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A SAMPLING POLICY DESIGN FOR WATER QUALITY ENFORCEMENT

by

Thomas Michael Heidtke

A dissertation submitted in partial fulfillment
of the requirements for the degree of
Doctor of Philosophy
(Civil Engineering: Water Resources Engineering)
in The University of Michigan
1977

Doctoral Committee:

Associate Professor John M. Armstrong, Chairman
Associate Professor Raymond P. Canale
Professor Eugene A. Glysson
Professor Stephen M. Pollock

Dedication

To my Mother and Father, whose constant love and support gave me the opportunity to achieve this goal; and to my wife, Susie, who makes it all worthwhile.

Judge a man not by the size of his estate
or the level of his education
or the strength of his arm
but by the number of people who would call him friend.

TMH

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TABLE OF CONTENTS

DEDICATION ii

ACKNOWLEDGEMENTS iii

LIST OF TABLES vi

LIST OF ILLUSTRATIONS. x

LIST OF APPENDICES xiv

LIST OF SYMBOLS. xv

CHAPTER

I. PROBLEM DESCRIPTION 1

II. LITERATURE REVIEW 5

III. SAMPLING POLICY DESIGN AND EVALUATION 17

The Probability Threshold Policies
The Optimum Sampling Policy
Chlorides As the Index of Pollution
Violation Definition

IV. A HYPOTHETICAL SYSTEM 23

The Selected Sampling Policies
The Stream Model
Generation of Point-Source Chloride Loads
Simulation Output
Required Number of Simulated Observations
Sampling Policy Costs
Selecting the Optimum Policy
Sensitivity of Optimum Policy to Response Time
Summary of Results

V. THE GREAT MIAMI RIVER SYSTEM 74

Sampling Policy Description
Mathematical Model of the Great Miami River
Background Chloride Concentration and Initial Conditions
Probabilistic Nature of Point-Source Chloride Loads

TABLE OF CONTENTS (continued)

Generation of Point-Source Chloride Loads
Streamflow Input
The Model and Computer Program
Derivation of Model Prediction Error
The Probability Threshold Sampling Policies
Sampling Policy Costs
Generation of Sample Chloride Profiles
Policy Evaluation
Comparison of Variable and Uniform Sampling Intervals
Analysis Using 1974-75 Historical Data
Conclusions
Recommendations

APPENDIX 141

BIBLIOGRAPHY 223

LIST OF TABLES

Table	Description	Page
4-1	point-source chloride load distributions	25
4-2	the selected probability threshold policies	27
4-3	list of stream model input parameters	28
4-4	steady-state chloride concentrations	32
4-5	initial conditions vs. output 100 hours later	34
4-6	hourly violation probabilities	43
4-7	sampling times and detection probabilities for policies 1-9 (standard = 85 mg/ℓ)	44
4-8	cost analysis for policies 1-9	49
4-9	optimum policies for specified P_{D1} (stream standard = 85 mg/ℓ)	54
4-10	optimum policies for specified P_{D2} (stream standard = 85 mg/ℓ)	55
4-11	optimum policy vs. chloride stream standard	57
4-12	optimum policy vs. response time using P_{D1} (stream standard = 85 mg/ℓ)	63
4-13	optimum policy vs. response time using P_{D2} (stream standard = 85 mg/ℓ)	63
4-14	same as 4-7 (standard = 81 mg/ℓ)	64
4-15	same as 4-7 (standard = 83 mg/ℓ)	65
4-16	same as 4-7 (standard = 85 mg/ℓ)	66
5-1	original chloride loading (lbs/day) from National Lead	77
5-2	original chloride loading (lbs/day) from Gulf Oil	78
5-3	the selected probability threshold policies	79
5-4	input parameters to the Great Miami River model	87

LIST OF TABLES (continued)

Table	Description	Page
5-5	chi-square goodness-of-fit test for normality of National Lead loads	89
5-6	chi-square goodness-of-fit test for normality of National Lead loads	90
5-7	chi-square goodness-of-fit test for log-normality of National Lead loads	91
5-8	predicted vs. actual chloride concentration using U.S.G.S. flows (1973 water year)	98
5-9	same as 5-8 (estimated flows)	99
5-10	chi-square goodness-of-fit test for normality of model prediction error using U.S.G.S. flows (1973 water year)	101
5-11	same as 5-10 (estimated flows)	106
5-12	optimum policies for various levels of P_{D1} and P_{D2} (U.S.G.S. flows)	119
5-13	same as 5-12 (estimated flows)	120
5-14	fixed-time sampling policies	122
5-15	same as 5-8 (January 1975-July 1975)	126
5-16	same as 5-9 (January 1975-July 1975)	127
5-17	chi-square goodness-of-fit test for normality of model prediction error using U.S.G.S. flows (1974 water year)	129
5-18	same as 5-17 (estimated flows)	130
5-19	same as 5-12	132
5-20	same as 5-13	133
A-1	individual storm recession curves for Great Miami River	143
A-2	1973 water year flows: U.S.G.S	151

LIST OF TABLES (continued)

Table	Description	Page
A-3	1973 water year flows: estimated	152
A-4	1974 water year flows: U.S.G.S	153
A-5	1974 water year flows: estimated	154
B-1	streamflow vs. background chlordie concentration	156
C-1	chloride standard vs. T_0/T_1	169
C-2	fixed-time sampling policy derivation	170
E-1	detection percentages and costs for policies 1-16 (U.S.G.S. flows; standard = 45 mg/ℓ)	185
E-2	same as E-1 (standard = 50 mg/ℓ)	186
E-3	same as E-1 (standard = 55 mg/ℓ)	187
E-4	same as E-1 (predicted flows; standard = 45 mg/ℓ)	188
E-5	same as E-1 (predicted flows; standard = 50 mg/ℓ)	189
E-6	same as E-1 (predicted flows; standard = 55 mg/ℓ)	190
E-7	detection percentages and costs for policies A-P (U.S.G.S. flows; standard = 45 mg/ℓ)	191
E-8	same as E-7 (standard = 50 mg/ℓ)	192
E-9	same as E-7 (standard = 55 mg/ℓ)	193
E-10	same as E-7 (predicted flows; standard = 45 mg/ℓ)	194
E-11	same as E-7 (predicted flows; standard = 50 mg/ℓ)	195
E-12	same as E-7 (predicted flows; standard = 55 mg/ℓ)	196
E-13	detection percentages and costs for policies 1-16 (1974 water year; standard = 45 mg/ℓ)	197

LIST OF TABLES (concluded)

Table	Description	Page
E-14	same as E-13 (standard = 50 mg/ℓ)	198
E-15	same as E-13 (standard = 55 mg/ℓ)	199
E-16	detection percentages and costs for policies A-P (1974 water year; standard = 45 mg/ℓ)	200
E-17	same as E-15 (standard = 50 mg/ℓ)	201
E-18	same as E-16 (standard = 55 mg/ℓ)	202
F-1	1973-74 chloride predictions using U.S.G.S flows	204
F-2	1973-74 chloride predictions using estimated flows	205
F-3	1974-75 chloride predictions using U.S.G.S flows	206
F-4	1974-75 chloride predictions using estimated flows	207
F-5	1973-74 chloride profile using U.S.G.S flows	208
F-6	1973-74 chloride profile using estimated flows	209
F-7	1974-75 chloride profile using U.S.G.S flows	210
F-8	1974-75 chloride profile using estimated flows	211
F-9	same as F-7	212
F-10	same as F-8	213

LIST OF ILLUSTRATIONS

Figure	Description
2-1	sampling delay for various water quality ranges 12
3-1	hypothetical receiving water system 17
3-2	chloride profile with stream standard violations 22
4-1	model of the receiving water and chloride loads 23
4-2	point-source loads to hypothetical receiving water system 24
4-3	generated point-source chloride load ...26
4-4	segmentation of stream 27
4-5	sensitivity of model output to variation in initial conditions..... 31
4-6	chloride profile at segment 19: replication 1 32
4-7	chloride profile at segment 19: replication 2 39
4-8	chloride profile at segment 19: replication 3 40
4-9	chloride profile at segment 19: replication 4 41
4-10	probability of stream standard violation vs. time of day..... 42
4-11	variable cost functions for detection delay..... 48
4-12	detection probability vs. total cost (violation definition 1; standard = 85 mg/ℓ) 52
4-13	same as 4-12 (violation definition 2; standard = 85 mg/ℓ) 53

LIST OF ILLUSTRATIONS (continued)

Figure	Description
4-14	same as 4-12 (violation definition 1; standard = 81 mg/ℓ)..... 67
4-15	same as 4-12 (violation definition 2; standard = 81 mg/ℓ)..... 68
4-16	same as 4-12 (violation definition 1; standard = 83 mg/ℓ)..... 69
4-17	same as 4-12 (violation definition 2; standard = 83 mg/ℓ)..... 70
4-18	same as 4-12 (violation definition 1; standard = 87 mg/ℓ)..... 71
4-19	same as 4-12 (violation definition 2; standard = 87 mg/ℓ)..... 72
4-20	violation detection vs. total cost for various response times (violation definition 1; non-linear cost function)..... 73
5-1	segmentation of Great Miami River system..... 82
5-2	model error vs. time.....105
5-3	predicted vs. actual chloride concentration (1973 water year; U.S.G.S. flows).....111
5-4	same as 5-3 (1974 water year; U.S.G.S. flows).....112
5-5	same as 5-3 (1973 water year; estimated flows)....113
5-6	same as 5-3 (1974 water year; estimated flows)....114

LIST OF ILLUSTRATIONS (continued)

Figure	Description
5-7	percentage of violations detected vs. required sample size 137
5-8	percentage of violations detected vs. average delay to detection 138
A-1	individual storm recession curves 142
A-2	groundwater depletion curve for Great Miami River 144
A-3	current flow vs. flow 24 hours later ...146
A-4	current flow vs. flow 24 hours later ...147
B-1	predicted vs. observed upstream chloride concentration 158
B-2	residual background concentration prediction error vs. streamflow 162
B-3	background chloride concentration vs. streamflow 163
C-1	stream standard violation process.....164
C-2	M vs. Δ/T_1 for various values of T_o/T_1..166
C-3	waste load to receiving water.....167
D-1	total cost vs. % of violations detected-1973 water year (linear costs; U.S.G.S. flows; standard = 45 mg/ℓ).....172
D-2	same as D-1 (non-linear costs).....173
D-3	same as D-1 (estimated flows).....174
D-4	same as D-1 (estimated flows; non-linear costs).....175
D-5	same as D-1 (stream standard = 50 mg/ℓ) 176

LIST OF ILLUSTRATIONS (concluded)

Figure	Description
D-6	same as D-2 (stream standard = 50 mg/ℓ) 177
D-7	same as D-3 (stream standard = 50 mg/ℓ) 178
D-8	same as D-4 (stream standard = 50 mg/ℓ) 179
D-9	same as D-1 (stream standard = 55 mg/ℓ) 180
D-10	same as D-2 (stream standard = 55 mg/ℓ) 181
D-11	same as D-3 (stream standard = 55 mg/ℓ) 182
D-12	same as D-4 (stream standard = 55 mg/ℓ) 183

LIST OF APPENDICES

APPENDIX		PAGE
A.	STREAMFLOW PREDICTION DEVELOPMENT	141
B.	BACKGROUND CHLORIDE CONCENTRATION	155
C.	DERIVATION OF FIXED-TIME SAMPLING POLICIES	164
D.	ILLUSTRATIONS OF DETECTION PERCENTAGE AND TOTAL COST OF VARIABLE AND FIXED-TIME POLICIES	171
E.	DETECTION AND COST TABLES FOR VARIABLE AND FIXED-TIME POLICIES	184
F.	SAMPLE CHLORIDE PROFILES AND MODEL PREDICTIONS FOR THE 1973 and 1974 WATER YEARS	203
G.	COMPUTER PROGRAMS	214
	Deckfile	
	Miami	

LIST OF SYMBOLS

Symbol	Description and Units
A	cross-sectional area of the stream ($ft.^2$)
$c_i(t)$	chloride concentration in segment i at time t (mg/ℓ)
$C_{v,r}$	total variable cost incurred during the r^{th} replication
c_M	model prediction which indicates $\hat{p} > p$
$c_p(t)$	model chloride prediction at time t (mg/ℓ)
c_r	depletion constant (dimensionless)
c_T	total cost over planning period
c_v	total variable cost over the planning period
$e(t)$	model prediction error at time t (mg/ℓ)
E	dispersion coefficient ($mi.^2$/day)
$f_j(x)$	probability function of chloride load from j^{th} source
FC	fixed cost of sampling
IC_i	initial chloride concentration in segment i (mg/ℓ)
K	number of flow measurements required over the planning period
ℓ_i	length of segment i (ft.)
L_i	characteristic length of segment i (ft.)
M	sampling frequency priority measure
n	number of samples required over planning period
n_t	number of violations at time t

LIST OF SYMBOLS (concluded)

$n_{v,r}$	number of violations detected during the r^{th} replication
N	number of segments contained in stream model
$N_{v,r}$	number of violations occurring during the r^{th} replication
N_t	number of observations at time t
P	sampling probability threshold
p_t	probability of violation at time t
$\hat{p}_t$	estimate of p_t
P_D	probability (percentage) of violation detection
Q	streamflow (volume/time)
Q_0	flow today (cfs)
Q_{24}	flow tomorrow (cfs)
Q_T	threshold streamflow (cfs)
SS	chloride stream standard (mg/ℓ)
t_i	time delay to detection of i^{th} violation
T_o	mean interval between violations (time)
T_1	mean duration of a violation (time)
UC(t)	upstream boundary chloride concentration at time t (mg/ℓ)
V_i	volume of stream segment i (ft^3)
$W_i(t)$	point-source chloride load to segment i at time t (lbs./time)
z	monitored position in stream
Δ	sampling interval (days)

CHAPTER I

PROBLEM DESCRIPTION

A major point of discussion and debate has evolved over the last several years regarding acceptable mechanisms for enforcing water quality legislation, in particular, water quality standards. Due to rising national interest and concern over the deterioration of many lakes and streams, the recent influx of proposed controls has been excessive, if not confusing as to practical feasibility. Widespread demands upon a limited water supply, e.g., municipal and industrial requirements, recreational and agricultural demands, etc., have in turn created a complex web of pollution control demands now facing governmental enforcement agencies. The end result is that these agencies must devote a great deal of their time and efforts toward reviewing and developing programs designed to monitor the quality of lakes and streams--programs which are not only effective in detecting violations of existing pollution "thresholds", but which are also feasible within the resource constraints of the agency itself.

Recent legislation such as the Water Pollution Control Act Amendments of 1972 further illustrates the need for developing effective monitoring programs. This legislation is one attempt to attain national water quality objectives. For instance, it requires that water quality standards shall be established on all navigable waters of the United States. Clearly, the legislation itself is principally a statement of pollution control goals, rather than a specific mechanism for reaching those goals. The bridge between legislation and its objectives is the enforcement system, and effective programs for monitoring the quality of a water resource are necessarily an integral part of this system.

In order to make clear the exact goal of this research, it is first necessary to distinguish between monitoring the resource for detection of long-range vs. short-range shifts in water quality. In the former case the major concern is trends in water quality which may occur over several months or even years--trends which may be reflected in gradual changes in the physical, chemical and biological condition of a water resource. More specifically, it may be of interest to detect long-term changes in such water quality parameters as temperature, dissolved oxygen, turbidity, or varieties of plant and fish populations occurring in a particular water body. In the latter case the primary concern is with short-range effects of pollutional stresses on the physical, chemical, and biological composition of a water resource--those which may be seen as weekly, daily, or perhaps hourly shifts in water quality. Short-term exposure to hazardous conditions such as low dissolved oxygen concentrations or high concentrations of toxic materials can result in rapid fish kills or contamination to municipal water intakes. Here it is necessary to design the monitoring program for the purpose of detecting these rapid shifts in water quality.

The emphasis in this research is on monitoring for the detection of short-term water quality changes. The objective of the investigation is to design and evaluate a sampling policy which will detect a required percentage of stream standard violations over a specified planning horizon. This approach attempts to utilize knowledge of the probabilistic nature of a pollutant concentration in obtaining the policy, thereby eliminating or at least reducing some of the wasteful sampling inherent in current monitoring practices. Today's state regulatory agencies may monitor a given water quality parameter by taking weekly or even monthly observations at random times on a particular stream. Although this approach may be useful in detecting a few isolated standard violations

over a comparatively long time period, it may also miss detecting a high percentage of unacceptable pollution levels which occur on a daily or hourly basis. Because, as previously discussed, such short-term fluctuations can result in irreparable damage to the biota of the stream, this type of sampling scheme may actually be ineffective for enforcing a specified standard.

Therefore it may prove advantageous to incorporate into the design of a monitoring program information concerning the probabilistic nature of both natural and man-made waste loadings to a receiving water. These stochastic inputs may be used in conjunction with a model of the stream system to arrive at a probabilistic description of the water quality parameter of interest. In this sense the discharge-receiving water system may be simulated. The generated pollutant concentration profiles may then be used as experimental cases for evaluating the effectiveness of a proposed sampling policy. This is the approach taken in the following investigation. Initially a class of sampling policies referred to as probability threshold policies are proposed for monitoring the stream. Each represents a specific sampling condition--a threshold probability which, if exceeded, requires an observation on the water quality parameter of interest. If, at any point in time ($t = 0, 1, 2, \ldots,$) the probability of a stream standard violation exceeds this threshold, a sample is taken. The required or "optimum" sampling policy is that which will detect at least a prespecified percentage of all violations at minimum cost. A subset of the single threshold policies--the selected policies--is then identified and each member of this subset is evaluated against the simulated pollutant profiles on the basis of detection capability and total cost over a given time period. The one which satisfies the definition of optimum is then selected.

It is intended that the methodology should be applicable

to a broad spectrum of pollutants. Although the following analysis is directed toward detection of stream standard violations for conservative[1] materials, the basic principles involved are retained in application to monitoring of more complex non-conservative substances.

[1]Materials which do not decay over time and do not leave the stream by means of settling or absorption.

CHAPTER II

LITERATURE REVIEW

Previous approaches to the design of water quality monitoring programs have focused primarily on the objective of detecting long-range changes in various water quality parameters. Additionally, these programs only deal with sampling policies that call for field observations to be taken at uniform intervals of time. Although there is evidence of a shift in emphasis toward incorporating information concerning the probabilistic nature of pollution in program design, the need for employing this information in intensive monitoring policies has been generally ignored.

Loucks and Lynn (1966) investigated the use of mathematical models for predicting the probability distribution of dissolved oxygen downstream of a wastewater treatment plant. The authors treat daily streamflows as a first-order Markov chain while using the oxygen-sag equation (Streeter and Phelps, 1925; Camp, 1963; Dobbins, 1964) to estimate dissolved oxygen concentrations for 1, 2, and 3 consecutive-day periods. However, rather than applying the results in a violation detection scheme, the information is used to derive a schedule of sewage treatment efficiency which would reduce the probability of low water quality levels occurring over a given length of time. The question of how to schedule sampling of the stream for detection of quality standard violations is not answered.

A more direct application of stochastic inputs to mathematical stream models in the area of water quality control was made by Ward (1973). The analysis focuses on a set of seven specific objectives of pollution prevention and abatement, the data requirements for attaining each objective, and an economic comparison of the types of sampling available

for obtaining this data, i.e., automatic monitoring, remote sensing and grab sampling. In particular, two specific designs for surveillance of water quality are investigated. The design for obtaining "base-level" or long-range trend data employs a surveillance network simulation model (Vanderholm,1972) for generation of water quality data according to known characteristics, e.g., monthly mean and standard deviation. A large set of arbitrary sampling frequencies are then evaluated on the basis of their ability to estimate the original characteristics of the given quality parameter and the associated cost of sampling. In the second case the primary concern is sampling for detection of short-term water quality fluctuations. Again, the simulation model is used to generate a series of "pollution events" or stream standard violations at random times and locations along a given reach of stream. A detection interval--a time period during which a sample must be taken for violation detection--is calculated by means of an equation developed by Glover (1964). With the detection interval specified for each pollution event, a set of arbitrary sampling frequencies are then tested as to detection capability and cost over a specified planning period.

One disadvantage of this approach is its failure to address the question of what specific policies should be considered for sampling, i.e., what are the sampling intervals for each policy? are these intervals uniform or variable? how are they determined? etc. The problem of developing an organized approach to the design of an effective and efficient sampling schedule remains unresolved. Also, no consideration is given to individual stochastic pollutional loadings and how they effect the probabilistic nature of the violation process.

Blackman and Proschan (1959) have researched the problem of scheduling looks for objects having unknown arrival times, and have applied principles of search theory

to arrive at an optimum solution. The arrival of objects (violations) is assumed to be a poisson process. The authors show that when looks are confined to a single position in space, the ideal sampling frequency is at uniformly spaced times. For the case in which the objective of the search is to mimimize the expected delay between the arrival of an object and its detection, the optimum frequency of scans is proportional to $((p_i/t_i)(1/P_i - 1/2))^{1/2}$, where p_i represents the probability of an object arriving at position i given it is present, t_i represents the time required for making an observation at position i, and P_i represents the probability of detecting the object with a single look at position i. Although a unique and optimum sampling frequency for target detection is derived, the underlying assumption that the arrival process is poisson can lead to difficulties in application to detection of environmental phenomena. The approach is most useful when continuous up-dating of information regarding the violation process is not possible. However, in cases where inputs effecting the process are known or can be closely approximated, e.g., streamflow and its effect on pollutant concentrations in a stream, the information should be used in scheduling observations.

Spectral analysis has been suggested as a mechanism for determining optimum sampling intervals for monitoring the quality of a water resource (Gunnerson, 1966; Wastler, 1969). A record of water quality data is analyzed by various statistical techniques until "the maximum understanding or insight is developed". Subsequently the quantity of data originally present is iteratively reduced until further reduction results in a data record which is inadequate for understanding the behavior of the phenomena. At this point the sampling interval is at its optimum.

Although analysis of power spectra may be useful in the design of various data acquisition systems, its applica-

tion as a tool for detection of specific pollution events has been challenged. Beckers et al. (1972) cite the following arguments:

> "The spectral approach can provide information about the 'goodness' of a particular sampling rate at portraying the original process. It cannot provide information about the ability of a sampling scheme to detect particular conditions of that process, such as a violation condition. This limitation is due to the elimination of phase information in the computation of the power spectrum.
>
> Thus, while the spectral approach is an excellent means of designing sampling systems to portray the full variation of the original process, it is not useful in designing sampling systems to detect only discrete states of the original process.
>
> A sampling scheme designed on the basis of spectral methods may detect violations, of course. There is, however, no way of measuring the likelihood of such detection using the spectral method. Even if sufficient water quality data were available to allow system design based on spectral methods, application of those methods to design of systems solely to detect water quality violations is not a preferred approach."

The value of spectral analysis is in its ability to characterize the probabilistic nature of a pollutant or pollutional loading. In this sense it may provide an effective means of designing data acquisition systems to understand the parameters of various environmental phenomena.

Another technqiue of modeling water quality data makes use of the Box-Jenkins method of time-series analysis (Huck and Farquhar, 1974). This approach is time-based rather than frequency-based. A model of an enviornmental process, e.g., chloride concentration, is developed by expressing a time-series of data as output from a linear filter receiving a random input and consisting of one or more transfer functions in series. The model design contains three distinct

stages. The first is model identification. Knowledge of the autocorrelation function and partial autocorrelation function is required to decide which model (autoregressive, moving average, autoregressive-moving average, autoregressive integrated moving average) is most applicable in any given case. In the second stage, maximum likelihood estimates of the model parameters are derived, e.g., the order of the model. The final stage of model building consists of diagnostic checks to validate the proposed model and to suggest possible modifications.

In spite of the fact that the Box-Jenkins method has had limited use in the field of water quality control, it represents a powerful tool which must be given more consideration in the future. The method is capable of producing good forecasts over selected lead times--a necessary ingredient for an effective water quality monitoring program. The present application by Huck and Farquhar does not address the problem of rapid detection of stream standard violations based upon a known damage function for delay to detection. However, the technique may be applicable to this particular objective of water quality monitoring. The Box-Jenkins method employs knowledge of the sequential dependence of a time series in deriving the forecast, thereby taking advantage of the statistical relationship between data points. Although the analysis presented in the following research uses the water quality forecast from a deterministic mathematical model to arrive at sampling times directly, it is noted that the degree of autocorrelation between pollutant observations is neglected. It is suggested, therefore, future research should be directed at the application of this technique to the problem currently under investigation.

Recently the design of sampling programs based upon estimation theory and the analysis of uncertainty has been investigated (Moore et al., 1974). In this approach the

emphasis is on sampling policies used to obtain input data to a mathematical model of a given water resource. A policy is evaluated on the basis of the resulting uncertainty in predicting the state of the resource, e.g., its trophic condition. Three types of uncertainty are explored: 1) uncertainty from errors in data due to sampling techniques; 2) uncertainty due to incomplete measurement of temporal and spatial water quality variables; and 3) uncertainty due to the natural probabilistic nature of the water quality phenomena. Moore et al. use this approach in analyzing the capability of various sampling frequencies to assess the trophic condition of an impoundment, In particular, the authors stress the advantages of estimation theory as a means of evaluating the adequacy of a sampling program. By looking at the trade-offs involved in increased sampling frequency versus reductions in uncertainty of prediction, policies may be systematically modified and improved as data is collected. This technique is shown to be advantageous for detecting long-range trends in a water quality parameter. Consideration is only given to policies which call for relatively infrequent sampling--between four and twelve observations per year. Therefore the value of this method in designing programs requiring intensive sampling for detection of short-term variations is not clear.

Moore (1973) has also investigated the problem of optimizing sampling intervals for water quality monitoring of streams through the application of filter theory. In short, observations from a dynamic water quality model and actual field measurements are assumed to represent independent estimates of the state of the receiving water. This information is then combined and weighted according to relative levels of prediction uncertainty. The filter output then produces an improved state estimate and predicted uncertainty associated with this new estimate. The objective of this application is to arrive at a least-cost sampling

strategy (in time and space) which will satisfy the specified constraints of the maximum allowable uncertainty in the state estimate.

The linear filter approach to determining sampling times for detection of water quality standard violations seems to possess many advantages over traditional sampling procedures. The technique incorporates information concerning random fluctuations in the variables effecting the state of the system, as well as the uncertainty associated with both system and measurement estimates. Although the methodology requires an assumption that the problem can be treated as linear, the overall formulation of the objective function and constraints seems to provide a unique and effective mechanism for sampling policy design. However, a few possible difficulties of the linear filter approach should be discussed.

The particular application discussed above does not include penalty costs for delays to detection of water quality standard violations. The efficiency of the filter as a tool in forecasting water quality is based upon 1) fixed costs of sampling 2) a variable cost dependent on the number and type of variables measured. Unfortunately, any penalty cost for delay to violation detection is neglected. For this reason the allocation of sampling resources is not directed toward the detection of the violation condition as much as it is toward accurate measurement of all conditions. The objective of the sampling strategy would shift if a penalty cost for detection delay were included.

The other disadvantages of the filter approach are 1) the necessity of an extensive data base for input to the filter and 2) the high computer costs associated with

the dimension of the state vector. These problems are discussed in greater detail in Moore's investigation of a simulated receiving water system.

Arnold (1970) has studied the problem of scheduling observations by treating the sampling procedure as a Markov process. In this approach water quality is classified by various levels of acceptance and a specified sampling interval or delay is associated with each level. A mathematical model of a stream is used to estimate the probability distribution for the parameter of interest. The parameters of the distribution define the individual probabilities of an observation falling into any given quality level. A graphic representation of this type of sampling plan is shown below:

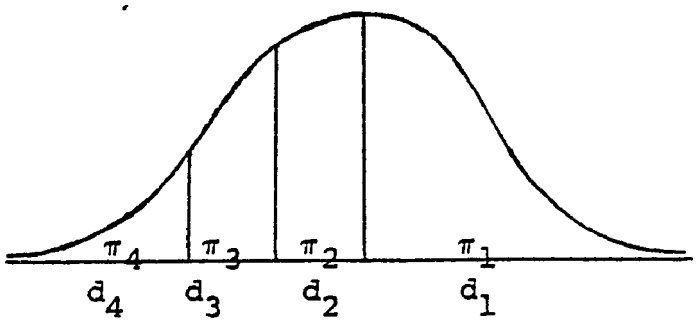


FIGURE 2-1

where $\pi_i \equiv$ probability of observing the water quality parameter at level i.

$x_i \equiv$ boundary for i^{th} acceptance level.

$d_i \equiv$ sampling delay associated with i^{th} acceptance level.

The author derives a set of sampling plans for monitoring the concentration of dissolved oxygen in a stream. The

deviations of DO about the expected value (from the mathematical model) are assumed to be normally distributed with known mean and variance. By varying the values of the d_i's, the expected sample size over a one year period can be immediately calculated for each feasible plan. Plans may then be selected prior to sampling on the basis of 2 criteria: 1) expected sample size over a given time range and, 2) probability of sampling within a specified number of time units.

A major advantage of this approach is that it addresses the problem of effective allocation of sampling resources. The use of a variable sampling delay reduces excessive sampling during periods when water quality levels are within acceptable limits, thus providing a more efficient overall monitoring program. On the other hand, the major drawbacks of this method should also be discussed.

The first of these regards the selection of the sampling delays (d_i's). The method of selecting an optimal policy requires enumeration of all possible combinations. This may become a most difficult task in treating complex systems. Secondly, the analysis fails to consider the penalties associated with an undetected violation level or the delay to detection. The author only suggests that the probability of sampling within a given number of time units should be a criterion for policy selection. The question of what constitutes a best or "optimum" policy is not discussed.

The need for establishing a definitive basis for sampling policy evaluation and selection has been recognized. Beckers et al. (1972) and Beckers and Chamberlain (1974) use an effectiveness rating (M) as a means of analyzing sampling programs. This effectiveness rating is defined as the ratio of the expected number of violations detected to the expected total number of violations for a given time period. The methodology is given extensive

discussion in Appendix C. Sampling frequencies derived from this technique are analyzed in Chapter 5.

A very recent study has been made of sampling procedures applied to monitoring the quality of effluent sources (Cohen et al., 1975). The method uses compliance and self-monitoring data from each source to establish a statistical description of the effluent standard violation process. This information is then used to set priorities regarding the order and frequency of sampling the various sources. Those having a high probability of violation and a large expected environmental damage are sampled first. The environmental damage or cost of undetected violations at a given source is a function of 1) the expected frequency of a standard violation, 2) the expected magnitude of the violation, 3) the toxicity of the pollutants, and, 4) the assimilative capacity of the receiving water at the discharge points. A maximum marginal return algorithm is used to obtain the optimum allocation of monitoring resources, i.e., that which minimizes the expected cost of undetected violations.

Cohen et al. recognized the need to consider the probabilistic nature of the violation process when designing a monitoring network. Their work reflects the benefits to be derived from statistical analysis of limited field data as a tool in deriving and evaluating monitoring programs. However, a few drawbacks of this particular treatment deserve discussion.

As previously mentioned, a criterion for acceptability of any given sampling policy is expected damage to the stream, where damage is shown to be a direct function of streamflow. In estimating the damage function, the authors suggest using the minimum average monthly flow over the months in the monitoring period. Therefore the random nature of streamflow is essentially neglected in obtaining an optimum allocation of sampling resources--a condition

which may result in reduced monitoring efficiency. Certainly the allocation which optimizes sampling effectiveness under low-flow conditions may differ significantly from the best allocation for high-flow conditions. The variability of streamflow should not be excluded from the allocation design without risking inaccurate damage estimates.

A second problem concerns the exclusion of time as a parameter in the analysis. For example, the optimum number of samples to be allocated to a particular effluent source is derived, but no means of selecting specific sampling times is given. This fact is a result of the condition that damage is assumed independent of time to detection. Thus, if an effluent source is found to be in violation of a standard at any time during a given monitoring period, the cost or damage for that source is considered zero. This assumption removes the need to consider the temporal variablility of the violation process as a critical parameter in determining the optimal resource allocation. Again, this exclusion can lead to an over-simplification of the monitoring problem.

The use of linear programming has been examined as a tool for obtaining water quality control. Anderson and Day (1968) studied five different regional treatment policies as a means of reducing treatment costs while adhering to existing river standards for dissolved oxygen. A linear programming formulation was used to derive optimum treatment levels for various waste sources on the Miami River. Revelle et al. (1967) followed a similar approach, employing a linear programming model based on the oxygen-sag equation to obtain the degree of treatment required of each effluent source. Treatment costs are minimized within the constraints of required water quality levels.

Although the above approaches may be effective in coordinating stream standards and required waste treatment

levels, they do not address the problem of how to monitor the resource for detecting violations of these standards. Water pollution legislation such as the Water Pollution Control Act Amendments of 1972 require that both effluent standards and stream standards be used as a means of obtaining overall water quality objectives. However, they should not be viewed as enforcement mechanisms. Stream and effluent standards are examples of national efforts to quantify pollution. In essence, they serve as a "speed limit" in controlling water pollution traffic, i.e., they can only function as a notice to those who use the resource that quality thresholds or limits are in effect. Enforcing the limits through violation detection then becomes a new problem, and it is toward this area that the following research is directed.

CHAPTER III

SAMPLING POLICY DESIGN AND EVALUATION

Predicting the response of a complex environmental system to a given set of pollutional stresses can present a difficult problem. If the particular policy for monitoring a stream is to depend upon the probabilistic nature of a specified water quality parameter, a means of response estimation must be an integral part of the policy design.

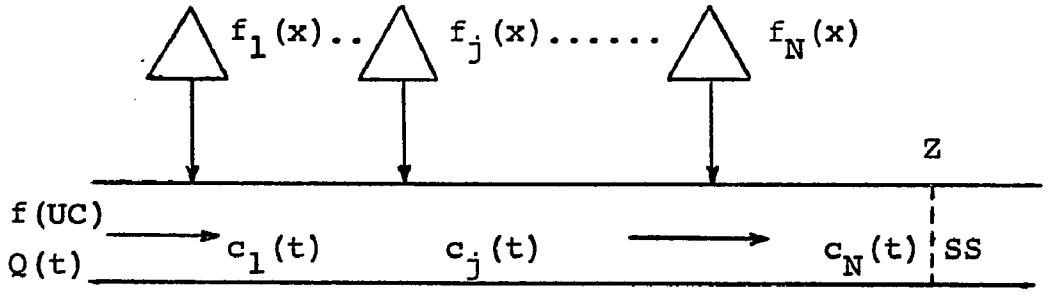


FIGURE 3-1

Figure 3-1 illustrates a stream receiving the waste discharges from a series of N point-sources. Each source j discharges according to a known probability distribution $f_j(x)$, where x represents the mass loading of pollutant (mass/time). In addition, the background concentration of the given pollutant has a density function f(UC) and the streamflow entering the system at the upstream boundary varies over time. A concentration threshold or stream standard SS is in effect at position Z on the stream. As previously stated, the objective of this research is to derive a sampling policy which will detect, at position Z, at least a specified percentage of all stream standard violations at minimum cost. It will

be shown that in order to accomplish this objective, knowledge of the probabilistic nature of $C_N(t)$--the concentration of material in the stream at position Z--is required.

The Probability Threshold Policies

As a means of attaining efficient allocation of available resources, a sampling policy is desired which will call for observations at times when the likelihood of a stream standard violation is high. At the same time the policy will have an associated risk of not sampling when a violation is actually occurring. The advantage of this approach is that it attempts to concentrate field observations in periods of high violation probability to avoid wasteful sampling. In other words, the increase in overall detection efficiency offsets the risk of a missed violation. For this reason the form of the sampling policies considered in the present investigation is based upon the violation probability at the specific points in time. In the following analysis time is divided into discrete time units such that samples may only be taken at times t = 0, 1, 2, ...
A probability threshold policy is defined as follows:

> If, at any point in time, the probability of a stream standard violation (p) exceeds a threshold probability P, a sample is taken.

In other words, samples are taken at those times for which

$$\hat{p} = p[C_N(t) \geq SS] > P$$

It can readily be seen, therefore, that the decision on whether or not to make a water quality observation depends upon the probability description of the stochastic process $C_N(t)$.

A probability threshold policy is defined by its associated violation probability threshold P. Obviously the total number of policies investigated must be limited to some manageable quantity while covering a wide range of

possible violation probabilities. In the analysis which follows the policies considered cover values of P from .01 to .50. Therefore the most intensive sampling policy included is one which calls for sampling at those times for which P = .01. Similarly, it is assumed that the maximum value of P would never exceed .50 in actual practice. The specific range of P covered in the subset of the probability threshold policies is left to the discretion of the user. For clarity, this subset will always be referred to as the selected policies.

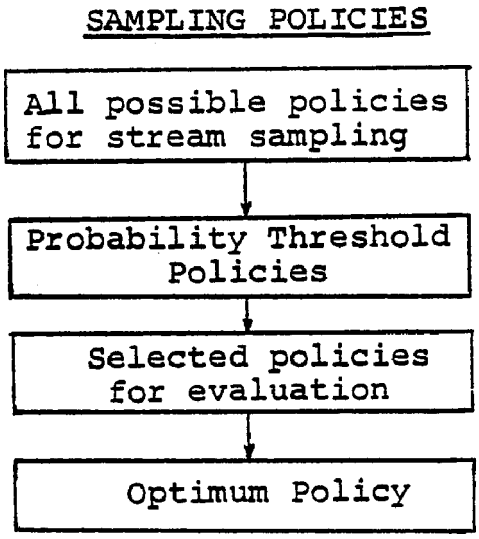
It is also necessary to emphasize that the family of sampling policies investigated here is only one of an infinite set of possible strategies for monitoring a stream. It is therefore possible that more effective policies do exist than those that follow. However, the present set of policies attempts to utilize information concerning the nature of all definable pollution sources in the determination of specific sampling times. In this manner a more effective allocation of sampling resources may be obtained than if this information is neglected in the design of the monitoring program. On the following page, a schematic representation is shown of the terminology applied to the various policies.

The Optimum Sampling Policy

Given the selected sampling policies, the optimum policy is selected on the basis of two criteria:

1) Probability of stream standard violation detection
2) Total cost over a given planning period.

More specifically, the optimum policy will be defined as the selected sampling policy which will detect at least X percent of the stream standard violations at minimum cost over the planning period. Evaluating each selected policy on the basis of these two performance criteria requires either 1) historical data for the water quality index of



interest, or 2) simulated profiles for that index. Because extensive historical data is often not available, selected policies are evaluated using simulated water quality profiles.

Chloride as the Index of Pollution

The concentration of chlorides in the stream is used throughout this research as the water quality index of interest. Reasons for the selection of chlorides are given below.

1. Chlorides are conservative in nature, i.e., they do not decay over time and do not leave the stream by means of settling or absorption.

2. The mathematical representation of the time rate-of change of chlorides in a stream is less complex than for non-conservative materials.

3. The concentration of chlorides is easily and accurately measureable.

4. Chlorides are commonly measured in existing monitoring

programs, i.e., some historical chloride data is usually available.

As previously discussed, the primary goal of the research is the development of a sampling methodology which can be applied for detection of violations with respect to both conservative and non-conservative pollutants. The present investigation illustrates the sampling policy design approach in the conservative case, but the technique is intended for application to a wide variety of contaminants. It is felt that further research into specific considerations and problems arising in the application to more complex materials is justified.

Violation Definition

Two different definitions of violation employed in the analysis of the selected policies. These definitions are listed below.

Definition 1: A violation occurs whenever the chloride concentration crosses the stream standard SS into the violation region. In this sense it may be considered a point event. The length of the violation is given by the time period until the chloride concentration again crosses the water quality threshold. This definition of a stream standard violation will always be referred to as either a violation or a violation event.

Definition 2: A single violation of a stream standard SS occurs any time the chloride concentration is equal to or greater than the given threshold. This definition will always be referred to as either a violation period or violation time.

Inspection of Figure 3-2 illustrates the difference between the two definitions. The chloride profile contained in Figure 3-2 contains a total of 2 violation events and 5 violation periods. The standard SS is first crossed at 3 P.M. At 6 P.M. the standard is again crossed, constituting a single 3-hour violation event. A second violation occurs over the time period from 8 P.M. to 10 P.M.--a 2 hour violation event. On the other hand, the same chloride

profile contains a total of 5 violation periods, i.e., 5 hours during which the stream standard is equalled or exceeded.

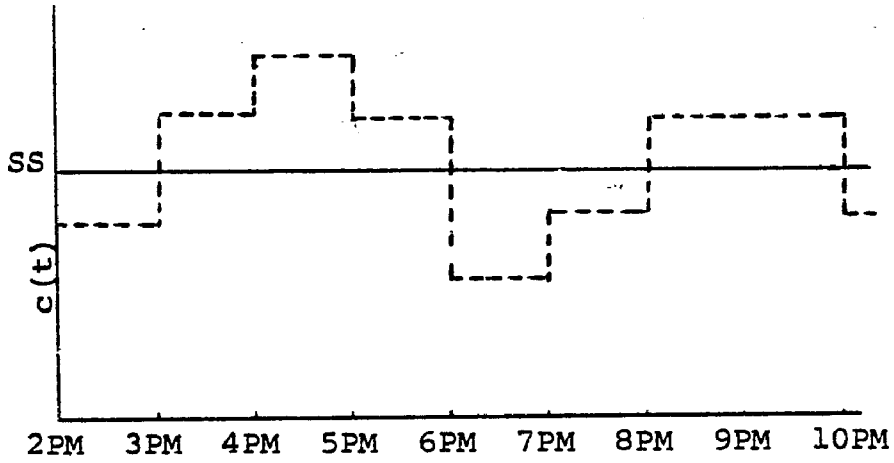


FIGURE 3-2

CHAPTER IV

A HYPOTHETICAL SYSTEM

The complex and dynamic nature of many environmental systems prohibits exact determination of their response to known inputs. For this reason it is necessary to approximate the response by formulating a model of the system, using the model as a means of transforming given inputs into an expected output. A mathematical statement is used to represent the manner in which the system reacts to natural or man-made pollutional stresses. In this way the physical characteristics of a receiving water can be broken down into a set of equations which describe the spatial and temporal effect of these stresses.

Once the mathematical model of the stream is derived and validated, the next step in evaluating the stochastic

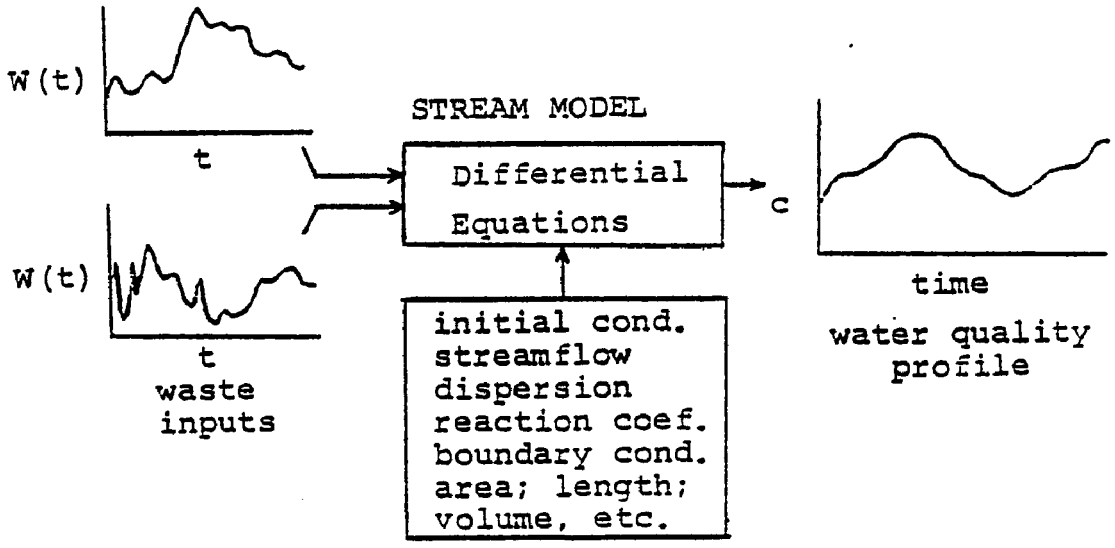


FIGURE 4-1

nature of the stream response is experimentation with the model over time. In this sense the discharge-receiving water system is simulated (Naylor et al., 1967). A computer program is used to evaluate the model output over time for a given set of system parameters and realizations of the stochastic input variables. The resulting data is then analyzed to determine specific sampling times for the probability threshold sampling strategies.

The first case investigated is an idealized system for which the characteristics of the stream and the waste loadings are known exactly. The mathematical model is assumed to be an exact representation of material dynamics in the stream. Similarly, the probability function and associated parameters which describe the waste-loading from each point-source are known explicitly. A schematic representation of the discharge-receiving water system is shown below.

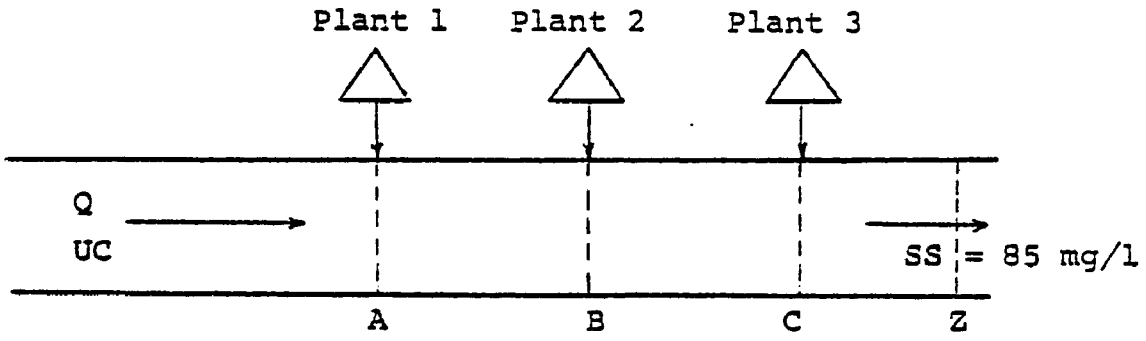


FIGURE 4-2

Time is treated as discrete 1 hour time units. Historical chloride loading data for the three plants is available at 2 hour intervals (M, 2 AM, 4 AM, ... , 10 PM).

The data reveals that the chloride loads from each point-source i follow a normal distribution with mean μ_i and variance σ_i^2. These distribution parameters change over time according to the following pattern:

TABLE 4-1

	MID - 6 AM	8 AM - 2 PM	4 PM - 10 PM
Plant 1	N(5,5)	N(20,20)	N(10,10)
Plant 2	N(10,8)	N(25,20)	N(12,10)
Plant 3	N(15,15)	N(15,15)	N(15,15)

The notation $N(\mu_i, \sigma_i^2)$ implies that the chloride load from Plant 1 is normally distributed with mean μ_i and variance σ_i^2. The mean load from two of the sources shifts with time in a manner which might be expected from the typical operating schedule of many industrial plants. Industries often operate over three distinct shifts, with the morning shift having the greatest production load. Of the 3 plants discharging to the receiving water, Plant 2 has the highest average daily chloride load associated with its effluent. Plant 1 has the lowest. As can be seen from the discharge patterns given in Table 4-1, the highest average chloride loading to the stream occurs during the interval from 8 AM to 2 PM. The chloride load from each plant at odd time points (1 AM, 3 AM, 5 AM..., 11 PM) is assumed to be an average of the loads at adjacent points in time. Therefore the chloride load at 3 PM is the average of the load at 2 PM and 4 PM. Figure 4-3 below shows a typical loading pattern from Plant 1 between the hours of midnight and 10 AM.

At loacation Ƶ on the stream (downstream of all 3 point-sources) a stream standard for chlorides is in effect. The standard states that the concentration of chlorides in the receiving water at Ƶ may never equal or exceed 85 mg/ℓ.

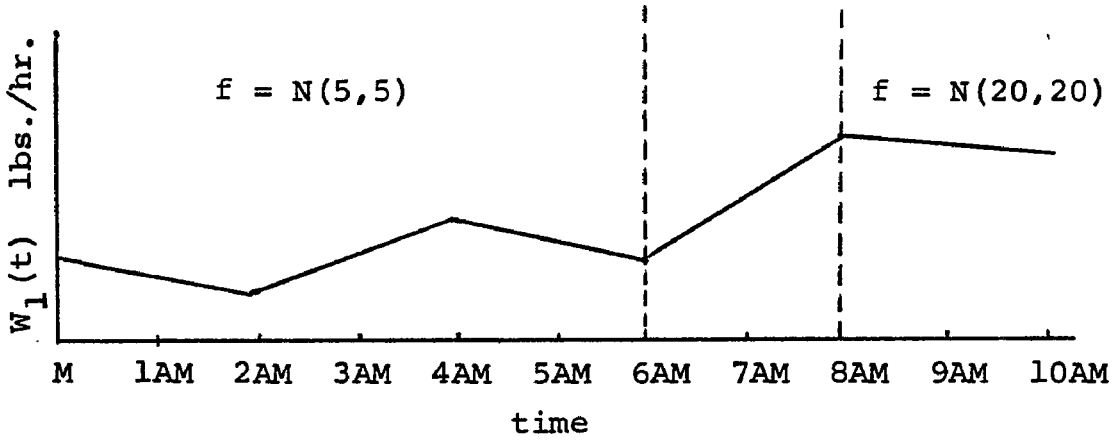


FIGURE 4-3

The objective is to find the optimum allowable selected policy for monitoring the stream at position Z.

The Selected Sampling Policies

Before proceeding with the derivation of the stream model, the sampling policies to be considered in the investigation are defined. On the following page is a list of the 9 selected policies for sampling the receiving water. All input variables to the stream model except the three point-source chloride loadings are assumed to be constant over time. The sampling policies may be viewed as fixed-time policies, i.e., each day samples are taken at the same time as derived from the simulation analysis. A formal statement of a sampling policy is then given as follows:

If, at time t, the estimated probability of violation Prob. ($C(t) \geq 85$ mg/ℓ) exceeds P, a chloride sample is taken. Otherwise a sample is not taken.

The Stream Model

The stretch of receiving water under investigation is broken up into a series of 25 sub-reaches as shown in

TABLE 4-2

Policy No.	Stream Standard (mg/ℓ)	P
1	85	.01
2	85	.05
3	85	.10
4	85	.15
5	85	.20
6	85	.25
7	85	.30
8	85	.40
9	85	.50

Figure 4-4. The time rate-of-change of chloride in each sub-reach is then approximated by means of a differential equation derived below. Mass transport in each stream segment assumes complete-mixing in the water volume, i.e., homogeneous properties throughout the sub-reach.

The differential equation which describes the time rate-of-change of the chloride concentration in the i^{th}

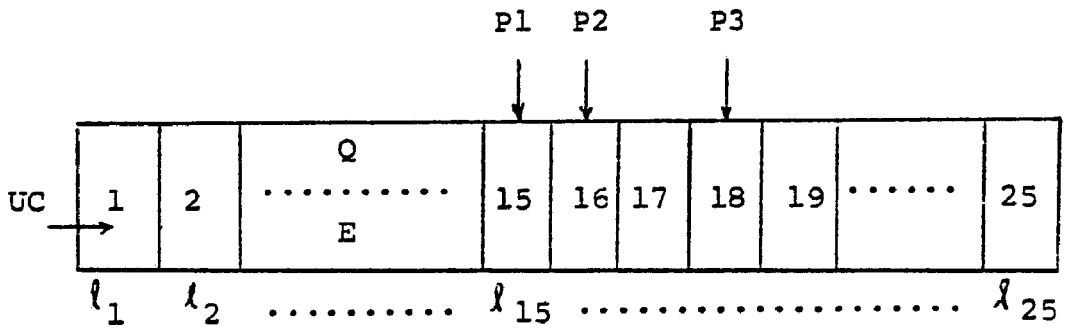


FIGURE 4-4

TABLE 4-3

LIST OF SYSTEM PARAMETERS

E	= .10 mi.^{2}/day	$\ell_1 - \ell_{10}$	= 1000 ft.
A	= 100 ft.^{2}	ℓ_{11}	= 500 ft.
UC	= 2.0 mg/ℓ	ℓ_{12}	= 500 ft.
Q	= 200,000 ft.^{3}/day	$\ell_{13} - \ell_{25}$	= 400 ft.

segment is given below:

$$\frac{dC_i}{dt} = \frac{Q}{V_i}(C_{i-1} - C_i) + \frac{W_i}{V_i} + \frac{EA}{V_i L_i}(C_{i-1} - C_i) + \frac{EA}{V_i L_{i+1}}(C_{i+1} - C_i) \qquad (4\text{-}1)$$

where

C_i = chloride concentration in the i^{th} stream segment at time t (mg/ℓ)

Q = streamflow (volume/time)

W_i = point-source chloride load to the i^{th} segment at time t (mass/time)

E = dispersion coefficient for the system (distance^2/time)

A = cross-sectional area of stream

ℓ_i = length of the i^{th} segment

L_i = characteristic length of the i^{th} segment $(\ell_{i-1} + \ell_i)/2$

V_i = volume of the i^{th} segment $(\ell_i \cdot A)$

It should be noted at this point that the mathematical model given by Equation (4-1) is a simplified representation of material transport in the receiving water. A derivation of a more sophisticated model is given by Thomann (1972). The present model does not weight the concentration at the segment interfaces by a linear interpolation between concentrations in adjacent segments. Although under appropri-

ate conditions the interface interpolation is recommended, the model used here is valid and assumed to provide an accurate description of material transport in the stream under investigation. In particular, the model employed in the present case is a special case of the generalized Thomann equation (6-27). The weighting factor at the interface is assumed to be unity for the upstream concentration and zero for the downstream concentration. This is referred to as a backward difference approximation of the derivative given in the Thomann equation. Equation (4-1) states that the time rate-of-change of chlorides in the i^{th} segment is a function of three phenomena:

1) the amount of chloride advected[2] into and out of the segment due to the flow in the stream.
2) the amount of chloride entering the segment from point-source loads.
3) the amount of chloride entering or leaving the segment due to dispersion.

Because the most upstream and most downstream segments are boundary segments (each having a single adjacent sub-reach), these two segments have unique differential equations associated with chloride transport.

Upstream boundary segment:

$$\frac{dC}{dt}1 = \frac{Q}{V_1}(UC-C_1) + \frac{W}{V_1}1 + \frac{EA}{V_1L_1}(UC-C_1) + \frac{EA}{V_1L_2}(C_2-C_1) \qquad (4\text{-}2)$$

Downstream boundary segment:

$$\frac{dC}{dt}25 = \frac{Q}{V_{25}}(C_{24}- C_{25}) + \frac{W}{V_{25}}25 + \frac{EA}{V_{25}L_{25}}(C_{24} - C_{25}) \qquad (4\text{-}3)$$

In Equation (4-2) the chloride concentration upstream of

---

[2]Material advected into the cell refers to material carried into the cell by means of river flow.

segment 1 is assumed to be a boundary constant (UC=2 mg/ℓ). Additionally, the level of dispersion of material at the upstream boundary is a function of the actual segment 1 length rather than an average characteristic length. Equation (4-3), which describes the time rate-of-change of chloride in the downstream boundary segment, contains only a single dispersive term. Thus it is assumed that there is no transport of material into or out of the stream system due to dispersion at the downstream boundary. This approach was used to simplify the overall analysis by eliminating the need to consider a specific downstream boundary condition. The assumption is not recommended in cases where field observations are available for accurately estimating a downstream boundary condition.

This system of 25 ordinary differential equations is the resulting model for describing the transport of chlorides in the hypothetical receiving water. The technique of solving these equations for the chloride concentration at any given time employs an Adams-Falkner predictor-corrector method. The algorithm, referred to as DVDQ, was written in 1969 by Fred T. Krough at the jet propulsion laboratory in Pasadena, California. The numerical technique employs a discrete set of vectors to approximate the solution at some time t. A multi-step approach is used which computes successive realizations of the vector set. Each member is evaluated as a weighted sum of previously computed vectors and their derivatives. A local error is specified to control the stepsize used in the integration procedure while the solution to the system of differential equations is given at user-specified points (times).

The model was checked by performing a steady-state mass balance on the entire system, i.e., at steady state the total amount of chloride entering the stream must equal the amount leaving. Each equation of the form

$$\frac{dC_i}{dt} = \frac{Q}{V_i}(C_{i-1}-C_i) + \frac{W_i}{V_i} + \frac{EA}{V_i L_i}(C_{i-1}-C_i) + \frac{EA}{V_i L_{i+1}}(C_{i+1}-C_i)$$

is rewritten as (at steady state $V_i \frac{dC_i}{dt} = 0$)

$$\left[-Q-\frac{EA}{L_i}\right] C_{i-1} + \left[Q + \frac{EA}{L_i} + \frac{EA}{L_{i+1}}\right] C_i + \left[-\frac{EA}{L_{i+1}}\right] C_{i+1} = W_i \qquad (4\text{-}4)$$

A tri-diagonal matrix of coefficients for C_{i-1}, C_i and C_{i+1} is obtained and the inverse of this matrix is referred to as a steady state response matrix. A Gaussian elimination algorithm was used to perform the matrix inversion. The receiving water system shown in Figure 4-2 was analyzed; the system parameters and resulting steady-state solution are given in Table 4-4. The total amount of chloride (in lbs/day) entering the system can be written as:

$$W_{entering} = UC \cdot Q + WC1 + WC2 + WC3$$
$$= 1032.79 \text{ lbs/day} \qquad (4\text{-}5)$$

Similarly, the total amount of chloride leaving the system at steady state is given as:

$$W_{leaving} = C_{ss}(25)\cdot Q + \frac{EA}{\ell_1}(C_{ss}(1) - UC)$$
$$= 1032.79 \text{ lbs/day} \qquad (4\text{-}6)$$

Because the values of $W_{entering}$ and $W_{leaving}$ are in agreement within .02%, the system of differential equations is accepted as valid. As a further check, however, the steady-state solution obtained from the computer algorithm (Appendix G) was also compared to the matrix inversion result. It can be shown that the time required for a system to reach 99% of steady state conditions is approximately 4.6 times the average retention time ($\frac{Vol}{Q}\cdot$) for the system. In the present case the average retention

TABLE 4-4

Steady-State Chloride Conc.'s (mg/l) for 25 Segment Stream Model

System Description:

A = 100.0 $ft.^2$	LOAD1 = 15	WC1 = 14.0 lbs/hr
E = 0.1 $mi.^2$/day	LOAD2 = 16	WC2 = 14.0 lbs/hr
Q = 200000 $ft.^3$/day	LOAD3 = 18	WC3 = 14.0 lbs/hr

REACH	LENGTH (ft.)	UC = 0.0	UC = 2.0
1	1000.	0.0424	2.0424
2	1000.	0.1153	2.1153
3	1000.	0.2404	2.2404
4	1000.	0.4553	2.4553
5	1000.	0.8243	2.8243
6	1000.	1.4580	3.4580
7	1000.	2.5465	4.5465
8	1000.	4.4157	6.4157
9	1000.	7.6260	9.6260
10	1000.	13.1392	15.1392
11	500.	20.2406	22.2406
12	500.	27.5222	29.5222
13	400.	36.4263	38.4263
14	400.	46.8961	48.8961
15	400.	60.3704	62.3704
16	400.	69.9911	71.9911
17	400.	74.6523	76.6523
18	400.	80.6511	82.6511
19	400.	80.6511	82.6511
20	400.	80.6511	82.6511
21	400.	80.6511	82.6511
22	400.	80.6511	82.6511
23	400.	80.6511	82.6511
24	400.	80.6511	82.6511
25	400.	80.6511	82.6511

time can be calculated as

$$T_{ret} = \frac{Vol.}{Q} = \frac{\sum_{i-1}^{25} V_i}{Q} = \frac{1620000\ ft^3}{2000000\ ft^3/day} = 8.1 \text{ days}$$

Therefore the time to steady state (t*) is

$$t^* = 4.6(8.1) \doteq 37.26 \text{ days}; \doteq 894 \text{ hours}$$

Rather than waste the computer time required to run the program over an 894-hour period, an alternative approach was used to check the computer algorithm. The steady-state output obtained by the matrix-inversion method was fed into the computer program as the initial condition for the stream model. The program was then run over a reduced 100-hour period to observe if the system response at the end of the interval had changed. Table 4-5 gives the results of the analysis. The computer output after 100 hours did not significantly change from the given initial conditions. It is therefore concluded that the computer algorithm provides an accurate solution to the system of ordinary differential equations.

An alternative check on the validity of the mathematical model involves a comparison of steady-state model output with a zero dispersion coefficient to a known analytical solution. Assuming steady-state chloride loads of W_1, W_2 and W_3 for plants 1, 2, and 3, respectively, and an upstream background chloride concentration of UC mg/ℓ, the chloride concentration in the stream between plants 1 and 2 may be written

$$C_1 = \frac{W_1}{Q} + UC$$

Similarly, the concentration between plants 2 and 3 at steady-state is

$$C_2 = \frac{W_1+W_2}{Q} + UC$$

TABLE 4-5

Segment	Initial Conditions to Computer Algorithm (Matrix Inversion steadystate result)	Computer Output after 100 hours
1	2.04	2.04
2	2.12	2.12
3	2.24	2.24
4	2.46	2.45
5	2.82	2.82
6	3.46	3.46
7	4.55	4.54
8	6.42	6.41
9	9.63	9.62
10	15.14	15.13
11	22.24	22.24
12	29.52	29.52
13	38.43	38.42
14	48.90	48.90
15	62.37	62.38
16	71.99	72.00
17	76.65	76.67
18	82.65	82.67
19	82.65	82.67
20	82.65	82.67
21	82.65	82.67
22	82.65	82.67
23	82.65	82.67
24	82.65	82.67
25	82.65	82.67

Finally, the steady-state concentration downstream of all 3 plants may be expressed as

$$C_3 = \frac{W_1 + W_2 + W_3}{Q} + UC$$

A steady-state chloride load of 14.0 lbs/hr. for each of the 3 plants is selected. The computer algorithm response and analytical solution for each of the 25 stream segments is given below:

Segment	Computer Output	Analytical Solution
1-14	2.0	2.0
15	28.9	28.9
16-17	55.8	55.7
18-25	82.7	82.6

Generation of Point-Source Chloride Loads

Once the stream model is derived and checked by means of a mass-balance, chloride loadings for each point-source are required for input to the model. As previously discussed, the probability functions describing the manner in which each plant discharges to the receiving water are known and given in Table 4-1. A random number generator is used to generate chloride loads from each plant according to these appropriate probability distributions.

Simulation Output

A computer program called Deckfile (Appendix G) is used to solve for the chloride concentration at segment 19 (Figure 4-4) of the stream model. The program employs DVDQ to integrate the system of 25 ordinary differential equations, and the solution to the equations is given at 1 hour time intervals. Four replications or simulation runs supply the chloride data, each run consisting of 720 output points. The first 120 hours of output data are ignored in the analysis, as these concentrations repre-

sent a transient-state. During the first 120 hours of each replication, the simulation output is effected by the initial conditions of the system. These observations are therefore not representative of the true chloride response in the receiving water. The sesnitivity of the output to variations in the initial conditions was tested and the results are plotted in Figure 4-5. It can be seen that after approximately 120 hours the effect of the initial conditions on the chloride concentration at segment 19 is significantly reduced. Therefore the remaining 600 hours of output comprise the chloride data analyzed from each replication.

A single simulation run provides 25 chloride observations for each hour of the day. Thus the four replications provide a total of 100 observations for each hour. Constraints on computer time and available data storage space made this a convenient data base for analysis. Figures 4-6, 4-7, 4-8 and 4-9 show the resulting chloride response from the 4 simulation runs.

The output data is now used to determine the probability of a stream standard violation ($C(t) \geq 85$ mg/ℓ) at each hour of the day. The probability of violation at time t ($\hat{\rho}_t$) can be estimated by Equation (4-7) below.

$$\hat{\rho}_t = \frac{n}{N}_t \qquad (4\text{-}7)$$

where

n_t = total number of times $C \geq 85$ mg/ℓ at time t

N_t = total number of observations at time t

Table 4-6 gives the resulting probabilities of violation for each of the four replications, as well as the overall average. A graphic representation of the data is shown in Figure 4-10.

The above probabilities provide the necessary information for exact definition of the selected sampling

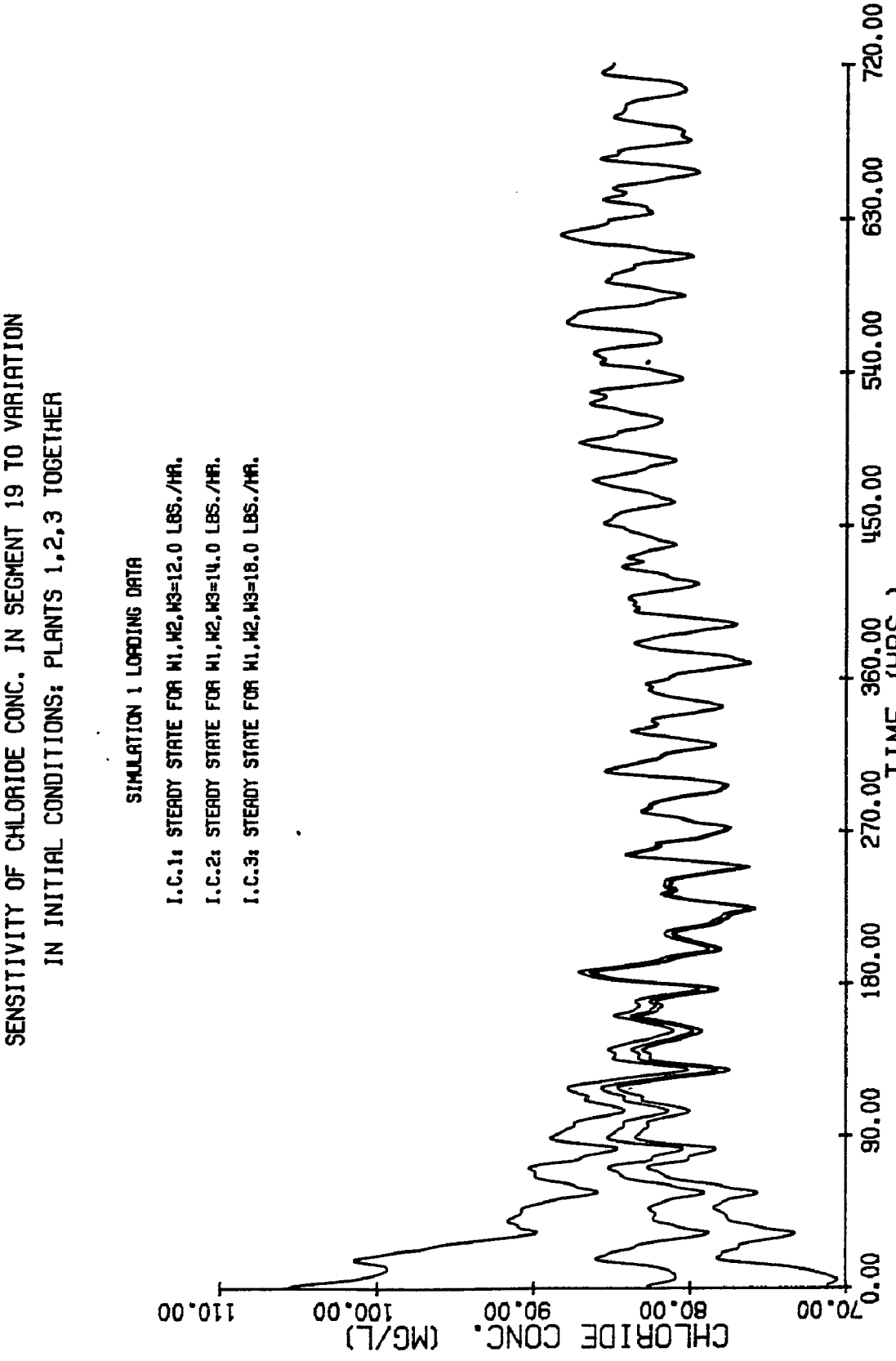

SENSITIVITY OF CHLORIDE CONC. IN SEGMENT 19 TO VARIATION
IN INITIAL CONDITIONS: PLANTS 1,2,3 TOGETHER
SIMULATION 1 LOADING DATA
I.C.1: STEADY STATE FOR W1,W2,W3=12.0 LBS./HR.
I.C.2: STEADY STATE FOR W1,W2,W3=14.0 LBS./HR.
I.C.3: STEADY STATE FOR W1,W2,W3=18.0 LBS./HR.
CHLORIDE CONC. (MG/L)
70.00
80.00
90.00
100.00
110.00
0.00
90.00
180.00
270.00
360.00
450.00
540.00
630.00
720.00
TIME (HRS.)

FIGURE 4-5

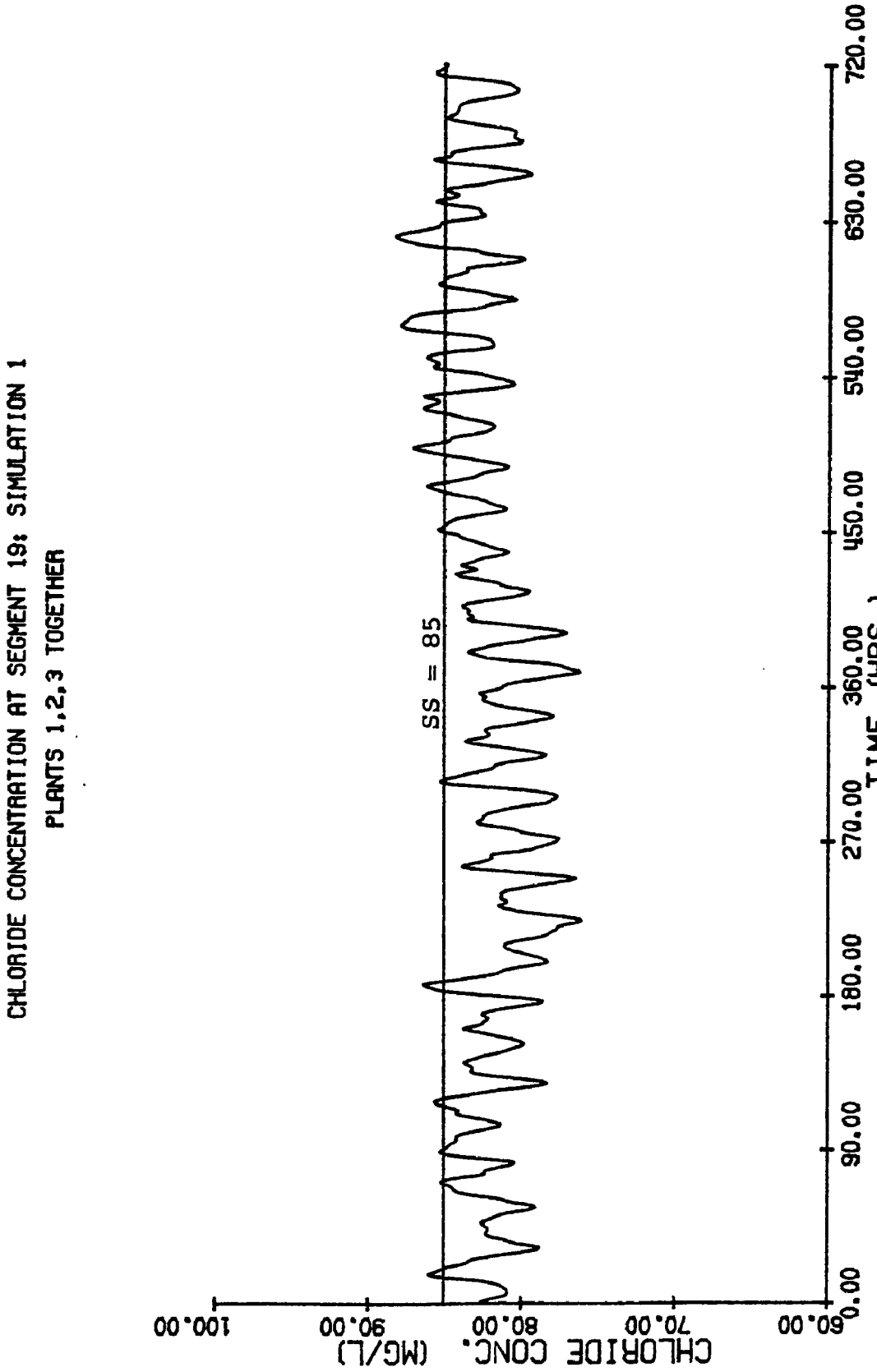

CHLORIDE CONCENTRATION AT SEGMENT 19: SIMULATION 1
PLANTS 1,2,3 TOGETHER
SS = 85
CHLORIDE CONC. (MG/L)
100.00
90.00
80.00
70.00
60.00
TIME (HRS.)
0.00
90.00
180.00
270.00
360.00
450.00
540.00
630.00
720.00

FIGURE 4-6

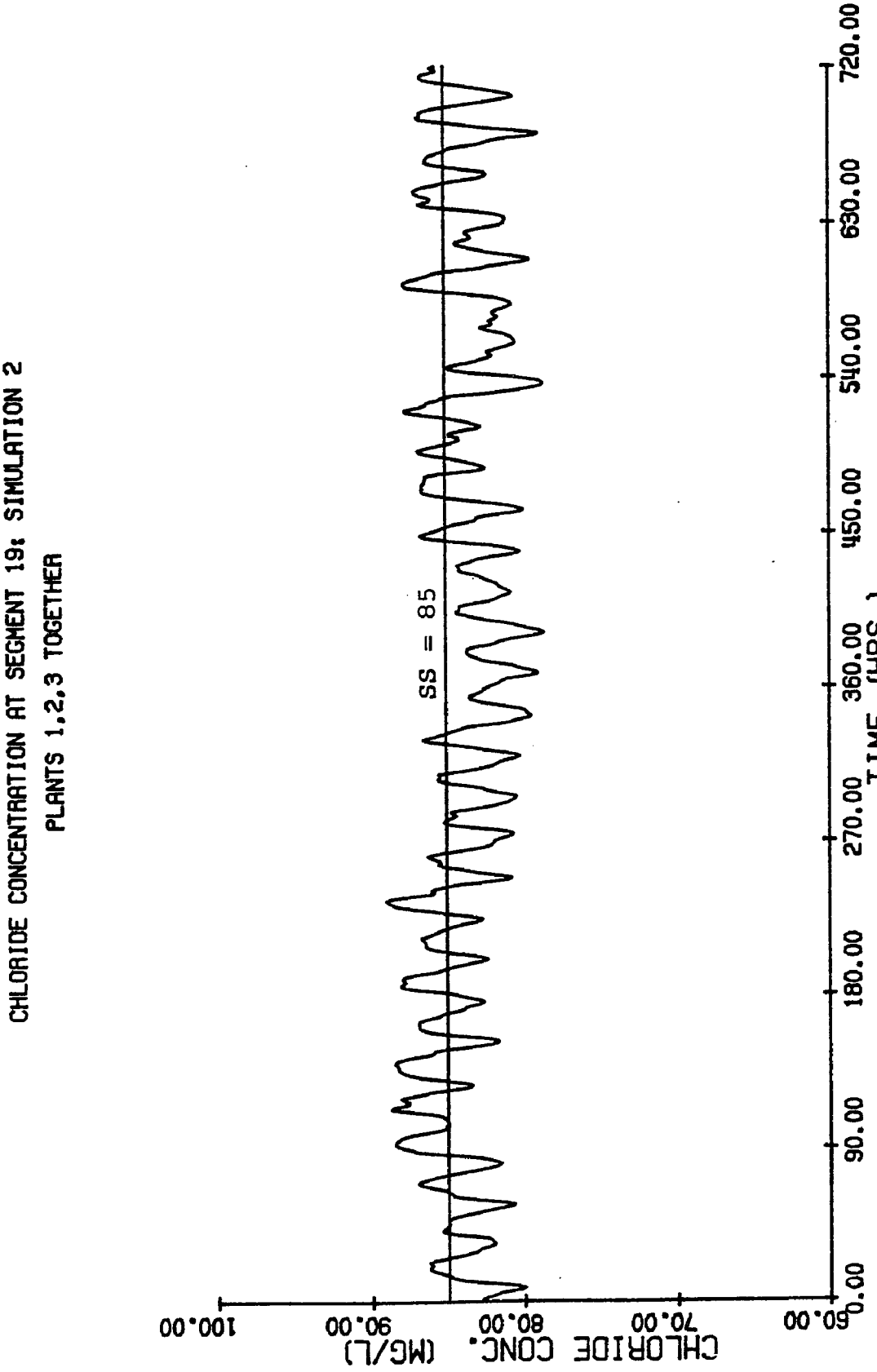

CHLORIDE CONCENTRATION AT SEGMENT 19: SIMULATION 2
PLANTS 1,2,3 TOGETHER
SS = 85
CHLORIDE CONC. (MG/L)
100.00
90.00
80.00
70.00
60.00
0.00
90.00
180.00
270.00
360.00
450.00
540.00
630.00
720.00
TIME (HRS.)

FIGURE 4-7

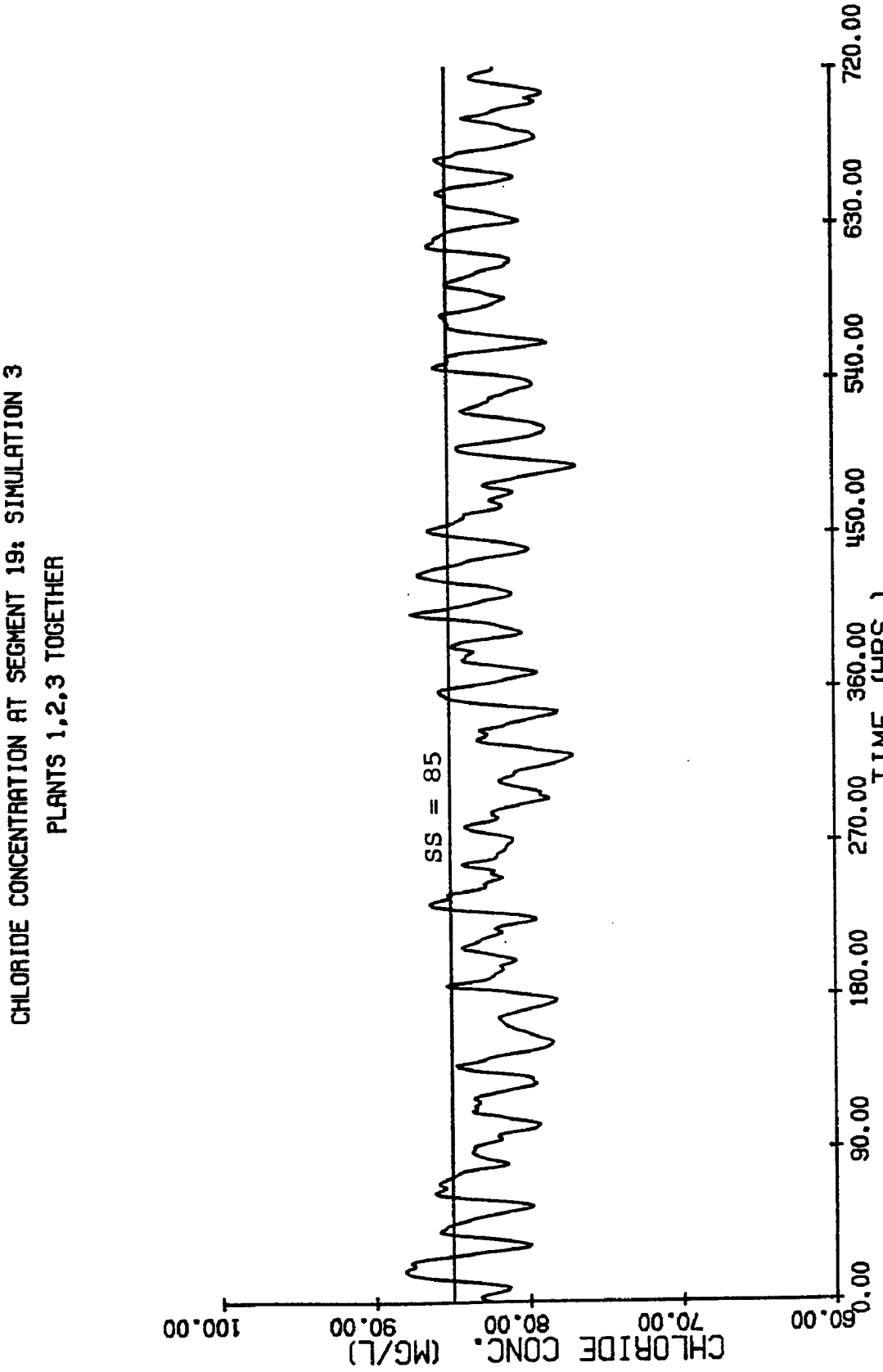

CHLORIDE CONCENTRATION AT SEGMENT 19: SIMULATION 3
PLANTS 1,2,3 TOGETHER
SS = 85
CHLORIDE CONC. (MG/L)
60.00
70.00
80.00
90.00
100.00
0.00
90.00
180.00
270.00
360.00
450.00
540.00
630.00
720.00
TIME (HRS.)

FIGURE 4-8

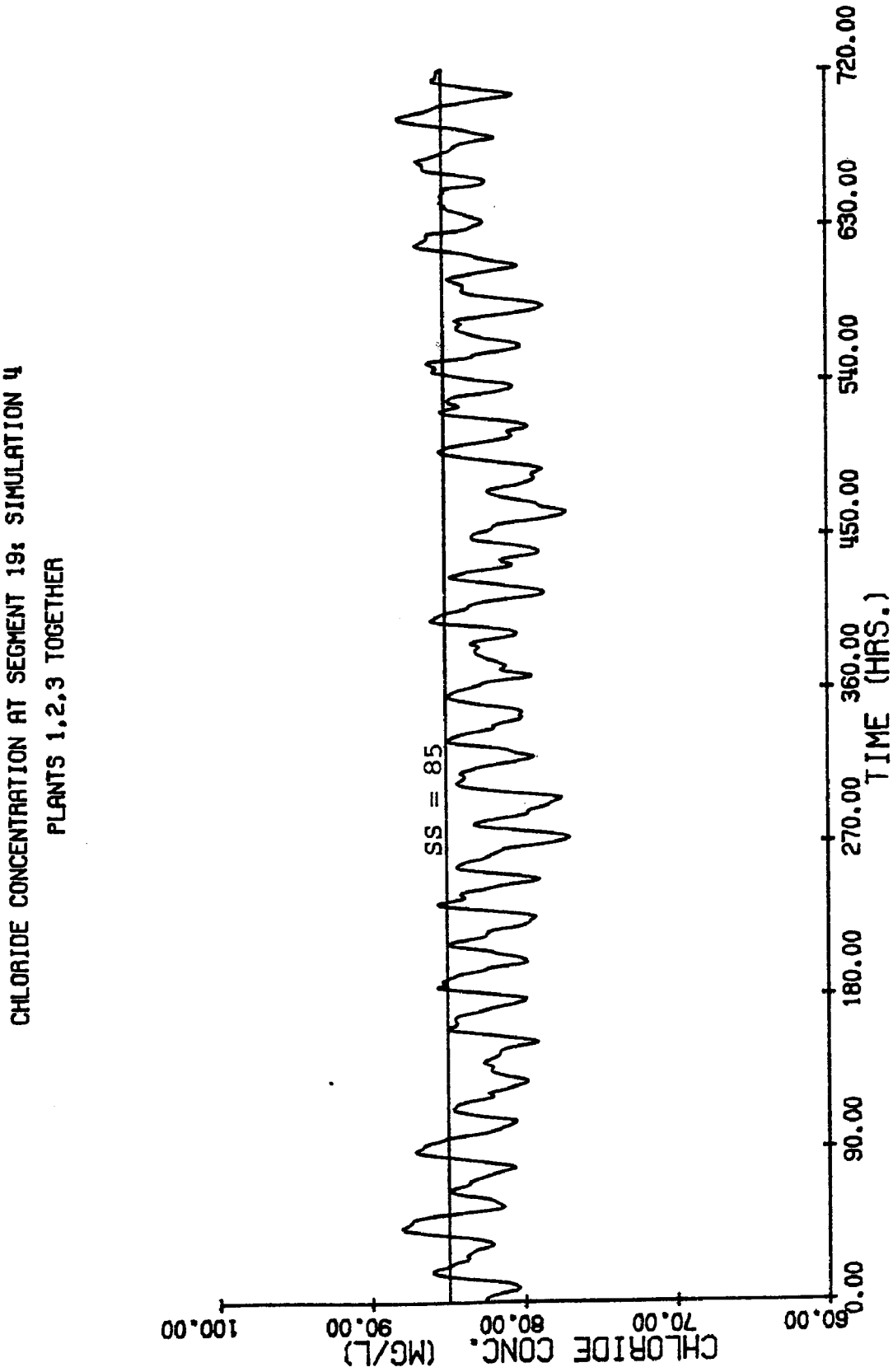

CHLORIDE CONCENTRATION AT SEGMENT 19: SIMULATION 4
PLANTS 1,2,3 TOGETHER
SS = 85
CHLORIDE CONC. (MG/L)
100.00
90.00
80.00
70.00
60.00
0.00
90.00
180.00
270.00
360.00
450.00
540.00
630.00
720.00
TIME (HRS.)

FIGURE 4-9

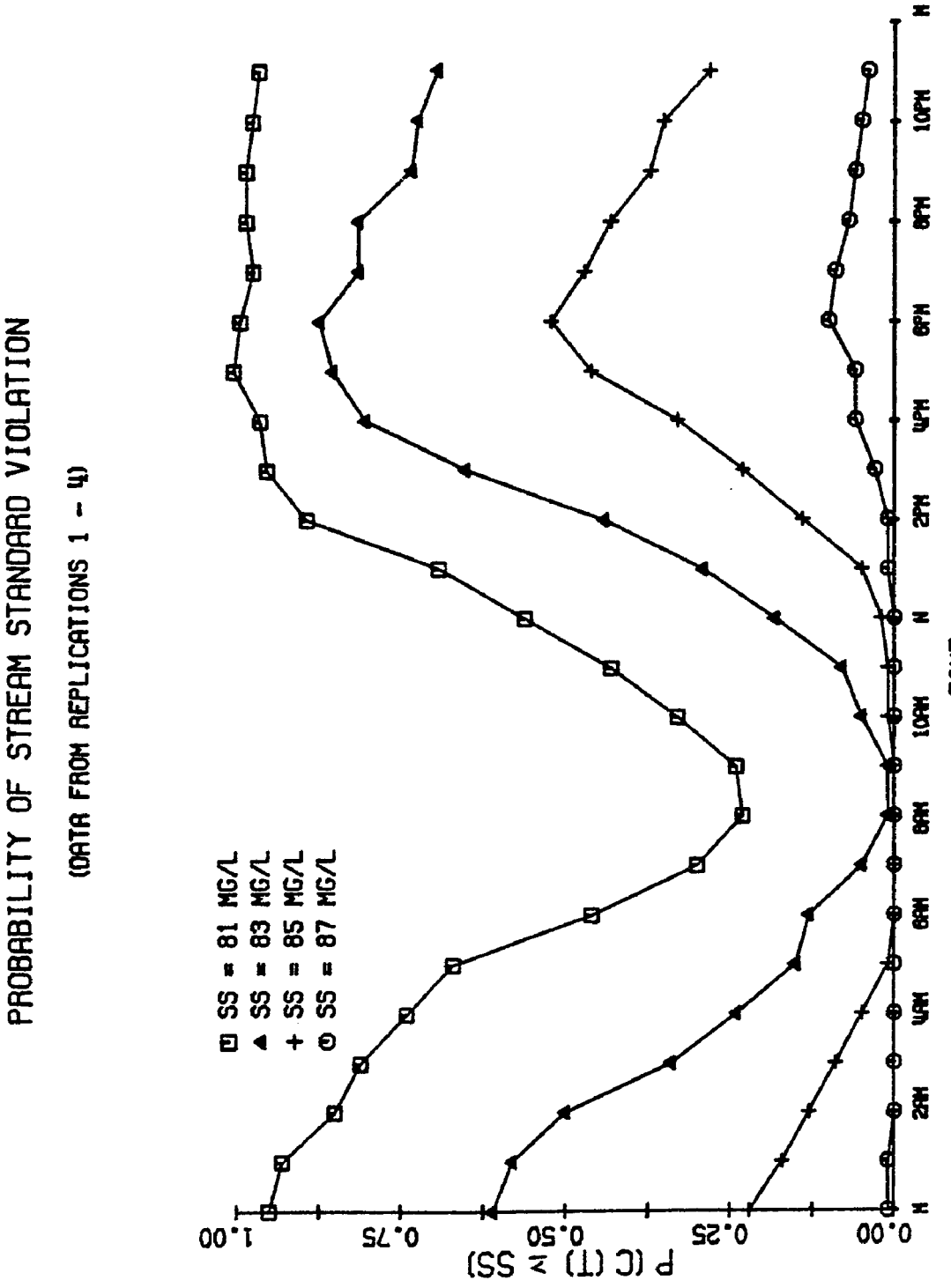

PROBABILITY OF STREAM STANDARD VIOLATION
(DATA FROM REPLICATIONS 1 - 4)
SS = 81 MG/L
SS = 83 MG/L
SS = 85 MG/L
SS = 87 MG/L
P (C (T) ≥ SS)
0.00
0.25
0.50
0.75
1.00
N
2AM
4AM
6AM
8AM
10AM
N
2PM
4PM
6PM
8PM
10PM
N
TIME

FIGURE 4-10

TABLE 4-6

PROBABILITY OF STREAM STANDARD VIOLATION
(CHLORIDE STANDARD = 85 MG/L)

TIME	RUN 1	RUN 2	RUN 3	RUN 4	MEAN
12M	.200	.440	.120	.120	.220
1AM	.160	.360	.080	.080	.170
2AM	.120	.280	.000	.120	.130
3AM	.040	.240	.000	.080	.090
4AM	.040	.160	.000	.000	.050
5AM	.040	.000	.000	.000	.010
6AM	.000	.000	.000	.000	.000
7AM	.000	.000	.000	.000	.000
8AM	.000	.000	.000	.000	.000
9AM	.000	.000	.000	.000	.000
10AM	.000	.040	.000	.000	.010
11AM	.000	.040	.000	.000	.010
12N	.000	.080	.000	.000	.020
1PM	.000	.160	.000	.040	.050
2PM	.040	.440	.040	.040	.140
3PM	.080	.600	.120	.120	.230
4PM	.280	.640	.160	.240	.330
5PM	.400	.720	.320	.400	.460
6PM	.480	.720	.440	.440	.520
7PM	.400	.680	.400	.400	.470
8PM	.360	.680	.320	.360	.430
9PM	.320	.640	.280	.240	.370
10PM	.280	.520	.320	.280	.350
11PM	.200	.480	.200	.240	.280

TABLE 4-7

SAMPLING TIMES AND DETECTION PROBABILITIES FOR POLICIES 1-9
CHLORIDE STREAM STANDARD = 85 MG/L

	SAMPLE IF $\hat{P}$(VIOLATION) > P								
TIME	P=.01 PLAN1	P=.05 PLAN2	P=.10 PLAN3	P=.15 PLAN4	P=.20 PLAN5	P=.25 PLAN6	P=.30 PLAN7	P=.40 PLAN8	P=.50 PLAN9
M	S	S	S	S	S	-	-	-	-
1AM	S	S	S	S	-	-	-	-	-
2AM	S	S	S	-	-	-	-	-	-
3AM	S	S	-	-	-	-	-	-	-
4AM	S	S	-	-	-	-	-	-	-
5AM	S	-	-	-	-	-	-	-	-
6AM	-	-	-	-	-	-	-	-	-
7AM	-	-	-	-	-	-	-	-	-
8AM	-	-	-	-	-	-	-	-	-
9AM	-	-	-	-	-	-	-	-	-
10AM	S	-	-	-	-	-	-	-	-
11AM	S	-	-	-	-	-	-	-	-
N	S	-	-	-	-	-	-	-	-
1PM	S	S	-	-	-	-	-	-	-
2PM	S	S	S	-	-	-	-	-	-
3PM	S	S	S	S	S	-	-	-	-
4PM	S	S	S	S	S	S	S	-	-
5PM	S	S	S	S	S	S	S	S	-
6PM	S	S	S	S	S	S	S	S	S
7PM	S	S	S	S	S	S	S	S	-
8PM	S	S	S	S	S	S	S	S	-
9PM	S	S	S	S	S	S	S	-	-
10PM	S	S	S	S	S	S	S	-	-
11PM	S	S	S	S	S	S	-	-	-
P(DET. OF EACH TIME)	1.00	0.99	0.95	0.88	0.84	0.74	0.73	0.43	0.12
P(DET. OF VIOLATION)	1.00	1.00	1.00	0.98	0.98	0.95	0.95	0.90	0.87
SAMPLES REQUIRED (25 DAYS)	500	400	325	275	250	200	175	100	25

policies given in Table 4-2. The specific sampling times for each policy are indicated in Table 4-7. As an example of the difference among the plans, Policy 1 calls for a sample to be taken at each hour of the day except 6 AM through 9 AM--a total of 20 observations per day. On the other hand, Policy 9 requires only a single chloride sample each day--that coming at 6 PM.

The Required Number of Simulated Observations

Although the probability of violation $\hat{p}$ is presently based upon the analysis of 100 simulated observations, the policy designer will most often by concerned with two questions in deriving these probabilities:

1) What is the reliability of the violation probabilities given by the simulation?
2) What sample size or total number of simulated observations is required to obtain a specified reliability for these probabilities?

In attempting to answer the first question, Kleijnen (1974) suggests that confidence intervals on these probabilities may be constructed from the table of the binominal distribution or from the normal approximation to the binomial distribution. Using the latter approach, a 95% confidence interval on the probability of violation at midnight is derived:

$$P\{.14 \leq \hat{p}_M \leq .30\} \doteq .95$$

Therefore $\hat{p}_M = .22$ has a 95% confidence interval ranging from .14 to .30. This same technique can be used to obtain a measure of reliability for each of the $\hat{p}$ given in Table 4-6.

The second question listed above presents a more complex problem--a different problem for the policy designer. Suppose it is necessary to simulate enough chloride observations so that the value of $\hat{p}$ will be within c of p at the (1-) confidence level:

$$\text{Prob.}\ \{|\hat{p} - p| \leq c\} = 1-\alpha.$$

The required sample size may be written as

$$n = \left[\frac{z^{\alpha/2}}{c}\right]^2 \cdot \left[\hat{p}(1-\hat{p})\right]$$

where $z^{\alpha/2}$ is the value of the standard normal variate at $\alpha/2$. Unfortunately the value of n will vary with the value of $\hat{p}$. Because the policy designer does not know the value of $\hat{p}$ before initiating the simulation, he therefore does not know how large a sample size is required. Kleijnen has suggested two alternatives for dealing with this difficulty. The first uses a sequential computation of $\hat{p}$ as the simulation proceeds. The simulation run is periodically stopped to check if the relationship $p(|\hat{p}-p| \leq c) = 1-\alpha$ is satisfied. If not, the stopping conditions become the new initial conditions and the simulation is continued over another specified period. This iterative procedure is followed until the desired reliability for $\hat{p}$ is achieved.

The second technique employs an initial run of length n_o to arrive at an estimate of the true variance of $\hat{p}$. This estimate is then used to solve for the required number of samples or observations in the total simulation:

$$n = \left[\frac{z^{\alpha/2}}{c}\right]^2 \cdot \left[\hat{p}\ (1-\hat{p})\right]_{n_o}$$

This procedure is referred to as double-sampling. Because of the high costs associated with many simulations, careful consideration must be given to the value of n_o used to determine $\hat{p}$.

As an example of how this method may be applied to the present analysis, suppose it is necessary to estimate the number of observations at midnight such that

$$\text{Prob}\ [|\hat{p}_M - p_M| \leq .05] = .90$$

Taking the data from replication 1 as the basis for computation (n_o = 25), the following result is obtained:

$$n = \left[\frac{1.645}{.05}\right]^2 \cdot \left[.2(1-.2)\right]$$

$$= 173$$

From the above computation, another 1973-25 = 148 observations at midnight are required to satisfy the given reliability constraint on $\hat{p}_M$. This same approach can then be applied to each discrete time under investigation in order to find the limiting number of required observations.

Sampling Policy Costs

Each of the selected sampling policies is evaluated on the basis of 1) total cost and 2) probability of violation detection over a 25-day planning period. The particular cost functions are hypothetical, representing typical sampling and damage costs which might be encountered in real systems. Looking at a given replication output, if a stream standard violation occurs at 5 PM and the policy calls for an observation each day at 5 PM, the violation is assumed to be detected. If a sample is not taken at 5 PM, suppose a cost is incurred proportional to the resulting delay to detection. Figure 4-11 shows two possible costs associated with the delay to detection. These two specific cost functions are used as a means of comparison in the analysis, one a linear function of delay and the other a non-linear function of delay. The total cost of a selected policy over the 25-day planning period has two components:

1. Fixed cost of sampling:

$$FC = 10 \cdot n \qquad (4\text{-}8)$$

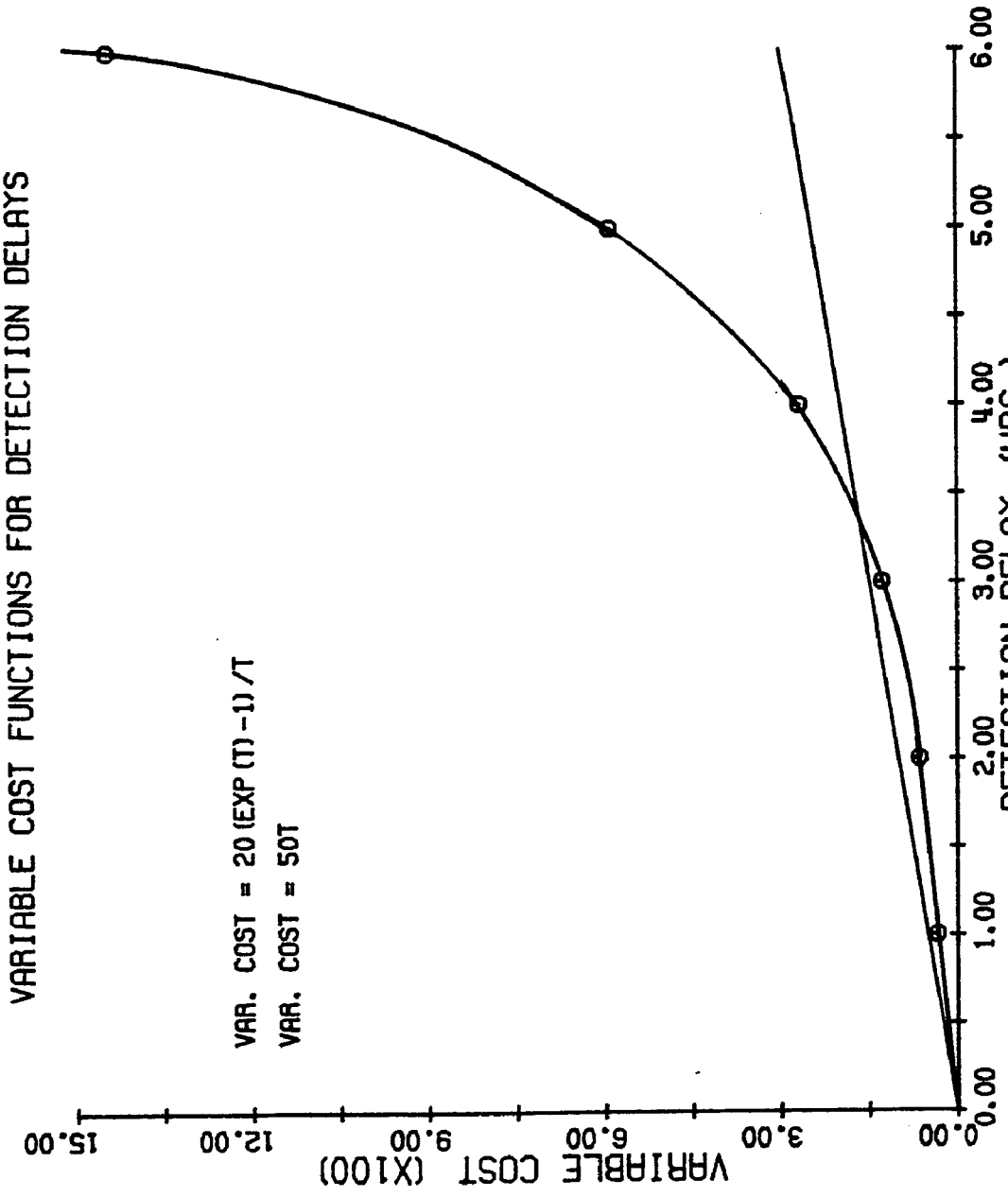

VARIABLE COST FUNCTIONS FOR DETECTION DELAYS
VAR. COST = 20(EXP(T)-1)/T
VAR. COST = 50T
VARIABLE COST (X100)
0.00
3.00
6.00
9.00
12.00
15.00
DETECTION DELAY (HRS.)
0.00
1.00
2.00
3.00
4.00
5.00
6.00

FIGURE 4-11

TABLE 4-8

COST ANALYSIS FOR SAMPLING POLICIES 1 - 9
(PLANNING PERIOD = 25 DAYS)

CHLORIDE STANDARD = 81 MG/L

POLICY NO.	FIXED COST	VAR. COST (EXP.)	TOTAL COST	VAR. COST (LIN.)	TOTAL COST
1	6000.	0.	6000.	0.	6000.
2	6000.	0.	6000.	0.	6000.
3	6000.	0.	6000.	0.	6000.
4	6000.	0.	6000.	0.	6000.
5	6000.	0.	6000.	0.	6000.
6	5500.	74.	5574.	113.	5613.
7	5500.	74.	5574.	113.	5613.
8	5000.	220.	5220.	300.	5300.
9	4500.	525.	5025.	613.	5113.

CHLORIDE STANDARD = 83 MG/L

1	6000.	0.	6000.	0.	6000.
2	5500.	0.	5500.	0.	5500.
3	4750	90.	4840.	138.	4888.
4	4500.	90.	4590.	138.	4638.
5	4000.	260.	4260.	350.	4350.
6	3750.	260.	4010.	350.	4100.
7	3500.	617.	4117.	700.	4200.
8	3250.	633.	3883.	725.	3975.
9	3000.	1420.	4420.	1250.	4250.

CHLORIDE STANDARD = 85 MG/L

1	5000.	0.	5000.	0.	5000.
2	4000.	40.	4040.	50.	4050.
3	3250.	109.	3359.	113.	3363.
4	2750.	320.	3070.	313.	3063.
5	2500.	320.	2820.	313.	2813.
6	2000.	890.	2890.	675.	2675.
7	1750.	890.	2640.	675.	2425.
8	1000.	2064.	3064.	1238.	2238.
9	250.	4381.	4631.	1888.	2138.

CHLORIDE STANDARD = 87 MG/L

1	3250.	0.	3250.	0.	3250.
2	1750.	65.	1815.	88.	1838.
3	250.	284.	534.	250.	500.
4	0.	7116.	7116.	713.	713.
5	0.	7116.	7116.	713.	713.
6	0.	7116.	7116.	713.	713.
7	0.	7116.	7116.	713.	713.
8	0.	7116.	7116.	713.	713.
9	0.	7116.	7116.	713.	713.

2. Variable detection delay cost:

$$\text{a)}\quad C_v = 50 \cdot \sum_{i=1}^{n_v} t_i \qquad t_i = 1, 2, \ldots \qquad (4\text{-}9)$$

$$\text{b)}\quad C_v = \sum_{i=1}^{n_v} \frac{20(e^{t_i}-1)}{t_i} \qquad t_i = 1, 2, \ldots, 11 \qquad (4\text{-}10)$$

where n = total number of samples required over the planning period.

n_v = number of violations occurring during the planning period, and

t_i = time delay (hrs.) to detection of i^{th} violation.

Thus the expressions for total cost are given as

$$1)\quad C_T = 10 \cdot n + 50 \cdot \sum_{i=1}^{n_v} t_i \qquad t_i = 1, 2, \ldots. \qquad (4\text{-}11)$$

$$2)\quad C_T = 10 \cdot n + \sum_{i=1}^{n_v} \frac{20(e^{t_i}-1)}{t_i} \qquad t_i = 1, 2, \ldots .11 \qquad (4\text{-}12)$$

The variable cost in Equation 4-12 is always zero for $t_i = 0$ and is assumed to reach a maximum after 11 time units, remaining constant for $t_i \geq 11$.

The probability of violation detection (P_D) is calculated as follows:

$$P_D = \sum_{r=1}^{4} n_{v,r} / \sum_{r=1}^{4} N_{v,r} \qquad (4\text{-}13)$$

where $n_{v,r}$ = number of violations detected in replication r, and

$N_{v,r}$ = number of violations occurring in replication r.

Similarly, the total variable cost over the 25-day planning period (C_v) is taken as the average over all 4 replications:

$$C_v = \sum_{r=1}^{4} C_{v,r} / 4 \qquad (4\text{-}14)$$

where $C_{v,r}$ = total variable cost incurred during the r^{th} replication.

Therefore the total cost (C_T) of a selected policy over the planning period is given as

$$C_T = 10n + \sum_{r=1}^{4} C_{v,r}/4 \qquad (4\text{-}15)$$

Table 4-7 contains the detection probabilities and Table 4-8 the total costs for each of the 9 selected probability threshold policies. The data is presented graphically in Figures 4-12 and 4-13. It should be noted that the damage functions are independent of the magnitude of the violation in terms of concentration above the standard.

Selecting the Optimum Policy

The optimum policy is that which yields the required detection probability P_D at minimum cost over the 25-day planning period. For clarity, the detection probability under definition 1 of a violation (a violation event) will be referred to as P_{D1}; P_{D2} will represent the detection probability under definition 2 (a violation period).

As an example of how the optimum policy is selected, suppose a policy is sought which will detect at least 95% of all stream standard violations, i.e., the detection requirement is $P_{D1} \geq 95\%$. Assuming (4-11) represents the damage function in effect, the upper half of Figure 4-12 is entered and those policies which lay below the .95 horizontal line are immediately eliminated from consideration. In this instance Policies 8 and 9 both lay outside the feasibility region. Of the remaining policies, the optimum is taken as that which lays to the extreme left-hand side of the feasibility region. Figure 4-13 shows Policy 7 to be the optimum in the present case. Table 4-9 gives the optimum policy for various values of P_{D1}. Under both the non-linear and linear cost functions, sampling Policy 3 yields the minimum cost for a required detection probability of at least .99. In this case 13

PROBABILITY OF VIOLATION DETECTION VS. TOTAL COST FOR SAMPLING POLICIES 1 - 9

CHLORIDE THRESHOLD = 85 MG/L

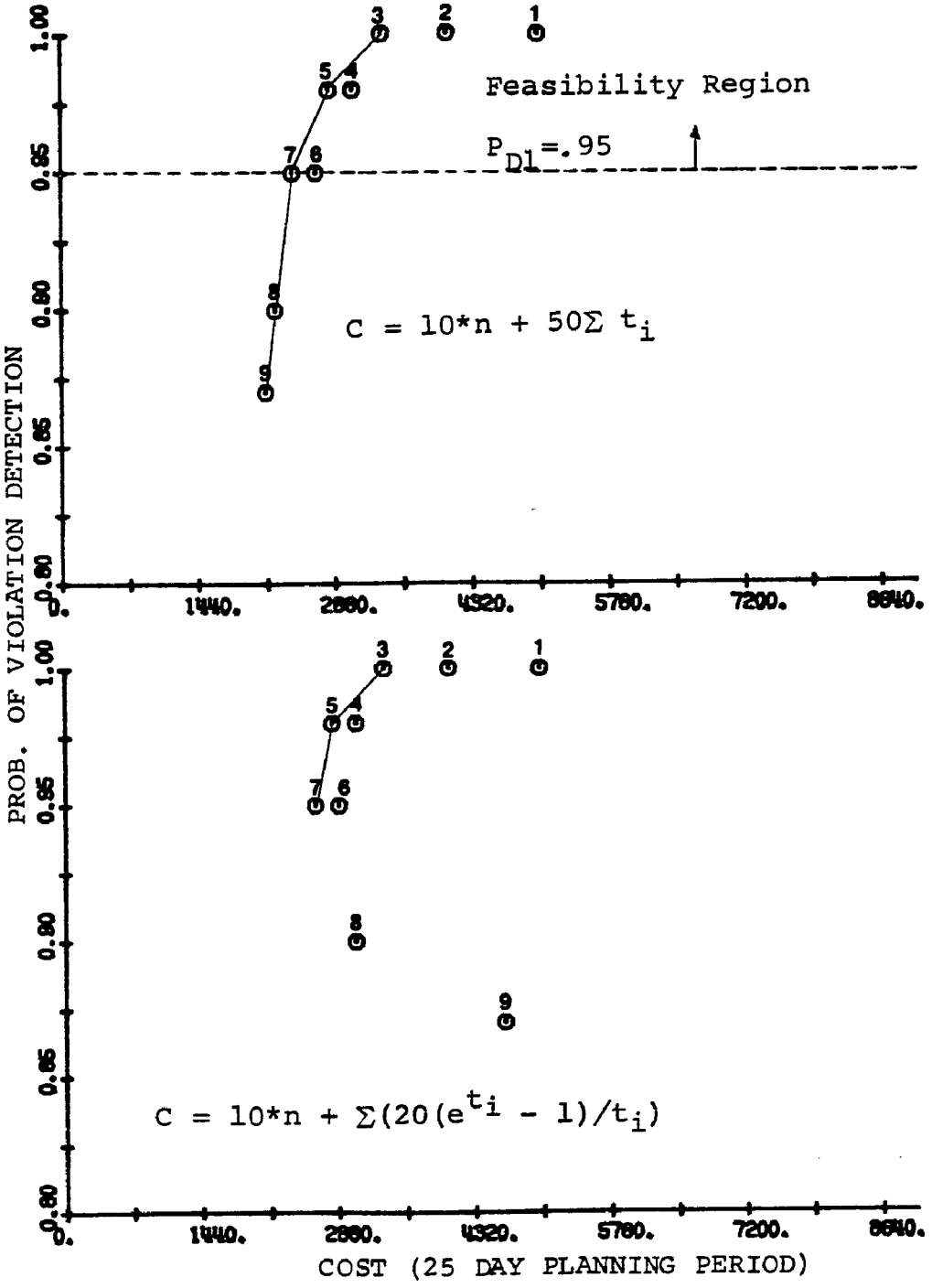


FIGURE 4-12

PROBABILITY OF ALL VIOLATION TIMES DETECTED
VS. TOTAL COST FOR SAMPLING POLICIES 1 - 9
CHLORIDE THRESHOLD = 85 MG/L

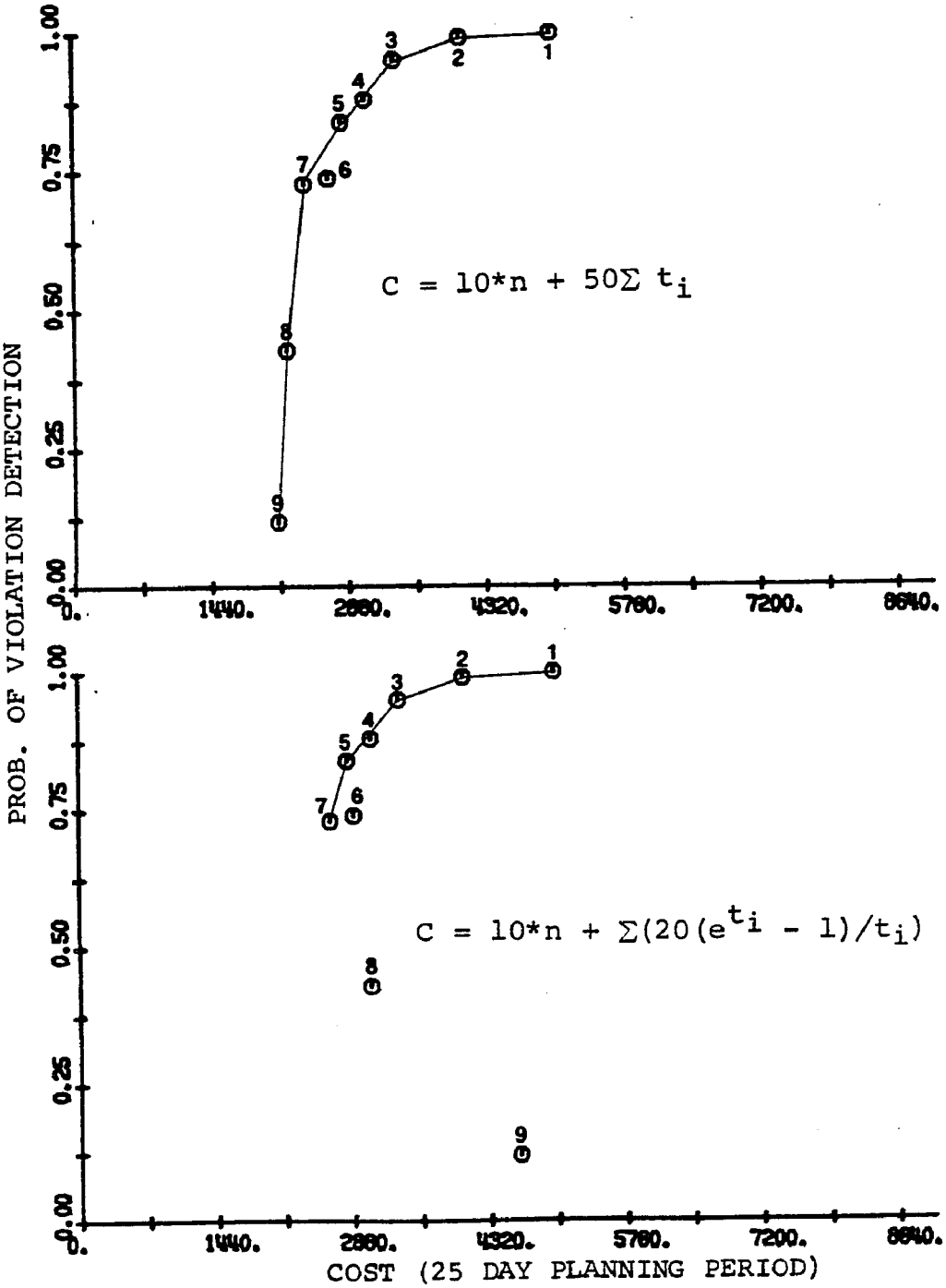


FIGURE 4-13

TABLE 4-9

STREAM STANDARD = 85 mg/ℓ

REQUIRED DETECTION PROBABILITY (P_{D1})	($C=10 \cdot n+\Sigma(20(e^{t}i-1)/t_i$)	($C=10 \cdot n+50\Sigma t_i$)
1.00	3	3
.99	3	3
.95	5	7
.90	7	8
.85	7	9
.80	7	9
.70	7	9
.60	7	9
.50	7	9
.25	7	9
.10	7	9

chloride observations are made each day, with no samples taken between the hours of 2 AM and 2 PM. As the specification for P_{D1} drops to .95, Policy 5 becomes the optimum under the non-linear cost structure, while Policy 7 is optimum for the linear cost structure. For values of P_{D1} below .90, Policy 7 is always selected as the optimum in the non-linear case and Policy 9 is always selected in the linear case.

Table 4-10 contains the optimum policy for various levels of detection under the second definition of a violation. From Table 4-10 it can be seen that the optimum policy is identical under both cost structures for values of P_{D2} above .25. Policy 1 is the only plan which yields a detection probability exceeding .99 and it is therefore selected as the optimum for $P_{D2} = 1.00$ and $P_{D2} = .99$. For all values of P_{D2} below .80, Policy 7 is again the optimum under the non-linear cost structure. This implies that at these levels of detection, the added cost associated with detection delays becomes more significant than the

TABLE 4-10

STREAM STANDARD = 85 mg/ℓ

REQUIRED DETECTION PROBABILITY (P_{D2})	($C=10 \cdot n+\Sigma(20(e^{t_i}-1)/t_i)$	($C=10 \cdot n+50\Sigma t_i$)
1.00	1	1
.99	1	1
.95	2	2
.90	3	3
.85	4	4
.80	5	5
.70	7	7
.60	7	7
.50	7	7
.25	7	8
.10	7	9

savings realized from less frequent sampling. For P_{D2}=.25 and P_{D2}=.10, Policy 8 and Policy 9 are the respective optimum plans under the linear cost structure.

An alternative way of evaluating the selected policies is to consider only those which lie on the minimum-cost boundary. The curved line passing through Policies 3, 5, 7, 8 and 9 in the upper half of Figure 4-12 represents such a boundary. Because Policies 1, 2, 4 and 6 fall to the right of this curve, they may be immediately eliminated from further consideration. Given that all policies falling on the minimum-cost boundary below P_{D1} = .95 must also be rejected, the set of alternatives is again reduced to Policies 3, 5 and 7. Of the remaining selected policies, Policy 7 falls on the lower left-hand edge of the minimum-cost boundary above P_{D1} = .95. Policy 7 is therefore chosen as optimum. The same approach to the policies contained in the bottom half of Figure 4-12 reduces the set of alternatives to only Policies 3, 5 and 7. The minimum-cost boundary stops at Policy 7, as

Policies 8 and 9 both have higher total costs than Policy 7. If the P_{D1} requirement is set at .95, Policy 7 is again selected as optimum.

The same procedure for determining a confidence interval on the $\hat{p}$'s can be applied to the P_D's. If the normal approximation to the binominial distribution is used, the variance of P_D can be approximated by $P_D(1-P_D)/n$, where n is the number of violations included in the analysis. As an example, suppose a 90% confidence interval is sought for the P_{D2} associated with Policy 4. Table 4-7 shows Policy 4 to have a P_{D2} of .88. Because this value is derived from an analysis of 434 violation periods, n is set equal to 434. The 95% confidence interval is then given by:

$$P[\leq .85 \leq P_{D2} \leq .91] \doteq .95$$

The same techniques discussed earlier may also be used to estimate the number of simulated observations required to determine the value of P_D within some specified limit.

Sensitivity of Optimum Policy to Chloride Stream Standard

In order to study the sensitivity of the optimum policy to changes in the specified chloride stream standard, each of the selected sampling policies is also evaluated for thresholds of 81 mg/ℓ, 83 mg/ℓ and 87 mg/ℓ. The data is plotted in Figures 4-14 through 4-19. Table 4-11 contains the optimum policies for the different chloride thresholds and under the two definitions of a violation. The probability of detection under Definition 1 of a violation is again denoted as P_{D1} in Table 4-11, while P_{D2} refers to detection probability under Definition 2. The specific sampling times for each policy and the resulting detection probabilities are given in Table 4-14, 4-15 and 4-16 for stream standards of 81, 83 and 87 mg/ , respectively.

TABLE 4-11

OPTIMUM POLICY VS. REQUIRED DETECTION PROBABILITY

REQ. DETECTION PROB. P_D	$C = 10*n + \Sigma(20(e^{t_i} - 1)/t_i)$								$C = 10*n + 50\Sigma\, t_i$							
	81 MG/L		83 MG/L		85 MG/L		87 MG/L		81 MG/L		83 MG/L		85 MG/L		87 MG/L	
	P_{D1}	P_{D2}	P_{D1}	P_{D2}	P_{D1}	P_{D2}	P_{D1}	P_{D2}	P_{D1}	P_{D2}	P_{D1}	P_{D2}	P_{D1}	P_{D2}	P_{D1}	P_{D2}
1.00	9	1-5	6	1	3	1	1	1	9	1-5	6	1	3	1	1	1
.99	9	1-5	6	2	3	1	1	1	9	1-5	6	2	3	1	1	1
.95	9	6,7	8	4	5	2	1	1	9	6,7	8	4	7	2	1	1
.90	9	8	8	6	7	3	2	1	9	8	8	6	8	3	2	1
.85	9	9	8	6	7	4	2	1	9	9	8	6	9	4	2	1
.80	9	9	8	8	7	5	3	2	9	9	8	8	9	5	3	2
.70	9	9	8	8	7	7	3	2	9	9	8	8	9	7	3	2
.60	9	9	8	8	7	7	3	2	9	9	8	8	9	7	3	2
.50	9	9	8	8	7	7	3	3	9	9	8	8	9	9	3	3
.25	9	9	8	8	7	7	3	3	9	9	8	8	9	9	3	3
.10	9	9	8	8	7	7	3	3	9	9	8	8	9	9	3	3

P_{D1} refers to detection under definition 1 of a violation

P_{D2} refers to detection under definition 2 of a violation

Analysis of Table 4-11 reveals that the optimum policy does not change with the cost structure under the chloride stream standards of 81 mg/ℓ, 83 mg/ℓ and 85 mg/ℓ, respectively. At a chloride threshold of 81 mg/ℓ Policy 9 is optimum for all values of P_{D1} (probability of detection of violation events). For a P_{D2} (probability of detection of each violation time) greater than or equal to .99, Policies 1 through 5 yield identical costs of 6000 and are therefore equivalent. For P_{D2} = .95 at a standard of 81 mg/ℓ, Policies 6 and 7 both yield a minimum cost. At levels of P_{D2} below .90, Policy 9 is again optimum under both cost structures.

As the chloride stream standard shifts to 83 mg/ℓ, Policy 6 becomes optimum for $P_{D1} \geq .99$, while Policy 8 is optimum for all values of P_{D1} less than .99. Policy 8 is also optimum at all levels of P_{D2} less than .85. At a standard of 87 mg/ℓ, the optimum policy shifts among Policies 1, 2 and 3. Policy 1 is optimum for $P_{D1} \geq .95$, Policy 2 for $.85 \leq P_{D1} < .95$, and Policy 3 for $P_{D1} < .85$. Similarly, Policy 1 is the optimum for $P_{D2} \geq .85$, Policy 2 for $.25 \leq P_{D2} < .85$, and Policy 3 for $P_{D2} = .10$. In all cases which specify a detection probability of $P_{D2} = 1$, Policy 1 is optimum.

Sensitivity of Optimum Policy to Response Time

Up to this point it has been assumed that a violation commencing at time t_i and detected at time t_{i+n} produces a damage or cost which is a function of the delay $(t_{i+n} - t_i)$. In other words once a violation is detected, it ceases to produce any further damage. Therefore a given sampling policy, although required to detect a given percentage of all violation times, will only incur damages over the delay to detection of a continuous violation period. This assumption seems reasonable in view of the fact that once a violation is detected, a response or corrective measure is immediately initiated. Although it may still be desir-

able to maintain detection of the continuous violation at all times, no specific additional penalty is assigned for failure to do so.

The specific time required to begin corrective action against a violation then becomes a critical factor in selecting the optimum policy. In order to test the sensitivity of the optimum policy to variation in the response time (T_r), the costs for each policy are calculated over a response range of 0-3 hours (at a chloride stream standard of 85 mg/ℓ). The results are plotted in Figure 4-20 for the non-linear cost function and under violation definition 1. Tables 4-12 and 4-13 on the following page give the optimum policies for the different values of P_{D1} and P_{D2}, respectively, for the various response times under consideration.

Inspection of Table 4-12 shows that the optimum policy does not change with the response time under the linear-cost structure. However, under the non-linear damage function the optimum policy does shift with differences in the given response time. At levels of P_{D1} less than .95 Policy 7 is optimum for $T_r = 0$, Policy 5 for $T_r = 1$, and Policy 3 for $T_r = 2$ and $T_r = 3$. At $P_{D1} = 1.00$ and $P_{D2} = .99$, Policy 3 is again optimum over all response times analyzed. For $P_{D1} = .95$, Policy 5 is optimum for $T_r = 0$-1 and Policy 3 for $T_r = 2$-3.

Table 4-13 again shows that under the linear-cost structure the optimum policy does not change with response time. The same pattern found under the non-linear damage function in Table 4-12 also holds in Table 4-13. Here Policies 7, 5 and 3 are optimum at T_r levels of 0, 1, and 2-3 hours, respectively, for $P_{D2} < .80$. For $P_{D2} \geq .80$, the optimum policy does not change with the response time under the non-linear cost structure.

Summary of Results

One very important conclusion reached from this

investigation is that the best policy under one set of conditions may not be the best under a different set of conditions. It has been shown that the process of selecting an optimum sampling policy for detection of stream standard violations in a given system must include:

a) explicit definition of a violation and the required detection criteria,
b) definition of the cost or damage functions and the overall cost constraints in effect, and
c) knowledge of the time required for responsive action against a detected violation.

As these conditions change, the optimum sampling policy for a given system may also change. Therefore the design of an effective water quality enforcement mechanism must include information concerning the probabilistic nature of system input and output. Neglect of the above considerations may result in poor detection capabilities and inefficient allocation of sampling resources

It is also important to recognize that the type of sampling policies considered are fixed-time policies, i.e., the optimum sampling plan calls for samples to be taken at specific times each day. The sampling times do not change from day to day. However, in actual practice the policy must provide additional flexibility. In situations where information not previously incorporated in the policy design suddenly becomes available, the sampling schedule must accommodate this additional input. For example, if a known accidential or intentional spill of material to the receiving water shows that a stream standard violation shall occur at a time for which no sample is scheduled, the policy should provide the capability for modifying the original observation times and initiating required responsive action. Similarly, if all discharge sources were known to be shut-off on a given day, e.g., a holiday, the

policy may be modified to call for no samples to be taken on that day. Therefore the implication of a fixed-time sampling policy does not rule out contingency planning for unexpected cases.

The previous analysis represents a very idealized case in which variation of stream chloride levels is only effected by point-source loads. Derivation of a fixed-time sampling policy is best suited to receiving waters which exhibit little day to day streamflow variation relative to monthly or seasonal variation. To be more specific, the effect on pollution levels due to daily streamflow variations must be minor in comparison to that caused by point-source loading variability. In such cases the policies may be derived on the basis of an average monthly or seasonal flow value over the past 10 years. Therefore a new policy may be derived and applied over each sampling period as required.

This technique also assumes that the characteristics of the waste-sources, the time rate-of-change of material in the receiving water and other system parameters, e.g., streamflow, are known exactly. Obviously this situation would not be expected in actual practice. In attempting to apply the above methodology to a real system, consideration may have to be given to such factors as streamflow and background chloride variation, as well as the resulting error in prediction for the stream model. The following chapter illustrates how this technique may be applied to the more complex system. However, before proceeding with the next application, some mention must be made of the possibilities for including other sources of statistical information into the sampling policy design. Although ignored in the present analysis, a high level of auto-correlation[3] exists between the hourly chloride concentra-

[3]Autocorrelation refers to the degree of correlation between neighboring observations. It may be used to provide insights as to the persistence of a given phenomena (Rich, 1973).

tions in the receiving water. This information may be extremely useful in deriving a sequential sampling procedure for detecting stream standard violations. As previously discussed in the literature review, the Box-Jenkins method incorporates information on the sequential dependence of data in the forecasting of a time-series. Although not considered here, the application of such techniques in the area of water quality monitoring may be extremely useful and should be further investigated.

TABLE 4-12

Optimum Policy for Response Times (T_r) of 0-3 Hours
(Stream Standard = 85 mg/ℓ)

	$C=10\cdot n+\Sigma(20(e^{t}i-1)/t_i)$				$C=10\cdot n+50\ \Sigma\ t_i$			
P_{D1}	$T_r=0$	$T_r=1$	$T_r=2$	$T_r=3$	$T_r=0$	$T_r=1$	$T_r=2$	$T_r=3$
1.00	3	3	3	3	3	3	3	3
.99	3	3	3	3	3	3	3	3
.95	5	5	3	3	7	7	7	7
.90	7	5	3	3	8	8	8	8
.85	7	5	3	3	9	9	9	9
.80	7	5	3	3	9	9	9	9
.70	7	5	3	3	9	9	9	9
.60	7	5	3	3	9	9	9	9
.50	7	5	3	3	9	9	9	9
.25	7	5	3	3	9	9	9	9
.10	7	5	3	3	9	9	9	9

TABLE 4-13

Optimum Policy for Response Times (T_r) of 0-3 Hours
(Stream Standard = 85 mg/ℓ)

	$C=10\cdot n+\Sigma(20(e^{t}i-1)/t_i$				$C=10\cdot n+50\ \Sigma\ t_i$			
P_{D2}	$T_r=0$	$T_r=1$	$T_r=2$	$T_r=3$	$T_r=0$	$T_r=1$	$T_r=2$	$T_r=3$
1.00	1	1	1	1	1	1	1	1
.99	1	1	1	1	1	1	1	1
.95	2	2	2	2	2	2	2	2
.90	3	3	3	3	3	3	3	3
.85	4	4	3	3	4	4	4	4
.80	5	5	3	3	5	5	5	5
.70	7	5	3	3	7	7	7	7
.60	7	5	3	3	7	7	7	7
.50	7	5	3	3	7	7	7	7
.25	7	5	3	3	8	8	8	8
.10	7	5	3	3	9	9	9	9

TABLE 4-14

SAMPLING TIMES AND DETECTION PROBABILITIES FOR POLICIES 1-9
CHLORIDE STREAM STANDARD = 81 MG/L

	SAMPLE IF P(VIOLATION) > P								
TIME	P=.01 PLAN1	P=.05 PLAN2	P=.10 PLAN3	P=.15 PLAN4	P=.20 PLAN5	P=.25 PLAN6	P=.30 PLAN7	P=.40 PLAN8	P=.50 PLAN9
M	S	S	S	S	S	S	S	S	S
1AM	S	S	S	S	S	S	S	S	S
2AM	S	S	S	S	S	S	S	S	S
3AM	S	S	S	S	S	S	S	S	S
4AM	S	S	S	S	S	S	S	S	S
5AM	S	S	S	S	S	S	S	S	S
6AM	S	S	S	S	S	S	S	S	-
7AM	S	S	S	S	S	S	S	-	-
8AM	S	S	S	S	S	-	-	-	-
9AM	S	S	S	S	S	-	-	-	-
10AM	S	S	S	S	S	S	S	-	-
11AM	S	S	S	S	S	S	S	S	-
N	S	S	S	S	S	S	S	S	S
1PM	S	S	S	S	S	S	S	S	S
2PM	S	S	S	S	S	S	S	S	S
3PM	S	S	S	S	S	S	S	S	S
4PM	S	S	S	S	S	S	S	S	S
5PM	S	S	S	S	S	S	S	S	S
6PM	S	S	S	S	S	S	S	S	S
7PM	S	S	S	S	S	S	S	S	S
8PM	S	S	S	S	S	S	S	S	S
9PM	S	S	S	S	S	S	S	S	S
10PM	S	S	S	S	S	S	S	S	S
11PM	S	S	S	S	S	S	S	S	S
P(DET. OF EACH TIME)	1.00	1.00	1.00	1.00	1.00	0.98	0.98	0.94	0.89
P(DET. OF VIOLATION)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
SAMPLES REQUIRED (25 DAYS)	600	600	600	600	600	550	550	500	450

TABLE 4-15

SAMPLING TIMES AND DETECTION PROBABILITIES FOR POLICIES 1-9
CHLORIDE STREAM STANDARD = 83 MG/L

	SAMPLE IF P(VIOLATION) > P								
TIME	P=.01 PLAN1	P=.05 PLAN2	P=.10 PLAN3	P=.15 PLAN4	P=.20 PLAN5	P=.25 PLANo	P=.30 PLAN7	P=.40 PLAN8	P=.50 PLAN9
M	S	S	S	S	S	S	S	S	S
1AM	S	S	S	S	S	S	S	S	S
2AM	S	S	S	S	S	S	S	S	S
3AM	S	S	S	S	S	S	S	-	-
4AM	S	S	S	S	S	-	-	-	-
5AM	S	S	S	S	-	-	-	-	-
6AM	S	S	S	-	-	-	-	-	-
7AM	S	S	-	-	-	-	-	-	-
8AM	S	-	-	-	-	-	-	-	-
9AM	S	-	-	-	-	-	-	-	-
10AM	S	S	-	-	-	-	-	-	-
11AM	S	S	-	-	-	-	-	-	-
N	S	S	S	S	-	-	-	-	-
1PM	S	S	S	S	S	S	-	-	-
2PM	S	S	S	S	S	S	S	S	-
3PM	S	S	S	S	S	S	S	S	S
4PM	S	S	S	S	S	S	S	S	S
5PM	S	S	S	S	S	S	S	S	S
6PM	S	S	S	S	S	S	S	S	S
7PM	S	S	S	S	S	S	S	S	S
8PM	S	S	S	S	S	S	S	S	S
9PM	S	S	S	S	S	S	S	S	S
10PM	S	S	S	S	S	S	S	S	S
11PM	S	S	S	S	S	S	S	S	S
P(DET. OF EACH TIME)	1.00	1.00	0.98	0.96	0.93	0.90	0.87	0.84	0.80
P(DET. OF VIOLATION)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.99	0.99
SAMPLES REQUIRED (25 DAYS)	600	550	475	475	400	375	350	325	300

TABLE 4-16

SAMPLING TIMES AND DETECTION PROBABILITIES FOR POLICIES 1-9
CHLORIDE STREAM STANDARD = 87 MG/L

	SAMPLE IF P(VIOLATION) > P								
TIME	P=.01 PLAN1	P=.05 PLAN2	P=.10 PLAN3	P=.15 PLAN4	P=.20 PLAN5	P=.25 PLAN6	P=.30 PLAN7	P=.40 PLAN8	P=.50 PLAN9
M	S	-	-	-	-	-	-	-	-
1AM	S	-	-	-	-	-	-	-	-
2AM	-	-	-	-	-	-	-	-	-
3AM	-	-	-	-	-	-	-	-	-
4AM	-	-	-	-	-	-	-	-	-
5AM	-	-	-	-	-	-	-	-	-
6AM	-	-	-	-	-	-	-	-	-
7AM	-	-	-	-	-	-	-	-	-
8AM	-	-	-	-	-	-	-	-	-
9AM	-	-	-	-	-	-	-	-	-
10AM	-	-	-	-	-	-	-	-	-
11AM	-	-	-	-	-	-	-	-	-
N	-	-	-	-	-	-	-	-	-
1PM	S	-	-	-	-	-	-	-	-
2PM	S	-	-	-	-	-	-	-	-
3PM	S	-	-	-	-	-	-	-	-
4PM	S	S	-	-	-	-	-	-	-
5PM	S	S	-	-	-	-	-	-	-
6PM	S	S	S	-	-	-	-	-	-
7PM	S	S	-	-	-	-	-	-	-
8PM	S	S	-	-	-	-	-	-	-
9PM	S	S	-	-	-	-	-	-	-
10PM	S	S	-	-	-	-	-	-	-
11PM	S	-	-	-	-	-	-	-	-
P(DET. OF EACH TIME)	1.00	0.83	0.17	0.00	0.00	0.00	0.00	0.00	0.00
P(DET. OF VIOLATION)	1.00	0.92	0.83	0.00	0.00	0.00	0.00	0.00	0.00
SAMPLES REQUIRED (25 DAYS)	325	175	25	0	0	0	0	0	0

PROBABILITY OF VIOLATION DETECTION VS. TOTAL COST FOR SAMPLING POLICIES 1 - 9
CHLORIDE THRESHOLD = 81 MG/L

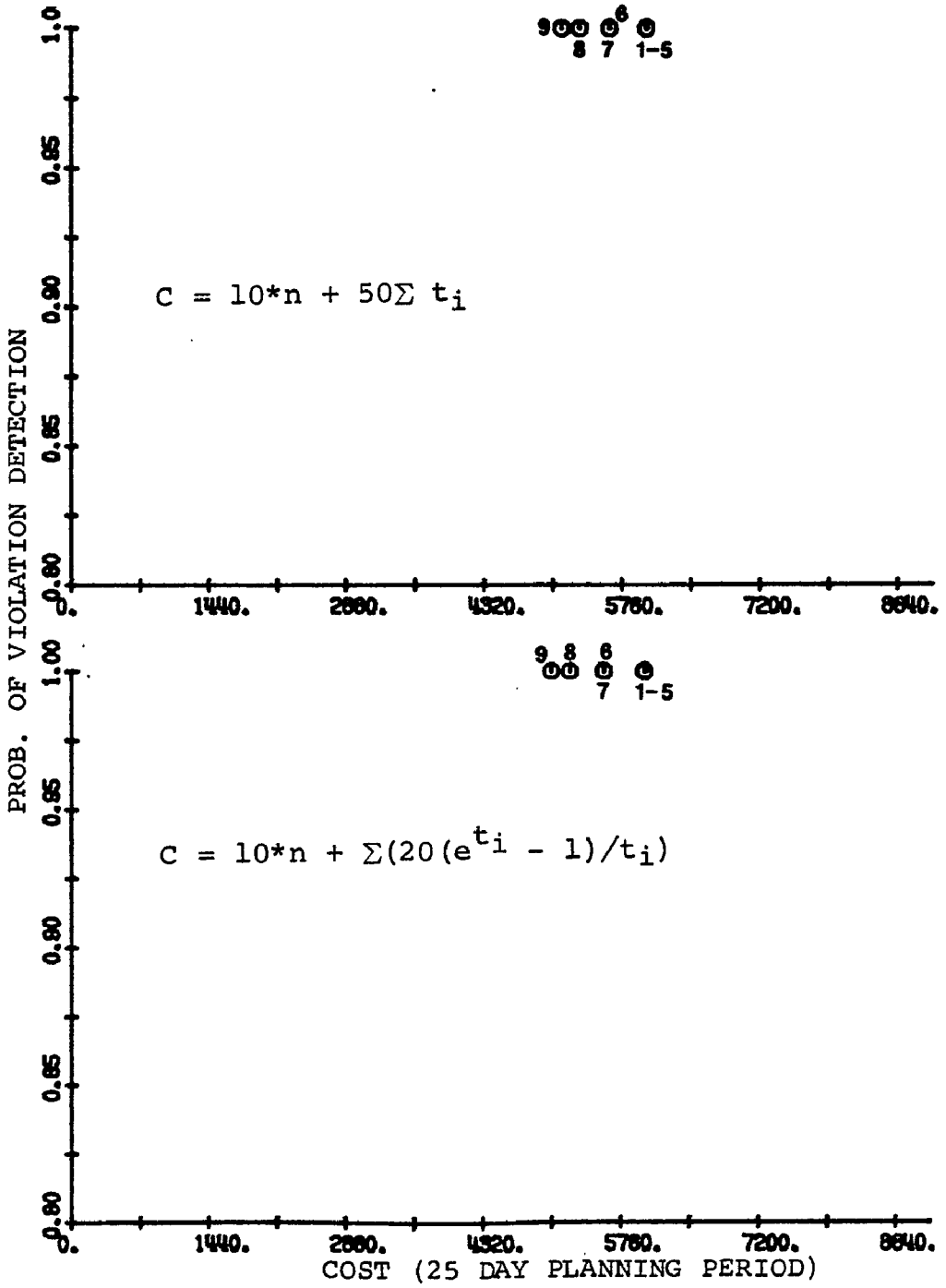


FIGURE 4-14

PROBABILITY OF ALL VIOLATION TIMES DETECTED
VS. TOTAL COST FOR SAMPLING POLICIES 1 - 9
CHLORIDE THRESHOLD = 81 MG/L

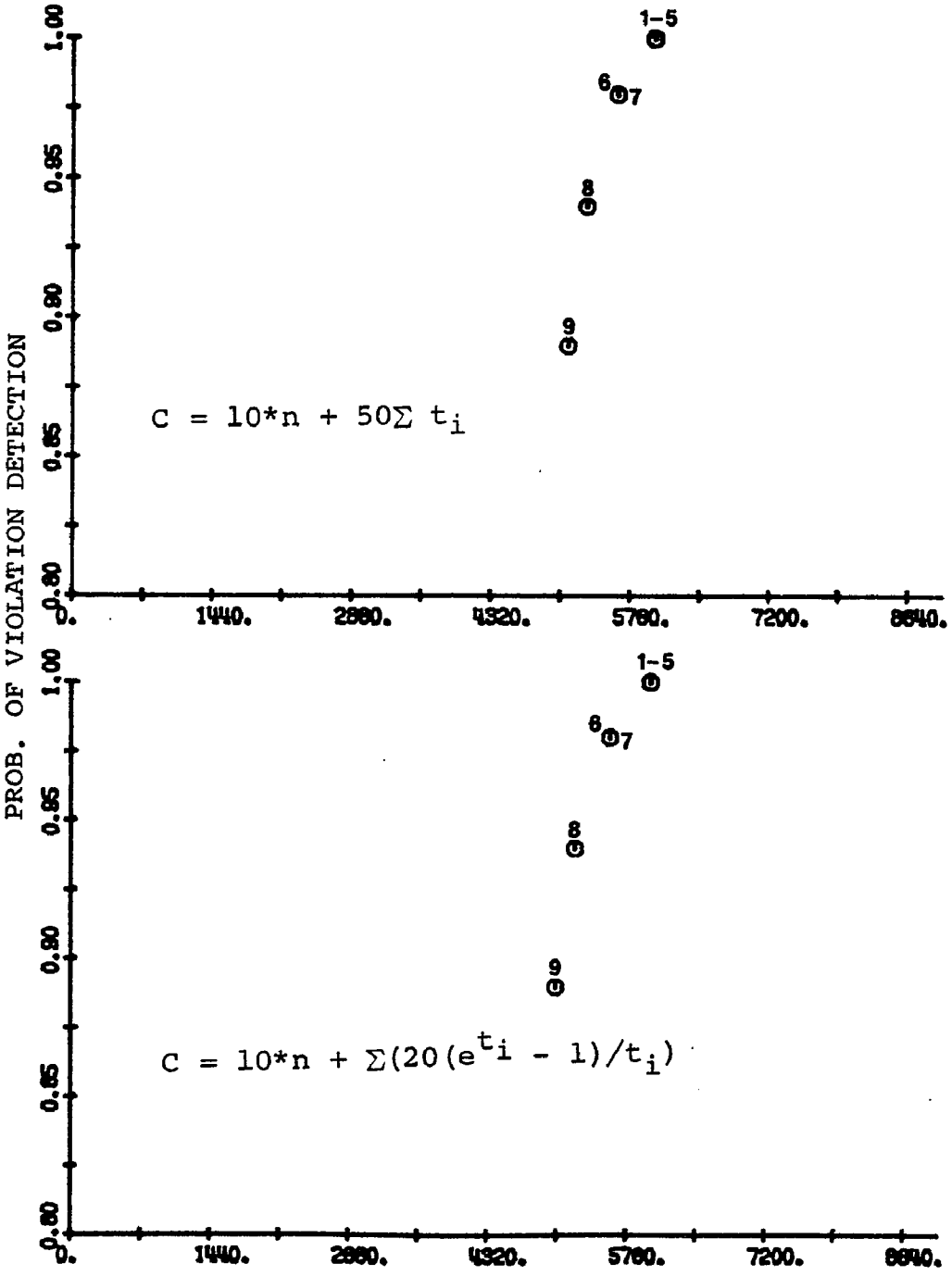


FIGURE 4-15

PROBABILITY OF VIOLATION DETECTION VS. TOTAL COST FOR SAMPLING POLICIES 1 - 9

CHLORIDE THRESHOLD = 83 MG/L

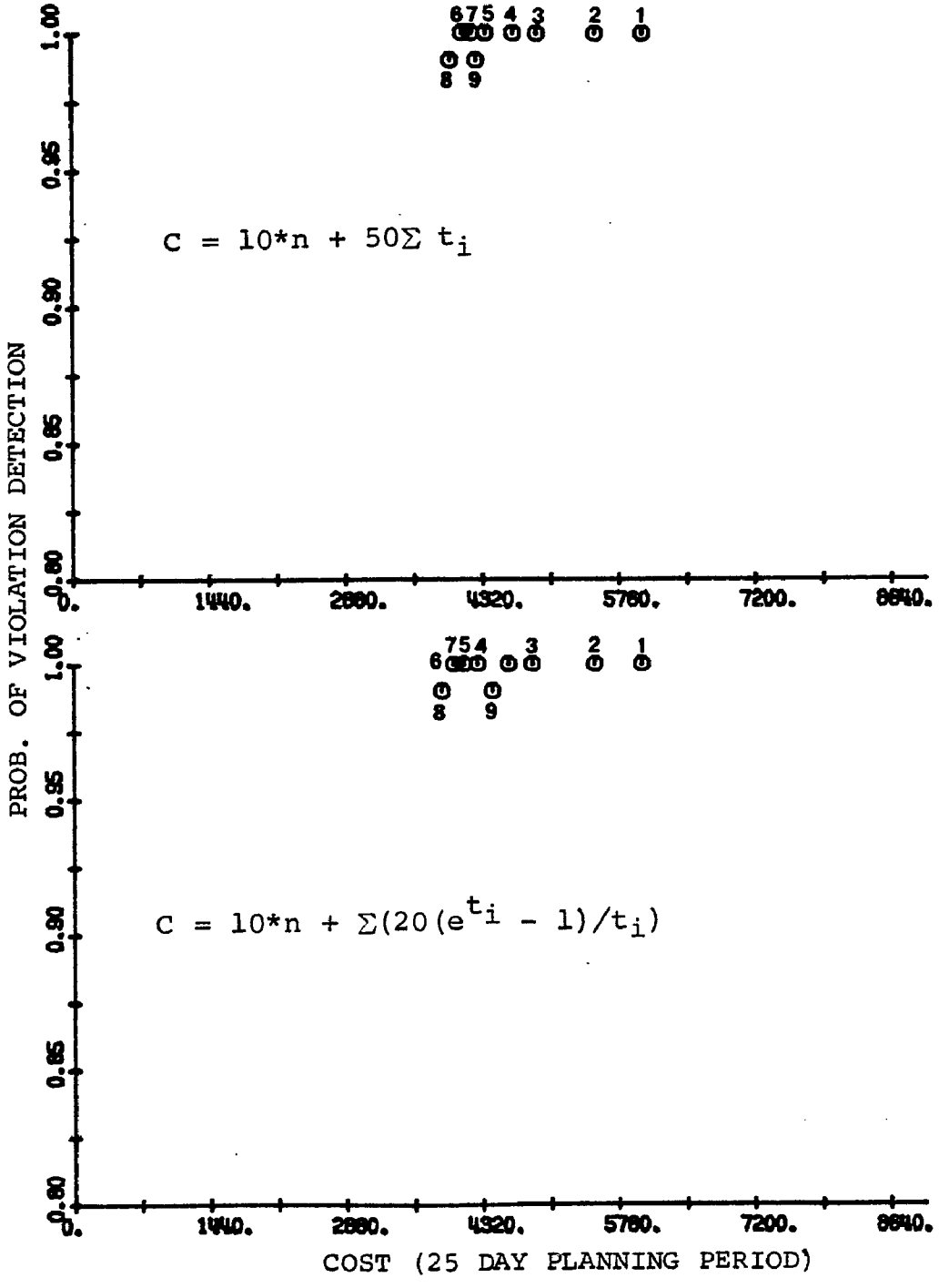


FIGURE 4-16

PROBABILITY OF ALL VIOLATION TIMES DETECTED
VS. TOTAL COST FOR SAMPLING POLICIES 1 - 9
CHLORIDE THRESHOLD = 83 MG/L

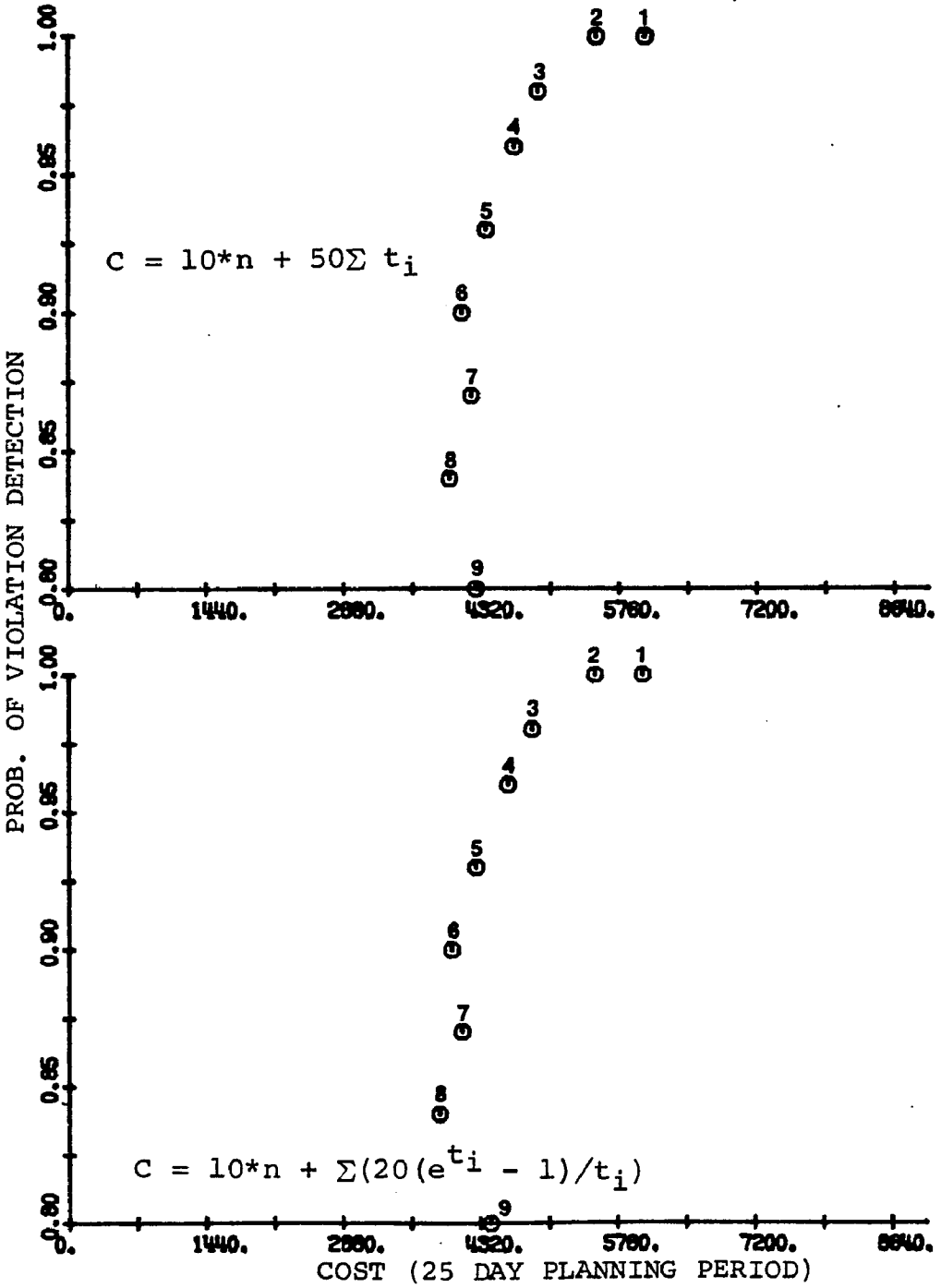


FIGURE 4-17

PROBABILITY OF VIOLATION DETECTION VS. TOTAL COST FOR SAMPLING POLICIES 1 - 9
CHLORIDE THRESHOLD = 87 MG/L

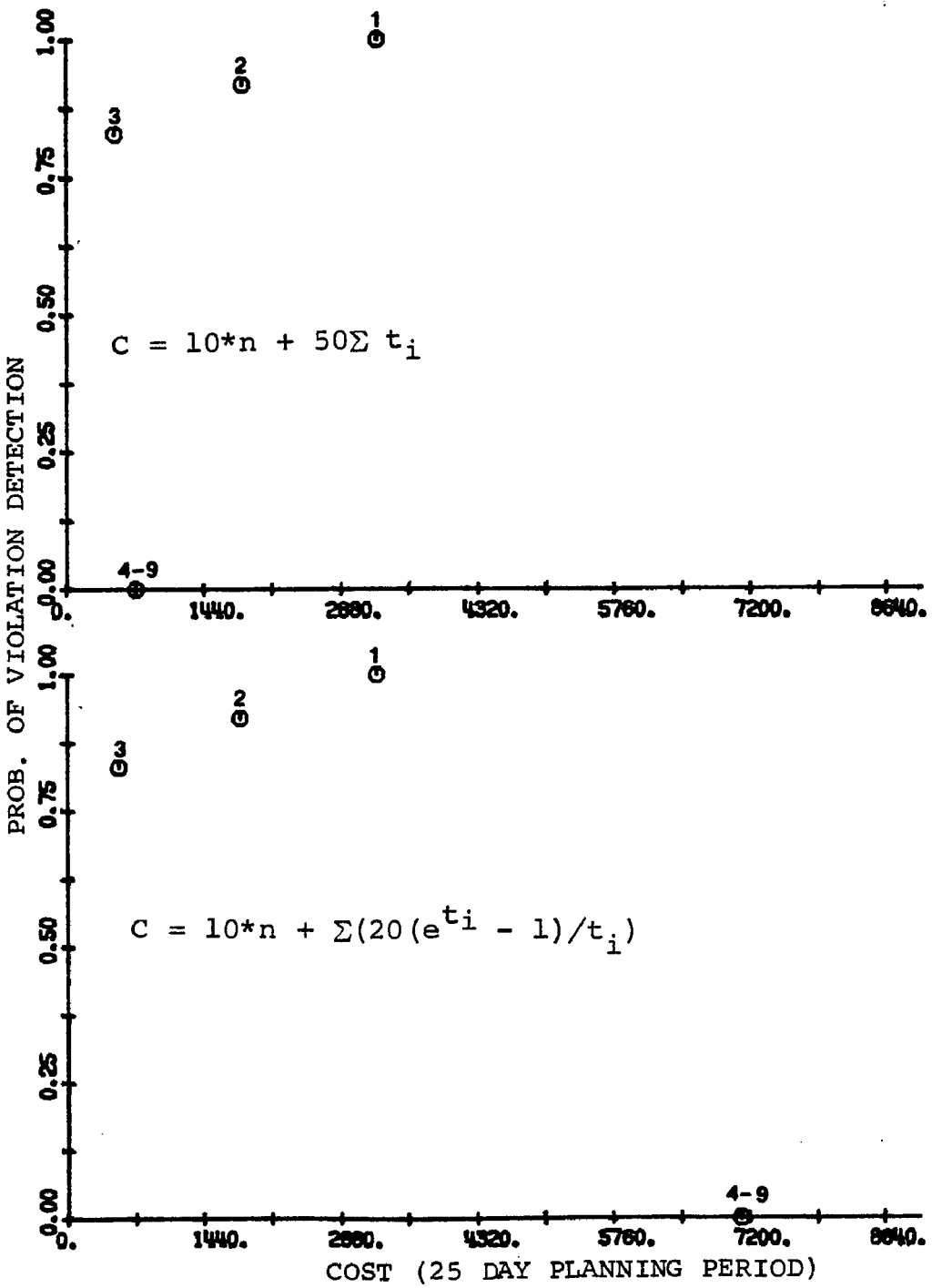


FIGURE 4- 18

PROBABILITY OF ALL VIOLATION TIMES DETECTED
VS. TOTAL COST FOR SAMPLING POLICIES 1 - 9
CHLORIDE THRESHOLD = 87 MG/L

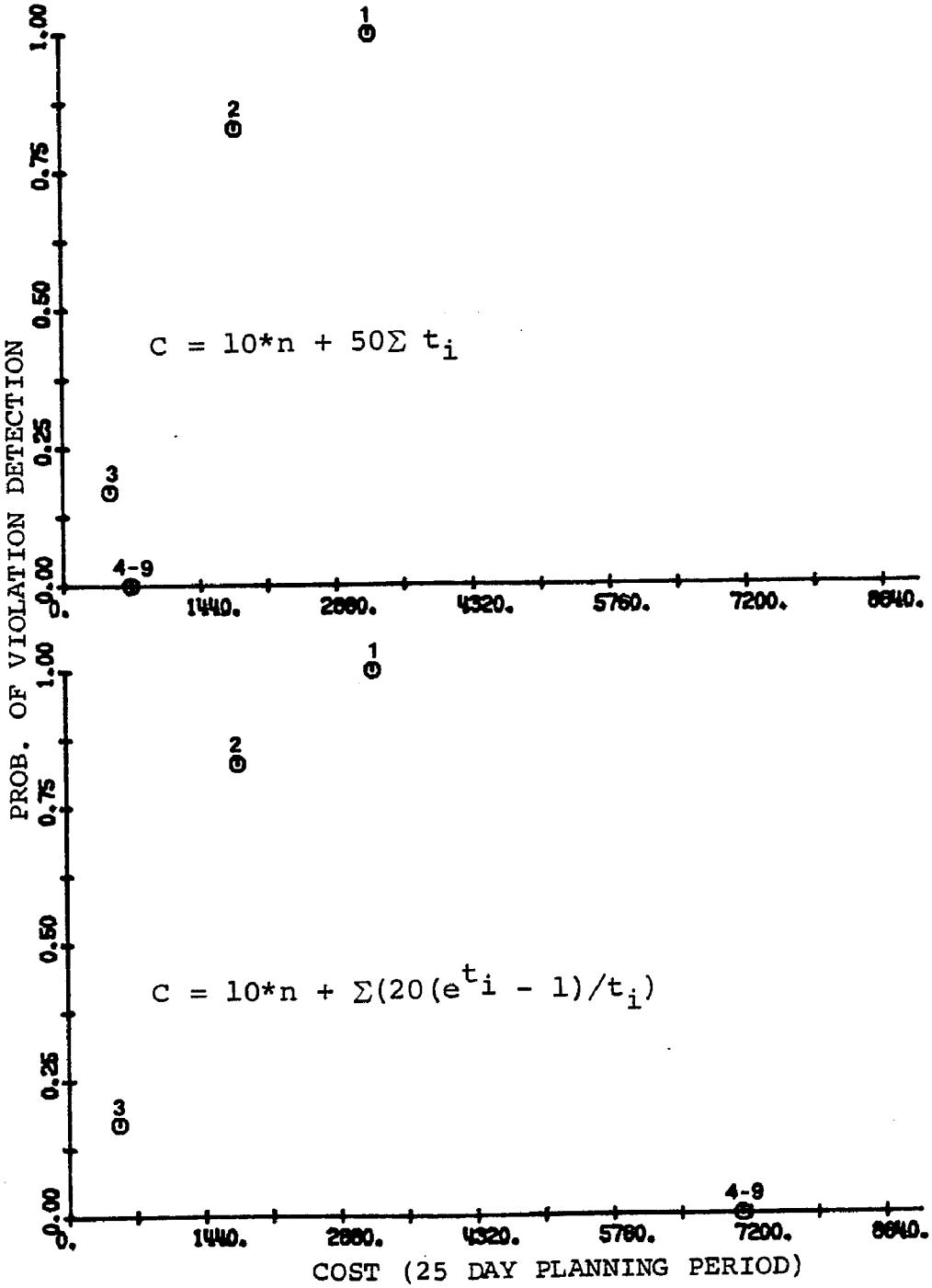


FIGURE 4-19

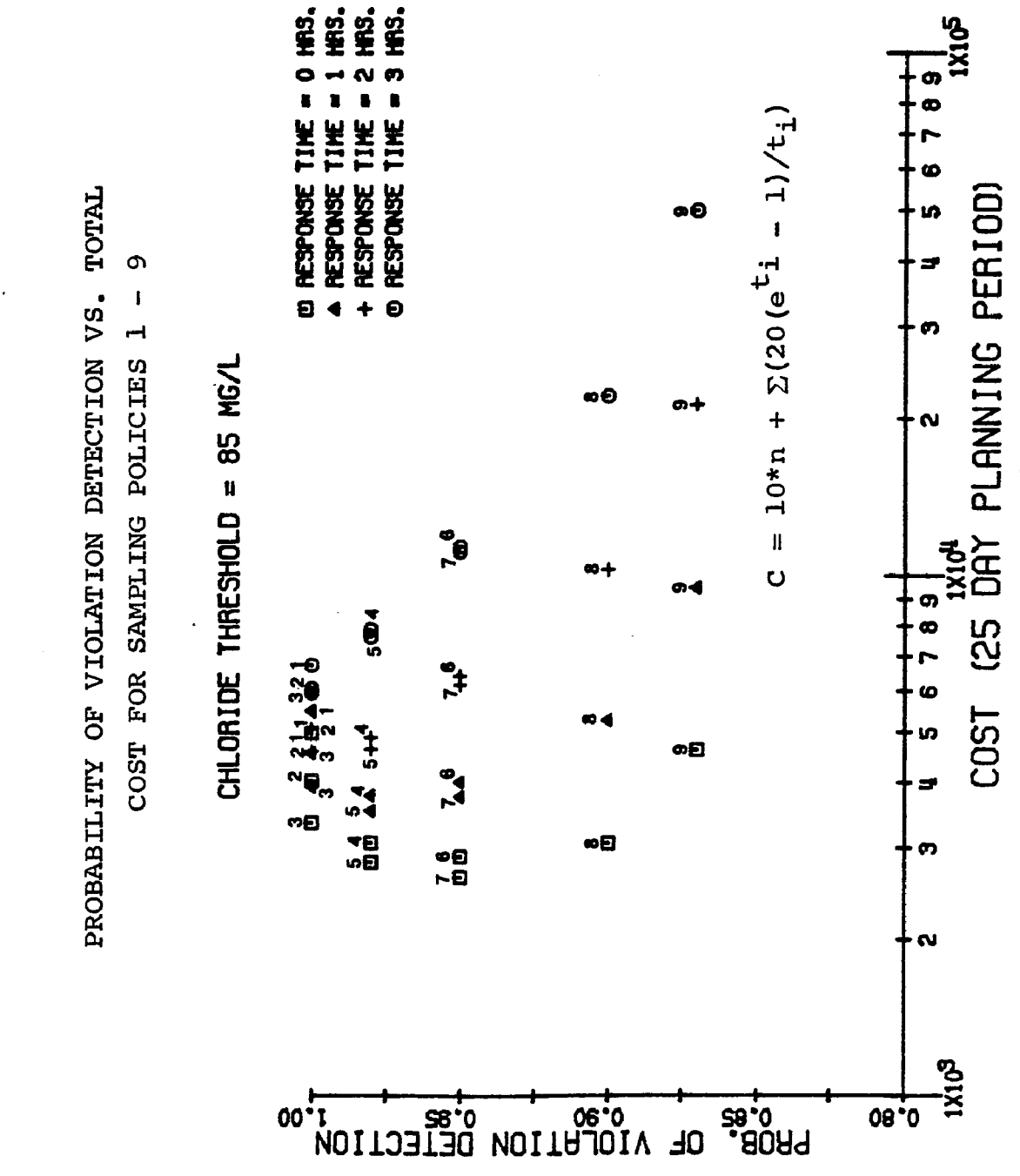
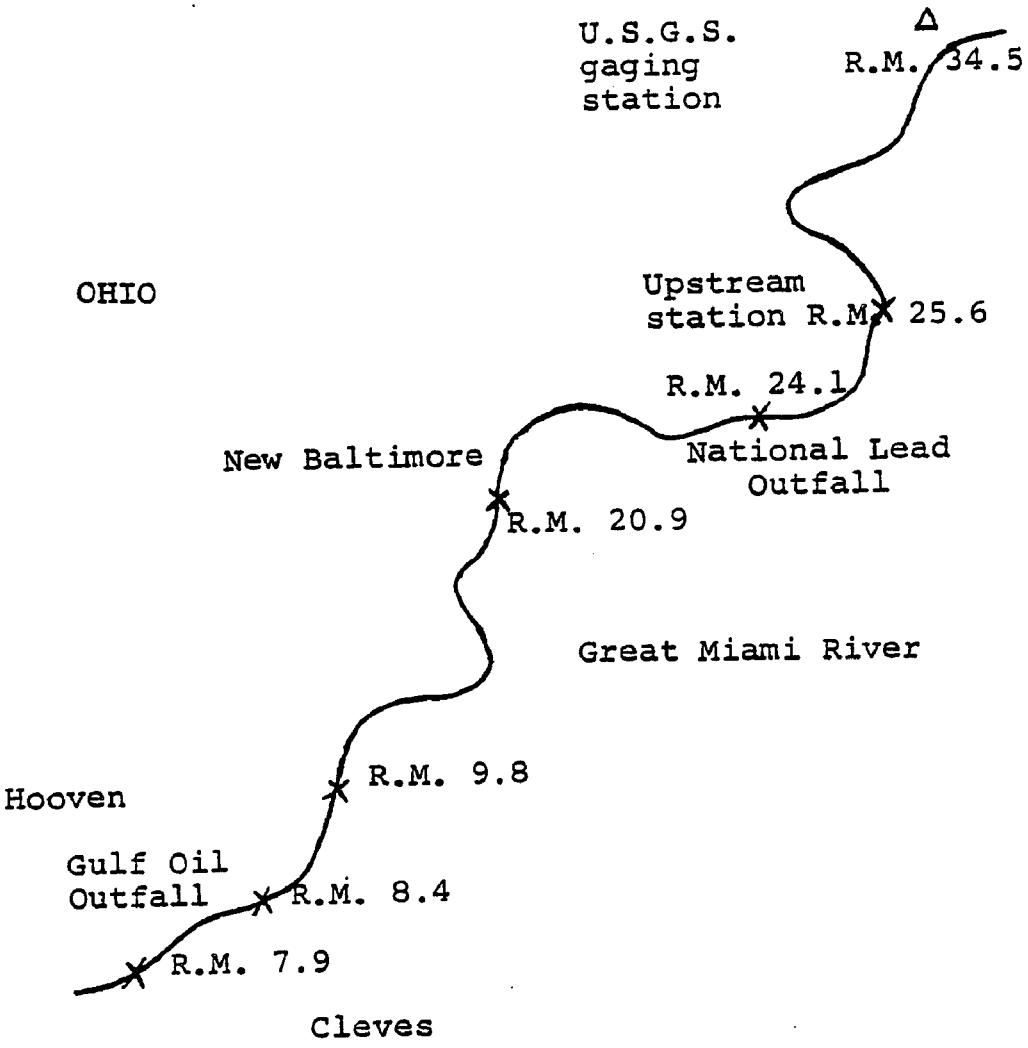

PROBABILITY OF VIOLATION DETECTION VS. TOTAL
COST FOR SAMPLING POLICIES 1 - 9
CHLORIDE THRESHOLD = 85 MG/L
RESPONSE TIME = 0 HRS.
RESPONSE TIME = 1 HRS.
RESPONSE TIME = 2 HRS.
RESPONSE TIME = 3 HRS.
PROB. OF VIOLATION DETECTION
1.00
0.85
0.90
0.85
0.80
C = 10*n + Σ(20(e^ti - 1)/ti)
1X10^3
2
3
4
5
6
7
8
9
1X10^4
2
3
4
5
6
7
8
9
1X10^5
COST (25 DAY PLANNING PERIOD)

FIGURE 4-20

CHAPTER V

THE GREAT MIAMI RIVER SYSTEM

The receiving water system under investigation in this section consists of a 25 mile stretch of the Great Miami River from river mile 28.0 down to river mile 3.0. Here the river receives waste loads from two point sources- National Lead Company at river mile 24.1 and a Gulf Oil refinery at river mile 8.4. Both companies discharge chlorides to the Great Miami River. A map of the area is shown below.



The first step in the analysis of the above system is to obtain available field data concerning the following:

1) Chloride loadings from Gulf Oil and National Lead
2) Background chloride levels
3) Streamflow
4) Chloride concentrations in the river at the point of interest

The State of Ohio Environmental Protection Agency was able to provide the chloride information for the 1973 water year - October 1, 1973 to September 30, 1974. The Streamflow data for the Great Miami River was obtained from the U.S.G.S. in Columbus, Ohio. The gaging station is located in Hamilton, Ohio at river mile 34.5. The flow information consisted of daily mean values, while the chloride information was in the form of both grab and continuous samples as given below:

	Location	Representation	Type of Sample
Station 1	R.M. 25.6	Background	Grab
Station 2	R.M. 24.1	National Lead Outfall	Continuous
Station 3	R.M. 20.9	Downstream of Nat. Lead	Continuous
Station 4	R.M. 9.8	Upstream of Gulf Oil	Grab
Station 5	R.M. 8.4	Gulf Oil Outfall	Grab
Station 6	R.M. 7.9	Point of Detection	Grab

The chloride data from stations 1, 2, and 5 covered the entire 1973 water year. However, the data at stations 3, 4, and 6 was only available for the 9-month period from October 1, 1973 through July 31, 1974. Additionally,

observations at all stations except one were taken on a weekly rather than daily basis. Only station 2 at the National Lead Outfall was sampled each day. The field observations at station 1 are given in Table B-1 of Appendix B. Tables 5-1 and 5-2 contain the original chloride loads from National Lead and Gulf Oil, respectively. Table 5-8 contains the field measurements taken at station 6--the most downstream location.

It is assumed in the following study that the objective is detection of chloride stream standard violations at station 6. Because at the outset of this investigation no specific chloride standard was in effect on the Great Miami River, a standard was selected which would reflect several violations over the course of the year. Later the threshold limit was changed to reveal sensitivities of the optimal sampling policy to variation in the specified standard.

Due to the rather dispersed data base available for the design and verification of the river model, the following assumptions were made at the outset:

1) Time is divided into discrete 1-day time units.
2) A point-scource load on any day is representative of that load over the entire 24-hour period.
3) A chloride stream measurement on any day is representative of the concentration over the entire day.
4) The U.S.G.S. flow measurements are representative of the stream discharge for the entire day, i.e., the stream flow remains constant within any 24-hour period and changes instantaneously from day to day.

Although the above assumptions may lend themselves to some criticism, they provide the necessary clarity and justification for treatment of the available data in the sampling policy design.

Sampling Policy Description

The initial step in the development is definition of

TABLE 5-1

ORIGINAL CHLORIDE LOADINGS (LBS./DAY) FROM NATIONAL LEAD
(OCTOBER 1973 - SEPTEMBER 1974)

DAY	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP
1	409.	474.	1622.	1769.	1481.	1296.	511.	574.	2117.	586.	380.	329.
2	389.	776.	1279.	659.	1475.	1303.	1048.	435.	952.	343.	820.	326.
3	989.	871.	1097.	758.	933.	751.	964.	907.	1381.	481.	647.	616.
4	777.	866.	1552.	2012.	881.	896.	836.	984.	374.	542.	397.	871.
5	830.	711.	1980.	1044.	1147.	916.	2459.	944.	433.	515.	475.	362.
6	1607.	321.	1346.	1128.	1086.	977.	2011.	702.	724.	727.	363.	402.
7	1697.	451.	1085.	1513.	1346.	803.	1583.	641.	446.	1271.	403.	392.
8	1443.	730.	1501.	1292.	1698.	1095.	1408.	652.	492.	625.	547.	909.
9	756.	415.	964.	931.	1047.	541.	1606.	812.	521.	393.	505.	514.
10	1159.	879.	688.	1934.	1036.	1118.	1397.	761.	301.	422.	591.	463.
11	1154.	616.	1003.	3628.	1414.	798.	1449.	638.	593.	532.	721.	1044.
12	652.	1026.	632.	2249.	998.	878.	776.	532.	375.	386.	419.	914.
13	642.	378.	639.	1069.	996.	1445.	642.	762.	216.	535.	450.	816.
14	814.	701.	740.	1204.	1061.	715.	713.	465.	791.	251.	596.	919.
15	762.	1038.	927.	1187.	1331.	949.	712.	608.	187.	213.	684.	1064.
16	779.	431.	1040.	1376.	753.	1786.	511.	546.	126.	225.	456.	766.
17	629.	643.	456.	1371.	744.	571.	637.	439.	123.	503.	743.	734.
18	776.	810.	608.	1391.	887.	485.	643.	1207.	113.	301.	397.	447.
19	613.	699.	509.	1746.	1202.	1002.	463.	744.	261.	591.	373.	652.
20	479.	363.	1666.	1452.	669.	446.	352.	894.	207.	971.	432.	431.
21	476.	891.	750.	1998.	557.	523.	655.	792.	520.	735.	397.	799.
22	557.	522.	1293.	1409.	828.	752.	468.	1212.	478.	420.	660.	474.
23	277.	795.	430.	1815.	1129.	609.	501.	1390.	508.	392.	721.	1204.
24	267.	1024.	1089.	1309.	1341.	626.	974.	765.	927.	395.	814.	473.
25	749.	210.	1635.	1202.	1207.	543.	663.	363.	1880.	333.	411.	792.
26	571.	416.	1005.	1164.	2377.	733.	370.	392.	1448.	532.	425.	1471.
27	1190.	1197.	1334.	2109.	2052.	441.	863.	915.	1247.	313.	396.	668.
28	521.	1846.	1016.	1472.	1636.	776.	521.	638.	1043.	225.	289.	489.
29	486.	1209.	1016.	1625.		575.	294.	336.	710.	332.	884.	429.
30	977.	2280.	994.	1209.		867.	593.	278.	537.	585.	1042.	851.
31	761.		2080.	1448.		540.		1103.		390.	438.	

TABLE 5-2

ORIGINAL CHLORIDE LOADINGS (LBS./DAY) FROM GULF OIL

(OCTOBER 1973 - SEPTEMBER 1974)

DATE	LOADING	DATE	LOADING
OCT. 4	1916.	APR. 4	3216.
OCT. 9	3119.	APR. 10	4201.
OCT. 16	3720.	APR. 17	5506.
OCT. 23	4021.	APR. 23	3507.
NOV. 1	4356.	MAY 7	6944.
NOV. 8	4208.	MAY 15	5010.
NOV. 14	3778.	MAY 23	3507.
NOV. 20	4173.	MAY 28	2344.
DEC. 3	3562.	JUN. 3	3507.
DEC. 12	2898.	JUN. 10	4810.
DEC. 19	3332.	JUN. 19	4033.
DEC. 26	5261.	JUN. 25	2104.
JAN. 4	3607.	JUL. 1	3788.
JAN. 9	4314.	JUL. 10	3066.
JAN. 16	4033.	JUL. 17	2886.
JAN. 24	3367.	JUL. 23	4173.
FEB. 5	4384.	AUG. 1	4068.
FEB. 12	5411.	AUG. 7	4559.
FEB. 20	5035.	AUG. 13	5225.
FEB. 26	5571.	AUG. 20	3928.
MAR. 5	4483.	SEP. 5	4509.
MAR. 14	3232.	SEP. 12	5373.
MAR. 20	3262.	SEP. 19	2856.
MAR. 26	3682.	SEP. 24	3262.

the selected probability threshold sampling policies. Table 5-3 contains the 16 policies under consideration. For example, Policy 6 states that a chloride observation shall be made if the probability of a stream standard violation exceeds .25. Otherwise no sample is taken.

Table 5-3
Selected Sampling Policies

Policy No.	P
1	.01
2	.05
3	.10
4	.15
5	.20
6	.25
7	.30
8	.40
9	.50
10	.60
11	.70
12	.80
13	.85
14	.90
15	.95
16	.99

The difficulty in obtaining the probability threshold sampling policies in this case as opposed to the hypothetical case previously analyzed is immediately apparent. In the hypothetical system, it was assumed that the random nature of the chloride concentration in the stream was due entirely to the probabilistic nature of the point-source loadings. Additionally, the stream model was assumed to provide an exact representation of the mass transport and time rate-of-change of material in the receiving water. Therefore, the

probability description of the concentration of chlorides at the point of interest and at any given time could be estimated by means of simulation. Repeated chloride observations could be generated, the resulting data analyzed, and the selected sampling policies explicitly defined in terms of time-of-observation. In the present case the random nature of the chloride concentration at the point of interest is not only a function of the probalistic nature of the point-source loadings (Gulf Oil and National Lead), but also the variability of the background chloride concentration and the streamflow. Also, the stream model is no longer a perfect representation of material transport in the stream. Therefore, the error in model prediction must be included in estimating violation probabilities.

The overall implication of these difficulties is that the sampling times are no longer fixed over a given planning period. Rather, specific sampling times must be determined on a continuous basis over the period. This enables the probability of violation to reflect current levels of the variables which effect it. For example, it would be inefficient to make a decision today that a chloride observation must be made each Tuesday of every week if the possibility exists of heavy rainfall producing a very small violation probability on some future date.

The derivation of the probability threshold sampling policies in this present case begins again with a description of the probability distributions for each of the point-sources of chloride. Given a satisfactory representation of these inputs, the background chloride concentrations and daily streamflows are also modeled as input variables to the overall mathematical model of the receiving water. The mathematical model is then solved for the chloride concentrations at the sampling position, and this output data is compared to actual field measurement to obtain the prediction error. Historical data is therefore

used to quantify the error between actual chloride levels and the concentrations given by the river model. Once the prediction error (or its probability distribution) is known, the information is used to determine the specific sampling times for each of the selected policies. The procedure is described in detail in the following sections of this chapter.

Mathematical Model of the Great Miami River System

Before commencing with the derivation of the Great Miami River model, it is important to emphasize that the mathematical model employed in the analysis is only a tool for illustrating the overall approach to policy design. For this reason, the model is a simplified version of more detailed models which can provide a better approximation of actual stream conditions. In practice it is expected that sophisticated mathematical models would be used which account for such factors as flow variation over the length of the receiving water, variable river geometry, variable dispersion levels at different locations, variable infiltration capacities in the watershed over time, etc. The use of such complex models in the present case would have exceeded the time and resource constraints in effect, but their importance to the actual success of the methodology must be clearly understood. The effectiveness of the derived monitoring policy will depend to a large degree on how accurately the pollutant-receiving water characteristics are modeled. At the same time, the simplified model employed in the following study is a useful tool for analyzing the available data and providing a clear example of the suggested approach to sampling policy design. Application of a very complex and detailed mathematical model here would have created problems in providing the required clarity, thereby justifying a less-detailed model.

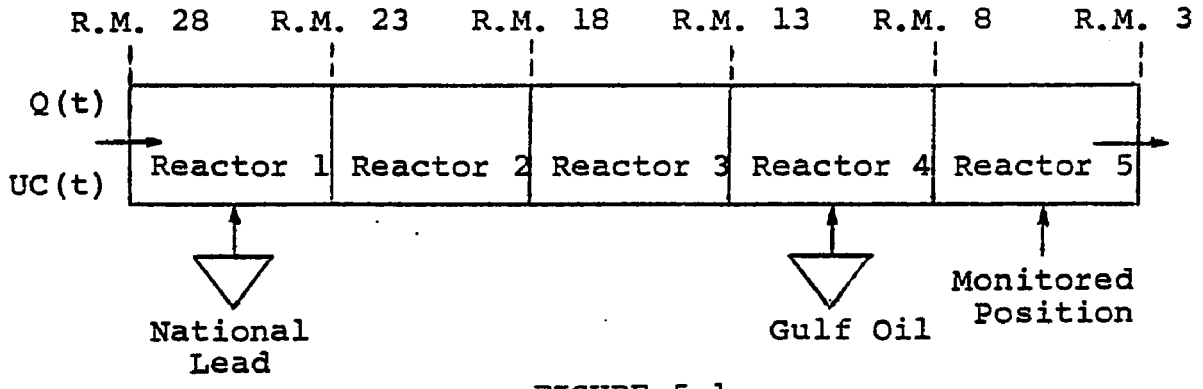


FIGURE 5-1

A series of 5 completely-mixed, flow reactors comprise the river model between river mile 28 and river mile 3, as shown in Figure 5-1. The overall effect of employing only 5 reactor cells in the river model is to dampen the system response to variation in pollutant loadings. On the other hand, the cost of a simulation run increases with the number of cells present, creating a trade-off between predictive accuracy and total cost. Because the extremely large streamflows in the Great Miami River may tend to reduce the chloride variation over short distances, the dampening effect of fewer cells is chosen over the higher costs of a more refined grid. The time rate-of-change of the chloride concentration in the i^{th} reactor is represented by an ordinary differential equation of the form:

$$\frac{dc_i}{dt} = \frac{Q(c_{i-1} - c_i)}{V_i} + \frac{W_i}{V_i} + \frac{EA}{L_i \cdot V_i}(c_{i-1} - c_i) + \frac{EA}{L_{i+1} \cdot V_i}(c_{i+1} - c_i) \quad (5\text{-}1)$$

where: Q = stream flow at time t (volume/time)

V_i = volume of i^{th} reactor at time t

W_i = chloride load from point-sources into i^{th} reactor at time t (mass/time)

E = dispersion coefficient (length^2/time)

c_i = chloride conc. in i^{th} reactor at time t
L_i = characteristic length of i^{th} reactor
A = cross-sectional area between adjacent reactors at time t

As opposed to the previous case in which streamflow was treated as a constant, the present system deals with a time-variable flow. This implies that the cross-sectional area between adjacent segments must also change with time to reflect the variation in reactor volumes. Equation (5-2) below was derived from a time-of-travel study on a 71.3 mile stretch of the Great Miami River between Dayton and Cleves, Ohio (Bauer, 1968).

$$t_R = 400. \times 10^2 \, Q^{-.85} \qquad (5\text{-}2)$$

where t_R = time of travel in hours
Q = discharge in cfs.

The assumption is made that the time-of-travel for the 25-mile stretch presently under investigation is approximately one-third (25/71.3) of that given by Equation (5-2) for any given flow. Therefore, the equation of time-of-travel for the present system may be written:

$$t_R = 1.4025 \times 10^4 \cdot Q^{-.85} \qquad (5\text{-}3)$$

The average detention time in the 5-reactor system is given by Equation (5-4):

$$t_R = \frac{\text{Volume}}{Q} \qquad (5\text{-}4)$$

The system volume (in ft^3) is:

$$\text{Volume } (ft^3) = (25 \text{ miles})\frac{(5280 \text{ ft})}{\text{mile}}(A) = 132000 \cdot A$$

where A is expressed in ft^2.

Substituting ($V = 132000 \cdot A$) back into Equation (5-4) yields:

$$t_R(\text{hrs}) = \frac{132000 \cdot A}{Q \cdot 3600} = 36.67 \cdot A \qquad (5\text{-}5)$$

Assuming that the system detention time is an acceptable estimate of the time-of-travel over the 25-mile stretch,

Equations (5-3) and (5-5) can be used to solve for the cross-sectional area as a function of flow:

$$\frac{36.67 \cdot A}{Q} = 1.4025 \times 10^{4} \cdot Q^{-.85} \qquad (5\text{-}6)$$

$$A = 383 \cdot Q^{.15}$$

Therefore, the values of V_i in Equation (5-1) are computed by the relationship:

$$V_i = \ell_i \cdot A \qquad (5\text{-}7)$$

where ℓ_i = length of the i^{th} reactor.

It is important to emphasize that Equation (5-2) is recommended for application over a limited flow range from 300-800 cfs. Many of the discharge levels in the current analysis exceed this range, and it may be argued that the relationship should not be used. However, attempting to quantify the variability of the cross-sectional area of the Great Miami River in the stretch of interest would have been extremely difficult, requiring time and resources beyond the scope of this research. On the other hand, it is important to incorporate available information concerning the dynamic characteristics of the receiving water into the stream model. The above time-of-travel study represented the best information available on the Great Miami River and was therefore applied in the analysis.

It is interesting to compare the relative effects of chloride transport by means of advection as opposed to dispersion in the present case. The average daily flow recorded at the Hamilton gaging station for the 1973 water year is 3982 cfs. Therefore, the net mass transport of material into segment i due to the average river flow may be written as:

$$3982 \ (c_{i-1} - c_i) \qquad (5\text{-}8)$$

The mass transport of chloride due to dispersion in the river is given by:

$$\frac{EA}{L_i}(c_{i-1} - c_i) + \frac{EA}{L_{i+1}}(c_{i+1} - c_i) \qquad (5\text{-}9)$$

A dispersion coefficient of 1.0 mi^2/day is selected as typical of dispersion levels in streams (Manhattan Summer Conference, 1976). Although a more accurate estimate of E could be obtained from an extensive development of the mathematical model, this value is representative of conditions in many rivers and streams. Substituting E = 1.0 mi^2/day, $A = 383 \cdot (3982)^{-.15} = 1328\ ft^2$ and L_i = 26400 ft into Equation (5-9) yields:

$$16(c_{i-1} - c_i) + 16\ (c_{i+1} - c_i). \qquad (5\text{-}10)$$

It is immediately obvious from comparison of Equations (5-8) and (5-10) that, at the average daily discharge level, the effect of dispersion is almost negligible in comparison with material transport due to net river flow.

The system of 5 ordinary differential equations is again solved by means of the Adams-falkner predictor-corrector method. The computer program MIAMI (Appendix G) contains the algorithm for obtaining the estimates of the chloride concentration at the sampling position. Output is given at 1-day intervals.

Background Chloride Concentration and Initial Conditions

One of the difficulties in modeling a realistic river system is the necessity of quantifying the boundary conditions. In the present case the concentration of chloride entering the system at the upstream boundary is required input to the river model:

$$\frac{dC_1}{dt} = \frac{Q(UC - C_1)}{V_1} + \frac{W_1}{V_1} + \frac{EA}{L_1 V_1}(UC - C_1) + \frac{EA}{L_2 V_1}(C_2 - C_1) \qquad (5\text{-}11)$$

Equation (5-11) describes the time rate-of-change of chloride

in the upstream boundary reactor. In order to provide a means of estimating the temporal variation in UC, a study was made of the relationship between incoming chloride concentration and streamflow. The analysis, described in detail in Appendix B, yields the following result:

$$UC = 273.\ Q^{-.24555} \tag{5-12}$$

Equation (5-12) provides for direct computation of the upstream boundary condition at any given level of discharge in the river. Obviously, the relationship is only valid for a specified range of flow values, i.e., as $Q \rightarrow 0$, $UC \rightarrow \infty$. In the present case it is suggested that the above relationship be restricted to flow values above 200 cfs.

The equation for the downstream boundary segment may be written as:

$$\frac{dC_5}{dt} = \frac{Q}{V_5}(C_4 - C_5) + \frac{W_5}{V_5} + \frac{EA}{L_5 V_5}(C_4 - C_5) \tag{5-13}$$

Here again the effect of dispersion at the downstream boundary is neglected in the river model. Because at an average streamflow level of 4000 cfs, mass transport due to dispersion is less than 1 percent of mass transport due to net river flow, the assumption seems justified. This simplifies the present analysis by eliminating the need to incorporate a downstream boundary condition into the river model.

The initial chloride concentration in each of the 5 reactors represents another set of required input data to the model. Ideally the initial conditions are available from field measurements taken at the various sampling stations on the Great Miami River. The major difficulty here centers around the fact that all field measurements are not taken on the same day and no data is available on the chloride concentration in all 5 reactors on any

given day. For this reason, the initial conditions for the stream model are assumed equal for all 5 segments. The particular value is obtained by substituting the observed streamflow on the starting date into Equation (5-12) and solving for the resulting chloride concentration. The specific initial conditions used in the present analysis are given in Table 5-4 along with additional input data to the river model.

Table 5-4

Input Parameters to the Great Miami River Model

Segment	1	2	3	4	5
Length (FT.)	26400.	26400.	26400.	26400.	26400.
Characteristic Length (FT.)	26400.	26400.	26400.	26400.	26400.
Dispersion Coeff. (Mi^2/Day)	1.0	1.0	1.0	1.0	1.0
Initial Cond. (MG/L)					
1973 Water Year	45	45	45	45	45
1974 Water Year	47	47	47	47	47

Probabilistic Nature of Point-Source Chloride Loads

In the study of the hypothetical discharge-receiving water system presented earlier, it was assumed that the probability distribution of each point-source chloride load was known exactly. Unfortunately, in actual practice this is not the case and therefore it is necessary to approximate these distributions from available field observations at the various outfalls. A goodness-of-fit test is performed on the field data to examine the validity of

assuming a specific distribution and related parameters for the data.

The first point-source analyzed was Gulf Oil Company located at river mile 8.4. Chloride samples of the effluent were taken on an average of one per week, with a total of 48 observations available over the entire 1973 water year. The original field observations are given in Table 5-2. The chloride loadings at Gulf Oil were assumed to follow a normal distribution with mean and standard deviation as estimated from the original field data. In order to test the validity of this assumption, a chi-square goodness-of-fit test was employed. This technique compares the number of times the chloride load is observed in specified ranges with the theoretical frequency for the assumed distribution and associated parameters. The resulting chi-square statistic becomes the basis for accepting or rejecting the assumed distribution. The test of the Gulf Oil data is presented in Table 5-5. The magnitude of the statistic in the present case falls very close to the .50 probability level for 9 degrees of freedom (3 degrees of freedom are lost because the theoretical mean is taken as the actual mean, the theoretical standard deviation is taken as the actual standard deviation, and the total number of cases is taken as the same for both the theoretical and actual frequencies). This implies there is a probability of approximately .50 of obtaining a chi-square statistic as large as 8.34 if the distribution of the original data was, in fact, normal with a mean of 4040 lbs/day and a standard deviation of 962 lbs/day. If it is decided that the assumed distribution should be rejected at a probability limit of .30 or less, then the distribution of the daily chloride loadings from Gulf Oil is accepted as normal with parameters μ = 4040 lbs/day and σ = 962 lbs/day.

The same approach is followed in attempting to describe the probability distribution of the daily chloride

TABLE 5-5

CHI-SQUARE GOODNESS OF FIT TEST FOR NORMALITY
OF GULF OIL LOADINGS
(OCT. 1 1973 - SEPT. 30 1974)

$n = 48$, $\bar{X} = 4040.$, $\sigma = 962.$

UPPER LIMIT OF INTERVAL	$X - \bar{X}$	STANDARD UNITS $Z = (X - \bar{X})/\sigma$	REL. FREQUENCY FROM $-\infty$ TO Z	CELL FREQUENCY	ABSOLUTE THEORETICAL FREQUENCY (f)	ACTUAL FREQUENCY (F)	F - f	$(F - f)^2/f$
3000.	-1040.	-1.081	.1401	.1401	6.7	5	1.7	.43
3250.	-790.	-.821	.2061	.0660	3.2	4	.8	.20
3500.	-540.	-.561	.2877	.0816	3.9	4	.1	.00
3750.	-290.	-.302	.3821	.0944	4.5	7	2.5	1.39
4000.	-40.	-.042	.4840	.1019	4.9	3	1.9	.74
4250.	210.	.218	.5871	.1031	4.9	8	3.1	1.96
4500.	460.	.478	.6844	.0973	4.7	3	1.7	.61
4750.	710.	.738	.7704	.0860	4.1	3	1.1	.30
5000.	960.	.998	.8413	.0709	3.4	1	2.4	1.69
5250.	1210.	1.258	.8962	.0549	2.6	4	1.4	.75
5500.	1460.	1.518	.9357	.0395	1.9	3	1.1	.64
∞	∞	∞	1.0000	.0643	3.1	3	.1	.00
							total =	8.71

Degrees of Freedom = 12 - 3 = 9

$\chi^2_{.30} = 10.70$ $\chi^2_{.50} = 8.34$

TABLE 5-6

CHI-SQUARE GOODNESS OF FIT TEST FOR NORMALITY
OF NATIONAL LEAD LOADINGS
(NOV. 24 1973 - APR. 13 1974)

$n = 141$, $\bar{X} = 1165.8$, $\sigma = 510.2$

UPPER LIMIT OF INTERVAL	$X - \bar{X}$	STANDARD UNITS $Z = (X - \bar{X})/\sigma$	REL. FREQUENCY FROM $-\infty$ TO Z	CELL FREQUENCY	ABSOLUTE THEORETICAL FREQUENCY (f)	ACTUAL FREQUENCY (F)	F - f	$(F - f)^2/f$
716.7	-449.1	-.880	.1894	.1894	26.6	26	.6	.01
927.8	-238.0	-.467	.3192	.1298	18.3	21	2.7	.40
1138.9	-26.9	-.053	.4801	.1609	22.7	30	7.3	2.35
1350.0	184.2	.361	.6406	.1605	22.6	21	1.6	.11
1561.1	395.3	.775	.7823	.1417	20.0	17	3.0	.45
1772.2	606.4	1.189	.8830	.1007	14.2	10	4.2	1.24
1983.3	817.5	1.602	.9452	.0622	8.8	5	3.8	1.64
2194.4	1028.6	2.016	.9783	.0331	4.7	6	1.3	.36
∞	∞	∞	1.0000	.0217	3.1	5	1.9	1.16
							total =	7.72

Degrees of Freedom = 9 - 3 = 6

$\chi^2_{.30} = 7.23$ $\chi^2_{.20} = 8.56$

TABLE 5-7

CHI-SQUARE GOODNESS OF FIT TEST FOR LOG-NORMALITY
OF NATIONAL LEAD LOADINGS
(NOV. 24 1973 - APR. 13 1974)

$n = 141$, $\bar{X} = 3.0256$, $\sigma = .19425$

UPPER LIMIT OF INTERVAL	$X - \bar{X}$	STANDARD UNITS $Z = (X - \bar{X})/\sigma$	REL. FREQUENCY FROM $-\infty$ TO Z	CELL FREQUENCY	ABSOLUTE THEORETICAL FREQUENCY (f)	ACTUAL FREQUENCY (F)	F - f	$(F - f)^2/f$
2.720	-.306	-1.575	.0571	.0571	8.1	10	1.9	.45
2.809	-.217	-1.117	.1314	.0743	10.5	12	1.5	.21
2.897	-.129	-.664	.2546	.1232	17.4	14	3.4	.66
2.985	-.041	-.211	.4168	.1622	22.9	16	6.9	2.08
3.074	.048	.247	.5987	.1819	25.6	27	1.4	.08
3.162	.136	.700	.7580	.1593	22.5	30	7.5	2.50
3.250	.224	1.153	.8749	.1169	16.5	16	.5	.02
3.338	.312	1.606	.9463	.0714	10.1	11	.9	.08
∞	∞	∞	1.0000	.0537	7.6	5	1.4	.26
							total =	6.34

Degrees of Freedom = 9 - 3 = 6

$\chi^2_{.50} = 5.35$ $\chi^2_{.30} = 7.23$

load from National Lead. Samples of the National Lead effluent were taken daily over the 1973 water year, providing a total data base of 365 observations. Examination of this data revealed a distinct shift in the magnitude of the chloride load from late-November through mid-April as compared to the remainder of the year. In particular, the mean daily load from National Lead over the former period is 1166 lbs/day, while the mean load over the latter period is approximately 647 lbs/day. Rather than combining all 365 field observations, the 141 cases from Nov. 24, 1973 through April 13, 1974 are analyzed separately in the goodness-of-fit test. This set of data provided acceptable results in the test of normality, while the analysis of all 365 observations produced a rejection of the normality assumption. For this reason the chloride load from National Lead over the entire year is assumed to follow the probability distribution derived from analysis of the 141 cases alone. Treatment of the National Lead data in this manner produces a negligible error. For example, the theoretical chloride concentration at the point of discharge may be written:

$$C(mg/\ell) = (.185)\ \frac{W\ (lbs/day)}{Q(cfs)} \qquad (5\text{-}14)$$

At the mean load of 647 lbs/day and the mean streamflow of 3982 cfs, the resulting chloride contribution to the river from National Lead is approximately .03 mg/ℓ . At the minimum streamflow recording during the 1973 water year (655 cfs), the concentration resulting from an average chloride load of 647 lbs/day is .18 mg/ℓ. It is quite apparent that the chloride concentration attributable to the National Lead effluent is almost always negligible at the lower mean discharge level. At the upper mean loading of 1166 lbs/day and a minimum streamflow of 655 cfs, the resulting theoretical concentration is .33 mg/ℓ- still very small in

comparison with overall stream levels.

The results of the goodness-of-fit test for the first 141 cases are given in Table 5-6. Under the assumption of normality with a mean of 1166 and a standard deviation of 510, the chi-square statistic is less than that given for a probability of .20. Because the probability of the theoretical distribution producing a statistic of 7.23 or more is only .30, the assumed distriubtion is rejected at the .30 probability limit. The next step is to test the data under the assumption of log-normality. The results of this test are presented in Table 5-7. In the present case the test statistic is less than 7.23, and therefore the log-normal assumption is accepted at the .30 probability level.

Rather than proceed with a separate analysis of the remaining loading data for National Lead, the log-normal distribution with a mean of 3.0256 and a standard deviation of .19425 is assumed to hold for the entire 1973 water year. As previously shown, this assumption introduces negligible error in estimating chloride levels in the river while reducing the total amount of data manipulation required in the analysis.

Generation of Point-Source Chloride Loads

Given the respective probability distributions and associated parameters which describe the daily chloride loadings from Gulf Oil and National Lead, a random number generator is again used to generate the individual loads. Unlike the previous system in which chloride loads between specified input times are derived by linear interpolation, the chloride load during any given day in the present system is assumed to be constant over the 24-hour period.

Two different approaches are employed for providing streamflow recordings to the river model. The first approach assumes that a streamflow observation is available directly from an on-site recording device, e.g., a Telemark station.[4] In this case, the flow information is obtained by phone and is known exactly. The specific readings here are assumed to correspond to the U.S.G.S. daily mean values given in Tables A-2 and A-4 of Appendix A.

The second approach assumes that no on-site recording device is available, and daily streamflows must, therefore, be either estimated or obtained by means of a field sample. The procedure for gathering streamflow data in this case employs direct field measurement during periods of significant rainfall and the subsequent period of rise. At all other times, daily values are estimated from typical recession curves derived for the watershed.

To determine the times of required field samples it is first necessary to define the terms "significant rainfall" and "period of rise." A significant rainfall is a precipitation level which will likely result in a deviation of streamflow from normal recession patterns. The period of rise is the time associated with a significant rainfall over which this deviation occurs. During the period of rise, the flow data cannot be adequately estimated using typical recession patterns and, therefore, flow measurements must be gathered by direct sampling. In order to determine the significant rainfall and period of rise for

---

[4]A Telemark station allows for immediate computation of streamflow from a stage-discharge curve. The station phone is called and a signal is given which is intrepreted as a specific gage height. The state-discharge curve is then used to determine the current streamflow value.

the system under investigation, 3 years of precipitation data at the Hamilton-Fairfield National Weather Service Station was obtained. This information, covering the period from January 1972 through December 1975, was used in conjunction with U.S.G.S. mean daily streamflow records over the same period to arrive at an estimate of these two parameters. A value of .1 inches of precipitation represented the minimum rainfall which could be expected to bring about deviation from normal recession patterns. Also, a span of 2 days represented the average time during which the deviation persisted (given no further precipitation).

The implication of these values with respect to the problem of obtaining streamflow data is as follows: given an initial streamflow measurement on the first day of a sampling period, flow values for all following days which receive less than significant rainfall (.1 inch) are estimated from appropriate recession equations (Appendix A); on all days receiving a significant level of precipitation and for one day following, flow data must be obtained by direct field measurement. This approach only seems reasonable in view of the great difficulties presented in attempting to predict streamflows during the period of rise. Sophisticated computer programs are available which provide very complex algorithms for computing daily flows, but the expense and time required for their application in the present case is beyond the scope of this research. For this reason, the previous two methods of gathering daily streamflow recordings seem to provide the most realistic solution. The streamflow data derived using the second method is given in Tables A-3 and A-5 of Appendix A.

The Model and Computer Program

The system of ordinary differential equations which comprise the river model are solved by the Adams-Falkner

predictor-corrector method discussed previously. A computer program MIAMI reads all input data, sets up the equations and calls upon the DVDQ subroutine to solve for the chloride concentration in the river at one day intervals. Output is therefore given on a daily basis over the specified period. A listing of MIAMI is contained in Appendix G.

Derivation of Model Prediction Error

Given the theoretical probability distributions of each point-source and the derived forecasting model of the background chloride concentration, several replications of the computer simulation over a given period are averaged to arrive at the predicted daily chloride concentration in the stream. The model output on days corresponding to available historical data is analyzed to evaluate the probability distribution of the model error. Once the distribution of the error between the model prediction and the true river concentration is known, then the decision of whether or not to sample is made each day based upon the probability of a stream standard violation for that day. In other words, if the model error follows a normal distribution with mean μ_E and standard deviation σ_E, then the probability of a stream standard violation can easily be computed from the model prediction. The application of this technique is given in the following section.

It is important to recognize that when comparing model predictions of the chloride concentration in the river with actual field observations, the model predictions are obtained using generated point-source loads rather than the original loading data. If an estimate of the model error is desired which only reflects the error associated with the transfer function itself, then input to the model should be comprised of historical data. This is referred

to as validation of the model. On the other hand, the present case calls for an estimate of the error under the condition that only the theoretical distributions of the point-source loads are known. Therefore, the resulting error is a reflection of not only the physical river model, but also the derived input descriptors.

The model predictions for the 1973 water year are given in Tables F-1 and F-2 respectively, of Appendix F (for both methods of gathering streamflow input). The model output is compared to the 30 available field measurements in order to evaluate the mean and standard deviation of the prediction error. Tables 5-8 and 5-9 contain the resulting prediction errors for the 30 dates in question using U.S.G.S. and estimated flows, respectively. This data is subsequently analyzed to provide a statistical description of the deviation between model output and actual chloride concentrations in the Great Miami River. A histogram of the error data is shown below for chloride predictions employing U.S.G.S. flows:

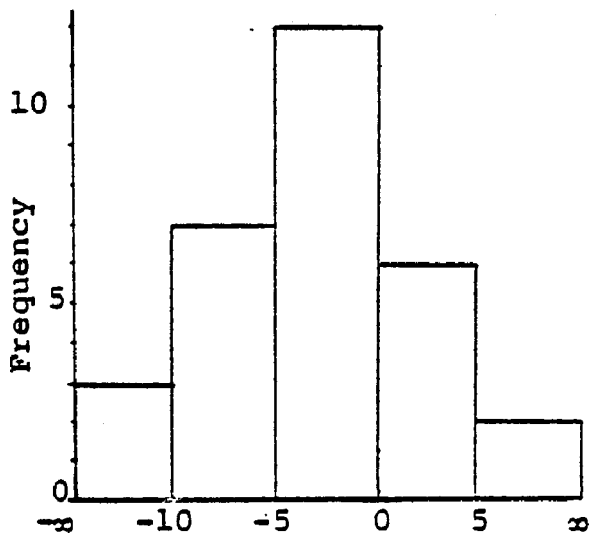


TABLE 5-8

PREDICTED VS. ACTUAL CHLORIDE CONC. USING U.S.G.S. FLOWS

OCTOBER 1973 - JUNE 1974

DATE	ACTUAL CONC. (MG/L)	PRED. CONC. (MG/L)	ERROR (MG/L)	DATE	ACTUAL CONC. (MG/L)	PRED. CONC. (MG/L)	ERROR (MG/L)
OCT. 4	40.	40.	0.	FEB. 20	39.	32.	-7.
OCT. 9	47.	47.	0.	FEB. 26	50.	34.	-16.
OCT. 16	55.	49.	-6.	MAR. 5	36.	35.	-1.
OCT. 23	59.	51.	-8.	MAR. 14	40.	38.	-2.
NOV. 1	55.	48.	-7.	MAR. 20	36.	39.	3.
NOV. 8	52.	50.	-2.	MAR. 26	50.	39.	-11.
NOV. 14	57.	52.	-5.	APR. 17	35.	36.	-1.
NOV. 20	50.	48.	-2.	APR. 23	38.	37.	-1.
DEC. 3	25.	36.	11.	MAY 7	53.	42.	-11.
DEC. 12	33.	40.	7.	MAY 15	45.	41.	-1.
DEC. 19	44.	42.	-2.	MAY 23	40.	38.	-2.
DEC. 26	40. *	28.	-12.	MAY 28	38.	42.	4.
JAN. 4	39.	36.	-3.	JUN. 3	40.	43.	3.
JAN. 9	46.	39.	-7.	JUN. 10	50.	46.	-4.
FEB. 5	37.	36.	-1.	JUN. 19	50.	49.	-1.
FEB. 12	46.	37.	-9.				

* excluded from analysis- rainfall

TABLE 5-9

PREDICTED VS. ACTUAL CHLORIDE CONC. USING PREDICTED FLOWS

OCTOBER 1973 - JUNE 1974

DATE	ACTUAL CONC. (MG/L)	PRED. CONC. (MG/L)	ERROR (MG/L)	DATE	ACTUAL CONC. (MG/L)	PRED. CONC. (MG/L)	ERROR (MG/L)
OCT. 4	40.	40.	0.	FEB. 20	39.	32.	-7.
OCT. 9	47.	46.	-1.	FEB. 26	50.	34.	-16.
OCT. 16	55.	49.	-6.	MAR. 5	36.	35.	-1.
OCT. 23	59.	52.	-7.	MAR. 14	40.	38.	-2.
NOV. 1	55.	48.	-7.	MAR. 20	36.	40.	4.
NOV. 8	52.	49.	-3.	MAR. 26	50.	40.	-10.
NOV. 14	57.	52.	-5.	APR. 17	35.	38.	3.
NOV. 20	50.	42.	-8.	APR. 23	38.	37.	-1.
DEC. 3	25.	34.	9.	MAY 7	53.	41.	-12.
DEC. 12	33.	41.	8.	MAY 15	45.	41.	-4.
DEC. 19	44.	42.	-2.	MAY 23	40.	38.	-2.
DEC. 26	40.*	28.	-12.	MAY 28	38.	39.	1.
JAN. 4	39.	36.	-3.	JUN. 3	40.	42.	2.
JAN. 9	46.	39.	-7.	JUN. 10	50.	46.	-4.
FEB. 5	37.	36.	-1.	JUN. 19	50.	48.	-2.
FEB. 12	46.	37.	-9.				

* excluded from analysis- rainfall

In particular, the chi-square goodness-of-fit test is used to test the initial assumption that the model error follows a normal distribution with mean $\mu = -2.77$ mg/ℓ and standard deviation $\sigma = 5.54$ mg/ℓ as determined from the 30 days of data. The results of the test are presented in Table 5-10. The assumption of normality is acceptable at the .75 probability level:

$$(F_i - f_i)^2/f_i < \chi^2_{.75} \qquad (5\text{-}15)$$

$$.433 < .580$$

Note that the predicted chloride concentrations for each day are obtained from a single replication rather than an average of several runs over the same period. Because the chloride contributions from point-sources are almost negligible in the present system, a single run is assumed to be representative of the average or expected output from several runs. Background chloride levels are determined explicitly from daily streamflows by means of (B-2) derived in Appendix B, and therefore they remain the same over several replications of a simulated period. Additionally, the background concentration represents almost all of the overall chloride concentration present in the river at the monitored position. Thus, the predicted chloride concentrations will have negligible variation from run to run, and a single replication output is treated as representative of the average of several.

Given that the derived error distribution holds for the entire 1973 water year, the probability of a stream standard violation on any day can easily be computed from the model prediction for that day, as shown in the next section. By convention the error will always be defined as the model prediction minus the true river concentration.

TABLE 5-10

CHI-SQUARE GOODNESS OF FIT TEST FOR NORMALITY
OF MODEL ERROR USING U.S.G.S. STREAMFLOWS
(OCT. 1 1973 - JUNE 30 1974)

$n = 30$, $\overline{X} = -2.77$, $\sigma = 5.54$

UPPER LIMIT OF INTERVAL	$X - \overline{X}$	STANDARD UNITS $Z = (X - \overline{X})/\sigma$	REL. FREQUENCY FROM $-\infty$ TO Z	CELL FREQUENCY	ABSOLUTE THEORETICAL FREQUENCY (f)	ACTUAL FREQUENCY (F)	F - f	$(F - f)^2/f$
-10.	-7.23	-1.31	.0951	.0951	2.9	3	.1	.003
-5.	-2.23	-.40	.3446	.2495	7.5	7	-.5	.033
0.	2.77	.50	.6915	.3469	10.4	12	1.6	.246
5.	7.77	1.40	.9192	.2277	6.8	6	-.8	.094
∞	∞	∞	1.0000	.0808	2.4	2	-.4	.067
							total =	.443

Degrees of Freedom = 5 - 3 = 2

$\chi^2_{.75} = .58$ $\chi^2_{.90} = .21$

The Probability Threshold Sampling Policies

The sampling policies for monitoring the Great Miami River are derived from an analysis covering the 1973 water year. Two separate sets of policies are investigated - one for the case in which streamflow measurements may be directly obtained at no cost from an on-site recording device, and another set for the case in which flow data is both estimated from recession equations and measured in the field at a known cost. In both instances the form of the sampling policy is expressed as before:

> If, on any day, the probability of a stream standard violation exceeds the threshold probability P, a chloride sample is taken.
> Otherwise no sample is taken.

The set of selected policies presently under consideration has been given in Table 5-3.

The optimum policy is defined as that policy which detects at least the specified percentage of all stream standard violations at minimum cost over a one-year planning period. The percentage of violations detected for a given policy is estimated on the basis of its performance over the 1973 water year. The assertion is not made that the optimum policy for one year is necessarily optimum for every year. However, the policies are re-evaluated over a generated chloride profile covering the 1974 water year to determine if the optimum policy has, in fact, changed over this 12-month period.

A probability threshold sampling policy is described by the concentration threshold which indicates an observation should be made, i.e., if on any day the model prediction exceeds some level C_M, the probability of violation $\hat{p}$ exceeds P. Therefore, each policy has an associated threshold probability P which corresponds to a specific threshold concentration C_M for the model. This implies

that $\hat{p} > P$ whenever $c(t) > C_M$. An example is given below which illustrates the method of deciding whether or not to sample on a given day.

ERROR DISTRIBUTION = $N(\mu_E = -2.77, \sigma = 5.54)$
STREAM STANDARD = 45 mg/ℓ
MODEL OUTPUT = 42 mg/ℓ
P = .50
ERROR = MODEL OUTPUT - TRUE CONCENTRATION
SAMPLE IF $\hat{p} \geq .50$; otherwise do not sample.

$$\hat{p} = \text{prob.}(\text{true conc.} \geq 45 \text{ mg/ℓ})$$
$$= \text{prob.}(\text{error} \leq (42-45))$$
$$= \text{prob.}\left[z \leq \frac{-3 - \mu_E}{\sigma_E}\right] = \text{prob.}\left[z \leq \frac{-3 - (-2.77)}{5.54}\right]$$
$$= \text{prob.}(z \leq -.0415)$$

FROM NORMAL TABLES (Gibra, 1963):

$$\hat{p} = .4832$$

Because the violation probability on this day is less than the .50 threshold, no sample is taken.

The model predictions which coincide with the probability thresholds for each of the selected policies are given in Table F-1 of Appendix F for a chloride standard of 45 mg/ℓ. These sampling policies are derived under the assumption that daily U.S.G.S. streamflow measurements are available instantaneously from the on-site recorder. The study of the system requiring estimates of daily streamflows (as well as direct field trips) gives slightly different results. In this case the prediction error is found to follow a normal distribution with the same mean as before, but with a standard deviation of 5.81 (up from 5.54).

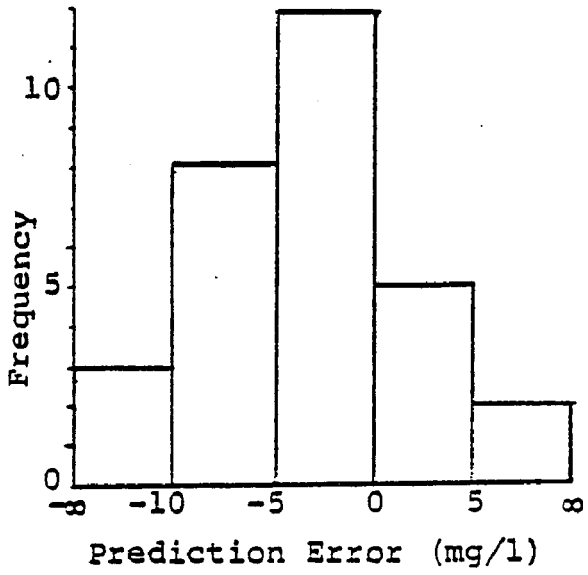


Table 5-11 gives the results of the goodness-of-fit test for the prediction errors using estimated streamflows. Although the test is not as conclusive in this particular instance, the assumption of normality is still accepted at the .50 probability level. The model predictions associated with the probability thresholds in this second system are presented in Table E-4 of Appendix E, again for a chloride standard of 45 mg/ℓ .

Although the technique is not used here, the model predictions could be increased by the absolute value of the prediction error in order to yield an overall mean error of zero. Therefore the predicted chloride concentration at time t would be given by $C_M(t) - \mu_E$. Figure 5-2 on following page shows the model errors as a function of time after incorporating the mean error into the model predictions. Inspection of Figure 5-2 shows that the distribution of the model error about the mean does not significantly change over time.

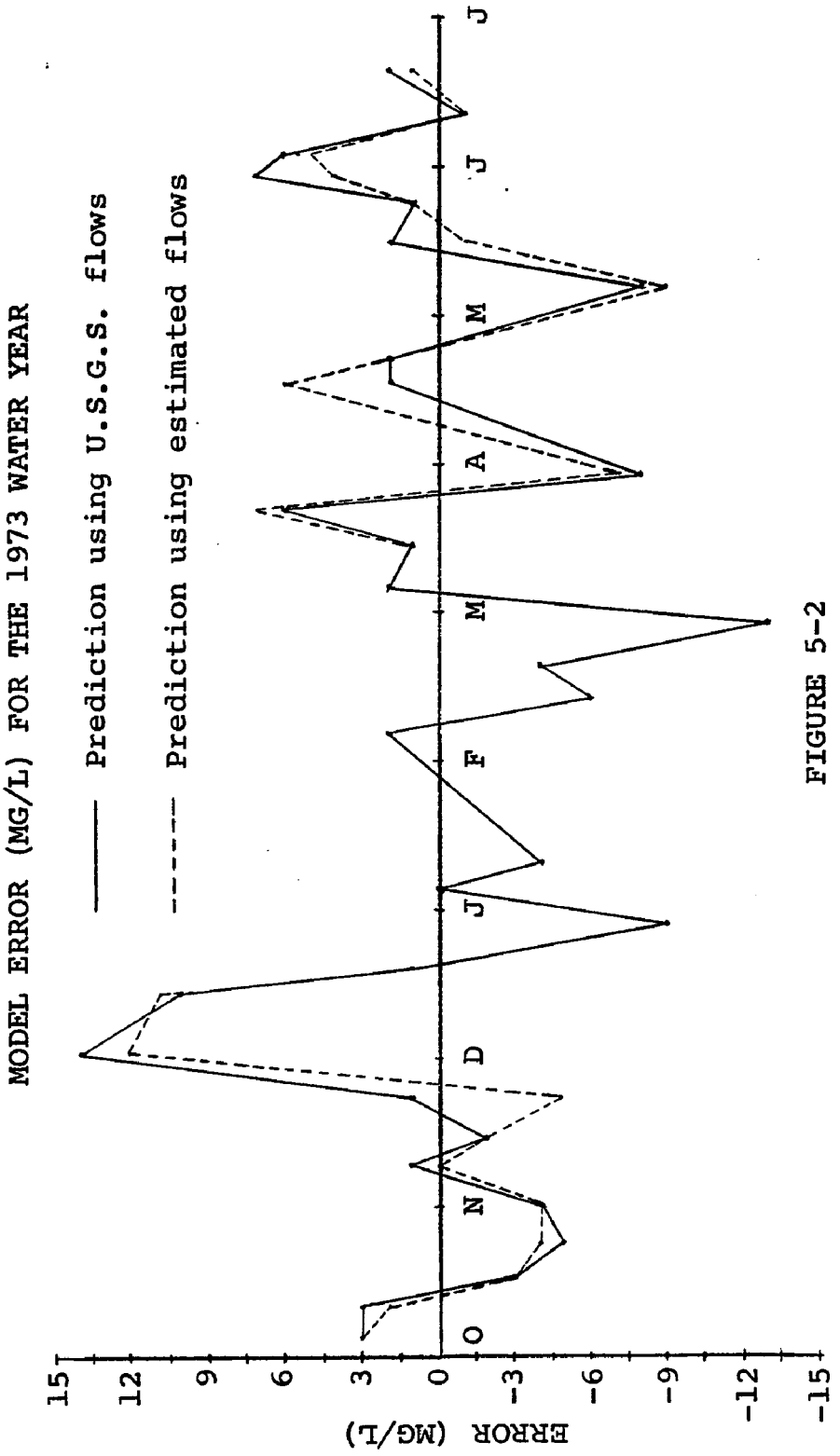

MODEL ERROR (MG/L) FOR THE 1973 WATER YEAR
Prediction using U.S.G.S. flows
Prediction using estimated flows
ERROR (MG/L)
15
12
9
6
3
0
-3
-6
-9
-12
-15
O
N
D
J
F
M
A
M
J
J

FIGURE 5-2

TABLE 5-11

CHI-SQUARE GOODNESS OF FIT TEST FOR NORMALITY
OF MODEL ERROR USING PREDICTED STREAMFLOWS
(OCT. 1973 - JUNE 30 1974)

$n = 30$, $\bar{X} = -2.77$, $\sigma = 5.81$

UPPER LIMIT OF INTERVAL	$X - \bar{X}$	STANDARD UNITS $Z = (X - \bar{X})/\sigma$	REL. FREQUENCY FROM $-\infty$ TO Z	CELL FREQUENCY	ABSOLUTE THEORETICAL FREQUENCY (f)	ACTUAL FREQUENCY (F)	F - f	$(F - f)^2/f$
-10.	-7.23	-1.24	.1075	.1075	3.2	3	-.2	.013
-5.	-2.23	-.38	.3520	.2445	7.4	8	.6	.049
0.	2.77	.48	.6844	.3324	10.0	12	2.0	.400
5.	7.77	1.34	.9099	.2255	6.7	5	-1.7	.431
∞	∞	∞	1.0000	.0901	2.7	2	-.7	.181
							total =	1.074

Degrees of Freedom = 5 - 3 = 2

$\chi^2_{.50} = 1.39$ $\chi^2_{.75} = .58$

The sensitivity of the sampling thresholds to the manner in which streamflow data is obtained may be seen by a comparison of Tables E-1 and E-4. The predicted chloride concentration which constitutes a violation probability exceeding P remains the same in both cases for policies 4 through 11, 14, 15, and 16. For policies 1, 2, 3, 12, and 13 the limiting chloride values differ by 1 mg/ℓ. This condition is a result of the difference in the standard deviation of the error for the two approaches.

Sampling Policy Costs

The same cost structure employed in the analysis of the hypothetical receiving water system is applied in the Great Miami River analysis. Again, two different costs are incurred over a given period - a fixed cost associated with sampling and a variable cost associated with damages arising from undetected violations. As discussed in detail in the Conclusions section of this chapter, these cost functions are hypothetical and not necessarily representative of actual costs in effect for the Great Miami River system. The specific damage functions under consideration are given below.

CASE 1: Daily streamflow data available at no cost; non-linear cost for detection delay.

$$C_T = 10\ n + \sum_{i=1}^{n_v} (20\ (e^{t_i} - 1)/t_i) \qquad (5\text{-}16)$$

CASE 2: Daily streamflow data available at no cost; linear cost for detection delay.

$$C_T = 10.n + 50 \sum_{i=1}^{n_v} \qquad (5\text{-}17)$$

CASE 3: Daily streamflow data not directly available; non-linear cost for detection delay.

$$C_T = 10.n + 2.K + \sum_{i=1}^{n_v} (20(e^{t_i}-1)t_i) \qquad (5\text{-}18)$$

CASE 4: Daily streamflow data not directly available; linear cost for detection delay.

$$C_T = 10.n + 2.K + 50 \sum_{i=1}^{n_v} t_i \qquad (5\text{-}19)$$

where t_i = 1,2,...11. As mentioned in Chapter IV, no delay cost is incurred for t_i = 0. Also, it is assumed that detection delay costs do not increase beyond that given for t_i=11. In Equations (5-16), (5-17), (5-18), and (5-19) n represents the total number of samples required over the one-year period, n_v is the total number of violations occurring during the year, and t_i represents the delay (in days) to detection of the i^{th} violation. In Equations (5-18) and (5-19) K represents the

total number of times streamflow measurements must be obtained by field sampling. The definitions of violation are the same as described in Chapter III.

Generation of Sample Chloride Profiles

In order to determine the percentage of violations detected and total cost associated with each selected policy, daily chloride measurements at the monitored position are required. This creates an immediate problem in view of the limited historical data available. Only 31 days were sampled during the 1973 water year, and the sample taken on Dec. 26 was eliminated from the analysis because of heavy rainfall on that date (resulting in a discrepancy between the streamflow magnitude at the time of the chloride observation and the mean daily streamflow measurement used as input to the predictive model). For this reason it was necessary to generate an artificial chloride profile from the daily model predictions. Given that the distribution of the prediction error is known, daily realizations of the deviation between model output and actual concentrations can be generated from this distribution by methods described earlier. The resulting errors are then subtracted from the model predictions to arrive at an artificial daily chloride profile over the entire year. If $C_M(t)$ represents the model chloride prediction on a particular day and $e(t)$ represents the generated prediction error for that day, the true river concentration $c(t)$ is taken as the difference between these two values:

$$C(t) = C_M(t) - e(t) \qquad (5\text{-}20)$$

Below is a list of the chloride profiles under investigation and the associated components used to derive them.

Profile 1: Oct. 1973 - Sept. 1974 (Table F-5)

$C_p(t)$ - model predictions using U.S.G.S. streamflow input (Table F-1)
$e(t)$ - generated prediction errors
$W_1(t)$ - generated chloride loads from Gulf Oil
$W_2(t)$ - generated chloride loads form National Lead

Profile 2: Oct. 1973 - Sept. 1974 (Table F-6)

$C_p(t)$ - model predictions using estimated streamflow input (Table F-2)
$e(t)$ - generated prediciton errors
$W_1(t)$ - generated chloride loads from Gulf Oil
$W_2(t)$ - generated chloride loads from National Lead

Profile 3: Oct. 1974 - Sept. 1975 (Table F-7)

$C_p(t)$ - model predictions using U.S.G.S. streamflow input (Table F-3)
$e(t)$ - generated prediction errors
$W_1(t)$ - generated chloride loads from Gulf Oil
$W_2(t)$ - generated chloride loads from National Lead

Profile 4: Oct. 1974 - Sept. 1975 (Table F-8)

$C_p(t)$ - model predictions using estimated streamflow input (Table F-4)
$e(t)$ - generated prediction errors
$W_1(t)$ - generated chloride loads from Gulf Oil
$W_2(t)$ - generated chloride loads from National Lead

Figures 5-3 through 5-6 show the distribution of the true river concentrations about the model predictions for the 1973 and 1974 water years, respectively. The artificial chloride profiles over the 1974 water year are generated on the assumption that the error distribution does not shift from that determined for the 1973 year.

Policy Evaluation

In order to test the sensitivity of the optimum policy to the particular stream standard in effect, three

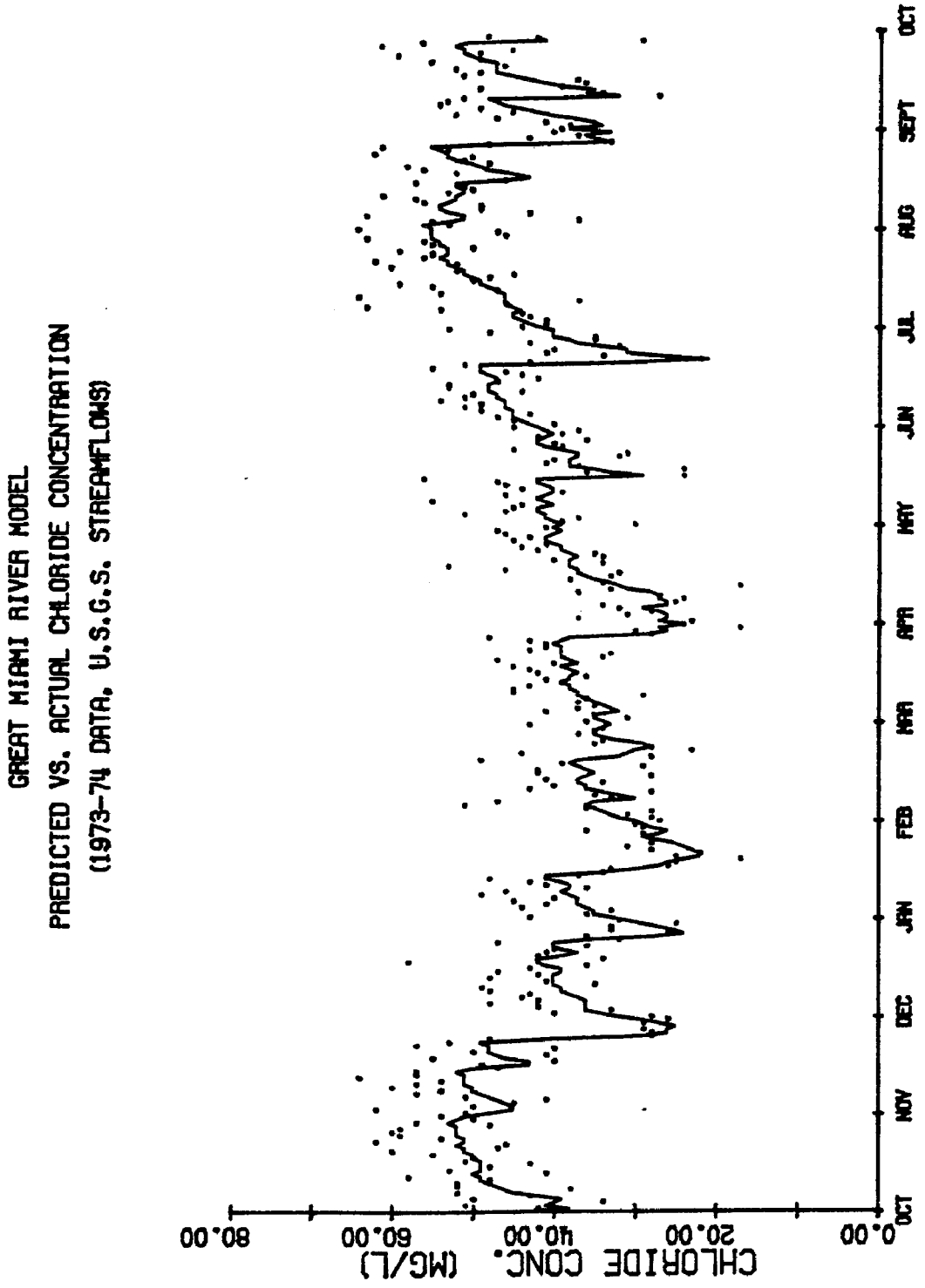

GREAT MIAMI RIVER MODEL
PREDICTED VS. ACTUAL CHLORIDE CONCENTRATION
(1973-74 DATA, U.S.G.S. STREAMFLOWS)
CHLORIDE CONC. (MG/L)
0.00
20.00
40.00
60.00
80.00
OCT
NOV
DEC
JAN
FEB
MAR
APR
MAY
JUN
JUL
AUG
SEPT
OCT
TIME

FIGURE 5-3

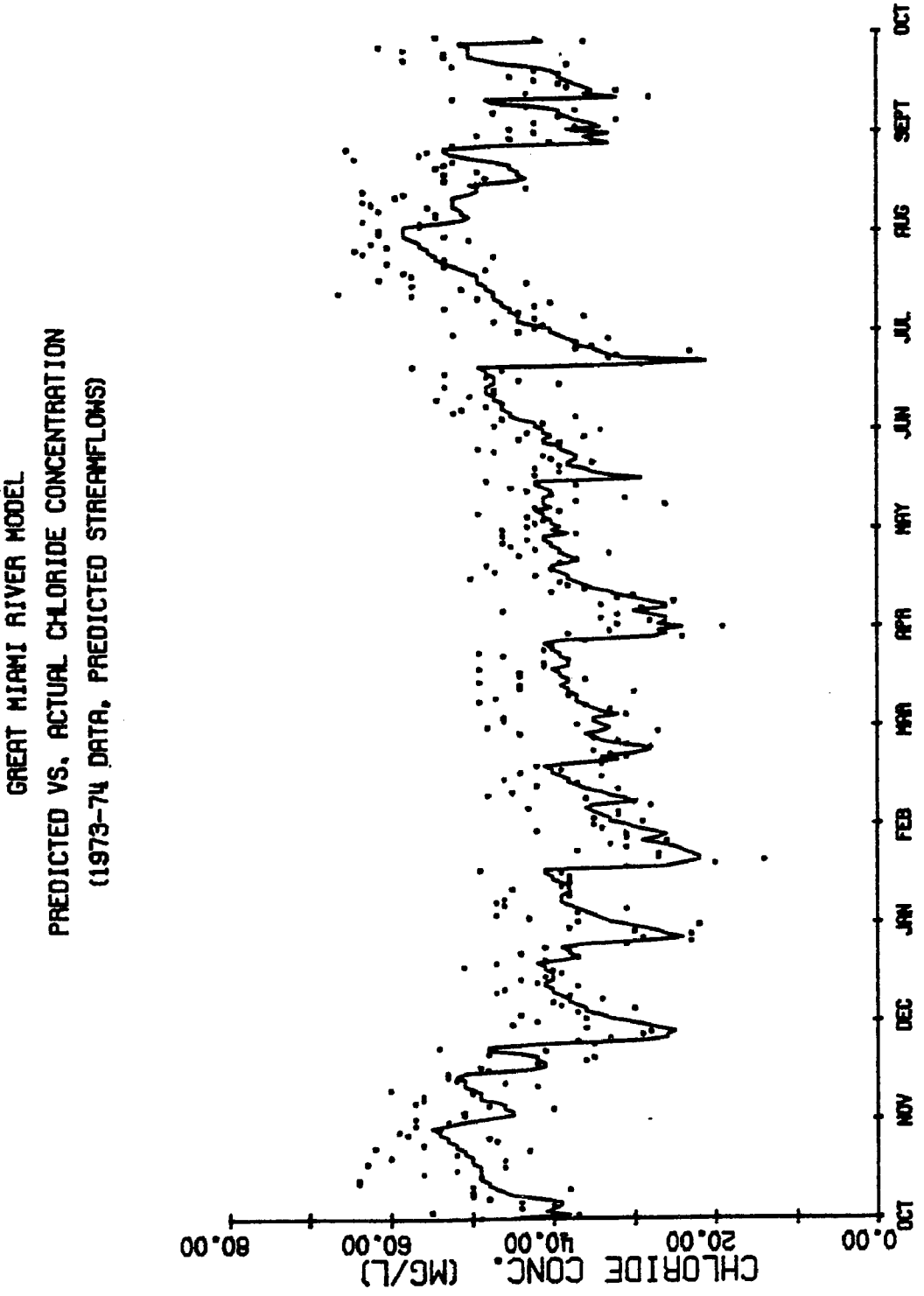

GREAT MIAMI RIVER MODEL
PREDICTED VS. ACTUAL CHLORIDE CONCENTRATION
(1973-74 DATA, PREDICTED STREAMFLOWS)
CHLORIDE CONC. (MG/L)
0.00
20.00
40.00
60.00
80.00
OCT
NOV
DEC
JAN
FEB
MAR
APR
MAY
JUN
JUL
AUG
SEPT
OCT
TIME

FIGURE 5-4

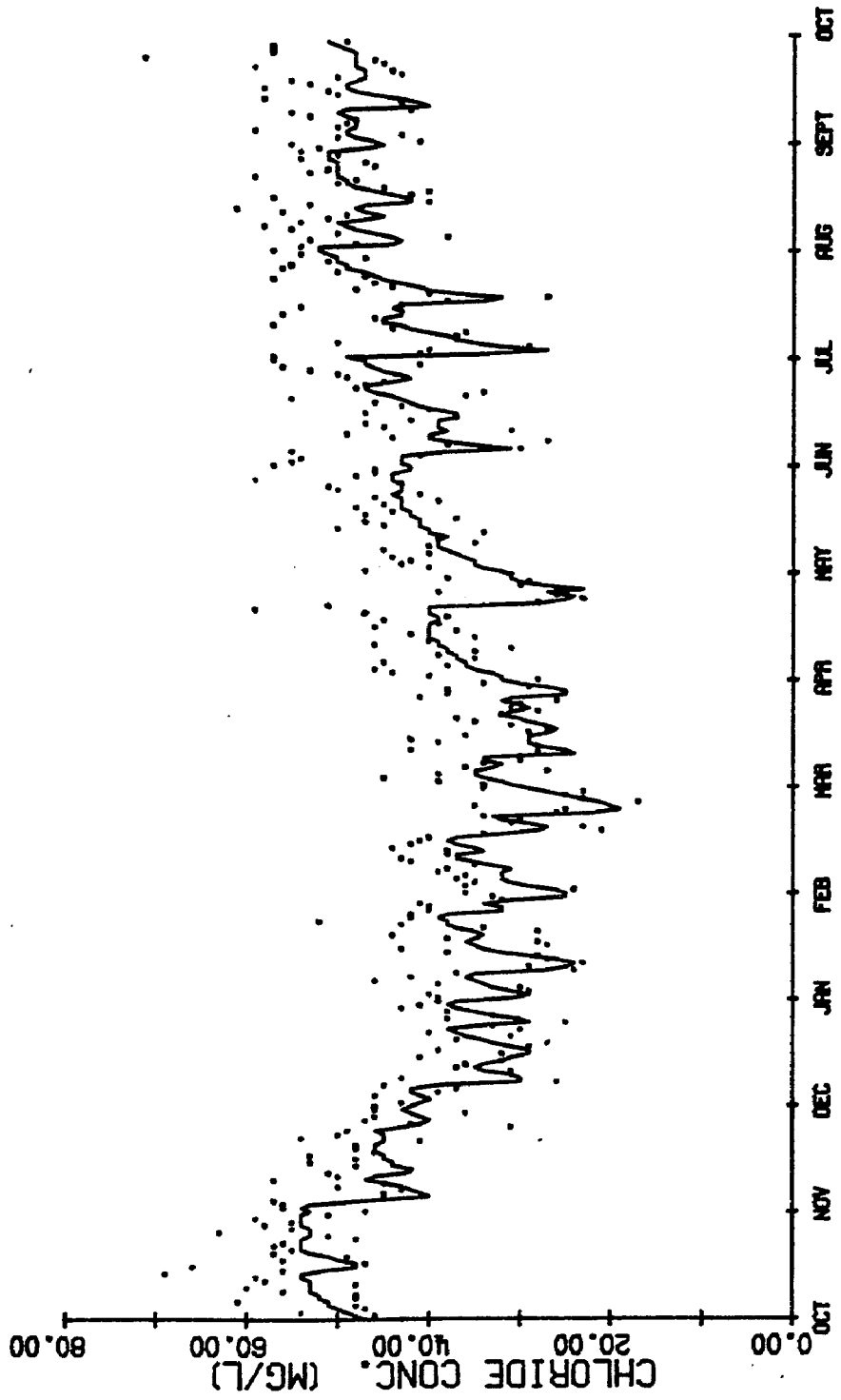

GREAT MIAMI RIVER MODEL
PREDICTED VS. ACTUAL CHLORIDE CONCENTRATION
(1974-75 DATA, U.S.G.S. STREAMFLOWS)
CHLORIDE CONC. (MG/L)
0.00
20.00
40.00
60.00
80.00
TIME
OCT
NOV
DEC
JAN
FEB
MAR
APR
MAY
JUN
JUL
AUG
SEPT
OCT

FIGURE 5-5

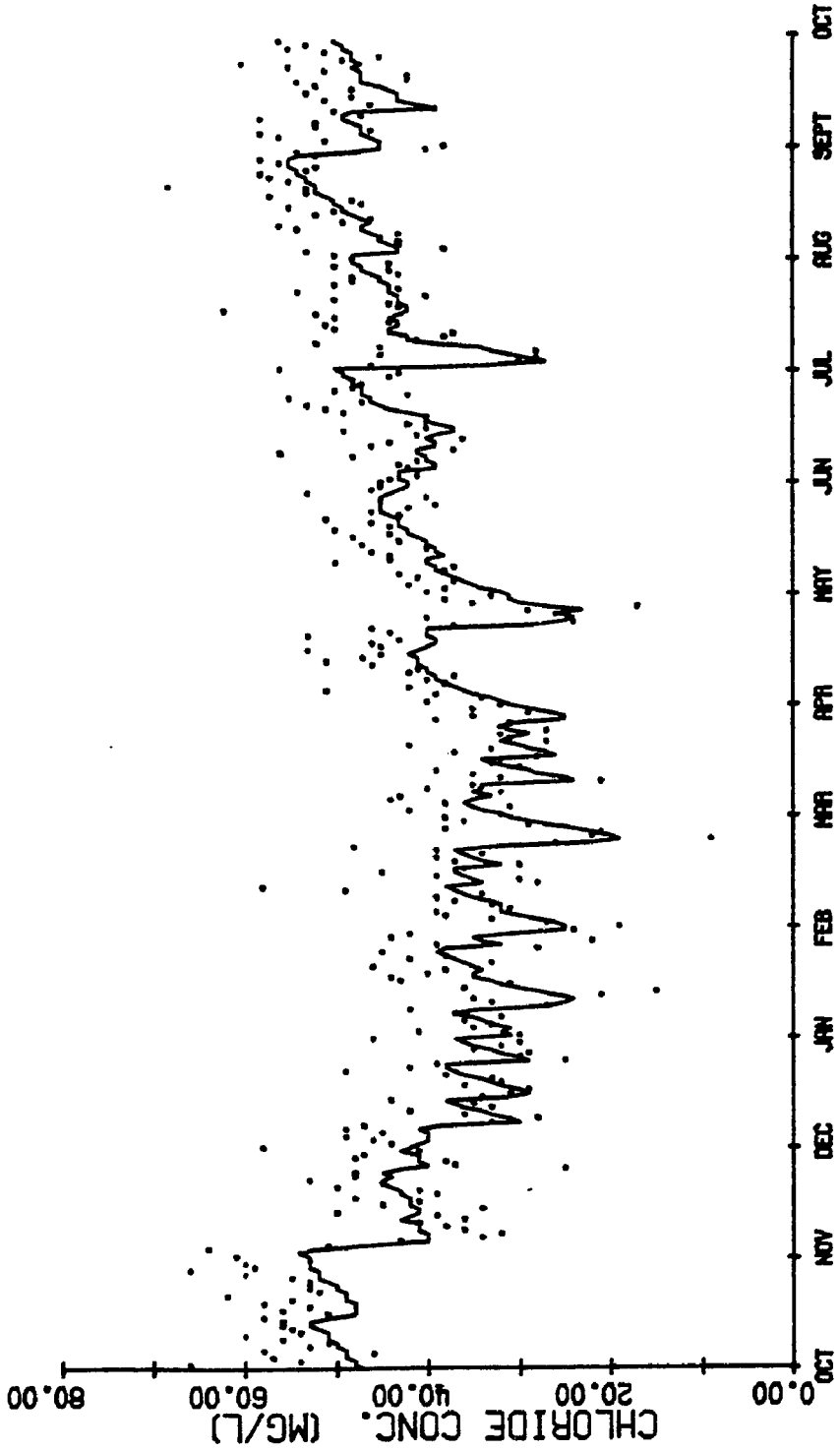

GREAT MIAMI RIVER MODEL
PREDICTED VS. ACTUAL CHLORIDE CONCENTRATION
(1974-75 DATA, PREDICTED STREAMFLOWS)
CHLORIDE CONC. (MG/L)
0.00
20.00
40.00
60.00
80.00
OCT
NOV
DEC
JAN
FEB
MAR
APR
MAY
JUN
JUL
AUG
SEPT
OCT
TIME

FIGURE 5-6

different hypothetical chloride standards are employed in the investigation - 45 mg/ℓ , 50 mg/ℓ and 55 mg/ℓ. The 16 selected sampling policies given in Table 5-3 are evaluated for each of the 4 profiles listed above. Looking first at Profile 1, the optimum policy is sought which will yield an estimated detection percentage of at least P_D over a 1 year period. Continuing with the convention employed in Chapter 4, the labels P_{D1} and P_{D2} shall refer to detection percentage under definitions 1 and 2 of a violation, respectively. Tables E-1, E-2, and E-3 of Appendix E contain the resulting detection percentages and associated costs for each of the selected policies during the 1973 and 1974 water years. Assuming a stream standard of 45 mg/ℓ and a value of $P_{D1} \geq 85\%$, examination of Table E-1 reveals that policy 4 is optimum under the cost structure represented by Equation (5-16). Given that policy 4 is then selected for monitoring the stream during the 1974 water year, it is of interest to determine if policy 4 remains optimum during this next 12-month period. Analysis of Table E-1 shows policy 4 represents the true optimum over the 1974 water year. The detection percentage and total cost for policy 4 during each year are:

1973 Water Year	1974 Water Year
$P_{D1} = 93.8\%$	$P_{D1} = 86.4\%$
Total Cost = 2682	Total Cost - 2574

Although the value of P_{D1} drops from 1973 to 1974, policy 4 still retains a detection percentage of over 85%. The associated cost differs by only 4% from 1973 to 1974. It is obvious that selection of an optimum policy based on its performance over a single year does not insure that this same policy will remain optimum for every other year. A more complete approach might use the average value of P_{D1} and total cost derived from an analysis of the last 10 or 20 years of data to arrive at an optimum policy for

future sampling. The time and computer costs involved are beyond the contraints of this research. However, it can be seen that the above results exhibit a consistency which may be used in determining future sampling strategies.

If the cost structure changes to that given by Equation (5-17), policy 4 is again optimum for both years.

1973 Water Year	1974 Water Year
P_{D1} = 93.8%	P_{D1} = 86.4%
Total Cost = 2750	Total Cost = 2670

If the detection percentage is now taken as $P_{D2} \geq 85\%$ and the cost function as that given by Equation(5-16), examination of Table E-1 reveals that policy 6 is selected as optimum on the basis of its performance over the 1973 water year:

P_{D2} = 91.4%
Total Cost = 2594

Policy 6 is therefore used as the plan for monitoring the river during the following 12 months. The results for policy 6 over the 1974 water year are:

P_{D2} = 94.6%
Total Cost = 2396

In this instance the performance of the selected policy is actually better than anticipated. Policy 6 does, in fact, represent the optimum policy over the 1974 water year.

Consider now the conditions in effect under Profile 2. In this case streamflow input for predicting chloride levels must be either estimated or directly measured in the field at some cost. The damage function is given here by Equation (5-18). The selected policies are each evaluated and the results contained in Table E-4 of Appendix E. From analysis of performances over Profile 2, policy 4 is again selected for a required $P_{D1} \geq 85\%$. Inspection of

Table E-4 reveals that policy 4 is also the optimum policy during the 1974 water year:

1973 Water Year	1974 Water Year
P_{D1} = 86.8%	P_{D1} = 91.7%
Total Cost = 3278	Total Cost = 2760

If the damage function shifts to that given by Equation (5-19), the same results hold true:

1973 Water Year	1974 Water Year
P_{D1} = 86.8%	P_{D1} = 91.7%
Total Cost = 3450	Total Cost = 2840

Assuming a detection specification $P_{D2} \geq 85\%$ and a cost function represented by Equation(5-18), policy 6 is selected for sampling during the 1974 water year. In this case, the optimum policy is actually policy 7. The result is a cost of 48 over that given by the missed optimum policy:

1974 Water Year

Policy 6	Policy 7
P_{D2} = 92.6%	P_{D2} = 88.9%
Total Cost = 2740	Total Cost = 2692

The policy selected (#6) satisifies the requirement of $P_{D2} \geq 85\%$, but yields a total cost of 2740 as compared to 2692 associated with policy 7.

If the cost function is changed to Equation (5-19), policy 5 and policy 6 both satisfy the definition of optimum. Assuming policy 5 is selected on the basis of its higher detection percentage over the 1973 water year, an excess cost of 40 is incurred during the next 12 months.

1974 Water Year

Policy 5	Policy 6
P_{D2} = 93.2%	P_{D2} = 92.6%
Total Cost = 2960	Total Cost = 2920

Although in certain instances, the optimum policy over the 1974 water year is not selected, it should be

clear that the discrepancy is slight. Inefficient policies are avoided and the policy employed is often the optimum. Table 5-12 gives the optimum policies for various values of P_{D1} and P_{D2} assuming streamflow data is known exactly at no cost. It can be seen that under the non-linear cost structure and at a stream standard of 45 mg/ℓ , the optimum policy is selected at levels of $P_{D1} <$ 80% and $P_{D2} <$ 95%. For P_{D1} and $P_{D2} \geq$ 99%, policy 2 is selected for monitoring over the next 12 months. However, policy 2 only yields a value of 97.7% for P_{D1}, slightly under the goal of the sampling program. Similarly, policy 2 gives a P_{D2} of 99.4% at a required P_{D2} of 100% - again slightly under the specification. Under a hypothetical chloride standard of 50 mg/ℓ, the optimum policy for sampling during the 1974 water year is selected for $P_{D1} \geq$ 95% and $P_{D2} \geq$ 99%. At required levels of P_{D1} and P_{D2} below 80%, policy 7 is selected rather than the optimum - policy 5. Plan 7 yields a P_{D1} of 62.8% and a P_{D2} of 76.3% and therefore meets the P_{D1} and P_{D2} requirements below 70% and 80% respectively. However, during the 1974 water year policy 7 yields an excess cost of 157 or 8.8% over the optimum given by policy 5.

As the chloride standard increases to a value of 55 mg/ℓ , policy 1 is selected for P_{D1} and $P_{D2} \geq$ 99%. Table E-3 of Appendix E reveals that none of the selected policies detects greater than or equal to 99% of the violations during the 1974 water year. The actual detection given by policy 1 is 96.9% for P_{D1} and 98.1% for P_{D2}. For P_{D1} and $P_{D2} <$ 60%, policy 6 is employed in favor of the true optimum - policy 5. In each case policy 6 satisfies the detection requirement during the 1974 water year, but at a cost of 26 or 2% over the cost given by policy 5.

As discussed in Chapter IV, the minimum-cost boundary can be used to restrict the number of candidates in the search for an optimum policy. For example, the top-half

TABLE 5-12

OPTIMUM POLICY VS. REQUIRED VIOLATION DETECTION (U.S.G.S. FLOWS)

REQUIRED DETECTION PERCENTAGE P_D	$C = 10*n + \Sigma(20(e^{t_i} - 1)/t_i)$						$C = 10*n + 50\Sigma t_i$					
	SS = 45 MG/L		SS = 50 MG/L		SS = 55 MG/L		SS = 45 MG/L		SS = 50 MG/L		SS = 55 MG/L	
	P_{D1}	P_{D2}	P_{D1}	P_{D2}	P_{D1}	P_{D2}	P_{D1}	P_{D2}	P_{D1}	P_{D2}	P_{D1}	P_{D2}
100%	2 (1)	2 (1)	1*	1*	1 (-)	1 (-)	2 (1)	2 (1)	1*	1*	1 (-)	1 (-)
99%	2 (1)	2*	1*	1*	1 (-)	1 (-)	2 (1)	2*	1*	1*	1 (-)	1 (-)
95%	3 (2)	4 (5)	2*	3 (2)	1*	1*	3 (2)	3 (5)	2*	3 (2)	1*	1*
90%	4 (2)	6*	3 (2)	6 (5)	2 (1)	2 (1)	3 (2)	3 (6)	3 (2)	6 (5)	2 (1)	2 (1)
80%	6 (5)	6*	4 (5)	7 (5)	3 (1)	5 (2)	3 (5)	3 (6)	4 (5)	6 (5)	3 (1)	5 (2)
70%	6*	6*	7 (5)	7 (5)	6 (2)	6 (4)	3 (6)	3 (6)	6 (5)	6 (5)	5 (2)	5 (4)
60%	6*	6*	7 (5)	7 (5)	6 (4)	6 (5)	3 (6)	3 (6)	6 (5)	6 (5)	5*	5*
50%	6*	6*	7 (5)	7 (5)	6 (5)	6 (5)	3 (6)	3 (6)	6 (5)	6 (5)	5*	5*
25%	6*	6*	7 (5)	7 (5)	6 (5)	6 (5)	3 (6)	3 (6)	6 (5)	6 (5)	5*	5*
10%	6*	6*	7 (5)	7 (5)	6 (5)	6 (5)	3 (6)	3 (6)	6 (5)	6 (5)	5*	5*

* INDICATES OPTIMUM POLICY FOR BOTH THE 1973 AND 1974 WATER YEARS
( ) INDICATES OPTIMUM POLICY FOR 1974 WATER YEAR
\- INDICATES OPTIMUM POLICY IS NOT AVAILABLE

TABLE 5-13

OPTIMUM POLICY VS. REQUIRED VIOLATION DETECTION (PREDICTED FLOWS)

REQUIRED DETECTION PERCENTAGE P_D	$C = 10*n + \Sigma(20(e^{ti} - 1)/t_i) + 2*k$						$C = 10*n + 50\Sigma\, t_i + 2*k$					
	SS = 45 MG/L		SS = 50 MG/L		SS = 55 MG/L		SS = 45 MG/L		SS = 50 MG/L		SS = 55 MG/L	
	P_{D1}	P_{D2}	P_{D1}	P_{D2}	P_{D1}	P_{D2}	P_{D1}	P_{D2}	P_{D1}	P_{D2}	P_{D1}	P_{D2}
100%	1*	1*	2 (1)	2 (1)	1 (-)	1 (-)	1*	1*	2 (1)	2 (1)	1 (-)	1 (-)
99%	1*	1 (2)	2 (1)	2*	1 (-)	1 (-)	1*	1 (2)	2 (1)	2*	1 (-)	1 (-)
95%	2 (3)	3 (4)	2*	4 (3)	4 (1)	4 (1)	2 (3)	3 (4)	2*	4 (3)	4 (1)	4 (1)
90%	3 (4)	5 (6)	3*	7 (4)	5 (1)	5 (3)	3 (4)	5 (4)	3*	7 (4)	4 (1)	4 (3)
80%	5 (6)	6 (7)	6 (3)	7 (6)	5 (4)	6 (4)	5 (4)	5 (4)	6 (3)	7 (6)	4*	4*
70%	6 (7)	6 (7)	7 (5)	7 (6)	6 (5)	6 (5)	5 (4)	5 (4)	7 (5)	7 (6)	4 (5)	4 (5)
60%	6 (7)	6 (7)	7 (6)	7 (6)	6*	6*	5 (4)	5 (4)	7 (6)	7 (6)	4 (5)	4 (5)
50%	6 (7)	6 (7)	7 (6)	7 (6)	6*	6*	5 (4)	5 (4)	7 (6)	7 (6)	4 (5)	4 (5)
25%	6 (7)	6 (7)	7 (6)	7 (6)	6*	6*	5 (4)	5 (4)	7 (6)	7 (6)	4 (5)	4 (5)
10%	6 (7)	6 (7)	7 (6)	7 (6)	6*	6*	5 (4)	5 (4)	7 (6)	7 (6)	4 (5)	4 (5)

* INDICATES OPTIMUM POLICY FOR BOTH THE 1973 AND 1974 WATER YEARS
( ) INDICATES OPTIMUM POLICY FOR 1974 WATER YEAR
\- INDICATES OPTIMUM POLICY IS NOT AVAILABLE

of Figure D-6 shows the minimum-cost boundary passing through policies 1, 2, 3, 4, 6 and 7. Any policy not positioned on this boundary need not be considered further, as it may be improved upon by either one of the boundary policies or some combination of boundary policies (although mixed-policies have not been included as possible sampling alternatives). Analyzing the set of selected policies in this manner reduces the candidates to a more restrictive sub-set.

Table 5-13 contains the results of optimum policy selection under the assumption that streamflow inputs to the predictive model must be either estimated or directly measured at a specific fixed cost. In each case it can be seen that the policy selected for the 1974 water year is either optimum or within a small percentage of the P_D and associated cost given by the optimum. As previously stated, the sampling policies do show a marked consistency in detection effectiveness and efficiency from year to year, and therefore the process of selection on the basis of previous performance seems to provide a reasonable approach for evaluating the selected policies. The results averaged over several years of violation data would provide better insurance that the optimum policy is employed in future sampling.

Comparison of Variable and Univorm Sampling Intervals

The previous policies investigated are referred to as variable-time sampling policies. The specific sampling time is not fixed in advance of a monitoring period, but is instead determined continuously on the basis of daily model predictions. For river systems which exhibit large day-to-day variation in those parameters which most directly effect pollution levels, e.g., streamflow, a variable-time policy provides the capability of incorporating current conditions into the process of determining specific sampling times. As discussed in the previous literature review,

Beckers, et al. (1972) have suggested an approach to water quality surveillance which utilizes fixed sampling intervals for detection of stream standard violations. In order to provide some insight into the differences between these two policy designs, a set of 16 fixed-time policies are derived for monitoring the Great Miami River (Appendix C). Table 5-14 lists each policy along with the expected percentage of stream standard violations detected over a given period.

Table 5-14

Fixed-Time Sampling Plans

Policy	P_D (%)
A	99.
B	95.
C	90.
D	85.
E	80.
F	70.
G	60.
H	50.
I	40.
J	30.
K	25.
L	20.
M	15.
N	10.
O	5.
P	1.

These policies are based upon the assumption that the violation process may be approximated as a poisson process. Each policy has a specific sampling interval associated with it, as given in Table C-2 of Appendix C. If a sampling plan is desired which will yield an expected detection percentage of 90%, policy C is selected. At a chloride standard of 45 mg/ℓ, policy C requires chloride observations to

be made at 3 day intervals over the monitoring period. Tables E-7 through E-12 of Appendix E contain the detection probabilities and total costs for these 16 fixed-time policies over the 4 different chloride profiles and 3 hypothetical stream standards under consideration.

As an example of how these policies compare in actual practice with the variable-time policies derived previously, suppose a plan is needed which will yield at least 85% detection of all violations of a 45 mg/ℓ stream standard over the 1974 water year. Because the fixed-time policies are only derived for detection of continuous violation periods rather than individual violation days, the comparison will initially be restricted to values of P_{D1}. Inspection of Table C-2 reveals that policy D, which requires sampling at 4-day intervals, is appropriate. Assuming Profile 3 represents the actual chloride profile at the monitored position during the 1974 water year, policy D is evaluated on the basis of P_{D1} and the associated damages (Equation (5-16)) incurred over the 12-month period. Table E-7 contains this information for each fixed-time policy and it can be seen that policy D yields a P_{D1} of 54.4% at a total cost of 2352. On the other hand, if a variable-time policy is employed, Table 5-12 is entered and policy 4, which calls for sampling whenever the predicted chloride concentration exceeds 36 mg/ℓ, is selected. The resulting value of P_{D1} contained in Table E-1 is given as 86.4% with an associated total cost of 2574. Although the total cost given by policy D is 222 (or 9%) less than that given by policy 4, the detection requirement is not satisfied under the fixed-time policy. The variable-time policy does, in fact, meet the requirement.

A more drastic result is given as the P_{D1} specification is lowered to 50%. Table C-2 calls for selection of policy H with a fixed sampling interval of 23 days. The variable-time approach employs policy 6 (sampling when the predicted concentration exceeds 38 mg/ℓ). The comparison

of these two policies is given below for the 1974 water year.

	Policy 6	Policy H
P_{D1}	79.5%	11.4%
Total Cost	2396	127004

For the case in which the predictive model uses estimated streamflow inputs, Profile 4 is taken as the true chloride profile. It is assumed that Equation (5-18) now represents the damage function in effect. A comparison of the fixed and variable-time policies selected for required P_{D1} levels of 85% and 50% are given below. The data is taken from Tables E-10 and E-4, respectively.

	Variable-time	Fixed-Time
Policy	3	D
Required P_{D1}	85%	85%
Actual P_{D1}	95.8%	39.6%
Total Cost	2976	3115

	Variable-time	Fixed-Time
Policy	6	H
Required P_{D1}	50%	50%
Actual P_{D1}	81.3%	12.5%
Total Cost	2570	53946

Figures D-1 through D-12 of Appendix D provide a graphical representation of the detection percentage and total cost for both the fixed and variable-time sampling policies. Figures D-1 through D-4 show policy A to be the only fixed-time policy which yields a P_{D1} greater than 80% during the 1973 water year. At the same time, 5 variable-time policies (1,2,3,4, and 5) all detect at least 80% of the violations at a total cost below that for policy A. The same result holds during the 1974 water year. As each particular chloride profile and associated stream standard is considered, the variable-time policies consistently provide higher levels of violation detection at lower cost than do the

fixed-time policies. For example, at a stream standard of 55 mg/ℓ the first 7 variable-time policies all give lower total cost than any of the fixed-time policies except for one isolated instance. The one exception occurs in Profile 3 where policy D has a total cost slightly less than that for policy 1. On the other hand, policy 1 detects 96.9% of the violations during the 1974 water year, while policy D detects only 56.3%. The results of P_{D2} are even more dramatic, but because design of the fixed time policies is directed toward detection of continuous violation periods rather than individual daily violations, these differences will not be discussed in detail. However, each plot does contain the P_{D2} data for both policy types.

Analysis Using 1974-1975 Historical Data

It has been previously stated that the model prediction error retains the same distribution over the 1973 and 1974 water years. Therefore, the sample chloride profiles 1 and 3 are both based on a prediction error which is normally distributed with a mean of -2.77 and a standard deviation of 5.54. Similarly, profiles 2 and 4 are based on a normally distributed prediction error with a mean of -2.77 and a standard deviation of 5.81. The assumption that the errors remained the same from year to year were necessary due to a lack of any historical data during the 1974 water year. All attempts at securring any 74-75 data were unsuccessful until after the previous analysis was completed. However, subsequently a set of 26 field ovservations taken at station 6 were forwarded by Mr. David J. Danford of the Ohio Environmental Protection Agency. The data covered the period from January, 1975 through July, 1975 and are given in Table 5-15. Comparisons of model predictions vs. actual chloride observations over the 1974 water year (for both methods of streamflow input) are given in Tables 5-15 and 5-16.

In order to construct a more representative sample chloride profile for the 1974 water year, this historical

TABLE 5-15

PREDICTED VS. ACTUAL CHLORIDE CONC. USING U.S.G.S. FLOWS

JANUARY 1975 - JULY 1975

DATE	ACTUAL CONC. (MG/L)	PRED. CONC. (MG/L)	ERROR (MG/L)	DATE	ACTUAL CONC. (MG/L)	PRED. CONC. (MG/L)	ERROR (MG/L)
JAN. 8	38.	35.	-3.	MAY 15	38.	40.	2.
JAN. 14	26.	32.	6.	MAY 20	47.	42.	-5.
JAN. 22	53.	38.	-15.	MAY 28	43.	43.	0.
FEB. 11	45.	37.	-8.	JUN. 4	50.	43.	-7.
MAR. 5	30.	34.	4.	JUN. 10	36.	40.	4.
MAR. 25	30.	29.	-1.	JUN. 17	43.	37.	-6.
APR. 2	25.	32.	7.	JUN. 26	34.	44.	10.
APR. 7	33.	36.	3.	JUL. 8	25.	37.	12.
APR. 16	35.	40.	5.	JUL. 15	45.	43.	-2.
APR. 22	34.	40.	6.	JUL. 22	30.	40.	10.
MAY 8	36.	37.	1.	JUL. 29	48.	49.	1.

TABLE 5-16

PREDICTED VS. ACTUAL CHLORIDE CONC. USING PREDICTED FLOWS

JANUARY 1975 - JULY 1975

DATE	ACTUAL CONC. (MG/L)	PRED. CONC. (MG/L)	ERROR (MG/L)	DATE	ACTUAL CONC. (MG/L)	PRED. CONC. (MG/L)	ERROR (MG/L)
JAN. 8	38.	35.	-3.	MAY 15	38.	39.	1.
JAN. 14	26.	30.	4.	MAY 20	47.	42.	-5.
JAN. 22	53.	37.	-16.	MAY 28	43.	45.	2.
FEB. 11	45.	38.	-7.	JUN. 4	50.	43.	-7.
MAR. 5	30.	35.	5.	JUN. 10	36.	41.	5.
MAR. 25	30.	29.	-1.	JUN. 17	43.	37.	-6.
APR. 2	25.	31.	6.	JUN. 26	34.	47.	13.
APR. 7	33.	37.	4.	JUL. 8	25.	33.	8.
APR. 16	35.	42.	7.	JUL. 15	45.	43.	-2.
APR. 22	34.	40.	6.	JUL. 22	30.	43.	13.
MAY 8	36.	39.	3.	JUL. 29	48.	46.	-2.

data was compared to model predictions on the appropriate dates to arrive at a new estimate of the prediction error. The chi-square goodness-of-fit test was again used to test for normality of the error. The test results are given in Table 5-17 for predictions employing U.S.G.S. flows, and in Table 5-18 for predictions employing estimated flows. It can be seen from inspection of Tables 5-17 and 5-18 that the mean and standard deviation of the error did, in fact, shift. For the model predictions employing U.S.G.S flows, the mean error changed to 1.09 and the standard deviation to 6.63. For predictions using estimated flows the mean error changed to 1.27 and the standard deviation to 6.96. The goodness-of-fit tests contained in Tables 5-17 and 5-18 both justify acceptance of the normality assumption at probability levels above .95.

Two new hypothetical chloride profiles - 3B and 4B - were generated for the 1974 water year by the same method previously discussed. Profile 3B, generated from prediction errors normally distributed with $\mu = 1.09$ and $\sigma = 6.63$, is contained in Table F-9 of Appendix F. Profile 4B employs model errors which are normally distributed with $\mu = 1.27$ and $\sigma = 6.96$ and is given in Table F-10.

The optimum sampling policies selected for the 1974 water year are now re-evaluated on the basis of their performance against profiles 3B and 4B. The results are presented in Tables E-13 through E-15 of Appendix E. Looking first at the policies derived on the basis of U.S.G.S. flow data, Table E-13 gives a complete cost analysis for each of the 16 selected policies at a chloride standard of 45 mg/ℓ. The optimum policy was selected for various P_{D1} and P_{D2} requirements, and a comparison of the optimum policies over the 1973 and 1974 water years is given in Table 5-19.

TABLE 5-17

CHI-SQUARE GOODNESS OF FIT TEST FOR NORMALITY
OF MODEL ERROR USING U.S.G.S. STREAMFLOWS
(JAN. 1975 - JULY 1975)

$n = 22$, $\bar{X} = 1.09$, $\sigma = 6.63$

UPPER LIMIT OF INTERVAL	$X - \bar{X}$	STANDARD UNITS $Z = (X - \bar{X})/\sigma$	REL. FREQUENCY FROM $-\infty$ TO Z	CELL FREQUENCY	ABSOLUTE THEORETICAL FREQUENCY (f)	ACTUAL FREQUENCY (F)	F - f	$(F - f)^2/f$
-10.	-11.09	-1.67	.0475	.0475	1.0	1	0.0	.000
-5.	-6.09	-.92	.1788	.1313	2.9	3	.1	.003
0.	-1.09	-.16	.4364	.2576	5.7	5	-.7	.086
5.	3.91	.59	.7224	.2860	6.3	6	-.3	.014
10.	8.91	1.35	.9115	.1891	4.2	5	.8	.152
∞	∞	∞	1.0000	.0885	1.9	2	.1	.005
							total =	.260

Degrees of Freedom = 6 - 3 = 3

$\chi^2_{.950} = .35$ $\chi^2_{.975} = .22$

TABLE 5-18

CHI-SQUARE GOODNESS OF FIT TEST FOR NORMALITY
OF MODEL ERROR USING PREDICTED STREAMFLOWS
(JAN. 1975 - JULY 1975)

$n = 22$, $\bar{X} = 1.27$, $\sigma = 6.96$

UPPER LIMIT OF INTERVAL	$X - \bar{X}$	STANDARD UNITS $Z = (X - \bar{X})/\sigma$	REL. FREQUENCY FROM $-\infty$ TO Z	CELL FREQUENCY	ABSOLUTE THEORETICAL FREQUENCY (f)	ACTUAL FREQUENCY (F)	F - f	$(F - f)^2/f$
-10.	-11.27	-1.62	.0526	.0526	1.1	1	-.1	.009
-5.	-6.27	-.90	.1841	.1315	2.9	3	.1	.003
0.	-1.27	-.18	.4286	.2445	5.4	5	-.4	.030
5.	3.73	.54	.7054	.2768	6.1	6	-.1	.002
10.	8.73	1.26	.8962	.1908	4.2	5	.8	.152
∞	∞	∞	1.0000	.1038	2.3	2	-.3	.039
							total =	.235

Degrees of Freedom = 6 - 3 = 3

$\chi^2_{.950} = .35$ $\chi^2_{.975} = .22$

One immediate result of interest is that detection probabilities are generally higher for all policies in the second profile, producing lower associated costs than were previously obtained. This is due to the improved predictive accuracy of the model as estimated from the new historical data. For example, under the previous error distribution there is a .1725 probability of a stream standard violation ($C(t) \geq 45$ m/ℓ) given the model yields a concentration of 37 mg/ℓ .

$$\hat{p} = P\left[c(t) \geq 45 \mid \text{model} = 37\right] = P\left[\text{error} \leq -8\text{mg}/\ell\right] = .1725$$

If the true distribution has actually shifted to normal ($\mu = 1.09$, $\sigma = 6.63$) the true probability of violation is

$$P\left[C(t) \geq 45 \mid \text{model} = 37\right] = .085$$

In this case, the true probability of violation is approximately one-half that originally estimated, rendering a much more conserative sampling policy than was anticipated. Therefore, policies 3 and 4, which call for sampling if $\hat{p}$ exceeds .10 and .15 resepctively, require samples to be taken at times when the probability of violation is less than that specified for the policy. For this reason more violations are detected during the 1974 water year than previously shown for profile 3, resulting in a reduction in the overall policy cost originally estimated.

Examination of Tables 5-12 and 5-19 reveals that the true optimum policy during the 1974 water year does change from that originally derived. For example, for a required P_{D1} and P_{D2} less than 80% (ss = 45 mg/ℓ ; non-linear cost) policy 6 was optimum for both years, but in Table 5-19, policy 8 is shown to be the true optimum policy during 1974-1975. At a stream standard of 55 mg/ℓ and under both cost structures, it was assumed that none of the selected policies were able to detect 100% of the violations during the 1974 water year, but it can be seen that policy 1 does

TABLE 5-19

OPTIMUM POLICY VS. REQUIRED VIOLATION DETECTION (U.S.G.S. FLOWS)

REQUIRED DETECTION PERCENTAGE P_D	$C = 10*n + \Sigma(20(e^{t_i} - 1)/t_i)$						$C = 10*n + 50\Sigma\, t_i$					
	SS = 45 MG/L		SS = 50 MG/L		SS = 55 MG/L		SS = 45 MG/L		SS = 50 MG/L		SS = 55 MG/L	
	P_{D1}	P_{D2}	P_{D1}	P_{D2}	P_{D1}	P_{D2}	P_{D1}	P_{D2}	P_{D1}	P_{D2}	P_{D1}	P_{D2}
100%	2 (1)	2 (-)	1 (5)	1 (5)	1*	1*	2 (1)	2 (-)	1 (5)	1 (5)	1*	1*
99%	2 (1)	2 (1)	1 (5)	1 (5)	1*	1*	2 (1)	2 (1)	1 (5)	1 (5)	1*	1*
95%	3 (5)	4 (6)	2 (6)	3 (5)	1*	1*	3 (5)	3 (6)	2 (6)	3 (5)	1*	1*
90%	4 (7)	6 (7)	3 (7)	6 (7)	2*	2*	3 (6)	3 (6)	3 (7)	6 (7)	2*	2*
80%	6 (8)	6 (8)	4 (7)	7*	3 (2)	5 (2)	3 (6)	3 (6)	4 (7)	6 (7)	3 (2)	5 (2)
70%	6 (8)	6 (8)	7*	7*	6 (4)	6 (4)	3 (6)	3 (6)	6 (7)	6 (7)	5 (4)	5 (4)
60%	6 (8)	6 (8)	7 (10)	7 (10)	6 (8)	6 (8)	3 (6)	3 (6)	6 (10)	6 (10)	5 (8)	5 (8)
50%	6 (8)	6 (8)	7 (11)	7 (11)	6 (9)	6 (9)	3 (6)	3 (6)	6 (11)	6 (11)	5 (9)	5 (9)
25%	6 (8)	6 (8)	7 (11)	7 (11)	6 (9)	6 (9)	3 (6)	3 (6)	6 (11)	6 (11)	5 (9)	5 (9)
10%	6 (8)	6 (8)	7 (11)	7 (11)	6 (9)	6 (9)	3 (6)	3 (6)	6 (11)	6 (11)	5 (9)	5 (9)

* INDICATES OPTIMUM FOR BOTH THE 1973 AND 1974 WATER YEARS
( ) INDICATES OPTIMUM POLICY FOR 1974 WATER YEAR
\- INDICATES OPTIMUM POLICY IS NOT AVAILABLE

TABLE 5-20

OPTIMUM POLICY VS. REQUIRED VIOLATION DETECTION (PREDICTED FLOWS)

REQUIRED DETECTION PERCENTAGE P_D	$C = 10*n + \Sigma(20(e^{t_i} - 1)/t_i) + 2*k$						$C = 10*n + 50\Sigma\, t_i + 2*k$					
	SS = 45 MG/L		SS = 50 MG/L		SS = 55 MG/L		SS = 45 MG/L		SS = 50 MG/L		SS = 55 MG/L	
	P_{D1}	P_{D2}	P_{D1}	P_{D2}	P_{D1}	P_{D2}	P_{D1}	P_{D2}	P_{D1}	P_{D2}	P_{D1}	P_{D2}
100%	1 (-)	1 (-)	2 (1)	2 (1)	1 (-)	1 (-)	1 (-)	1 (-)	2 (1)	2 (1)	1 (-)	1 (-)
99%	1 (-)	1*	2 (1)	2 (1)	1 (-)	1 (-)	1 (-)	1*	2 (1)	2 (1)	1 (-)	1 (-)
95%	2 (1)	3 (4)	2 (1)	4 (2)	4 (-)	4 (2)	2 (1)	3 (4)	2 (1)	4 (2)	4 (-)	4 (2)
90%	3 (4)	5 (6)	3 (2)	7 (3)	5 (3)	5 (3)	3 (4)	5 (6)	3 (2)	7 (3)	4 (3)	4 (3)
80%	5 (6)	6*	6 (3)	7 (6)	5 (3)	6 (3)	5 (6)	5 (6)	6 (3)	7 (6)	4 (3)	4 (3)
70%	6*	6*	7 (5)	7*	6 (4)	6 (4)	5 (6)	5 (6)	7 (5)	7 (6)	4 (3)	4 (3)
60%	6*	6*	7*	7*	6 (5)	6 (5)	5 (6)	5 (6)	7 (6)	7 (6)	4 (5)	4 (5)
50%	6*	6*	7*	7*	6 (5)	6*	5 (6)	5 (6)	7 (6)	7 (6)	4 (5)	4 (6)
25%	6*	6*	7*	7*	6 (8)	6 (8)	5 (6)	5 (6)	7 (6)	7 (6)	4 (8)	4 (8)
10%	6*	6*	7*	7*	6 (8)	6 (8)	5 (6)	5 (6)	7 (6)	7 (6)	4 (8)	4 (8)

* INDICATES OPTIMUM FOR BOTH THE 1973 AND 1974 WATER YEARS
( ) INDICATES OPTIMUM POLICY FOR 1974 WATER YEAR
\- INDICATES OPTIMUM POLICY IS NOT AVAILABLE

satisy this extreme P_{D1} and P_{D2} requirement. For linear costs and at a standard of 45 mg/ℓ , the optimum policy does not change from that originally derived.

Analysis of profile 4B (using estimated flows) reveals that the true optimum policy is selected more frequently than was originally anticipated. A comparison of Tables 5-13 and 5-20 shows that policy 1 does not, in fact, detect 100% of all violations of the 45 mg/ℓ stream standard. Also at levels of P_{D1} and P_{D2} below 80%, the selected policy (6) is actually the true optimum for the 1974 water year. Previously it was shown that policy 7 was optimum for this period. At a stream standard of 50 mg/ℓ , the true optimum policy is again selected at P_{D1} and P_{D2} < 70%. Under the linear cost structure, there is almost no change from the first results given for a standard of 50 mg/ℓ .

A re-examination of the performance of the fixed-time policies A through P was completed and the results are also presented in Tables E-16 and E-18. Again, there is a sharp drop in detection effectiveness from the variable-time policies to the fixed-time policies according to this second analysis. Only policy A produces a P_{D1} and P_{D2} above 78% over profiles 3B and 4B, but at a higher estimated cost than that derived for several of the variable-time policies which yield a P_{D1} and P_{D2} above 78%.

Conclusions

Although the given method of policy design and selection does not insure that the optimum policy for future sampling is always obtained, the analysis does show that inefficient policies are always avoided and often the optimum policy is used. It has been asserted that if the selection process were based on several years of policy evaluation, the number of cases in which the optimum policy is employed would significantly increase. This technique provides a practical means of incorporating critical information into the policy design, e.g., point-source loading distributions,

streamflow variability, background pollution levels, physical characteristics of the receiving water and the distribution of the prediction error. At the same time, the process of policy selection is given a sensitivity to the exact definition of a violation, existing damage functions, as well as current levels of the particular stream standard. This approach to monitoring a receiving water attempts to capitalize on known information regarding those factors which most directly effect pollution levels. In doing so the sampling process avoids excessive observations during times when the condition of the stream is within acceptable levels, thereby providing an efficient allocation of available sampling resources.

The investigation makes use of historical data from the waste effluents, background levels and monitored position to calibrate the predictive model. It is suggested that if such data is not available from outside sources, some additional field sampling must be established to continuously up-date the estimators of the above conditions. Data received directly through the monitoring program will center around high chloride concentrations, but the model should also be occasionally checked for accuracy at times associated with low chloride predictions. Similarly, field samples from the point-sources and at the upstream boundary would insure that the model incorporates current information on all chloride sources. The specific format of this parallel sampling, although not considered in the present research, must depend upon the judgements and objectives of the policy designer.

If the approach presented here is to be successful, careful consideration must be given to the criteria used to evaluate the selected sampling policies. It has been shown that the optimum policy may vary with the particular damage function in effect, illustrating a need for extensive examination of all costs which may arise in policy application.

It is important to emphasize that the cost functions employed in the previous analysis are not necessarily representative of those in effect for the Great Miami River. Because of a lack of specific information concerning samling costs and stream damages, the hypothesized functions are able to provide a conceptual aid for application of the design methodology. However, in actual practice a complete study of available cost information is necessary to derive accurate representations of existing damage functions.

As a means of eliminating any inappropriate conclusions resulting from the use of these subjective cost functions, an alternative method of policy evaluation is suggested. Figure 5-7 shows a plot of P_{D1} (detection percentage) vs. the total number of required samples for the 1973 water year (using U.S.G.S. flow data). The information contained in Figure 5-7 may be used to select the optimum policy when detection delay costs are either unknown or insignificant in comparison to sampling costs. Alternatively, it may be helpful to select the optimum policy from Figure 5-7 and to subsequently examine the average delay to violation detection associated with that policy. Figure 5-8 provides a plot of average detection delay vs. percent of violations detected for each of the 16 selected policies. Knowledge of the average detection delay may then be incorporated into the process of policy selection at the discretion of the designer. For example, suppose the optimum policy is sought for $P_{D1} \geq 80\%$. Using the required sample size as the initial criterion for policy cost, Figure 5-7 is entered and Policy 6 is selected as optimum. The policy designer then enters Figure 5-8 and finds that this policy yields an average detection delay of .29 days. This information can then be given appropriate weight in the final decision as to the actual effectiveness of Policy 6. Obviously, the optimum policy for any given system will depend upon the relative magnitude of sampling costs versus costs arising from violation detection delays.

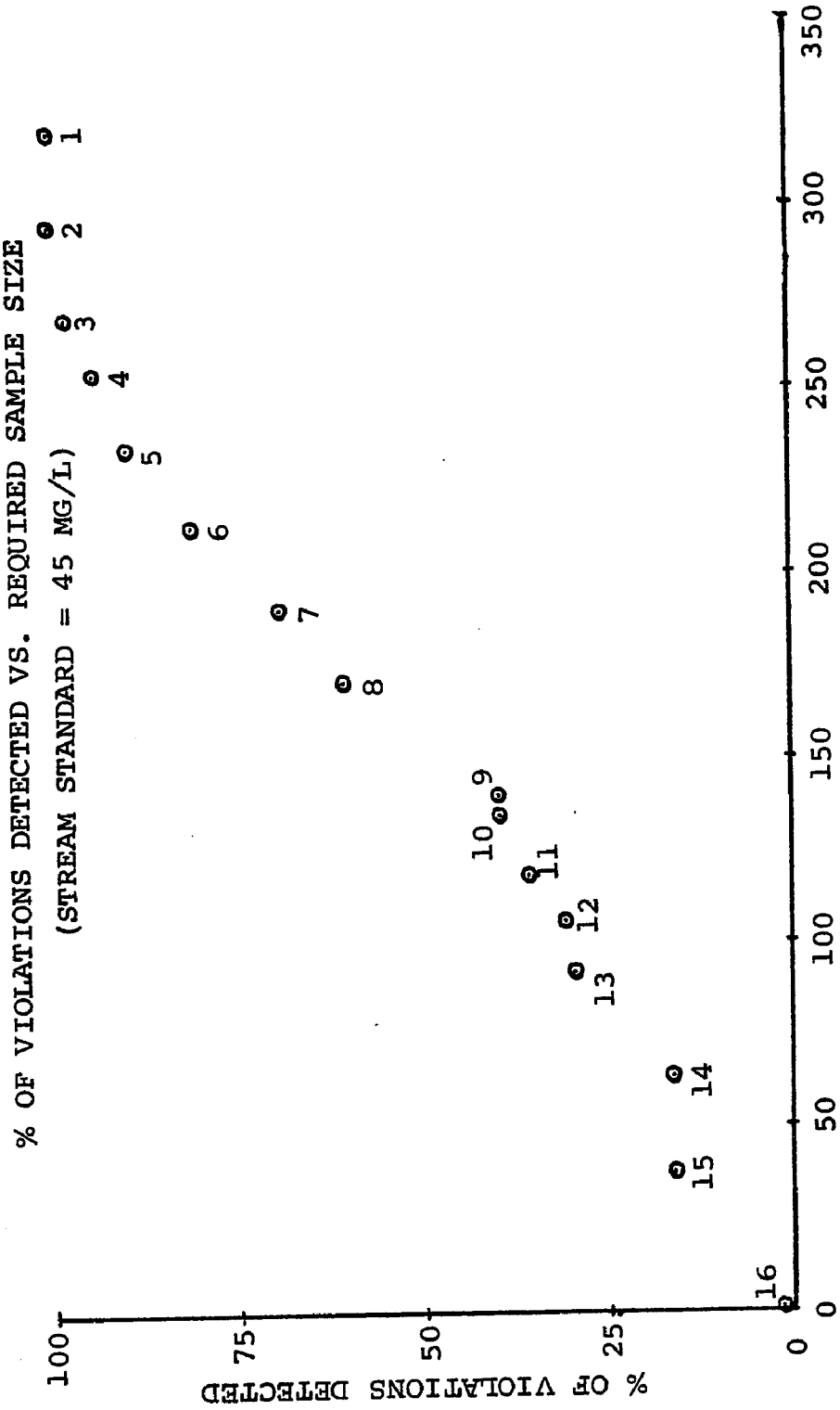

% OF VIOLATIONS DETECTED VS. REQUIRED SAMPLE SIZE
(STREAM STANDARD = 45 MG/L)
% OF VIOLATIONS DETECTED
100
75
50
25
0
0
50
100
150
200
250
300
350
REQUIRED SAMPLE SIZE
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16

FIGURE 5-7

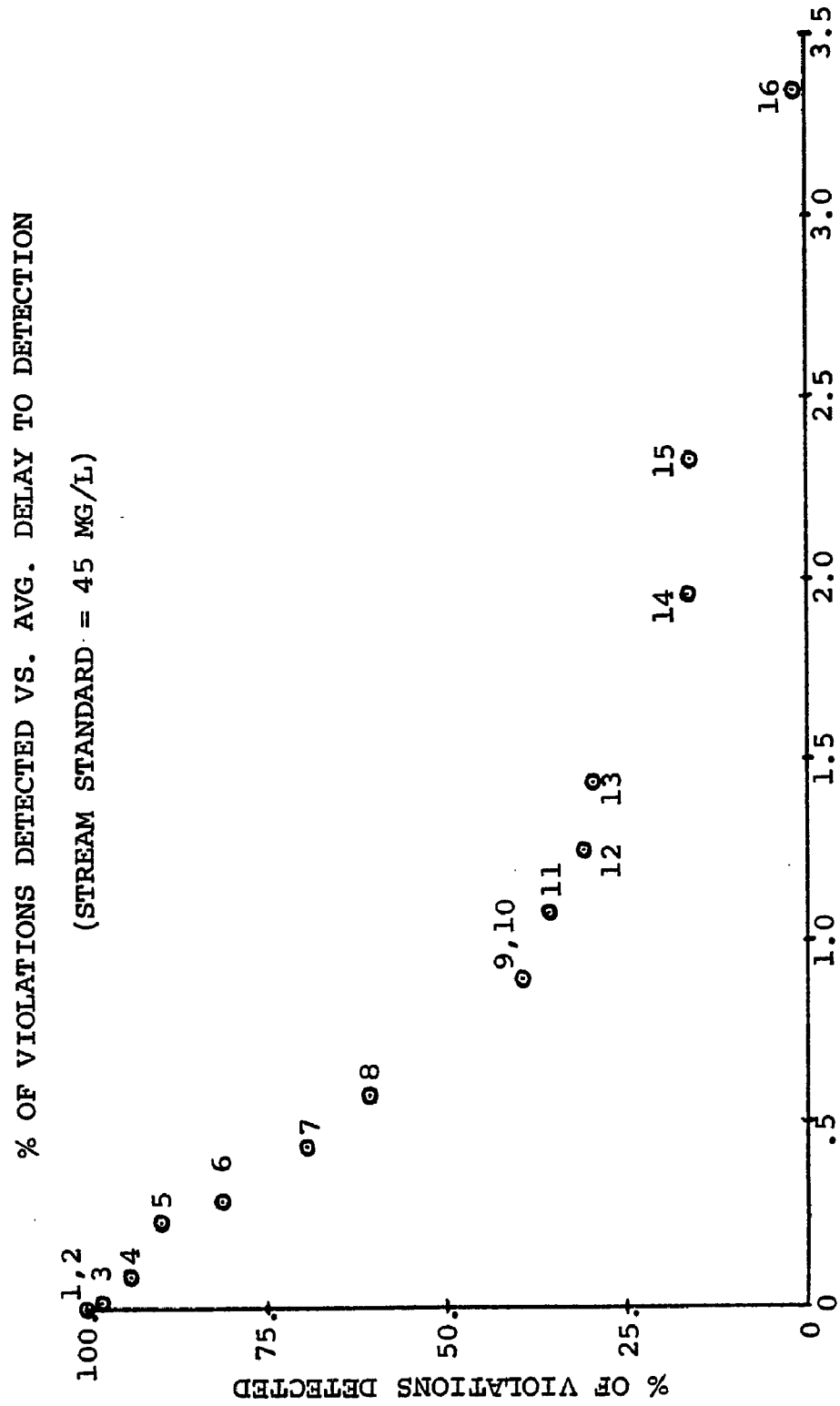

% OF VIOLATIONS DETECTED VS. AVG. DELAY TO DETECTION
(STREAM STANDARD = 45 MG/L)
% OF VIOLATIONS DETECTED
100
75
50
25
0
0
.5
1.0
1.5
2.0
2.5
3.0
3.5
AVG. DELAY TO DETECTION (DAYS)
1,2
3
4
5
6
7
8
9,10
11
12
13
14
15
16

FIGURE 5-8

Finally, it is necessary to explicitly define what constitutes a stream standard violation in any given system. A violation may be a function of time spent above a given threshold, the magnitude above the threshold, the average pollution level over a specified time period, or any one of several other definitions. Any regulatory agency attempting to implement this methodology must perform a complete analysis of violation interpretation if effective results are to be achieved. On the other hand, if sufficient pre-planning is devoted to assessment of system damages and definition of stream standard violations, it is felt that the above approach can provide a realistic and practical method of allocating sampling resources in an effective monitoring program.

Recommendations

It is difficult to generalize as to which of the selected policies would be most effective in application to a different receiving water system. Each system must be viewed as a unique problem containing a unique set of pollution sources and physical characteristics. Therefore, the optimum policy for one system may be quite different from the optimum policy for some other system. However, an interestigg aspect of the preceding analysis is the consistent effectiveness and efficiency associated with Policies 5, 6, and 7. In a high percentage of cases in which the P_{D1} and P_{D2} requirements are below 80%, the optimum policy is seen to be generally distributed among these three policies. In these cases the decrease in cost associated with less frequent sampling (than that required for Policies 1-4) offsets any additional costs incurred due to increased violation detection delays. At P_D levels above 80%, the more rigid detection constraint forces the optimum away from the overall minimum-cost policy contained in the set of selected policies. In this P_D region

it may be said that a regulatory agency must "pay" for increased detection capabilities. Although in the preceding study the percent increase in overall cost for more intensive sampling is not always high, it should be clear that this is only a reflection of the particular cost functions employed here. For example, a different cost function may create a large discrepancy between the cost of the optimum policy for $P_D \geq 90\%$ and the cost for $P_D < 80\%$. This fact must be understood in the process of setting the P_D requirement. Therefore it is critical that careful consideration be given to the relationship between total cost and required violation detection for a given receiving water system.

A final comment is also given to a point previously discussed at the conclusion of Chapter IV. It has been stated that the sampling policy design did not make use of the autocorrelation function for the chloride data. Further research should certainly be devoted to the design of sequential sampling policies which incorporate information on the statistical dependence between pollution observations.

APPENDIX A

Streamflow Prediction Development

When daily streamflow observations cannot be directly obtained by means of Telemark gaging stations , it becomes necessary to estimate this information required for input to the stream model. The method of prediction employs past runoff records for a given watershed to arrive at a typical total flow-recession curve (Linsley, Kohler, Paulhus, 1958). Sets of runoff observations taken several days after a peak flood are plotted and superimposed to arrive at a base-flow recession or groundwater depletion curve, as shown in Fig. A-1. The first several days following a peak flow are eliminated in the analysis to insure that the curve will likely represent only base-flow recession rather than total-flow recession. Once the individual recession curves have been superimposed and a groundwater depletion curve has been "pieced together", the result is used in deriving a groundwater depletion equation as discussed below. Runoff data which includes the first several days following a peak flood is then used to develop an equation describing total-flow recession. This equation becomes the model for predicting runoff values above base-flow levels.

In the case of the Great Miami River, U.S.G.S. runoff records at Hamilton, Ohio for the years 1965-1975 were examined and twelve depletion curves were selected for deriving the recession equations. The list of curves is given in Table A-1. After constructing the groundwater-depletion curve shown in Fig. A-1, runoff values were read from the curve at 4-day time intervals and the data was plotted on semi-log paper as shown in Fig. A-2. A linear regression was then performed to arrive at a straight-line fit of the resulting data. The groundwater-depletion curve may be described by a characteristic equation

$$Q_{24} = Q_0 C_r$$

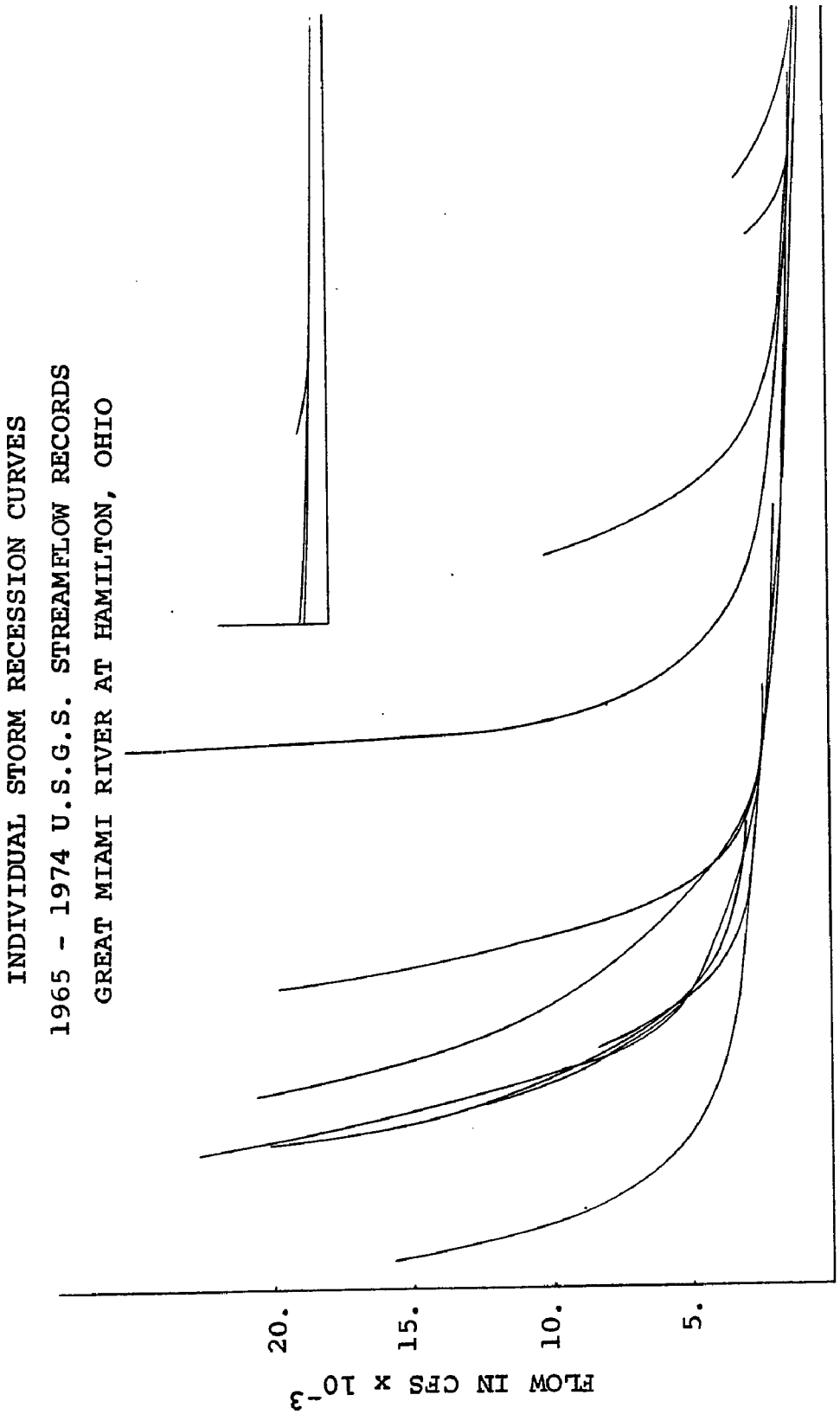

INDIVIDUAL STORM RECESSION CURVES
1965 - 1974 U.S.G.S. STREAMFLOW RECORDS
GREAT MIAMI RIVER AT HAMILTON, OHIO
FLOW IN CFS x 10^-3
20.
15.
10.
5.
TIME IN DAYS

FIGURE A-1

TABLE A-1

INDIVIDUAL STORM RECESSION CURVES

1965 - 1974 U.S.G.S. STREAMFLOW RECORDS

GREAT MIAMI RIVER AT HAMILTON, OHIO

DATE	MAX. RUNOFF (CFS)	MIN. RUNOFF (CFS)
10/23/65 - 11/07/65	3100.	652.
1/06/66 - 1/31/66	10200.	664.
2/11/66 - 2/28/66	20800.	1700.
12/10/66 - 1/19/67	30700.	973.
5/15/67 - 5/28/67	20300.	2500.
12/29/68 - 1/16/69	19800.	1510.
2/09/69 - 3/08/69	15800.	1850.
1/04/73 - 1/18/73	8500.	2340.
4/09/74 - 4/21/74	14500.	2950.
6/23/74 - 8/02/74	33400.	655.
9/29/74 - 10/14/74	2660.	784.

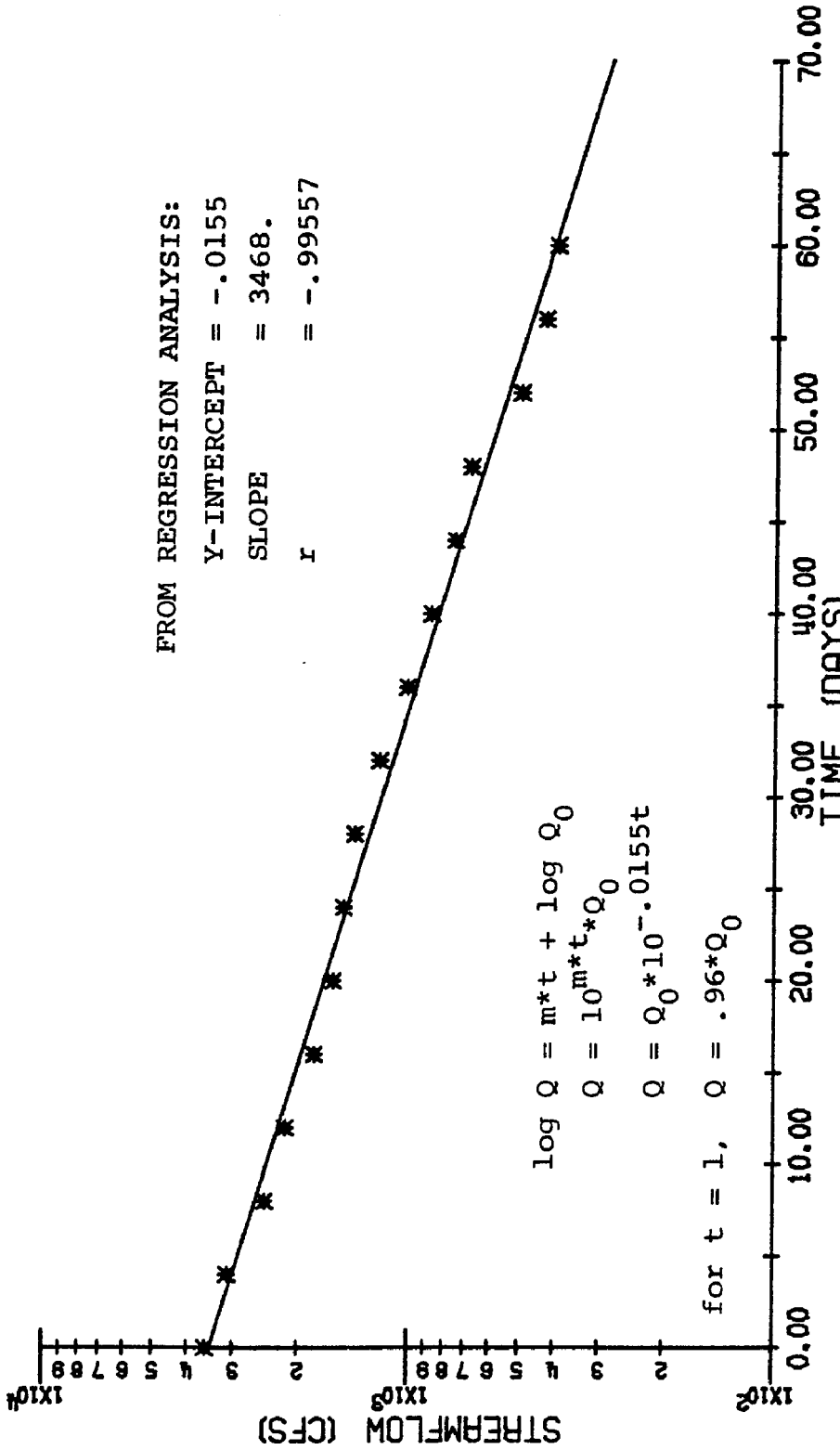

GROUNDWATER DEPLETION CURVE FOR GREAT MIAMI RIVER
FROM REGRESSION ANALYSIS:
Y-INTERCEPT = -.0155
SLOPE = 3468.
r = -.99557
log Q = m*t + log Q0
Q = 10^m*t*Q0
Q = Q0*10^-.0155t
for t = 1, Q = .96*Q0
STREAMFLOW (CFS)
TIME (DAYS)
0.00
10.00
20.00
30.00
40.00
50.00
60.00
70.00

FIGURE A-2

where Q_{24} represents the streamflow tomorrow, Q_0 represents the streamflow today, and C_r is a depletion constant less than unity. To obtain an estimate of this depletion constant, the equation describing the relationship of Q_{24} and t is used:

$$\log Q_{24} = .0155t + 3.5401 \qquad \text{(A-1)}$$

$$Q_{24} = 10^{-.0155t + 3.5401}$$

$$Q_{24} = 10^{-.0155t} \cdot 10^{3.5401}$$

(A-1) is used to obtain the flow value t days subsequent to a flow of $10^{3.5527}$ cfs, assuming base-flow recession. However, the equation can be generalized to solve for the flow one day subsequent to any given level by substituting Q_0 for $10^{3.5527}$ and setting t = 1. The result is given by:

$$Q_{24} = 10^{-.0159t} \cdot 10^{3.5527}$$

$$= 10^{-.0159} \cdot Q_0$$

$$Q_{24} = .965Q_0 \qquad \text{(A-2)}$$

(A-2) may now be used to predict daily streamflows once a value of Q_0 has been measured, given the condition of base-flow recession.

When attempting to estimate daily streamflows which include direct runoff as well as base flow, a slightly different approach must be taken. Again, data on the twelve recession curves selected previously from U.S.G.S. records on the Great Miami River is employed in developing the predictive model. However, now runoff values recorded after a peak flood are included in the analysis. Initially a plot is constructed of Q_0, vs. Q_{24} as shown in Fig. A-3. Q_0 represents the current runoff level in cfs and Q_{24} represents the runoff level 24 hours later as taken from the twelve recession curves. The data is bounded on the right by a 45° straight line which represents the slowest recession possible. Similarly, the data is bounded on the left by a line representing the fastest rate of recession: depletion

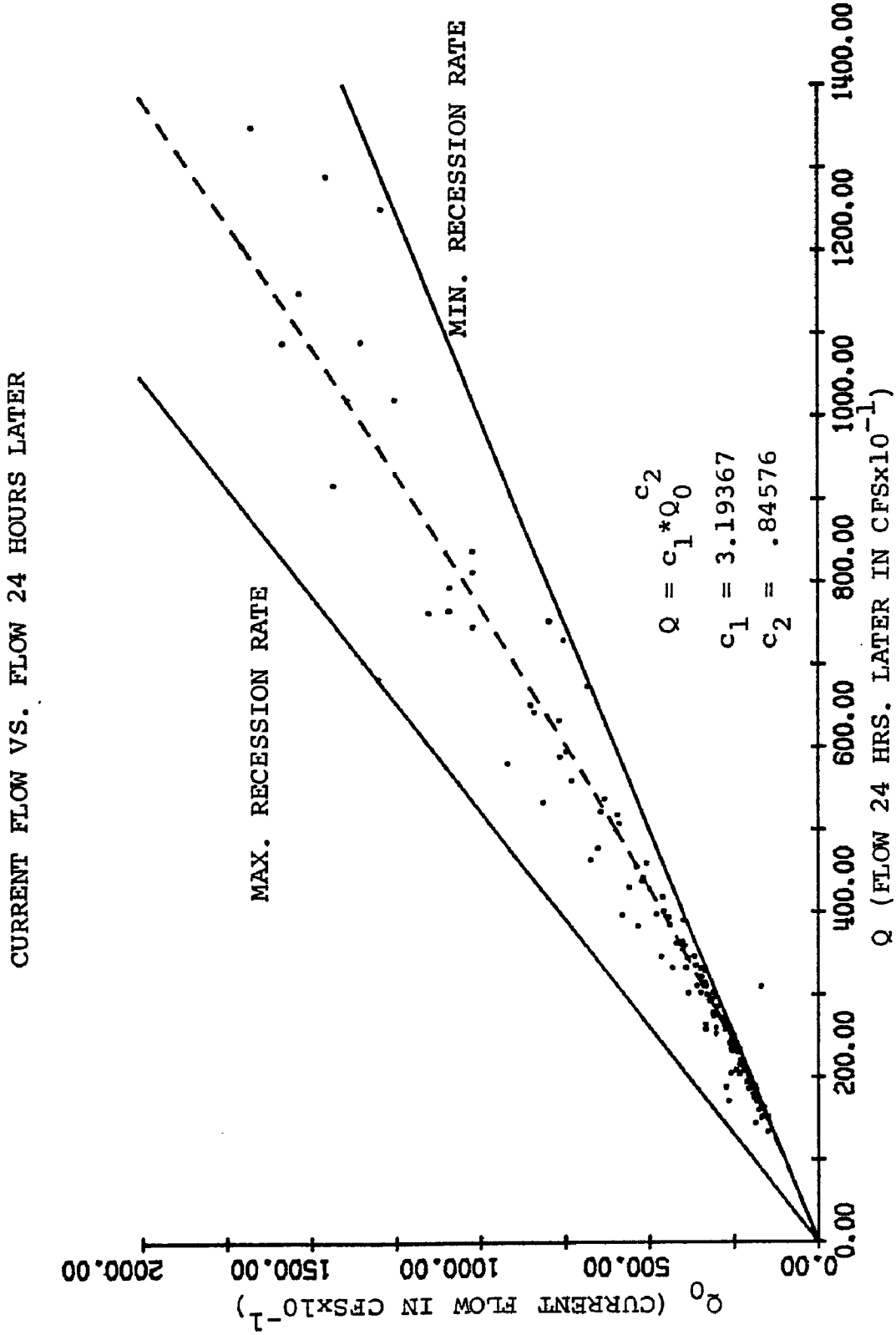

CURRENT FLOW VS. FLOW 24 HOURS LATER
MAX. RECESSION RATE
MIN. RECESSION RATE
$Q = c_1 * Q_0^{c_2}$
c_1 = 3.19367
c_2 = .84576
Q_0 (CURRENT FLOW IN CFSx10^{-1})
Q (FLOW 24 HRS. LATER IN CFSx10^{-1})
0.00
500.00
1000.00
1500.00
2000.00
200.00
400.00
600.00
800.00
1000.00
1200.00
1400.00

FIGURE A-3

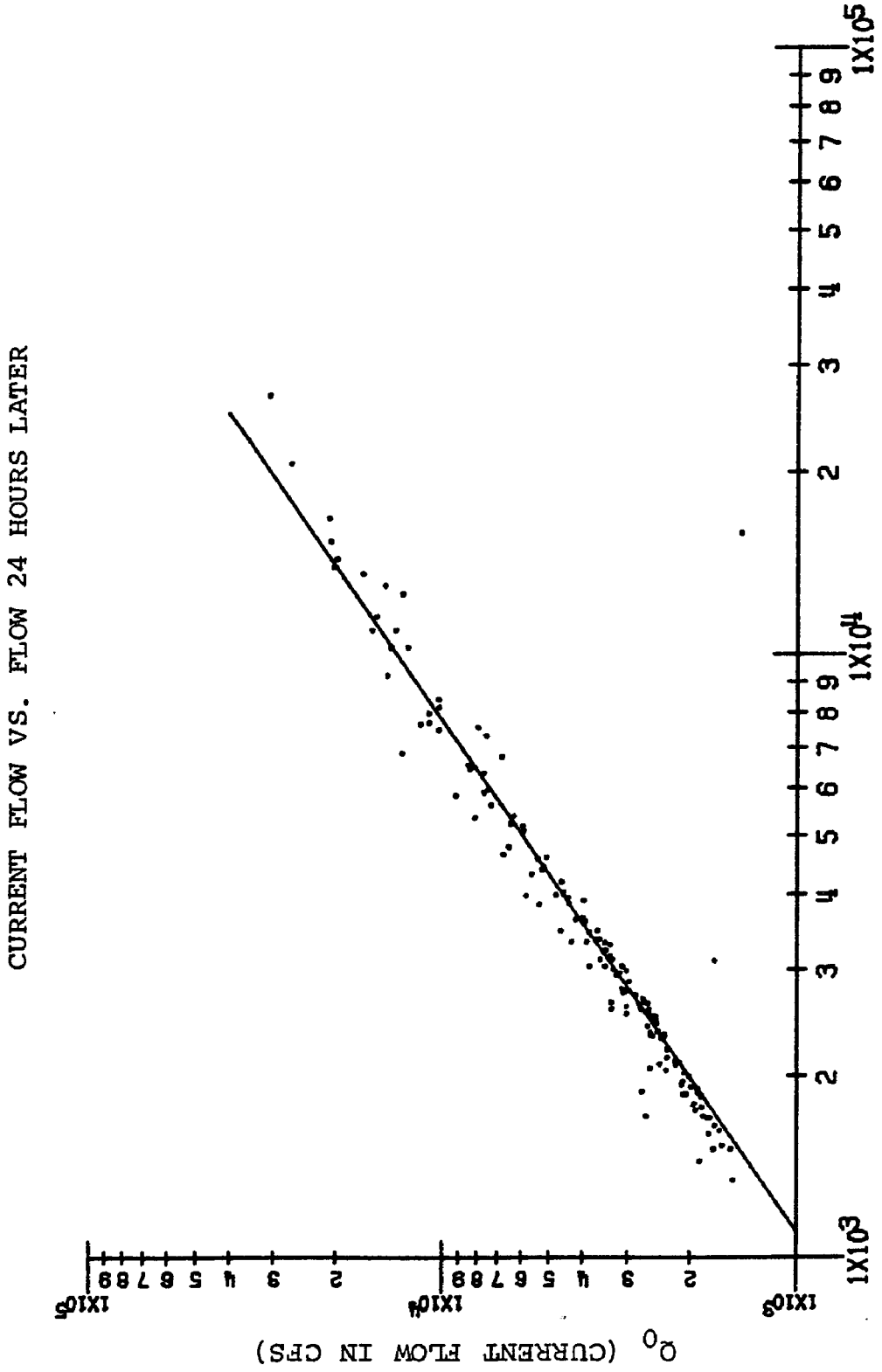

CURRENT FLOW VS. FLOW 24 HOURS LATER
Q_O (CURRENT FLOW IN CFS)
1X10³
1X10⁴
1X10⁵
Q (FLOW 24 HRS. LATER IN CFS)

FIGURE A-4

from direct runoff. Actual rates of recession will lie within these two boundaries. In order to fit a model to the data in Fig. A-3, a new plot was constructed of log Q_0 vs. log Q_{24} (Fig. A-4). Again, a linear regression was performed to obtain a least-squares fit of the transformed coordinate pairs. The resulting slope, y-intercept and correlation coefficient for the least-squares line are given below.

y intept. = .50429

m = .84576

r = .93566

The equation of the fitted line is

$$\log Q_{24} = .84576 \log Q_0 = .50429 \qquad \text{(A-3)}$$

Taking the antilog of each side of (A-3) leads to the following result:

$$Q_{24} = 3.194 Q_0^{.846} \qquad \text{(A-4)}$$

Equation (A-4) now becomes the model for predicting stream flows during recessions which include direct as well as base-flow runoff.

At this point it is necessary to determine the respective streamflow ranges over which Equations (A-2) and (A-4) apply, i.e., the level at which total recession is described by the characteristics of base-flow alone rather than base-flow and direct runoff combined. Because the rate of recession which includes direct runoff is greater than that of runoff including base-flow alone, Equation (A-4) can only be applied for levels of Q_0 which give a value of Q_{24} less than that derived from Equation (A-2). The critical value of Q_0 is that which gives the same result for both equations.

(Base-Flow) $\quad Q = .965\ Q_0 \qquad \text{(A-2)}$

(Base and Direct) $\quad Q_{24} = 3.194\ Q_0^{.846} \qquad \text{(A-4)}$

Substracting Equation (A-4) from Equation (A-2) gives:

$$0 = .965 Q_0 - 3.194\ Q_0^{.846}$$

$$.946 Q_0 = 3.194 Q_0^{.846} \qquad \text{(A-5)}$$

By trial and error it was found that Q_0 = 2360 cfs satisfied the relationship given in (A-5). At runoff levels less than 2360 cfs, the rate of recession given by (A-4) is slower than the rate given by (A-2). Because recession due to base-flow alone is the slowest recession possible (A-4) is used to predict streamflows above 2360 cfs, i.e., streamflows which include direct runoff. At levels below 2360 cfs, (A-2) is used.

The capability of equations (A-2) and A-4) in predicting streamflows is best illustrated by examining the resulting errors in estimated runoffs over the period from October 1973 to October 1975. Tables A-2 and A-4 contain the U.S.G.S. average daily runoff data on the Great Miami River at Hamilton, Ohio for the water years 1973-74 and 1974-1975 respectively. Tables A-3 and A-5 contain the streamflow data obtained either by model prediction or by direct measurement as discussed in Chapter 5. Those generated through model prediction are indicated by means of an asterisk to the right of the flow value. The resulting errors in estimation were compiled for both years; the mean and standard deviation of the percent error are given below.

	1973-1974	1974-1975
μ_{ERROR} =	3.65%	.96%
σ_{ERROR} =	22.69%	24.44%

Although the mean % error in streamflow prediction is quite low, the standard error of approximately 23% is high enough to justify a search for more sophisticated methods of estimating future flow levels. In fact, such techniques are available in the form of complex computer algorithms which keep track of the physical characteristics of the watershed - infiltration capacity, soil moisture content, temperature, residual snowfall, etc. This method of prediction would be expected to provide better estimates of daily runoff than that currently used in the present

research. However, the objectives of the investigation herein are served adequately by the more crude estimators of (A-2) and (A-4). If time and money are sufficient, it is suggested that the sampling policy design include these sophisticated techniques of runoff estimation.

TABLE A-2

1973-74 U.S.G.S. STREAMFLOW MEASUREMENTS: GREAT MIAMI RIVER AT HAMILTON, OHIO

(DISCHARGE IN CUBIC FEET PER SECOND)

DAY	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP
1	1530.	1510.	6330.	5670.	6280.	4930.	11500.	3070.	2270.	2490.	706.	5460.
2	3380.	1650.	4770.	4440.	5550.	5630.	21400.	2550.	1960.	2440.	655.	3140.
3	2060.	1530.	4030.	4130.	5040.	4810.	12200.	2990.	1760.	2090.	886.	4980.
4	2860.	1330.	3840.	3990.	4550.	4150.	14600.	2510.	1660.	1860.	1110.	4330.
5	2890.	1280.	3920.	3600.	4030.	4350.	14500.	2230.	1570.	1770.	943.	3460.
6	1730.	1230.	3640.	3420.	4130.	5960.	12900.	2180.	1600.	1600.	813.	2550.
7	1500.	1090.	3310.	3260.	8360.	5590.	9010.	2120.	1540.	1630.	778.	1890.
8	1410.	1060.	2960.	2940.	5980.	4660.	14100.	2130.	1510.	1740.	813.	1520.
9	1320.	1040.	2740.	2750.	4800.	4340.	14500.	2860.	1460.	1590.	825.	1320.
10	1180.	1010.	2640.	3140.	3990.	3770.	12900.	2300.	1360.	1460.	977.	1260.
11	1080.	949.	2550.	3180.	3680.	3520.	12500.	2130.	1340.	1600.	794.	1250.
12	1100.	913.	2410.	2780.	3510.	3560.	10200.	2450.	1310.	1520.	1180.	6130.
13	1060.	945.	2410.	2340.	3480.	3170.	7460.	2620.	1240.	1360.	900.	3770.
14	1220.	947.	2620.	2390.	3920.	3040.	5950.	2660.	1170.	1190.	888.	4220.
15	1160.	1360.	2790.	4790.	4550.	2890.	5180.	2350.	1400.	1120.	952.	2700.
16	1160.	2310.	2630.	7790.	4010.	3070.	4420.	2130.	1400.	1170.	933.	2040.
17	1100.	1570.	2320.	11900.	3510.	3040.	3940.	2270.	1210.	1040.	1990.	1670.
18	1000.	1400.	2100.	14200.	3110.	3270.	3600.	8920.	1170.	1000.	1790.	1460.
19	956.	1290.	2000.	20100.	3510.	3020.	3360.	5370.	1100.	902.	1140.	1300.
20	930.	1220.	3070.	26500.	5910.	2770.	3150.	4370.	1170.	926.	1180.	1270.
21	936.	1210.	3560.	28400.	6950.	3340.	2950.	3180.	1220.	849.	1030.	1360.
22	969.	1230.	2770.	23100.	8020.	3230.	3190.	2920.	7230.	784.	879.	1170.
23	919.	1130.	2430.	20100.	11300.	2840.	3520.	2940.	33400.	812.	822.	996.
24	923.	2800.	2390.	17100.	8760.	2730.	3060.	3620.	13000.	861.	874.	979.
25	909.	9100.	3980.	12100.	5880.	2710.	2810.	3290.	6830.	824.	770.	947.
26	892.	13400.	11300.	9220.	4630.	2650.	2660.	2680.	6740.	789.	674.	927.
27	872.	13900.	19400.	12800.	4140.	2570.	2480.	2270.	4640.	784.	679.	911.
28	846.	18100.	16500.	13400.	4280.	2700.	2300.	2030.	3470.	754.	2040.	1060.
29	852.	11500.	12700.	11100.		3030.	2250.	2130.	3030.	714.	5300.	2660.
30	1070.	8870.	9520.	9320.		11400.	2510.	2560.	2530.	734.	4250.	1710.
31	1130.		7250.	7530.		15000.		2480.		729.	3780.	

TABLE A-3

1973-74 PREDICTED STREAMFLOW MEASUREMENTS: GREAT MIAMI RIVER AT HAMILTON, OHIO

(DISCHARGE IN CUBIC FEET PER SECOND)

DAY	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP
1	1530.	1510.	*6846.	5670.	*5481.	4930.	11500.	3070.	2270.	*2782.	* 567.	5460.
2	3380.	1650.	*5600.	*4774.	5550.	5630.	21400.	2550.	*2188.	*2615.	* 547.	3140.
3	2060.	*1591.	*4724.	*4128.	5040.	4810.	12200.	2990.	*2110.	*2481.	886.	4980.
4	2860.	*1533.	3840.	*3651.	*4322.	*4154.	14600.	2510.	*2034.	1860.	1110.	4330.
5	2890.	1280.	3920.	*3290.	*3795.	4350.	14500.	*2397.	1570.	1770.	*1070.	3460.
6	*2700.	1230.	*3494.	*3013.	4130.	5960.	*10564.	*2305.	1600.	*1706.	*1032.	*3144.
7	1500.	*1186.	*3170.	*2797.	8360.	*4980.	* 8081.	*2222.	*1542.	*1645.	* 994.	*2899.
8	1410.	*1143.	*2920.	*2626.	*6630.	*4278.	14100.	2130.	*1487.	*1586.	* 959.	*2708.
9	*1359.	1040.	*2724.	2750.	*5450.	*3762.	14500.	2860.	*1433.	*1529.	* 924.	1320.
10	*1310.	1010.	*2568.	3140.	*4617.	*3375.	*10564.	2300.	*1382.	*1474.	* 891.	1260.
11	*1263.	* 974.	*2444.	3180.	*4013.	3520.	* 8081.	*2217.	1340.	*1420.	794.	1250.
12	*1218.	* 939.	*2343.	*2928.	*3564.	3560.	* 6443.	2450.	1310.	*1369.	1180.	6130.
13	1060.	* 905.	2410.	*2730.	*3224.	*3221.	*5319.	2620.	*1263.	*1320.	*1138.	3770.
14	1220.	* 872.	2620.	*2573.	*2962.	*2959.	*4523.	*2485.	*1217.	*1273.	*1097.	4220.
15	*1176.	1360.	*2485.	*2447.	*2757.	2890.	*3944.	2350.	1400.	*1227.	*1057.	*3719.
16	*1134.	2310.	*2377.	*2346.	*2594.	3070.	*3512.	2130.	1400.	*1183.	*1019.	*3342.
17	*1093.	*2227.	*2291.	*2262.	*2465.	3040.	*3184.	2270.	*1350.	*1140.	1990.	*3053.
18	*1054.	*2147.	*2209.	14200.	*2360.	*2818.	*2931.	8920.	*1301.	*1099.	1790.	*2829.
19	*1016.	*2069.	*2129.	20100.	3510.	*2643.	*2732.	5370.	*1254.	*1059.	*1726.	*2651.
20	* 979.	*1995.	3070.	26500.	5910.	*2504.	*2575.	4370.	1170.	926.	*1663.	*2510.
21	* 944.	1210.	3560.	28400.	*4945.	3340.	*2449.	*3831.	1220.	849.	*1604.	1360.
22	* 910.	1230.	*3221.	23100.	8020.	3230.	3190.	2920.	7230.	* 818.	*1546.	1170.
23	* 877.	*1186.	*2959.	20100.	11300.	*2966.	3520.	2940.	33400.	* 789.	822.	*1128.
24	* 846.	2800.	*2755.	17100.	8760.	2730.	3060.	3620.	6830.	* 761.	874.	*1087.
25	* 815.	9100.	3980.	*12145.	5880.	2710.	*2834.	3290.	*5588.	* 733.	* 843.	*1048.
26	* 786.	13400.	11300.	9220.	*4924.	*2557.	*2656.	*3013.	*4716.	* 707.	* 812.	*1010.
27	* 757.	13900.	19400.	12800.	*4237.	*2435.	*2514.	*2797.	*4086.	* 681.	679.	* 974.
28	* 730.	18100.	*13513.	13400.	*3712.	*2336.	*2400.	*2626.	*3619.	* 657.	2040.	1060.
29	852.	11500.	12700.	11100.		3030.	*2307.	2130.	*3266.	* 633.	5300.	2660.
30	1070.	*8683.	9520.	*8427.		11400.	2510.	2560.	*2994.	* 610.	4250.	1710.
31	1130.		7250.	*6675.		15000.		2480.		* 588.	3780.	

TABLE A-4

1974-75 U.S.G.S. STREAMFLOW MEASUREMENTS: GREAT MIAMI RIVER AT HAMILTON, OHIO

(DISCHARGE IN CUBIC FEET PER SECOND)

DAY	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP
1	1290.	818.	2210.	7460.	14200.	13200.	8330.	7520.	2260.	1250.	964.	1590.
2	1130.	894.	2590.	8810.	9820.	9590.	6340.	7260.	1910.	1160.	920.	1200.
3	1050.	838.	2620.	6640.	7480.	7130.	6040.	5840.	1930.	1070.	1010.	1070.
4	1000.	1850.	2350.	5570.	6310.	6080.	5150.	4930.	1830.	4660.	2190.	1180.
5	968.	2760.	2130.	4890.	5870.	4810.	4470.	4520.	1970.	11600.	1790.	1220.
6	909.	2110.	1970.	4280.	6220.	4290.	3940.	4100.	2920.	6570.	1510.	1250.
7	864.	2010.	3170.	3810.	7430.	4520.	3650.	3680.	6930.	4200.	1300.	1280.
8	873.	1740.	8430.	4310.	5710.	5170.	3470.	3320.	4510.	3420.	1170.	1140.
9	852.	1470.	8400.	12000.	4390.	5960.	3270.	3100.	3120.	3110.	1090.	1020.
10	856.	1270.	6740.	16100.	3600.	5080.	3120.	2890.	2430.	2550.	1020.	931.
11	833.	1610.	4990.	21200.	3440.	4640.	3020.	2680.	2400.	2130.	1740.	1390.
12	822.	2400.	4380.	14500.	4940.	20900.	2880.	2770.	2930.	1730.	1390.	3160.
13	808.	1810.	5080.	9040.	4260.	16100.	2720.	3050.	2670.	1580.	1190.	2020.
14	784.	1740.	6220.	6150.	3530.	11400.	2620.	2620.	2760.	1900.	1070.	1600.
15	1120.	1650.	6880.	4960.	3200.	8850.	2580.	2470.	2900.	1970.	1700.	1350.
16	1320.	1630.	9770.	4450.	3590.	8780.	2520.	2340.	3620.	1780.	2380.	1210.
17	997.	1550.	9170.	3880.	6020.	9420.	2480.	2200.	3370.	1520.	1820.	1120.
18	934.	1420.	6920.	4230.	11000.	11900.	2420.	2160.	2700.	2170.	1670.	1180.
19	871.	1390.	5380.	4850.	11700.	13900.	2820.	2090.	2230.	4870.	1250.	1450.
20	811.	1510.	4630.	4150.	8770.	12400.	2650.	2030.	1980.	5900.	1110.	1280.
21	799.	1740.	3920.	3500.	6230.	9620.	2540.	1940.	1890.	3490.	1070.	1370.
22	815.	1650.	3550.	3110.	5200.	7590.	2390.	1890.	1550.	2570.	1090.	1220.
23	808.	1520.	3190.	2960.	26700.	6220.	2510.	1890.	1440.	2000.	995.	1170.
24	849.	1530.	4180.	2840.	55100.	6990.	11200.	1840.	1340.	1640.	1130.	1290.
25	845.	2310.	8620.	3140.	45800.	8850.	17400.	1800.	1250.	1480.	1140.	1240.
26	826.	2450.	7860.	5910.	33700.	7890.	21700.	1780.	2100.	1330.	1100.	1150.
27	775.	2440.	5710.	6290.	23400.	5870.	12400.	2060.	2160.	1170.	987.	1140.
28	753.	2240.	4340.	4580.	16800.	6820.	24400.	1900.	1590.	1120.	905.	1070.
29	788.	1950.	3610.	7260.		17300.	11500.	1750.	1590.	1120.	846.	962.
30	821.	1810.	3150.	18300.		17600.	8310.	1840.	1380.	1080.	1120.	954.
31	854.		3470.	18300.		12800.		1910.		1000.	1650.	

TABLE A-5

1974-75 PREDICTED STREAMFLOW MEASUREMENTS: GREAT MIAMI RIVER AT HAMILTON, OHIO

(DISCHARGE IN CUBIC FEET PER SECOND)

DAY	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP
1	1290.	823.	2210.	7460.	14200.	9581.	9282.	7520.	2260.	1114.	1299.	1591.
2	1244.	894.	2590.	6021.	10378.	7441.	7244.	7260.	1910.	1074.	1252.	1533.
3	1199.	838.	2620.	6640.	7961.	6008.	6040.	5885.	1930.	1036.	1207.	1478.
4	1156.	1850.	2485.	5570.	6310.	5014.	5150.	4927.	1830.	4660.	2170.	1425.
5	1114.	2760.	2376.	4703.	5870.	4303.	4401.	4239.	1970.	11600.	1790.	1374.
6	1074.	2597.	2291.	4076.	6220.	3781.	3854.	3734.	2920.	8747.	1726.	1324.
7	1035.	2467.	3170.	3611.	5163.	4520.	3448.	3353.	2724.	6889.	1663.	1277.
8	998.	2362.	8430.	4310.	4411.	5170.	3132.	3062.	2568.	5629.	1604.	1231.
9	962.	2277.	6677.	12000.	3861.	4416.	2890.	2835.	2443.	4745.	1546.	1186.
10	927.	2195.	5483.	16100.	3450.	5080.	2700.	2657.	2343.	2550.	1020.	1144.
11	894.	1610.	4641.	21200.	3136.	4640.	2549.	2515.	2400.	2130.	1740.	1390.
12	862.	2400.	4030.	14500.	4940.	20900.	2428.	2770.	2930.	2053.	1390.	3160.
13	831.	2307.	3577.	10564.	4260.	16100.	2331.	3050.	2670.	1580.	1190.	2020.
14	801.	2224.	3234.	8081.	3749.	11400.	2247.	2826.	2525.	1900.	1147.	1947.
15	1320.	2144.	6880.	6443.	3365.	8850.	2166.	2649.	2900.	1970.	1106.	1877.
16	1320.	2069.	9770.	5319.	3590.	6958.	2088.	2509.	3620.	1899.	1066.	1810.
17	1272.	1992.	7565.	4523.	6020.	5677.	2480.	2396.	3370.	1520.	1028.	1744.
18	1227.	1921.	6093.	4230.	5023.	4779.	2420.	2160.	2700.	2170.	991.	1180.
19	1183.	1852.	5380.	4850.	4309.	13900.	2820.	2090.	2549.	2092.	955.	1450.
20	1140.	1510.	4630.	4184.	3785.	12400.	2650.	2015.	2428.	2017.	921.	1398.
21	1099.	1740.	4022.	3692.	3392.	9254.	2509.	1942.	1890.	1944.	887.	1347.
22	1059.	1677.	3571.	3321.	5200.	7590.	2396.	1872.	1550.	1874.	855.	1299.
23	1021.	1617.	3229.	3037.	26700.	6220.	2510.	1805.	1494.	1807.	825.	1252.
24	984.	1530.	2966.	2816.	55100.	6990.	11200.	1740.	1440.	1741.	795.	1290.
25	949.	2310.	8620.	3140.	45800.	8850.	17400.	1677.	1389.	1679.	766.	1240.
26	915.	2450.	7860.	5910.	27942.	6958.	21700.	1617.	1339.	1618.	739.	1195.
27	882.	2348.	6293.	4945.	18398.	5870.	14856.	1559.	1290.	1560.	712.	1152.
28	850.	2261.	5215.	4251.	12920.	6820.	24400.	1503.	1244.	1504.	687.	1111.
29	820.	2182.	4448.	7260.		17300.	11500.	1750.	1199.	1450.	662.	1071.
30	821.	1810.	3888.	18300.		17600.	8310.	1840.	1156.	1398.	1120.	1032.
31	854.		3470.	18300.		12444.		1910.		1347.	1650.	

APPENDIX B

Background Chloride Concentration

In order to accurately forecast the daily chloride concentration in the Great Miami River, the predictive model requires an estimate of the background concentration entering the system at river mile 28.0 the upstream boundary. Chloride field measurements gathered by the Ohio EPA from October 1973 through September 1974 are listed in Table B-1; these observations were taken at river mile 25.7. Attempting to model variations in background input is difficult because little information is available on chloride loadings to the Great Miami River upstream of river mile 26.0. For this reason it is necessary to try to relate variations in background chloride concentrations to the variation in some other parameter. Because of the conservative nature of chlorides, it would be expected that a relatively high degree of correlation may exist between streamflow and background chloride levels. In particular, if the background chloride load (pounds per day) entering the system were constant over time, the correlation between background concentration and streamflow would be extremely high.

The first attempt to model the relationship between streamflow and background chloride concentration employs a linear regression analysis on the data given in Table B-1. As stated previously, the chloride field observations were gathered by the Ohio EPA, while the streamflow observations were obtained from U.S.G.S. records for the Great Miami River at Hamilton, Ohio. The resulting equation and the correlation coefficient from the analysis are given as

$$UC = -.00128Q + 46.57 \qquad (B\text{-}1)$$
$$r = -.49928$$

where UC = background chloride concentration (mg/ℓ),

Q = streamflow (cfs)

r = correlation coefficient.

In an attempt to improve upon the above model, a second regression analysis is performed, this time on the logarithms

TABLE B-1

STREAMFLOW VS. BACKGROUND CHLORIDE CONC.
OCTOBER 1973 - SEPTEMBER 1974

DATE	FLOW (CFS)	ACTUAL CONC. (MG/L)	PRED. CONC. (MG/L)	DATE	FLOW (CFS)	ACTUAL CONC. (MG/L)	PRED. CONC. (MG/L)
OCT. 2	3380.	38.	37.	APR. 3	12200.	21.	27.
OCT. 10	1180.	46.	48.	APR. 9	14500.	27.	26.
OCT. 17	1100.	52.	49.	APR. 16	4420.	30.	35.
OCT. 23	919.	59.	51.	APR. 24	3060.	35.	38.
OCT. 31	1130.	60.	49.	MAY 1	3070.	36.	38.
NOV. 7	1090.	49.	49.	MAY 7	2120.	39.	42.
NOV. 14	947.	58.	51.	MAY 14	2660.	39.	39.
NOV. 20	1220.	50.	48.	MAY 22	2920.	34.	38.
NOV. 28	18100.	56.	25.	MAY 28	2030.	37.	42.
DEC. 5	3920.	29.	36.	JUN. 4	1660.	41.	44.
DEC. 12	2410.	36.	40.	JUN. 12	1310.	47.	47.
DEC. 18	2100.	48.	42.	JUN. 19	1100.	47.	49.
DEC. 25	3980.	40.	36.	JUN. 26	6740.	25.	31.
JAN. 1	5670.	37.	33.	JUL. 2	2440.	35.	40.
JAN. 9	2750.	39.	39.	JUL. 9	1590.	40.	45.
JAN. 15	4790.	46.	34.	JUL. 16	1170.	49.	48.
JAN. 22	23100.	18.	23.	JUL. 24	861.	59.	52.
JAN. 31	9320.	22.	30.	JUL. 30	734.	62.	54.
FEB. 6	4130.	31.	35.	AUG. 6	813.	57.	53.
FEB. 12	3510.	41.	37.	AUG. 14	888.	57.	52.
FEB. 19	3510.	35.	37.	AUG. 20	1180.	53.	48.
FEB. 27	4140.	53.	35.	AUG. 27	679.	39.	55.
MAR. 6	5960.	35.	32.	SEPT. 3	4980.	25.	34.
MAR. 12	3560.	34.	37.	SEPT. 11	1250.	42.	47.
MAR. 19	3020.	36.	38.	SEPT. 18	1460.	36.	46.
MAR. 27	2570.	62.	40.	SEPT. 24	979.	50.	50.

of the data contained in Table B-1. The results of this analysis are given below:

$$UC = 273\ Q^{-.246}$$
$$r = -.70 \qquad \text{(B-2)}$$

The higher degree of correlation given by Equation (B-2) implies that it is a more effective model for predicting boundary conditions than Equation (B-1). However, the two equations given above are also compared on the basis of 1) the mean error in mg/ℓ and 2) the standard deviation of the error in mg/ℓ . The results for both equations are given below.

From Equation (B-1)	From Equation (B-2)
$UC = .00128 \cdot Q + 46.57$	$UC = 273 \cdot Q^{-.246}$
$\mu_{error} = -.038$	$\mu_{error} = -.79$
$\sigma_{error} = 9.90$	$\sigma_{error} = 8.02$

Inspection of the information above shows that both equations provide a mean error in prediction of less than 1 mg/ℓ . However, Equation (B-2) gives a 20 percent reduction in the standard error over Equation (B-1). For this reason, Equation (B-2) is selected as the model to be used in predicting daily background chloride concentrations on the Great Miami River system. Figure B-1 provides a profile of actual vs. predicted background levels. It can be seen that the predictive model is effective in tracking fluctuations in the true process with the exception of a few isolated cased listed below.

Date	Actual	Pred
Nov. 28	56	25
Feb. 27	53	35
Mar. 27	62	40
Aug. 27	39	55

In order to reveal possible explanations for the large deviations between observed and predicated background levels on these four dates, records of climatological observations from October 1973 through September 1974 were obtained from the National Climatic Center in Asheville, North Carolina. Thefirst data point in question, Nov. 28, represents the

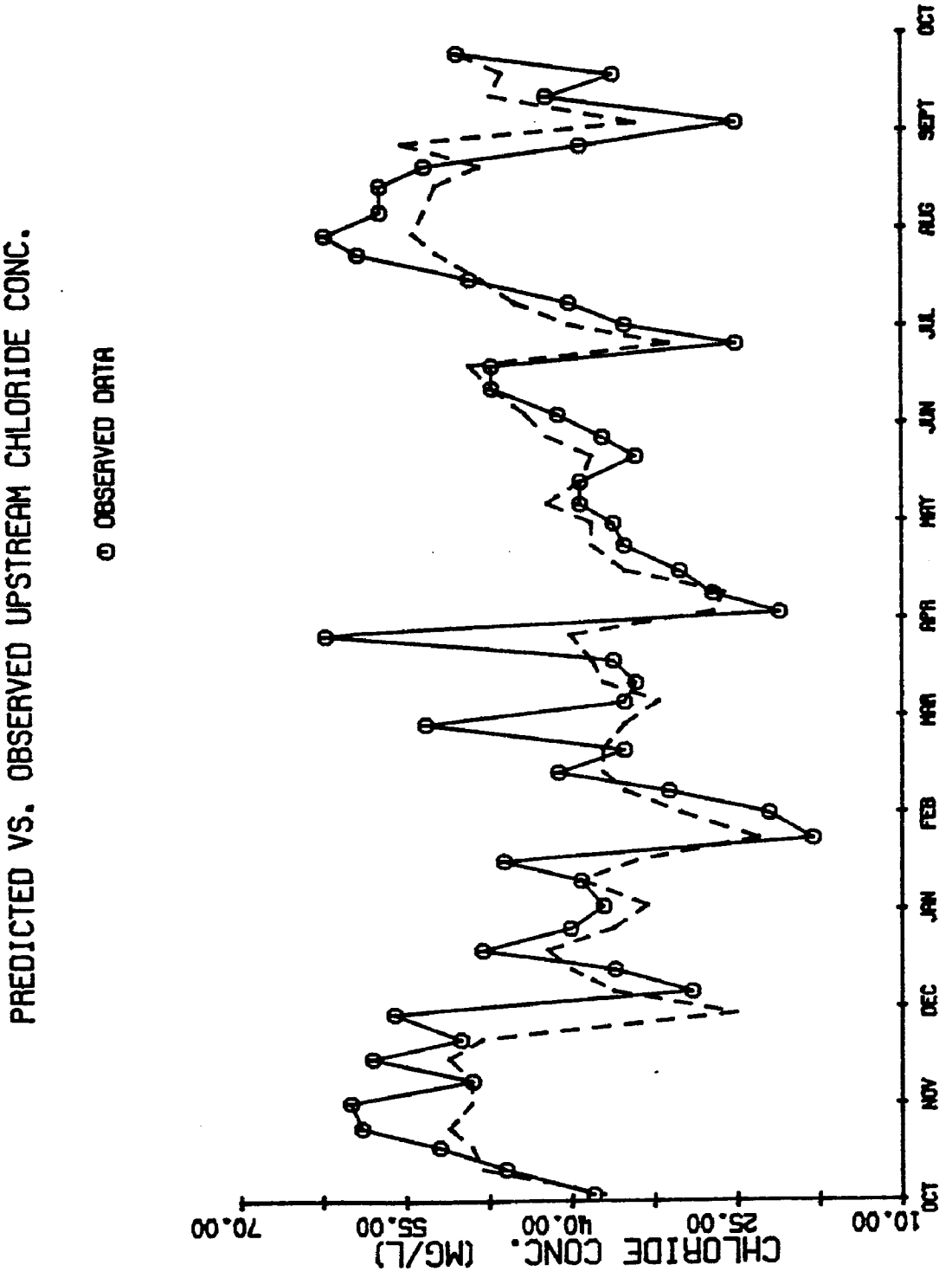

PREDICTED VS. OBSERVED UPSTREAM CHLORIDE CONC.
O OBSERVED DATA
CHLORIDE CONC. (MG/L)
10.00
25.00
40.00
55.00
70.00
OCT
NOV
DEC
JAN
FEB
MAR
APR
MAY
JUN
JUL
AUG
SEPT
OCT
TIME

FIGURE B-1

greatest error in prediction for the 1973-74 water year. Examination of U.S.G.S. discharge records and climatological data disclosed that for five consecutive days prior to, and including, Nov. 28, heavy rainfall was observed at the Ham-Fairfield station in Butler County, Ohio.

Date	Rainfall (inches)	Streamflow
Nov. 24	1.24	2800
Nov. 25	.99	9100
Nov. 26	.61	13400
Nov. 27	.22	13900
Nov. 28	.69	18100

The third highest runoff level recorded for the months of September, October and November between 1964 and 1974 occurred on Nov. 28, 1973. The dilution effect expected from the high streamflow would seem to indicate the presence of an extremely low chloride concentration in the river. To be more specific, a chloride loading of approximately 6 million pounds per day would be required to effect a concentration of 56 mg/ℓ at a streamflow level of 18,100 cfs. This information may be sufficient evidence to discount the chloride observation as an erroneous measurement. However, it is possible that the continuous heavy rainfall over a five-day period may have resulted in a flushing of chloride deposits within the watershed directly into the river. Similarly, the high runoff condition may have resulted in an intentional or unintentional waste discharge from an industrial point-source located on the Great Miami River.

On inspection of flow and climatological records for Feb. 27, 1974, it was noted that no significant precipitation occurred on this date. However, for several days previous to this date, freezing temperatures were recorded, dropping as low as 10° F and 11° F on Feb. 25 and 26, respectively. In addition, on Feb. 24, 17 inches of rain and 2.5 inches of snow were recorded at the Ham-Fairfield station. On the date in question, Feb. 27, the temperature rose significantly to a high of 55° F. It is quite possible that

large quantities of rock salt were used on Feb. 24 for the purpose of melting snow and ice on local highway systems. Given that this salt was temporalily held on top of frozen ground and in snow deposits in the region, the sharp temperature increase on Feb. 27 may have resulted in a release of chlorides to the river. This would account for the large discrepency between observed and predicted background chloride levels on this data.

The prediction error for March 27 may be explained by exactly the same reasoning. Three days previous, on March 24, .33 inches of rainfall and 3.5 inches of snowfall were recorded. Temperatures on March 25 fell to a low of 8° F but on March 26 and 27, temperatures rapidly rose, reaching a high of 62°F on March 27. Thus, the exact same conditions which prevailed on Feb. 27 were also present on March 27.

The final data point in question occurred on Aug. 27, and on this date there seems to be no explanation for the prediction error other than the weakness of the model used to relate background concentration and streamflow. Examination of runoff and climatological records revealed no unusual phenomena which could have accounted for the discrepancy.

Rather than excluding the data for Nov. 28, Feb. 27, and March 27 from the analysis used to derive Equation (B-2), all of the original field observations were retained. This was done to protect the assertion that the model is, in fact, representative of the entire set of field observations available and that no data was unjustly left out of the analysis to improve prediction accuracy. However, in order to illustrate the effect of removing these three observations in estimating the prediction error, below is a comparison of the mean and standard deviation of the error for each case.

All Data

Equation (B-1):

$\mu_{error} = -.038$

$\sigma_{error} = 9.90$

Equation (B-2):

$\mu_{error} = -.79$

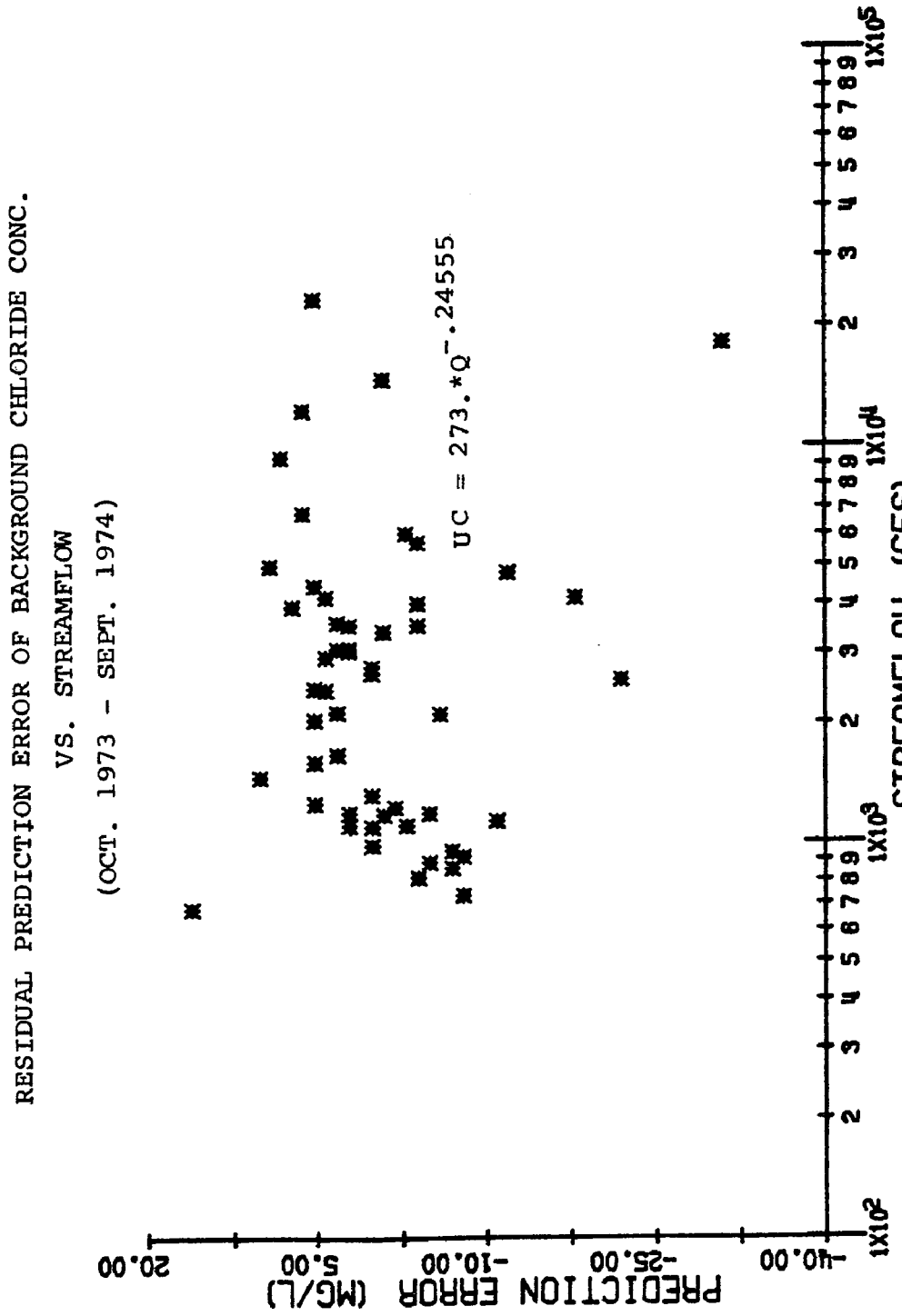
$\sigma_{error} = 8.02$

Excluding Nov. 28, Feb. 27, March 27

Equation (B-1):	Equation (B-2):
$\mu_{error} = -1.134$	$\mu_{error} = .27$
$\sigma_{error} = 8.45$	$\sigma_{error} = 5.17$

Examination of the above information reveals that removal of the data for the dates in question reduces the standard error by approximately 35% and the mean error by over 60% for Equation (B-2). In each case Equation (B-2) has a significantly smaller standard error than Equation (B-1).

Having arrived at an acceptable model for predicting background chloride levels, it is then only necessary to obtain a daily streamflow value in order to estimate the magnitude of this input variable to the computer simulation of the Great Miami River. Fig. B-3 provides a plot of predicted background chloride concentration versus streamflow using Equation (B-2). In order to reveal possible trends in the error of estimation as a function of streamflow magnitude, a plot of residual error versus streamflow is presented in Figure B-2. Examination of this data shows that the estimation error does not exhibit significant trends which might warrant using different estimators of background concentration in different flow regions.


RESIDUAL PREDICTION ERROR OF BACKGROUND CHLORIDE CONC.
VS. STREAMFLOW
(OCT. 1973 - SEPT. 1974)
UC = 273.*Q^-.24555
PREDICTION ERROR (MG/L)
20.00
5.00
-10.00
-25.00
-40.00
STREAMFLOW (CFS)
1X10^2
1X10^3
1X10^4
1X10^5

FIQURE B-2

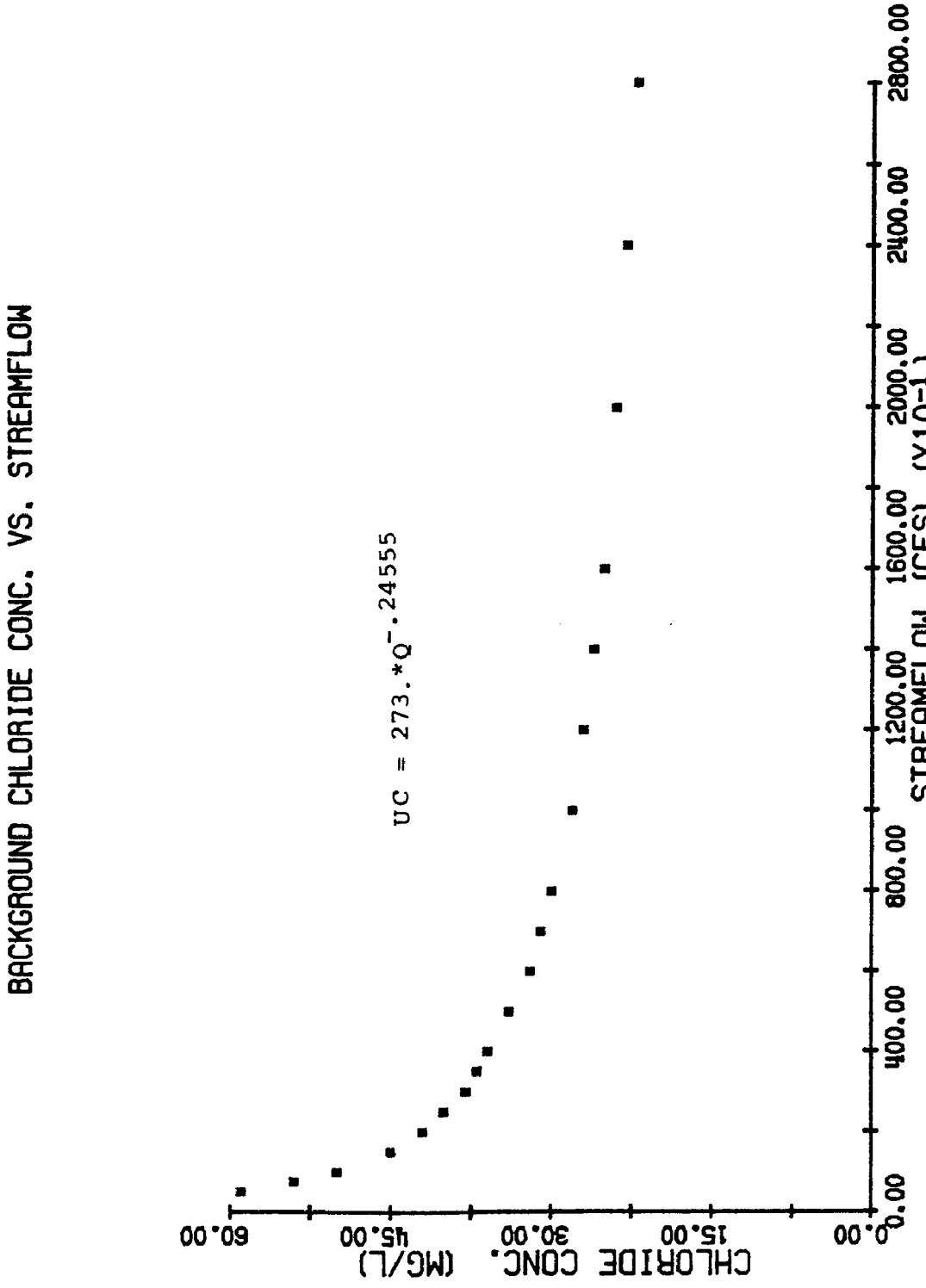

BACKGROUND CHLORIDE CONC. VS. STREAMFLOW
UC = 273.*Q^-.24555
CHLORIDE CONC. (MG/L)
0.00
15.00
30.00
45.00
60.00
0.00
400.00
800.00
1200.00
1600.00
2000.00
2400.00
2800.00
STREAMFLOW (CFS) (X10^-1)

FIGURE B-3

APPENDIX C

Derivation of Fixed-Time Sampling Policies

As a means of evaluating the effectiveness of the sampling policies which employ the computer model to determine specific sampling times, policies 1-16 were compared with several other plans which employ uniform sampling intervals for detecting stream standard violations (Beckers, et al., 1972) In deriving these uniform sampling policies, the value of a sampling frequency priority measure (M) must first be specified:

$$M = \frac{\text{Expected Number of Violations Detected}}{\text{Expected Number of Violations}}. \qquad \text{(C-1)}$$

If c(t) represents the concentration of a given pollutant at some time t, then a violation of the water quality standard SS for this material occurs when $c(t) \geq SS$. A normalized two-state process is then defined which takes the value unity if a violation occurs and the value zero otherwise: Fig. C-1 shows y(t) to be such a process.

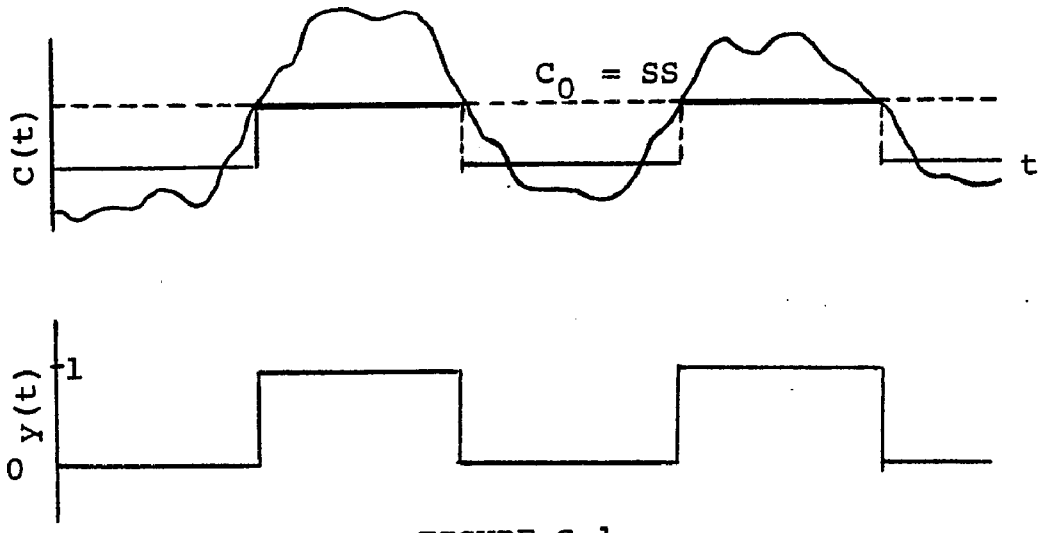


FIGURE C-1

In order to estimate the transition probabilities from one state to the other for the stochastic process y(t), the assumption is made that these probabilities are explicitly dependent on the present state of the process (0 or 1) and on the length of time y(t) has been in that state and implicitly dependent on the standard SS. The stochastic process y(t) can then be described by the values

$$\Pi_i(t) \equiv Pr[y(t) = i \mid y(0) = i] \quad i=0, 1 \qquad (C\text{-}2)$$

where $\Pi_i(t)$ is the probability that, for time $t>0$, y is in state i, given that it entered state i at time $t = 0$, i.e., no transitions occurred in time t, given a transition at time 0.

The second assumption made is that y(t) is a poisson process. The transition probabilities then become

$$\Pi_i(t) = e^{-t/T_i} \qquad i = 0, 1 \qquad (C\text{-}3)$$

where T_i represents the mean time spent in state i before a transition occurs. T_i is therefore dependent on the magnitude of SS. The violation process may then be described by means of the two parameters T_1 (the mean duration of a violation) and T_0 (the mean interval between violations). The probability of a violation is therefore given by

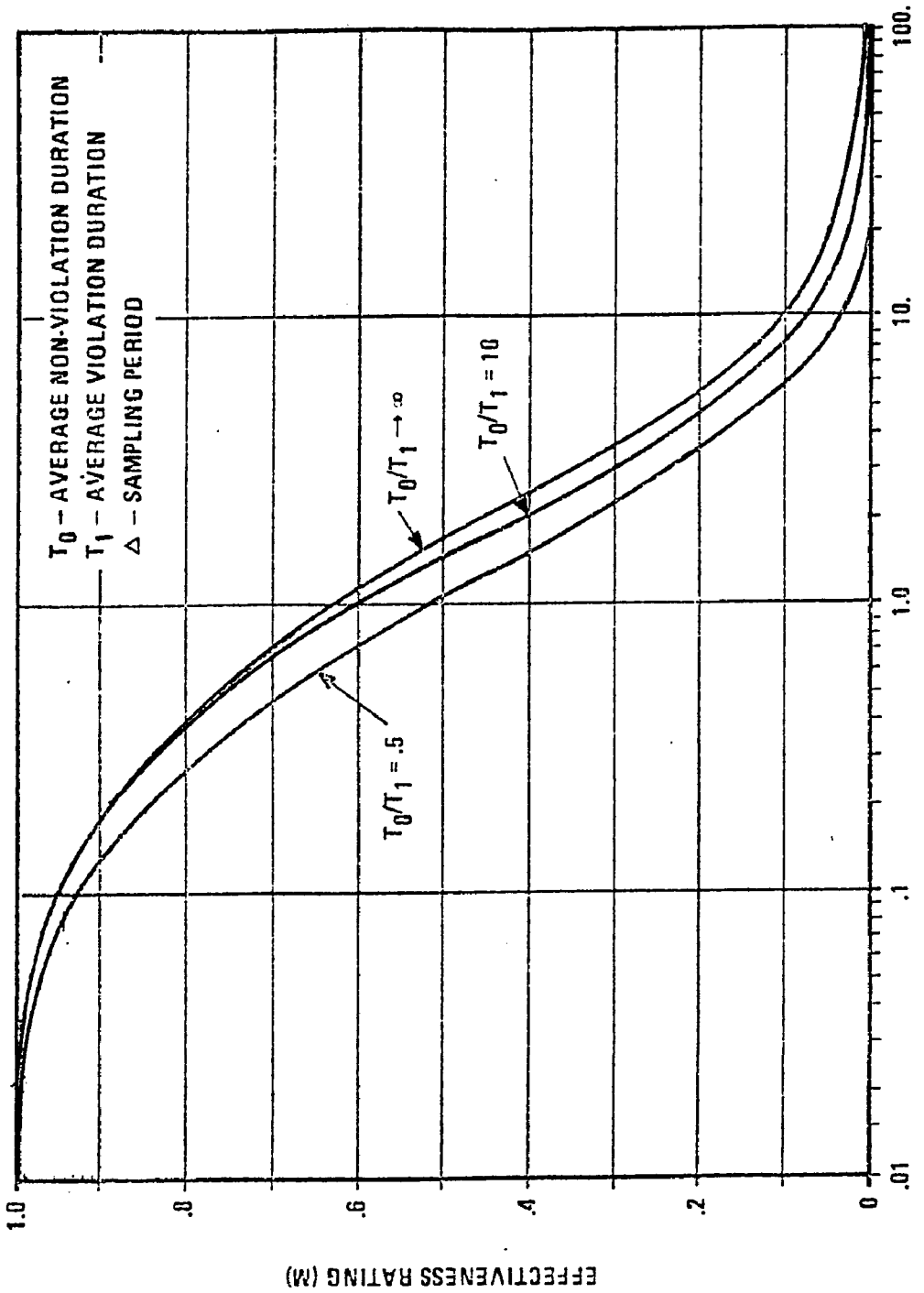
$$\Pi_V = \frac{T_1}{T_1 + T_0} \qquad (C\text{-}4)$$

Thus, to evaluate the probability Π_V it is necessary to estimate the values of T_1 and T_0. The method of estimation is discussed in detail later in this appendix.

The expression for M* (the approximate for M) is given below in terms of T_0, T_i, and the sampling interval Δ:

$$M^*(\Delta, T_0, T_1) = \frac{T_0}{\Delta} \frac{e^{-\Delta/T_0} - e^{-\Delta/T_1}}{T_0/T_1 - 1} \qquad (C\text{-}5)$$

Beckers, et al. (1972) provide a detailed derivation of Equation(C-5). This approximation is valid for the case in which $\Delta < T_0 + T_1$. Figure C-2 gives a graphic representation of M versus Δ/T_1 for various values of T_0/T_1. Once estimates



(Beckers et al., 1972)

FIGURE C-2

of T_0 and T_1 have been obtained, these curves can then be employed in determining the required sampling period Δ for a pre-specified effectiveness rating M.

Estimates of T_0 and T_1 are derived by examining several years of discharge records for the river basin under investigation. In order to understand how these estimates are obtained, consider a river system receiving a waste load from two sources, as shown in Fig. C-3.

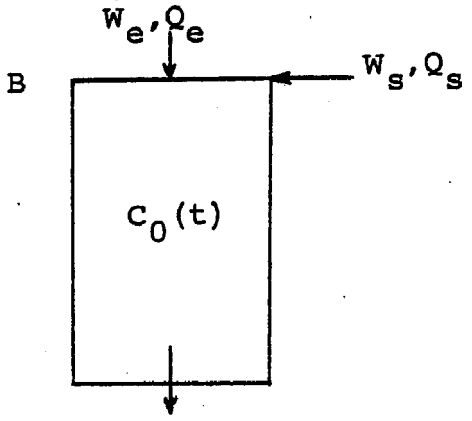


FIGURE C-3

Assuming completely-mixed, flow conditions with no dispersion for the stream segment shown in Fig. C-3, the concentration of a conservative material such as chlorides below point B can be approximated by the following relationship:

$$C_0 = \frac{W_e + W_s}{Q_e + Q_s} \qquad \text{(C-6)}$$

where: $C_0 \equiv$ concentration of contaminant below point B
$W_e \equiv$ contaminant loading from sources upstream of B
$W_s \equiv$ flow entering stream segment from above point B
$Q_s \equiv$ flow entering segment from point-source at B.

Often the temporal variations in W_s, W_e and Q_s are very small in comparison to the variations in Q_e. In such cases it may be assumed that fluctuations in the value of C_0 are due entirely to the variation in Q_e. Additionally, because

detailed information about the values of W_e, Q_e and Q_s is generally not known, average values for these variables must often be used. Under such conditions Equation C-6 becomes

$$C_0 = \frac{\text{Constant.}}{Q_e + Q_s} \qquad \text{(C-7)}$$

For cases in which $Q_s \ll Q_e$, the above equation may be rewritten as

$$C_0 = \frac{\text{Constant}}{Q_e} \qquad \text{(C-8)}$$

Once a threshold contaminant concentration SS has been specified, a threshold streamflow Q_T is immediately defined, i.e., if a violation occurs when $C_0 \geq SS$, then a violation occurs if $Q_e \leq Q_T$. The equation used to define the threshold streamflow Q_T is therefore given by

$$Q_T = \frac{W_{Avg.}}{SS} \qquad \text{(C-9)}$$

Thus, in estimating the values of T_0 and T_1, T_0 is set equal to the average length of time Q is greater than Q_T and T_1 the average length of time Q is less than Q_T. These average times are obtained by analyzing several years of past discharge records for the watershed in question.

In attempting to apply the above technique for monitoring the level of chlorides in the Great Miami River, the Ohio EPA data given in Table B-1 was used in conjunction with chloride loading information from Gulf Oil and National Lead to evaluate W_{Avg}. The average chloride load from Gulf Oil, as estimated from original field observations given in Table 5-2 for the water year 1973-74, is 4040 lbs/day. Similarly, from the data contained in Table 5-1, National Lead has an average chloride load of 1166 lbs/day. The average daily background chloride loading to the system was estimated as follows:

$$W_{BC_{avg.}} = (K_T \sum_{i}^{N} C_i Q_i)/N$$

where C_i ≡ chloride concentration (mg/ ℓ) from Table B-1

$Q_i \equiv$ streamflow (cfs) from Table B-1

$N \equiv$ number of observations (= 52)

$K_T \equiv$ factor to convert $W_{BC_{avg.}}$ to lbs/day (= 5.39)

The overall value of $W_{avg.}$ is finally given as

$$W_{avg.} = W_{BC_{avg.}} + W_{Gulf} + W_{Lead} \qquad \text{(C-10)}$$

$$= 535733. + 4040 + 1166.$$

$$W_{avg.} = 540939. \text{ lbs/day}$$

This is the value of $W_{avg.}$ used in Equation(C-9)to estimate the Q_T associated with a specific chloride water quality standard SS.

Several sampling strategies, each representing a different sampling frequency priority measure (M), were selected for evaluation on the basis of percentage of violations detected and total cost over a one year period. These strategies were tested for each of three different hypothetical chloride standards - 45, 50, and 55 mg/ℓ using U.S.G.S. discharge records from October 1965 through September 1973 to estimate T and T_1 in each case. The information used in deriving each sampling policy is given in Tables C-1 and C-2.

TABLE C-1

Chloride Standard Vs. T_0/T_1

Chloride Standard (mg/ℓ)	Q_T (cfs)	T_0 (days)	T_1 (days)	T_0/T_1
45	2228	15.24	21.01	.725
50	2006	17.99	21.26	.846
55	1823	20.14	22.33	.902

TABLE C-2

FIXED-TIME SAMPLING POLICY DERIVATION

Policy	(M)	Δ/T_1 *	Sampling Interval Δ (days)			Samples per year		
			SS=45	SS=50	SS=55	SS=45	SS=50	SS=55
A	.99	.04	1	1	1	365	365	365
B	.95	.08	2	2	2	183	183	183
C	.90	.13	3	3	3	122	122	122
D	.85	.19	4	4	4	91	91	91
E	.80	.27	6	6	6	61	61	61
F	.70	.42	9	9	9	41	41	41
G	.60	.70	15	15	16	24	24	23
H	.50	1.10	23	23	25	16	16	15
I	.40	1.50	32	32	34	11	11	11
J	.30	2.20	46	47	49	8	8	7
K	.25	2.80	59	60	63	6	6	6
L	.20	3.50	74	74	78	5	5	5
M	.15	4.50	95	96	100	4	4	4
N	.10	6.00	126	128	134	3	3	3
O	.05	8.30	174	176	185	2	2	2
P	.01	16.00	336	340	357	1	1	1

* values of Δ/T_1 are estimated from curves given in Fig. C-2

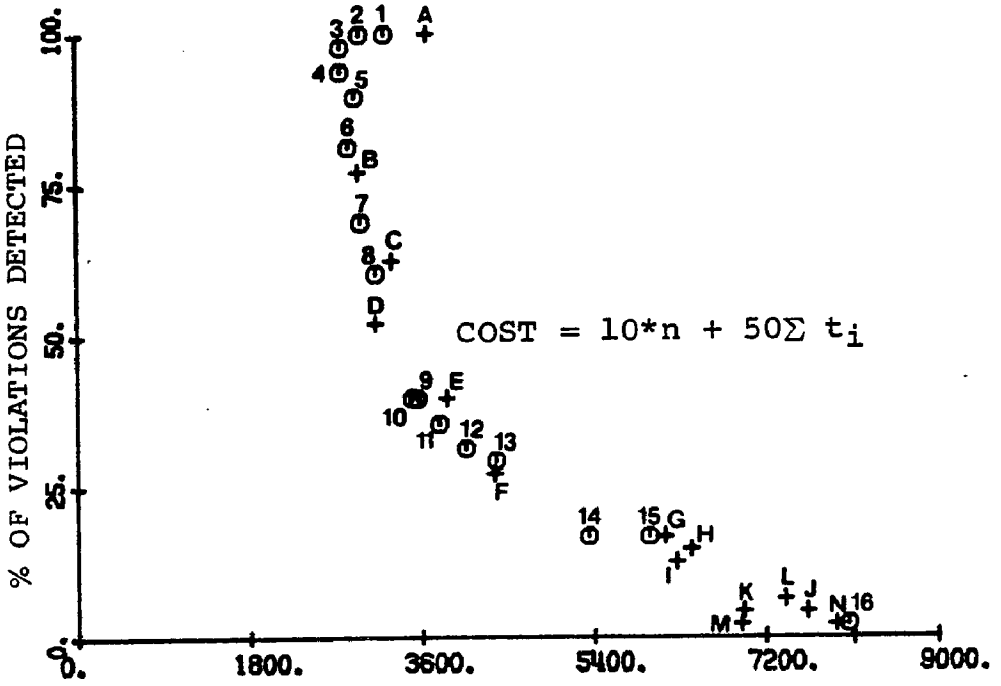
APPENDIX D

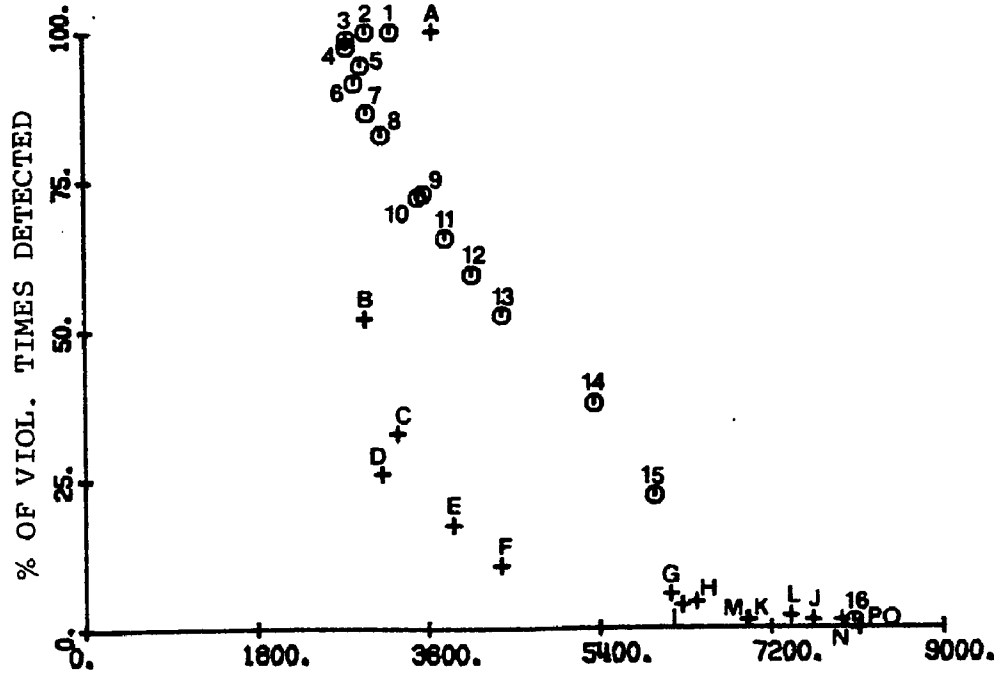
Illustrations of Detection Percentage and Total Cost of Variable and Fixed-Time Policies

TOTAL COST VS. % OF VIOLATIONS DETECTED

1973-74 U.S.G.S. STREAMFLOWS

CHLORIDE STANDARD = 45 MG/L





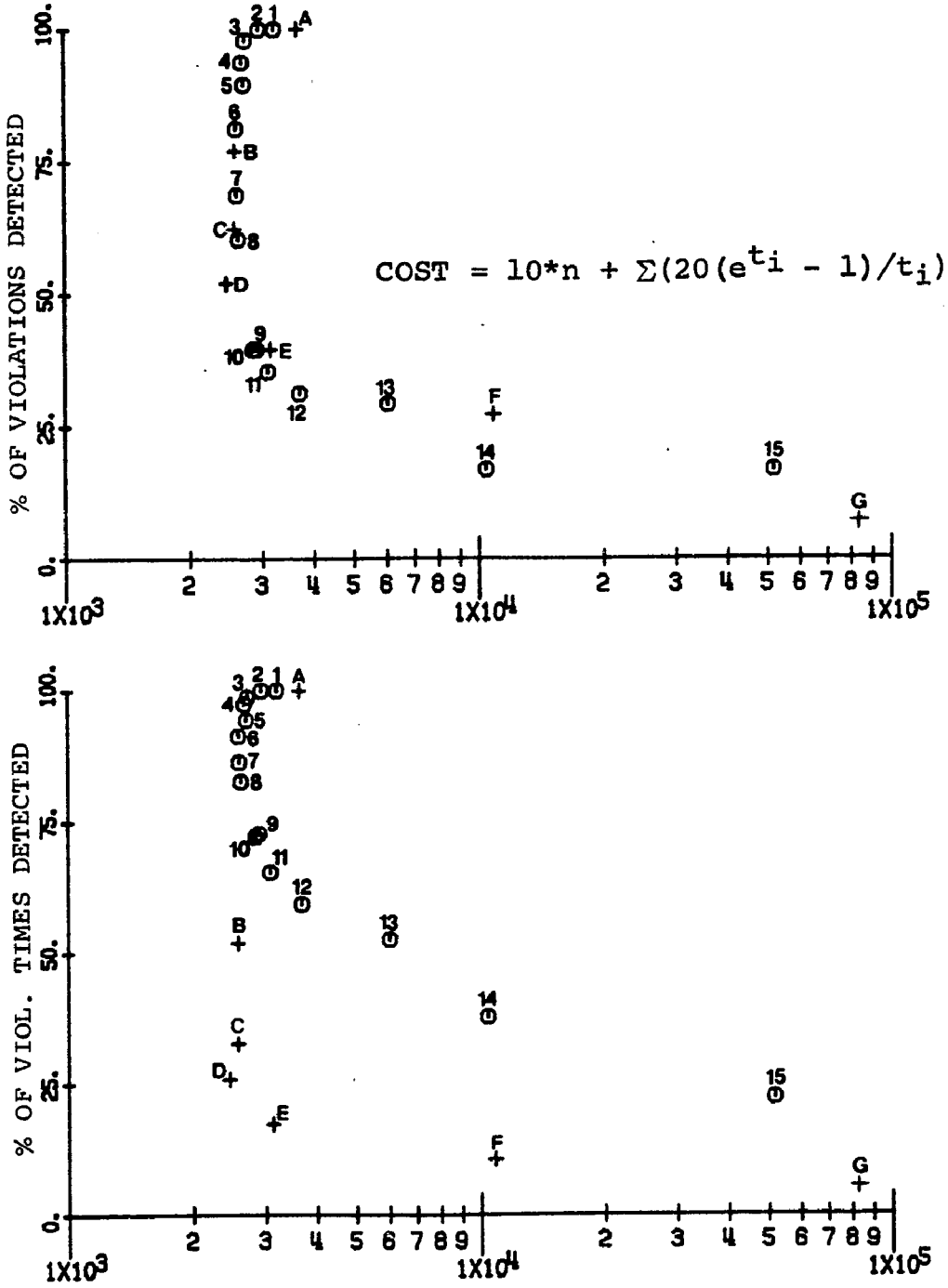
TOTAL COST (LINEAR)

FIGURE D-1

TOTAL COST VS. % OF VIOLATIONS DETECTED

1973-74 U.S.G.S. STREAMFLOWS

CHLORIDE STANDARD = 45 MG/L



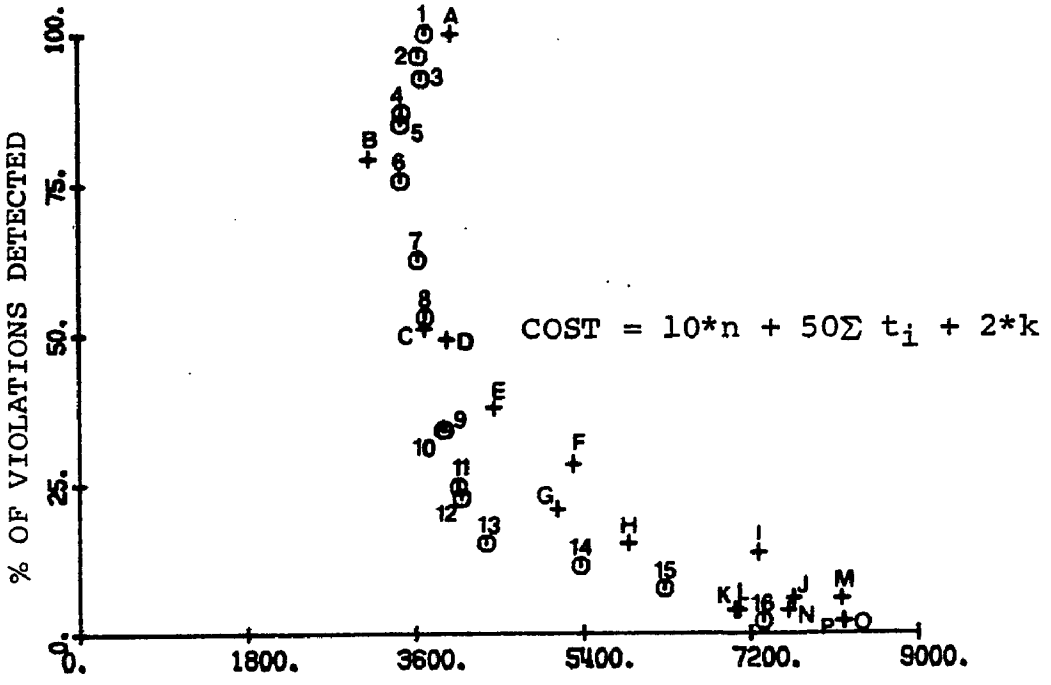
TOTAL COST (NON-LINEAR)

FIGURE D-2

TOTAL COST VS. % OF VIOLATIONS DETECTED

1973-74 PREDICTED STREAMFLOWS

CHLORIDE STANDARD = 45 MG/L



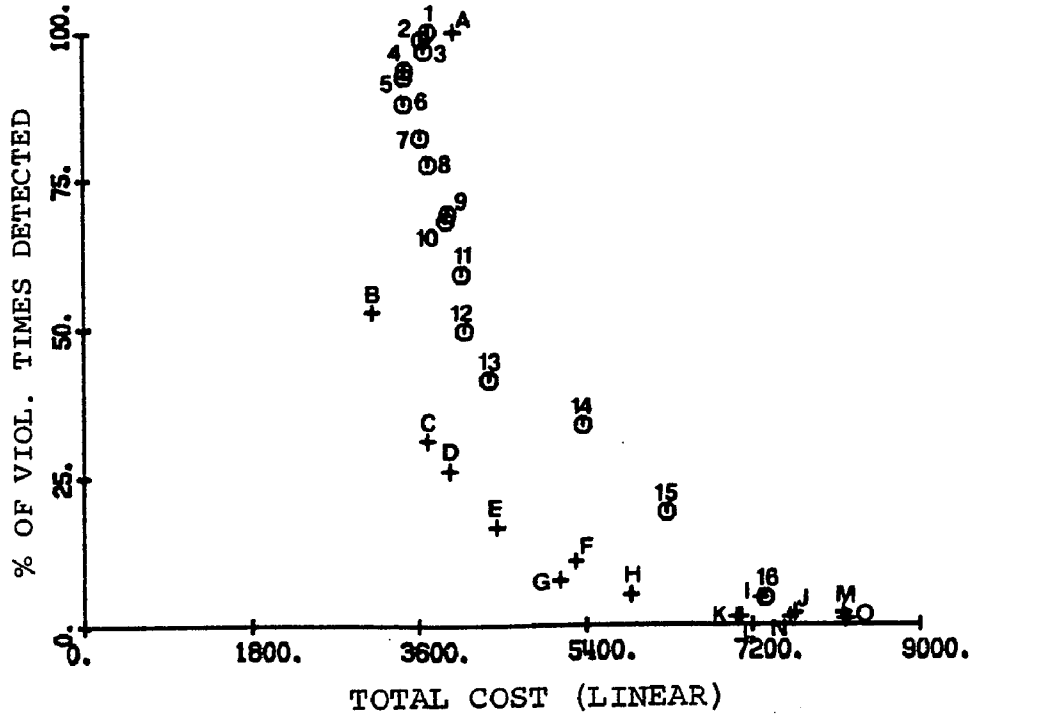
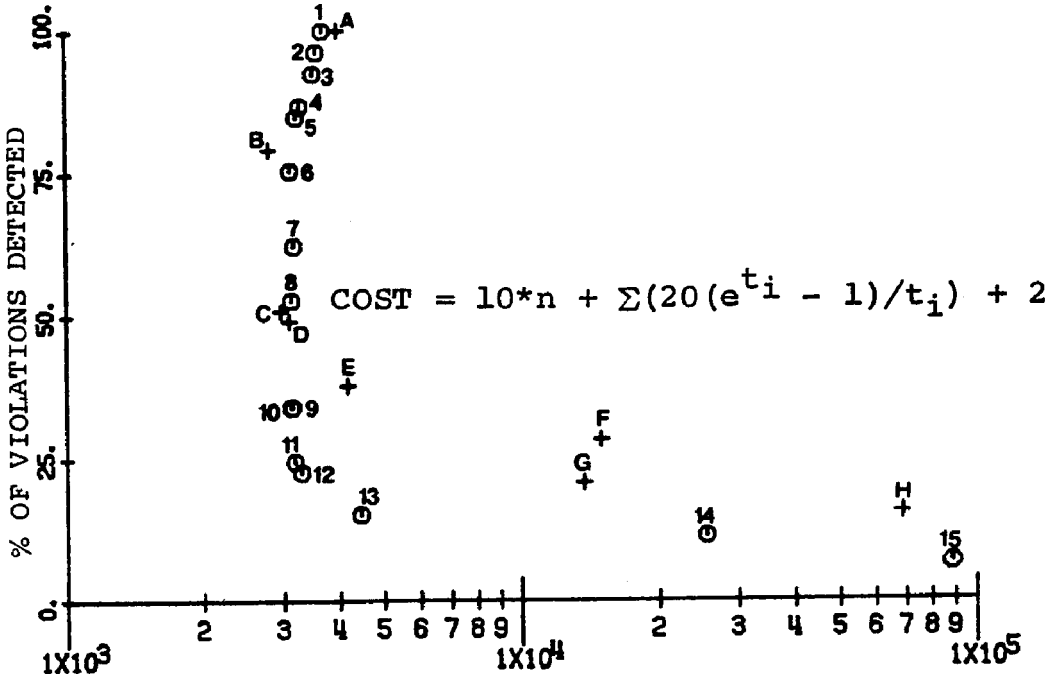


FIGURE D-3

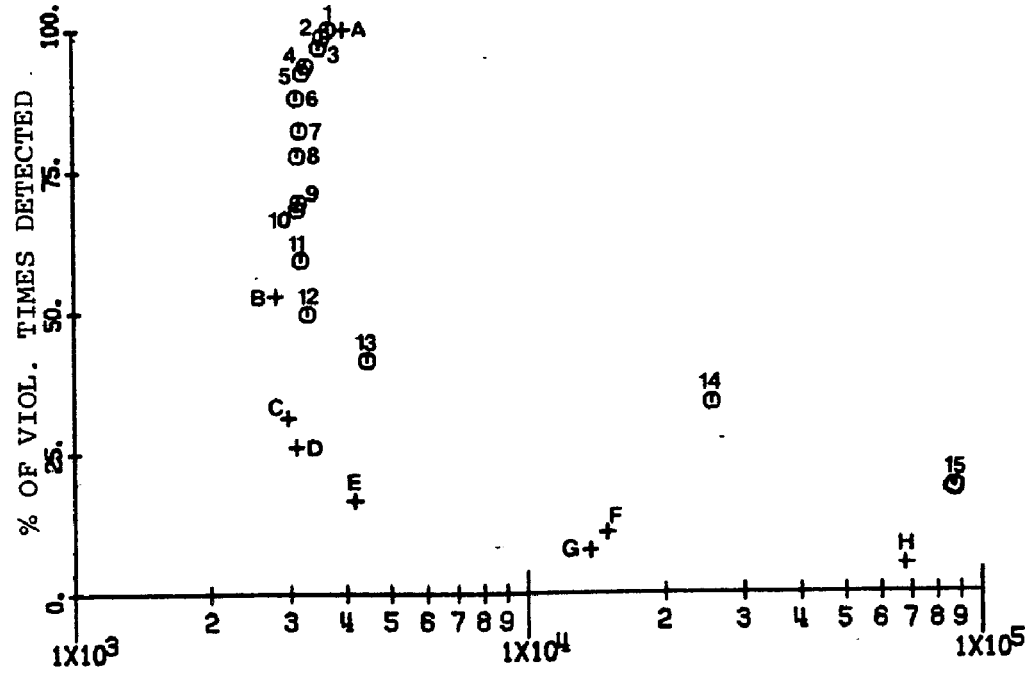
TOTAL COST VS. % OF VIOLATIONS DETECTED

1973-74 PREDICTED STREAMFLOWS

CHLORIDE STANDARD = 45 MG/L



$$\text{COST} = 10*n + \Sigma(20(e^{t_i} - 1)/t_i) + 2*k$$



TOTAL COST (NON-LINEAR)

FIGURE D-4

TOTAL COST VS. % OF VIOLATIONS DETECTED

1973-74 U.S.G.S. STREAMFLOWS

CHLORIDE STANDARD = 50 MG/L

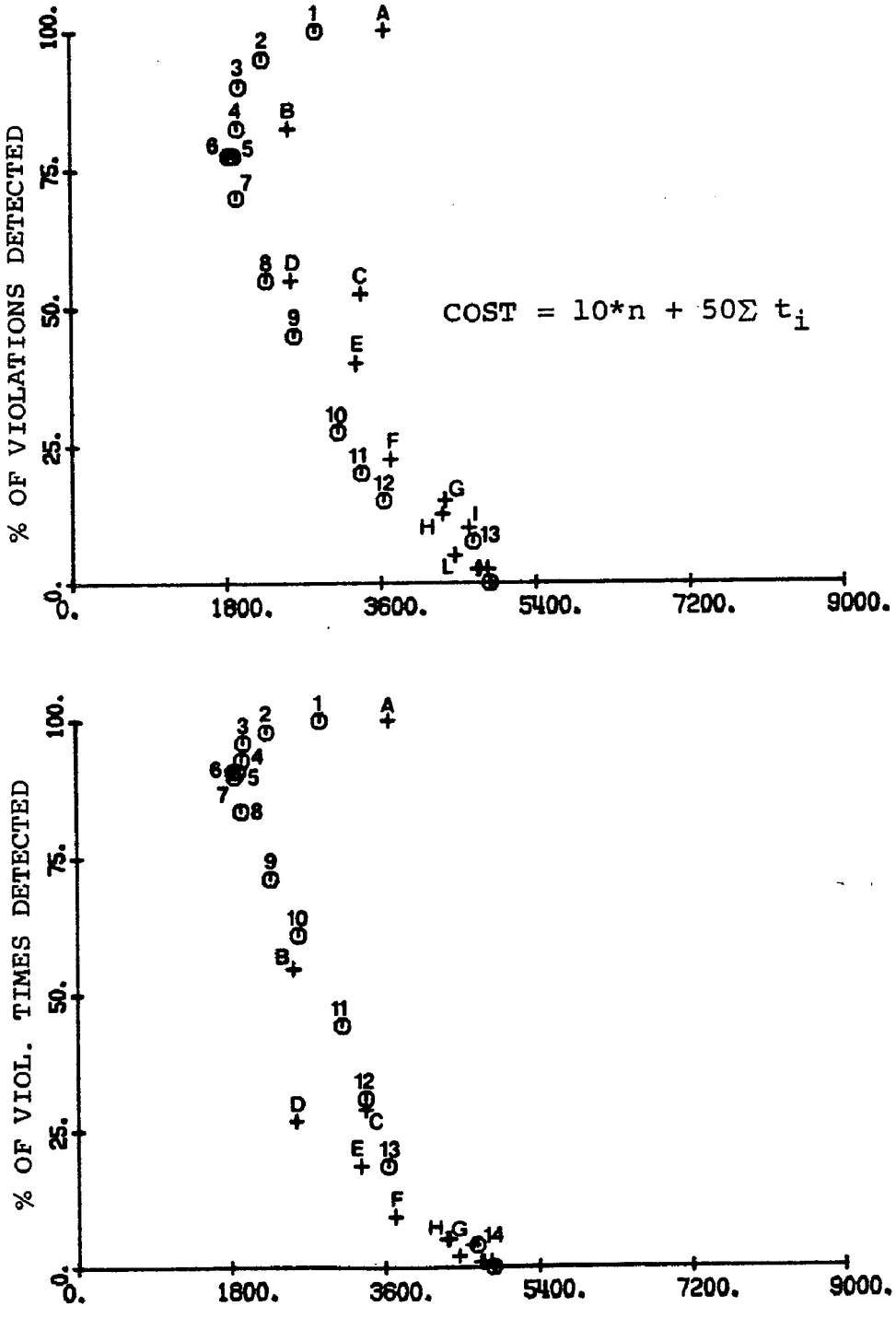


FIGURE D-5

TOTAL COST VS. % OF VIOLATIONS DETECTED

1973-74 U.S.G.S. STREAMFLOWS

CHLORIDE STANDARD = 50 MG/L

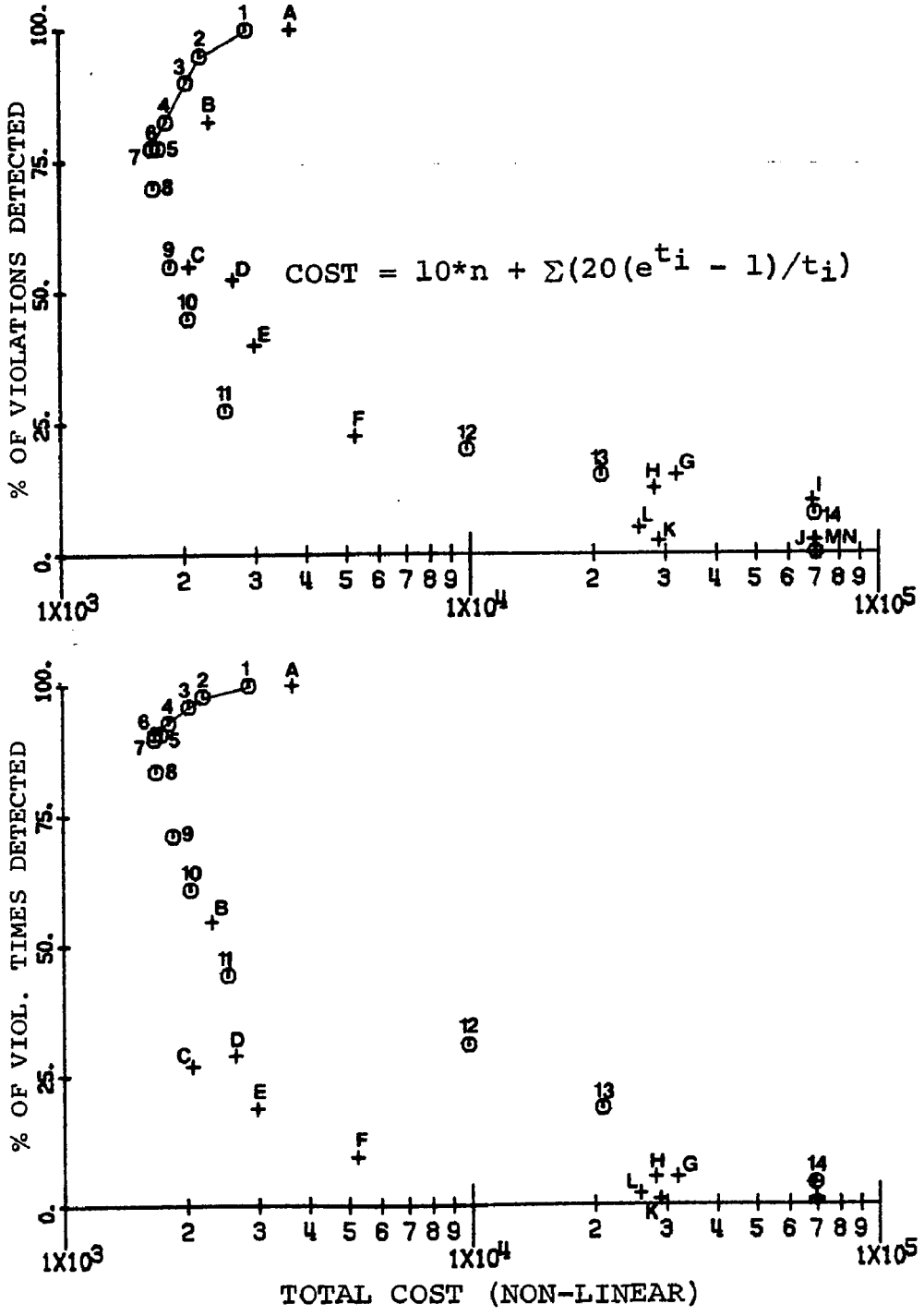
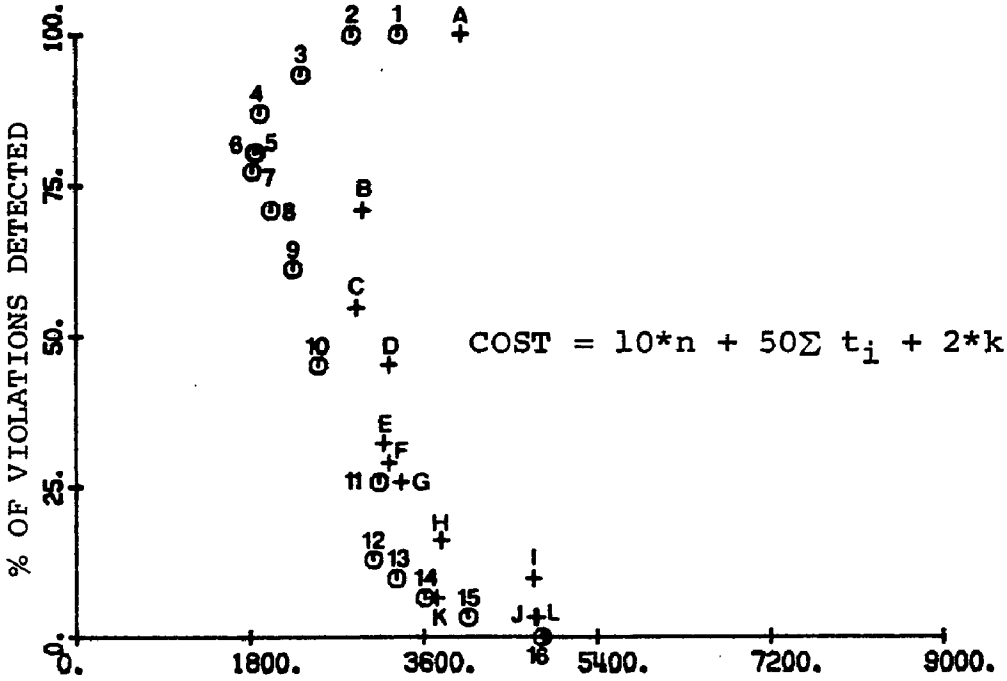


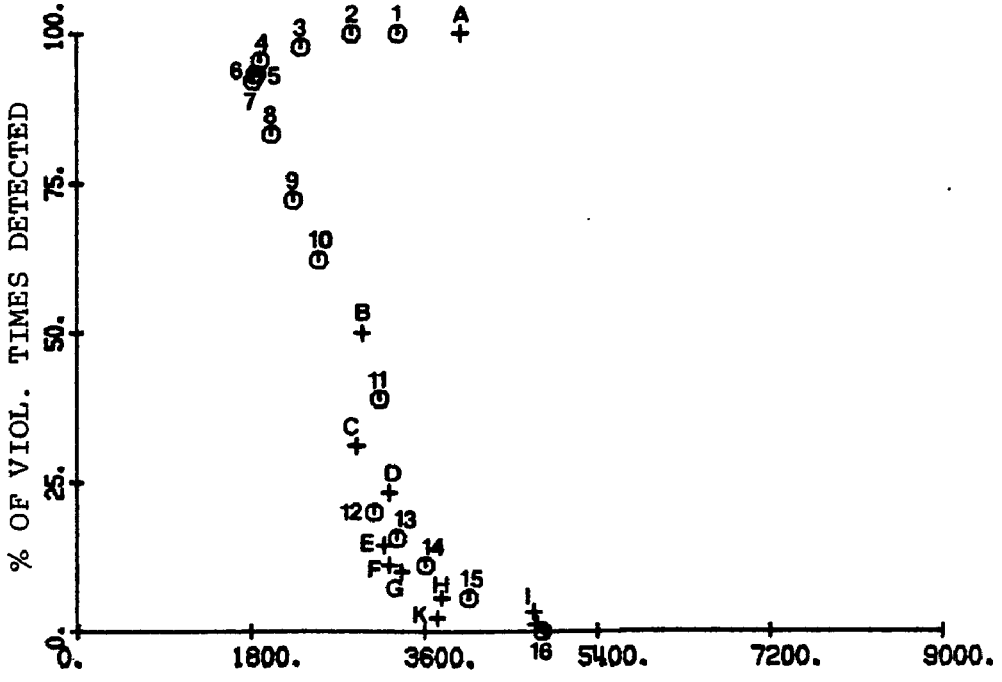
FIGURE D-6

TOTAL COST VS. % OF VIOLATIONS DETECTED

1973-74 PREDICTED STREAMFLOWS

CHLORIDE STANDARD = 50 MG/L





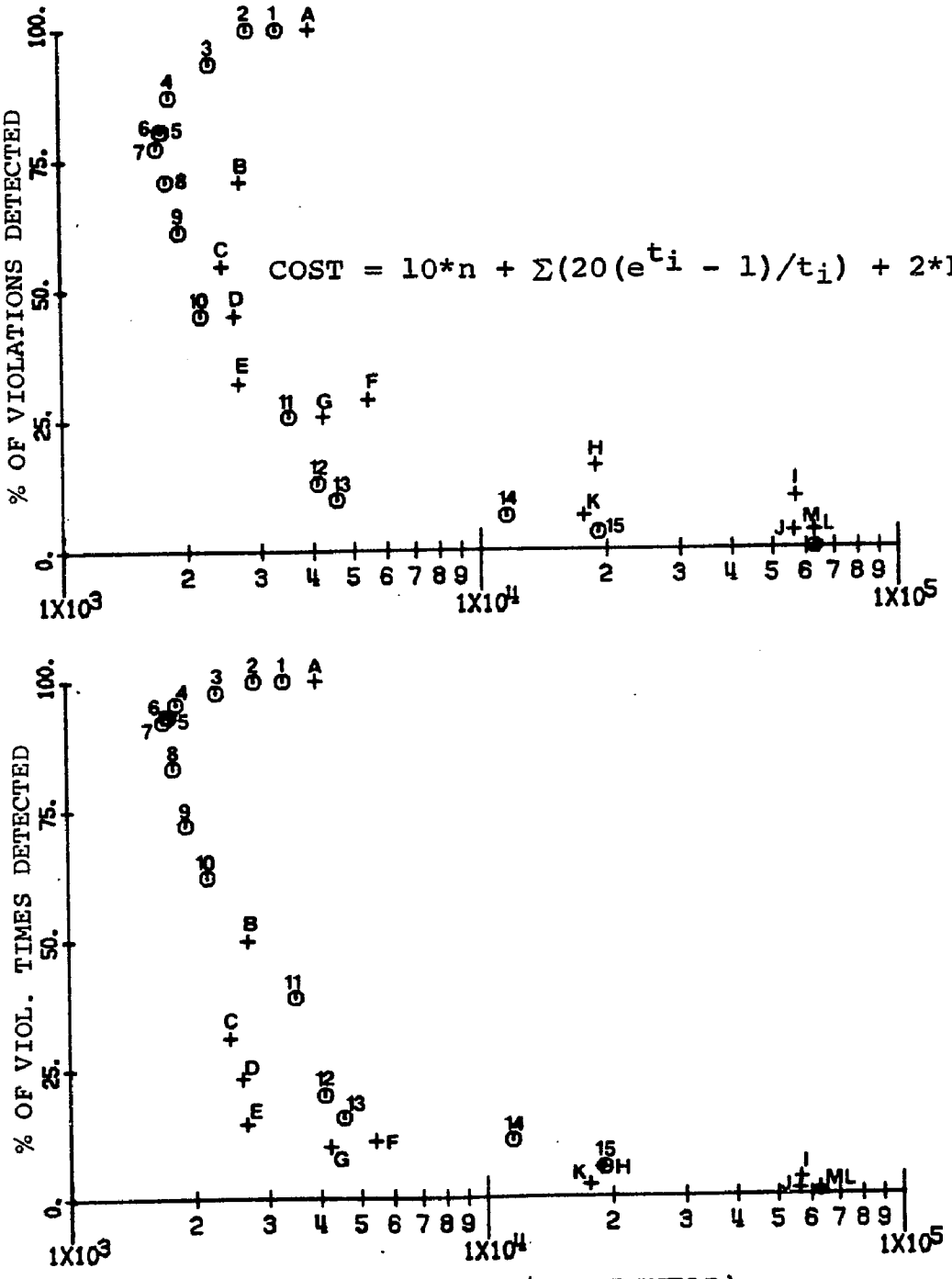
TOTAL COST (LINEAR)

FIGURE D-7

TOTAL COST VS. % OF VIOLATIONS DETECTED

1973-74 PREDICTED STREAMFLOWS

CHLORIDE STANDARD = 50 MG/L



TOTAL COST (NON-LINEAR)

FIGURE D-8

TOTAL COST VS. % OF VIOLATIONS DETECTED

1973-74 U.S.G.S. STREAMFLOWS

CHLORIDE STANDARD = 55 MG/L

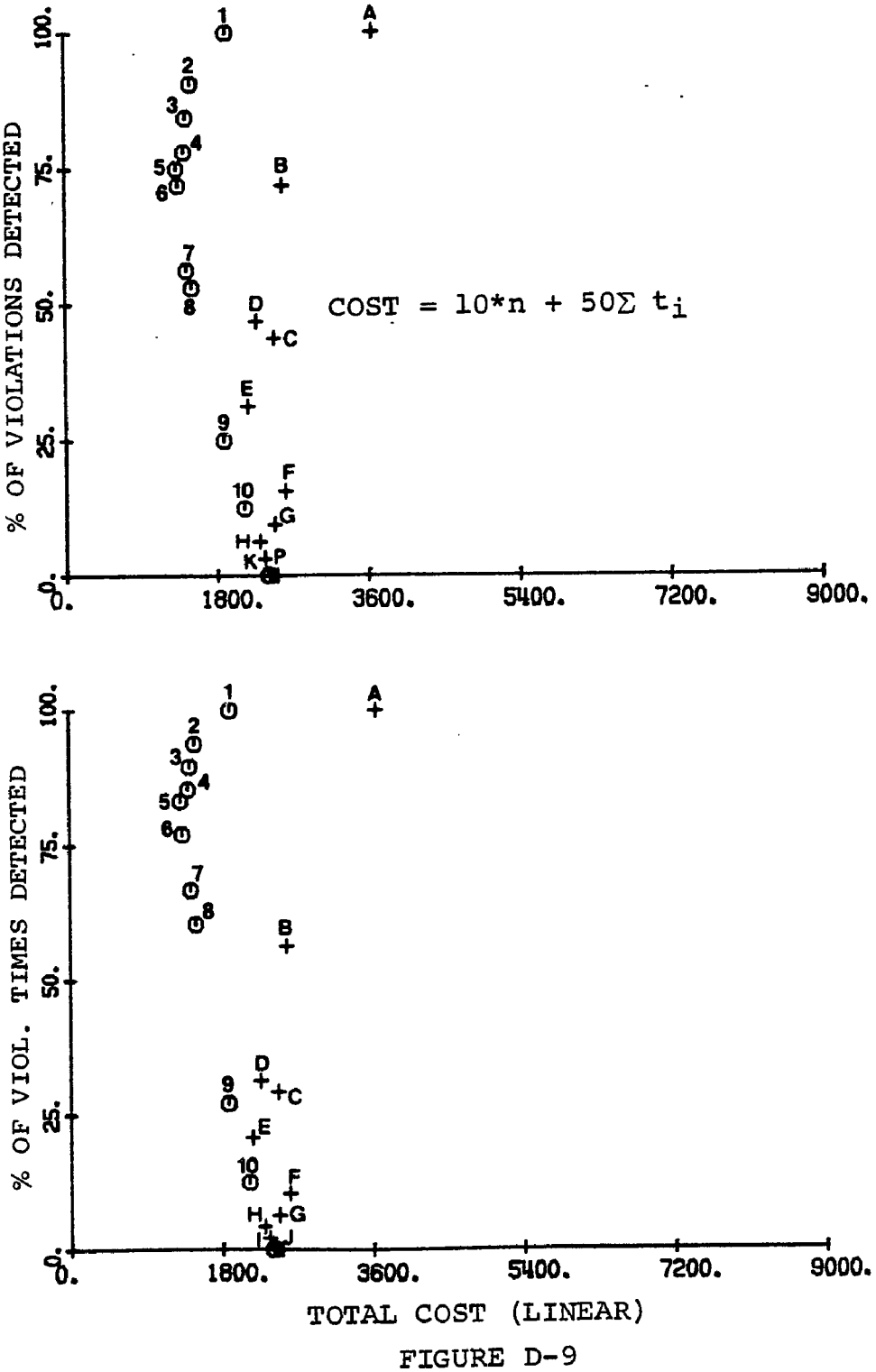


FIGURE D-9

TOTAL COST VS. % OF VIOLATIONS DETECTED

1973-74 U.S.G.S. STREAMFLOWS

CHLORIDE STANDARD = 55 MG/L

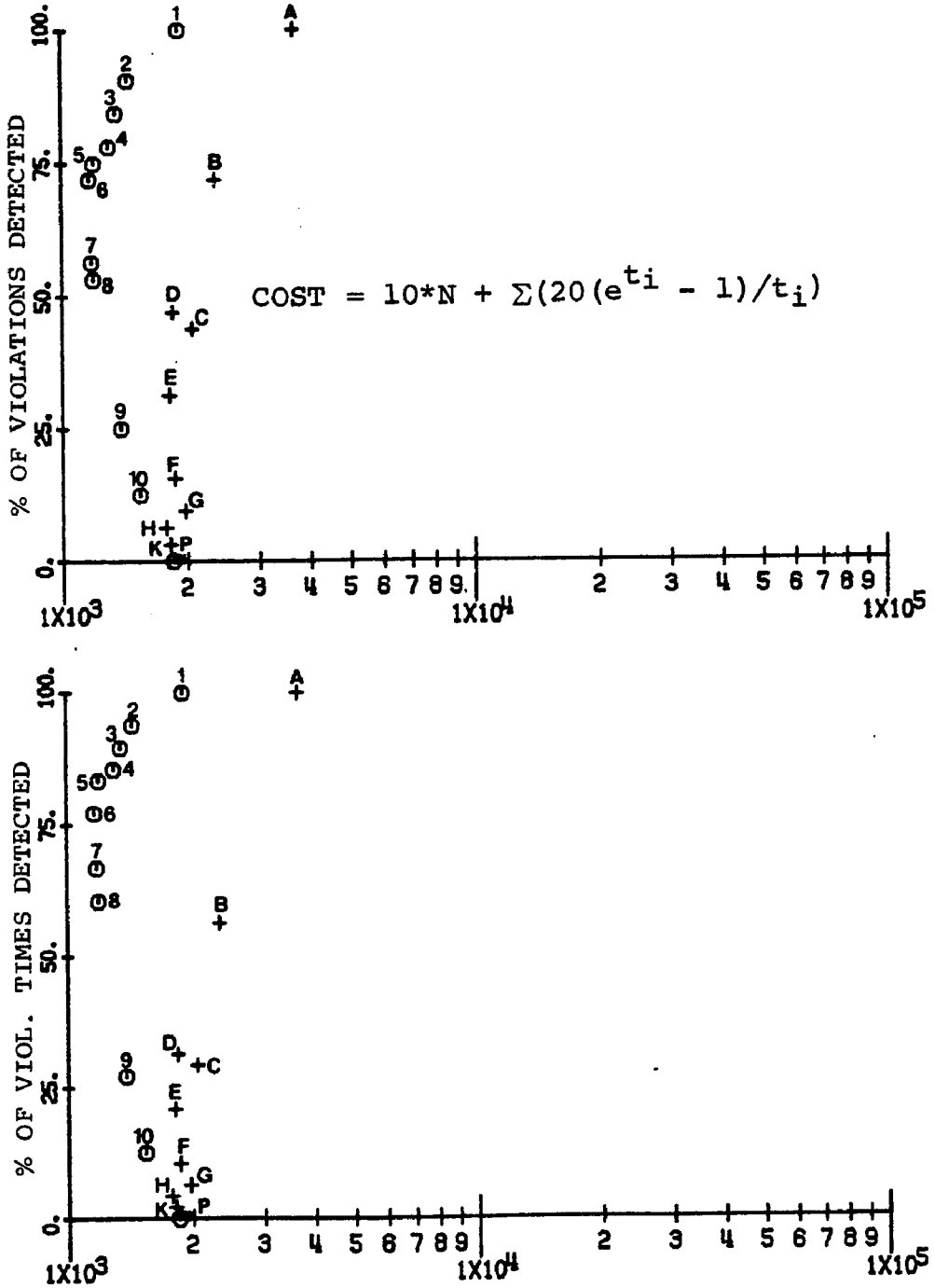
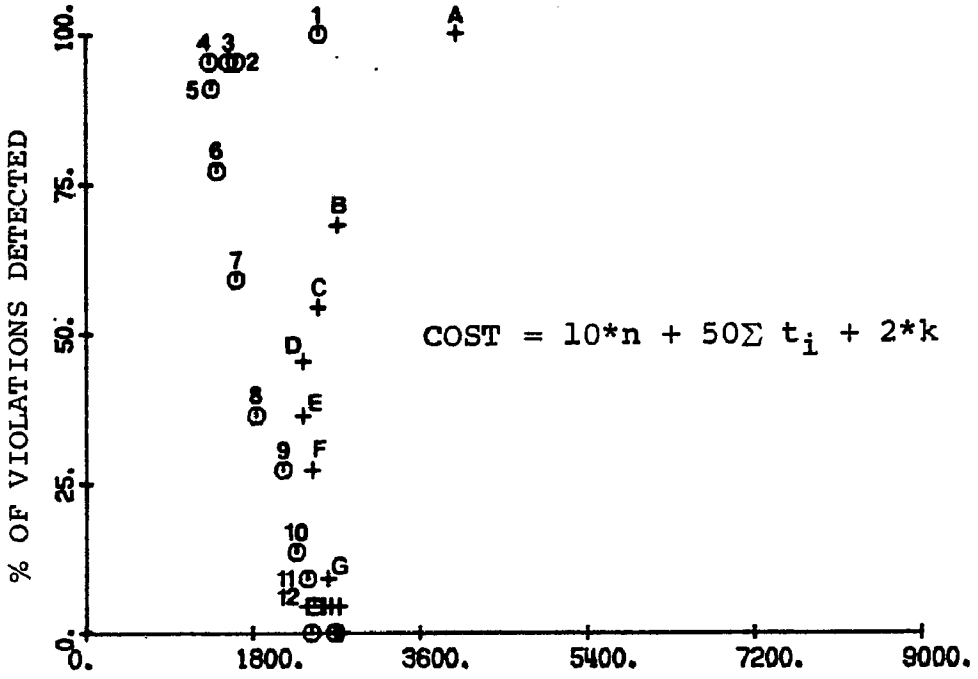


FIGURE D-10

TOTAL COST VS. % OF VIOLATIONS DETECTED

1973-74 PREDICTED STREAMFLOWS

CHLORIDE STANDARD = 55 MG/L



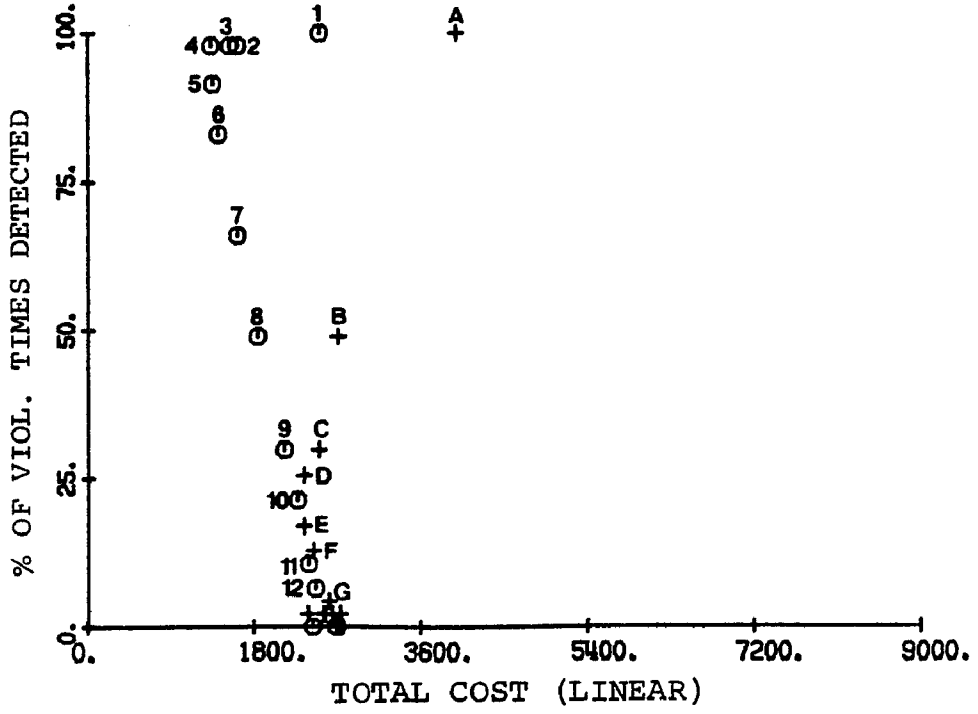
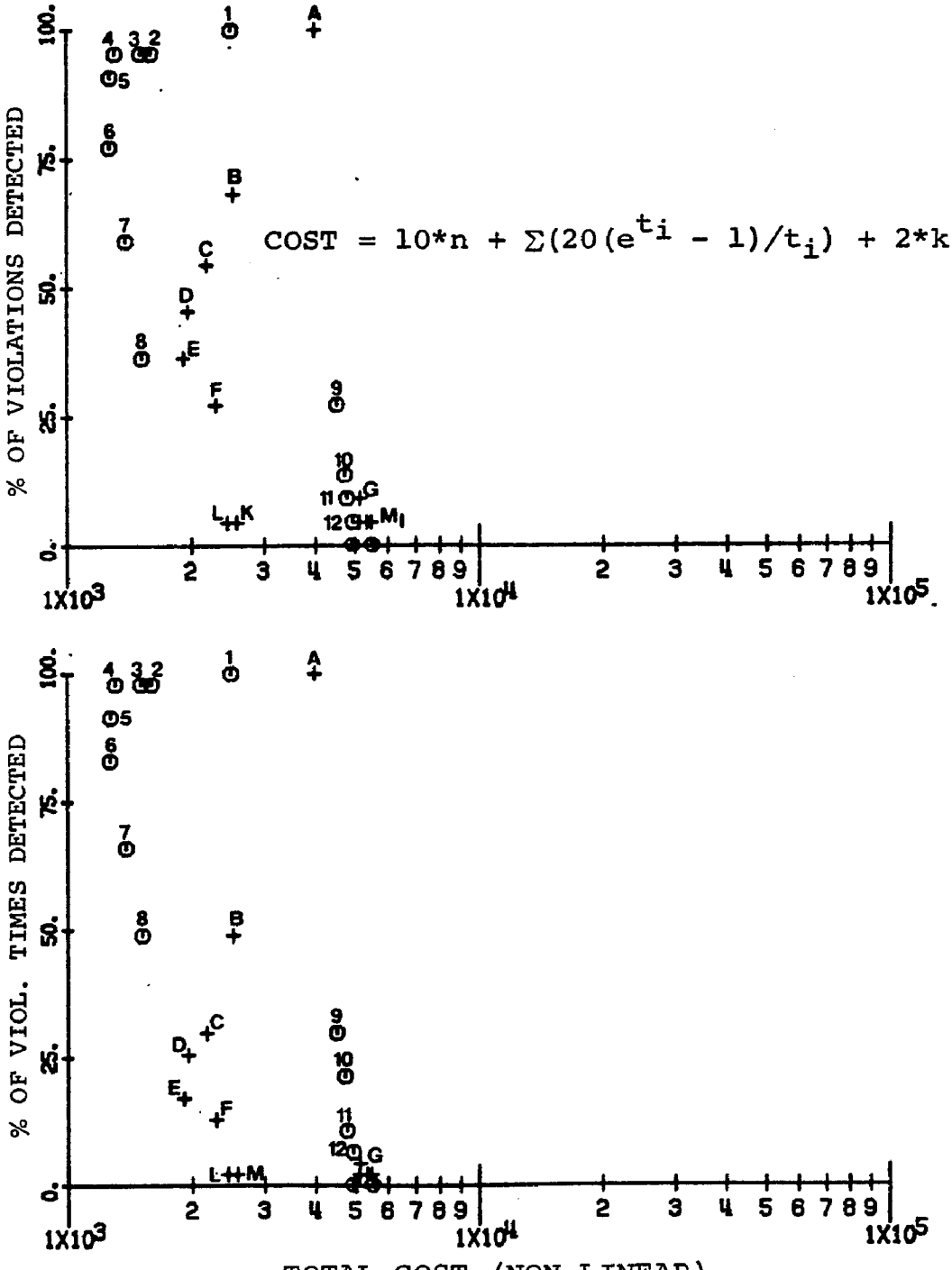


FIGURE D-11

TOTAL COST VS. % OF VIOLATIONS DETECTED

1973-74 PREDICTED STREAMFLOWS

CHLORIDE STANDARD = 55 MG/L



TOTAL COST (NON-LINEAR)

FIGURE D-12

APPENDIX E

Detection and Cost Tables for Variable and Fixed-Time Policies

TABLE E-1

1973-74 CHLORIDE PROFILE; U.S.G.S. STREAMFLOWS; STANDARD = 45 MG/L

PLAN	SAMPLE IF MODEL >	SAMPLES REQUIRED	% VIOL. TIMES DETECTED	% VIOLATIONS DETECTED	FIXED COST	VARIABLE COST (EXPONENTIAL)	TOTAL COST (EXPONENTIAL)	VARIABLE COST (LINEAR)	TOTAL COST (LINEAR)
1	29	321	100.0	100.0	3210.	0.	3210.	0.	3210.
2	33	295	100.0	100.0	2950.	0.	2950.	0.	2950.
3	35	270	98.8	97.9	2700.	34.	2734.	50.	2750.
4	36	255	97.5	93.8	2550.	132.	2682.	200.	2750.
5	37	235	94.4	89.6	2350.	362.	2712.	550.	2900.
6	38	213	91.4	81.3	2130.	464.	2594.	700.	2830.
7	39	191	86.4	68.8	1910.	694.	2604.	1050.	2960.
8	40	171	82.7	60.4	1710.	920.	2630.	1400.	3110.
9	42	140	72.8	39.6	1400.	1509.	2909.	2150.	3550.
10	43	134	72.2	39.6	1340.	1509.	2849.	2150.	3490.
11	45	118	65.4	35.4	1180.	1914.	3094.	2600.	3780.
12	46	106	59.3	31.3	1060.	2631.	3691.	3000.	4060.
13	47	92	52.5	29.2	920.	5108.	6028.	3450.	4370.
14	49	64	37.7	16.7	640.	9766.	10406.	4700.	5340.
15	51	37	22.2	16.7	370.	51211.	51581.	5600.	5970.
16	55	1	1.0	2.1	10.	271165.	271175.	8050.	8060.

1974-75 CHLORIDE PROFILE; U.S.G.S. STREAMFLOWS; STANDARD = 45 MG/L

1	29	320	100.0	100.0	3200.	0.	3200.	0.	3200.
2	33	274	99.4	97.7	2740.	34.	2774.	50.	2790.
3	35	243	97.0	88.6	2430.	170.	2600.	250.	2680.
4	36	237	96.4	86.4	2370.	204.	2574.	300.	2670.
5	37	222	95.8	84.1	2220.	238.	2458.	350.	2570.
6	38	209	94.6	79.5	2090.	306.	2396.	450.	2540.
7	39	195	91.0	68.2	1950.	506.	2456.	750.	2700.
8	40	177	85.6	59.1	1770.	803.	2573.	1150.	2920.
9	42	150	77.8	47.7	1500.	1062.	2562.	1500.	3000.
10	43	128	68.3	36.4	1280.	1454.	2734.	2100.	3380.
11	45	104	56.9	29.5	1040.	5244.	6284.	3050.	4090.
12	46	92	51.5	25.0	920.	8296.	9216.	3400.	4320.
13	47	77	44.9	20.5	770.	12127.	12897.	4000.	4770.
14	49	46	27.5	13.6	460.	18545.	19005.	5350.	5810.
15	51	26	15.6	4.5	260.	104397.	104657.	6600.	6860.
16	55	0	0.0	0.0	0.	173904.	173904.	8350.	8350.

TABLE E-2

1973-74 CHLORIDE PROFILE; U.S.G.S. STREAMFLOWS; STANDARD = 50 MG/L

PLAN	SAMPLE IF MODEL >	SAMPLES REQUIRED	% VIOL. TIMES DETECTED	% VIOLATIONS DETECTED	FIXED COST	VARIABLE COST (EXPONENTIAL)	TOTAL COST (EXPONENTIAL)	VARIABLE COST (LINEAR)	TOTAL COST (LINEAR)
1	34	285	100.0	100.0	2850.	0.	2850.	0.	2850.
2	38	213	97.9	95.0	2130.	68.	2198.	100.	2230.
3	40	171	95.9	90.0	1710.	320.	2030.	250.	1960.
4	41	154	92.8	82.5	1540.	272.	1812.	400.	1940.
5	42	140	90.7	77.5	1400.	340.	1740.	500.	1900.
6	43	134	90.7	77.5	1340.	340.	1680.	500.	1840.
7	44	130	89.7	77.5	1300.	374.	1674.	550.	1850.
8	45	118	83.5	70.0	1180.	506.	1686.	750.	1930.
9	47	92	71.1	55.0	920.	931.	1851.	1350.	2270.
10	48	79	60.8	45.0	790.	1254.	2044.	1800.	2590.
11	50	55	44.3	27.5	550.	1967.	2517.	2550.	3100.
12	51	37	30.9	20.0	370.	9422.	9792.	3000.	3370.
13	52	23	18.6	15.0	230.	20631.	20861.	3400.	3630.
14	54	6	4.1	7.5	60.	69323.	69383.	4600.	4660.
15	56	0	0.0	0.0	0.	69772.	69772.	4850.	4850.
16	60	0	0.0	0.0	0.	69772.	69772.	4850.	4850.

1974-75 CHLORIDE PROFILE; U.S.G.S. STREAMFLOWS; STANDARD = 50 MG/L

1	34	260	100.0	100.0	2600.	0.	2600.	0.	2600.
2	38	209	97.9	95.3	2090.	68.	2158.	100.	2190.
3	40	177	93.8	88.4	1770.	200.	1970.	300.	2070.
4	41	166	93.8	88.4	1660.	200.	1860.	300.	1960.
5	42	150	90.7	86.0	1500.	268.	1768.	400.	1900.
6	43	128	79.4	69.8	1280.	663.	1943.	950.	2230.
7	44	116	76.3	62.8	1160.	765.	1925.	1100.	2260.
8	45	104	70.1	55.8	1040.	927.	1967.	1350.	2390.
9	47	77	58.8	51.2	770.	1283.	2053.	1800.	2570.
10	48	60	48.5	44.2	600.	1601.	2201.	2200.	2800.
11	50	34	26.8	18.6	340.	5955.	6295.	3350.	3690.
12	51	26	21.6	14.0	260.	6613.	6873.	3700.	3960.
13	52	21	18.6	11.6	210.	6700.	6910.	3800.	4010.
14	54	0	0.0	0.0	0.	10885.	10885.	4850.	4850.
15	56	0	0.0	0.0	0.	10885.	10885.	4850.	4850.
16	60	0	0.0	0.0	0.	10885.	10885.	4850.	4850.

TABLE E-3

1973-74 CHLORIDE PROFILE; U.S.G.S. STREAMFLOWS; STANDARD = 55 MG/L

PLAN	SAMPLE IF MODEL ▲	SAMPLES REQUIRED	% VIOL. TIMES DETECTED	% VIOLATIONS DETECTED	FIXED COST	VARIABLE COST (EXPONENTIAL)	TOTAL COST (EXPONENTIAL)	VARIABLE COST (LINEAR)	TOTAL COST (LINEAR)
1	39	191	100.0	100.0	1910.	0.	1910.	0.	1910.
2	43	134	93.8	90.6	1340.	102.	1442.	150.	1490.
3	45	118	89.6	84.4	1180.	170.	1350.	250.	1430.
4	46	106	85.4	78.1	1060.	238.	1298.	350.	1410.
5	47	92	83.3	75.0	920.	272.	1192.	400.	1320.
6	48	79	77.1	71.9	790.	374.	1164.	550.	1340.
7	49	64	66.7	56.3	640.	536.	1176.	800.	1440.
8	50	55	60.4	53.1	550.	638.	1188.	950.	1500.
9	52	23	27.1	25.0	230.	1152.	1382.	1650.	1880.
10	53	12	12.5	12.5	120.	1415.	1535.	2000.	2120.
11	55	1	0.0	0.0	10.	1844.	1854.	2400.	2410.
12	56	0	0.0	0.0	0.	1844.	1844.	2400.	2400.
13	57	0	0.0	0.0	0.	1844.	1844.	2400.	2400.
14	59	0	0.0	0.0	0.	1844.	1844.	2400.	2400.
15	61	0	0.0	0.0	0.	1844.	1844.	2400.	2400.
16	65	0	0.0	0.0	0.	1844.	1844.	2400.	2400.

1974-75 CHLORIDE PROFILE; U.S.G.S. STREAMFLOWS; STANDARD = 55 MG/L

1	39	195	98.1	96.9	1950.	34.	1984.	50.	2000.
2	43	128	83.3	75.0	1280.	302.	1582.	450.	1730.
3	45	104	77.8	65.6	1040.	404.	1444.	600.	1640.
4	46	92	75.9	62.5	920.	438.	1358.	650.	1570.
5	47	77	68.5	59.4	770.	540.	1310.	800.	1570.
6	48	60	55.6	50.0	600.	736.	1336.	1100.	1700.
7	49	46	42.6	34.4	460.	991.	1451.	1450.	1910.
8	50	34	35.2	21.9	340.	1234.	1574.	1650.	1990.
9	52	21	29.6	15.6	210.	1302.	1512.	1750.	1960.
10	53	11	16.7	12.5	110.	1498.	1608.	2050.	2160.
11	55	0	0.0	0.0	0.	2941.	2941.	2700.	2700.
12	56	0	0.0	0.0	0.	2941.	2941.	2700.	2700.
13	57	0	0.0	0.0	0.	2941.	2941.	2700.	2700.
14	59	0	0.0	0.0	0.	2941.	2941.	2700.	2700.
15	61	0	0.0	0.0	0.	2941.	2941.	2700.	2700.
16	65	0	0.0	0.0	0.	2941.	2941.	2700.	2700.

TABLE E-4

1973-74 CHLORIDE PROFILE; PREDICTED STREAMFLOWS; STANDARD = 45 MG/L

PLAN	SAMPLE IF MODEL >	SAMPLES REQUIRED	% VIOL. TIMES DETECTED	% VIOLATIONS DETECTED	FIXED COST (1)	FIXED COST (2)	VARIABLE COST (EXPONENTIAL)	TOTAL COST (EXPONENTIAL)	VARIABLE COST (LINEAR)	TOTAL COST (LINEAR)
1	28	333	100.0	100.0	3330.	320.	34.	3684.	50.	3700.
2	32	311	98.7	96.2	3110.	320.	136.	3566.	200.	3630.
3	34	289	96.8	92.5	2890.	320.	298.	3508.	450.	3660.
4	36	263	93.6	86.8	2630.	320.	328.	3278.	500.	3450.
5	37	247	92.4	84.9	2470.	320.	426.	3216.	650.	3440.
6	38	217	87.9	75.5	2170.	320.	630.	3120.	950.	3440.
7	39	190	82.2	62.3	1900.	320.	953.	3173.	1400.	3620.
8	40	163	77.7	52.8	1630.	320.	1183.	3133.	1750.	3700.
9	42	124	69.4	34.0	1240.	320.	1587.	3147.	2350.	3910.
10	43	122	68.2	34.0	1220.	320.	1587.	3127.	2350.	3890.
11	45	104	59.2	24.5	1040.	320.	1821.	3181.	2700.	4060.
12	47	82	49.7	22.6	820.	320.	2152.	3292.	2950.	4090.
13	48	68	41.4	15.1	680.	320.	3450.	4450.	3350.	4350.
14	49	54	33.8	11.3	540.	320.	24629.	25489.	4500.	5360.
15	51	29	19.1	7.5	290.	320.	85881.	86491.	5650.	6260.
16	55	6	4.5	1.9	60.	320.	201038.	201418.	6950.	7330.

1974-75 CHLORIDE PROFILE; PREDICTED STREAMFLOWS; STANDARD = 45 MG/L

1	28	335	100.0	100.0	3350.	170.	0.	3520.	0.	3520.
2	32	292	99.4	97.9	2920.	170.	68.	3150.	100.	3190.
3	34	267	97.5	95.8	2670.	170.	136.	2976.	200.	3040.
4	36	242	96.3	91.7	2420.	170.	170.	2760.	250.	2840.
5	37	229	93.2	83.3	2290.	170.	340.	2800.	500.	2960.
6	38	220	92.6	81.3	2200.	170.	370.	2740.	550.	2920.
7	39	202	88.9	72.9	2020.	170.	502.	2692.	750.	2940.
8	40	180	85.3	64.6	1800.	170.	799.	2769.	1150.	3120.
9	42	147	71.0	43.8	1470.	170.	4758.	6398.	2150.	3790.
10	43	124	64.8	39.6	1240.	170.	5090.	6500.	2450.	3860.
11	45	94	52.5	20.8	940.	170.	7602.	8712.	3750.	4860.
12	47	70	41.4	14.6	700.	170.	8095.	8965.	3900.	4770.
13	48	51	30.2	10.4	510.	170.	10844.	11524.	4550.	5230.
14	49	37	22.8	8.3	370.	170.	40175.	40715.	5750.	6290.
15	51	23	16.2	4.2	230.	170.	134426.	134826.	6300.	6700.
16	55	0	0.0	0.0	0.	170.	134426.	134596.	8100.	8270.

TABLE E-5

1973-74 CHLORIDE PROFILE; PREDICTED STREAMFLOWS; STANDARD = 50 MG/L

PLAN	SAMPLE IF MODEL >	SAMPLES REQUIRED	% VIOL. TIMES DETECTED	% VIOLATIONS DETECTED	FIXED COST (1)	FIXED COST (2)	VARIABLE COST (EXPONENTIAL)	TOTAL COST (EXPONENTIAL)	VARIABLE COST (LINEAR)	TOTAL COST (LINEAR)
1	33	300	100.0	100.0	3000.	320.	0.	3320.	0.	3320.
2	37	247	100.0	100.0	2470.	320.	34.	2824.	50.	2840.
3	39	190	97.8	93.5	1900.	320.	68.	2288.	100.	2320.
4	41	138	95.6	87.1	1380.	320.	136.	1836.	200.	1900.
5	42	124	93.3	80.6	1240.	320.	204.	1764.	300.	1860.
6	43	122	93.3	80.6	1220.	320.	204.	1744.	300.	1840.
7	44	115	92.2	77.4	1150.	320.	238.	1708.	350.	1820.
8	45	104	83.3	71.0	1040.	320.	438.	1798.	650.	2010.
9	47	82	72.2	61.3	820.	320.	786.	1926.	1100.	2240.
10	48	68	62.2	45.2	680.	320.	1161.	2161.	1500.	2500.
11	50	41	38.9	25.8	410.	320.	2770.	3500.	2400.	3130.
12	52	20	20.0	12.9	200.	320.	3585.	4105.	2550.	3070.
13	53	14	15.6	9.7	140.	320.	4098.	4558.	2850.	3310.
14	54	9	11.1	6.5	90.	320.	11115.	11525.	3200.	3610.
15	56	4	5.6	3.2	40.	320.	18769.	19129.	3700.	4060.
16	60	0	0.0	0.0	0.	320.	62552.	62872.	4500.	4820.

1974-75 CHLORIDE PROFILE; PREDICTED STREAMFLOWS; STANDARD = 50 MG/L

1	33	281	100.0	100.0	2810.	170.	0.	2980.	0.	2980.
2	37	229	99.0	97.4	2290.	170.	68.	2528.	100.	2560.
3	39	202	97.0	92.3	2020.	170.	170.	2360.	250.	2440.
4	41	162	91.1	76.9	1620.	170.	306.	2096.	450.	2240.
5	42	147	88.1	71.8	1470.	170.	374.	2014.	550.	2190.
6	43	124	81.2	61.5	1240.	170.	510.	1820.	750.	2160.
7	44	111	74.3	46.2	1110.	170.	871.	2151.	1250.	2530.
8	45	94	70.3	41.0	940.	170.	1003.	2113.	1450.	2560.
9	47	70	57.4	25.6	700.	170.	1394.	2264.	2000.	2870.
10	48	51	44.6	17.9	510.	170.	1776.	2456.	2500.	3180.
11	50	29	28.7	5.1	290.	170.	2468.	2928.	3050.	3510.
12	52	16	15.8	5.1	160.	170.	3716.	4046.	3250.	3580.
13	53	7	6.9	5.1	70.	170.	5739.	5979.	3450.	3690.
14	54	3	3.0	0.0	30.	170.	53842.	54042.	4800.	5000.
15	56	0	0.0	0.0	0.	170.	90443.	90613.	5050.	5220.
16	60	0	0.0	0.0	0.	170.	90443.	90613.	5050.	5220.

TABLE E-6

1973-74 CHLORIDE PROFILE: PREDICTED STREAMFLOWS: STANDARD = 55MG/L

PLAN	SAMPLE IF MODEL ^	SAMPLES REQUIRED	% VIOL. TIMES DETECTED	% VIOLATIONS DETECTED	FIXED COST (1)	FIXED COST (2)	VARIABLE COST (EXPONENTIAL)	TOTAL COST (EXPONENTIAL)	VARIABLE COST (LINEAR)	TOTAL COST (LINEAR)
1	38	217	100.0	100.0	2170.	320.	0.	2490.	0.	2490.
2	42	124	97.9	95.5	1240.	320.	34.	1594.	50.	1610.
3	44	115	97.9	95.5	1150.	320.	34.	1504.	50.	1520.
4	46	95	97.9	95.5	950.	320.	34.	1304.	50.	1320.
5	47	82	91.5	90.9	820.	320.	132.	1272.	200.	1340.
6	48	68	83.0	77.3	680.	320.	268.	1268.	400.	1400.
7	49	54	66.0	59.1	540.	320.	523.	1383.	750.	1610.
8	50	41	48.9	36.4	410.	320.	786.	1516.	1100.	1830.
9	52	20	29.8	27.3	200.	320.	3980.	4500.	1600.	2120.
10	53	14	21.3	13.6	140.	320.	4252.	4712.	1800.	2260.
11	55	6	10.6	9.1	60.	320.	4380.	4760.	2000.	2380.
12	57	3	6.4	4.5	30.	320.	4584.	4934.	2100.	2450.
13	58	0	0.0	0.0	0.	320.	4584.	4904.	2100.	2420.
14	59	0	0.0	0.0	0.	320.	5174.	5494.	2350.	2670.
15	61	0	0.0	0.0	0.	320.	5174.	5494.	2350.	2670.
16	65	0	0.0	0.0	0.	320.	5174.	5494.	2350.	2670.

1974-75 CHLORIDE PROFILE: PREDICTED STREAMFLOWS: STANDARD = 55 MG/L

1	38	220	97.6	96.2	2200.	170.	34.	2404.	50.	2420.
2	42	147	90.5	84.6	1470.	170.	136.	1776.	200.	1840.
3	44	111	90.5	84.6	1110.	170.	136.	1416.	200.	1480.
4	46	88	85.7	80.8	880.	170.	204.	1254.	300.	1350.
5	47	70	78.6	73.1	700.	170.	268.	1138.	400.	1270.
6	48	51	69.0	61.5	510.	170.	434.	1114.	650.	1330.
7	49	37	52.4	38.5	370.	170.	697.	1237.	1000.	1540.
8	50	29	47.6	30.8	290.	170.	765.	1225.	1100.	1560.
9	52	16	26.2	23.1	160.	170.	1058.	1388.	1500.	1830.
10	53	7	7.1	7.7	70.	170.	1916.	2156.	1950.	2190.
11	55	0	0.0	0.0	0.	170.	2121.	2291.	2100.	2270.
12	57	0	0.0	0.0	0.	170.	2121.	2291.	2100.	2270.
13	58	0	0.0	0.0	0.	170.	2121.	2291.	2100.	2270.
14	59	0	0.0	0.0	0.	170.	2121.	2291.	2100.	2270.
15	61	0	0.0	0.0	0.	170.	2121.	2291.	2100.	2270.
16	65	0	0.0	0.0	0.	170.	2121.	2291.	2100.	2270.

TABLE E-7

1973-74 CHLORIDE PROFILE; U.S.G.S. STREAMFLOWS; STANDARD = 45 MG/L

PLAN	SAMPLING INTERVAL	SAMPLES REQUIRED	% VIOL. TIMES DETECTED	% VIOLATIONS DETECTED	FIXED COST	VARIABLE COST (EXPONENTIAL)	TOTAL COST (EXPONENTIAL)	VARIABLE COST (LINEAR)	TOTAL COST (LINEAR)
A	1	365	100.0	100.0	3650.	0.	3650.	0.	3650.
B	2	183	51.9	77.1	1830.	748.	2578.	1100.	2930.
C	3	122	32.7	62.5	1220.	1350.	2570.	2050.	3270.
D	4	91	25.9	52.1	910.	1543.	2453.	2200.	3110.
E	6	61	17.3	39.6	610.	2522.	3132.	3250.	3860.
F	9	41	10.5	27.1	410.	10408.	10818.	3950.	4360.
G	15	24	5.6	16.7	240.	82328.	82568.	5950.	6140.
H	23	16	4.3	14.6	160.	183301.	183461.	6250.	6410.
I	32	11	3.7	12.5	110.	157162.	157272.	6150.	6260.
J	46	8	1.2	4.2	80.	227148.	227228.	7550.	7630.
K	59	6	1.2	4.2	60.	227182.	227242.	6900.	6960.
L	74	5	1.9	6.3	50.	271131.	271181.	7350.	7400.
M	95	4	1.2	2.1	40.	227148.	227188.	6900.	6940.
N	126	3	1.2	2.1	30.	269922.	269952.	7900.	7930.
O	174	2	0.0	0.0	20.	271199.	271219.	8100.	8120.
P	336	1	0.0	0.0	10.	271199.	271209.	8100.	8110.

1974-75 CHLORIDE PROFILE: U.S.G.S. STREAMFLOWS; STANDARD = 45 MG/L

A	1	365	100.0	100.0	3650.	0.	3650.	0.	3650.
B	2	183	53.3	77.3	1830.	544.	2374.	800.	2630.
C	3	122	31.7	56.8	1220.	1260.	2480.	1900.	3120.
D	4	91	26.3	54.5	910.	1442.	2352.	2100.	3010.
E	6	61	17.4	40.9	610.	3436.	4046.	3450.	4060.
F	9	41	10.8	29.5	410.	11222.	11632.	4150.	4560.
G	15	24	7.2	25.0	240.	55012.	55252.	4900.	5140.
H	23	16	3.0	11.4	160.	126846.	127004.	6800.	6960.
I	32	11	3.0	11.4	110.	170914.	171024.	7750.	7860.
J	46	8	1.2	4.5	80.	168264.	168344.	7950.	8030.
K	59	6	2.4	9.1	60.	173836.	173896.	8250.	8310.
L	74	5	0.0	0.0	50.	173904.	173954.	8350.	8400.
M	95	4	1.0	2.3	40.	173636.	173676.	8150.	8190.
N	126	3	0.0	0.0	30.	173904.	173934.	8350.	8380.
O	174	2	0.0	0.0	20.	173904.	173924.	8350.	8370.
P	336	1	1.0	2.3	10.	173904.	173914.	8350.	8360.

TABLE E-8

1973-74 CHLORIDE PROFILE; U.S.G.S. STREAMFLOWS; STANDARD = 50 MG/L

PLAN	SAMPLING INTERVAL	SAMPLES REQUIRED	% VIOL. TIMES DETECTED	% VIOLATIONS DETECTED	FIXED COST	VARIABLE COST (EXPONENTIAL)	TOTAL COST (EXPONENTIAL)	VARIABLE COST (LINEAR)	TOTAL COST (LINEAR)
A	1	365	100.0	100.0	3650.	0.	3650.	0.	3650.
B	2	183	54.6	82.5	1830.	476.	2306.	700.	2530.
C	3	122	28.9	52.5	1220.	1414.	2634.	2150.	3370.
D	4	91	26.8	55.0	910.	1152.	2062.	1650.	2560.
E	6	61	18.6	40.0	610.	2365.	2975.	2700.	3310.
F	9	41	9.3	22.5	410.	4834.	5244.	3300.	3710.
G	15	24	5.2	15.0	240.	31674.	31914.	4100.	4340.
H	23	16	5.2	12.5	160.	28012.	28172.	4150.	4310.
I	32	11	4.1	10.0	110.	68508.	68618.	4500.	4610.
J	47	8	1.0	2.5	80.	69309.	69389.	4750.	4830.
K	60	6	1.0	2.5	60.	28851.	28911.	4650.	4710.
L	74	5	2.1	5.0	50.	25814.	25864.	4400.	4450.
M	96	4	1.0	2.5	40.	69645.	69685.	4700.	4740.
N	128	3	1.0	2.5	30.	69738.	69768.	4800.	4830.
O	176	2	0.0	0.0	20.	69772.	69792.	4850.	4870.
P	340	1	0.0	0.0	10.	69772.	69782.	4850.	4860.

1974-75 CHLORIDE PROFILE; U.S.G.S. STREAMFLOWS; STANDARD = 50 MG/L

A	1	365	100.0	100.0	3650.	0.	3650.	0.	3650.
B	2	183	53.6	74.4	1830.	646.	2476.	950.	2780.
C	3	122	33.0	60.5	1220.	1320.	2540.	2000.	3220.
D	4	91	28.9	39.5	910.	1068.	1978.	1600.	2510.
E	6	61	17.5	34.9	610.	2530.	3140.	2900.	3510.
F	9	41	11.3	25.6	410.	3759.	4169.	3700.	4110.
G	15	24	7.2	16.3	240.	8077.	8317.	4150.	4390.
H	23	16	1.0	2.3	160.	10617.	10777.	4650.	4810.
I	32	11	2.1	7.0	110.	7862.	7972.	4500.	4610.
J	47	8	0.0	2.3	80.	10851.	10931.	4800.	4880.
K	60	6	1.0	2.3	60.	10617.	10677.	4650.	4710.
L	74	5	0.0	0.0	50.	10885.	10935.	4850.	4900.
M	96	4	1.0	2.3	40.	10851.	10891.	4800.	4840.
N	128	3	0.0	0.0	30.	10885.	10915.	4850.	4880.
O	176	2	1.0	2.3	20.	7882.	7902.	4650.	4670.
P	340	1	1.0	2.3	10.	10821.	10831.	4750.	4760.

TABLE E-9

1973-74 CHLORIDE PROFILE; U.S.G.S. STREAMFLOWS; STANDARD = 55 MG/L

PLAN	SAMPLING INTERVAL	SAMPLES REQUIRED	% VIOL. TIMES DETECTED	% VIOLATIONS DETECTED	FIXED COST	VARIABLE COST (EXPONENTIAL)	TOTAL COST (EXPONENTIAL)	VARIABLE COST (LINEAR)	TOTAL COST (LINEAR)
A	1	365	100.0	100.0	3650.	0.	3650.	0.	3650.
B	2	183	56.3	71.9	1830.	510.	2340.	750.	2580.
C	3	122	29.2	43.8	1220.	838.	2058.	1250.	2470.
D	4	91	31.3	46.9	910.	931.	1841.	1350.	2260.
E	6	61	20.8	31.3	610.	1203.	1813.	1550.	2160.
F	9	41	10.4	15.6	410.	1453.	1863.	2200.	2610.
G	16	23	6.3	9.4	230.	1746.	1976.	2250.	2480.
H	25	15	4.2	6.3	150.	1624.	1774.	2150.	2300.
I	34	11	0.0	0.0	110.	1844.	1954.	2400.	2510.
J	49	7	0.0	0.0	70.	1844.	1914.	2400.	2470.
K	63	6	2.1	3.1	60.	1751.	1811.	2300.	2360.
L	78	5	0.0	0.0	50.	1844.	1894.	2400.	2450.
M	100	4	0.0	0.0	40.	1844.	1884.	2400.	2440.
N	134	3	0.0	0.0	30.	1844.	1874.	2400.	2430.
O	185	2	0.0	0.0	20.	1844.	1864.	2400.	2420.
P	357	1	2.1	3.1	10.	1810.	1820.	2350.	2360.

1974-75 CHLORIDE PROFILE; U.S.G.S. STREAMFLOWS; STANDARD = 55 MG/L

A	1	365	100.0	100.0	3650.	0.	3650.	0.	3650.
B	2	183	46.3	65.6	1830.	544.	2374.	800.	2630.
C	3	122	31.5	46.9	1220.	966.	2186.	1450.	2670.
D	4	91	33.3	56.3	910.	735.	1645.	1050.	1960.
E	6	61	11.1	18.8	610.	1955.	2565.	2050.	2660.
F	9	41	9.3	15.6	410.	2049.	2459.	2200.	2610.
G	16	23	5.6	9.4	230.	2746.	2976.	2450.	2680.
H	25	15	3.7	6.3	150.	2257.	2407.	2300.	2450.
I	34	11	3.7	6.3	110.	2844.	2954.	2600.	2710.
J	49	7	1.9	3.1	70.	2911.	2981.	2650.	2720.
K	63	6	0.0	0.0	60.	2941.	3001.	2700.	2760.
L	78	5	1.9	3.1	50.	2877.	2927.	2600.	2650.
M	100	4	0.0	0.0	40.	2941.	2981.	2700.	2740.
N	134	3	0.0	0.0	30.	2941.	2971.	2700.	2730.
O	185	2	0.0	0.0	20.	2941.	2961.	2700.	2720.
P	357	1	0.0	0.0	10.	2941.	2951.	2700.	2710.

TABLE E-10

1973-74 CHLORIDE PROFILE; PREDICTED STREAMFLOWS; STANDARD = 45 MG/L

PLAN	SAMPLING INTERVAL	SAMPLES REQUIRED	% VIOL. TIMES DETECTED	% VIOLATIONS DETECTED	FIXED COST (1)	FIXED COST (2)	VARIABLE COST (EXPONENTIAL)	TOTAL COST (EXPONENTIAL)	VARIABLE COST (LINEAR)	TOTAL COST (LINEAR)
A	1	365	100.0	100.0	3650.	320.	0.	3970.	0.	3970.
B	2	183	52.9	79.2	1830.	320.	646.	2796.	950.	3100.
C	3	122	31.2	50.9	1220.	320.	1430.	2970.	2150.	3690.
D	4	91	26.1	49.1	910.	320.	1867.	3097.	2700.	3930.
E	6	61	16.6	37.7	610.	320.	3235.	4165.	3500.	4430.
F	9	41	10.8	28.3	410.	320.	14196.	14926.	4550.	5280.
G	15	24	7.6	20.8	240.	320.	13162.	13722.	4550.	5110.
H	23	16	5.1	15.1	160.	320.	68181.	68661.	5400.	5880.
I	32	11	4.5	13.2	110.	320.	141726.	142156.	6850.	7280.
J	46	8	1.9	5.7	80.	320.	156041.	156441.	7250.	7650.
K	59	6	1.3	3.8	60.	320.	157255.	157635.	6650.	7030.
L	74	5	1.3	3.8	50.	320.	157577.	157947.	6700.	7070.
M	95	4	1.9	5.7	40.	320.	201004.	201364.	7800.	8160.
N	126	3	1.3	3.8	30.	320.	182507.	182857.	7250.	7600.
O	174	2	1.0	1.9	20.	320.	201038.	201378.	7850.	8190.
P	336	1	1.0	1.9	10.	320.	201038.	201368.	7850.	8180.

1974-75 CHLORIDE PROFILE; PREDICTED STREAMFLOWS; STANDARD = 45 MG/L

A	1	365	100.0	100.0	3650.	170.	0.	3820.	0.	3820.
B	2	183	47.5	64.6	1830.	170.	986.	2986.	1450.	3450.
C	3	122	34.6	62.5	1220.	170.	1366.	2756.	2050.	3440.
D	4	91	22.2	39.6	910.	170.	2035.	3115.	2850.	3930.
E	6	61	17.9	45.8	610.	170.	3591.	4371.	3250.	4030.
F	9	41	13.6	35.4	410.	170.	3872.	4452.	3650.	4230.
G	15	24	7.4	18.8	240.	170.	14634.	15044.	4450.	4860.
H	23	16	4.3	12.5	160.	170.	53616.	53946.	5450.	5780.
I	32	11	3.1	8.3	110.	170.	46357.	46636.	5300.	5580.
J	46	8	1.9	6.3	80.	170.	64298.	64548.	5650.	5900.
K	59	6	2.5	8.3	60.	170.	88487.	88717.	6150.	6380.
L	74	5	1.0	2.1	50.	170.	90374.	90594.	6350.	6570.
M	95	4	1.0	2.1	40.	170.	90374.	90584.	6350.	6560.
N	126	3	1.0	4.2	30.	170.	90310.	90510.	6250.	6450.
O	174	2	1.0	4.2	20.	170.	87244.	87434.	6000.	6190.
P	336	1	1.0	2.1	10.	170.	90374.	90554.	6350.	6530.

TABLE E-11

1973-74 CHLORIDE PROFILE; PREDICTED STREAMFLOWS; STANDARD = 50 MG/L

PLAN	SAMPLING INTERVAL	SAMPLES REQUIRED	% VIOL. TIMES DETECTED	% VIOLATIONS DETECTED	FIXED COST (1)	FIXED COST (2)	VARIABLE COST (EXPONENTIAL)	TOTAL COST (EXPONENTIAL)	VARIABLE COST (LINEAR)	TOTAL COST (LINEAR)
A	1	365	100.0	100.0	3650.	320.	0.	3970.	0.	3970.
B	2	183	50.0	71.0	1830.	320.	544.	2694.	800.	2950.
C	3	122	31.1	54.8	1220.	320.	890.	2430.	1350.	2890.
D	4	91	23.3	45.2	910.	320.	1374.	2604.	2000.	3230.
E	6	61	14.4	32.3	610.	320.	1730.	2660.	2250.	3180.
F	9	41	11.1	29.0	410.	320.	4689.	5419.	2500.	3230.
G	15	24	10.0	25.8	240.	320.	3665.	4225.	2800.	3360.
H	23	16	5.6	16.1	160.	320.	18373.	18853.	3300.	3780.
I	32	11	3.3	9.7	110.	320.	56268.	56698.	4300.	4730.
J	47	8	1.1	3.2	80.	320.	55692.	56092.	4350.	4750.
K	60	6	2.2	6.5	60.	320.	17258.	17638.	3350.	3730.
L	74	5	1.1	3.2	50.	320.	62459.	62829.	4400.	4770.
M	96	4	1.1	3.2	40.	320.	62348.	62708.	4400.	4760.
N	128	3	0.0	0.0	30.	320.	62552.	62902.	4500.	4850.
O	176	2	0.0	0.0	20.	320.	62552.	62892.	4500.	4840.
P	340	1	0.0	0.0	10.	320.	62552.	62882.	4500.	4830.

1974-75 CHLORIDE PROFILE; PREDICTED STREAMFLOWS; STANDARD = 50 MG/L

PLAN	SAMPLING INTERVAL	SAMPLES REQUIRED	% VIOL. TIMES DETECTED	% VIOLATIONS DETECTED	FIXED COST (1)	FIXED COST (2)	VARIABLE COST (EXPONENTIAL)	TOTAL COST (EXPONENTIAL)	VARIABLE COST (LINEAR)	TOTAL COST (LINEAR)
A	1	365	100.0	100.0	3650.	170.	0.	3820.	0.	3720.
B	2	183	49.5	69.2	1830.	170.	646.	2646.	950.	2950.
C	3	122	35.6	59.0	1220.	170.	1026.	2416.	1550.	2940.
D	4	91	18.8	28.2	910.	170.	1732.	2812.	2350.	3430.
E	6	61	18.8	35.9	610.	170.	1619.	2399.	2100.	2880.
F	9	41	10.9	20.5	410.	170.	2146.	2726.	2650.	3230.
G	15	24	5.9	12.8	240.	170.	53506.	53916.	3500.	3910.
H	23	16	4.0	10.3	160.	170.	46090.	46420.	3550.	3880.
I	32	11	3.0	7.7	110.	170.	90111.	90391.	4600.	4880.
J	47	8	3.0	7.7	80.	170.	49220.	49470.	4500.	4750.
K	60	6	1.0	2.6	60.	170.	90175.	90405.	4850.	5080.
L	74	5	1.0	2.6	50.	170.	90175.	90395.	4850.	5070.
M	96	4	2.0	5.1	40.	170.	90145.	90355.	4800.	5010.
N	128	3	1.0	2.6	30.	170.	90175.	90375.	4850.	5050.
O	176	2	1.0	2.6	20.	170.	90175.	90365.	4850.	5040.
P	340	1	2.0	5.1	10.	170.	90112.	90292.	4800.	4980.

TABLE E-12

1973-74 CHLORIDE PROFILE; PREDICTED STREAMFLOWS; STANDARD = 55 MG/L

PLAN	SAMPLING INTERVAL	SAMPLES REQUIRED	% VIOL. TIMES DETECTED	% VIOLATIONS DETECTED	FIXED COST (1)	FIXED COST (2)	VARIABLE COST (EXPONENTIAL)	TOTAL COST (EXPONENTIAL)	VARIABLE COST (LINEAR)	TOTAL COST (LINEAR)
A	1	365	100.0	100.0	3650.	320.	0.	3970.	0.	3970.
B	2	183	48.9	68.2	1830.	320.	374.	2524.	550.	2700.
C	3	122	29.8	54.5	1220.	320.	634.	2174.	950.	2490.
D	4	91	25.5	45.5	910.	320.	728.	1958.	1100.	2330.
E	6	61	17.0	36.4	610.	320.	982.	1912.	1400.	2330.
F	9	41	12.8	27.3	410.	320.	1561.	2291.	1700.	2430.
G	16	23	4.3	9.1	230.	320.	4580.	5130.	2050.	2600.
H	25	15	2.1	4.5	150.	320.	5000.	5470.	2250.	2720.
I	34	11	2.1	4.5	110.	320.	4678.	5108.	2200.	2630.
J	49	7	0.0	0.0	70.	320.	5204.	5594.	2350.	2740.
K	63	6	2.1	4.5	60.	320.	2201.	2581.	2150.	2530.
L	78	5	2.1	4.5	50.	320.	2074.	2444.	2000.	2370.
M	100	4	2.1	4.5	40.	320.	4970.	5330.	2200.	2560.
N	134	3	0.0	0.0	30.	320.	5204.	5554.	2350.	2700.
O	185	2	0.0	0.0	20.	320.	5204.	5544.	2350.	2690.
P	357	1	0.0	0.0	10.	320.	5204.	5534.	2350.	2680.

1974-75 CHLORIDE PROFILE; PREDICTED STREAMFLOWS; STANDARD = 55 MG/L

A	1	365	100.0	100.0	3650.	170.	0.	3820.	0.	3820.
B	2	183	42.9	50.0	1830.	170.	578.	2578.	850.	2850.
C	3	122	28.6	46.2	1220.	170.	736.	2126.	1100.	2490.
D	4	91	21.4	34.6	910.	170.	1024.	2104.	1450.	2530.
E	6	61	7.1	11.5	610.	170.	1726.	2506.	1750.	2530.
F	9	41	9.5	15.4	410.	170.	1434.	2014.	1750.	2330.
G	16	23	2.4	3.8	230.	170.	2087.	2487.	2050.	2450.
H	25	15	7.1	11.5	150.	170.	1229.	1549.	1600.	1920.
I	34	11	0.0	0.0	110.	170.	2121.	2401.	2100.	2380.
J	49	7	2.4	3.8	70.	170.	2087.	2327.	2050.	2290.
K	63	6	2.4	3.8	60.	170.	2087.	2317.	2050.	2280.
L	78	5	0.0	0.0	50.	170.	2121.	2341.	2100.	2320.
M	100	4	0.0	0.0	40.	170.	2121.	2331.	2100.	2310.
N	134	3	4.8	7.7	30.	170.	2053.	2253.	2000.	2200.
O	185	2	0.0	0.0	20.	170.	2121.	2311.	2100.	2290.
P	357	1	0.0	0.0	10.	170.	2121.	2301.	2100.	2280.

TABLE E-13

1974-75 CHLORIDE PROFILE; U.S.G.S. STREAMFLOWS; STANDARD = 45 MG/L

PLAN	SAMPLE IF MODEL >	SAMPLES REQUIRED	% VIOL. TIMES DETECTED	% VIOLATIONS DETECTED	FIXED COST (1)	FIXED COST (2)	VARIABLE COST (EXPONENTIAL)	TOTAL COST (EXPONENTIAL)	VARIABLE COST (LINEAR)	TOTAL COST (LINEAR)
1	29	320	99.1	100.0	3200.	0.	0.	3200.	0.	3200.
2	33	274	98.1	98.1	2740.	0.	64.	2804.	100.	2840.
3	35	243	97.2	96.3	2430.	0.	98.	2528.	150.	2580.
4	36	237	97.2	96.3	2370.	0.	98.	2468.	150.	2520.
5	37	222	97.2	96.3	2220.	0.	98.	2318.	150.	2370.
6	38	209	95.3	94.4	2090.	0.	166.	2256.	250.	2340.
7	39	195	91.5	92.6	1950.	0.	293.	2243.	400.	2350.
8	40	177	87.7	87.0	1770.	0.	454.	2224.	600.	2370.
9	42	150	79.2	74.1	1500.	0.	752.	2252.	1050.	2550.
10	43	128	72.6	66.7	1280.	0.	978.	2258.	1400.	2680.
11	45	104	61.3	57.4	1040.	0.	1276.	2316.	1850.	2890.
12	46	92	52.8	44.4	920.	0.	8862.	9782.	2450.	3370.
13	47	77	44.3	38.9	770.	0.	8964.	9734.	2600.	3370.
14	49	46	34.0	27.8	460.	0.	9788.	10248.	3200.	3660.
15	51	26	19.8	14.8	260.	0.	10430.	10690.	4100.	4360.
16	55	0	0.0	0.0	0.	0.	13454.	13454.	5300.	5300.

1974-75 CHLORIDE PROFILE; PREDICTED STREAMFLOWS; STANDARD = 45 MG/L

1	28	335	99.1	98.2	3350.	170.	34.	3554.	50.	3570.
2	32	292	97.3	94.7	2920.	170.	102.	3192.	150.	3240.
3	34	267	96.4	93.0	2670.	170.	136.	2976.	200.	3040.
4	36	242	95.5	91.2	2420.	170.	170.	2760.	250.	2840.
5	37	229	93.7	87.7	2290.	170.	238.	2698.	350.	2810.
6	38	220	93.7	87.7	2200.	170.	238.	2608.	350.	2720.
7	39	202	85.6	78.9	2020.	170.	532.	2722.	800.	2990.
8	40	180	82.0	73.7	1800.	170.	804.	2774.	1000.	2970.
9	42	147	75.7	63.2	1470.	170.	1042.	2682.	1350.	2990.
10	43	124	68.5	54.4	1240.	170.	1208.	2618.	1600.	3010.
11	45	94	55.0	45.6	940.	170.	1786.	2896.	2400.	3510.
12	47	70	44.1	35.1	700.	170.	2313.	3183.	2950.	3820.
13	48	51	31.5	26.3	510.	170.	3045.	3725.	3650.	4330.
14	49	37	24.3	21.1	370.	170.	4450.	4990.	4050.	4590.
15	51	23	15.3	10.5	230.	170.	6532.	6932.	4500.	4900.
16	55	0	0.0	0.0	0.	170.	10711.	10881.	5550.	5720.

TABLE E-14

1974-75 CHLORIDE PROFILE; U.S.G.S. STREAMFLOWS; STANDARD = 50 MG/L

PLAN	SAMPLE IF MODEL >	SAMPLES REQUIRED	% VIOL. TIMES DETECTED	% VIOLATIONS DETECTED	FIXED COST (1)	FIXED COST (2)	VARIABLE COST (EXPONENTIAL)	TOTAL COST (EXPONENTIAL)	VARIABLE COST (LINEAR)	TOTAL COST (LINEAR)
1	34	260	100.0	100.0	2600.	0.	0.	2600.	0.	2600.
2	38	209	100.0	100.0	2090.	0.	0.	2090.	0.	2090.
3	40	177	100.0	100.0	1770.	0.	0.	1770.	0.	1770.
4	41	166	100.0	100.0	1660.	0.	0.	1660.	0.	1660.
5	42	150	100.0	100.0	1500.	0.	0.	1500.	0.	1500.
6	43	128	94.4	95.7	1280.	0.	34.	1314.	50.	1330.
7	44	116	91.7	91.3	1160.	0.	68.	1228.	100.	1260.
8	45	104	80.6	82.6	1040.	0.	230.	1270.	350.	1390.
9	47	77	69.4	69.6	770.	0.	361.	1131.	500.	1270.
10	48	60	66.7	65.2	600.	0.	395.	995.	550.	1150.
11	50	34	55.6	56.5	340.	0.	531.	871.	750.	1090.
12	51	26	38.9	47.8	260.	0.	723.	983.	1050.	1310.
13	52	21	33.3	39.1	210.	0.	849.	1059.	1150.	1360.
14	54	0	0.0	0.0	0.	0.	1304.	1304.	1800.	1800.
15	56	0	0.0	0.0	0.	0.	1304.	1304.	1800.	1800.
16	60	0	0.0	0.0	0.	0.	1304.	1304.	1800.	1800.

1974-75 CHLORIDE PROFILE; PREDICTED STREAMFLOWS; STANDARD = 50 MG/L

1	33	281	100.0	100.0	2810.	170.	0.	2980.	0.	2980.
2	37	229	95.2	91.9	2290.	170.	102.	2562.	150.	2610.
3	39	202	90.5	83.8	2020.	170.	204.	2394.	300.	2490.
4	41	162	85.7	75.7	1620.	170.	306.	2096.	450.	2240.
5	42	147	85.7	75.7	1470.	170.	306.	1946.	450.	2090.
6	43	124	81.0	67.6	1240.	170.	408.	1818.	600.	2010.
7	44	111	76.2	64.9	1110.	170.	510.	1790.	750.	2030.
8	45	94	65.1	59.5	940.	170.	761.	1871.	1100.	2210.
9	47	70	52.4	45.9	700.	170.	1127.	1997.	1450.	2320.
10	48	51	41.3	35.1	510.	170.	1331.	2011.	1750.	2430.
11	50	29	27.0	18.9	290.	170.	1650.	2110.	2200.	2660.
12	52	16	17.5	13.5	160.	170.	1875.	2205.	2500.	2830.
13	53	7	7.9	2.7	70.	170.	2500.	2740.	2850.	3090.
14	54	3	4.8	2.7	30.	170.	2534.	2734.	2900.	3100.
15	56	0	0.0	0.0	0.	170.	3841.	4011.	3150.	3320.
16	60	0	0.0	0.0	0.	170.	3841.	4011.	3150.	3320.

TABLE E-15

1974-75 CHLORIDE PROFILE; U.S.G.S. STREAMFLOWS; STANDARD = 55 MG/L

PLAN	SAMPLE IF MODEL >	SAMPLES REQUIRED	% VIOL. TIMES DETECTED	% VIOLATIONS DETECTED	FIXED COST (1)	FIXED COST (2)	VARIABLE COST (EXPONENTIAL)	TOTAL COST (EXPONENTIAL)	VARIABLE COST (LINEAR)	TOTAL COST (LINEAR)
1	39	195	100.0	100.0	1950.	0.	0.	1950.	0.	1950.
2	43	128	90.9	90.0	1280.	0.	34.	1314.	50.	1330.
3	45	104	72.7	70.0	1040.	0.	102.	1142.	150.	1190.
4	46	92	72.7	70.0	920.	0.	102.	1022.	150.	1070.
5	47	77	63.6	60.0	770.	0.	136.	906.	200.	970.
6	48	60	63.6	60.0	600.	0.	136.	736.	200.	800.
7	49	46	63.6	60.0	460.	0.	136.	596.	200.	660.
8	50	34	63.6	60.0	340.	0.	136.	476.	200.	540.
9	52	21	54.5	50.0	210.	0.	170.	380.	250.	460.
10	53	11	18.2	20.0	110.	0.	302.	412.	450.	560.
11	55	0	0.0	0.0	0.	0.	370.	370.	550.	550.
12	56	0	0.0	0.0	0.	0.	370.	370.	550.	550.
13	57	0	0.0	0.0	0.	0.	370.	370.	550.	550.
14	59	0	0.0	0.0	0.	0.	370.	370.	550.	550.
15	61	0	0.0	0.0	0.	0.	370.	370.	550.	550.
16	65	0	0.0	0.0	0.	0.	370.	370.	550.	550.

1974-75 CHLORIDE PROFILE; PREDICTED STREAMFLOWS; STANDARD = 55 MG/L

1	38	220	96.2	94.1	2200.	170.	34.	2404.	50.	2420.
2	42	147	96.2	94.1	1470.	170.	34.	1674.	50.	1690.
3	44	111	92.3	94.1	1110.	170.	68.	1348.	100.	1380.
4	46	88	73.1	70.6	880.	170.	230.	1280.	350.	1400.
5	47	70	61.5	64.7	700.	170.	294.	1164.	450.	1320.
6	48	51	50.0	47.1	510.	170.	396.	1076.	600.	1280.
7	49	37	42.3	41.2	370.	170.	464.	1004.	700.	1240.
8	50	29	38.5	35.3	290.	170.	494.	954.	750.	1210.
9	52	16	23.1	23.5	160.	170.	655.	985.	950.	1280.
10	53	7	11.5	11.8	70.	170.	783.	1023.	1150.	1390.
11	55	0	0.0	0.0	0.	170.	881.	1051.	1300.	1470.
12	57	0	0.0	0.0	0.	170.	881.	1051.	1300.	1470.
13	58	0	0.0	0.0	0.	170.	881.	1051.	1300.	1470.
14	59	0	0.0	0.0	0.	170.	881.	1051.	1300.	1470.
15	61	0	0.0	0.0	0.	170.	881.	1051.	1300.	1470.
16	65	0	0.0	0.0	0.	170.	881.	1051.	1300.	1470.

TABLE E-16

1974-75 CHLORIDE PROFILE; U.S.G.S. STREAMFLOWS; STANDARD = 45 MG/L

PLAN	SAMPLING INTERVAL	SAMPLES REQUIRED	% VIOL. TIMES DETECTED	% VIOLATIONS DETECTED	FIXED COST (1)	FIXED COST (2)	VARIABLE COST (EXPONENTIAL)	TOTAL COST (EXPONENTIAL)	VARIABLE COST (LINEAR)	TOTAL COST (LINEAR)
A	1	365	100.0	100.0	3650.	0.	0.	3650.	0.	3650.
B	2	183	47.2	70.4	1830.	0.	1020.	2850.	1500.	3330.
C	3	122	40.6	66.7	1220.	0.	974.	2194.	1450.	2670.
D	4	91	23.6	44.4	910.	0.	1969.	2879.	2850.	3760.
E	6	61	17.9	35.2	610.	0.	2130.	2740.	3050.	3660.
F	9	41	10.4	20.4	410.	0.	12428.	12838.	4500.	4910.
G	15	24	11.3	22.2	240.	0.	11340.	11580.	4300.	4540.
H	23	16	3.8	7.4	160.	0.	5906.	6066.	4850.	5010.
I	32	11	4.7	7.4	110.	0.	12075.	12185.	4950.	5060.
J	46	8	2.8	5.6	80.	0.	5936.	6016.	4800.	4880.
K	59	6	2.8	5.6	60.	0.	12800.	12860.	4950.	5010.
L	74	5	0.9	1.9	50.	0.	13420.	13470.	5250.	5300.
M	95	4	0.9	1.9	40.	0.	13420.	13460.	5250.	5290.
N	126	3	0.9	1.9	30.	0.	13420.	13450.	5250.	5280.
O	174	2	0.9	1.9	20.	0.	13420.	13440.	5250.	5270.
P	336	1	0.9	1.9	10.	0.	13420.	13430.	5250.	5260.

1974-75 CHLORIDE PROFILE; PREDICTED STREAMFLOWS; STANDARD = 45 MG/L

A	1	365	100.0	100.0	3650.	170.	0.	3820.	0.	3820.
B	2	183	49.5	73.7	1830.	170.	952.	2952.	1400.	3400.
C	3	122	27.0	47.4	1220.	170.	1864.	3254.	2800.	4190.
D	4	91	23.4	45.6	910.	170.	2099.	3179.	2950.	4030.
E	6	61	17.1	33.3	610.	170.	3036.	3816.	3650.	4430.
F	9	41	9.0	17.5	410.	170.	3568.	4148.	4400.	4980.
G	15	24	5.4	10.5	240.	170.	4273.	4683.	4650.	5060.
H	23	16	1.8	3.5	160.	170.	10316.	10646.	5200.	5530.
I	32	11	2.7	5.3	110.	170.	10486.	10766.	5250.	5530.
J	46	8	0.9	1.8	80.	170.	10443.	10693.	5350.	5600.
K	59	6	1.8	3.5	60.	170.	10584.	10814.	5400.	5630.
L	74	5	1.8	3.5	50.	170.	10443.	10663.	5350.	5570.
M	95	4	0.0	0.0	40.	170.	10711.	10921.	5550.	5760.
N	126	3	0.0	0.0	30.	170.	10711.	10911.	5550.	5750.
O	174	2	0.0	0.0	20.	170.	10711.	10901.	5550.	5740.
P	336	1	0.9	1.8	10.	170.	8922.	9102.	5500.	5680.

TABLE B-17

1974-75 CHLORIDE PROFILE; U.S.G.S. STREAMFLOWS; STANDARD = 50 MG/L

PLAN	SAMPLING INTERVAL	SAMPLES REQUIRED	% VIOL. TIMES DETECTED	% VIOLATIONS DETECTED	FIXED COST (1)	FIXED COST (2)	VARIABLE COST (EXPONENTIAL)	TOTAL COST (EXPONENTIAL)	VARIABLE COST (LINEAR)	TOTAL COST (LINEAR)
A	1	365	100.0	100.0	3650.	0.	0.	3650.	0.	3650.
B	2	183	44.4	56.5	1830.	0.	408.	2238.	600.	2430.
C	3	122	44.4	69.6	1220.	0.	464.	1684.	700.	1920.
D	4	91	19.4	30.4	910.	0.	762.	1672.	1150.	2060.
E	6	61	22.2	34.8	610.	0.	914.	1524.	1300.	1910.
F	9	41	19.4	30.4	410.	0.	1040.	1450.	1400.	1810.
G	15	24	13.9	21.7	240.	0.	1104.	1344.	1500.	1740.
H	23	16	8.3	13.0	160.	0.	1109.	1269.	1550.	1710.
I	32	11	5.6	8.7	110.	0.	1143.	1243.	1600.	1710.
J	47	8	8.3	13.0	80.	0.	1143.	1223.	1600.	1680.
K	60	6	2.8	4.3	60.	0.	1270.	1330.	1750.	1810.
L	74	5	2.8	4.3	50.	0.	1270.	1320.	1750.	1800.
M	96	4	5.6	8.7	40.	0.	1240.	1280.	1700.	1740.
N	128	3	2.8	4.3	30.	0.	1270.	1300.	1750.	1780.
O	176	2	2.8	4.3	20.	0.	1270.	1290.	1750.	1770.
P	340	1	2.8	4.3	10.	0.	1270.	1280.	1750.	1760.

1974-75 CHLORIDE PROFILE; PREDICTED STREAMFLOWS; STANDARD = 50 MG/L

A	1	365	100.0	100.0	3650.	170.	0.	3820.	0.	3820.
B	2	183	44.4	62.2	1830.	170.	782.	2782.	1150.	3150.
C	3	122	33.3	51.4	1220.	170.	944.	2334.	1400.	2790.
D	4	91	22.2	37.8	910.	170.	1449.	2529.	2050.	3130.
E	6	61	19.0	32.4	610.	170.	1916.	2696.	2150.	2930.
F	9	41	9.5	16.2	410.	170.	2080.	2660.	2650.	3230.
G	15	24	6.3	10.8	240.	170.	2381.	2791.	2700.	3110.
H	23	16	1.6	2.7	160.	170.	3714.	4044.	3000.	3330.
I	32	11	1.6	2.7	110.	170.	3807.	4087.	3100.	3380.
J	47	8	1.6	2.7	80.	170.	2500.	2750.	2850.	3100.
K	60	6	4.8	8.1	60.	170.	3616.	3846.	2850.	3080.
L	74	5	1.6	2.7	50.	170.	3807.	4027.	3100.	3320.
M	96	4	0.0	0.0	40.	170.	3841.	4051.	3150.	3360.
N	128	3	0.0	0.0	30.	170.	3841.	4041.	3150.	3350.
O	176	2	0.0	0.0	20.	170.	3841.	4031.	3150.	3340.
P	340	1	0.0	0.0	10.	170.	3841.	4021.	3150.	3330.

TABLE E-18

1974-75 CHLORIDE PROFILE; U.S.G.S. STREAMFLOWS; STANDARD = 55 MG/L

PLAN	SAMPLING INTERVAL	SAMPLES REQUIRED	% VIOL. TIMES DETECTED	% VIOLATIONS DETECTED	FIXED COST (1)	FIXED COST (2)	VARIABLE COST (EXPONENTIAL)	TOTAL COST (EXPONENTIAL)	VARIABLE COST (LINEAR)	TOTAL COST (LINEAR)
A	1	365	100.0	100.0	3650.	0.	0.	3650.	0.	3650.
B	2	183	27.3	30.0	1830.	0.	238.	2068.	350.	2180.
C	3	122	36.4	40.0	1220.	0.	238.	1458.	350.	1570.
D	4	91	9.1	10.0	910.	0.	306.	1216.	450.	1360.
E	6	61	18.2	20.0	610.	0.	302.	912.	450.	1060.
F	9	41	9.1	10.0	410.	0.	340.	750.	500.	910.
G	16	23	0.0	0.0	230.	0.	370.	600.	550.	780.
H	25	15	0.0	0.0	150.	0.	370.	520.	550.	700.
I	34	11	0.0	0.0	110.	0.	370.	480.	550.	660.
J	49	7	0.0	0.0	70.	0.	370.	440.	550.	620.
K	63	6	0.0	0.0	60.	0.	370.	430.	550.	610.
L	78	5	0.0	0.0	50.	0.	370.	420.	550.	600.
M	100	4	0.0	0.0	40.	0.	370.	410.	550.	590.
N	134	3	0.0	0.0	30.	0.	370.	400.	550.	580.
O	185	2	0.0	0.0	20.	0.	370.	390.	550.	570.
P	357	1	0.0	0.0	10.	0.	370.	380.	550.	560.

1974-75 CHLORIDE PROFILE; PREDICTED STREAMFLOWS; STANDARD = 55 MG/L

A	1	365	100.0	100.0	3650.	170.	0.	3820.	0.	3820.
B	2	183	42.3	64.7	1830.	170.	374.	2374.	550.	2550.
C	3	122	30.8	47.1	1220.	170.	438.	1828.	650.	2040.
D	4	91	19.2	29.4	910.	170.	625.	1705.	900.	1980.
E	6	61	19.2	29.4	610.	170.	629.	1409.	900.	1660.
F	9	41	3.8	5.9	410.	170.	881.	1461.	1300.	1880.
G	16	23	0.0	0.0	230.	170.	881.	1291.	1300.	1710.
H	25	15	11.5	17.6	150.	170.	656.	986.	1000.	1330.
I	34	11	3.8	5.9	110.	170.	851.	1131.	1250.	1530.
J	49	7	0.0	0.0	70.	170.	881.	1131.	1300.	1550.
K	63	6	0.0	0.0	60.	170.	881.	1111.	1300.	1530.
L	78	5	3.8	5.9	50.	170.	881.	1101.	1300.	1520.
M	100	4	0.0	0.0	40.	170.	881.	1091.	1300.	1510.
N	134	3	0.0	0.0	30.	170.	881.	1081.	1300.	1500.
O	185	2	0.0	0.0	20.	170.	881.	1071.	1300.	1490.
P	357	1	0.0	0.0	10.	170.	881.	1061.	1300.	1480.

APPENDIX F

Sample Chloride Profiles and Model Predictions for the 1973 and 1974 Water Years

TABLE F-1

1973-74 MODEL PREDICTIONS AT SEGMENT 5 USING U.S.G.S. STREAMFLOWS

(CHLORIDE CONCENTRATIONS IN MG/L)

DAY	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP
1	45.	48.	32.	33.	32.	34.	27.	39.	41.	40.	55.	33.
2	38.	45.	34.	35.	33.	33.	24.	40.	42.	40.	55.	38.
3	41.	45.	36.	35.	34.	34.	27.	39.	43.	42.	56.	34.
4	40.	46.	36.	36.	35.	35.	26.	40.	44.	43.	53.	35.
5	39.	47.	36.	37.	36.	35.	26.	41.	45.	44.	51.	37.
6	42.	48.	36.	37.	35.	32.	27.	41.	45.	45.	51.	40.
7	45.	49.	37.	37.	30.	33.	29.	42.	45.	45.	53.	42.
8	46.	50.	38.	38.	32.	34.	26.	42.	46.	44.	54.	44.
9	47.	50.	39.	39.	34.	35.	26.	40.	46.	45.	54.	46.
10	48.	51.	39.	38.	36.	36.	27.	41.	46.	46.	53.	47.
11	49.	51.	40.	38.	36.	37.	27.	42.	47.	46.	52.	48.
12	49.	51.	40.	39.	37.	37.	28.	41.	47.	46.	52.	32.
13	50.	51.	40.	41.	37.	38.	31.	40.	48.	46.	51.	36.
14	49.	52.	40.	41.	36.	38.	32.	40.	48.	47.	51.	35.
15	49.	50.	39.	34.	35.	39.	33.	41.	48.	48.	52.	39.
16	49.	43.	39.	30.	36.	38.	35.	42.	47.	49.	52.	41.
17	49.	43.	41.	27.	37.	38.	36.	42.	47.	49.	46.	43.
18	50.	46.	42.	26.	38.	37.	37.	29.	48.	50.	43.	45.
19	50.	47.	42.	24.	37.	38.	37.	33.	49.	51.	45.	47.
20	51.	48.	39.	22.	32.	39.	38.	35.	49.	51.	48.	47.
21	51.	48.	37.	22.	31.	37.	38.	38.	49.	52.	49.	47.
22	52.	48.	39.	23.	30.	38.	38.	38.	31.	53.	50.	47.
23	51.	49.	40.	24.	28.	39.	37.	38.	21.	53.	52.	49.
24	51.	41.	40.	25.	29.	39.	38.	37.	27.	54.	53.	50.
25	52.	29.	36.	27.	32.	39.	39.	37.	31.	53.	53.	51.
26	52.	26.	28.	29.	34.	39.	39.	39.	31.	53.	54.	51.
27	52.	26.	24.	27.	35.	40.	40.	41.	34.	53.	55.	52.
28	52.	25.	25.	26.	35.	39.	41.	42.	37.	54.	47.	51.
29	53.	27.	27.	28.		38.	41.	42.	38.	54.	33.	41.
30	52.	29.	29.	29.		28.	40.	41.	40.	55.	35.	42.
31	51.		31.	30.		26.		40.		55.	36.	

TABLE F-2

1973-74 MODEL PREDICTIONS AT SEGMENT 5 USING PREDICTED STREAMFLOWS

(CHLORIDE CONCENTRATIONS IN MG/L)

DAY	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP
1	45.	48.	31.	33.	33.	34.	27.	38.	41.	39.	58.	33.
2	38.	45.	33.	34.	33.	33.	24.	40.	41.	40.	58.	38.
3	41.	45.	34.	35.	34.	34.	27.	39.	42.	40.	58.	34.
4	40.	46.	36.	36.	35.	35.	26.	40.	42.	42.	54.	35.
5	39.	46.	36.	37.	36.	35.	26.	40.	44.	44.	51.	37.
6	39.	47.	37.	38.	35.	32.	28.	41.	45.	44.	50.	38.
7	42.	49.	38.	39.	10.	34.	30.	41.	45.	44.	51.	39.
8	45.	49.	38.	39.	31.	35.	26.	42.	46.	45.	51.	39.
9	46.	49.	39.	39.	33.	36.	26.	40.	46.	45.	52.	42.
10	47.	50.	40.	38.	34.	37.	28.	41.	46.	46.	52.	47.
11	48.	51.	40.	38.	36.	37.	30.	41.	47.	46.	52.	48.
12	48.	51.	41.	38.	37.	37.	32.	41.	47.	47.	52.	32.
13	49.	51.	41.	39.	38.	38.	33.	40.	48.	47.	50.	36.
14	49.	52.	40.	40.	38.	38.	35.	40.	48.	47.	49.	35.
15	49.	51.	40.	40.	39.	39.	36.	41.	48.	48.	49.	36.
16	49.	43.	40.	41.	40.	38.	37.	42.	47.	48.	50.	37.
17	49.	41.	41.	41.	40.	38.	38.	42.	47.	49.	45.	38.
18	50.	41.	41.	26.	41.	39.	38.	29.	47.	49.	43.	39.
19	50.	42.	42.	24.	37.	39.	39.	33.	48.	49.	44.	39.
20	50.	42.	39.	22.	32.	40.	40.	35.	48.	50.	44.	40.
21	51.	44.	37.	22.	34.	38.	40.	36.	49.	51.	45.	42.
22	51.	48.	38.	23.	30.	38.	38.	38.	31.	52.	45.	46.
23	52.	48.	38.	24.	28.	38.	37.	38.	21.	53.	47.	48.
24	52.	41.	39.	25.	29.	39.	38.	37.	31.	54.	50.	50.
25	53.	29.	36.	27.	32.	39.	39.	37.	33.	54.	52.	50.
26	53.	26.	28.	29.	34.	40.	39.	38.	34.	55.	53.	50.
27	54.	26.	24.	27.	35.	40.	40.	39.	35.	55.	53.	50.
28	54.	25.	26.	26.	36.	41.	40.	39.	37.	56.	47.	51.
29	55.	27.	27.	28.		39.	41.	41.	37.	56.	33.	41.
30	53.	29.	29.	30.		28.	41.	41.	38.	57.	35.	42.
31	51.		31.	31.		26.		40.		58.	36.	

TABLE F-3

1974-75 MODEL PREDICTIONS AT SEGMENT 5 USING U.S.G.S. STREAMFLOWS

(CHLORIDE CONCENTRATIONS IN MG/L)

DAY	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP
1	47.	53.	42.	31.	26.	27.	30.	31.	42.	47.	51.	45.
2	48.	54.	41.	29.	29.	29.	32.	31.	42.	47.	52.	46.
3	49.	53.	40.	31.	31.	31.	32.	32.	43.	49.	52.	48.
4	50.	47.	41.	33.	32.	32.	33.	34.	43.	34.	44.	49.
5	51.	40.	42.	34.	32.	34.	35.	35.	43.	27.	43.	49.
6	51.	41.	42.	35.	32.	35.	36.	35.	39.	32.	44.	48.
7	52.	42.	39.	36.	31.	35.	36.	36.	31.	35.	46.	48.
8	52.	43.	30.	35.	33.	33.	37.	37.	35.	37.	48.	48.
9	51.	45.	30.	27.	35.	32.	37.	38.	38.	38.	49.	49.
10	53.	47.	31.	25.	37.	34.	38.	39.	40.	40.	50.	50.
11	51.	46.	34.	24.	37.	34.	38.	39.	40.	42.	47.	49.
12	53.	42.	35.	26.	34.	24.	39.	39.	38.	43.	45.	40.
13	54.	42.	34.	29.	35.	25.	39.	38.	39.	45.	47.	41.
14	54.	44.	32.	32.	37.	28.	40.	40.	39.	45.	48.	43.
15	52.	44.	31.	34.	38.	29.	40.	40.	39.	43.	47.	46.
16	48.	45.	29.	35.	37.	29.	40.	41.	37.	43.	42.	48.
17	48.	45.	29.	36.	32.	29.	40.	41.	37.	44.	42.	49.
18	50.	46.	31.	35.	28.	27.	40.	41.	39.	43.	44.	49.
19	51.	46.	31.	34.	27.	26.	39.	42.	41.	34.	46.	48.
20	53.	46.	34.	35.	29.	27.	39.	42.	42.	32.	48.	47.
21	54.	45.	36.	37.	32.	29.	40.	43.	43.	37.	49.	47.
22	54.	45.	37.	38.	33.	30.	40.	43.	44.	40.	49.	47.
23	54.	45.	38.	38.	22.	32.	40.	43.	46.	41.	50.	48.
24	54.	46.	35.	39.	19.	31.	28.	43.	47.	43.	50.	48.
25	53.	43.	29.	38.	20.	29.	25.	44.	47.	45.	50.	48.
26	53.	41.	30.	32.	21.	30.	24.	43.	44.	46.	50.	48.
27	53.	40.	33.	32.	23.	32.	27.	43.	42.	47.	50.	48.
28	54.	41.	35.	34.	25.	31.	23.	43.	43.	49.	51.	49.
29	54.	42.	37.	31.		25.	27.	44.	45.	49.	51.	50.
30	54.	43.	38.	25.		25.	30.	44.	46.	50.	51.	51.
31	54.		37.	25.		27.		44.		50.	47.	

TABLE F-4

1974-75 MODEL PREDICTIONS AT SEGMENT 5 USING PREDICTED STREAMFLOWS

(CHLORIDE CONCENTRATIONS IN MG/L)

DAY	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP
1	47.	53.	42.	31.	26.	29.	29.	31.	42.	49.	48.	45.
2	48.	54.	41.	32.	28.	31.	31.	31.	42.	49.	48.	45.
3	48.	53.	40.	31.	30.	32.	32.	32.	43.	50.	48.	45.
4	49.	47.	40.	33.	32.	34.	33.	34.	43.	34.	43.	46.
5	49.	40.	40.	34.	32.	35.	35.	35.	43.	27.	43.	47.
6	49.	40.	41.	35.	32.	36.	36.	36.	39.	29.	44.	47.
7	50.	40.	39.	37.	33.	35.	37.	37.	39.	31.	45.	47.
8	50.	41.	30.	35.	35.	33.	38.	38.	40.	33.	45.	48.
9	51.	41.	31.	27.	36.	35.	39.	39.	40.	34.	46.	49.
10	51.	41.	33.	25.	37.	34.	39.	39.	41.	39.	47.	49.
11	51.	43.	34.	24.	38.	34.	40.	40.	41.	42.	47.	48.
12	52.	42.	36.	26.	34.	24.	40.	40.	39.	42.	46.	39.
13	53.	41.	37.	28.	35.	25.	41.	38.	39.	44.	47.	41.
14	53.	41.	38.	30.	36.	28.	41.	39.	40.	44.	48.	43.
15	51.	42.	31.	32.	37.	29.	41.	39.	39.	43.	49.	43.
16	48.	42.	29.	33.	37.	31.	42.	40.	37.	43.	49.	43.
17	48.	42.	30.	35.	32.	33.	41.	40.	37.	44.	50.	44.
18	48.	43.	32.	35.	34.	34.	40.	41.	39.	43.	50.	45.
19	48.	41.	31.	34.	35.	26.	39.	42.	40.	42.	51.	47.
20	49.	44.	34.	35.	36.	27.	39.	42.	40.	42.	52.	47.
21	49.	45.	36.	36.	37.	29.	40.	43.	42.	43.	52.	47.
22	49.	45.	37.	37.	33.	30.	40.	43.	44.	43.	52.	47.
23	50.	44.	38.	38.	22.	32.	40.	43.	45.	43.	53.	48.
24	50.	45.	38.	39.	19.	31.	28.	44.	46.	44.	53.	47.
25	51.	43.	29.	38.	20.	29.	25.	45.	46.	44.	54.	48.
26	52.	40.	30.	32.	22.	31.	24.	45.	47.	44.	54.	48.
27	52.	41.	32.	34.	24.	32.	26.	45.	47.	45.	55.	48.
28	52.	41.	33.	35.	27.	31.	23.	45.	47.	45.	55.	49.
29	53.	41.	35.	31.		25.	27.	45.	48.	46.	55.	49.
30	53.	43.	36.	25.		25.	30.	44.	48.	47.	54.	50.
31	53.		37.	25.		27.		43.		47.	48.	

TABLE F-5

1973-74 CHLORIDE PROFILE AT SEGMENT 5 USING U.S.G.S. STREAMFLOWS

(CONCENTRATIONS IN MG/L)

DAY	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP
1	39.	51.	28.	43.	28.	34.	17.	47.	37.	48.	47.	40.
2	42.	62.	40.	36.	31.	43.	24.	41.	45.	53.	64.	39.
3	51.	50.	31.	33.	28.	36.	23.	30.	40.	44.	53.	38.
4	34.	45.	42.	44.	36.	31.	37.	39.	45.	41.	55.	41.
5	50.	41.	48.	41.	51.	35.	31.	37.	47.	41.	37.	47.
6	43.	51.	42.	45.	47.	41.	28.	51.	43.	43.	63.	52.
7	52.	57.	44.	40.	33.	37.	32.	46.	49.	44.	43.	45.
8	38.	54.	43.	49.	35.	35.	34.	45.	51.	54.	49.	49.
9	52.	60.	48.	46.	28.	37.	25.	44.	49.	63.	49.	54.
10	52.	57.	49.	36.	43.	36.	24.	55.	54.	46.	56.	53.
11	48.	54.	35.	41.	40.	29.	27.	42.	51.	37.	57.	51.
12	58.	64.	41.	43.	34.	45.	37.	46.	50.	64.	61.	27.
13	41.	57.	48.	48.	41.	45.	33.	39.	44.	54.	53.	34.
14	53.	57.	42.	37.	28.	43.	17.	44.	53.	47.	50.	49.
15	48.	47.	47.	34.	42.	39.	34.	46.	53.	55.	51.	39.
16	43.	49.	43.	33.	42.	41.	38.	47.	47.	59.	57.	36.
17	51.	40.	36.	26.	29.	36.	33.	56.	42.	50.	46.	37.
18	50.	55.	58.	25.	39.	42.	32.	24.	44.	48.	44.	51.
19	55.	41.	34.	17.	49.	43.	46.	36.	46.	45.	52.	49.
20	60.	50.	42.	25.	28.	45.	53.	24.	55.	52.	56.	52.
21	47.	40.	41.	22.	44.	40.	34.	37.	51.	60.	58.	46.
22	46.	57.	40.	28.	23.	47.	40.	41.	43.	52.	48.	55.
23	62.	53.	36.	31.	28.	34.	34.	40.	22.	62.	51.	49.
24	54.	48.	47.	28.	35.	33.	35.	32.	34.	56.	50.	59.
25	59.	28.	32.	28.	34.	43.	42.	31.	41.	55.	62.	49.
26	60.	28.	36.	28.	34.	41.	44.	45.	40.	59.	53.	45.
27	59.	29.	26.	29.	37.	41.	41.	39.	32.	50.	61.	61.
28	48.	26.	33.	36.	35.	43.	45.	40.	43.	55.	48.	56.
29	57.	29.	33.	29.		48.	43.	36.	35.	56.	33.	29.
30	50.	26.	25.	30.		28.	46.	42.	35.	63.	43.	48.
31	54.		32.	27.		30.		39.		46.	37.	

TABLE F-6

1973-74 CHLORIDE PROFILE AT SEGMENT 5 USING PREDICTED STREAMFLOWS

(CONCENTRATIONS IN MG/L)

DAY	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP
1	49.	57.	42.	37.	29.	27.	33.	37.	43.	52.	61.	42.
2	37.	51.	36.	43.	35.	48.	19.	46.	34.	44.	61.	45.
3	55.	51.	44.	47.	32.	37.	32.	43.	47.	42.	56.	37.
4	44.	40.	37.	37.	32.	46.	28.	42.	41.	41.	56.	42.
5	40.	48.	30.	31.	43.	46.	34.	41.	46.	47.	63.	32.
6	44.	57.	39.	46.	28.	31.	32.	43.	37.	42.	54.	39.
7	48.	56.	40.	47.	30.	33.	46.	49.	52.	36.	54.	47.
8	50.	56.	34.	46.	36.	33.	29.	39.	51.	44.	61.	37.
9	50.	50.	38.	38.	48.	36.	34.	42.	48.	53.	55.	43.
10	38.	60.	47.	38.	45.	49.	25.	26.	46.	42.	62.	46.
11	50.	42.	46.	45.	32.	47.	32.	37.	54.	40.	63.	52.
12	64.	46.	37.	39.	42.	38.	30.	31.	50.	49.	59.	28.
13	64.	52.	42.	38.	37.	30.	40.	40.	47.	57.	58.	43.
14	49.	53.	44.	38.	46.	44.	36.	43.	47.	66.	63.	32.
15	56.	51.	41.	38.	34.	44.	38.	45.	53.	51.	43.	38.
16	52.	48.	39.	39.	44.	49.	39.	37.	39.	57.	49.	39.
17	46.	49.	40.	49.	39.	36.	50.	48.	44.	43.	53.	42.
18	63.	42.	51.	31.	42.	44.	42.	31.	48.	61.	48.	45.
19	46.	36.	47.	20.	34.	44.	47.	42.	53.	57.	53.	39.
20	60.	35.	41.	14.	34.	49.	40.	39.	46.	58.	49.	42.
21	52.	48.	37.	27.	31.	41.	48.	42.	57.	48.	54.	52.
22	43.	41.	44.	27.	33.	41.	39.	39.	29.	53.	53.	38.
23	62.	54.	38.	37.	35.	38.	37.	35.	22.	60.	52.	58.
24	56.	35.	41.	31.	28.	46.	44.	40.	33.	53.	64.	53.
25	47.	37.	31.	33.	31.	49.	33.	41.	32.	47.	56.	53.
26	48.	33.	23.	26.	32.	41.	42.	47.	23.	63.	55.	58.
27	58.	29.	29.	31.	35.	40.	45.	49.	37.	64.	65.	61.
28	59.	28.	23.	31.	44.	40.	46.	39.	35.	60.	52.	51.
29	54.	36.	30.	42.		36.	43.	39.	37.	62.	40.	36.
30	57.	45.	38.	34.		24.	46.	44.	33.	50.	45.	54.
31	53.		22.	35.		38.		36.		53.	49.	

TABLE F-7

1974-75 CHLORIDE PROFILE AT SEGMENT 5 USING U.S.G.S. STREAMFLOWS

(CONCENTRATIONS IN MG/L)

DAY	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP
1	43.	47.	46.	39.	24.	23.	34.	38.	46.	41.	54.	55.
2	46.	56.	43.	40.	36.	34.	28.	31.	57.	57.	57.	41.
3	54.	56.	40.	29.	35.	34.	40.	47.	55.	57.	54.	50.
4	47.	57.	46.	30.	37.	39.	44.	40.	54.	41.	48.	43.
5	48.	45.	39.	39.	36.	45.	46.	43.	41.	40.	53.	59.
6	61.	45.	37.	46.	39.	39.	38.	42.	55.	29.	38.	50.
7	48.	43.	45.	42.	38.	27.	45.	44.	30.	50.	50.	49.
8	48.	50.	26.	37.	35.	36.	35.	40.	38.	37.	55.	48.
9	56.	45.	43.	24.	42.	34.	39.	45.	27.	37.	58.	53.
10	60.	57.	31.	29.	43.	30.	35.	40.	44.	36.	54.	55.
11	48.	50.	31.	23.	38.	30.	31.	35.	49.	44.	51.	42.
12	58.	51.	37.	27.	38.	28.	46.	39.	31.	57.	49.	43.
13	59.	42.	36.	28.	44.	42.	40.	39.	45.	45.	56.	43.
14	69.	48.	32.	38.	42.	28.	35.	34.	47.	46.	61.	58.
15	56.	53.	41.	43.	41.	39.	42.	50.	42.	56.	53.	50.
16	66.	48.	32.	27.	40.	42.	37.	45.	37.	43.	40.	51.
17	47.	53.	39.	28.	34.	36.	46.	47.	47.	54.	57.	58.
18	56.	46.	29.	38.	21.	29.	39.	37.	40.	43.	42.	53.
19	49.	48.	27.	44.	23.	27.	45.	47.	43.	38.	40.	55.
20	57.	48.	37.	28.	31.	31.	38.	44.	46.	27.	45.	50.
21	55.	41.	31.	34.	30.	35.	47.	48.	55.	40.	50.	43.
22	57.	54.	40.	43.	34.	37.	59.	45.	36.	48.	48.	44.
23	56.	50.	30.	52.	26.	32.	51.	39.	34.	44.	59.	59.
24	48.	49.	33.	42.	25.	28.	28.	46.	48.	46.	53.	45.
25	51.	31.	25.	42.	35.	31.	23.	41.	47.	57.	51.	46.
26	63.	42.	38.	40.	17.	30.	26.	50.	45.	47.	46.	71.
27	55.	47.	33.	40.	23.	26.	39.	51.	49.	50.	47.	57.
28	58.	46.	38.	41.	25.	41.	26.	43.	50.	56.	54.	57.
29	55.	36.	43.	32.		38.	30.	59.	53.	55.	50.	57.
30	59.	46.	41.	33.		38.	29.	48.	56.	51.	54.	49.
31	51.		39.	36.		29.		46.		47.	52.	

TABLE F-8

1974-75 CHLORIDE PROFILE AT SEGMENT 5 USING PREDICTED STREAMFLOWS

(CONCENTRATIONS IN MG/L)

DAY	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP
1	50.	61.	58.	30.	27.	36.	35.	38.	45.	44.	44.	40.
2	66.	54.	44.	41.	33.	38.	32.	33.	45.	43.	48.	38.
3	54.	64.	46.	35.	38.	32.	40.	40.	44.	56.	50.	51.
4	57.	51.	49.	33.	39.	42.	34.	38.	41.	46.	53.	56.
5	46.	43.	45.	36.	31.	31.	39.	42.	41.	30.	38.	58.
6	58.	34.	49.	32.	33.	38.	51.	37.	42.	28.	43.	46.
7	53.	32.	47.	37.	37.	44.	42.	41.	43.	45.	43.	52.
8	51.	36.	33.	42.	39.	43.	38.	43.	41.	28.	45.	52.
9	56.	38.	28.	27.	34.	32.	40.	38.	48.	45.	43.	58.
10	60.	41.	36.	33.	49.	40.	37.	37.	56.	52.	54.	47.
11	54.	36.	42.	35.	58.	35.	42.	50.	37.	41.	56.	50.
12	55.	39.	33.	21.	28.	21.	41.	44.	46.	38.	49.	39.
13	56.	50.	35.	15.	30.	32.	46.	44.	44.	37.	46.	46.
14	56.	34.	44.	36.	39.	35.	51.	46.	36.	50.	52.	51.
15	58.	45.	34.	31.	45.	39.	47.	40.	41.	51.	50.	48.
16	51.	41.	31.	43.	36.	30.	45.	47.	49.	44.	55.	53.
17	56.	48.	29.	44.	30.	33.	53.	43.	40.	50.	47.	48.
18	53.	39.	36.	40.	37.	32.	45.	48.	42.	52.	48.	52.
19	58.	41.	32.	48.	39.	28.	46.	44.	40.	62.	57.	54.
20	55.	50.	33.	46.	34.	37.	43.	50.	40.	43.	53.	42.
21	62.	45.	38.	33.	39.	33.	53.	44.	49.	44.	53.	42.
22	52.	53.	49.	42.	48.	42.	44.	46.	51.	50.	68.	51.
23	53.	48.	42.	38.	26.	27.	46.	51.	53.	40.	55.	55.
24	53.	48.	39.	45.	9.	31.	37.	43.	47.	54.	57.	60.
25	53.	25.	25.	28.	22.	32.	24.	46.	55.	43.	58.	49.
26	55.	37.	30.	39.	21.	27.	40.	42.	42.	50.	53.	45.
27	60.	38.	29.	22.	38.	32.	26.	39.	50.	48.	52.	51.
28	66.	48.	35.	44.	29.	31.	29.	45.	48.	48.	56.	55.
29	59.	47.	32.	42.		39.	17.	40.	47.	43.	58.	53.
30	60.	41.	30.	24.		35.	35.	53.	48.	44.	52.	56.
31	53.		46.	19.		29.		46.		50.	54.	

TABLE F-9

1974-75 CHLORIDE PROFILE AT SEGMENT 5 USING U.S.G.S. STREAMFLOWS

(CONCENTRATIONS IN MG/L)

DAY	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP
1	51.	49.	35.	36.	27.	42.	14.	39.	42.	40.	50.	41.
2	43.	52.	42.	26.	28.	22.	25.	22.	38.	38.	53.	42.
3	39.	52.	37.	17.	23.	43.	39.	29.	47.	37.	45.	47.
4	48.	54.	28.	41.	33.	23.	28.	33.	36.	24.	49.	44.
5	51.	26.	46.	46.	28.	30.	39.	33.	45.	17.	37.	47.
6	57.	25.	45.	32.	31.	25.	30.	21.	48.	37.	42.	37.
7	51.	27.	38.	35.	28.	35.	41.	29.	33.	43.	44.	46.
8	42.	40.	41.	28.	29.	29.	27.	32.	40.	22.	36.	44.
9	55.	49.	41.	36.	41.	32.	36.	40.	36.	49.	48.	39.
10	61.	39.	29.	25.	33.	30.	38.	33.	39.	42.	51.	46.
11	38.	48.	30.	11.	33.	35.	42.	40.	35.	29.	46.	53.
12	64.	33.	25.	18.	34.	40.	30.	42.	45.	31.	40.	37.
13	47.	40.	40.	30.	36.	17.	32.	26.	47.	31.	37.	35.
14	54.	37.	38.	33.	30.	22.	45.	48.	49.	50.	37.	44.
15	48.	37.	45.	33.	41.	34.	22.	38.	40.	55.	47.	46.
16	48.	53.	49.	41.	22.	23.	31.	31.	40.	44.	43.	39.
17	40.	55.	30.	36.	29.	29.	35.	49.	31.	55.	46.	42.
18	41.	52.	32.	31.	25.	29.	41.	46.	42.	35.	41.	43.
19	51.	46.	38.	34.	12.	20.	41.	27.	45.	39.	45.	49.
20	56.	45.	23.	30.	35.	34.	49.	47.	43.	43.	44.	57.
21	44.	46.	32.	37.	40.	28.	45.	36.	37.	35.	40.	47.
22	66.	45.	36.	43.	30.	23.	47.	35.	42.	30.	41.	49.
23	53.	49.	44.	40.	3.	34.	38.	48.	49.	35.	49.	47.
24	37.	43.	44.	39.	16.	30.	17.	50.	44.	41.	40.	43.
25	47.	35.	36.	44.	19.	12.	22.	42.	46.	51.	49.	44.
26	52.	47.	25.	36.	11.	39.	24.	49.	31.	48.	34.	50.
27	45.	35.	30.	19.	31.	38.	26.	48.	42.	41.	51.	40.
28	51.	44.	33.	34.	24.	38.	16.	43.	34.	48.	50.	41.
29	39.	45.	33.	37.		21.	13.	48.	40.	49.	42.	47.
30	49.	44.	43.	28.		30.	30.	36.	49.	41.	52.	47.
31	58.		37.	22.		22.		45.		51.	48.	

TABLE F-10

1974-75 CHLORIDE PROFILE AT SEGMENT 5 USING PREDICTED STREAMFLOWS

(CONCENTRATIONS IN MG/L)

DAY	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP
1	41.	52.	48.	38.	21.	26.	15.	18.	43.	46.	46.	49.
2	48.	42.	42.	17.	32.	38.	32.	19.	34.	59.	55.	39.
3	51.	55.	34.	34.	25.	34.	34.	31.	45.	52.	47.	44.
4	54.	56.	42.	35.	26.	19.	30.	37.	50.	23.	50.	45.
5	50.	29.	45.	29.	30.	33.	33.	41.	42.	29.	44.	43.
6	42.	35.	43.	31.	29.	30.	30.	25.	51.	30.	50.	40.
7	51.	42.	37.	23.	23.	38.	40.	38.	38.	29.	53.	47.
8	38.	41.	33.	39.	29.	25.	41.	40.	35.	33.	58.	53.
9	46.	50.	39.	29.	63.	44.	53.	54.	37.	37.	51.	41.
10	51.	39.	23.	32.	28.	33.	46.	49.	42.	41.	44.	36.
11	49.	47.	27.	23.	36.	30.	34.	45.	42.	43.	53.	42.
12	37.	26.	31.	17.	33.	29.	39.	50.	45.	44.	50.	41.
13	59.	51.	50.	20.	31.	27.	42.	39.	48.	36.	41.	34.
14	56.	39.	34.	32.	37.	30.	35.	41.	42.	48.	50.	46.
15	47.	38.	38.	31.	23.	28.	40.	49.	32.	41.	53.	34.
16	41.	40.	34.	36.	43.	33.	32.	41.	32.	46.	37.	44.
17	48.	46.	36.	38.	26.	22.	49.	46.	32.	52.	53.	29.
18	46.	39.	20.	37.	35.	33.	31.	43.	46.	43.	39.	46.
19	26.	51.	37.	46.	38.	26.	30.	35.	34.	17.	48.	58.
20	45.	33.	33.	27.	44.	38.	39.	38.	36.	48.	49.	57.
21	47.	30.	44.	30.	50.	21.	40.	39.	38.	47.	52.	41.
22	43.	45.	41.	41.	41.	38.	40.	30.	55.	38.	58.	53.
23	49.	44.	37.	38.	19.	35.	36.	35.	55.	32.	38.	36.
24	43.	39.	44.	42.	23.	35.	35.	30.	36.	43.	37.	37.
25	42.	33.	22.	22.	20.	34.	20.	42.	27.	28.	42.	55.
26	57.	39.	34.	40.	16.	25.	18.	37.	40.	39.	55.	42.
27	55.	38.	24.	35.	21.	20.	31.	40.	48.	50.	56.	49.
28	58.	47.	26.	37.	20.	46.	31.	56.	46.	55.	53.	44.
29	54.	42.	41.	29.		12.	14.	53.	53.	59.	56.	70.
30	52.	42.	43.	23.		22.	45.	46.	49.	33.	50.	58.
31	43.		37.	25.		45.		44.		38.	61.	

APPENDIX G

Computer Programs

DECKFILE

```
C     SIMULATION OF STREAM RESPONSE TO TIME-VARIABLE, POINT-SOURCE
C     CHLORIDE LOADINGS USING CMF REACTORS IN SERIES
C
C
C     DEFINITIONS OF SYSTEM VARIABLES
C
C     WT(J,I1):     LOAD FROM PLANT J AT TIME I1 IN LBS./HR. (ARRAY)
C     NREAC:        TOTAL NUMBER OF REACTORS IN SERIES
C     NINC:         TOTAL NUMBER OF LOADINGS FOR EACH PLANT
C     TINC:         TIME BETWEEN LOADINGS (HRS.)
C     V:            SEGMENT VOLUMES (CUBIC FEET)
C     L:            SEGMENT LENGTHS (FEET)
C     CL(I):        CHARACTERISTIC LENGTH OF SEGMENT I (FEET)
C     E:            DISPERSION COEFFICIENT (SQ. MILES/DAY)
C     A:            INTERFACIAL AREA BETWEEN SEGMENTS (SQ. FEET)
C     SF1:          SCALE FACTOR TO CONVERT (Q/V)(C(I-1)-C(I))
C                   TO MG/L
C     SF2:          SCALE FACTOR TO CONVERT (W(T,J)/V) TO MG/L
C     SF3:          SCALE FACTOR TO CONVERT DISPERSION TERM TO MG/L
C     UC:           UPSTREAM BOUNDARY CHLORIDE CONCENTRATION (MG/L)
C     TIMC(I1):     ARRAY CONTAINING INDIVIDUAL LOAD TIMES
C     NEQ:          NUMBER OF EQUATIONS (INPUT)
C     T:            INDEPENDENT VARIABLE- TIME (HRS.)
C     C:            CURRENT VALUE OF DEPENDENT VARIABLE- CONC. (MG/L)
C     KD:           ORDER OF THE DIFFERENTIAL EQUATIONS IN THE SYSTEM
C     EP(I):        PARAMETER USED TO CONTROL THE LOCAL ERROR IN
C                   REACTOR I
C     H:            CURRENT VALUE OF THE STEPSIZE
C     DELT:         SPECIFIES POINTS WHERE OUTPUT IS DESIRED
C     TFINAL:       TOTAL LENGTH OF SIMULATION (HRS.)
C     KSTEP:        NUMBER OF INTEGRATION STEPS TAKEN
C
C
      IMPLICIT REAL*8 (A-H,O-Z)
      REAL*8 L(25),V(25),CL(25),AC(721)
      REAL*4 WT(361,3),TIMC(361),S(361,3),EP(25)
      COMMON WT,TIMC,I1,S
      DIMENSION F(25),DT(17,25),C(25),YN(25),KQ(25)
C      SET PARAMETERS AND INITIAL CONDITIONS
      CALL TIME(0)
      KD=1
      MXSTEP=5000
      HMINA=0.
      H=1.
      READ(5,1000) NREAC,NREAC1,NREAC2,NREAC3,NREAC4,NINC,NPLOT,LT
 1000 FORMAT(5I5)
      READ(5,1001) E,A,UC,SF1,SF2,SF3,TQ,TINC,Q,DELT,TF,(L(I),
     1I=1,NREAC),TSTART,TSTOP,TOUT
 1001 FORMAT(4F14.6)
      DO 45 JP=1,3
   45 READ(5,1002) (WT(IP,JP),IP=1,361)
 1002 FORMAT(4F9.2)
      NREACM=NREAC-1
      V(1)=L(1)*A
      V(NREAC)=L(NREAC)*A
      CL(1)=L(1)
      CL(NREAC)=(L(NREACM)+L(NREAC))/2.
      DO 13 I=2,NREACM
      V(I)=L(I)*A
   13 CL(I)=(L(I-1)+L(I))/2.
```

```
      IF(LT .EQ. 1)GO TO 12
      DO 8 I=1,NREAC
    8 C(I)=UC
      GO TO 15
   12 READ(5,1009) (C(I),I=1,NREAC)
 1009 FORMAT(4F10.4)
   15 NEQ=NREAC
      TFINAL=TSTOP
      T=TSTART
      HMAXA=TINC/TF
      E2=SF2
      E3=E*A*SF3
      WRITE(6,195) NREAC,NREAC1,NREAC2,NREAC3,NREAC4,NINC
  195 FORMAT('1',5X,'NREAC  = ',I2,9X,'NREAC1 = ',I2,9X,'NREAC2 = ',I2,/
     16X,'NREAC3 = ',I2,9X,'NREAC4 = ',I2,9X,'NINC  = ',I3)
      WRITE(6,196) TQ,TINC,DELT,HMINA,HMAXA,NEQ
  196 FORMAT(6X,'TQ = ',F4.0,11X,'TINC = ',F2.0,11X,'DELT = ',F2.0,/
     16X,'HMINA = ',F2.0,10X,'HMAXA = ',F2.0,10X,'NEQ = ',I2)
      WRITE(6,197) Q,E,UC,A,NREACM
  197 FORMAT(6X,'Q = ',F7.0,9X,'E = ',F2.1,/
     16X,'UC = ',F2.0,13X,'A = ',F4.0,12X,'NREACM = ',I2)
      WRITE(6,198) WT(1,1),WT(1,2),WT(1,3)
      WRITE(6,199) WT(2,1),WT(2,2),WT(2,3)
      WRITE(6,201) WT(361,1),WT(361,2),WT(361,3)
  198 FORMAT(6X,'WT(1,1) = ',F5.2,5X,'WT(1,2) = ',F5.2,5X,'WT(1,3) = ',F5.2)
  199 FORMAT(6X,'WT(2,1) = ',F5.2,5X,'WT(2,2) = ',F5.2,5X,'WT(2,3) = ',F5.2)
  201 FORMAT(6X,'WT(361,1) = ',F5.2,3X,'WT(361,2) = ',F5.2,3X,'WT(361,3) = ',F5.2)
      DO 191 I=1,21,5
      I1=I+1
      I2=I+2
      I3=I+3
      I4=I+4
  191 WRITE(6,192) I,L(I),I1,L(I1),I2,L(I2),I3,L(I3),I4,L(I4)
  192 FORMAT(6X,'L(',I2,') = ',F5.0,7X,'L(',I2,') = ',F5.0,7X,'L(',I2,')
     1 = ',F5.0,7X,'L(',I2,') = ',F5.0,7X,'L(',I2,') = ',F5.0)
      DO 189 I=1,21,5
      I1=I+1
      I2=I+2
      I3=I+3
      I4=I+4
  189 WRITE(6,202) I,C(I),I1,C(I1),I2,C(I2),I3,C(I3),I4,C(I4)
  202 FORMAT(6X,'C(',I2,') = ',F8.4,4X,'C(',I2,') = ',F8.4,4X,'C(',I2,')
     1 = ',F8.4,4X,'C(',I2,') = ',F8.4,4X,'C(',I2,') = ',F8.4)
      DO 5 IR=1,NINC
    5 TIMC(IR)=DFLOAT(IR-1)*TINC
      I1=T/2+1
C      NOW MAKE FIRST ENTRY
      CALL DVDQ(NEQ,T,C,F,KD,EP,IFLAG,H,HMINA,HMAXA,DELT,TFINAL,
     1MXSTEP,KSTEP,KEMAX,EMAX,KQ,YN,DT)
      GO TO 2
    1 CALL DVDQ1
    2 GO TO (10,10,30,30,70,50,50,50), IFLAG
C      EVALUATE DIFFERENTIAL EQUATION, IFLAG=1 OR 2
   10 DO 9 I=1,NEQ
      EP(I)=.02*DABS(C(I))
    9 EP(I)=-AMAX1(EP(I),.02)
      E1=Q*SF1
C      FIRST REACTOR NEVER RECEIVES LOADINGS
      J=0
      F(1)=E1*(UC-C(1))/V(1)+E2*W(T,J)/V(1)+
```

```
     1(E3*(UC-C(1)))/(V(1)*CL(1))+(E3*(C(2)-C(1)))/(V(1)*CL(2))
C      LAST REACTOR NEVER RECEIVES LOADINGS
       J=0
       F(NREAC)=E1*(C(NREACM)-C(NREAC))/V(NREAC)+E2*W(T,J)/V(NREAC)+
     1(E3*(C(NREACM)-C(NREAC)))/(V(NREAC)*CL(NREAC))
       DO 6 I=2,NREACM
       J=0
       IF(I .EQ. NREAC1) J=1
       IF(I .EQ. NREAC2) J=2
       IF(I .EQ. NREAC3) J=3
       IF(I .EQ. NREAC4) J=4
    6  F(I)=E1*(C(I-1)-C(I))/V(I)+E2*W(T,J)/V(I)+
     1(E3*(C(I-1)-C(I)))/(V(I)*CL(I))+(E3*(C(I+1)-C(I)))/(V(I)*CL(I+1))
       GO TO 1
C      OUTPUT, IFLAG=3, OR FINISHED, IFLAG=4
   30  IF(T .GE. TOUT)GO TO 32
       GO TO 44
   32  WRITE(6,1003)T
 1003  FORMAT('0','TIME= ',F6.2,' HRS. ')
       WRITE(6,1004)(K,C(K),K=1,NREAC)
 1004  FORMAT('0',3('SEGMENT= ',I2,'     C = ',
     1E11.5,' MG/L    '))
       LZ=T+1.D0
       AC(LZ)=C(NPLOT)
   44  IF(IFLAG .EQ. 3) GO TO 1
       IF(IFLAG .EQ. 4) GO TO 75
C      ERROR CONDITION, IFLAG=6, 7 OR 8
   50  CALL ERROR
       STOP
   70  GO TO 1
   75  WRITE(7,1020)(AC(I),I=1,LZ)
 1020  FORMAT(8F10.4)
       STOP
       END
C
C      LINEAR INTERPOLATION OF TIME-VARIABLE CHLORIDE LOADS FROM POINT-SOURCES
       DOUBLE PRECISION FUNCTION W(T,J)
       IMPLICIT REAL*8 (A-H,O-Z)
       REAL*4 WT(361,3),TIMC(361),S(361,3),TT,VV
       COMMON WT,TIMC,I1,S
       IF(J .EQ. 0) GO TO 6
       IF(T .GT. TIMC(I1+1)) I1=I1+1
       IF(T .LE. TIMC(I1+1)) GO TO 5
       WRITE(6,100)
  100  FORMAT('TIME INCREMENT GREATER THAN TINC')
       RETURN
     5 W=((((T-TIMC(I1))/(TIMC(I1+1)-TIMC(I1)))*(WT(I1+1,J)
      1-WT(I1,J)))+WT(I1,J))
       RETURN
     6 W=0.
       RETURN
       END
```

MIAMI

```
C     GREAT MIAMI RIVER CHLORIDE MODEL USING CMF REACTORS IN SERIES
C
C      INPUT DATA SOURCES: OHIO EPA, MIAMI CONSERVANCY DISTRICT, U.S.G.S. STREAMFLOW RECORDS
C      TIME REFERENCE    : OCTOBER 1973 - SEPTEMBER 1975
C      POINT-SOURCE LOADS: NATIONAL LEAD AND GULF OIL
C      RIVER STRETCH     : FROM RIVER MILE 28.0 TO RIVER MILE 3.0
C
C      W(T,J)            : CHLORIDE LOAD FROM PLANT J AT TIME T (LBS./DAY)
C      Q(T)              : STREAMFLOW AT TIME T (CFS)
C      UC(T)             : UPSTREAM BOUNDARY CONDITION AT TIME T (MG/L)
C      A(T)              : INTERFACIAL AREA BETWEEN SEGMENTS AT TIME T (SQ. FEET)
C      L(I)              : LENGTH OF SEGMENT I (FEET)
C      NREAC             : TOTAL NUMBER OF REACTORS IN SERIES
C      TINC              : TIME BETWEEN CHLORIDE LOADING DATA POINTS (DAYS)
C      NINC              : TOTAL NUMBER OF LOADING DATA POINTS PER PLANT
C      E                 : DISPERSION COEFFICIENT (SQ. MILES/DAY)
C      SF1               : SCALE FACTOR CONVERTING (Q*C)/(L*A) TO MG/L-DAY
C      SF2               : SCALE FACTOR CONVERTING W/(L*A) TO MG/L-DAY
C      SF3               : SCALE FACTOR CONVERTING (E*A*C)/(L*A) TO MG/L-DAY
C      TIMC(I1)          : ARRAY CONTAINING INDIVIDUAL LOAD TIMES
C      NEQ               : NUMBER OF EQUATIONS
C      T                 : INDEPENDENT VARIABLE- TIME(DAYS)
C      C                 : CURRENT VALUE OF DEPENDENT VARIABLE - CONC.(MG/L)
C      KD                : ORDER OF THE DIFFERENTIAL EQUATIONS IN THE SYSTEM
C      EP(I)             : PARAMETER USED TO CONTROL THE LOCAL ERROR IN REACTOR I
C      H                 : CURRENT VALUE OF THE STEPSIZE
C      DELT              : SPECIFIES POINTS WHERE OUTPUT IS DESIRED
C      TFINAL            : TOTAL LENGTH OF THE SIMULATION (DAYS)
C      KSTEP             : NUMBER OF INTEGRATION STEPS TAKEN
C
C
      IMPLICIT REAL*8 (A-H,O-Z)
      REAL*8 L(5)
      REAL*4 WT(365,2),TIMC(365),EP(5),QT(365),UCT(365)
      COMMON TIMC,I1
      COMMON/SUB1/WT
      COMMON/SUB2/UCT
      COMMON/SUB3/QT
      DIMENSION F(5),DT(17,5),C(5),YN(5),KQ(5)
C      SET PARAMETERS AND INITIAL CONDITIONS
      KD=1
      MXSTEP=5000
      HMINA=0.
      HMAXA=1.
      H=1.
      NPLOT1=5
      READ(5,1000)NREAC,NREAC1,NREAC2,NINC,NEQ,NREACM,LT,KT
      READ(5,1000)KZ,MZ,NZ,MQ,NQ,MT,NT
      READ(5,1001)E,SF1,SF2,SF3,TINC,DELT,(L(I),I=1,NREAC),T,TFINAL
      READ(5,1002)(WT(IP,1),IP=1,365)
      READ(5,1002)(WT(IP,2),IP=1,365)
      IF(KT .EQ. 1)GO TO 12
      READ(5,1004)(C(I),I=1,NREAC)
      GO TO 14
   12 DO 17 I=1,NREAC
   17 C(I)=0.
   14 IF(LT .EQ. 1)GO TO 16
      IF(KZ .EQ. 1)GO TO 21
      GO TO 13
   21 READ(5,1007)(UCT(I),I=MZ,NZ)
```

```
      GO TO 22
   13 READ(5,1007) (UCT(I),I=MQ,NQ)
      READ(5,1007) (UCT(I),I=MT,NT)
      GO TO 22
   16 DO 20 I=1,365
   20 UCT(I)=0.0
   22 READ(5,1008) (QT(J),J=1,365)
      WRITE(6,1010) NREAC,DELT,E
      WRITE(6,1011) NREAC1,HMINA,NREACM
      WRITE(6,1012) NREAC2,HMAXA,SF1
      WRITE(6,1013) NINC,KD,SF2
      WRITE(6,1014) TINC,NEQ,SF3
      WRITE(6,1039) T,TFINAL
      WRITE(6,1015) WT(1,1),WT(2,1),WT(365,1)
      WRITE(6,1016) WT(1,2),WT(2,2),WT(365,2)
      WRITE(6,1019) QT(1),QT(2),QT(365)
      WRITE(6,1020) UCT(1),UCT(2),UCT(365)
      I=1
      I1=I+1
      I2=I+2
      I3=I+3
      I4=I+4
      WRITE(6,1021) I,L(I),I1,L(I1),I2,L(I2),I3,L(I3),I4,L(I4)
      WRITE(6,1023) I,C(I),I1,C(I1),I2,C(I2),I3,C(I3),I4,C(I4)
 1000 FORMAT(8I5)
 1001 FORMAT(6F10.0)
 1002 FORMAT(8F10.0)
 1004 FORMAT(8F10.0)
 1007 FORMAT(16F5.0)
 1008 FORMAT(7F10.0)
 1010 FORMAT('1',13X,'NREAC      =    ',I2,6X,'DELT       =    ',F2.0,
     15X,'E          =        ',F2.0)
 1011 FORMAT(14X,'NREAC1     =    ',I2,6X,'HMINA      =    ',F2.0,
     15X,'NREACM       =       ',I2)
 1012 FORMAT(14X,'NREAC2     =    ',I2,6X,'HMAXA      =    ',F2.0,
     15X,'SF1',10X,'=     ',F6.0)
 1013 FORMAT(14X,'NINC',7X,'=  ',I3,6X,'KD',9X,'=   ',I2,6X,'SF2',10X,'=   ',
     1F7.0)
 1014 FORMAT(14X,'TINC',7X,'=',4X,F2.0,5X,'NEQ',8X,'=   ',I2,6X,'SF3',10X,'= ',
     1F9.0)
 1015 FORMAT('-',13X,'WT(1,1)',4X,'=',F6.0,5X,'WT(2,1)',4X,'=',F6.0,5X,
     1'WT(365,1)',4X,'=',F10.0)
 1016 FORMAT(14X,'WT(1,2)',4X,'=',F6.0,5X,'WT(2,2)',4X,'=',F6.0,5X,
     1'WT(365,2)',4X,'=',F10.0)
 1019 FORMAT(14X,'QT(1)',6X,'= ',F5.0,5X,'QT(2)',6X,'= ',F5.0,
     15X,'QT(365)',6X,'=',5X,F5.0)
 1020 FORMAT(14X,'UCT(1)     =    ',F3.0,5X,'UCT(2)',5X,'=',3X,F3.0,
     15X,'UCT(365)',5X,'=',7X,F3.0,/)
 1021 FORMAT(16X,'L(',I1,') = ',F6.0,6X,'L(',I1,') = ',F6.0,6X,
     1'L(',I1,') = ',F6.0,6X,'L(',I1,') = ',F6.0,6X,'L(',I1,') = ',F6.0)
 1023 FORMAT(16X,'C(',I1,') = ',F6.0,6X,'C(',I1,') = ',F6.0,6X,
     1'C(',I1,') = ',F6.0,6X,'C(',I1,') = ',F6.0,6X,'C(',I1,') = ',F6.0)
 1039 FORMAT(14X,'T',10X,'=',2X,F4.0,5X,'TFINAL',5X,'=  ',F4.0)
      DO 5 IR=1,NINC
    5 TINC(IR)=DFLOAT(IR)*TINC
      I1=T
C      NOW MAKE FIRST ENTRY
      CALL DVDQ(NEQ,T,C,F,KD,EP,IFLAG,H,HMINA,HMAXA,DELT,TFINAL,
     1MXSTEP,KSTEP,KEMAX,EMAX,KQ,YN,DT)
      GO TO 2
```

```
    1 CALL DVDQ1
    2 GO TO (10,10,30,30,70,50,50,50), IFLAG
C      EVALUATE DIFFERENTIAL EQUATION, IFLAG=1 OR 2
   10 DO 9 I=1,NEQ
      EP(I)=.02*DABS(C(I))
    9 EP(I)=-AMAX1(EP(I),.02)
C      FIRST REACTOR NEVER RECEIVES LOADINGS
      J=0
      A=383.*Q(T)**.15
      F(1)=SF1*Q(T)*(UC(T)-C(1))/(L(1)*A)+SF2*W(T,J)/(L(1)*A)+(E*A*SF3*(
     1C(2)-2*C(1)+UC(T)))/(L(1)*L(1)*A)
C      LAST REACTOR NEVER RECEIVES LOADINGS
      J=0
      F(NREAC)=SF1*Q(T)*(C(NREACM)-C(NREAC))/(L(NREAC)*A)+SF2*W(T,J)/(L(
     1NREAC)*A)+(E*A*SF3*(C(NREACM)-C(NREAC)))/(L(NREAC)*L(NREAC)*A)
      DO 6 I=2,NREACM
      J=0
      IF(I .EQ. NREAC1) J=1
      IF(I .EQ. NREAC2) J=2
    6 F(I)=SF1*Q(T)*(C(I-1)-C(I))/(L(I)*A)+SF2*W(T,J)/(L(I)*A)+(E*A*SF3*
     1(C(I+1)-2*C(I)+C(I-1)))/(L(I)*L(I)*A)
      GO TO 1
C      OUTPUT, IFLAG=3, OR FINISHED, IFLAG=4
   30 IT=T
      IF(T .EQ. 0) GO TO 70
      IF(T .GE. 1 .AND. T .LE. 31) GO TO 53
      IF(T .GE. 32 .AND. T .LE. 61) GO TO 57
      IF(T .GE. 62 .AND. T .LE. 92) GO TO 58
      IF(T .GE. 93 .AND. T .LE. 123) GO TO 60
      IF(T .GE. 124 .AND. T .LE. 151) GO TO 63
      IF(T .GE. 152 .AND. T .LE. 182) GO TO 68
      IF(T .GE. 183 .AND. T .LE. 212) GO TO 72
      IF(T .GE. 213 .AND. T .LE. 243) GO TO 75
      IF(T .GE. 244 .AND. T .LE. 273) GO TO 78
      IF(T .GE. 274 .AND. T .LE. 304) GO TO 80
      IF(T .GE. 305 .AND. T .LE. 335) GO TO 83
      IF(T .GE. 336 .AND. T .LE. 365) GO TO 88
   53 WRITE(6,1025) IT,(K,C(K),K=1,NREAC)
      GO TO 90
   57 IT=T-31.
      WRITE(6,1026) IT,(K,C(K),K=1,NREAC)
      GO  TO 90
   58 IT=IT-61.
      WRITE(6,1027) IT,(K,C(K),K=1,NREAC)
      GO TO 90
   60 IT=T-92.
      WRITE(6,1028) IT,(K,C(K),K=1,NREAC)
      GO TO 90
   63 IT=T-123.
      WRITE(6,1029) IT,(K,C(K),K=1,NREAC)
      GO TO 90
   68 IT=T-151.
      WRITE(6,1030) IT,(K,C(K),K=1,NREAC)
      GO TO 90
   72 IT=T-182.
      WRITE(6,1031) IT,(K,C(K),K=1,NREAC)
      GO TO 90
   75 IT=T-212.
      WRITE(6,1032) IT,(K,C(K),K=1,NREAC)
      GO TO 90
```

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   78 IT=T-243.
      WRITE(6,1033)IT,(K,C(K),K=1,NREAC)
      GO TO 90
   80 IT=T-273.
      WRITE(6,1034)IT,(K,C(K),K=1,NREAC)
      GO TO 90
   83 IT=T-304.
      WRITE(6,1035)IT,(K,C(K),K=1,NREAC)
      GO TO 90
   88 IT=T-335.
      WRITE(6,1036)IT,(K,C(K),K=1,NREAC)
   90 WRITE(7,1038)C(NPLOT1)
 1025 FORMAT('0',10X,'OCTOBER ',I2,2X,5(5X,'C',I1,' = ',F4.0,' MG/L'))
 1026 FORMAT('0',10X,'NOVEMBER ',I2,1X,5(5X,'C',I1,' = ',F4.0,' MG/L'))
 1027 FORMAT('0',10X,'DECEMBER ',I2,1X,5(5X,'C',I1,' = ',F4.0,' MG/L'))
 1028 FORMAT('0',10X,'JANUARY ',I2,2X,5(5X,'C',I1,' = ',F4.0,' MG/L'))
 1029 FORMAT('0',10X,'FEBRUARY ',I2,1X,5(5X,'C',I1,' = ',F4.0,' MG/L'))
 1030 FORMAT('0',10X,'MARCH ',I2,4X,5(5X,'C',I1,' = ',F4.0,' MG/L'))
 1031 FORMAT('0',10X,'APRIL ',I2,4X,5(5X,'C',I1,' = ',F4.0,' MG/L'))
 1032 FORMAT('0',10X,'MAY ',I2,6X,5(5X,'C',I1,' = ',F4.0,' MG/L'))
 1033 FORMAT('0',10X,'JUNE ',I2,5X,5(5X,'C',I1,' = ',F4.0,' MG/L'))
 1034 FORMAT('0',10X,'JULY ',I2,5X,5(5X,'C',I1,' = ',F4.0,' MG/L'))
 1035 FORMAT('0',10X,'AUGUST ',I2,3X,5(5X,'C',I1,' = ',F4.0,' MG/L'))
 1036 FORMAT('0',10X,'SEPTEMBER ',I2,5(5X,'C',I1,' = ',F4.0,' MG/L'))
 1038 FORMAT(' ',F7.0)
      IF(IFLAG .EQ. 3) GO TO 1
      IF(IFLAG .EQ. 4) STOP
C      ERROR CONDITION, IFLAG=6, 7 OR 8
   50 CALL ERROR
      STOP
   70 GO TO 1
      END
C
C EVALUATION OF TIME-VARIABLE CHLORIDE LOADS FROM PLANT EFFLUENTS
      DOUBLE PRECISION FUNCTION W(T,J)
      IMPLICIT REAL*8 (A-H,O-Z)
      REAL*4 WT(365,2),TIMC(365)
      COMMON TIMC,I1
      COMMON/SUB1/WT
      IF(J .EQ. 0) GO TO 6
      IF(T .GT. TIMC(I1+1)) I1=I1+1
      IF(T .LE. TIMC(I1+1)) GO TO 5
      WRITE(6,100)
  100 FORMAT('TIME INCREMENT GREATER THAN TINC')
      RETURN
    5 W=WT(I1+1,J)
      RETURN
    6 W=0.
      RETURN
      END
C
C EVALUATION OF TIME-VARIABLE BACKGROUND CHLORIDE CONC.S
      DOUBLE PRECISION FUNCTION UC(T)
      IMPLICIT REAL*8 (A-H,O-Z)
      REAL*4 UCT(365),TIMC(365)
      COMMON TIMC,I1
      COMMON/SUB2/UCT
      IF(T .GT. TIMC(I1+1)) I1=I1+1
      IF(T .LE. TIMC(I1+1)) GO TO 5
      WRITE(6,100)
```

```
 100 FORMAT('TIME INCREMENT GREATER THAN TINC')
     RETURN
   5 UC=UCT(I1+1)
     RETURN
     END
C
C EVALUATION OF TIME-VARIABLE STREAMFLOW
     DOUBLE PRECISION FUNCTION Q(T)
     IMPLICIT REAL*8 (A-H,O-Z)
     REAL*4 QT(365),TINC(365)
     COMMON TINC,I1
     COMMON/SUB3/QT
     IF(T .GT. TINC(I1+1)) I1=I1+1
     IF(T .LE. TINC(I1+1)) GO TO 5
     WRITE(6,100)
 100 FORMAT('TIME INCREMENT GREATER THAN TINC')
     RETURN
   5 Q=QT(I1+1)
     RETURN
     END
```

BIBLIOGRAPHY

Anderson, M.W., and Day, H.J. "Regional Management of Water Quality - A Systems Approach", J. Water Pollut. Contr. Fed., 40(10): 1679-1687, 1968.

Arnold, J.C., "A Markowian Sampling Policy Applied to Water Quality Monitoring of Streams", Biometrics, 26(4): 739-747, 1970.

Bauer, D.P., "Time of Travel of Water in the Great Miami River - Dayton to Cleves, Ohio", Geological Survey Circular 546, Washington 1968.

Beckers, V. and Chamberlain, G., "Design of Cost-Effective Water Quality Surveillance Systems", Environmental Protection Agency, Socioeconomic Studies Series Report EPA-600/5-74-004, January 1974.

Backers, V., Chamberlain, G., and Grimsrud, G., "Quantitative Methods for Preliminary Design of Water Quality Surveillance Systems", Environmental Studies Series Report EPA-R5-72-001, November 1972.

Blachman, N. and Proschan, F., "Optimum Search for Objects Having Unknown Arrival Times", Operations Research, 7(5): 625-638, 1959.

Boyce, W.E. and DiPrima, R.C., Elementary Differential Equations and Boundary Value Problems, Second Edition, John Wiley and Sons, Inc., 1969.

Canale, R.P., Modeling Biochemical Prossesses in Aquatic Ecosystems, Ann Arbor Science Publishers,Inc., 1976.

Cochran, W.G., Sampling Techniques, John Wiley and Sons, Inc, 1963.

Deininger, R.A., Models for Environmental Pollution Control, Ann Arbor Science Publishers, Inc., 1973.

Donigian, A., Jr. and Waggy, W.H., "Simulation - A Tool for Water Resource Management", Water Resources Bulletin, 10 (2): 229-244, 1974.

Duncan, A.J., Quality Control and Industrial Statistics, 3rd Edition, Richard D. Irwin, Inc., 1965.

Gibra, I.N., Probability and Statistical Inference for Scientists and Engineers, Prentice-Hall, Inc., 1973.

Glover, R.F., "Dispersion of Dissolved or Suspended Materials in Flowing Streams", U.S.Geological Survey Professional Paper 433B, 1964.

Gunnerson, C.G., Optimizing Sampling Intervals in Tidal Estuaries", J. Sanit. Eng. Div., Proc. Amer. Soc. Eng., 92(SA2): 103-125, 1966.

Horton, R.K., "An Index - Number System for Rating Water Quality", J. Water Pollut. Contr. Fed., 37(3): 300-306, 1965.

Huck, P.M. and Farquhar, G.J., "Water Quality Models Using the Box-Jenkins Method", J. Env. Eng. Div., Proc. Amer. Soc. Civ. Eng., 100(EE3): 733-752,1974.

Hydroscience, Inc., Simplified Mathematical Modeling of Water Quality, Environmental Protection Agency, Washington, D.C., March 1971.

Kerri, K.D., "A Dynamic Model for Water Quality Control", J. Water Pollut. Contr. Fed., 39(5): 772-786, 1967.

Kleijnen, J.P.C., Statistical Techniques in Simulation, Marcel Dekker, Inc., 1974.

Kneese, A., and Bower, B.T., Managing Water Quality Economics Technology, Institutions, John Hopkins Press, Baltimore, 1968.

Koivo, A.J. and Phillips, G., "Optimal Estimation of DO, BOD, and Stream Parameters Using a Dynamic Discrete Time Model", Water Resources Research, 12(4): 705-711, 1976.

Linsley, R.K. Jr., Kohler, M.A., and Paulhus, J.L.H., Hydrology for Engineers, McGraw-Hill, Inc., 1958.

Lombardo, P.S., and Ott, R.F., "Water Quality Simulation and Application", Water Resources Bulletin, 10(1): 1-9, 1974.

Loucks, D.P. and Lynn, W.R., "Probabilistic Models for Predicting Stream Quality", Water Resources Research, 2(3): 593-605, 1966.

Manhattan Summer Conference, Mathematical Modeling of Natural Systems, Manhattan College, New York, N.Y, 1976.

Mood, A.M. and Graybill, F.A., Introduction to the Theory of Statistics, McGraw-Hill, Inc., 1974.

Moore, S.F., "Estimation Theory Applications to Design of Water Quality Monitoring Systems", J. Hydraul. Div., Proc. Amer. Soc. Civ. Eng., 99(HY5): 815-831, 1973.

Moore, S.F., Dandy, G.C., and deLucia, R.J., "Describing Variance With a Simple Water Quality Model and Hypothetical Sampling Programs", Water Resources Research, 12(4): 795-804, 1976.

Naylor, T.H., Balintfy, J.S., Burdick, D.S., and Chu, K., Computer Simulation Techniques, Wiley, 1967.

NUS Corporation, "Design of Water Quality Surveillance Systems, U.S.Dept. of the Interior, Federal Water Quality Administration, Washington, D.C., Report No. 16090 DBJ 08/70, August 1970.

O'Connor, D.J., and Mueller, J.A., "A Water Quality Model of Chlorides in the Great Lakes", J. Sanit. Eng. Div., Prox. Amer. Soc. Civ. Eng., 96(SA4): 955-975, 1970.

Ott, W.R., Proceedings of the Conference on Environmental Modeling and Simulation, U.S. Environmental Protection Agency, EPA 600/9-76-016, July 1976.

Oxley, N.C. and Wallis, B.F.J., "The Application of Statistical Techniques to River Quality Management", Water Pollution Control, 73(4): 449-455, 1974.

Page, C. and Warn, A.E., "Water Quality Considerations in the Design of Water Resource Systems", Water Research, 8(11): 969-975, 1974.

Pollock, Stephen M., "Minimum-Cost Checking Using Imperfect Information", Management Science, 13(7): 454-465, 1967.

Pratt, J.W., Statistical and Mathematical Aspects of Pollution Problems, Marcel Dekker, Inc., 1974.

Public Law 92-500, "Federal Water Pollution Control Act Amendments of 1972", STAT 816 et. seq., October 18, 1973.

Revelle, C.S., Loucks, D.P., and Lynn, W.R., "A Management Model for Water Quality Control", J. Water Pollut-Contr. Fed., 39(7): 1164-1183, 1967.

Rich, L.G., Environmental Systems Engineering, McGraw-Hill, Inc., 1973.

Sayers, W.T., "Water Quality Surveillance", Environmental Science and Technology, 5(2): 114-119, 1971.

Schofield, W.R., and Krutchkoff, R.G. "Stochastic Model of a Dynamic Eutrophic Estuary", J. Env. Eng. Div. Proc. Amer. Soc. Civ. Eng., 100(EE3): 613-628, 1974.

Shih, Chia Shun, "System Optimization for River Basin Water Quality Management", J. Water Pollut. Contr. Fed., 42(10): 1792-1804, 1970.

Smith, E.T. and Morris, A.R., "Systems Analysis for Optimal Water Quality Management", J. Water Pollut. Contr. 41(9): 1635-1646, 1969.

Thoman, R.V., "Time Series Analysis of Water Quality Data", J. Sanit. Eng. Div., Proc. Amer. Soc. Civ. Eng., 93(SA1): 1-23, 1967.

Thoman , R.V., Systems Analysis and Water Quality Management, Environmental Research and Applications, Inc., 1972.

Vanderholm, D.H. "Planning Water Quality Surveillance", Unpublished Ph.D. Thesis, Department of Agricultural Engineering, Colorado State University, Fort Collins, 1972.

Vansteenkiste, G.C., Computer Simulation of Water Resource Systems, North-Holland Publishing Co., 1975.

Velz, C.J., "Sampling for Effective Evaluation of Stream Pollution", Sewage and Industrial Wastes, 22(5) 666-682, 1950.

Volk, W., Applied Statistics for Engineers, McGraw-Hill, Inc., 1968.

Ward, R.C. Data Aquisition Systems in Water Quality Management, Report No. 16909 FUO 12/71 Environmental Protection Agency, Washington, D.C. 1971.

Wastler, R.A., Spectral Analysis Applications in Water Pollution Control, Federal Water Pollution Control Administration, Washington, D.C. 1969.

Water Quality Criteria, Report of the National Technical Advisory Committee, U.S. Dept. of Interior, Washington, D.C. 1968.

Weber, W.J., Jr., Physiochemical Processes for Water Quality Wiley-Interscience, 1972.

Wisler, C.O. and Brater, E.F., Hydrology, John Wiley and Sons, Inc., 1959.

Yevjevich, V., Probability and Statistics in Hydrology, Water Resources Publications, 1972.