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Use of Mathematical Models in Water Quality Control Studies

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USE OF MATHEMATICAL MODELS
IN
WATER QUALITY CONTROL STUDIES

FEDERAL WATER POLLUTION CONTROL ADMINISTRATION

DEPARTMENT OF THE INTERIOR

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ABSTRACT

Mathematical models were utilized to study water pollution control programs in a river basin. Sensitivity analyses, with a steady state model, showed substantial variation of cost for sewage treatment, depending upon stream purification parameter selections. When actual parameters are less favorable than design values, quality standards may not be met; these effects are more serious with lower levels of treatment.

An unsteady state model was developed to trace a time profile at any specified station in terms of flow and quality as BOD, dissolved oxygen, coliforms, and chlorides while upstream discharge, water temperature, and solar radiation vary. The techniques assume that, for short reaches and/or times, steady state conditions apply without undue loss of accuracy. A new empirical procedure was developed to route unsteady stream flow.

The time varying model was used to investigate the effectiveness of an assumed configuration of treatment plants when the stream's assimilative capacity varies with distance and time. Studies showed that DO values are worse at times than the steady state value, and that susceptibility to poorer conditions increases with higher BOD releases. Lower treatment levels also result in a greater range of river conditions than high levels. Sensitivity analyses of stream parameters were also made with the time varying model.

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Introduction

Mathematical models, including both steady state and time varying versions, were developed and utilized to study water pollution control programs. Although hypothetical, these models are based to a large extent upon population and other river and community data for the Merrimack River Basin in northeastern Massachusetts. This basin is 302 square miles in area, with a design population of 579,000 persons. The stream is 50.9 miles long and has a discharge of 650 mgd during the critical low flow period. It may be stated, without substantial qualification, that the models were developed with realistic data such as those obtained with an actual river basin. The studies were limited to effects for non-tidal streams.

In the initial proposal for this project, it was stated that an analysis of data collection and processing programs of the water pollution control agencies of the New England States would permit a comprehensive study to establish a scientific basis for judging the economic efficiency of such programs. Because adequate information was not available for a systematic statistical analysis, the research focused on methodologies for utilizing data rather than on a quantitative evaluation of the costs of data collection and processing. The emphasis was on the relationships between stream parameter selection and the cost and effectiveness of systems of waste water treatment plants.

Although the models were studied within a rather limited framework of assumptions and results, the capabilities of such models, in terms of producing better design decisions, have been demonstrated. It is most desirable to coordinate individual field and laboratory studies, automatic monitoring programs, and mathematical model investigations. All of these methods are needed in the efficient operation of water pollution control programs.

Significant Results

The following are the most significant conclusions of this research project:

1. While a steady state river model is a practicable tool for selecting levels of waste water treatment in a system, the levels of treatment and the consequent costs can be quite sensitive to the river parameters chosen for the analysis.

2. An unsteady state river model incorporating variations, with distance and time, of flow, temperature, solar radiation, and stream parameters can be developed, and can be used to produce a time profile of river quality at any specified location.

3. Considering the present inadequate state of the art in estimating stream parameters, there can be no assurance that treatment plant systems will maintain specified river quality. If parameters are less favorable than assumed, susceptibility to stream conditions which do not meet quality standards increases with lower levels of waste water treatment.

Studies with Steady State Mathematical Model

Input data for the steady state model include hydraulic constants and stream purification rate constants for each reach of the river, population and economic information for each community, and assumptions with respect to administrative decisions. If the levels of sewage treatment are specified for each community, a computer program evaluates water quality throughout the length of the stream, and the costs and benefits of the water pollution control program. If a single water quality criterion is specified for each river section (e.g. in terms of dissolved oxygen), an optimizing routine determines desirable levels of sewage treatment.

Two surveys made prior to this research served as the basis of field data. Both surveys used the dissolved oxygen concentration as the principal criterion of stream quality. Equations determining changes in BOD and DO are by Streeter and Phelps, as modified by Camp. These formulas require values of rate parameters for deoxygenation (k_1), reaeration (k_2), settling out of BOD to bottom deposits (k_3), resuspension of BOD from bottom deposits (p), and oxygenation by photosynthesis ~~of algae (a)~~. Each parameter chosen as a basis for design affects the planning of a system of treatment plants, each with an appropriate performance level in terms of BOD removal fraction. More than one set of parameters, even if individual parameters are incorrect, can give a satisfactory agreement of estimated and measured BOD and DO values for existing stream conditions. Considering the inadequate state of the art in adjusting parameters for future conditions, however, good projections are not necessarily assured.

A sensitivity analysis by computer simulation was carried out, in which each of the stream parameters entering in the BOD and DO equations was investigated within a possible range, in order to include many of the possible combinations which might occur. The resulting river water qualities, degrees of treatment, and economic effects, in combination, form a picture of how different assumptions might affect the ultimate design. The studies demonstrated that the k_3 and p values were the most insensitive, while the a value was the most sensitive.

Of particular interest was the effect of parameter selection on sewage treatment plant cost. The technique used was to hold all parameters constant at the basic design values except one, and to test that parameter at the upper and lower bounds of its range. In each case, an optimum set of treatment plants was established to achieve a 3 ppm DO deficit at all points in the river. For all testing, the investment cost varied between $30.5 million and $43.0 million, a range of 92 to 130 percent of an investment cost of $33.1 million based on the basic design values. The highest cost corresponded to an assumption that photosynthesis is not active at all, showing that for the modeled stream the algae can have a large favorable effect if the growth can be controlled to avoid a nuisance.

Similar results were obtained with an objective of 4 ppm DO deficit at all points in the river. It was found, however, that the costs of treatment appear to be less sensitive to parameter selection for lower standards of river water quality.

While cost is important in water pollution control planning, a more essential consideration is the effectiveness of treatment. If treatment plant levels are selected on the basis of assumed values of parameters, and the actual values of parameters turn out later to be unfavorable, the water quality standards would not be met. It was found that the effects on stream quality for such an unfavorable set of parameters were more serious with lower levels of treatment than with higher levels. This would indicate that if the treatment process for a plant were limited to primary treatment on the basis of computation with a mathematical model, the plant ought to be constructed to permit expansion to secondary or higher levels of treatment. Furthermore, communities which construct primary treatment plants should be advised that such plants may turn out to be inadequate.

On the basis of these studies, it would appear to be prudent to base every system design on the premise that the estimated values of parameters may be inaccurate, and possible ranges of parameters should be investigated.

The methods used to determine stream parameters should be improved. Emphasis should be placed on the development of good techniques to evaluate parameters individually and as directly as possible. Only then, can engineers make dependable adjustments to parameters in order to predict future conditions.

While steady state models for estuarine streams are often unsatisfactory, it is judged that steady state models can continue to be useful to determine approximate plans for water pollution control for many non-tidal rivers. Such models can be developed using the data from brief field surveys, and can provide comparisons of alternative plans for pollution control at reasonable cost for the engineering studies. It is clear, however, that adequate margins of safety over the results indicated by the model studies should be employed.

Time Varying Mathematical Model

For important streams with existing or potential trouble spots with respect to water quality, an approximate model of river conditions based on the steady state does not provide a means for a sufficiently adequate simulation of conditions when streamflow and environmental factors vary substantially over a period of operation. An unsteady state model can be used to investigate the effectiveness of an assumed configuration of treatment plants while a stream's assimilative capacity changes with distance and time. Also when a simulation showing the variation of quality with time is available, the suitability of the stream for fish propagation or other specific purpose may be studied in detail.

The techniques which have been developed can trace a time profile at any specified station for flow and quality in terms of BOD, DO, coliforms and chlorides while the upstream discharge, water temperature, and solar radiation are changing. With a change in programming instructions, values could be shown for all stations at any specified time.

Most previous investigators of time varying models have developed sophisticated differential equations and numerical methods for computer solutions of these equations. When "plug" flow can be assumed, as is the case for almost all non-tidal conditions, different methods can be used based upon relatively uncomplicated elaborations of a classical approach such as the oxygen sag expression employed by Streeter and Phelps.

The principal objectives in the development of the time varying mathematical model described in this report have been to make maximum use of the types of data which can typically be obtained from records and field surveys, and to produce the kinds of results that are needed to examine the effectiveness of water pollution control programs. It was found possible to accomplish these objectives by working with short reaches and/or times, over which steady state conditions could be assumed to apply without undue loss of accuracy. It was also found possible to accept input data in any available form (such as a table, when an equation cannot be created without compromising the data).

The adopted system utilizes moving stations in addition to the fixed stations of the steady state model. The moving stations are located so that the time of travel between two successive stations for any flow is an arbitrary time interval. It was found that the analysis of six intervals per day was sufficient to show the diurnal variation of dissolved oxygen due to photosynthesis.

The computations proceed in a downstream direction. A set of "distance varying parameters" are assumed to remain at these values until the next fixed river station is reached. At the beginning of each time interval which corresponds to a moving station location, one or all of the "time varying parameters" may change, and these are assumed to remain constant until the beginning of the next time interval. Many of the functions used for evaluating river conditions utilize both time varying and distance varying parameters.

Six values of flow are routed for each day. All of the computed values for the time, coliforms, BOD, DO, chlorides, and discharge remain in computer storage until a flow increment has passed all the way downstream, after which these values may be printed for as many of the fixed stations as desired. After the printing operation is complete, the procedure cycles back to the upstream end of the stream, and a new incremental flow is processed.

Stream factors influencing the oxygen balance which have been considered for the time varying model include: deoxygenation, reaeration, algae activity, benthal demand, settling and resuspension of BOD, temperature, sunlight, and streamflow.

A new empirical procedure was developed and tested for the representation of unsteady streamflow. The procedure is especially suited for studying normal and low flow periods. The only input data required are an upstream hydrograph and velocity-discharge relationships for the reaches of the stream. Unlike many flood routing procedures, this method of low flow routing is concerned with the time displacement of pollutants for different discharges rather than representing the movement of translatory waves. Much of the computer time required for processing with the time varying model is attributable to the routing procedure.

The report discusses appropriate relationships to handle variations with time of the other input data for the stream parameters. The programming methods are quite flexible, and can facilitate revisions in computational procedures to accommodate different relationships for the parameters which may be preferred by other analysts.

Features which could have been added to the model but which have not been included for these studies are: the varying of treatment plant operation with time, the inclusion of overland flow and major tributaries along the stream, changing population configurations, the direct addition of molecular oxygen to the stream, the employment of low flow augmentation, and thermal discharge effects. In general, these were not analyzed because they were not of particular importance in previous studies of the Merrimack River, on which the model was patterned. They can be added, however, without much difficulty if the variations with time and/or distance are known.

The time varying model was based on a deterministic approach which primarily considers the changes due to flow, temperature, and sunlight. As with practically any simulation vehicle, however, the model can be modified to include stochastic fluctuations in the rate parameters and other input data, or by adding random components to the output values.

<u>Simulation Studies with Time Varying Model</u>

The examples described in the report are limited to a few basic demonstrations. Due to the comprehensive nature of the time varying model and its flexibility in accepting modifications, however, many other input-output studies are possible. Although output was normally available from a computer run for every preselected ("fixed") river station within the study stream, the examples present results for only one river station, which was shown by the steady state model studies to be critical with respect to DO for the specified design period.

Although the time varying mathematical model and the input data are patterned on information for the Merrimack River in Massachusetts, it is not claimed that the results reproduce values for this stream. It is also emphasized that the conclusions are based upon only limited testing work. Nevertheless, the research has effectively studied different techniques for analyses, and the implications of using available data and design assumptions on the planning and operation of water pollution control programs. Furthermore, the conclusions conform to what may be expected from experience and judgment.

Initial time varying studies, in which typical input vs. output relationships were studied, demonstrated that by merely inspecting the input data, it may not be possible to determine which month's output will contain the lowest single DO value, the lowest average DO value, or the longest period with the DO values below some specified value.

It was not possible to obtain a natural record with a coincidence of low flow, high temperature, and little sunlight, that would occur for a long enough period of time, to cause the kinds of stream conditions indicated by the steady state analysis. In order to provide results for the time varying model which could be used for comparison with the steady state model, it was necessary to modify the input. Two months of synthetic sequential input data were constructed, one in which the averages for the worst seven day period conformed to the input values for the steady state model, and the other in which the averages for the month conformed to steady state values.

For each of these modified months, biological conditions were predicted for three different plant configurations--all primary treatment with 38% BOD removal, an optimum set of plants to satisfy an objective DO deficit of 3 ppm for the whole stream which was obtained from the steady state model, and all secondary treatment with 90% BOD removal.

At the critical station, and for the first modified month, the lowest DO for the unsteady state model for the "all secondary treatment" assumption was virtually the same as that for the steady state model, but the lowest DO's for the "optimum treatment" and for the "all primary treatment" assumptions were about 0.7 ppm less and 1.4 ppm less, respectively, than the steady state results. For the second modified month, the lowest DO's were 0.3 ppm less for "all secodary treatment," 1.0 ppm less for "optimum treatment," and 1.9 ppm less for "primary treatment." These results demonstrate that worse values than the steady state value can occur at times, and that the susceptibility and degree of such differences increases with higher BOD loads, corresponding to lower treatment plant performance levels.

The ranges of DO results are also significant. For the second modified month, values were between 1.82 ppm and 4.42 ppm for primary treatment, 4.48 ppm and 5.79 ppm for optimum treatment, and 6.28 ppm and 7.30 ppm for secondary treatment. Therefore, DO ranges for the three forms of treatment (primary, optimum, and secondary) were 2.60 ppm, 1.31 ppm, and 1.02 ppm, respectively. It would appear from these studies that a greater range of river conditions over a period of time may result when higher BOD loads are applied to the stream.

The methods used to test the sensitivity of the various model parameters in the unsteady state model were similar to that used with the steady state model. Such analyses were performed for the deoxygenation (k_1) and reaeration (k_2) constants, the amount of algae activity (a), and the water temperature. In each case, the values of the parameter being studied were altered, while the values of the other parameters remained the same.

The most apparent indication of the sensitivity analyses for k_1 and k_2 was that with lower levels of treatment (allowing greater BOD loads to enter the stream), the DO is prone to much greater variation. Thus, design values chosen for these stream parameters are very critical. Another way of looking at this is that if the planner hopes to use the stream's assimilative capacity to adjust for lower forms of treatment, it is likely that even if the average values for a 7 day or 30 day period are acceptable, worse conditions will probably occur for a portion of the time.

Although the concept has not been thoroughly investigated, the testing work suggests that there may be some merit in considering BOD concentration as a criterion for receiving waters.

As for the steady state model, the effect of the a assumption is marked and is emphasized in the time variable studies which take account of the diurnal changes in photosynthesis. It was assumed that the a values are not affected by the treatment levels chosen.

For the ranges studied, changes in temperature produced about the same average change in DO for the different levels of treatment. A $3^{\circ}C$ change in temperature for the stream produced a fairly consistent change in DO of nearly 1 ppm at the critical station.

The work of other investigators was reviewed in order to gain some insight in the matter of statistical variations. Since the model is deterministic, it does not explicitly consider either the random variations in the input or the random variations in local environmental conditions. It would appear reasonable to expect that the results for DO obtained for the time varying model represent the most probable values, while the actual values of DO would range above and below these values. A tentative judgment is that the differences between probable and actual values would be less than 1 ppm for at least 90 percent of the time. Additional studies would be needed to establish a firmer relationship.

CHAPTER I

INTRODUCTION

A. Research Objectives

The principal objective of this research project has been to study relationships between data collection programs and water pollution control plans. The ultimate aim of research such as this is to improve the efficiency of data collection and processing programs, to make them as responsive as practicable (within budget constraints) to the planning and operation requirements of water pollution control agencies.

Water pollution control agencies have programs for collecting water quality data and other physical and economic data, and for processing them in accordance with procedures that have evolved over the years. Certain data are collected more or less routinely (e.g. data on performance of sewage treatment plants) while other data are obtained to meet specific needs (e.g. sanitary surveys of river water quality and pollution discharges for purposes of river basin planning).

The agencies do not have unlimited budgets. It is, therefore, desirable for the agencies to know the relative importance of various field and laboratory data used in making studies leading to: 1) selection of required performance levels for waste treatment plants; 2) selection of objective qualities for watercourses; and 3) administrative regulations to control the operation of water pollution control programs.

In the initial proposal for this project, it was stated that information on data collection and processing programs would be obtained from the water pollution control agencies of the New England states, whose work is partially coordinated by the New England Interstate Water Pollution Control Commission. It had been hoped that the analysis of such information would indicate the feasibility of a comprehensive study to establish a scientific basis for judging the economic efficiency of data collection and processing programs. Discussions with these agencies were rather disappointing, however, in that it appeared that adequate information was not available for systematic statistical and economic analyses.

Some attempt was made to obtain useful data from other states but this was not successful. The scope of this project did not include a more extensive survey of possible sources.

In view of the above, this research has focused on methodologies for utilizing data rather than on a quantitative evaluation of the costs of data collection and processing. The vehicles for the studies have been mathematical models. Steady state and time varying simulation models have been used, both patterned on community and stream data for the Merrimack River Basin in northeastern Massachusetts.

The preceding discussion has referred to the needs of governmental agencies. To a large extent, the results of this research are also applicable to the activities of planning and design groups under private control.

B. Authority

A proposal entitled "Data Collection Economics for Water Pollution Studies" was submitted to the Federal Water Pollution Control Administration on 25 May 1966. On March 31, 1967, a notice of grant awarded for Grant Number WP-01090-01 was issued covering the two year period March 1, 1967 through February 28, 1969. Because graduate research assistance was not available until July 1967, the termination of the project was extended by the FWPCA to June 30, 1969.

C. Outline of Research Procedures

Water pollution control criteria for wastes treatment and for quality of receiving waters were reviewed. Methods for predicting river water quality were reviewed. Data used for predictive formulas and for program examination were classified.

Two studies, by the FWPCA (1)[a] and Camp Dresser and McKee (2), Consulting Engineers, have recently been made for the Merrimack River in Massachusetts. These studies were examined in order to establish possible ranges of river parameters that could be selected for planning water pollution control programs. Using a slightly modified version of a steady state mathematical model previously developed by the Principal Investigator, electronic computer studies were made which varied data within possible ranges of values and determined resulting river water qualities and economic effects.

The steady state studies indicated the need for a model to study the unsteady state effects. The development and testing of the time varying model has been the major effort of this

[a]Note: Numbers in parentheses refer to corresponding items in bibliography.

research project. Operating studies were made to compare results with those of the steady state model and to investigate the sensitivity of model parameters in terms of river water quality predictions.

This report includes a brief discussion of the data collection and processing programs of the New England regulatory agencies. As mentioned above, however, this portion of the project could not be pursued at a desirable level.

D. Needs for Better Mathematical Tools

In the following chapters, steady state and unsteady state models are discussed within a rather limited framework of assumptions and results. However, the capabilities of such models, in terms of producing better design decisions, are demonstrated.

Improved mathematical tools should serve as aids in activities such as the following:

1. Selection of treatment plant capacities to meet water quality criteria.
2. Selection of operating modes for plants to meet water quality criteria.
3. Prediction of ecological effects (e.g., fish kills) with varying plant treatment levels, flows, and climatological elements.
4. Establishing field and laboratory programs to obtain required data for initial selections of plant capacities and operating modes.
5. Establishing monitoring programs to ensure compliance with water quality requirements, and to measure precision of mathematical techniques.

It is most desirable to coordinate the individual field and laboratory studies, automatic monitoring programs, and mathematical model investigations. All of these methods are needed in the efficient operation of water pollution control programs.

E. Significant Results

It is believed that the following are the most significant conclusions of this research project:

1. While a steady state river model is a practicable tool for selecting levels of waste water treatment in a system, the levels of treatment and the consequent costs can be quite sensitive to the river parameters chosen for the analysis.

2. An unsteady state river model incorporating variations with distance and time of flow, temperature, solar radiation, and stream parameters can be developed, and can be used to produce a record of river quality at any specified location.

3. Considering the present state of the art in estimating river purification parameters, there can be no assurance that treatment plant systems will operate as planned to maintain river quality. If parameters are less favorable than assumed, susceptibility to stream conditions which do not meet quality standards increases with lower levels of waste water treatment.

CHAPTER II

SETTING FOR RESEARCH

A. Description of Goodman-Dobbins Mathematical Model

In a doctoral thesis (3) completed at New York University in 1965, a methodology was proposed by the Principal Investigator for studying the physical, economic, and administrative interrelationships of water pollution control programs. The work was later outlined in a paper co-authored with Dobbins, published by the American Society of Civil Engineers (4).

The methodology featured a steady state mathematical model for a stretch of river where bordering populations and industries use the flowing water for municipal water supply, disposal of treated sewage, and recreation. A computer program was used with simulation techniques to obtain numerical values for the characteristics of importance in the planning, design and operation of water pollution control programs. The computer program was comprised of three essential components: 1) statements to control data processing; 2) equations for a "community and river model," and 3) statements for an "optimizing routine." Subprograms were included for processing "functions" for repetitive calculations, such as for the oxygen sag curve and for the cost estimates.

Input data for the model included hydraulic constants and stream purification rate constants for each reach of the river, population and other economic information for each community, and assumptions with respect to administrative decisions. If the levels of sewage treatment were specified for each community, the program evaluated the water quality throughout the length of the stream, and the costs and benefits of the water pollution control program. If a single water quality criterion for the stream was specified for each river section (e.g. in terms of dissolved oxygen), an optimizing routine was used to determine desirable levels of sewage treatment. The work described in detail in the aforementioned publications was based upon operating a model designated "CARM-1" (for "community and river model number 1").

The steady state studies which are discussed in the next chapter were made with essentially the same model CARM-1. An improved time varying river model, whose development, testing, and operation are discussed in Chapters IV and V, is patterned upon the same stream in Massachusetts as was used for the steady state model.

Although hypothetical, these models are based to a large extent upon the geographical and population configurations and other river and community data for the Merrimack River Basin in northeastern Massachusetts. The Merrimack River has been studied in recent years by Massachusetts governmental agencies (5); the U. S. Public Health Service (6); Camp Dresser, & McKee (2,7), Consulting Engineers, Boston; and the Federal Water Pollution Control Administration (1). It may be stated without substantial qualification, that the models have been developed with realistic data such as those obtained with an actual river basin.

The area serving as the basis for these models is shown on Figure 1. The distance covered by the stream is 50.9 miles. There are nine communities with a total area of 302 square miles, and a total population of 579,000 persons. Each community is shown with locations of water supply intake facilities (designated I) and sewage discharge facilities (designated D).

Table 1 contains information for each community with respect to area, population, industry, type of sewage system, treatment and disposal, and water supply. The steady state model CARM-1 also included locations for recreational facilities. The studies with the time varying model have not included all aspects of water supply and recreation potential. Community "THREE" does not front on the river but instead discharges its treated sewage to the river via a tributary. This tributary is assumed to have an insignificant flow in comparison to the larger stream. Therefore, the mouth of this tributary can be considered to be the sewage outfall for this community.

The time setting for this study is assumed to be the year 2000 or later when urban pressures are expected to intensify the need to exploit many rivers for water supply, recreation, and wastewater disposal. The use of this projected time with its estimated population values also permits more interesting model studies, because the current populations do not exert as much stress on what is a fairly large stream.

The time varying model discussed in this report is functional for the warm months of the year (May-October). To extend its use requires only the addition of more detailed data for colder and higher flow periods. Generally, the warm months contain the periods of low-flow and high temperature, with associated high water supply demands. Such conditions provide the setting for waste water treatment design and for critical examination of river water quality.

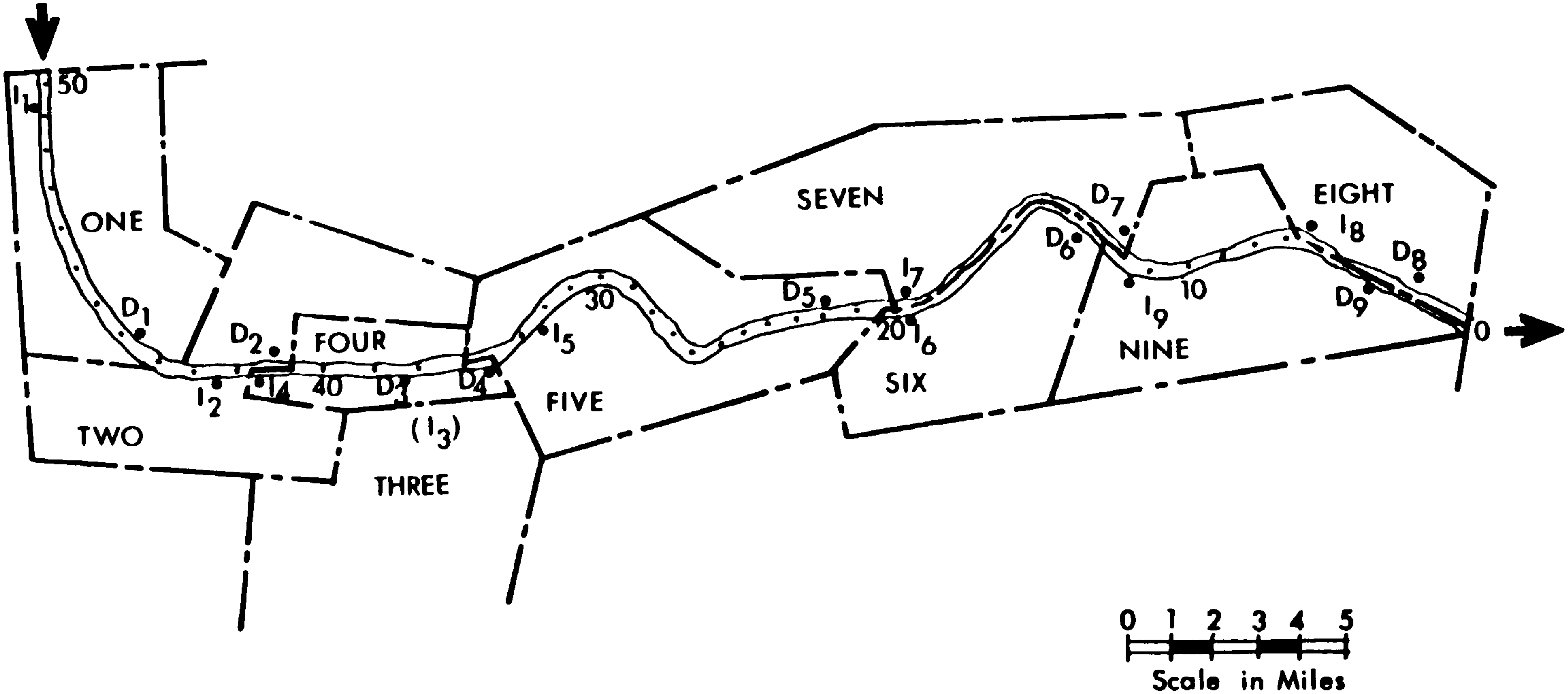


Figure 1. Sketch of Study Stream Showing Locations of Sewage Discharges and Water Supply Intakes

TABLE 1

General Information for Communities for River Models

Community Name	J	Area (sq.mi.)	Population	Employees in Community	Water Supply	Sewage Disposal	Sewage System	% Homes Served by Public Sewers	% BOD Removal by Treatment Plant Set 1	Set 2	Set 3
ONE	1	30.8	14,000	0	On-river	On-river	Sanitary	100	38	38	90
TWO	2	43.4	67,000	5,650	On-river	On-river	Sanitary	100	38	80	90
THREE	3	25.5	48,000	2,750	On-river	On-river	Sanitary	100	38	80	90
FOUR	4	13.4	94,000	29,500	On-river	On-river	Sanitary	100	38	38	90
FIVE	5	39.9	142,000	55,000	On-river	On-river	Sanitary	100	38	65	90
SIX	6	22.4	45,000	41,650	On-river	On-river	Sanitary	100	38	38	90
SEVEN	7	51.7	70,000	33,000	On-river	On-river	Sanitary	100	38	38	90
EIGHT	8	36.4	49,000	4,300	On-river	On-river	Sanitary	100	38	38	90
NINE	9	38.5	50,000	9,550	On-river	On-river	Sanitary	100	38	38	90
		302.0	579,000	144,400							

B. Optimization Routine

In the next chapters of the report, references are made to an "optimum" set of plants, required in order to meet specified water quality criteria. In each case, the optimum set is derived for the steady state model, using an optimizing routine based on the concept of "path of steepest ascent." This is an operations research procedure which makes changes leading to a specified set of objectives by means of a series of steps, each step being the most efficient that can be taken. Some studies using this method have been made for hydraulic and sanitary engineering problems (8,9).

The optimizing routine makes successive improvements to the quality of water (in terms of DO deficit) at one river station at a time. The objective quality is reached for the most upstream station for which a quality is specified, before proceeding to adjust the quality at the next station downstream with a specification.

The water quality at a river station can be adjusted by raising the level of treatment at one or more upstream sewage treatment plants. Only substantial increments of treatment are practical from the standpoint of engineering economy. Up to nine levels of treatment may be considered with the routine, ranging from a minimum of comminution and chlorination (with 10% BOD removal) to a maximum of tertiary treatment (with 99% BOD removal).

Starting with an initial state of performance levels for the treatment plants, one increment of level is examined for each plant and the most efficient of these increments is selected. Proceeding from the new state of performance levels, one increment is again examined for each plant and the most efficient selected. The process is repeated until the objective quality specified for the river station is reached. The routine then goes on to make adjustments for the next station.

The "efficiency" of an increment (increase in level of treatment) requires a definition by the planner. The term "effectiveness" is preferred as being more descriptive of the intent of an increment. Three alternative measures of effectiveness are accepted by the routine and may be specified by the planner as part of the input data. For all of the work done for this report, the effectiveness was defined in terms of least cost of sewage treatment. Thus, the optimization process examined a ratio of the reduction of DO deficit at a station to increase in annual cost of sewage treatment.

In the discussion of the Goodman-Dobbins model presented in the ASCE paper, several reviewers criticized the method of optimization on the grounds that it does not guarantee a global minimum. As pointed out in the closure discussion (10), the difference is small between the results of the method based on path of steepest ascent and a dynamic programming method developed by Liebman and Lynn (11). A linear programming method has been proposed for this problem by Revelle, Louks and Lynn (12). And a mixed integer programming method is now being developed by Camp, Dresser and McKee for a study of the Merrimack River, on which the principal author is a consultant.

The path of steepest ascent has been retained for the optimizing procedures used in this report. The authors are confident that changes in the interpretation of results would be negligible if another slightly more precise method were used for optimizing.

CHAPTER III

STUDIES WITH STEADY STATE MODEL

A. Introduction

A brief description of the mathematical model by Goodman, patterned after the Merrimack River in Massachusetts, was given in Chapter II. The sensitivity analyses discussed in this chapter are based on the assumption of the steady state. Stream purification parameters have been estimated for this stream and reported on by Camp, Dresser and McKee in December 1963 (2) and by the staff of the Merrimack River Project, Northeast Region of the Federal Water Pollution Administration in August 1966 (1). For convenience, these study groups will be referred to as "CDM" and "FWPCA." The CDM work is also the subject of a paper by Camp (13), published in October 1965.

The principal reasons for using the Goodman model and the CDM and FWPCA surveys as the basis of the steady state studies are as follows: 1) The Merrimack River, while larger than most, is typical of the New England rivers being studied for pollution abatement; 2) The river has been the object of several extensive surveys which have developed considerable data; 3) Only minor modifications to the Goodman model are needed for the studies.

Both surveys, like most stream investigations, have used the dissolved oxygen concentration as the principal criterion of stream quality. Equations determining changes in BOD and DO are by Streeter and Phelps (14), as modified by Camp (15). These formulas require values of rate parameters for deoxygenation, reaeration, settling out of BOD to bottom deposits, resuspension of BOD from bottom deposits, and oxygenation by photosynthesis of algae.

Various approaches may be taken for a sensitivity analysis of the rate parameters. For the 50 mile stream with 9 sewage treatment plants, the focus was on the effects of parameter selection on the cost of sewage treatment necessary to maintain a specified DO throughout the stream length. Thus, each stream parameter chosen as a basis for design affects the planning of a system of treatment plants, each with an appropriate performance level in terms of BOD removal fraction.

The values of parameters proposed by CDM and by the FWPCA were examined. In addition, a range of values of each of the parameters was considered. All of these values were proposed as estimates by responsible engineers. Nevertheless, all investigators in this field work under the restrictions of an inadequate state of knowledge concerning field and laboratory procedures and the interpretation of data.

$$L_b = \left[L_a - \frac{p}{2.3(K_1+K_3)}\right] 10^{-(K_1+K_3)t} + \frac{p}{2.3(K_1+K_3)}$$

in which L_a and L_b are BOD's (ppm)
t is the time of flow (days)
K_1 is the deoxygenation rate constant (days^{-1})
K_3 is the rate constant for settling out of BOD to bottom deposits (days^{-1})
p is the rate of addition of BOD to the overlying water from bottom deposits (ppm per day)

The oxygen sag curve is in terms of the oxygen saturation deficit, the difference between the saturation concentration and actual value of DO. The change in deficit between stations a and b is computed by the following formula:

$$D_b = \frac{K_1}{K_2-K_1-K_3}\left[L_a - \frac{p}{2.3(K_1+K_3)}\right]\left[10^{-(K_1+K_3)t} - 10^{-K_2t}\right] - \frac{K_1}{K_2}\left[\frac{p}{2.3(K_1+K_3)} - \frac{a}{2.3K_1}\right](1-10^{-K_2t}) + D_a 10^{-K_2t}$$

in which D_a and D_b are DO deficits (ppm)
K_2 is the atmospheric reaeration rate constant (days^{-1})
a is the rate of production of DO by algae through photosynthesis (ppm per day)

All BOD's obtained in the CDM and FWPCA surveys and utilized in these studies are carbonaceous demands based on

5 day measurements. Current studies on the Merrimack by CDM, with which the Principal Investigator is familiar, also consider the nitrogenous demand. If the nitrogenous demands were included in the model studies, individual values of results would be expected to change somewhat, but it is judged that no concepts or significant conclusions would be modified.

C. Values of Parameters Selected in Surveys

For purposes of the computer work, it was convenient to designate each parameter as a "Z" value. Thus Z(3) is k_1, Z(4) is k_2, Z(5) is k_3, Z(6) is p, and Z(7) is a. The 50 mile length of study river was divided into 62 river stations at unequal distances. Values of parameters for existing conditions are shown in Table 2, based on the CDM report, and in Table 3, based on the FWPCA report. Values are projected for the year 2010 by CDM as shown in Table 4, and for the year 1985 by FWPCA in Table 5. Goodman adopted the values shown in Table 6 for use in previous mathematical modeling studies.

Comparing Tables 2 and 3, it will be noted that there are large differences between values of the parameters determined by CDM and FWPCA. The values in Tables 4 and 5 for future conditions are also different. Despite this, design recommendations made by CDM and FWPCA based on these parameters were similar.

More than one set of parameters, even if individual parameters are incorrect, can give the same overall results. This is true when reproducing actual oxygen sag cruves, and also if approximately parallel adjustments are applied for forecasting future conditions. Considering the inadequate state of the art in adjusting parameters for future conditions, however, good projections are not necessarily assured.

D. Techniques for Evaluation of Stream Parameters

For completeness, the procedures used by each study group for estimating values of stream parameters for conditions during the field surveys will be outlined. The CDM studies, based on surveys in the summers of 1962 and 1963, will be described first.

The k_1 value was obtained from measurements of oxygen uptake in BOD bottle tests in the laboratory at constant temperature. Camp suggests that a better procedure would have been to submerge the bottles in the river water at the locations of the samples so that the sample temperatures would vary with the stream temperature. The p value was determined by measuring the oxygen demand of samples of bottom deposits; the demand in

TABLE 2

River Parameters for Existing Conditions
(Camp, Dresser and McKee Report 1963)

River Station	Z(3)-(k_1)	Z(4)-(k_2)	Z(5)-(k_3)	Z(6)-(p)	Z(7)-(a)
1 27	0.06	0.04	0.11	1.0	0.5
28 34	0.06	0.07	0.13	1.0	1.0
35 62	0.07	0.15	0.05	0.5	0.5

TABLE 3

River Parameters for Existing Conditions
(Federal Water Pollution Control Administration August 1964 & August 1965)

River Station		Z(3)-k_1)	Z(4)-k_2)	Z(5)-(k_3)	Z(6)-(p)	Z(7)-(a)
1–6	1964 (1125 cfs)	0.130	0.210	0.140	0.50	1.70
	1965 (770 cfs)	0.095	0.230	0.040	1.0	2.0
19–30		0.161	0.160	0.010	0.50	0.80
31–40		0.175	0.220	0.010	0.20	1.0
41–58		0.175	0.140	0.00	0.90	1.70

TABLE 4

River Parameters for Year 2010
(Camp, Dresser and McKee Report)

River Station	Z(3)-(k_1)	Z(4)-(k_2)	Z(5)-(k_3)	Z(6)-(p)	Z(7)-(<u>a</u>)
1 27	0.05	0.04	0.07	0.5	0.3
28 34	0.05	0.07	0.08	0.5	0.5
35 62	0.05	0.15	0.05	0.3	0.3

TABLE 5

River Parameters for 1985
(Federal Water Pollution Control Administration Report)

River Station	Z(3)-(k_1)	Z(4)-(k_2)	Z(5)-(k_3)	Z(6)-(p)	Z(7)-($\underline{a}$)
1 19	0.080	0.170	0.010	0.30	0.20
20 30	0.080	0.170	0.010	0.30	0.20
31 41	0.100	0.230	0.010	0.10	0.40
42 58	0.100	0.150	0.010	0.50	0.10

TABLE 6

River Parameters
(Adopted for CARM 1)

River Station	Z(3)-(k_1)	Z(4)-(k_2)	Z(5)-(k_3)	Z(6)-(p)	Z(7)-($\underline{a}$)
1 8	.05	.03	.07	.50	.30
9 27	.05	.04	.07	.50	.30
28 34	.05	.06	.08	.50	.40
35 62*	.05	.08	.08	.50	.50

* Note that in this section (Haverhill to the Mouth) the Merrimack is an estuary and because the original model was not designed to cope with this, these are ficticious values.

gms per square meter was divided by the hydraulic depth in meters to obtain the parameter in the proper units, where the hydraulic depth is defined as the total volume in a reach divided by the water surface area. L_a and L_b were determined from BOD bottle tests for samples collected at stations a and b. With the value of k_1, p, L_a and L_b thus obtained and the travel time estimated, the value of k_3 was determined from the BOD equation given above. The method of determining the travel time was not discussed in the CDM report.

The value of a was evaluated by the light- and dark-bottle technique, with the bottles suspended at the points where the samples were collected; values of a for various locations were averaged for the reach in accordance with the hydraulic depth calculations. The k_2 value can then be estimated from the oxygen sag equation, if the oxygen deficits at the ends of the reach are measured and all of the other parameters are as found above. Instead, and the CDM report does not indicate why this was done, it was assumed that at a point in the receiving waters, both D and L are constant ("preferably averaged over a period of several days") and the rate of supply of oxygen is equal to the rate of deoxygenation, as:

$$2.3\ k_2 D + \underline{a} = 2,3\ k_1\ L$$

and the value of k_2 may be determined if the other parameters are known.

As indicated in the FWPCA report, it was decided to use the basic approach of CDM but "with more emphasis on certain of the variables considered to be weak. In addition, gaps in water quality information, such as the biological condition of the river, were to be filled." The field work was done in the summers of 1964 and 1965.

The a value was determined by the light- and dark-bottle technique. Data were plotted as oxygen production per day versus depth to obtain a parabolic curve extending from a maximum effect in the first foot of depth to zero effect at 8 feet depth. The value of a was obtained by dividing the area of the curve by the hydraulic depth. Studies were also made to relate measured sunlight intensity to the oxygen production.

Values of p were determined in several ways. The first method was based upon running bottle tests and Warburg analyses on samples taken from the benthic deposits. The second method was based upon pilot plant studies of bottom sediments. The FWPCA report states that a representative value of p was selected for each reach based upon the two methods and that the "selection was influenced by field observations of the area, and the relationship of p with the observed oxygen sag equations." Comparisons of values of p are not available in the report.

Time of travel was determined by the use of the Rhodamine B dye and fluorometer technique. Values of k_1 were available from laboratory bottle tests. With known values of L_a, L_b, k_1, t, and p, the k_3 was obtained from the BOD equation. The k_2 was found from the oxygen sag equation with the known values of D_a, D_b, k_1, k_3, p, a and t.

The k_3 obtained as described above was generally negative or of very low positive value. The report states that "considering the low dissolved oxygen levels and physical characteristics of the Merrimack River, such k_2 results were not considered representative." Of the various key parameters, it was felt that the a and p were acceptable but that the values of k_1 and k_3 should be adjusted to give reasonable values of k_2. Bottle k_1 values were used to compute L values. By plotting L vs time of flow, a combined $(k_1 + k_3)$ term was calculated. Values of k_1, k_3 and k_2 were then determined that would result in duplicating observed field conditions for the oxygen sag curve, on the assumption that the "bottle k_1" was not representative of the "river k_1." It was made clear that various values of k_1, k_2 and k_3 would result in satisfactory matching of the sag curve.

There is no basis at this time for judging which study's values best represent the present and anticipated stream conditions. The studies were made in different years and projected conditions for different periods. It may be questioned, however, why two surveys using similar sampling techniques would obtain values of parameters over ten times different.

It may be reasoned that perhaps the differences are attributable to the data collection systems. These studies relied on standard bottle tests. Those parameters which could not be evaluated directly were determined by substituting into the BOD and DO equations those values which could be determined directly. Bottle tests do not simulate the current of the stream and the benthal material, in producing a "trickling filter" action. Furthermore, these values of parameters are so dependent on the experimental values that a reasonable degree of accuracy may not be obtained. For example, k_3 (constant for the settling out of BOD to bottom deposits) depends on the correct evaluation of k_1 (deoxygenation constant), p (addition of BOD to the stream from bottom deposits) and

L_a and L_b (the BOD at upstream and downstream locations).

The separation of the parameters affecting dissolved oxygen into a number of components is conceptually satisfying, but implies a degree of precision in the estimate of the parameters which does not currently exist. Due to the way in which these parameters are evaluated, no way exists of checking their accuracy. Although it has been impossible to make an accurate diagnosis of the degree of error, the magnitudes of the differences between the results of the two studies would indicate that it might have been large.

The search for improved methods of laboratory testing is documented by many reports in the literature. Much less has been written on the improvement of field measuring techniques. In a report dated April 1967, the FWPCA described experiments with more sophisticated sampling devices (16). Using the Blue River in Oklahoma, they attempted to better separate the stream parameters by evaluating them individually. It was shown earlier in this chapter that light and dark bottle tests employed in the two river surveys of the Merrimack River can only include the effects of suspended plant material. In the Oklahoma study, not only was an attempt made to include the benthos, but a system of paddles was devised to simulate the current as well. Methods of this type should provide better estimates of k_1.

In view of the difficulties of obtaining good field measurements of stream parameters, continued research to improve field sampling is needed.

E. Sensitivity Analyses by Computer Simulation

With the availability of a new advanced electronic computer facility recently installed at Northeastern University and methodologies based on those previously developed by the Principal Investigator, a large number of runs were made for a sensitivity analysis. The procedure was to multiply each of the river parameters (Table 6) selected for design, by coefficients to individually vary them within possible ranges. These ranges were taken from the extremes of the values measured in each survey studied. The resulting river water qualities, degrees of treatment, and economic effects were graphed and charted to form a picture of how different assumptions might affect the ultimate design.

These studies have shown that for the Merrimack River the constants for the settling out and resuspension of BOD are the most insensitive, while the rate for the production of oxygen by photosynthesis of algae is the most sensitive. With primary waste treatment for all sewage discharging communities, DO deficit ranges for a particular station were observed to vary from 1 ppm to over 6 ppm while the parameter for the oxygen production by algae varied from approximately 1.5 ppm/day to 0.0 ppm/day. These values would correspond to an assumption of sunlight for the former and darkness for the latter. These conclusions are in agreement with those determined by the FWPCA, who applied a type of sensitivity analysis to a single oxygen sag curve. The CDM and FWPCA studies both decided to employ values for design corresponding to a cloudy day.

The results were even more impressive when the data were processed by passing values, assumptions, and criteria through a computer routine which derives an optimum configuration of treatment plants. As discussed in Chapter II, this selection is based on a required DO minimum in the river being met by one or more up-stream systems of treatment plants each having incremental performance levels giving the most benefits downstream for the invested capital. These results are best explained by Tables 7 through 10. Again, the parameter associated with photosynthesis of algae produces the most significant changes in the results. For a condition of darkness, secondary treatment in all plants was not sufficient to keep the DO deficit below 3 ppm, while with a condition associated with much sunlight, primary treatment in all plants was more than sufficient, as the DO deficit remained substantially below 2 ppm throughout the study area. The following paragraphs describe the results in greater detail.

Table 7 presents the results of a sensitivity analysis in terms of the effect of parameter on sewage treatment plant cost. The technique used was to hold all parameters constant at the design values of Table 6 except one, and to test the remaining parameter at the upper and lower bounds of its range. In each case, a set of treatment plants is established by the optimizing routine, with allowable levels of treatment between 38% and 90% BOD removal, to achieve a 3 ppm DO deficit at all points in the river. The investment cost for sewage treatment plants based upon the design values of Table 6 was estimated at $33.1 million. For all testing, the investment cost varied between $30.5 million and $43.0 million, a range of 92 to 130 percent of the design investment cost of $33.1 million. The highest cost corresponds to an assumption that photosynthesis is not active at all; this shows that for the Merrimack River, the activity of algae has a large favorable economic effect provided that the growth can be controlled to avoid a nuisance.

With respect to individual parameters, the following

TABLE 7

Sensitivity Analysis
(using optimizing routine to determine treatment plant removal fraction and location)

Each river parameter (Z value) is varied through given range, while holding constant the other river parameter values originally adopted for CARM-1
Minimum allowable treatment = 38% BOD removal
Maximum allowable treatment = 90% BOD removal
Objective DO deficit = 3 ppm

		Community - Percent Treatment									Investment Cost of Sewage Treatment
Parameter	Assumed Values	1	2	3	4	5	6	7	8	9	
Z_3	0.01	38	38	38	38	38	38	38	38	38	30,512,828
	0.05	38	80	80	38	65	38	38	38	38	33,074,040
	0.10	80	90	85	80	80	65	65	38	38	39,536,362
Z_4	0.010-0.027	80	80	80	80	80	65	38	38	38	36,473,606
	0.030-0.080	38	80	80	38	65	38	38	38	38	33,074,040
	0.10-0.267	38	38	38	38	38	38	38	38	38	30,512,828
Z_5	0.014-0.016	65	80	80	65	80	65	38	38	38	35,624,589
	0.070-0.080	38	80	80	38	65	38	38	38	38	33,074,040
	0.140-0.160	38	65	65	38	65	38	38	38	38	32,365,717
Z_6	0.00	38	80	80	38	65	38	38	38	38	33,074,040
	0.50	38	80	80	38	65	38	38	38	38	33,074,040
	1.50	65	80	80	80	80	38	38	38	38	36,004,233
Z_7	0.00	80	90	90	90	80	65	38	38	38	43,012,646
	0.30-0.50	38	80	80	38	65	38	38	38	38	33,074,040
	1.0 -1.667	38	38	38	38	38	38	38	38	38	30,512,828

table shows the results obtained:

Range of Investment Costs

Parameter	Million $	% of $33.1 Million	% Difference
k_1	30.5-39.5	92-120	8-20
k_2	30.5-36.5	92-110	8-10
k_3	32.4-35.6	98-108	2-8
p	33.1-36.0	100-109	0-9
a	30.5-43.0	92-130	8-30

Table 8 shows the maximum DO deficit upstream and downstream of each of the sewage treatment plants, for two conditions - all primary treatment and with the optimum set of plants from Table 7 (selected in each case based upon the assumed parameters). For the optimum sets, this table merely confirms that each set will result in meeting the criterion of 3 ppm maximum DO deficit at all points on the stream. For the set of primary plants, the maximum DO deficit with the design parameters is 5.0 ppm while the DO deficit can go to as much as 7.9 ppm if the parameter turns out to be less favorable than assumed. Figures 2 and 3 show the effects of varying the photosynthesis parameter, for all primary treatment, and all secondary treatment.

The tables and exhibits show that lower levels of treatment, which correspond to discharge of higher BOD loads to the stream, may result in stream conditions which are quite sensitive to the stream parameters that actually obtain.

Table 9 shows results similar to those on Table 7, except that the maximum DO deficit specified is 4 ppm rather than 3 ppm. For 4 ppm, optimum configurations of plants range in investment cost from $30.5 million to $35.2 million, compared with $32.4 for design values of parameters. For 3 ppm, the costs were $30.5, $43.0, and $33.1 million respectively. This might indicate that costs are less sensitive to parameter selection, when standards of quality are lower. Table 10 is similar to Table 8, except for using a criterion of 4 ppm DO deficit.

F. Discussion

The sensitivity analyses have been made in terms of the

TABLE 8

DO Deficits with Primary Treatment and with Optimum Treatment*

Minimum allowable treatment = 38% BOD removal
Maximum allowable treatment = 90% BOD removal
Objective DO deficit = 3 ppm

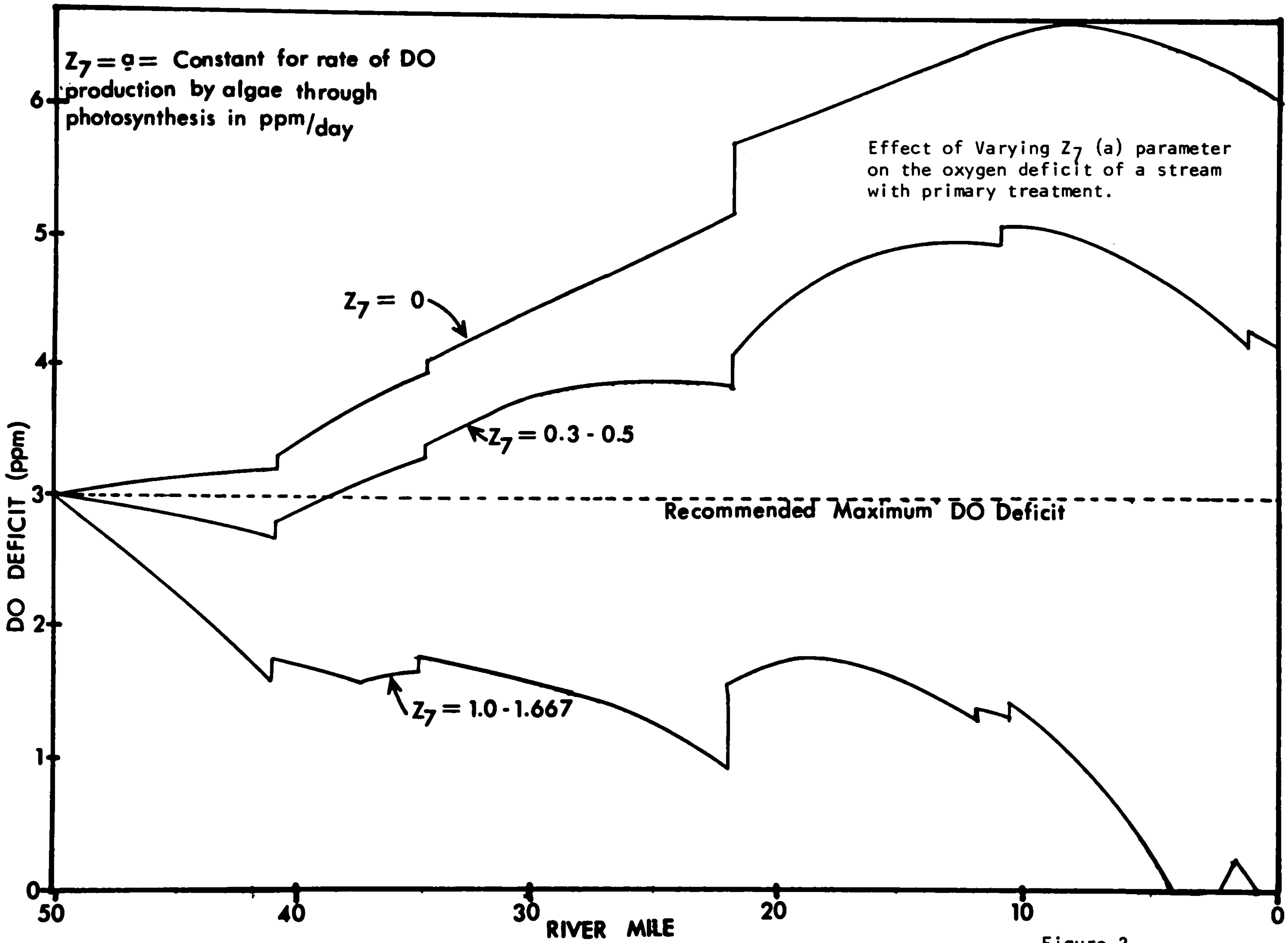
		Maximum DO Deficit (ppm) for Stations									
Parameter	Assumed Values	1-6	7-16	17-23	24-26	27-32	33-43	44-47	48-57	58-59	60-62
z_3	0.01	3.0	2.5	2.4	2.3	2.4	2.1	1.5	1.6	0.6	0.4
	0.05	3.0	2.8	3.0	3.2	3.8	4.8	4.9	5.0	4.1	3.9
		3.0	2.8	2.7	2.6	2.9	2.8	2.8	3.0	2.5	2.4
	0.10	3.1	3.2	3.8	4.3	5.8	7.9	7.9	7.9	6.7	6.3
		3.1	3.1	3.1	3.1	3.1	2.9	2.8	2.8	2.2	2.4
z_4	0.010-0.027	3.0	3.0	3.3	3.6	4.8	6.6	6.8	7.3	7.1	7.1
		3.0	2.9	2.9	2.9	2.9	2.8	2.7	3.0	2.8	2.9
	0.030-0.080	3.0	2.8	3.0	3.2	3.8	4.8	4.9	5.0	4.1	3.9
		3.0	2.8	2.7	2.6	2.9	2.8	2.8	3.0	2.5	2.4
	0.10-0.267	3.0	2.4	2.2	2.2	2.4	2.5	2.2	2.2	1.1	1.0
z_5	0.014-0.016	3.0	2.8	3.1	3.4	4.4	6.1	6.2	6.7	6.6	6.6
		3.0	2.8	2.7	2.7	2.7	2.7	2.6	2.9	2.8	3.0
	0.070-0.080	3.0	2.8	3.0	3.2	3.8	4.8	4.9	5.0	4.1	3.9
		3.0	2.8	2.7	2.6	2.9	2.8	2.8	3.0	2.5	2.4
	0.140-0.160	3.0	2.8	2.9	3.1	3.4	3.9	3.8	3.9	2.5	2.2
		3.0	2.8	2.7	2.7	2.9	2.6	2.4	2.5	1.5	1.6
z_6	0.00	3.0	2.8	2.9	3.1	3.5	4.4	4.4	4.5	3.5	3.2
		3.0	2.8	2.6	2.5	2.7	2.5	2.4	2.6	1.8	1.8
	0.50	3.0	2.8	3.0	3.2	3.8	4.8	4.9	5.0	4.1	3.9
		3.0	2.8	2.7	2.6	2.9	2.8	2.8	3.0	2.5	2.4
	1.50	3.0	2.9	3.2	3.5	4.4	5.7	5.8	5.9	5.4	5.2
		3.0	2.9	2.9	2.9	2.8	2.8	2.8	3.0	2.8	2.9

(concluded on next page)

Table 8--concluded

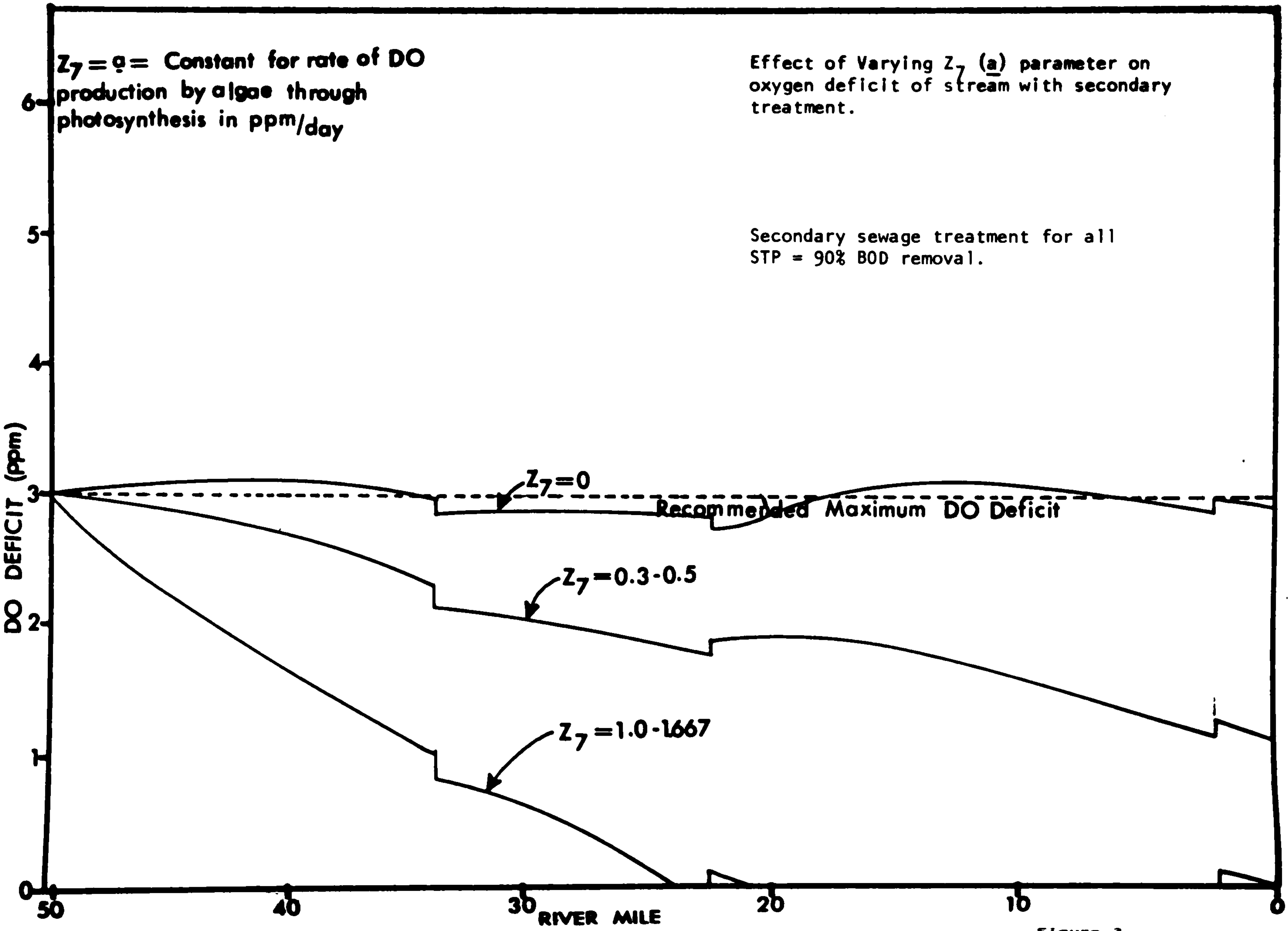
Parameter	Assumed Values	1-6	7-16	17-23	24-26	27-32	33-43	44-47	48-57	58-59	60-62
Z_7	0.0	3.1	3.2	3.6	3.9	5.0	6.3	6.4	6.5	6.5	5.9
		3.1	3.1	3.1	3.1	3.0	2.9	2.9	2.9	3.0	3.0
	0.30-0.50	3.0	2.8	3.0	3.2	3.8	4.8	4.9	5.0	4.1	3.9
		3.0	2.8	2.7	2.6	2.9	2.8	2.8	3.0	2.5	2.4
	1.00-1.67	3.0	2.0	1.8	1.7	1.0	1.8	1.4	1.4	0.1	0.2

* When two rows are shown, first row is for primary treatment and second row is for optimum treatment.



$Z_7 = a =$ Constant for rate of DO production by algae through photosynthesis in ppm/day
Effect of Varying Z_7 (a) parameter on the oxygen deficit of a stream with primary treatment.
$Z_7 = 0$
$Z_7 = 0.3 - 0.5$
$Z_7 = 1.0 - 1.667$
Recommended Maximum DO Deficit
DO DEFICIT (ppm)
RIVER MILE
0
1
2
3
4
5
6
50
40
30
20
10
0

Figure 2.



Z7 = a = Constant for rate of DO production by algae through photosynthesis in ppm/day
Effect of Varying Z7 (a) parameter on oxygen deficit of stream with secondary treatment.
Secondary sewage treatment for all STP = 90% BOD removal.
Z7=0
Recommended Maximum DO Deficit
Z7=0.3-0.5
Z7=1.0-1.667
DO DEFICIT (ppm)
RIVER MILE
0
1
2
3
4
5
6
50
40
30
20
10
0

Figure 3.

TABLE 9

Sensitivity Analysis
(using optimizing routine to determine treatment plant removal fraction and location)

Each river parameter (Z value) is varied through given range, while holding constant the other river parameter values originally adopted for CARM-1
Minimum allowable treatment = 38% BOD removal
Maximum allowable treatment = 90% BOD removal
Objective DO deficit = 4 ppm

Parameter	Assumed Values	Community - Percent Treatment									Investment Cost of Sewage Treatment
		1	2	3	4	5	6	7	8	9	
Z_3	0.01	38	38	38	38	38	38	38	38	38	30,512,828
	0.05	38	65	65	38	65	38	38	38	38	32,365,717
	0.10	38	80	80	65	80	38	38	38	38	35,173,474
Z_4	0.010-0.027	38	80	80	65	80	38	38	38	38	35,173,474
	0.030-0.080	38	65	65	38	65	38	38	38	38	32,365,717
	0.01-0.267	38	38	38	38	38	38	38	38	38	30,512,828
Z_5	0.014-0.016	38	80	80	65	65	38	38	38	38	33,692,019
	0.070-0.080	38	65	65	38	65	38	38	38	38	32,365,717
	0.14-0.16	38	38	38	38	38	38	38	38	38	30,512,828
Z_6	0.000	38	65	65	38	38	38	38	38	38	31,112,178
	0.50	38	65	65	38	65	38	38	38	38	32,365,717
	1.50	38	80	80	38	65	38	38	38	38	33,074,040
Z_7	0.00	38	80	80	65	80	38	38	38	38	35,173,474
	0.30-0.50	38	65	65	38	65	38	38	38	38	32,365,717
	1.0 -1.667	38	38	38	38	38	38	38	38	38	30,512,828

TABLE 10

DO Deficits with Primary Treatment and with Optimum Treatment*

Minimum allowable treatment = 38% BOD removal
Maximum allowable treatment = 90% BOD removal
Objective DO deficit = 4 ppm

Parameter	Assumed Values	Maximum DO Deficit (ppm) for Stations 1-6	7-16	17-23	24-26	27-32	33-43	44-47	48-57	58-59	60-62
Z_3	0.01	3.0	2.5	2.4	2.3	2.4	2.1	1.5	1.6	0.6	0.4
	0.05	3.0	2.8	3.0	3.2	3.8	4.8	4.9	5.0	4.1	3.9
		3.0	2.8	2.7	2.8	3.1	3.1	3.2	3.3	2.7	2.8
	0.10	3.1	3.2	3.8	4.3	5.8	7.9	7.9	7.9	6.7	6.3
		3.1	3.2	3.3	3.4	3.6	3.6	3.5	3.7	3.2	3.3
Z_4	0.010-0.027	3.0	3.0	3.3	3.6	4.8	6.6	6.8	7.3	7.1	7.1
		3.0	3.0	3.0	3.0	3.1	3.1	3.1	3.4	3.3	3.4
	0.030-0.080	3.0	2.8	3.0	3.2	3.8	4.8	4.9	5.0	4.1	3.9
		3.0	2.8	2.7	2.8	3.1	3.1	3.2	3.3	2.7	2.8
	0.10-0.267	3.0	2.4	2.2	2.2	2.4	2.5	2.2	2.2	1.1	1.0
Z_5	0.014-0.016	3.0	2.8	3.1	3.4	4.4	6.1	6.2	6.7	6.6	6.6
		3.0	2.8	2.8	2.8	2.8	3.2	3.3	3.8	3.8	3.9
	0.070-0.080	3.0	2.8	3.0	3.2	3.8	4.8	4.9	5.0	4.1	3.9
		3.0	2.8	2.7	2.8	3.1	3.1	3.2	3.3	2.7	2.8
	0.140-0160	3.0	2.8	2.9	3.1	3.4	3.9	3.8	3.9	2.5	2.2
Z_6	0.00	3.0	2.8	2.9	3.1	3.5	4.4	4.4	4.5	3.5	3.2
		3.0	2.8	2.6	2.1	2.9	3.8	3.8	4.0	3.1	3.2
	0.50	3.0	2.8	3.0	3.2	3.8	4.8	4.9	5.0	4.1	3.9
		3.0	2.8	2.7	2.8	3.1	3.1	3.2	3.3	2.7	2.8
	1.50	3.0	2.9	3.2	3.5	4.4	5.7	5.8	5.9	5.4	5.2
		3.0	2.9	2.9	2.9	3.4	3.6	3.7	3.9	3.7	3.8

(concluded on next page)

Table 10--<u>concluded</u>

Parameter	Assumed Values	1-6	7-16	17-23	24-26	27-32	33-43	44-47	48-57	58-59	60-62
Z_7	0.0	3.1	3.2	3.6	3.9	5.0	6.3	6.4	6.5	6.1	5.9
		3.1	3.2	3.2	3.3	3.5	3.5	3.6	3.8	3.7	3.8
	0.30-0.50	3.0	2.8	3.0	3.2	3.8	4.8	4.9	5.0	4.1	3.9
		3.0	2.8	2.7	2.8	3.1	3.1	3.2	3.3	2.7	2.8
	0.0-1.667	3.0	2.0	1.8	1.7	1.9	1.8	1.4	1.4	0.1	0.2

* When two rows are shown, first row is for primary treatment and second row is for optimum treatment.

effect of parameter on design decision. In each case, the river quality criteria were specified, and it was assumed that the designer would have to provide the appropriate levels of treatment in order to meet this standard.

The sensitivity analyses showed that the selection of parameters can greatly influence the cost of treatment, if the plants are permitted to vary from primary treatment to secondary treatment. For a model based on the Merrimack River in Massachusetts, the investment cost ranged from $30.5 million to $43.0 million for nine plants, with $33.1 million based on the "design" values of parameters. The estimates were particularly sensitive to the a assumption, indicating that as much as 30 percent more would be spent on treatment if photosynthesis were assumed inoperative. The costs of treatment appear to be more sensitive to parameter selection for higher standards of river water quality.

While cost is an important factor in water pollution control planning, the following is undoubtedly a more essential consideration. If treatment plant levels are selected on the basis of assumed values of parameters, and the actual values of parameters turn out later to be unfavorable, the water quality standards would not be met. The effects on stream quality for such an unfavorable set of parameters are more serious with lower levels of treatment than with higher levels. This would indicate that if the treatment process for a plant is limited to primary treatment on the basis of computation with a mathematical model, the plant ought to be constructed for possible expansion to secondary or higher levels of treatment. Furthermore, communities which construct primary treatment plants should be advised that they may turn out to be inadequate.

While the Camp-Dobbins equations for BOD and DO sag are theoretically an improvement over the Streeter-Phelps equation, they are only as good for predictive purposes as the stream parameters employed for the solutions. Because existing technology does not include satisfactory procedures for measuring all parameters separately, the practice is to estimate some of the parameters by trial-and-error, so that computed oxygen sag curves match field measured curves. More than one set of parameters are possible from this approach. If judgments are used to adjust the parameters for assumed future conditions, such procedures are risky and can lead to highly unreliable results.

Both CDM and FWPCA attempted to employ the latest techniques for estimating parameters. While some analysts would prefer other techniques, there is really no way to check results. Such estimates by experts do not assure that future conditions will agree closely with predictions. On the basis of the studies for this report, it would appear to be prudent to base every system design on the premise that the assumed values of parameters may be inaccurate, and possible ranges of parameters should be investigated.

The methods used to determine stream parameters should be improved. Emphasis should be placed on the development of good techniques to evaluate parameters individually and as directly as possible. Only then, can engineers make confident adjustments to parameters in order to predict future conditions.

CHAPTER IV

TIME VARYING MATHEMATICAL MODEL

A. Introduction

The preceding two chapters have discussed a steady state mathematical model for studying water pollution control programs for a non-tidal river. The authors started with a steady state model because, despite its obvious limitations, most regulatory agencies assume a steady state corresponding to a low flow summer period with a specified return interval as the basis for estimating the efficacy of water pollution control programs.

While steady state models for estuarine streams are rarely satisfactory, it is judged that steady state models can continue to be useful to determine approximate plans for water pollution control for many non-tidal rivers. Such models can be developed using the data from brief field surveys, and can provide comparisons of alternative plans for pollution control at reasonable cost for the engineering studies. It is clear, however, that adequate margins of safety over the results indicated by the model studies should be employed.

For important streams with existing or potential trouble spots with respect to water quality, an approximate model of river conditions based on the steady state will not provide the means for a sufficiently adequate simulation of conditions when streamflow and environmental factors vary substantially over a period of operation. Thus, whereas tentative plans for construction and operation of waste water treatment plants can be formulated for steady state assumptions, changes in construction and/or operation may be indicated to be necessary or desirable after studies are made with a time varying mathematical model.

This chapter discusses studies to develop a mathematical model for unsteady conditions, which represents system performance more realistically than the steady state mathematical model. The unsteady state model can be used to investigate the effectiveness of an assumed configuration of treatment plants while a stream's assimilative capacity changes with distance and time. Effectiveness can be determined in terms of compliance with specified stream quality criteria. Also, when a simulation showing the variation of quality with time is available, the suitability of the environment for fish propagation or other specific purpose may be studied in detail.

For any river station over the length of stream upon which the model is patterned, the techniques can trace a time profile of flow and quality in terms of BOD, dissolved oxygen, coliforms and chlorides while the upstream discharge, water temperature and solar radiation are changing. In the next chapter, the results for model operation are discussed for a single station which was shown to be critical by the steady state studies. It would be possible, however, to develop a record of flows and any or all of the water quality parameters at any other specified station.

With a change in the printing instructions, values could be shown for all stations at any specified time. This is the form in which some of the other investigators have shown their results.

Many investigators have recognized the need for a satisfactory method of determining the spatial and temporal variations of dissolved oxygen in a flowing stream. It is only in recent years, however, that methods of "operations research" and the electronic computer have been available for practical investigations of unsteady conditions. Significant advances in this regard have been made by Thomann and collaborators (17, 18, 19, 20, 21, 22) who, since 1963, have presented results for mathematical models designed for the estuarine conditions of the Delaware River. Such models determine the dissolved oxygen at various locations in a body of water in response to the variations of BOD inputs (and other changing environmental effects such as temperature and photosynthesis). A feature of Thomann's approach has been the partitioning of a body of water into segments which are affected, not only by local factors, but also by interrelationships with all other adjacent segments and through them with more distant segments. In order to employ his approach, it is necessary to have equations to describe the variation of any parameter with respect to distance, or time, or both. The techniques lead to a number of linear differential equations with time included as a variable, which are solved by means of numerical integration using a high speed computer. The computer program developed for the Delaware River (23) was adapted for the simulation of the Connecticut River by Arnoldi and Hoover (24); this application was limited by an assumption of constant values of river flow (except for tidal effects) and other parameters.

Another serious investigator has been O'Connor (25, 26, 27, 28). Like Thomann, he has aimed at developing differential equations and appropriate methods of computer solution. Again, the approach requires sophisticated mathematical forms incorporating the parameters which vary with distance and/or time.

Dobbins, with Dresnack and Bella (28, 29) have focused on the methods of numerical analyses to solve the differential equations underlying the BOD and DO profiles for streams.

All of the investigators, whose work is outlined above, have developed models applicable to both estuarine and non-tidal streams. Under estuarine conditions, the diffusion of BOD and DO, and any other substance may be considerable at certain times. With the non-tidal stream, however, even with typical low velocities in the downstream direction during critical low flow periods, the effect of diffusion may be ignored. This may be considered to be a special case, "plug flow," which would partially reduce the complexities of the methods employed by these investigators.

It has been found, however, that when "plug" flow can be assumed, as is the case for almost all non-tidal conditions, different methods can be used, based upon relatively uncomplicated elaborations of a classical approach such as the oxygen sag expression employed by Streeter and Phelps. This can be accomplished with virtually no loss of precision, compared with the more sophisticated methods. Thus Frankel and Hansen (31, 32) have used an expanded version of the Streeter and Phelps equation, particularly as modified to include diurnal variations of photosynthesis. The DO deficit may be plotted versus time, or versus distance, when the velocity of flow along the reach is known. The reaches are processed successively in the downstream direction. By making a number of simulations with different conditions at different times at the starting point of each tributary, enough results are produced to plot variations with respect to distance and time.

The Frankel and Hansen model assumes that the stream flows for respective calculations are available as input. Le Feuvre and Pogge (33), with Mac Roberts, have worked on simulation procedures in which flow variations are routed from reach to reach throughout a river system; the results are being used to study quality variations on the Lower Kansas River System.

The examples cited above are believed to be representative of the most advanced work, and some features have influenced the direction of development of the time varying mathematical model described herein. The principal objectives have been to make maximum use of the types of data which can typically be obtained from records and field surveys, and to produce the kinds of results that are needed to examine the effectiveness of water pollution control programs. It has been found possible to accomplish these objectives without sophisticated manipulations of differential equations -- by working with short reaches and/or times, over which steady state conditions may be assumed to apply without undue loss of accuracy, and by accepting input data in any available form (such as a table when an equation cannot be created without compromising the data).

While the procedures have been developed with the Merrimack River data and previous investigations as background, it is believed that the time varying mathematical model should have wide application. The remainder of this chapter is devoted to the details of the procedures, while the results of the computer runs are described in the next chapter.

It is noted, at the outset of this exposition, that the procedures are deterministic. Random variations of input data may, however, easily be accommodated in a simulation model. Alternatively, the output values may be assumed to be the means in a band of values encompassing the variations of an output having stochastic properties.

B. General Methodology for Time Varying River Model

It was decided to develop the methodology to show the effects of time varying river parameters, for use with currently accepted steady state formulas for determining the quality characteristics. For this purpose, it was necessary to work with sufficiently small time intervals during which these parameters can be assumed to be constant. The previous steady state study divided the stream into 62 reaches with distances between stations varying between 0.01 and 5.90 miles. The travel times for the constant design discharge of 650 mgd varied from 0.02 to 1.04 days. Preliminary studies for the time varying river model indicated the desirability of providing additional river stations, in order to properly process the input data.

The adopted system utilizes moving stations in addition to the 62 fixed stations of the steady state model. The moving stations are located so that the time of travel between two successive stations for any flow is an arbitrary time interval. For this study, it was found that the analysis of six intervals per day was sufficient to show the diurnal variation of dissolved oxygen due to photosynthesis. Because the locations of the moving stations are dependent only on the travel times, which are in turn dependent on the discharge, it follows that the moving stations are in different locations for every discharge. A procedure was devised which performs an oxygen sag analysis for the short stretches between fixed and moving stations.

Figure 4 shows a possible configuration of fixed and moving stations. These stations are identified by "I" and "N" designations, respectively. The figure also shows the subprograms used to compute the changes in oxygen concentration between the stations; the appropriate subprogram depends on the relative positions of fixed and moving stations.

The locations of the fixed river stations remain unchanged for the particular river studied. The moving river stations, however, may be closer or farther apart depending on the magnitude of the flow. If, as in the case of Figure 4, there are no changes in the discharge or hydraulic characteristics of the stream channel, the moving stations will be evenly spaced. This is because the velocity would remain constant and therefore the distance traveled in each constant time interval would remain constant.

For a larger discharge, corresponding to a greater velocity, the distance traveled in a specified time interval would be greater; thus, the moving river station N+3 may be reached after the flow has passed fixed river station I+3. For a smaller discharge, N+3 may be reached before station I+2.

Six values of flow are routed for each day. As each incremental volume of flow, which is assumed to originate at the upstream end of the stream, moves downstream it must pass all of the fixed river stations, which are indexed by I, I+1....., etc. At these stations, sewage discharges carrying pollutants may be added; the river flow may decrease because of water supply intakes; the discharge rate may change in accordance with the routing procedure for unsteady flow; and any or all of the parameters which vary with distance and time may change.

A description of all of the elements of the model and a summary of the respective functions used for repetitive evaluations are contained in Appendix I and Appendix II. Functions 1, 2, 5, and 6 were especially developed or were obtained by modifying similar functions for the steady state model, in order to accommodate the needs of the time varying model. The remaining functions are the same for the steady and unsteady state models. It is possible to bypass those functions which are not needed for desired results, and/or to print only those results which are of interest. This is a matter of detailed programming.

An outline of the step-by-step computation procedure which takes account of changing conditions at both fixed and moving stations is shown in Figure 5.

The computations proceed in a downstream direction. A

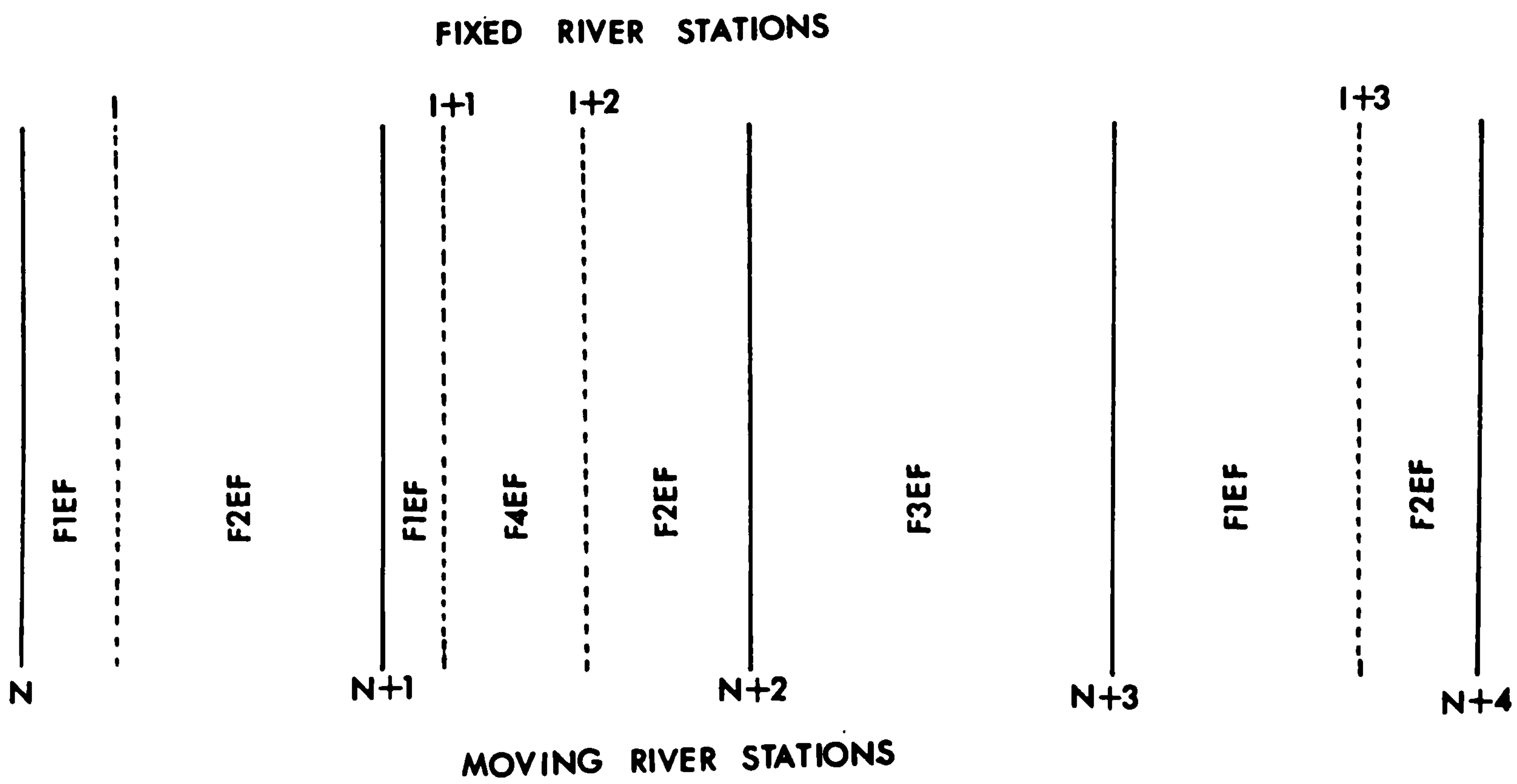


Figure 4 Example of Configuration of Fixed and Moving Stations

NOTES: (1) PATHS SHOWN FOR ONLY ONE FLOW INCREMENT AS IT PASSES BETWEEN 2 FIXED RIVER STATIONS. REPEATED FOR 6 INCREMENTS PER DAY, AND FOR ALL REACHES.
(2) FLOW ROUTING COEFFICIENTS ESTABLISHED PRIOR TO BEGINNING OPERATIONS SHOWN.

I-corresponds to river station number
N-corresponds to time interval number

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I=Number of next river station
Call F1T (I)
         Compute Q (I)
         Compute T (I)
Call F2EF (I,N)
          Is T(I) less than TZ(N) + 1/6 ?

    NO                                                    YES
Call F5A1 (I,N)                                        Call F4EF (I,N)
     Compute ZZ3(N); ZZ4(N); ZZ7(N)                       Call F5A1 (I,N)
Compute F(I-1) for new Temp.                                   Compute ZZ3(N); ZZ4(N); ZZ7(N)
N=N+1                                                     Compute F(I-1) for new temp.
Compute TZ(N); EZ(N); FZ(N); DOZ(N)                       Compute E(I); F(I); DO(I)
Call F3EF (I,N)
     Is TZ(N) + 1/6 less than T(I) ?

   NO                                        YES
Call F1EF (I,N)                           Call F5A1 (I.N)
     Call F5A1 (I,N)                           Compute ZZ3(N); ZZ4(N); ZZ7(N)
       Compute ZZ3(N); ZZ4(N); ZZ7(N)     Compute FZ(N) for new temp.
     Compute FZ(N) for new temp.          N= N+1
     Compute E(I); F(I); DO(I)            Compute TZ(N); FZ(N); EZ(N); DOZ(N)

Call statements for routines computing coliforms (MPN), chlorides (PPM), water supply for
community (MGD), quantity and quality of sewage discharge if any, and resulting condition
of the stream Q(I), F(I), DO(I), E(I), etc. at outfall
```

Figure 5 - Paths for Mathematical Analysis - Time Varying River Model

set of "distance varying parameters" are assumed to remain at these values until the next fixed river station is reached. At the beginning of each time interval which corresponds to a moving station location, one or all of the "time varying parameters" may change. These are assumed to remain constant until the beginning of the next time interval. Many of the functions utilize both time varying and distance varying parameters; thus, a reach of the river where all parameters are constant is always equal to or less than the distance between moving stations.

With the configuration of fixed and moving river stations described above, it was possible to use the oxygen sag equation developed by Streeter and Phelps (14) as modified by Camp (15), and other steady state formulas previously applied by the Principal Investigator. Four subprograms are available for implementing Function 2 to compute the BOD and dissolved oxygen (F1EF, F2EF, F3EF, or F4EF) and the appropriate one in each case depends on the relative locations of the fixed and moving stations (see Figures 4 and 5).

Because the location of a moving station is derived from the stream flow and, therefore, may be in a single place only once there is little reason to print out the computed values for such a station. These values are merely used as the initial values for the next river reach which will end at either a fixed or another moving station,whichever comes first.

All of the computed values for the time, coliforms, BOD, DO, chlorides and discharge remain in computer storage until a flow increment has passed all the way downstream, after which these values may be printed for as many of the fixed stations as desired. After the printing operation is complete, the procedure cycles back to the upstream end of the stream, and a new incremental flow is processed. This procedure may be repeated for as long a period as data are available.

Because of the limitations on computer storage and the computer time required, the authors have generally processed only one month of data at a time. To obtain approximately one month of output for three systems of treatment plants, about ten minutes of computer time were required. Input data for several days before the start of the period for which output data is desired are needed in order to process the first flows of the period.

Previous analyses using a steady state model have demonstrated that particular stations in this river are critical with respect to river quality criteria. For the operational studies of the time varying mathematical model, the most downstream of these stations was retained for the production of results.

C. Factors Affecting Oxygen Variations

Most of the stream factors which influence the oxygen balance have been studied extensively by many investigators. Those parameters which are most often considered are as follows:

1. Deoxygenation
2. Reaeration
3. Algae Activity
4. Benthal Demand
5. Settling and Resuspension of BOD
6. Temperature
7. Sunlight
8. Streamflow

All of these factors were included in the steady state model, but they varied only with distance and were assumed constant with time. The rate of benthal demand was included in the parameter defining the rate of resuspension of BOD; the temperature was assumed to be $25^{o}C$; the stream flow was 650 mgd; and the sunlight condition was assumed to be cloudy.

For the development of a realistic unsteady state model, acceptable relationships had to be developed to demonstrate the time variation of the parameters. Satisfactory existing relationships between water temperature and oxygen saturation, reaeration, and deoxygenation could easily be adopted for the model. Water temperature is easily measured and was in terms of a daily input value. Water temperature data for the operation of the model were taken from the records of treatment plants along the river (34). A relationship between algae activity and sunlight based on experimental data for the study stream was also available. Daily values for sunlight intensity at Boston, Mass. were used for input (35).

For benthal demand, and the settling and resuspension of BOD, little useful information was available. A relationship for these parameters would have to consider discharge as it effects scour, and account would also have to be kept of available benthal material subject to resuspension as the discharge changes. If better data were available, it might be possible to develop an empirical relationship. The studies with the steady state mathematical model (see Chapter III) indicated that the dissolved oxygen values for the stream were not very sensitive to changes in the rate at which BOD is settling or is resuspended. Accordingly, it was decided to make these factors constant with time for each computer run. Since each run covers a limited portion of the year, any inaccuracies due to this assumption are small.

As noted in the next section, the representation of unsteady streamflow, using a new empirical procedure, was developed and tested. An upstream hydrograph (36) was determined by combining the recorded flows of the main stream and tributaries upstream of the study reach.

Succeeding sections provide details on the relationships which were adopted for deoxygenation, reaeration, and photosynthetic oxygen production.

It is emphasized that the programming methods for the time varying mathematical model are quite flexible, and can facilitate revisions in computational procedures to accommodate different relationships for the parameters which may be preferred by other analysts.

Variable inputs from waste treatment plants have not been included in the time varying model. These were not considered to be of importance for a model patterned on treatment arrangements for the Merrimack River, but could easily be accommodated by relatively minor programming modifications. Similarly, it would not be difficult to include the effects of contributions of pollutants from overland flow and tributaries, if estimates were available.

D. Streamflow Routing Procedure

For the biological analysis of a stream where the time varying nature of streamflow is taken into account, two concepts should be considered. One of these is the changing of the dilution of pollutants. The other is the differing travel times for pollutants. To properly consider these effects, an empirical flow routing procedure has been developed, which is especially suited for studying normal and low flow periods.

The only input data required are an upstream hydrograph and velocity-discharge relationships for the reaches. Unlike many flood routing procedures, this method of low flow routing is concerned with the time displacement of pollutants for different discharges rather than representing the movement of translatory waves. This procedure proved to be quite complex and a major portion of the time used for the study was spent on its development. Also, much of the computer time required for processing with the time varying model is attributable to the routing procedure. A detailed explanation of this procedure is contained in Appendix IV. The establishment of coefficients for flow routing is completed prior to the operations in Figure 5.

It is noted that the effects of overland flow and ground water flow on stream discharge are included only implicitly in the streamflow routing which aims at correlation of available stream records. If these types of data are available, they can be included explicitly with little difficulty.

E. Deoxygenation

For a flowing stream, the measurement of the oxidation coefficient or deoxygenation constant, usually abbreviated k_1 in the literature, is very difficult. The rate of removal of BOD in a stream may be determined from the BOD values at an upstream and a downstream station and the travel time between them. However, because BOD may be removed in a stream by processes other than oxidation, the oxidation coefficient may not equal the computed stream rate. Current methods of analysis cannot separate the oxidation of the organic matter as determined in a BOD test, from the other effects in a flowing stream. Usually the k_1 factor for a stream is higher than the laboratory k_1 but less than the total BOD removal rate of a stream. This problem in evaluating k_1 is evidenced by the fact that Camp Dresser and McKee (2) and the FWPCA (1) each ran extensive surveys of the study stream and determined very different values.

O'Connor (37) lists the factors which may influence the oxidation rate as turbulence, biological growths on the stream bed, algae, immediate or chemical demands, nutrients and lag or adaptive periods. Other factors which may affect the rate of BOD removal, but not necessarily the oxidation rate, are sedimentation and flocculation, scour, and volatilization. It is obviously impractical to quantify the effects of all of these factors. Some distinction was recognized by Streeter and Phelps (14) and others who included effects other than oxidation by incorporating the coefficient k_3. The k_3 constant is usually interpreted as the BOD which settles out to bottom deposits rather than exerting its demand on the flowing stream.

For the basic data in the time varying model, the authors have adopted the values of k_1 used for the steady state model (3, 4) which were taken from the Camp Dresser and McKee report (2).

It is generally accepted that as the temperature of a body of water increases, factors affecting the k_1 value also increase their activity. Recent work by Zanoni (38, 39) demonstrates that a modified Arrhenius expression using two breaks, one at $15^{o}C$ and the other at $32^{o}C$, gives a good approximation of the effect of temperature on the value of k_1.

These equations are as follows:

Equation	Range
$K_t/K_{20} = (0.796)\ (1.126)^{(T-15)}$	2-15°C
$K_t/K_{20} = (1.047)^{(T-20)}$	15-32°C
$K_t/K_{20} = (1.728)\ (0.985)^{(T-32)}$	32-40°C

These equations were reproduced in the form of a matrix of values which is used to modify the distance varying values previously adopted for the steady state model, in accordance with the time varying temperature.

It is recognized that other factors may alter the deoxygenation constant. Temperature was, however, the only time varying parameter which has an adequate explicit relationship which could be incorporated in the mathematical model.

F. <u>Reaeration</u>

The factors affecting the equilibrium between oxygen diffusion into and out of a liquid are many and complex. As far back as 1925, Streeter and Phelps (14) found that the reaeration coefficient, k_2, is influenced by the hydraulic and physical characteristics of a river channel. Two well known formulas determine k_2 as a function of velocity and depth of flow. One by O'Connor and Dobbins (40, 41) is as follows:

$$k_2 = \frac{(D_m V)^{1/2}}{2.303\ H^{3/4}}$$

k_2 = reaeration coefficient as constant per day

D_m = molecular diffusivity of oxygen in water in sq. ft. per day (varies with temperature)

V = average velocity in ft. per day

H = depth in feet

Another formula has been proposed by Churchill, Elmer and Buckingham (42):

$$k_2 = \frac{5.026\ V^{0.969}}{H^{1.673}} (1.024)^{T-20}$$

k_2 = reaeration coefficient as a constant per day

T = temperature in degrees centigrade

V = velocity in ft. per sec.

H = depth in feet

Issacs and Gaudy (43) in one of the most recent studies have developed the following formula:

$$k_2\ (T^{o}) = 3.739\ \frac{V}{H^{3/2}} (1.0241)^{T^{o}-20}$$

$k_2\ (T^{o})$ = reaeration coefficient as a constant per day at a temperature T^{o}

V = average stream velocity in ft. per sec.

H = average stream depth in feet

All of these formulas are basically similar in that they show that the reaeration coefficient depends on the stream turbulence, which by fluid mechanics theory would be expected to increase with increasing velocity and with decreasing depth. Temperature also enters into each formula since it affects the diffusivity of oxygen. Some of the studies used data for actual streams while other investigations were based on laboratory simulations. The nature of the equations implies that each class of stream could have a unique formula for relating k_2, V, H, and T.

Another approach by Camp (2, 13) estimated the value of k_2, along with other parameters entering the standard oxygen sag formulas, from light- and dark-bottle oxygen analysis. Camp demonstrated that the Merrimack River, on which the mathematical model has been based, exhibits k_2 values which are much less than values indicated by the above formulas for streams of this type. The FWPCA (1), using similar techniques, estimated higher values for the reaeration constant while also determining proportionately higher values for the deoxygenation constant (k_1). Since this method for evaluating k_2 depends upon first estimating k_1, the differences between the CDM and FWPCA estimates are attributable to the determinations of k_1.

Theoretical formulas are particularly useful for mathematical model studies. To use them effectively, however, more stream factors would have to be developed which vary with distance and time. Channel roughness and velocity vs. stage data appear to be most important although wind, precipitation, ice cover, and barometric pressure also have an effect. None of the above formulas gives results for the Merrimack River consistent with those determined in the two stream studies.

All the effects except temperature have been grouped into one constant which is unique to each river stretch. This is essentially what the two steady state studies have done. This factor implicitly includes the channel roughness and average values of other parameters affecting aeration.

The following formula is based on the concept that an interfacial liquid film is in a continuous state of random renewal (44, 45):

$$k_t = k_{20}\theta^{(t-20)}$$

k_t = reaeration constant at any temperature "t"

k_{20} = measured reaeration constant at 20° centigrade

θ = temperature coefficient

In regard to an appropriate θ value, Dobbins and Metzger (46) have observed in experiments with helium and nitrogen that the absorption coefficients for gases of different diffusivity are affected differently by temperature, and that for a given gas the effects of temperature on the absorption coefficient depends on the level of turbulence. Metzger (47) extended these results to include oxygen absorption and interpreted the variation in reported θ values. He has proposed a series of graphs and formulas for predicting this value. It seems likely, in light of this recent work, that a different value of θ may be appropriate for each river stretch.

Considering the available information, the following values for θ were adopted:

Range Centigrade Temp.	θ Value
10-20	1.025
20-30	1.035

With this formula, a table of coefficients was developed to modify the values of k_2 which were originally estimated for the steady state model at 25 degrees centigrade. These original values, as mentioned earlier, vary from one river stretch to another and probably reflect the physical characteristics of the river during the low flow period. The table of coefficients applies to temperatures from 10^o to 30^o centigrade.

By using a table rather than an inflexible formula, it is simple to introduce modifications as more accurate information becomes available. In the future, it should be possible to develop additional tables of coefficients which would allow the k_2 parameter to vary with flow and with other stream and atmospheric parameters which vary with distance and time.

G. Photosynthetic Oxygen Production

There appears to be no uniformity in design practice as to whether to evaluate the effects of algae activity on stream water quality. This is probably due largely to difficulties in obtaining appropriate basic data and incorporating them in a predictive formula that can be used with confidence. The sensitivity of algae to changes in light intensity, as influenced by cloud cover in the atmosphere and turbidity in the water,have been cited as reasons why photosynthetic oxygen production is too variable to be a reliable addition to stream oxygen resources.

Literature is in general agreement that algae growths which are concentrated at a river location are not desirable from an oxygen balance standpoint. O'Connell and Thomas (48) concluded that the oxygen production by benthic algae and other attached plants have little beneficial effect on the oxygen balance of streams. On the contrary, they state that oxygen demand associated with algal respiration and decay negate any temporary gains made as a result of photosynthesis. Since these algae must undergo their entire life cycle in the same place, this location must eventually be subject to oxygen demands of the same magnitude as the amount of oxygen originally produced due to photosynthesis.

The effects of phytoplankton may, however, be more significant in the stream environment. When these plants die they will not be in the same place they were when they were producing oxygen. In fact, they could decompose quite harmlessly in the ocean.

Hull (4) has the following interesting comment regarding these concepts:

"I have been frequently reminded that for every gram of oxygen released by photosynthesis, a gram of biochemical oxygen demand is produced by the synthesis of organic matter, and therefore there can be no net gain of oxygen. This is a gross oversimplification---it must be qualified by consideration of the relative rates, periods, and locations of the oxygen liberation and subsequent oxygen demand. To support this, one need only point to the vast standing crop of fixed carbon on this earth in the form of petroleum, coal, plants and animals, including several billions of human beings. All of this carbon was fixed by photosynthesis, but some of it has waited millions of years to be oxidized, before it will eventually take back the oxygen liberated at its birth."

Although the authors do not mean to imply its general applicability for all streams, it appears that in the Merrimack River, which was used as a basis for this model, some benefit can be gained from the action of phytoplankton. Camp (13) attributes two thirds of the available oxygen in the Merrimack to photosynthesis and states that reliance must be placed on the photosynthetic production of dissolved oxygen. Both the FWPCA and the CDM reports have included an analysis of the effects of algae. They have assumed positive alpha (a) values of approximately 0.3 - 0.5 ppm per day corresponding to a cloudy day period.

The oxygen model which has been formulated represents as closely as possible the effect of algae on the stream. Using records from the U.S. Weather Bureau (34) and correlative studies by the FWPCA, relationships were developed between total daily solar radiation in Langleys (gm-cal/cm^2/day) and the photosynthetic oxygen production in ppm per day (see Figure 6). A series of parabolic curves were drawn (see Figure 7) of gross oxygen production vs depth, similar to those shown in the FWPCA report and developed in other studies (50). The area above one of these curves represents the total oxygen production for the indicated solar radiation value. The alpha (a) value can be determined by dividing the area over the curve by the average depth of the reach in question. With this method, an estimate of the oxygen contribution from algae can be determined from daily solar radiation (time varying) and depth (distance varying).

Also incorporated in this procedure is a technique which takes account of the diurnal variation in the rate of photosynthesis. Each daily solar radiation value has six rate constants which correspond to the six 4-hour periods occurring each day. For example, the first value is for 12 midnight to 4 A.M. and the second is from 4 A.M. to 8 A.M. In the absence

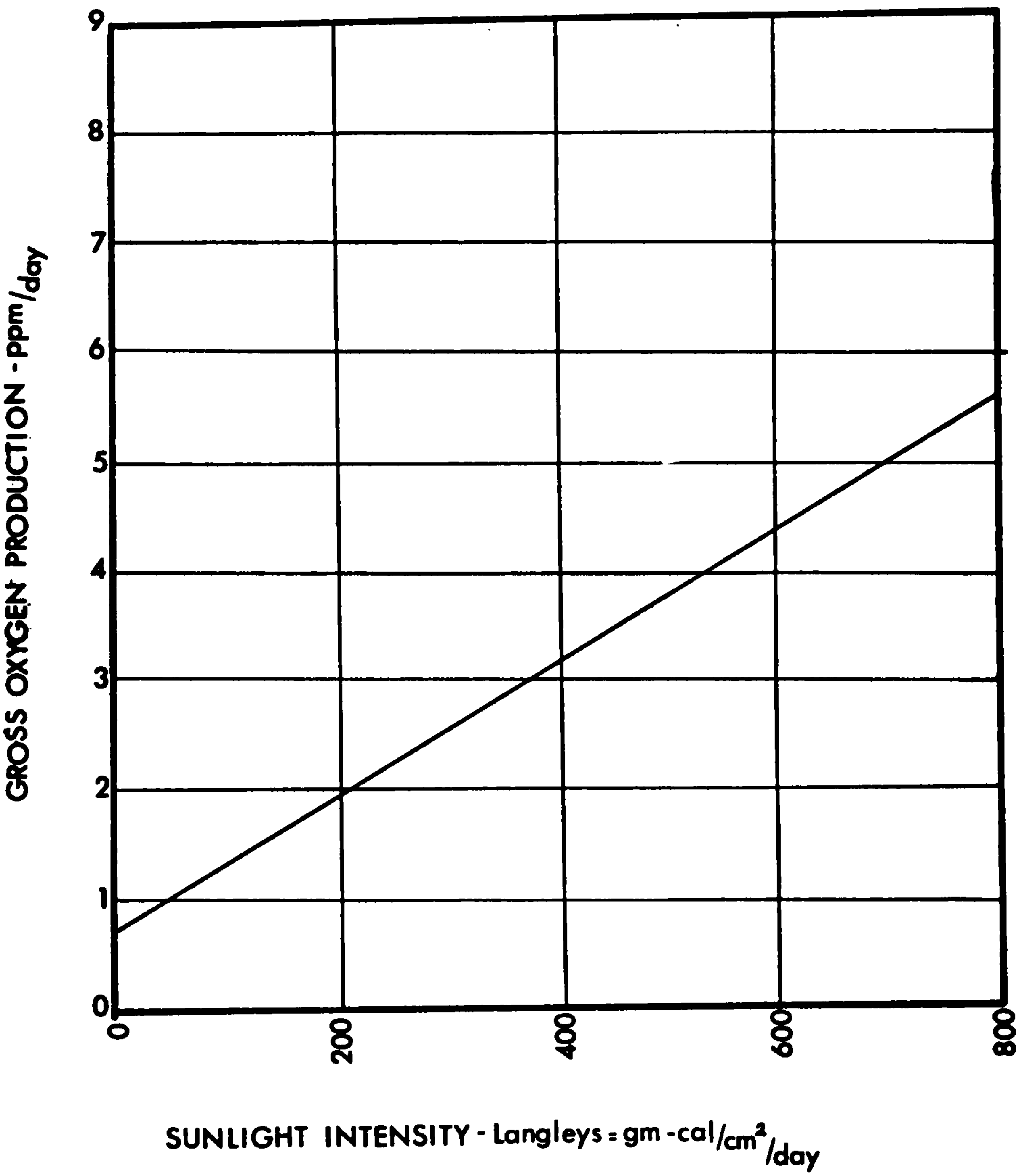


Figure 6 Gross Oxygen Production vs. Sunlight Intensity

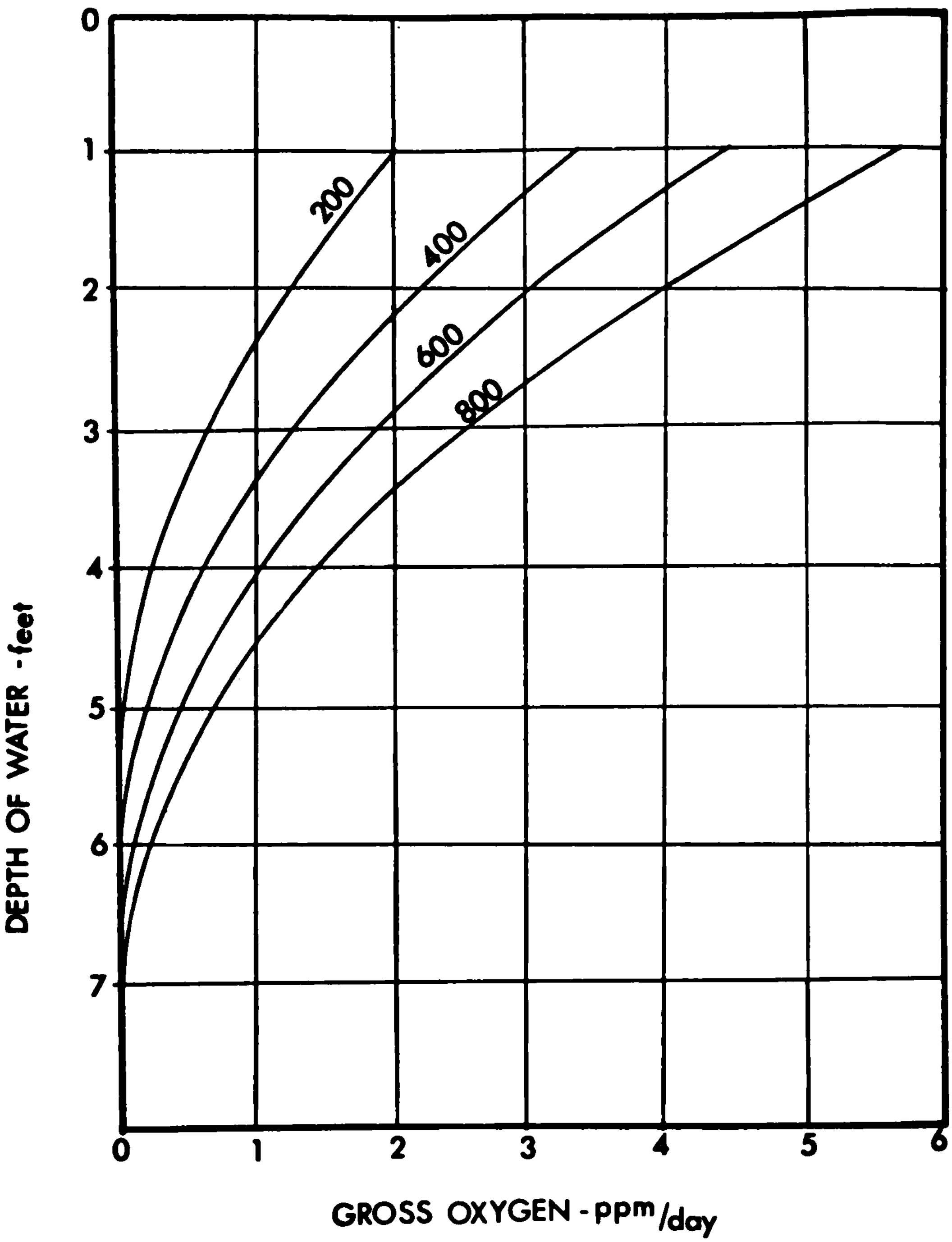


Figure 7 Gross Oxygen Production vs. Depth for Indicated Values of Solar Radiation in Langleys

of light energy it is known that plants respire so night-time values must be negative. Also cloudy day values must be low. Most studies of the Merrimack River have observed stream dissolved oxygen variations of about 1.0 ppm for sunny days and something less than that for cloudy periods.

In the literature it has been reported that respiration accounts for 10 percent of the gross photosynthesis (51); however, this percentage should vary considerably for different sunlight intensities. A matrix of values was developed which approximates this variation while maintaining the measured average daily rates.

The use of matrices permits flexibility in the computations. For example, while maintaining the same total average daily oxygen contribution, it is possible to increase the activity and to simulate the venting to the atmosphere of molecular oxygen when the stream becomes supersaturated. The corresponding night-time demand will be proportionately greater and, therefore, lower early morning values of dissolved oxygen will be observed. Although studies have indicated that significant supersaturation does occur because of algae (52), this effect has not been included because it can be quite localized and not readily estimated. For the initial studies, a constant night-time alpha value of -0.5 ppm per day was selected.

There were some factors which were not taken into account in this study because of insufficient information. In the natural case the peaks and valleys in the dissolved oxygen curve lag about one day behind the extreme values of the sunshine curve. This has been explained by the fact that with a large amount of sunshine for two or more consecutive days, there may be an increasing production potential caused by the development of a young and vigorously growing phytoplankton population. Consecutive cloudy days result in a loss of phytoplankton and are followed by increased BOD and decreased DO (51). Recent studies have tried to clarify the factors which control this kind of activity but further research is needed before these phenomena are fully understood (53). The possibility was also investigated of including the effects of varying temperature and nutrient concentrations on the rate of oxygen production by algae (54) but, like the lag effect described above, the proposed methods have not been sufficiently proven to warrant their inclusion at this time.

The time varying model is based on an actual stream. Studies made of this stream previously indicated that benthic algae would not be a significant factor; however, it would be quite simple to include these effects.

The matrices were developed for summer conditions. It would be a simple matter to insert other input tables which would be applicable to fall or winter temperatures.

In summary, the model is based on the following assumptions: 1) the rate of algae respiration is constant during all dark periods; 2) any instantaneous rate is independent of the algae's previous history; 3) algal synthessis is independent of temperature; and 4) there is no significant relationship between nutrient concentrations, due to differing sewage treatment levels, and algal synthesis.

H. Description of Computer Program

The computer program developed for the time varying river model consists of a main program and seventeen subroutines. These source programs were written using Fortran II language and programming rules, and were processed by the CDC 3300 computer system installed at Northeastern University in 1967. The computer as of January 1, 1969 had a core storage of 64,000 words and a processing speed of more than 300,000 basic instructions per second.

The main computer program has three major components in addition to the standard statements required by the specific computer system used. They are as follows:

1. Statements to control data processing--to place data into storage; to cause a "path" for data processing; and to print results
2. Procedure for the routing of the unsteady inflow
3. Equations for the river model which perform simple calculations or call upon the subroutines where the more complex analyses are handled.

The subroutines are coordinated with the main program with respect to the use of common symbols and common areas of computer storage.

Appendix I contains precise descriptions of the various input data needed for this river model. The input data are of several types and are designated by the following symbols:

1. "Z" Data - hydraulic and stream purification constants associated with river location
2. "CZ" Data - temperature coefficients to vary "Z" data with temperature
3. "ALA" Data - data varying with solar radiation
4. "NT" Data - input data varying daily (sunlight, temperature, streamflow)
5. "G" Data - physical and population data for communities

6. "Y" Data - economic data for communities
7. "A" Data - administrative assumptions for communities
8. "KD" Input Data - Table defining coefficients in equations evaluating treatment plant efficiencies
9. Miscellaneous -- miscellaneous input data (for computer processing).

All of these data are used in the solution of the functions contained in the main program and subroutines. A summary of these functions appears in Appendix II. A general flow diagram for the entire process is shown in Figure 8.

The river modeling statements are of three types: 1) identities; 2) simple additions and subtractions; and 3) "call" statements, each of which brings a corresponding subroutine into play. A given subroutine may "call" another subroutine which may "call" still another subroutine etc. Appendix III lists the computer call" statements for the seventeen subroutines and gives a precise description of the arguments of these statements.

A total of 269 statements were required to model the portion of the river used for the studies of time-varying relationships, which covered slightly over thirty miles and had 41 stations. The model was developed for a full fifty miles and 62 stations but it was found that the critical sections occurred above station 41. These modeling statements are the only part of the program which is unique for a specific stream. Once the principles of the model and the pattern for these modeling statements are fully understood, the programming of the model for another stream should not be difficult.

Each river station has a group of equations which evaluate the quantity of flow and various quality properties. Each community has two groups of equations -- for water supply and sewage treatment. The water supply equations are placed just after the equations for the river station where the raw water quality is established. The sewage treatment equations are placed just before the equations for the discharge of treated sewage.

Appendix I also contains descriptions of computed output. Generally the only computed results which are printed are the flow and water quality characteristics at one of more specified river stations. Depending on the routing of unsteady flow an average of six values of each parameter are printed daily. The number may depart from the six values in accordance with the time intervals selected for discharge values, as discussed in the description of the river routing procedure (Appendix IV).

[A]Read and print fixed input data
(KD; ALA; CZ)

[B]Read and print variable input data
(NSTA; NCOM; NDAY; NOPT)
(Z; G; Y; Z; and NT)

Read river routing input data TS, ZN
River routing procedure
Compute CQ value for every input flow

Read new initial values at upstream end ← Read and print all or a portion of new variable input (B) according to option number (NOPT)
(Q; T; E; F; DO; H; and V)

Run river model for one input flow

Print results for specified downstream stations
(Q; T; E; F; DO; H; V; WI; WR)

Have all study periods been evaluated?

NO

YES
Are there more options to be run?

NO

END

YES

Figure 8 - General Flow Diagram of Time Varying River Model

The print-out instructions provide for printing all input data as well as output results. This enables the verification of card punching and the debugging of errors which may be caused by incorrect data or data presented in a form which is unacceptable to the computer.

This program required the use of nearly all available core storage, and six minutes were needed to complete output from one month of daily input values. About two thirds of this time is required for the river routing procedure; therefore, once the inflows and river routing data are available, to run the program for different sewage treatment combinations, administrative assumptions, or river parameters takes only about two more minutes per run. At current computer prices of from $200 to $300 per hour, the initial input set for one month can be processed for about $25 and each additional option can be processed for about $10 more.

The computer output obtained was found to be very voluminous and, therefore, it was quite difficult to examine the relationships between input and output data. Because graphing of the results proved to be quite tedious, computer graphing techniques were investigated. The C.D.C. 3300 computer has a Cal-Comb plotter,but the techniques for its use were not yet sufficiently developed for use in this study. It is judged, however, that automatic graphing with the computer can be shown to be feasible and can greatly facilitate the analysis of the computer output.

CHAPTER V

SIMULATION STUDIES WITH TIME VARYING MODEL

A. Introduction

The preceding chapter has discussed the need for a time varying model and has described the general methodology and details for its employment. This chapter shows various types of results which can be obtained with this model. Results are compared with those obtained with a steady state model patterned on the same stream. Sensitivity analyses are also made to show the effect of stream parameter selection on the prediction of stream quality conditions.

A steady state model based on a seven day low flow period corresponding to a specified return interval cannot give a complete simulation of stream conditions. A time varying model can be used not only to study conditions for the variations within the worst seven day low flow period, but can also cover longer periods at other times of the year.

The time varying model is based on a deterministic approach which primarily considers the changes due to flow, temperature, and sunlight. Although not included in the studies for this report, the model could be modified to include stochastic fluctuations in the rate parameters and other input data.

Time varying results should enable planners to investigate the effectiveness of treatment plant configurations in terms of a realistic appraisal of stream conditions as they may be expected to occur in a stream. The model described herein enables the analyst to use the types of data which are typically available, without the need for sophisticated construction and manipulation of differential equations. Another advantageous feature of the model is its flexibility, which permits the replacement of any of the individual functions used for repetitive computations as better relationships become available.

Although the time varying mathematical model and the input data used to produce the results are patterned on information for the Merrimack River in Massachusetts, it is not claimed that the results reproduce values for this stream. Rather, the intent has been to study different techniques for analyses, and the implications of using available data and design assumptions in the planning and operation of water pollution control programs.

The examples described in the following sections are limited to a few basic demonstrations. It is important, however, to emphasize that, due to the comprehensive nature of the time varying model and its flexible procedures, many other input-output studies are possible. Although output is normally available from a computer run for every preselected ("fixed") river station within the study stream, the examples included herein present results for only one river station. This station was chosen because it was shown by the steady state model studies to be critical with respect to DO for the specified design period. It is judged that, unless unforeseen combinations of unusual river conditions and/or treatment plant configurations were to exist, this station would remain critical for the time varying model.

Features which could have been included for the studies are: the varying of treatment plant operations with time, the inclusion of overland flow and major tributaries along the stream, changing population configurations, the direct addition of molecular oxygen to the stream, the employment of low flow augmentation, and thermal discharge effects. In general, these have not been analyzed because they have not been of particular importance in previous studies of the Merrimack River, on which the model is patterned. They could have been included, however, without much difficulty.

The representative results for the critical station which are described in this chapter are of three types. The first demonstrates the form of input vs. output which can be obtained with the model. The second set compares the results obtainable with the time varying model with those previously determined with the steady state model. Finally, the third set comprises the results of a sensitivity analysis performed on the model parameters.

The final section of this chapter discusses various results which have been obtained by other investigators who have incorporated a statistical approach to evaluate the variation of results. This is used as a basis for discussing the possibility of expanding the time varying model to consider random variations in the input, and the variations of output produced by effects other than those included thus far by the mathematical relationships.

It is emphasized that the conclusions discussed in this chapter are based upon only limited testing work with the time varying model. They conform, however, to what may be expected from experience and intuition.

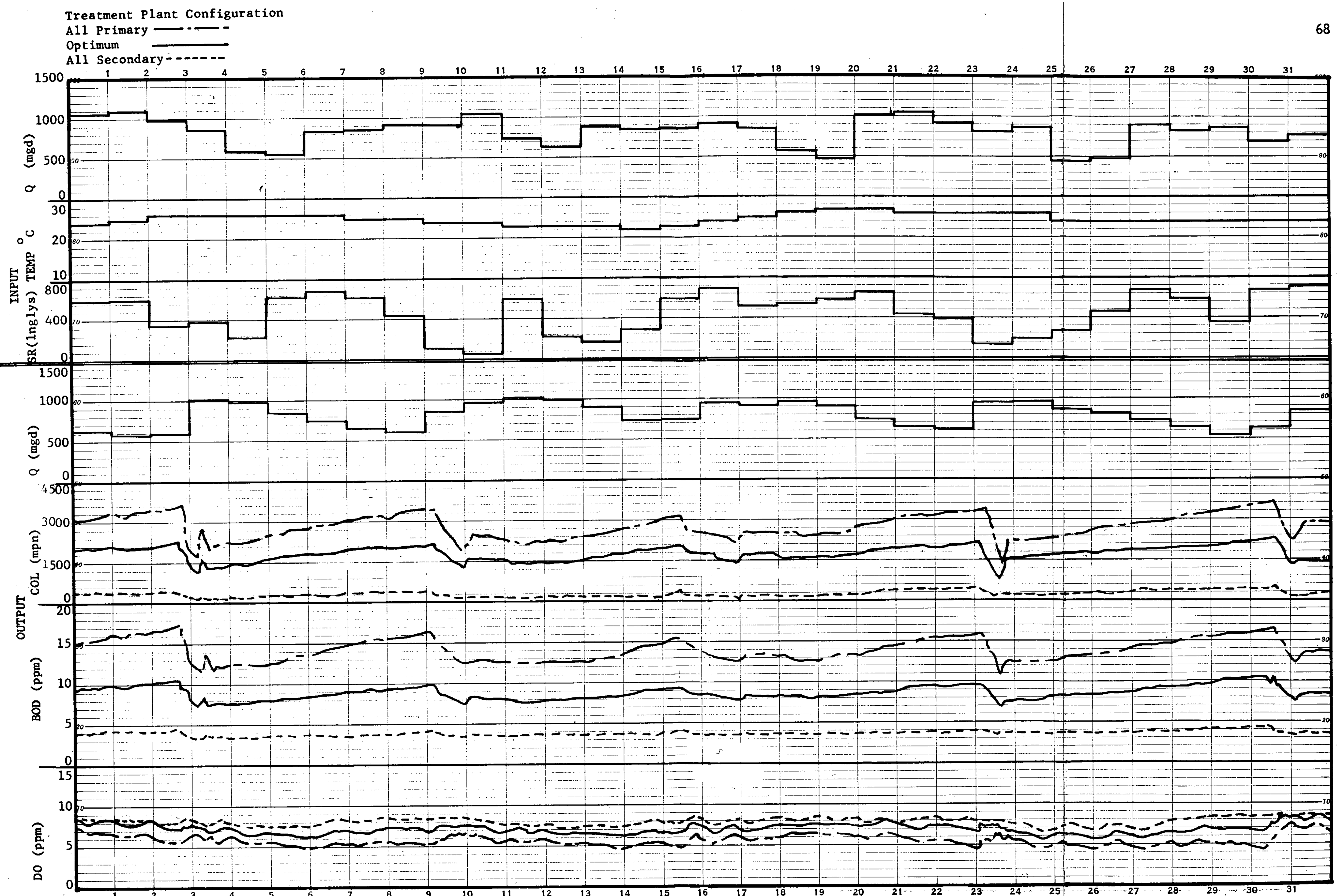
B. Results Obtained With Time Varying Model

Figures 9, 10, and 11 are charts which represent typical input vs. output conditions which may occur during periods when the heaviest demands are being placed on a stream. These charts show simulations of conditions similar to those for the Merrimack River for July and August 1964, and for August 1965. These months were chosen because they are drought periods and the inputs for the model contain the worst combinations which could be found in the records for recent years. By merely inspecting the input data, it may not be possible to determine which month's output will contain the lowest single DO value, the lowest average DO value, or the longest period with the DO values below some specified value. As shown, the output with "optimum" treatment for August 1964, with an average DO of 7.41 ppm, was generally better than July 1964, which had an average DO of 7.10 ppm. The controlling factor in this case seemed to be the temperature which averaged 24.8° C for July and 21.8° F for August, even though the discharges averaged 819 mgd and 700 mgd, respectively.

The month of August 1965 contained the worst input which could be found from the natural records. This month was used as a basis for the remaining analyses described in this chapter.

Since each chart has 32 vertical spaces available, the first space on each chart shows the input and output for the last day of the preceding month. The upper graphs of the chart show the daily averages for input data (flow, temperature, and solar radiation) as they occur at the upstream station. The bottom four graphs of the chart show output for the critical downstream station, comprising continuous graphs of discharge (mgd), coliform (mpn per 100 ml), BOD (ppm), and DO (ppm). To facilitate comparison with upstream discharge values, output discharge values are shown as daily averages, instead of the computed output which was more variable. Also computed but not shown here are chlorides (ppm), DO deficit (ppm), and two index values which could be used by expanded versions of the program to determine the utility of the stream for water supply and/or recreation.

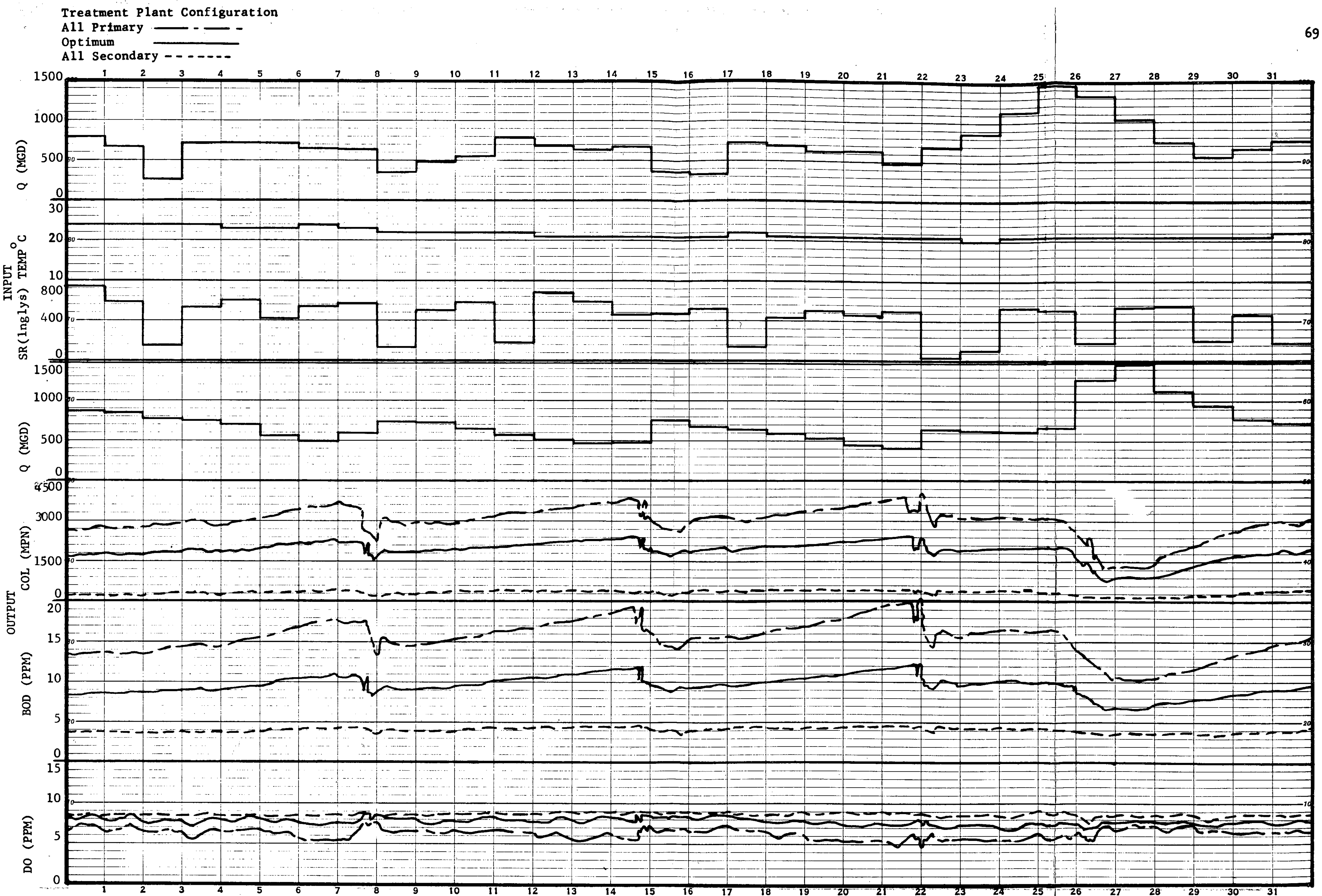
Outputs pertaining to biological conditions are shown for three different treatment plant configurations - all primary treatment with 38% BOD removal, all secondary treatment with 90% BOD removal, and an "optimum" (least cost) configuration. The optimum configuration was previously determined by the steady state model studies, and was based on satisfying an objective DO deficit of 3 ppm for the whole stream including the critical station. For the steady state model, the DO



Simulation of River Conditions at Station 41

MONTH OF JULY **19**64

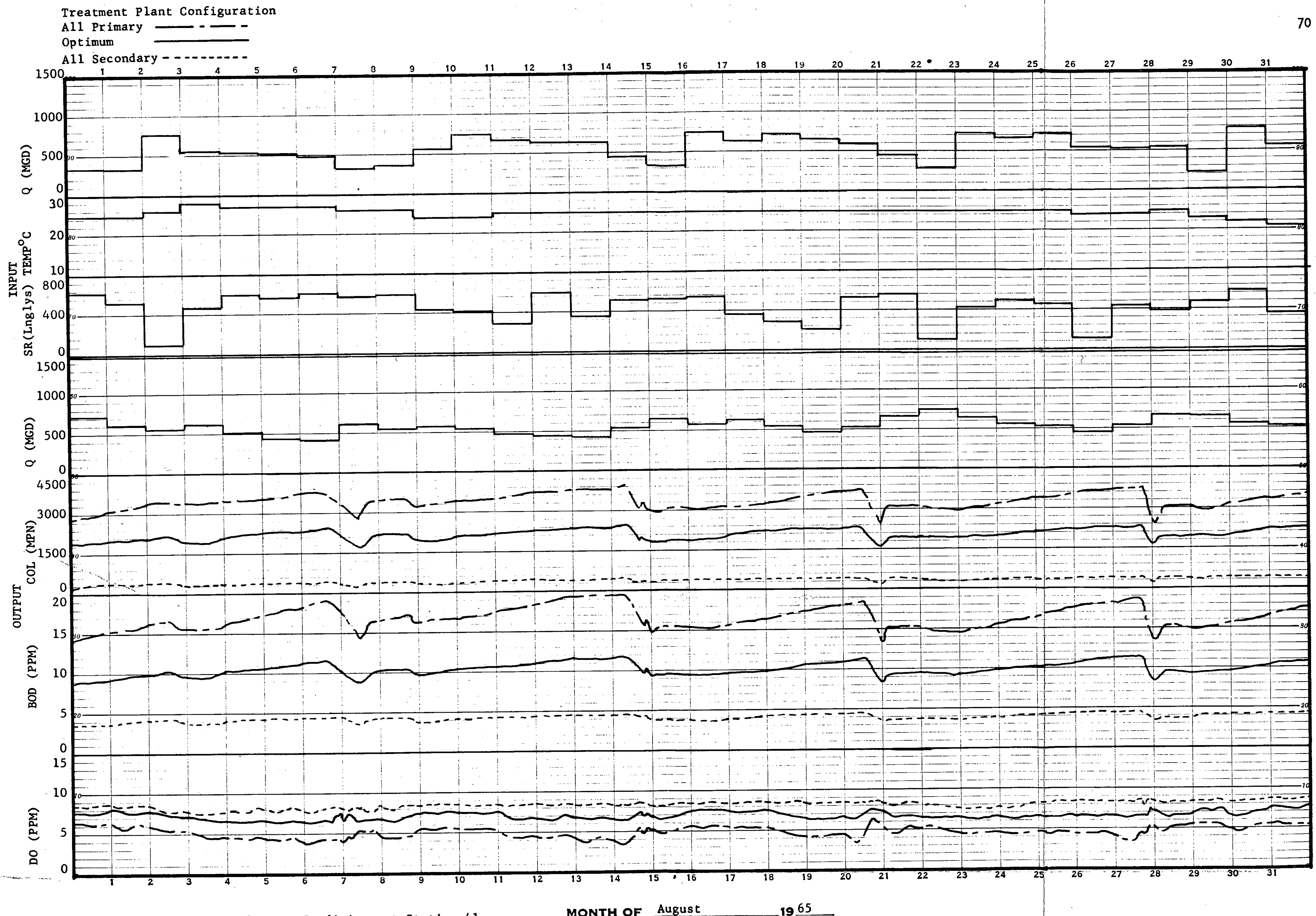
Figure 9



Simulation of River Conditions at Station 41

MONTH OF AUGUST 19 64

Figure 10



Simulation of River Conditions at Station 41

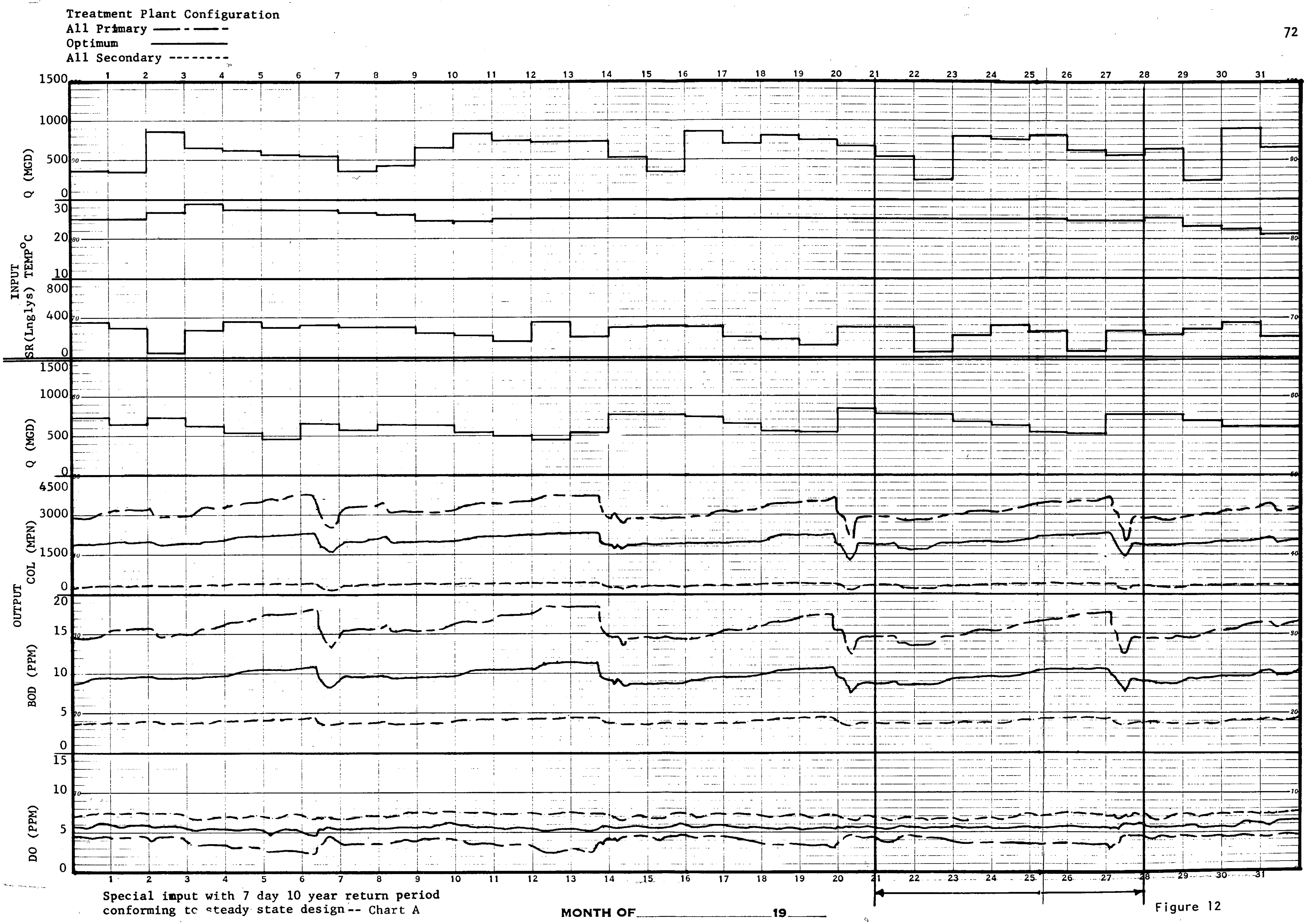
Figure 11

deficit for the critical station for the three forms of treatment (primary, optimum, and secondary) equalled 4.63 ppm, 2.94 ppm, and 1.83 ppm, respectively. At 25° C, the oxygen saturation value is 8.38 ppm, which would give corresponding DO values of 3.75 ppm, 5.44 ppm, and 6.55 ppm, respectively.

C. Comparison With Steady State Model

As shown by Figures 9, 10, and 11, the resulting conditions in the stream for the three months were generally better than those projected with the steady state model. This is interesting because, individually, each input value meets the steady state input assumption at certain times during these periods. However, the coincidence of low flow, high temperatures, and little sunlight did not occur for a long enough period of time to cause the kinds of stream conditions indicated by the steady state analysis. For example, the average DO values for July and August 1964 with only primary treatment were at a generally acceptable 5.85 ppm and 6.28 ppm respectively.

In order to make a comparison between the steady and unsteady state models, synthetic sequential input data whose averages would conform to the steady state design values were developed. August 1965 was used as a start for developing the synthetic data. The averages conforming to the steady state design values were 650 mgd for discharge, 25° C for temperature, and 200 Langleys per day for sunlight. The latter value was chosen to represent the average cloudy day. The steady state study was based on these values which were estimated by previous investigators to occur for seven days once every ten years. For the month of August 1965, the worst seven days occurring naturally appeared to be the 21st to the 27th, inclusive (see Figure 12). In constructing the first set of synthetic data, the input data for the entire month were multiplied by coefficients so that the averages for this seven day period conformed to the input values for the steady state model. It is interesting to note, however, that the lowest values for DO occurred during a high temperature period near the beginning of the month, and not during the low flow period. The lowest values of DO for the three forms of treatment (primary, optimum, and secondary) for the month were 2.34 ppm, 4.79 ppm, and 6.48 ppm respectively; for the "all secondary treatment" assumption, the lowest DO for the unsteady state model was virtually the same as that for the steady state model,but the lowest DO's for the "all primary treatment" and the "optimum treatment" assumptions were about 1.4 ppm less and 0.7 ppm less, respectively, than the steady state results.



Special input with 7 day 10 year return period conforming tc steady state design -- Chart A

Figure 12

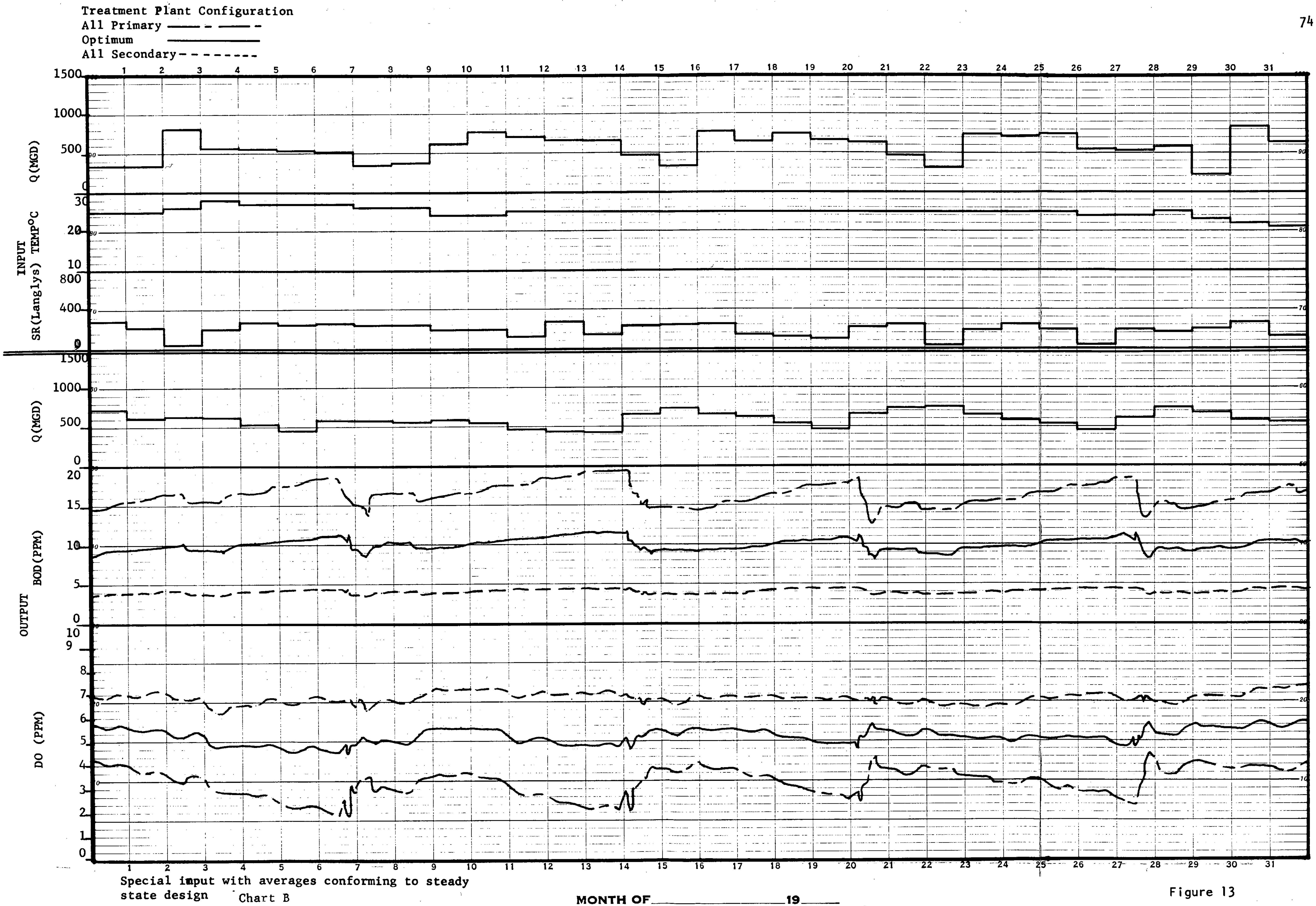
It is important to note that while with the steady state model it was sufficient to consider merely the DO deficit, the unsteady state model operations make it necessary also to consider the relationships between dissolved oxygen, the DO saturation value, and the DO deficit as they vary with water temperature. At 25° C, the steady state design temperature, a DO deficit of 3 ppm is equal to a DO of 5.38 ppm. At 28° C, however, the same deficit equals a DO of 4.92 ppm. Therefore, a true comparison of the results requires the inclusion of water temperature.

Another approach utilized for comparing steady and unsteady state models was to adjust the average input values for the entire month to the steady state design values. As shown in Figure 13, the average values of DO for the various configurations of treatment were approximately equal to the steady state values. On this figure, the coliform output has not been included and DO results have been shown to larger scale to give better representation of the variation in these values. For primary treatment, the DO range was 1.82 ppm to 4.42 ppm. With optimum treatment, this range was 4.48 ppm to 5.79 ppm, and, finally, for all secondary treatment, the range was 6.28 ppm to 7.30 ppm. This gives ranges between extremes for the three forms of treatment (primary, optimum, and secondary) of 2.60 ppm, 1.31 ppm, and 1.02 ppm, respectively. It would appear from these studies that a greater range of river conditions over a period of time may result when higher BOD loads are applied to the stream.

The lower bounds of the ranges were 1.93 ppm less than the steady state value for primary treatment, 0.96 ppm less for optimum treatment, and 0.27 less for secondary treatment. This again appears to demonstrate that worse values than the steady state value can result at times, and that the susceptibility and degree of such differences increases with higher BOD loads.

D. Sensitivity Analysis

The method used to test the sensitivity of the various model parameters in the unsteady state model was similar to that used with the steady state model (see Chapter III). Each of the river parameters recommended for design was multiplied by coefficients to individually vary them within their possible ranges of values. For this study, the analysis of temperature required that a given number of degrees were added or subtracted. The ranges are similar to those for the steady state analysis but, due to the nature of the way input data is presented to the program, it was not possible to make them identical in all cases.



Special input with averages conforming to steady state design   Chart B

Figure 13

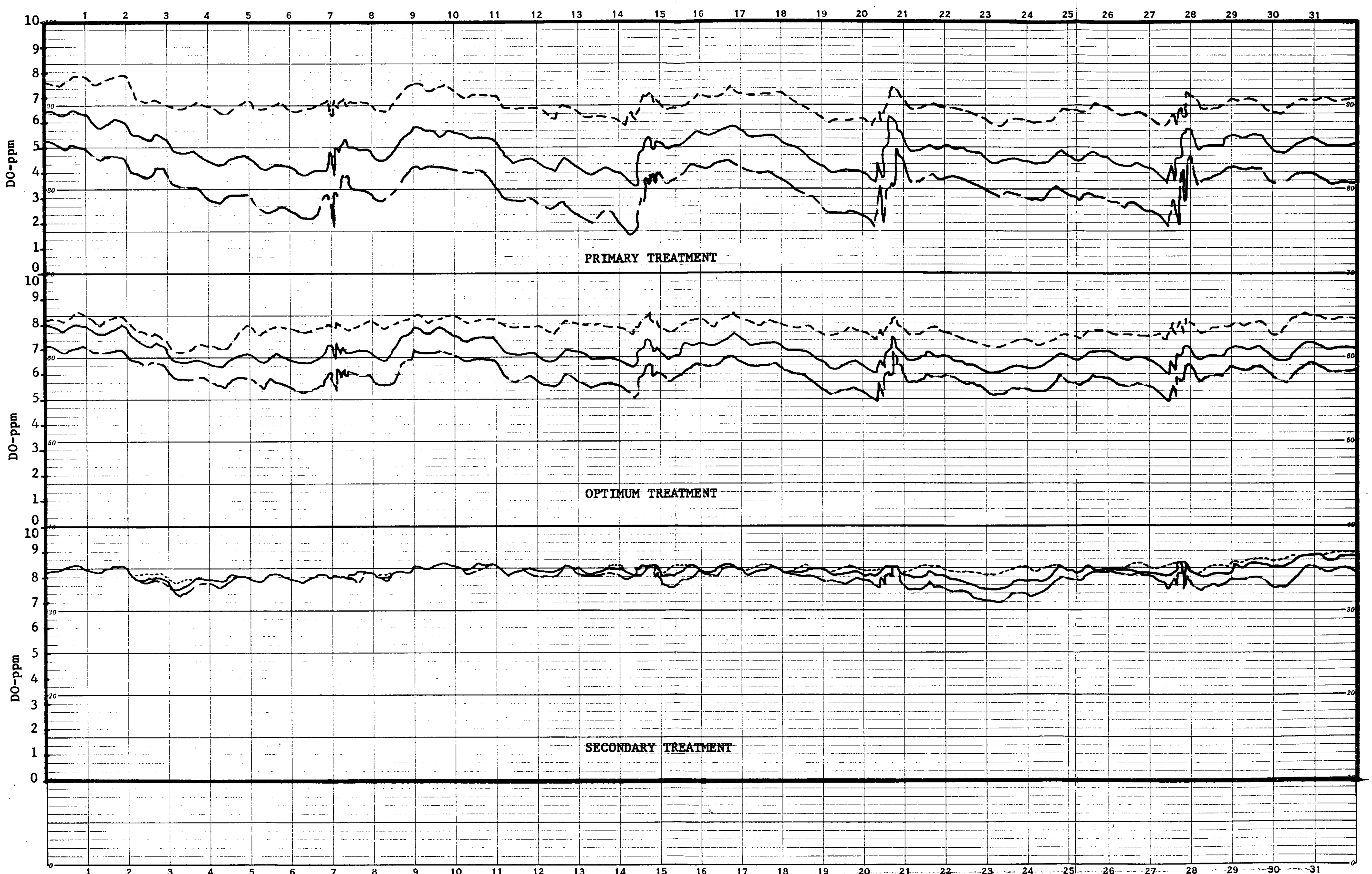
This sensitivity analysis was performed on the parameters which were assumed to vary with time. These included the deoxygenation and reaeration constants, the amount of algae activity and the water temperature. In each case, the values of the parameter being studied were altered while the values of the other parameters remained the same.

Figures 14 and 15 show the results of the sensitivity analysis of the deoxygenation (k_1) and reaeration (k_2) constants. The most apparent indication of this analysis is that with lower levels of treatment (allowing greater BOD loads to enter the stream), the DO is prone to much greater variation. Thus, design values chosen for these stream parameters are very critical. Another way of looking at this is that if the planner hopes to use the stream's assimilative capacity to adjust for lower forms of treatment, it is likely that even if the average values of DO are acceptable, worse conditions will probably occur for a portion of the time.

Although the concept has not been thoroughly investigated, it is suggested that the analysis may indicate some merit in considering BOD concentration as a criterion for receiving waters. In the above case, the BOD values for the critical river station during the study period for three levels of treatment (primary, optimum, and secondary) averaged approximately 16 ppm, 10 ppm and 4 ppm respectively.

In the sensitivity analysis, a range of values of the parameter a implies that the amount of algae in the stream would either increase or decrease, while the ratio of photosynthesis to respiration would remain the same. In other words, this method assumes an increase in the numbers of algae while their individual synthesis rates remain unchanged. Due to the nature of the oxygen sag equation and because for this study algae concentrations were assumed to be independent of treatment plant levels, different algae concentrations would produce about the same effect on the stream regardless of treatment levels.

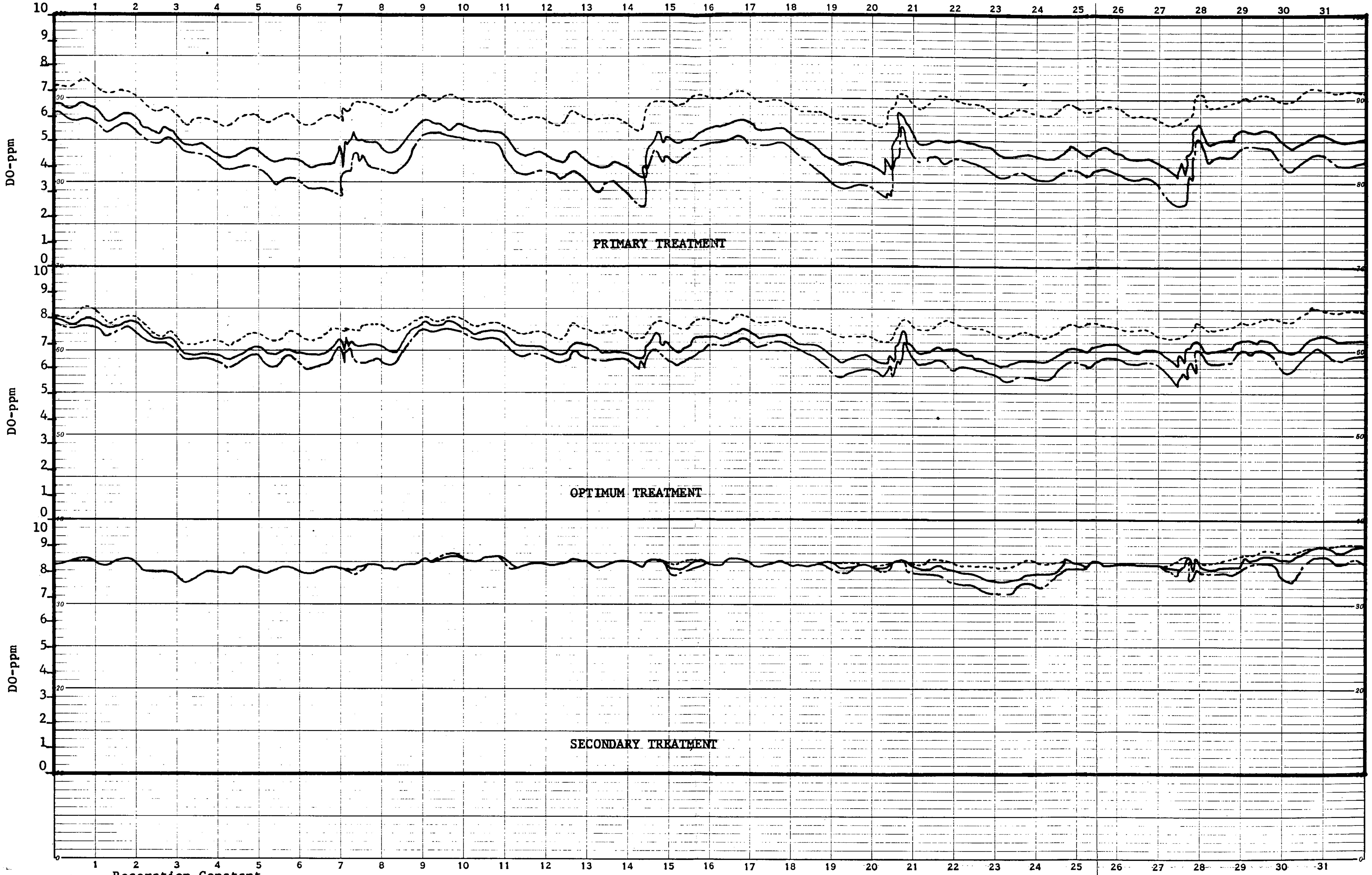
Figure 16 demonstrates the effect of different algae concentrations on the study stream. It is reiterated that the methodology assumes supersaturation cannot occur, so that excess oxygen vents to the atmosphere. This is the reason why, for this stream, which obtains a net gain in dissolved oxygen from algae, the DO values for different forms of treatment at high algae concentrations is not greatly different. It is pointed out that the variations caused by different algae concentrations may be quite different for another stream. Using



Deoxygenation Constant
0.25 = - - - - -
$Z3=k_1=$ 0.50 = ———

Sensitivity Analysis
MONTH OF AUGUST 19 65

Figure 14



θ.010 - 0.027 = — — —
Z4=k2=θ.030 - 0.080 = ————
0.10 - 0.267 = - - - - - -

Figure 15

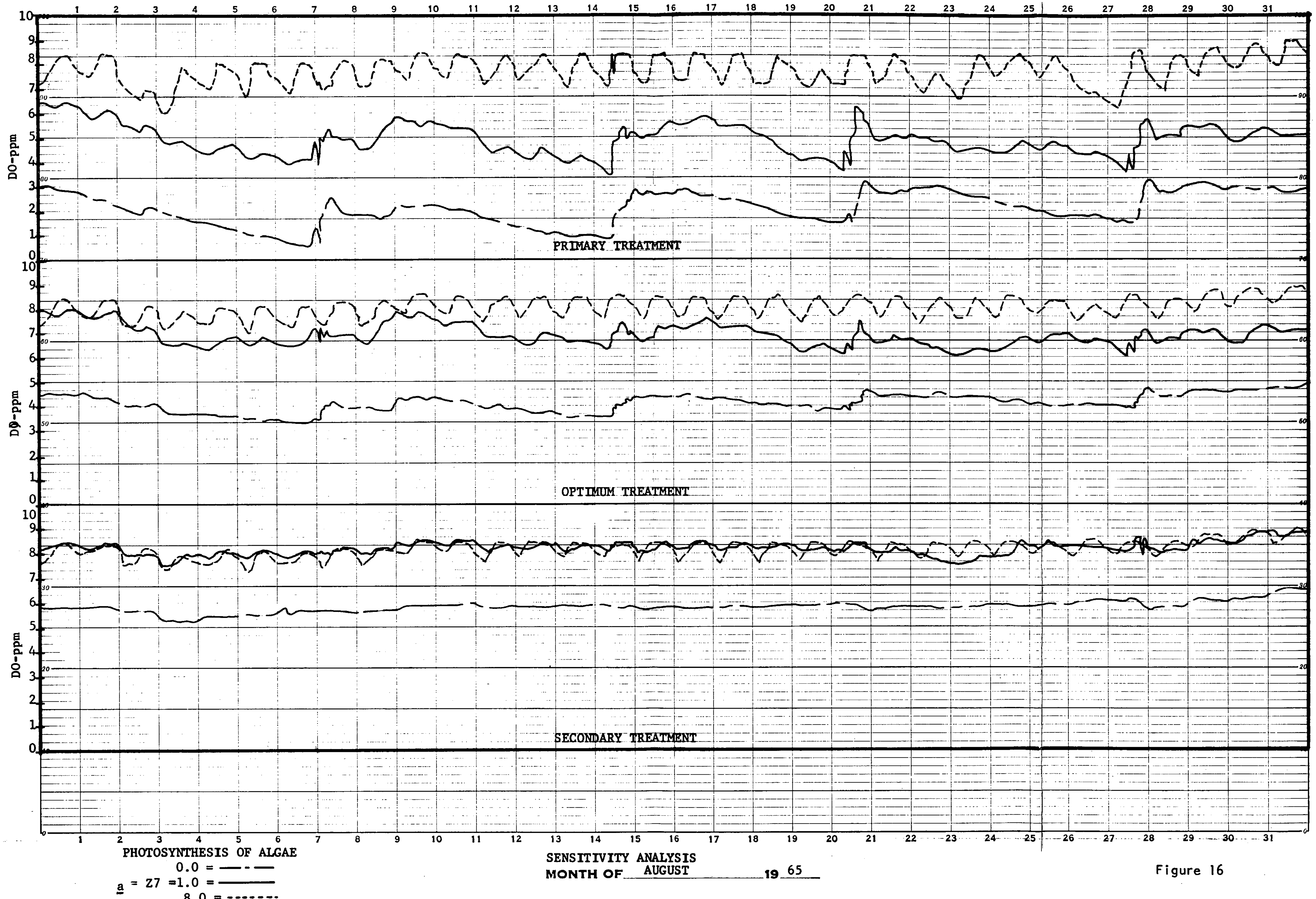

DO-ppm
PRIMARY TREATMENT
DO-ppm
OPTIMUM TREATMENT
DO-ppm
SECONDARY TREATMENT
PHOTOSYNTHESIS OF ALGAE
0.0 =
a = Z7 =1.0 =
8.0 =
SENSITIVITY ANALYSIS
MONTH OF AUGUST 19 65

Figure 16

relationships for this stream previously developed between sunlight intensity and the action of algae (see Chapter IV) and the photosynthesis-respiration ratios reported in the literature, it was found that many times the normal algae concentration would have to occur in this stream before nighttime respiration would become a problem in reducing DO levels.

Figure 17 shows the effect of varying the temperature of the stream on the resultant DO at the critical station. The procedure was to add or subtract a given number of degrees to each daily temperature for the entire study period. For the ranges studied, the changes in temperature produced about the same average change in DO for the different levels of treatment. A 3° C change in temperature for the stream produced a fairly consistent change in DO of nearly 1 ppm at the critical station. This fact stresses the need for accurate data on temperature profiles for a stream before an effective treatment plant system can be designed.

E. Effect of Statistical Variations

Although the time varying model described herein produces continuous estimates of river conditions which demonstrate the true nature of their variations, it is not claimed that they are the exact values of DO or other quality characteristics which would occur. It is recognized that this model is deterministic and, therefore, does not consider either the random variations in the input or the random variations in local environmental conditions.

Thomann and Sobel (18) state that a good forecasting scheme must contain an adequate representation of output which may be caused by combinations of periodic, transient, and random phenomena. O'Connor and Thomann (21) recognize in their work with estuaries, that the various processes are composed of largely deterministic components on which are superimposed random elements. Thomann (2) made estimates of the random variations of dissolved oxygen in the Delaware estuary and determined a standard deviation of 0.85 ppm. Thayer and Krutchkoff (55) determined a variation of about 0.3 ppm above and below predicted DO's, which were found in laboratory analyses. For tests with values on the Sacramento River, they determined a range of about 0.5 ppm between the mean and the 10% lower confidence limit. Kothandaraman and Ewing (56) made sensitivity analyses using a steady state model. They determined the probability distributions for the model rate parameters k_1 and k_2. With these results, a band of values for DO was created having limits up to 1 ppm above and below the most probable values.

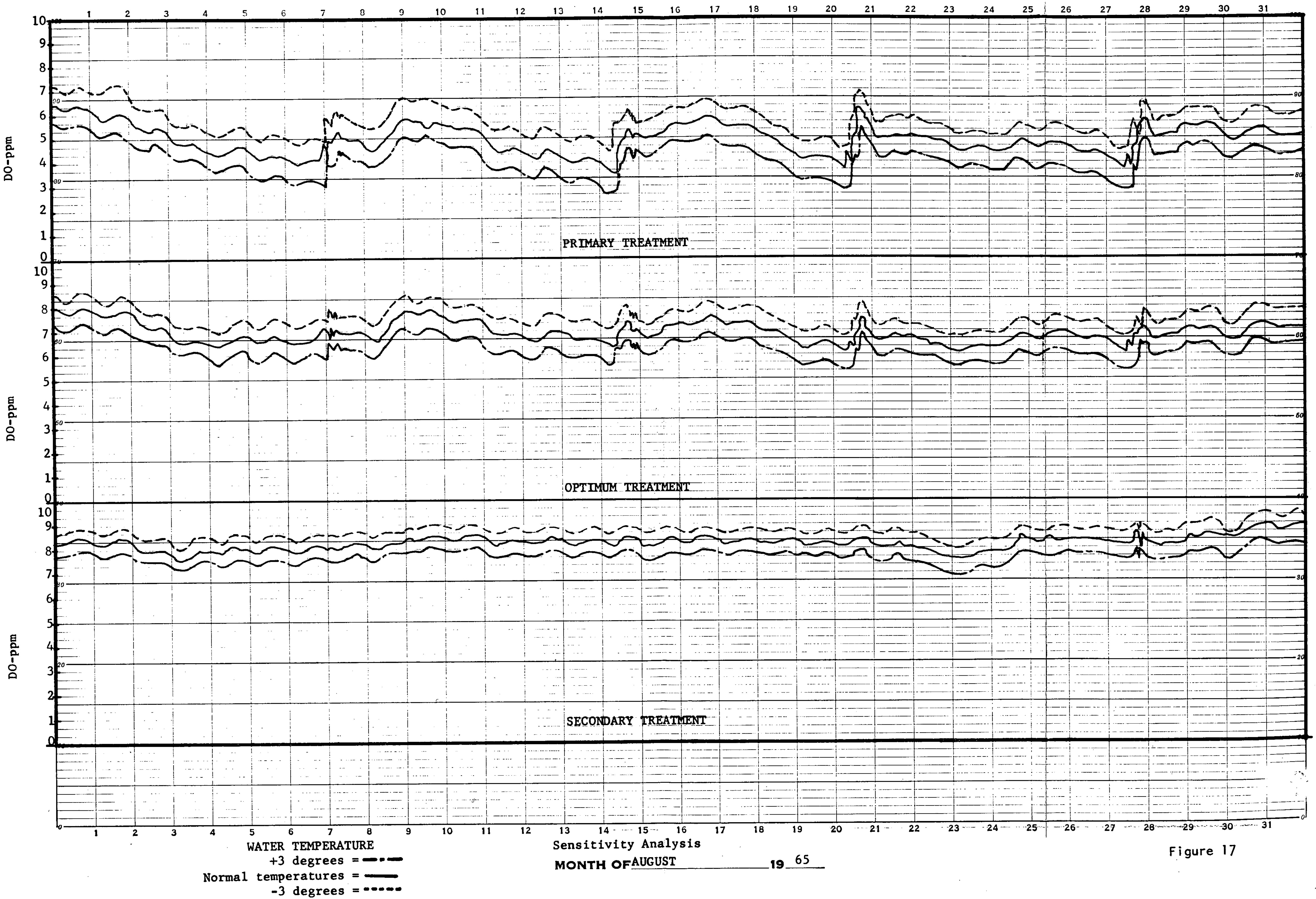

DO-ppm
PRIMARY TREATMENT
OPTIMUM TREATMENT
SECONDARY TREATMENT
WATER TEMPERATURE
+3 degrees =
Normal temperatures =
-3 degrees =
Sensitivity Analysis
MONTH OF AUGUST 19 65

Figure 17

In light of the above, it would appear reasonable to expect that the results for DO obtained for the time varying model represent the most probable values, while the actual values of DO would range above and below these values. A tentative judgment is that the differences between probable and actual values would be less than 1 ppm for at least 90 percent of the time. Additional studies comparing predicted and actual values would be needed to establish a firmer relationship.

An extension of the time varying model could superimpose additional statistical fluctuations on the computed values. These fluctuations could be estimated by generating stochastic input, or by adding random components to the output. The use of a Monte Carlo procedure could be one approach to generating these values randomly.

CHAPTER VI

CURRENT SAMPLING PRACTICES BY NEW ENGLAND REGULATORY AGENCIES

A. Introduction

When this project was originally conceived it was planned to collect all available information pertaining to the data collection and processing programs of the water pollution control agencies of the New England states. It was thought that this information could be used to make a comprehensive study to establish a scientific basis for judging the economic efficiency of data collection programs. It was planned to determine the importance of various field and laboratory data in making studies leading to: 1) selection of required performance levels for waste treatment plants; 2) selection of objective qualities for watercourses; and 3) administrative regulations for reports of participants in water resources operational programs. An ultimate objective for this work was to be able to indicate 1) for portions of such programs where field and laboratory techniques are well developed, the extent of sampling needed for useful results and 2) for portions of such programs where field and laboratory techniques are not well developed, the relative importance that ought to be placed on researching and developing such techniques.

B. Current Sampling Practices

The discussions with the pollution control agencies were quite disappointing in that they did not produce the amount and/or kinds of information necessary to make the planned analyses. Generally, all of the New England states have data collection programs, but most of them have not been in existence long enough to adequately study their performance. These programs have been set up more or less intuitively with sampling stations at the "trouble spots." These locations have usually been specified by complaints, which would indicate that they would tend to be in the most populated areas. The frequency of sampling is quite variable and is often limited to only a few times a year because of insufficient personnel. Depending on their available laboratory facilities, these samples may be either analyzed by the agency or contracted to a private laboratory. The use of automatic samplers by the states is quite new and in most cases are not being used or are just used experimentally.

It should be pointed out that several government and private agencies have conducted special surveys which have been quite extensive. Some rivers where such studies have been made include: the Merrimack in New Hampshire and Massachusetts, the Penobscot in Maine, and the Hudson in New York. In these cases a great deal of data has been obtained and, in the case of the Merrimack, it has been possible to make a comparison of two different studies in this report.

The only NEIWPCC member state which currently appears to have a program directed toward an optimum sampling program is New York (57). New York currently has established a network of more than 100 stations which were selected as being representative of the general quality of the water in a given basin or as being indicative of the effect of upstream discharges on water quality. A total of 33 different parameters are evaluated to demonstrate the condition of a receiving body of water. The cost for a single sample is from $150 to $200--which includes the transportation from the sampling site to the lab. Several automatic samplers have been installed and seem to provide a worthwhile supplement for the periodic sampling data. For the manual program, a sampling frequency of once per month has been adopted. This frequency may be changed if conditions demonstrate that more or less frequent sampling is necessary. As with the other states, New York's data collection program has not been operated long enough for an evaluation of its performance by the authors.

For the New York program, much use is made of data processing systems. All water quality data is placed on magnetic tape and is therefore readily retrievable for processing by computer to produce stream analyses, raw data listings, summary listings, statistical evaluations, and reports. New York has also been attempting to analyze this data from a statistical standpoint (58). They have not as yet found any theoretical statistical distribution which is applicable. They have, however, been studying the applicability of control charts to administer water quality control programs. Although these studies have some promise, the results are still inconclusive and more data will be needed before a complete evaluation can be made.

In view of the above, the work for this report has concentrated on methodologies for utilizing data rather than a quantitative evaluation of the cost and efficiency of data collection and processing. The models discussed in the previous chapters have been used to analyze the effect of assuming various values of stream purification parameters and other factors on the cost and effectiveness of water pollution control.

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APPENDIX I

INPUT - OUTPUT DATA DESCRIPTIONS[a]

Flow Characteristics and Water Quality at River Station I

Q(I) River flow in MGD

T(I) Time in days

E(I) BOD in ppm

F(I) Oxygen deficit in ppm

DO(I) Dissolved oxygen in ppm

H(I) Coliform count in MPN/100 ml

V(I) Chlorides in ppm

WI(I) Water quality index for water supply

WR(I) Water quality index for recreation

Flow Characteristics and Water Quality at End of Time Interval (Not Printed)

TZ(N) = Time in days

EZ(N) = BOD in ppm

FZ(N) = Oxygen deficit ppm

DOZ(N) = Dissolved oxygen in ppm

"Z" Data (Hydraulic and Stream Purification Constants Associated with River Location)

Z1(I), Z2(I) Constants for velocity expression, in which velocity equals

Appendix I -- Continued

$Z_1 Q^{Z_2}$; velocity is miles per day and Q is MGD

Z3(I) Deoxygenation constant in ppm/day

Z4(I) Reaeration constant in ppm/day

Z5(I) Rate constant for settling out of BOD to bottom deposits in ppm/day

Z6(I) Rate constant for addition of BOD to overlying water from bottom deposits in ppm/day

Z7(I) Constant to alter oxygen production by algae in ppm/day

Z8(I) Constant defining the die-off rate of coliform bacteria in fraction of coliforms per day

Z9(I) River station in miles

"CZ" Data (Temperature Coefficients to Vary "Z" Data with Temperature)

CZ3(MT) Deoxygenation coefficient

CZ4(MT) Reaeration coefficient

OSAT(MT) Oxygen saturation value

"ALA Data" (Data Varying with Solar Radiation)

ALA(NSR,NTN) Constant for rate of production of dissolved oxygen by algae through photosynthesis in ppm/day NSR = Daily solar radiation (Langleys) value /10 NTN = time period during day.

"NT" Data (Input Data Varying Daily)

SR(NT) Total solar radiation in Langleys

TM(NT) Average river water temperature degrees Centigrade

RQ(NT) Average daily flow at upstream station I = 1

Appendix I--Continued

"ZZ" Data (Significant Internally Computed Data - Not Printed)	
ZZ3(N)	Value of Z3 as modified by CZ3
ZZ4(N)	Value of Z4 as modified by CZ4
ZZ7(N)	Value of ALA(NSR, NTN) as modified by Z7
CQ(NTC)	Constant for modification of Q as flow moves downstream to compensate for varying Q

"G" Data (physical and population data for communities)	
G1(J)	Maximum feasible off-river water supply in MGD
G2(J)	Community area in square miles (or drainage area in sq. mi.)
G3(J)	Ratio of average infiltration to 1000 gal. per acre per day
G4(J)	Ratio of infiltration to average infiltration for period of study
G5(J)	DO for saturation of raw sewage discharge
G6(J)	Chlorides in off-river water supply in ppm
G14(J)	Residential population
G15(J)	Employees of commercial enterprises
G16(J)	Employees of fabrication industries
G17(J)	Employees of wet-process industries

"Y" Data (economic data for communities)	
Y1(J)	Ratio of residential water supply demand to 90 gal. per capita per day
Y2(J)	Ratio of commercial water supply demand to 250 gal. per employee per day

Appendix I -- Continued

"Y" Data -- Continued

Y3(J) Ratio of fabrication industry water demand to 250 gal. per employee per day

Y4(J) Ratio of wet-process industry water demand to 900 gal. per employee per day

Y5(J) Ratio of public water supply demand to 25 gal. per capita per day

Y6(J), Y7(J) Maximum weekly municipal water supply demand equals $Y_6Q^{Y_7}$ where Q is average water supply demand

Y8(J) Ratio of residential average sewage discharge to 75 gal. per capita per day

Y9(J) Ratio of commercial average sewage discharge to 225 gal. per employee per day

Y10(J) Ratio of fabrication industry average sewage discharge to 235 gal. per employee per day

Y11(J) Ratio of wet-process industry average sewage discharge to 800 gal. per day.

Y12(J) Ratio of public average sewage discharge to 20 gal. per capita per day

Y13(J) Y14(J) Maximum daily municipal sewage discharge (without infiltration equals $Y_{13}Q^{Y_{14}}$ where Q is average discharge without infiltration

Y15(J) Ratio of expected BOD to 0.2 pounds per resident per day for all sewage except wet-process industrial waste.

Appendix I--Continued

Y16(J) Ratio of expected BOD to 0.0025 pounds per gallon of wet-process industrial waste

Y17(J) Ratio of expected coliforms in raw sewage to 80 billion MPN per capita per day

Y18(J) Ratio of expected coliforms in raw sewage to 0.07 pounds per capita per day

Y19(J) Factor to adjust efficiency of BOD removal for operation of sewage treatment plant

Y20(J) Factor to adjust efficiency of DO additions for operation of sewage treatment plant

Y21(J) Factor to adjust efficiency of coliform removals for operation of sewage treament plant

Y42(J), Y43(J), Y44(J), Y45(J) Constants used to define annual cost (in \$) of on-river water supply (Q_h in MGD), where $C_H = Y_{42}Q_H^{Y43} + Y_{44}Q_H^{Y45}$ The value of C_H may be the estimated actual value or a relative value suitable for making a comparison with an off-river water supply.

Y46(J), Y47(J), Y48(J), Y49(J) Constants used to define annual cost of off-river water supplies (Q_B in MGD), Where: $C_B = Y_{46}Q_B^{Y47} + Y48\,Q_B^{Y49}$ The value of C_B may be computed to reflect credits or penalities for intangible values entering into judgment of off-river supplies.

"A" Data (administrative assumptions for communities)

A1(J) Defines the extent to which the average municipal water supply demand (Q_O) will be met by on-river supply (Q_H) and off-river

Appendix I -- Continued

"A" Data (administrative assumptions for communities)

Supplies (Q_B), as follows:

If 0 A_1 1.0, $Q_B = A_1 Q_O$

If $A_1 = 2.0$, split between Q_B and Q_H is determined by analysis using standard subroutine.

A2(J) Define whether there is to be a municipal sewage treatment plant discharging to the river, as follows:

If $A_2 = 0$, $Q_S = 0$ and $Q_D = 0$

If $A_2 = 1.0$, Q_S and Q_D are defined by standard subroutines

A3(J) Defines the degree of sewage treatment; A_3 equals the fraction of BOD to be removed, and this in turn fixes the quality of the effluent in terms of BOD, DO deficit and coliforms.

"KD" Input Data

Table defining coefficients in equations evaluating treatment plant efficiencies

Miscellaneous Input Data (for computer processing)

NSTA Total number of river stations

NCOM Total number of communities

NOPT Two digit number defining path for computer processing

NDAY Total number of consecutive study days

ZN Exponent controlling pattern of flow additions and subtractions in river routing procedure

Appendix I--Concluded

TS - Time last flow reaches downstream station, before start of study period.

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[a] Using notation for oomputer statements in Fortran, in which I is river station number and J is community number; N = number of time interval, NT = number of the study day, MT = study day temperature, NTC = number of time interval when flow passes most upstream station

APPENDIX II

SUMMARY OF EQUATIONS AND PROCEDURES FOR FUNCTIONS

FUNCTION -1-- TIME AT RIVER STATION (I) -- DAYS, Q AT RIVER STATION (I) -- MGD

$$Q_i = Q_k + (M_k - M_i)\ CQ_n\ (M_i / LN)^{ZN}$$

$$T_i = T_k + (M_k - M_i) / z_1 \left[(Q_k + Q_i) / 2\right]^{z_2}$$

K is first station upstream of I

Input data: z_1, z_2, ZN, LN, M_k (same as z_9 at Sta. K), M_i (same as z_9 at Sta. I)

Determined previously: T_k, Q_k, CQ_n

Output: T_i, Q_i

FUNCTIONS - 2A; 2B; 2C; 2D - BOD AND DO AT RIVER STATION (I) OR (N) - PPM

$$E_i = \left[E_k - (z_6 / 2.3) / (zz_3 + z_5)\right] X\ 10^{-(zz_3 + z_5)\ (T_i - T_k)} + (z_6 / 2.3) / (zz_3 + z_5)$$

$$F_k = OSAT_{Mt} - DO_k$$

$$F_i = \left[zz_3 / (zz_4 - zz_3 - z_5)\right] \left[E_k - (z_6 / 2.3) / (zz_3 + z_5)\right]$$
$$X \left[10^{-(zz_3 + z_5)\ (T_i - T_k)} - 10^{-zz_4\ (T_i - T_k)}\right]$$
$$+ (zz_3 / z_4) \left[(z_6 / 2.3) / (zz_3 + z_5) - zz_7 / 2.3\ z_3\right]$$
$$X \left[1 - 10^{-zz_4\ (T_i - T_k)}\right]$$
$$- F_k\ X\ 10^{-zz_4\ (T_i - T_k)}$$

$$DO_i = OSAT_{Mt} - F_i$$

Appendix II--Continued

K is first station upstream of I

Input data: OSAT $_{Mt}$, z_5, z_6

Determined previously: E_k, DO_k, T_k, T_i, zz_3, zz_4, zz_7

Output: E_i, F_i, DO_i

FUNCTION -5- DEOXYGENATION REAERATION AND PHOTOSYNTHETIC CONSTANTS

AT RIVER STATION I OR N

$t = TM_{Nt}$

$zz_3 = z_3 \text{ X } CZ_{3_t}$

$zz_4 = z_4 \text{ X } CZ_{4_t}$

$Sr = Sr_{Nt} \ / \ 10$

$Td = T_k - Nt$

$zz_7 = ALA_{Sr,Td} \text{ X } z_7$

K is first station upstream of I

Nt is day number

Fixed Input data: z_3, z_4, z_7

Daily input data: SR, TM

Input data from matrices: CZ_3, CZ_4, ALA

Determined previously: T_k, Nt

Output: zz_3, zz_4, zz_7

Appendix II--Continued

FUNCTION 6 -- COLIFORMS AT RIVER STATION (I) -- MPN/100 ML

$H_cQ_c = N_c$ for inflow at most upstream "control" section

$H_1Q_1 = N_1$ for first sewage discharge Q_1 below control section

$H_kQ_k = N_k$ for last sewage discharge Q_k before reaching Station I

$n_c = N_c \times 10^{-Z_8 (T_i - T_c)}$

$n_1 = N_1 \times 10 (T_i - T_1)$, if $(T_i - T_1) \leq 0.5$

or $n_1 = N_1 \times 5 \times 10^{-z_8 (T_i - T_1 - 0.5)}$, if $(T_i - T_1) > 0.5$

$n_2, \ldots \ldots n_k$ are each computed as for n_1

Then:

$H_i = (n_c + n_1 + \ldots \ldots n_k) / Qi$

Input data: z_8, H_c, Q_c, T_c

Determined previously: $H_1 \ldots H_k$; $Q_1 \ldots Q_i$; $T_1 \ldots T_i$

Output: H_i

FUNCTION 11 -- QUALITY OF MIXTURE OF TWO FLOWS AT RIVER STATION (I) -- BOD (PPM), DO DEFICIT (PPM), COLIFORMS (MPN/100 ML), CHLORIDES (PPM)

$E_i = (E_kQ_k + E_jQ_j) / (Q_k + Q_j)$

$F_i = (F_kQ_k + F_jQ_j) / (Q_k + Q_j)$

$H_i = (H_kQ_k + H_jQ_j) / (Q_k + Q_j)$

$V_i = (V_kQ_k + V_jQ_j) / (Q_k + Q_j)$

Q_k is river flow just upstream of point of sewage discharge Q_j

Q_i is river flow just downstream of Q_j (and $Q_i = Q_k + Q_j$)

Input data: None

Determined previously: Q_k, E_k, F_k, H_k, V_k;Q_j, E_j, F_j, H_j, V_j

Output: E_i, F_i, H_i, V_i

Appendix II--Continued

FUNCTION 12 -- WATER QUALITY RATING AT RIVER STATION (I) FOR MUNICIPAL WATER SUPPLY -- INDEX NUMBER

WI_i is given a base value of 0.00

0.75 is credited for meeting each of the following criteria:

$E_i \leq 4$

$F_i \leq 4$

$H_i \leq 5000$

$V_i \leq 250$

Maximum value for WI_i is 3.00

Input data: None

Determined previously: E_i, F_i, H_i, V_i

Output: WI_i

FUNCTION 13 -- WATER QUALITY RATING AT RIVER STATION (I) FOR RECREATION -- INDEX NUMBER

WR_i is given a base value of 0.00

1.00 is credited for meeting each of following sets of criteria:

$E_i \leq 50$, $F_i \leq 7$, and $H_i \leq 20{,}000$

$E_i \leq 15$, $F_i \leq 4$, and $H_i \leq 2{,}000$

$E_i \leq 10$, $F_i \leq 3$, and $H_i \leq 1{,}000$

Maximum value for WR_i is 3.00

Input data: None

Determined previously: E_i, F_i, H_i

Output: WR_i

Appendix II--Continued

FUNCTION 15-- AVERAGE MUNICIPAL WATER SUPPLY DEMAND OF COMMUNITY (J) -- MGD

$QO_j = (90\ n_1 Y_1 + 250\ n_2 Y_2 + 250\ n_3 Y_3 + 900\ n_4 Y_4 + 25\ n_1\ Y_5)$ / 1,000,000

Input data: n_1 (same as g_{14} for J), n_2 (g_{15}), $n_3 (g_{16})$, n_4 (g_{17}) Y_1, Y_2, Y_3, Y_4, Y_5

Determined previously: None

Output: QO_j

FUNCTION 16 -- AVERAGE OFF-RIVER MUNICIPAL WATER SUPPLY OF COMMUNITY (J) -- MGD

If $a_1 = Q$, $QB_j = 0$

If $a_1 > 0$ and $a_1 \leq 1.0$, $QB_j = a_1\ QO_j$

If $a_1 = 2$, procedure follows:

$$QB_j = 0.05\ pQO_j$$

$$QH_j = QO_j - QB_j$$

$$CH_j = Y_{42}\ QH_j^{\ Y43} + Y_{44}\ QH_j^{\ Y45}$$

$$CB_j = Y_{46}\ QB_j^{\ Y47} + Y_{44}\ QB_j^{\ Y49}$$

$$CO_j = CH_j + CB_j$$

QB_j is determined for minimum CO_j by comparing values for p = 0, 1,. . . . 20 and for $QB_j \leq g_1$

Input data: a_1, Y_{42}, Y_{43}, Y_{44}, Y_{45}, Y_{46}, Y_{47}, Y_{48}, Y_{49}, g_1

Appendix II--Continued

FUNCTION 19 -- Continued

Input data: g_2, g_3, g_4; Y_{13}, Y_{14}

Determined previously QS_j

Output: QD_j

FUNCTION 26 -- QUALITY OF RAW SEWAGE OF COMMUNITY (J) -- BOD (PPM), DO DEFICIT (PPM), COLIFORMS (MPN/100 ML), CHLORIDES (PPM)

If $QS_j = 0$, dummy values are as follows:

$$ES_j = 0,\ FS_j = 0.\ HS_j = 0,\ VS_j = 0$$

If $QS_j > 0$, equations follow:

$$ES_j = (0.2\ n_1\ Y_{15} + 2.0\ n_4\ Y_{11}\ Y_{16})\ /\ 8.34\ QS_j$$

$$FS_j = g_5$$

$$HS_j = 2100\ n_1\ Y_{17}\ /\ QS_j$$

$$VS_j = (0.5\ V_{ij}\ X\ QH_j + g_6\ QB_j)\ /\ (QH_j + QB_j) + 0.0085\ n_1\ Y_{18}\ /\ QS_j$$

Input data: n_1 (same as g_{14} for J), n_4 (g_{17}); g_5, g_6; Y_{11}, Y_{15}, Y_{16}, Y_{17}, Y_{18}

Determined previously: QS_j, V_{ij}, QH_j, QB_j

Output: ES_j, FS_j, HS_j, VS_j

FUNCTION 16 -- Continued

Determined previously: QO_j

Output: QB_j

FUNCTION 17 -- MAXIMUM WEEKLY ON-RIVER MUNICIPAL WATER SUPPLY OF COMMUNITY (J) -- MGD

$$QI_j = (QH_j \ / \ QO_j) \quad (Y_6 \ QO_j^{\ Y_7})$$

Input data: Y_6, Y_7

Determined previously: QO_j, QH_j

Output: QI_j

FUNCTION 18 -- AVERAGE SEWAGE DISCHARGE OF COMMUNITY (J) -- MGD

If $a_2 = 0$, $QS_j = 0$

If $a_2 = 1$, equation follows:

$$QS_j = (75 \ n_1 \ Y_8 + 225 \ n_2 \ Y_9 + 235 \ n_3 \ Y_{10} + 800 \ n_4 \ Y_{11} + 20 \ n_1 \ Y_{12}) \ / \ 1{,}000{,}000 \quad + 0.64 \ g_2 g_3$$

Input data: n_1 (same as g_{14} for J), n_2 (g_{15}), n_3 (g_{16}) n_4 (g_{17}); Y_8, Y_9, Y_{10}, Y_{11}, Y_{12}; g_2, g_3,; a_z

Determined previously: None

Output: QS_j

FUNCTION 19 -- MAXIMUM DAILY SEWAGE DISCHARGE OF COMMUNITY (J) -- MGD

If $a_2 = 0$, $QD_j = 0$

If $a_2 = 1$, equation follows:

$$QD_j = Y_{13} \ (QS_j - 0.64 \ g_2 \ g_3)^{Y_{14}} + 0.64 \ g_2 \ g_3 \quad g_4$$

Appendix II--Concluded

FUNCTION 27 --QUALITY OF TREATED SEWAGE OF COMMUNITY (J) -- BOD (PPM), DO DEFICIT (PPM), COLIFORMS (MPN/100 ML), CHLORIDES (PPM)

If $QD_j = 0$, dummy values are as follows:

$$ED_j = 0,\ FD_j = 0,\ HD_j = 0,\ VD_j = 0$$

If $QD_j > 0$, equations follow:

$$ED_j = (1.0 - d_1)\ Y_{19}^{ES_j}$$

$$FD_j = d_2\ g_5\ Y_{20}$$

$$HD_j = 5{,}000\ d_3\ Y_{21}$$

$$VD_j = (V_{ij} \times QH_j + g_6\ QB_j) / (QH_j + QB_j) + 0.0085\ n_1\ Y_{18} / QS_j$$

In the above equations, d_1, d_2, and d_3 are obtained from a fixed input table and depend upon the value of a_3 (given in the input data or generated by special routine)

Input data: n_1 (same as g_{14} for J); g_5, g_6; Y_{18}, Y_{19}, Y_{20}, Y_{21}; a_3

Determined previously: QS_j, V_{ij}, QH_j, QB_j, ES_j

Output: ED_j, FD_j, HD_j, VD_j

APPENDIX III COMPUTER PROGRAM STATEMENTS CORRESPONDING TO FUNCTIONS

Function Description	Call Statement in Main Program or Subprogram	Arguments of Call Statement	Title of Subprogram with Instructions for Evaluation
Function 1 - Time at river sta. (I)-days; at river sta. (I) CFS	CALL F1T (I)	I = river station number	SUBROUTINE F1T (I)
Function 2A - BOD and DO in ppm at river sta. when no river stations have passed since completion of last time interval	CALL F1EF (I,N)	I = river station number N = time interval number	SUBROUTINE F1EF (I,N)
Function 2B - BOD and DO in ppm at end of time interval when no time intervals have elapsed since last river station	CALL F2EF	I = river station number N = time interval number	SUBROUTINE F2EF (I,N)
Function 2C - BOD and DO in ppm at end of time interval when two or more time intervals will elapse between river stations	CALL F3EF (I,N)	I = river station number N = time interval number	SUBROUTINE F3EF (I,N)
Function 2D - BOD and DO in ppm at river sta. when two or more river stations will be passed between time intervals	CALL F4EF (I,N)	I = river station number N = time interval number	SUBROUTINE F4EF (I,N)

Function Description	Call Statement in Main Program or Subprogram	Arguments of Call Statement	Title of Subprogram with Instructions for Evaluation
Function 5 - Time changing of river parameters; reaeration constant, deoxygenation constant and constant for oxygen production by algae	CALL F5A1 (I,N)	I = river station number N = time interval number	SUBROUTINE F5A1 (I,N)
Function 6 - Coliforms at River Station (I) - MPN/100 ml.	CALL F6H (I,J,JST 6)	I = river station number J = community number for first sewage discharge upstream of Station I. JST6 = river station number for first sewage discharge upstream of station I	SUBROUTINE F6H (I,J, JST6)
Function 11 - Quality of mixture of two flows at River station (I) - BOD (ppm), Do deficit (ppm), Coliforms (MPN/100 ml.), Chloirdes (ppm)	CALL F11P (I,J)	I = river station number J = community number for sewage discharge mixing with river flow at station I	SUBROUTINE F11P (I,J)
Function 12 - Water Quality Rating at river station (I) for municipal water supply (index ranging from 0 to 3)	CALL F12WI (I)	I = river station number	SUBROUTINE F12WI (I)
Function 13 - Water quality rating at river station (I) for recreation (index ranging from 0 to 3)	CALL F13WR (I)	I = river station number	SUBROUTINE F13WR (I)

Function Description	Call Statement in Main Program or Subprogram	Arguments of Call Statement	Title of Subprogram with Instructions for Evaluation
Function 15 - Average municipal water supply demand of community (J) - mgd	CALL F15QO (J)	J = community number	SUBROUTINE F15QO (J)
Function 16 - Average off-river municipal water supply of community (J) - mgd	CALL F16QB (J)	J = community number	SUBROUTINE F16QB (J)
Function 17 - Maximum weekly on-river municipal water supply of community (J) - mgd	CALL F17QI (J)	J = community number	SUBROUTINE F17QI (J)
Function 18 - Average sewage discharge of community (J) - mgd	CALL F18QS (J)	J = community number	SUBROUTINE F18QS (J)
Function 19 - Maximum daily sewage discharge of community (J) - mgd	CALL F19QD (J)	J = community number	SUBROUTINE F19QD (J)
Function 26 - Quality of raw sewage of community (J) - BOD (ppm), DO deficit (ppm), coliforms (MPN/100 ml.), chlorides (ppm)	CALL F26PS (J,I)	J = community number I = river station number of municipal water supply intake of community J	SUBROUTINE F26PS (J,I)

Function Description	Call Statement in Main Program or Subprogram	Arguments of Call Statement	Title of Subprogram with Instructions for Evaluation
Function 27 - Quality of treated sewage of community (J) - BOD (ppm), DO deficit (ppm), coliforms (MPN/100 ml.), chlorides (ppm)	CALL F27PD (J,I)	J = community number I = river station number of municipal water supply intake of community J	SUBROUTINE F27PD (J,I)

APPENDIX IV

STREAMFLOW ROUTING FOR WATER POLLUTION STUDIES

Introduction

For the analysis of the flow regimen in a channel receiving discharges of pollutants, two concepts are important. First, account must be taken of the change in concentration of pollutants at each location where a pollutant enters the stream. Second, the relationship between discharge and velocity for any reach must be maintained. For example, the change in BOD between two points on a stream, in which the pollutional load is delivered at the upstream end, depends on the travel time for the reach. For water pollution studies, this is estimated by determining the modified discharge and concentration at the point receiving the pollutant, estimating the velocity for the reach, and computing the time from velocity and distance traveled. In the natural case, the BOD and other stream quality characteristics are continuously changing with both time and distance.

As part of a larger study which utilized a time varying model patterned on the Merrimack River in northeastern Massachusetts, a special flow routing procedure has been developed to study these effects. Although the time varying model also included other time varying parameters, the unsteady flow condition present in natural streams appears to have been the major problem in the development of practical models of this type.

The engineering literature describes many different procedures for solving flow-routing problems. The theoretical analysis of the movement of flood waves is quite complex, and methods based on a strict mathematical treatment are not practical for routing through natural river channels. Methods applicable to a river basin such as the ones studied for this work rely on some simplification of the concepts in the theoretical approach. Most of these methods utilize a relationship between stage and storage or between discharge and storage. Procedures which assume an invariable discharge-storage relationship are useful only for reservoir routing (59). Of the methods using a variable discharge-storage relationship, both the "Muskingum method" (60) and the "working value method" (59) have been satisfactory for routing in a natural stream. The more empirical "lag method" (61) has also been found to yield adequate approximations. These methods are discussed extensively in publications by the U.S. Army Corps of Engineers

(62) and by Chow (59), and computer programs have been written to facilitate the great number of computations which must be made (63, 64).

Recently, several newer methods have been proposed which recognize more of the parameters involved in the theoretical approach. Baltzer and Lai (69) use a set of non linear, partial differential equations describing one dimensional translatory wave motion. They have included such effects as fluid friction, variable channel geometry, wind, lateral inflow and outflow, Coriolis acceleration, and over bank storage. Another method by Himmelblau and Yates (66) represents the excess flow for pulse-like stream releases. Amein and Fang (67) have reviewed three methods in detail and evaluated their utility for computing the unsteady flow in artificial and natural channels. They note that the advent of the digital computer has made it feasible to obtain complete solutions of the equations by numerical methods. They have also found that different methods, although producing almost identical solutions, vary considerably with respect to speed, reliability, simplicity, and convenience.

After completing the investigation of the existing flow routing procedures, it was concluded that none of them would give the discharge-velocity-time results necessary to study the effects of unsteady flow on the pollution of a stream. It was, therefore, necessary to develop a new procedure which would be especially suited for pollution studies.

The continued use of the Merrimack River data created several problems for the development of a routing procedure. The stream is subject to reservoir regulation at many sections, making short term correlations between gaging stations impractical. Also, although much information is available on the routing of unsteady flows occurring during high flow periods, a thorough search of literature revealed no satisfactory procedure for the routing of normal and low flows in a natural stream.

It was decided to use the river data for the Merrimack which was available and in the form obtained for many other streams where pollution abatement is being studied. For the Merrimack, the available low flow information comprised a hydrograph upstream of the region studied and measurements of the travel times of various steady discharges through the study reach. The upstream hydrograph was available from the USGS (36) and the time of travel studies were made in field surveys by the FWPCA (1), in 1965, and Camp, Dresser and McKee (2), in 1963. This information was used as the basis for deriving a downstream flow configuration. After the routing procedure for the Merrimack River was well developed, similar stream flow information was obtained for the Susquehanna River in New York and Pennsylvania (68, 69). For the Susquehanna, a downstream hydrograph unaffected by reservoir regulation was also available; this additional information was used to confirm that the method is a valid routing technique.

<u>Description of River Routing Procedure</u>

This routing procedure accepts existing hydrograph data at a station, and estimates hydrograph data at any downstream location. In the next portion of the discussion, discharges at the upstream and downstream ends of a reach will be referred to as "inflows" and "outflows" respectively.

Because recorded daily inflows at the station where hydrograph data are available often change radically between two successive values, the routing procedure must take account of their differing travel times. A key premise of the procedure is that the travel time required for any inflow to reach a downstream station may be related to a weighted average of the travel times for succeeding inflows. This is a reasonable assumption because the merging of successive flows is obvious. The principle employed is similar to the "lag method" (61) which makes approximations by time displacement of average inflows.

The flow-velocity equation which was used for this procedure is as follows:

$$V = Z_1 \, Q^{Z_2} \qquad (1)$$

Where V is velocity in miles per day
Q is discharge in mgd
Z_1 and Z_2 are distance varying river constants

This equation, used by Goodman (3, 4) in his steady state mathematical model of the Merrimack was demonstrated by Leopold and Maddock (7) to have general applicability for many streams. Between two successive stations on the river, the discharge and velocity are assumed to be constant. This approximation is reasonable for the Merrimack River model because the distance between stations averages less than 1 mile in length, and a new station is always located where any significant change in flow characteristics can occur. The time of travel can be computed from the following simple relationship:

$$T = D/V = D/Z_1 Q^{Z_2} \qquad (2)$$

where T is time in days
D is distance in miles

In order to estimate a change in discharge between successive stations, an increase or decrease of discharge is computed using the following empirical formula:

$$Q_2 = Q_1 + (D_1 - D_2) \times CQ_y \times (D_2/Ln)^{Zn} \qquad (3)$$

where Q_1 = inflow at section 1, in mgd
Q_2 = flow at section 2, the next station downstream, in mgd
D_2 = distance of river station 2, above the farthest downstream station in miles
D_1 = distance of river station 1 above the farthest downstream station in miles
CQ_y = dimensionless coefficient determined by the flow routing procedure, which may be either positive or negative
L_n = total length of river channel studied in miles
Z_n = "Fitting coefficient" which controls the pattern of flow additions or subtractions

The essentials of the routing procedure are as follows. First, an "objective" travel time is found for an inflow. This travel time is evaluated by equation (2) for some weighted average of the succeeding inflows. In order for this inflow or a portion thereof to reach a downstream station in the objective travel time, while maintaining the integrity of equation (1), the discharge as it moves downstream changes by amounts given by equation (3).

The incremental flow computed according to equation (3) is added or subtracted to obtain an adjusted discharge. This adjusted flow applies for the stretch to the next station, where another adjustment in discharge is made. As noted previously, equations (1) and (2) assume a steady discharge between stations.

It is recognized that more than one pattern of flow additions or subtractions will give the same travel time. The exponent "Zn," which has been designated as a "fitting coefficient," determines the distribution of these flow alterations. As shown on Figure 1, by increasing the "fitting coefficient" (Zn) a greater amount of the total change in discharge takes place near the beginning of the reach. With lower values, the additions or subtractions are distributed more evenly. Also, Figure 1 shows that less total change of flow over a reach is required to meet the objective time with higher values of Zn. The corresponding effects on discharge over a period of time at a downstream station are shown on Figure 2.

To ensure that the general shape of the inflow hydrograph is retained for downstream stations, maximum values for compression or spreading of time intervals are specified. For example, the normal time interval between successive inflows is 0.167 days (4 hours), whereas the allowable intervals at the farthest downstream station could range from 0.02 to 0.314 days. An inflow, when followed by a sequence of larger inflows, is "compressed" so that the corresponding outflow rate, after routing, increases and is maintained for a shorter interval. When an inflow is followed by a sequence of smaller flows, a "spreading" of time intervals is effected.

Hydrograph data used for the initial studies were obtained from USGS records (36), and were in terms of daily values. Six flow routings are made per day (at 4 hour intervals) in which each inflow is set equal to the daily value. Also available were 2 hour discharge values (71); these could be used to refine the routing procedure, but are not discussed herein.

The method has a great deal of flexibility since, depending on the particular flow conditions, different values may be used for the parameters in the routine. For less variation of inflows and/or shorter reach of river, a weighted average of the succeeding discharges may be computed from perhaps the next two days of inflow values. Other conditions may require 5 or more days of inflow values in order to adequately adjust for the downstream effects. The selection of the "fitting coefficient" in equation (3) determines the pattern by which the incremental flow changes are to be made. And the specification of allowable time intervals can also be modified.

After working with this procedure, one develops an understanding of the combined effects of changes in the parameters. These parameters can be employed advantageously to obtain good agreement between a derived outflow hydrograph and a recorded hydrograph when available. Because appropriate outflow hydrograph data were not available for the initial studies, parameters were selected to obtain an outflow hydro-

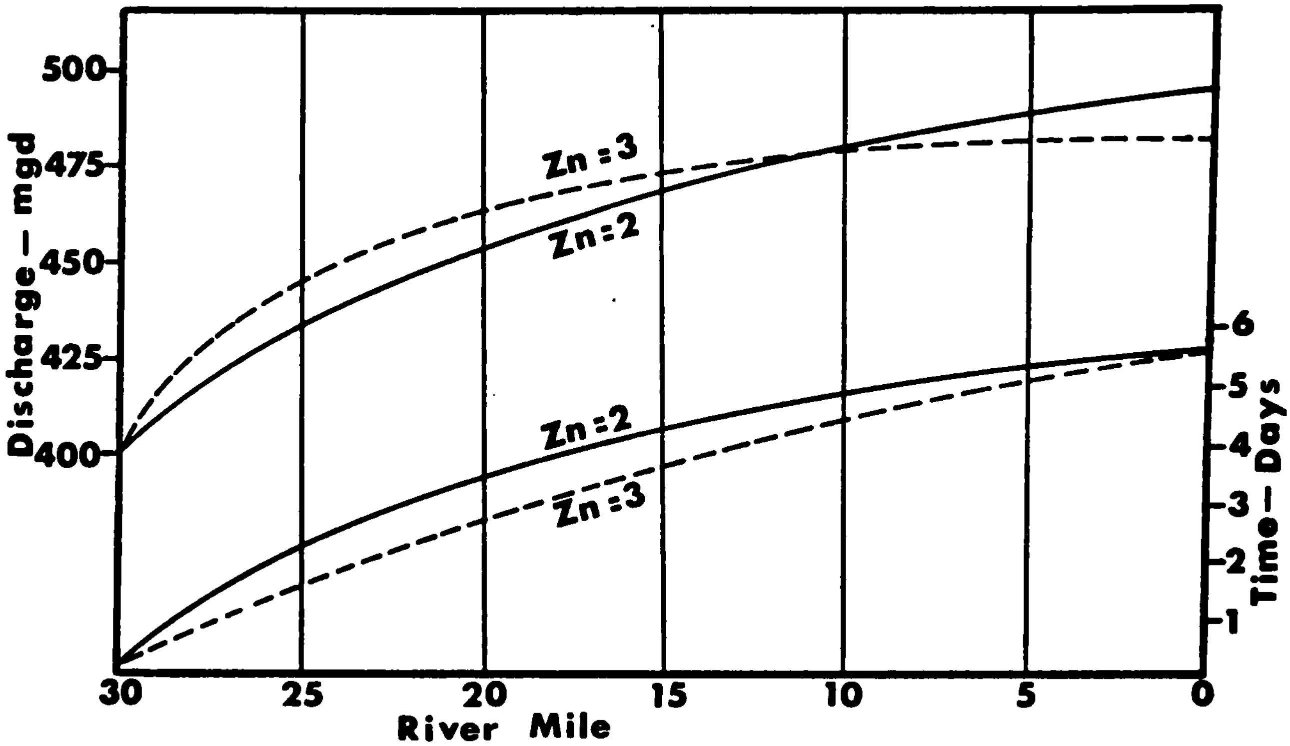

Figure 1 Effect of Fitting Coefficient on Routing

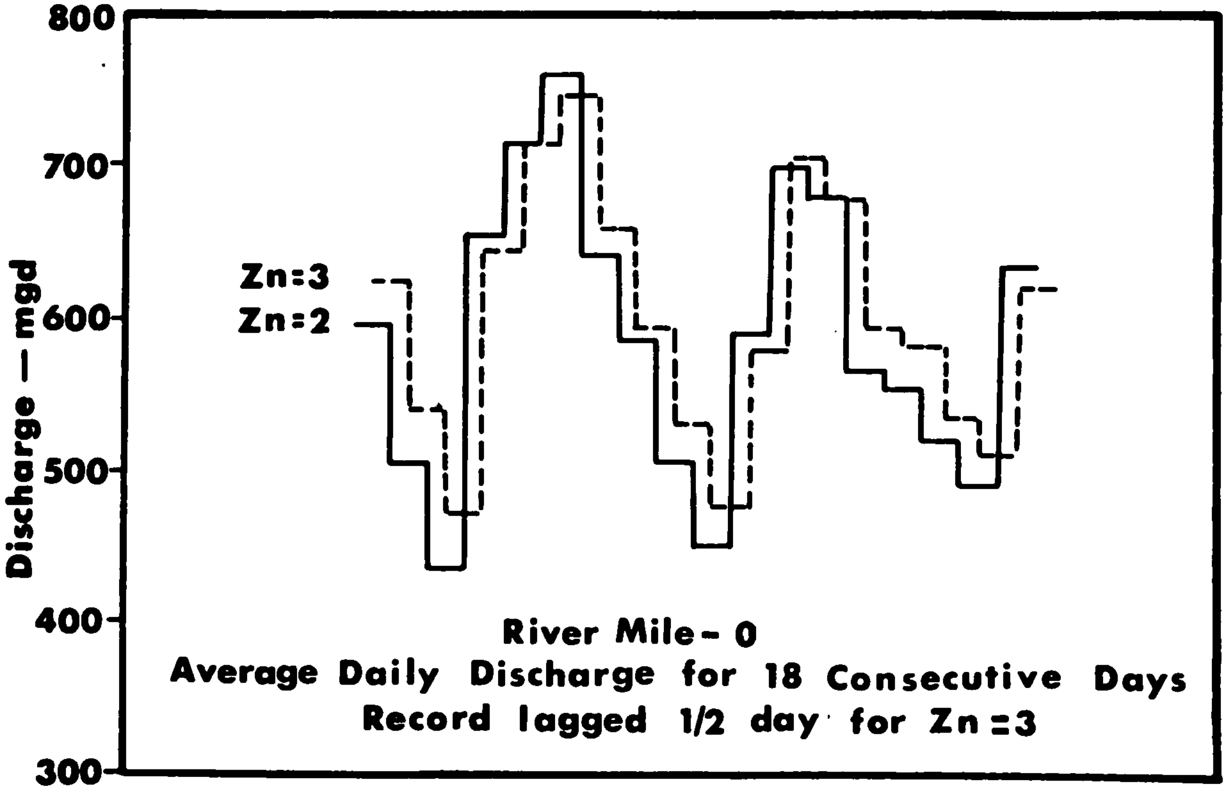

Figure 2 Effect of Fitting Coefficient on Computed Discharges at Downstream Station

graph which seemed most reasonable. Figure 3 shows daily inflows together with computed outflows for a 30 mile long reach of the Merrimack. This figure also includes the outflows in terms of daily values, in order to compare inflows and outflows on the same basis.

In the absence of measured outflows, an approximate checking procedure was developed to continuously compare cumulative inflow and outflow (see Figure 4). By adjusting the weighted average of succeeding discharges, the mass balance can be maintained for as long a study period as desired.

This work employed a C.D.C. 3300 computer at Northeastern University. The computer time required ranges widely depending upon the precision desired for the outflows. The computer routine determines the values of CQ_y in equation (3) by successive increments. If the adjustments of CQ_y are small, the results will have improved accuracy but the computer time will be correspondingly greater. Increments requiring about 6 minutes of computer time per month gave the best results for developing good fit with an actual, downstream hydrograph. Reasonable approximations, however, were obtained with larger increments which required only 2 minutes of computer time. The latter computations were found to be adequate to demonstrate the effect of time varying discharges on the oxygen balance of a stream. Based on the present charges of 200 to 300 dollars per hour for use of a medium sized computer, it would cost about 5 to 15 dollars to process one month of data for use in a time varying mathematical model.

<u>Checking of Procedure</u>

The data used for the validation of this procedure was for the Susquehanna River in New York and Pennsylvania. Information included a daily discharge hydrograph at Vestal, New York (69) which was just upstream of a reach where a recent study by the USGS (68) has related discharge and time of travel. The magnitudes of the discharges and the corresponding velocities were not greatly different from those obtained for the Merrimack. The checking was made possible by the availability of another hydrograph located at Waverly, Pennsylvania (69) which was just downstream of the reach for the USGS study. The total length of this reach was just over 30 miles.

Comparisons of the hydrographs at Vestal and Waverly indicated that in this reach the Susquehanna was very much affected by ground water and local surface water inflows

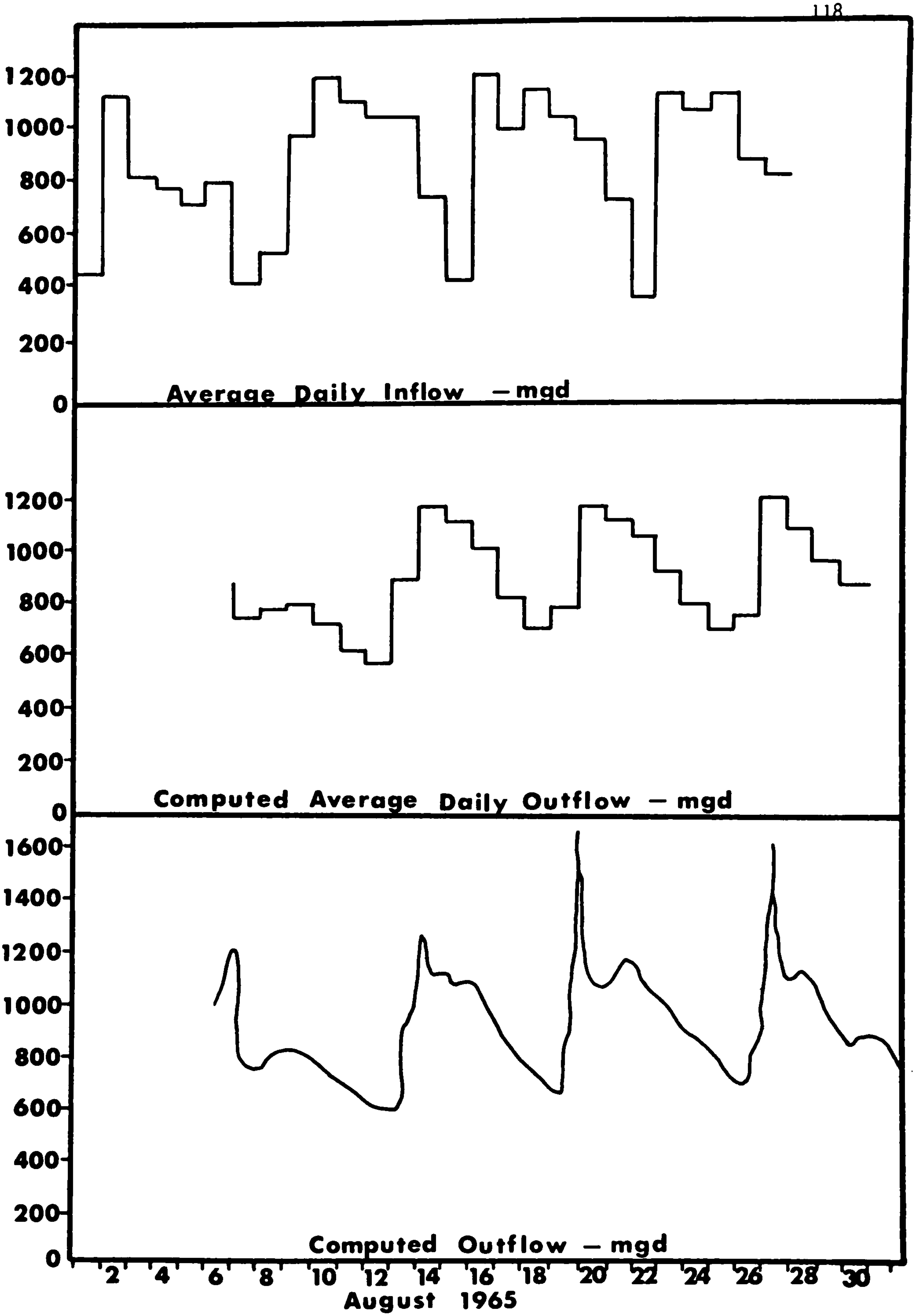

Figure 3 Comparison of Inflow and Outflow Hydrographs

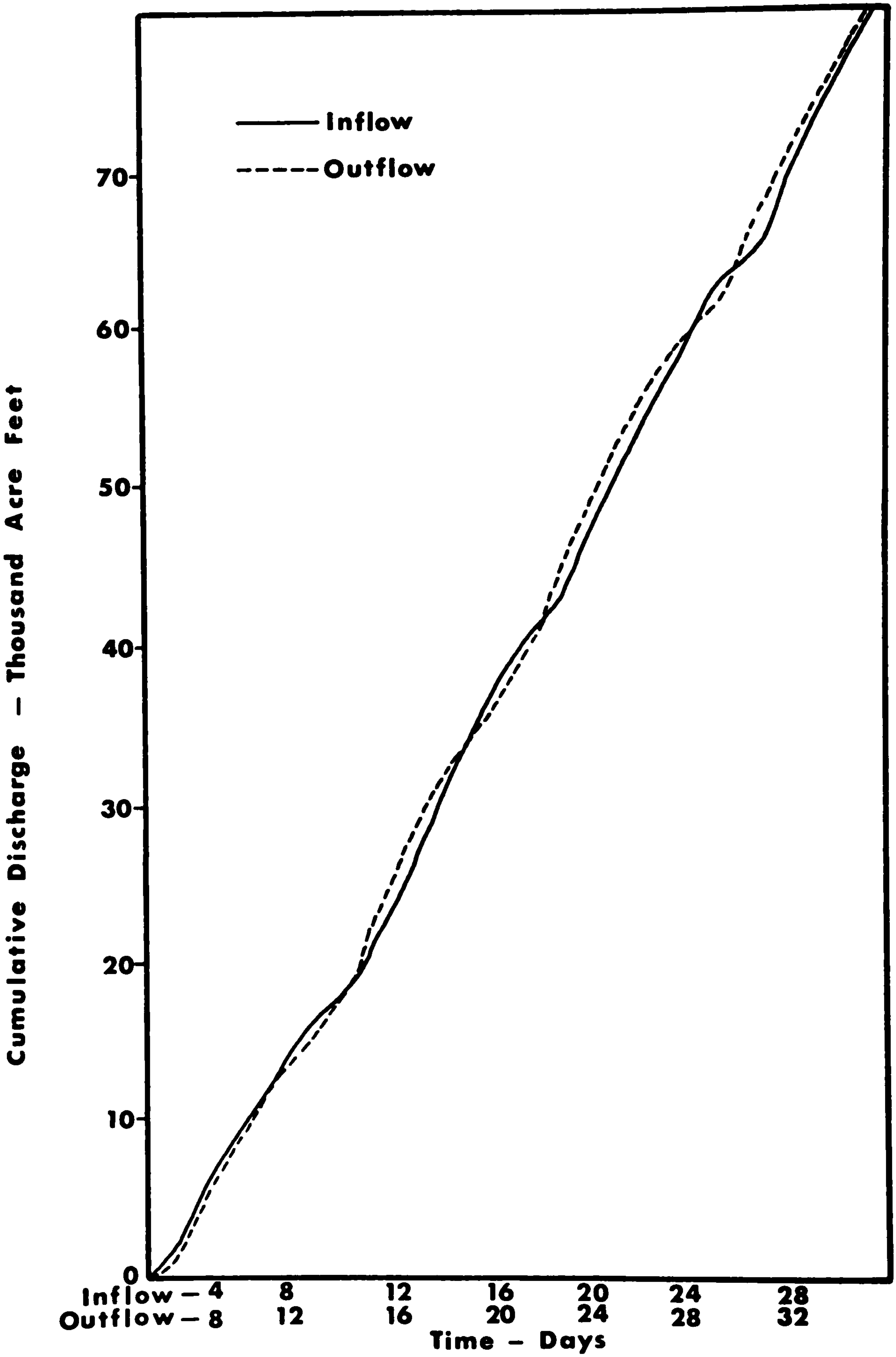

Figure 4 Mass-Diagram of Inflow and Outfl...

during the low flow period. This caused some complications in trying to obtain a good correlation between the computed and observed hydrographs.

Figure 5 shows the upstream measured hydrograph at Vestal and the corresponding measured downstream hydrograph at Waverly. Superimposed on the Waverly hydrograph are the instantaneous values determined by the routing procedure. The instantaneous values computed for Waverly were derived from the daily mean discharges measured at Vestal. Because of the nonrecorded fluctuations about the daily averages at the measured hydrographs, and because of the effects of ground water and local surface inflow, one could not expect to obtain a perfect fit, and the results are judged to be satisfactory.

Conclusions

For water pollution studies the travel times of the pollutants are quite important. Unlike available flood routing methods, this low flow routing procedure emphasizes this fact. Reasonable approximations of unsteady flow effects are obtained by utilization of an inflow hydrograph and information on the travel times of pollutants for varying discharges. If an outflow hydrograph is available, the method can be adapted to provide reasonable correlation of computed and measured outflows.

Although the method is somewhat empirical, and is derived from intuitive concepts rather than a rigorous analysis of unsteady flow theory, it is suitable for incorporation in a mathematical model used in the biological analysis of a stream environment.

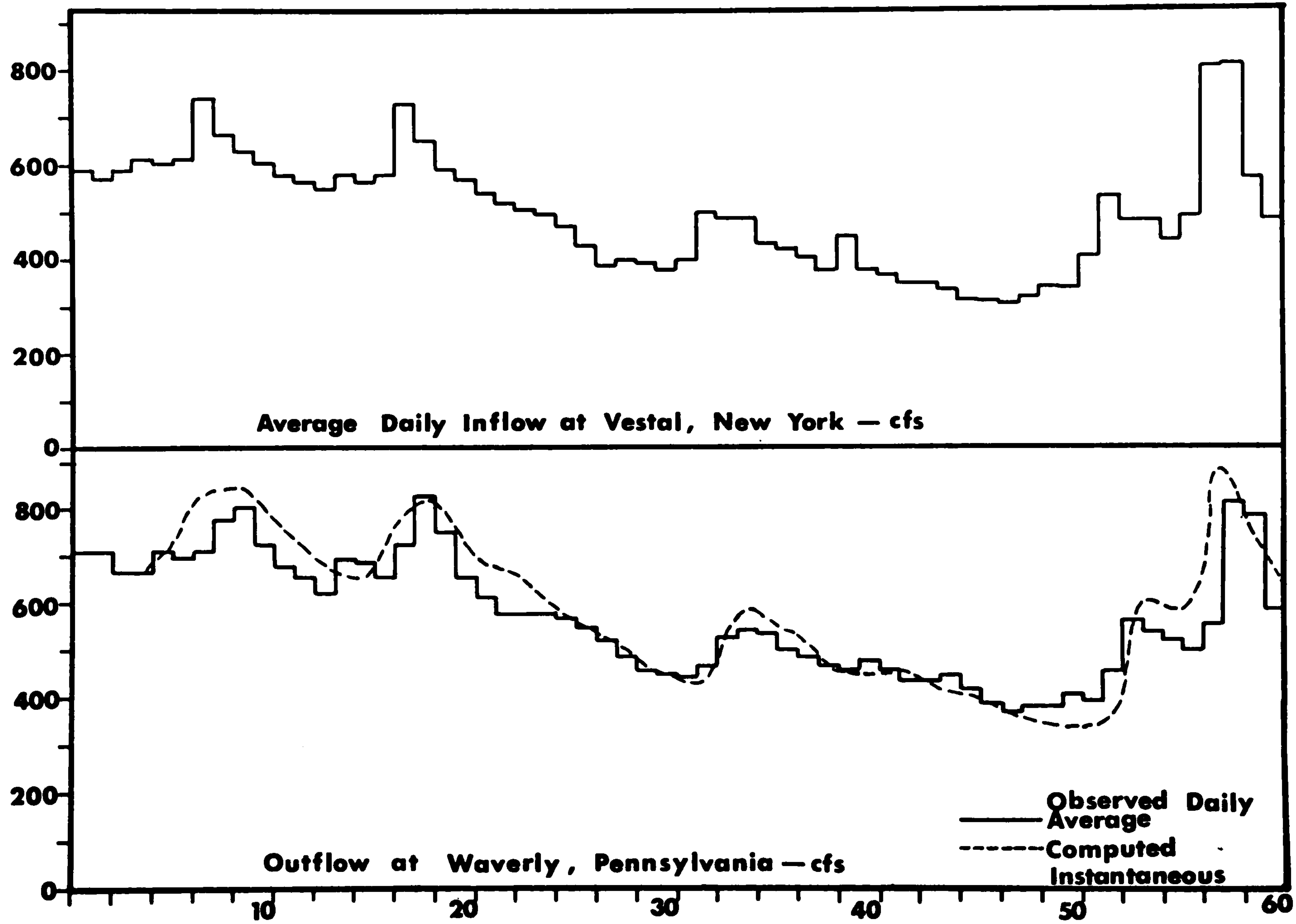

Figure 5 Comparison of Measured and Computed Hydrographs for Susquehanna River

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