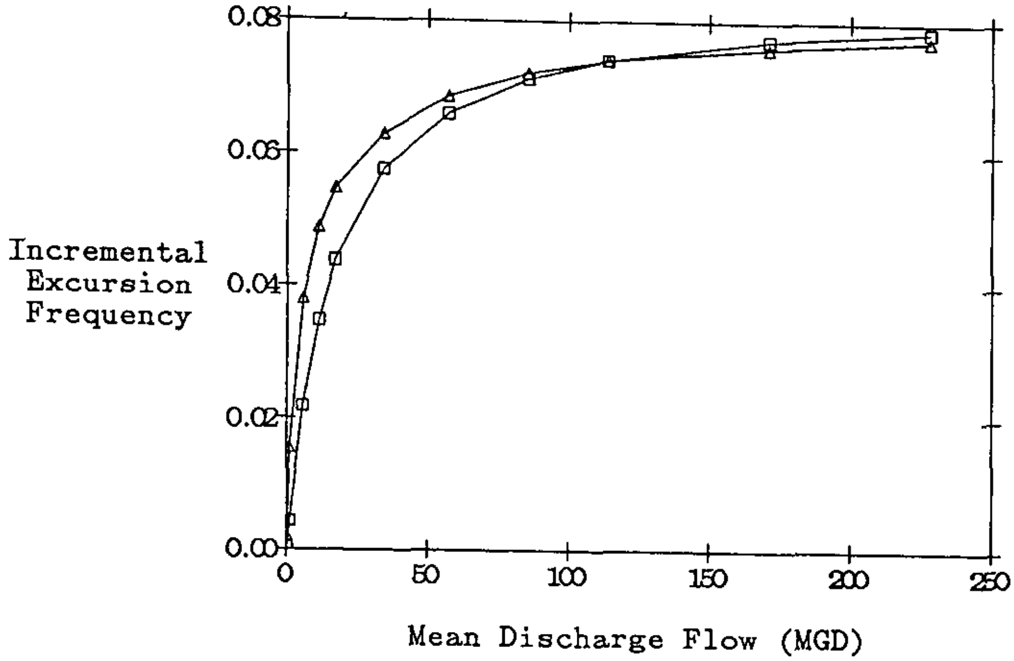


Figure A-6: Excursion Frequencies for TSS in the Quinnipiac River



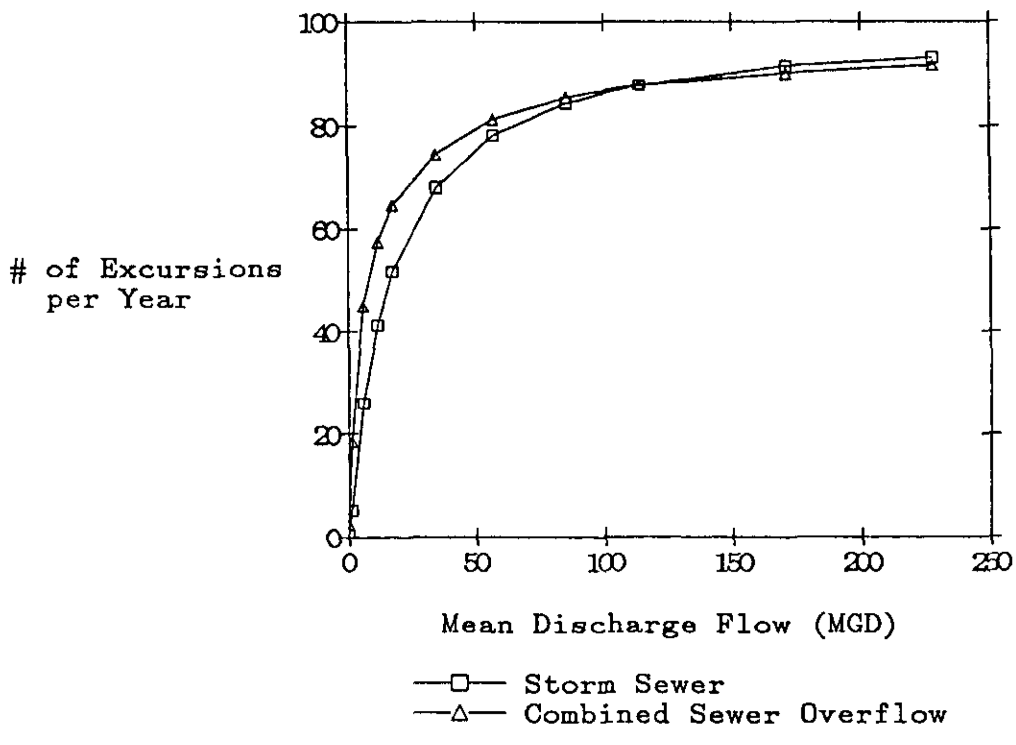


Figure A-7: Excursion Frequencies for Fecal Coliform in the Blackstone River

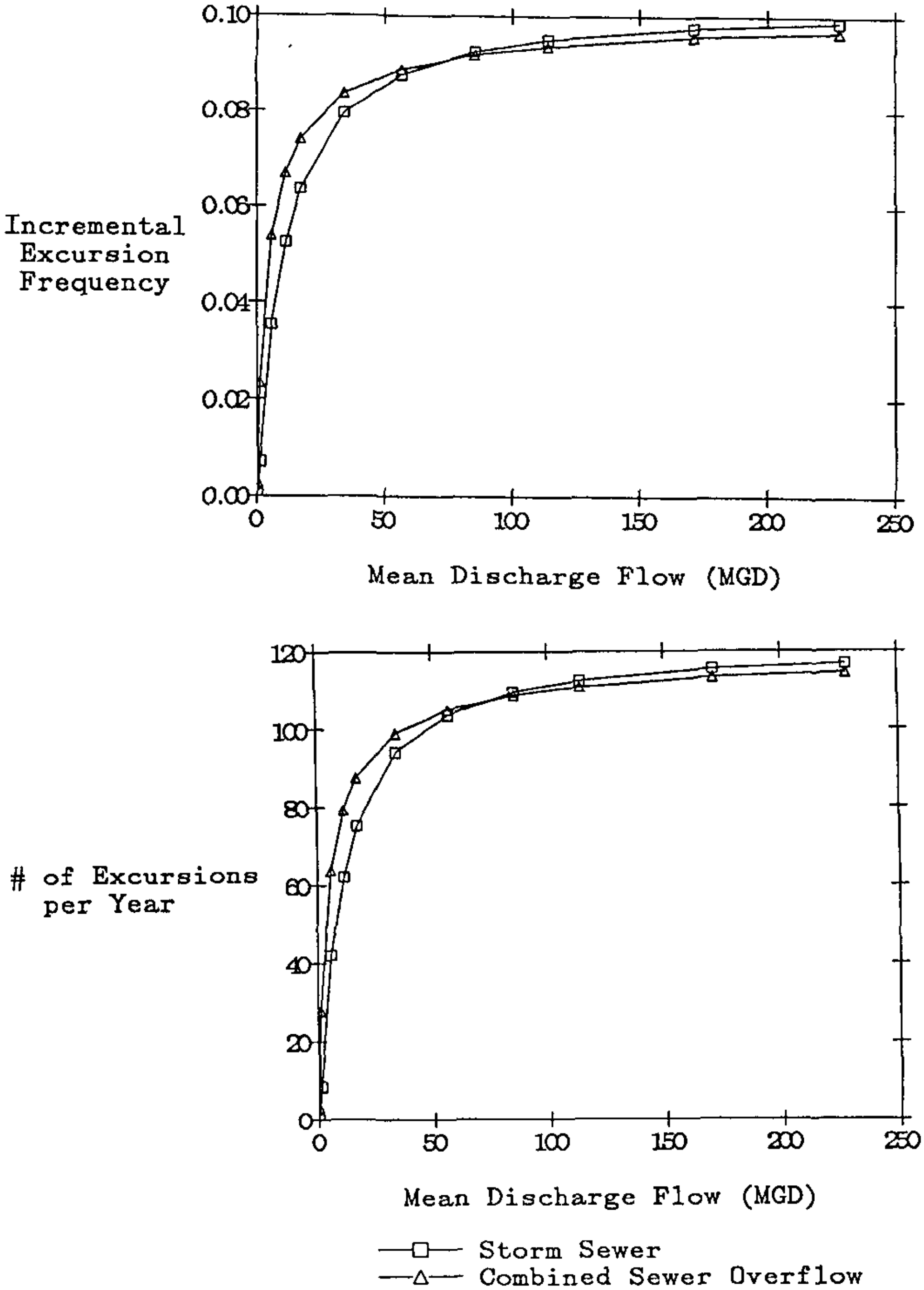
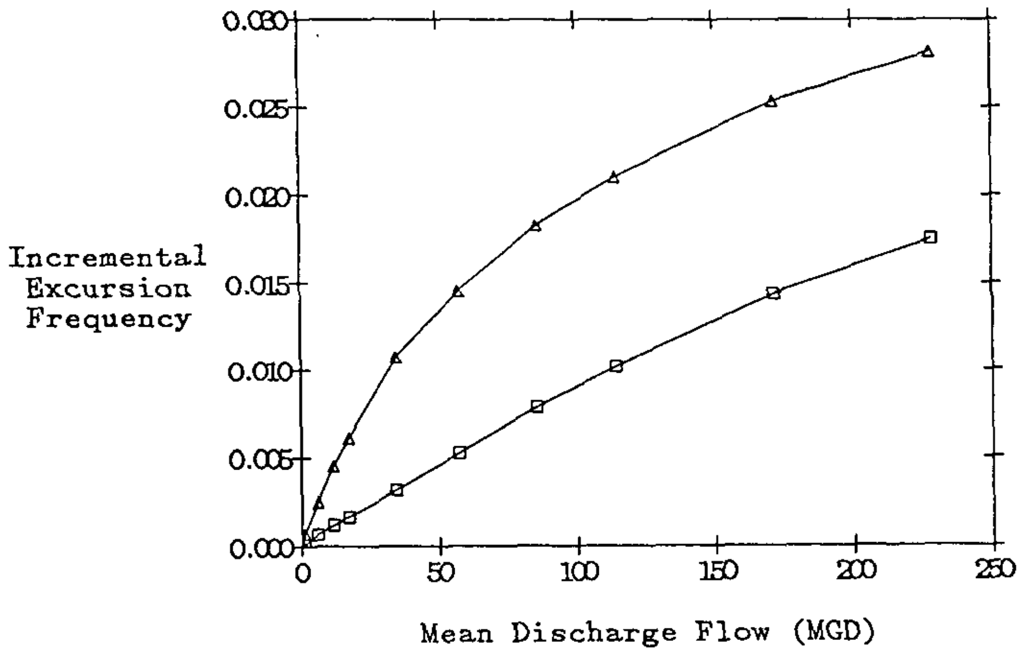


Figure A-8: Excursion Frequencies for Fecal Coliform in the Charles River



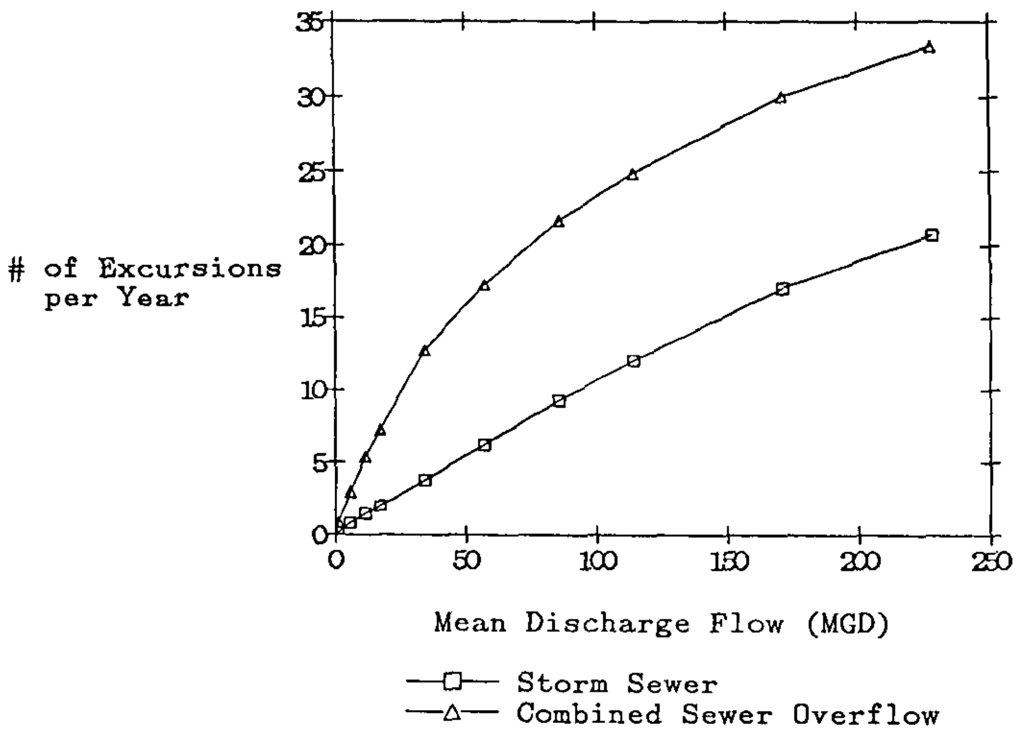


Figure A-9: Excursion Frequencies for Fecal Coliform in the Connecticut River

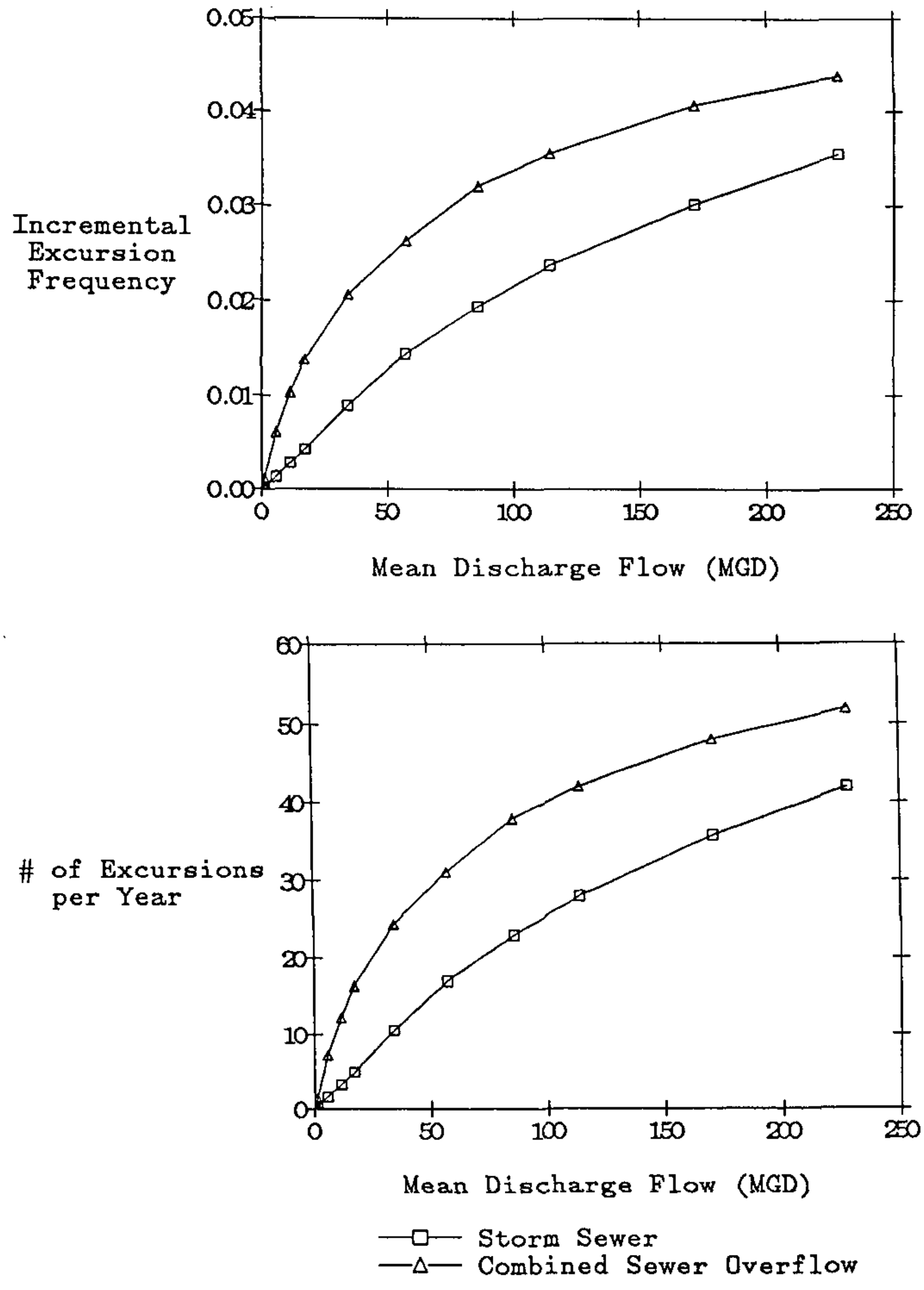


Figure A-10: Excursion Frequencies for Fecal Coliform in the Merrimack River

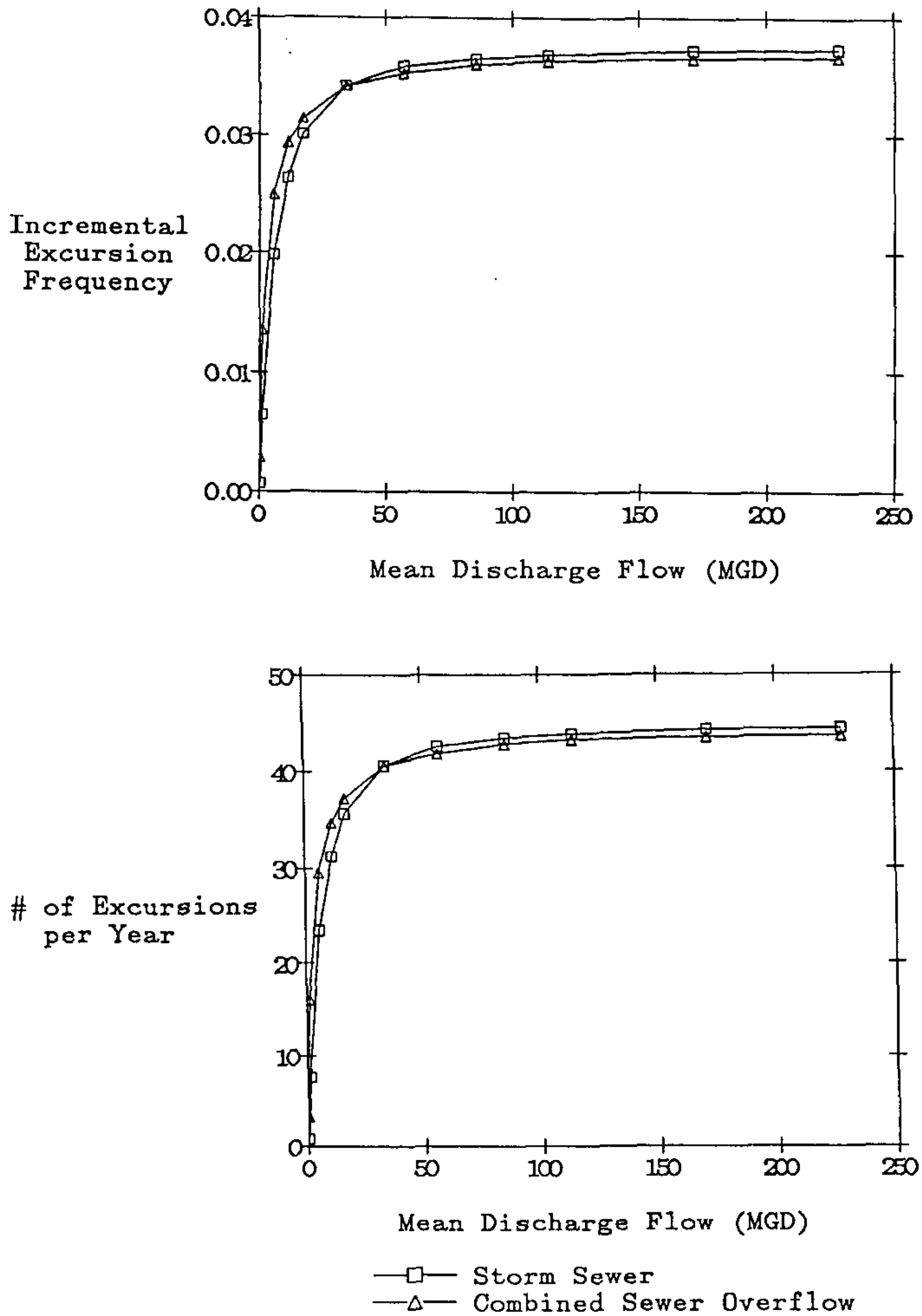


Figure A-11: Excursion Frequencies for Fecal Coliform in the Quinnipiac River

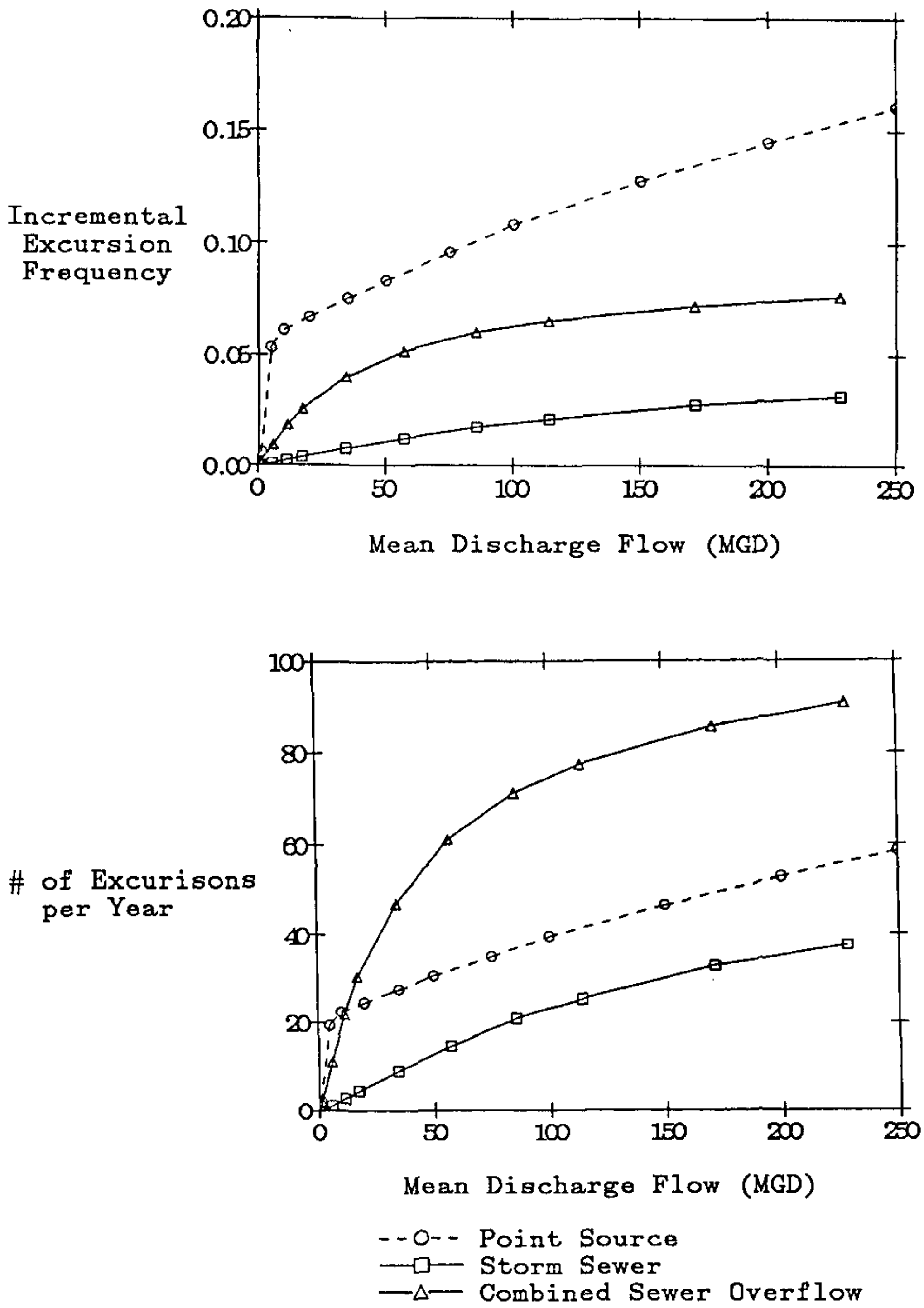


Figure A-12: Excursion Frequencies for BOD_5 in the Blackstone River