SIMPLIFIED MATHEMATICAL MODELING OF WATER QUALITY

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Under sub-contract with:

THE MITRE CORPORATION

ENVIRONMENTAL PROTECTION AGENCY
Water Quality Office

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ENVIRONMENTAL PROTECTION AGENCY
WATER QUALITY OFFICE
WASHINGTON, D.C. 20242

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Memorandum

To : All Regional Directors

Attn : Planning Directors

From : Director, Division of Planning and Interagency Programs

Subject : Guide: Simplified Mathematical Modeling of Water Quality

This Guide was developed under a subcontract with the Hydroscience Corporation through the Mitre Corporation. The purpose as stated on the cover is to assist our planning effort during the interim process by providing a simplified methodology for applying mathematical models to the analysis of water quality.

Since the methodology developed in this Guide has not been widely tested, we are providing a limited number of copies for in-house use. We would like your evaluation as to the usefulness and reliability of the procedures. If we find that this Guide would be helpful to local and State personnel, additional copies will be printed. Please maintain control of your copies until our evaluation is completed.

I would like to acknowledge the efforts and cooperation of Messrs Donald J. O'Connor, Robert V. Thoman, John L. Mancini, and Henry J. Salas of Hydroscience in developing this Guide and Messrs William P. Somers and Ken Feigner of this office in reviewing the document.

Frank M. Covington

Enclosure
Guide

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USER GUIDE

The guidelines in this report are to be used with discretion and apply only to interim planning needs. Complex river and estuarine systems or complex water quality problems, such as eutrophication, are not covered by the simplified analysis given in these guidelines. The user should carefully familiarize himself with all of the assumptions that are made for each analysis. The results of the simplified water quality analysis, especially with respect to the projected effects in water quality of different treatment levels, should be considered as trend indications only, and not as a precise, certain prediction. In any event, the results of the simplified analysis should be used only as a supplement and aid to engineering judgment and never as a substitute.

The analyses are structured in such a manner that estimates can be made of water quality responses with a minimum amount of information. This does not mean that the analyst should use only that minimum level of input data. Every attempt should be made to collect, interpret, and utilize all relevant data on water quality, waste loads, river flows, population, industrial growth, etc. Therefore, whatever data are available or can be obtained within the constraints of the total planning effort should be incorporated into the analysis.

In order to aid the analyst in utilizing the analyses presented in this report, the following breakdown may be of assistance (see Table of Contents for a more detailed structure of the report).

The results obtained from the applicaton of the simplified techniques presented have compared favorably with a very limited number of specific water quality situations where a more detailed analysis was available. It is anticipated that comments and questions may develop from further application of the simplified techniques presented. These can be directed to either:

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I. INTRODUCTION

This report presents a general framework for the methods of application of mathematical models to the analysis of water quality. These models relate wastewater discharge to water quality in the receiving body. The modeling effort is considered to be a part of the overall water quality planning operation. The types of models necessary to address various water quality problems in streams and estuaries are discussed.

The report contains a detailed presentation of the necessary Tables, nomographs, and technical data needed to evaluate receiving water quality for interim planning purposes.

In particular, the interim procedures address problems concerning total dissolved solids, coliform organisms, nutrients, oxidation of carbonaceous, and nitrogenous compounds, and the dissolved oxygen analysis in streams or estuaries. Both single and multiple sources of waste discharge are considered. These constituents may be analyzed employing the interim planning tools presented in this manual provided the water body is approximately described as one-dimensional and the basic geometry of the system is relatively simple.

The simplified analysis presented for the interim planning function, requires knowledge of the present population, drainage area, and water depth to develop a first approximation of the degree of treatment required to meet water quality standards. For estuaries, data are also needed on the cross-sectional area of the estuary.

Problems associated with eutrophication and contamination of parts of the food chain, cannot be adequately analyzed by the simplified procedures presented in this report. Furthermore, the procedures given in this report are not applicable to complex stream and estuarine systems or to lakes, impoundments and bays.

It should be pointed out that the intent of the interim water quality analysis is to protect the investment by insuring that appropriate levels of treatment may be installed and that land area is available for the required treatment at present and future population levels. It is anticipated that the application of the interim planning techniques presented in this report will result in the classification of a particular application for construction monies, in one of three categories.

1. The proposed treatment schemes will be capable of meeting water quality

standards under present and future conditions. No action appears required to protect the investment in facilities.

2. The proposed treatment plants will not meet water quality standards under present or future conditions. Advanced treatment systems will be required to meet water quality criteria. Additional planning appears to be needed with the time frame for this planning, dictated in part, by the population level at which water quality criteria will not be met.

3. Proposed and advanced waste treatment systems will not meet water quality criteria under present or future conditions. This should be a warning that the effectiveness of the proposed investment in facilities may be diminished because of the inability to meet water quality standards. Additional planning is necessary and changes in the size of plant, discharge location, low flow and population served should be examined.

There are a number of situations around the country which are characterized by complex water quality problems and substantial investments in water pollution control facilities. It is felt that the interim analysis contains too many simplifying assumptions to justify its application to areas where complex water quality problems must be solved and where the investment to solve these problems is substantial. Examples are areas such as the New York Harbor complex, San Francisco Bay, Los Angeles, and Galveston Bay. Situations of this type which require more detailed planning should not be addressed within the context of the interim planning procedures presented in this report.

II. MATHEMATICAL MODELING FOR DETAILED PLANNING

A. Introduction

One element of the detailed planning process is the development and application of models which will relate waste inputs to water quality in the receiving body. The following sections discuss the elements which influence model selection, construction, verification, and application. Water quality modeling can cover a wide spectrum of effort ranging from crude "first-cut" analysis, to highly detailed time-varying analysis of complex water quality problems.

The models can be employed to evaluate alternate engineering plans for control and management of water quality. Alternatives such as varing degress of treatment, relocation of the waste discharge points, low flow augmentation, regional treatment systems versus multiple plants, are some of the specific alternatives whose influence on receiving water quality can be assessed by application of the water quality models. The models can also assist in evaluating the relative benefit to water quality from removal of different constituents.

The factors which influence the degree of complexity of the modeling effort include the water quality problem of concern, the time and funds available, the observed data available or obtainable on present and past water quality and waste discharges, and finally, the uncertainty and risk associated with employing a simplified model in lieu of a more complex analysis. This section of the report deals specifically with the content and elements of detailed water quality models. For detailed planning, it is most desirable to develop fully, all required information for each individual body of water and problem context. It is recognized, however, that practical constraints may necessitate use of approximations and broad rules of thumb for certain inputs and parameters. Simplification in the modeling effort and the use of generalized information, can provide a valid platform for water quality management decisions. When possible, indications are presented, in this section, of methods for simplifying the analysis and developing approximations for gaps in available information. Thus, a framework is presented which will allow development and application of simplified models as well as detailed models for water quality analysis.

The framework for detailed mathematical modeling is made up of the following principal components:

1. water quality problem identification
2. mathematical model classification

3. identification of wastewater inputs
4. specification of kinetic interaction
5. verification analysis
6. analysis of control alternatives

B. Water Quality Problem Identification

From the point of view of water quality control and management, it is desirable to examine water quality problems in terms of specific constituents or groups of constituents which are discharged as a result of man's activities and natural phenomena. Table II-1 presents a list which relates constituents to water quality problems and indicates those consitituents and variables which have been or can be modeled with current technology. The list of variables and specific water quality problems in Table II-1 is not exhaustive but can provide a guide for the content of fully-developed plans.

One of the initial steps in the planning effort with respect to water quality modeling, is the identification of the water quality problems presently observed and those projected under future conditions of population growth and development. Having identified the significant present and future water quality problems, it is then necessary to select the constituents which are discharged to the environment, from

TABLE II-1

SOME WASTE DISCHARGE CONSTITUENTS
AND
RELATED WATER QUALITY PROBLEMS

	Waste Discharge Constituent	Water Quality and Water Use Problem
(*)	TDS & CL	Agricultural, Industrial and domestic water supply
(*)	Temperature	Dissolved oxygen, aquatic balance
(*)	Carbonaceous BOD & COD, Total Carbon	Dissolved oxygen - Nutrient
(*)	Organic Nitrogen	Dissolved oxygen - Nutrient
(*)	Ammonia	Dissolved oxygen - Nutrient
(*)	NO_2 and NO_3	Nutrient - Dissolved oxygen - water supply
(*)	Phosphate	Nutrient
(*)	CCE	Water supply - food chain
	Toxic metals and inorganics	Water supply - food chain
	Toxic organics	Water supply - food chain
(*)	Bacteria	Water supply - recreational usage
	Virus	Water supply - recreational usage
	Floating substances	Recreational usage
	Suspended Solids	Recreational usage - dissolved oxygen - nutrient - light limitations
	Color and Turbidity	Recreational usage - light limitations

Note: (*) Variables which have been modeled or can be modeled with current technology.

natural and manmade activities, that are responsible for water quality problems. It is then appropriate to consider a meaningful engineering framework, such as a mathematical model, for analysis of the methods available for improving and managing the system. The factors which should be included in the mathematical analysis, include the hydrology and the climatology of the area, from which water balances, hydraulic circulation, and temperature structures can be defined. In the mathematical model, these factors are combined with knowledge of the assimilation mechanisms and reactions that are involved in the specific water quality problem.

Within this framework, each specific water quality problem may be viewed from a characteristic scale which would set the degree of simplicity or complexity of the required mathematical model.

Certain problems can be attacked relatively quickly, employing the simplier conceptual hydraulic and quality models associated with analysis of long-term phenomena. The type of problem which is properly addressed in this context is related to the long-term patterns of substances which are conservative, such as chlorides, or those substances which change at such slow rates that they may be regarded as conservative.

A second scale of time which is appropriate in the investigation of water quality problems is the annual cycle in which the time unit is a week, month, or season. At this intermediate time scale, it may be necessary to account for lateral and vertical spatial variation in water quality. The eutrophication problem is amenable to analysis, utilizing this intermediate time scale.

A third time scale is one in which the time unit is hours extending over an interval of one day to possibly one week's period. This time scale establishes a comparable spatial dimension. The spatial scale may therefore involve two and possibly three dimensional analyses. Typical problems addressed in this respect would be transient algae blooms, unexpected spills or discharges of pollutional mass, such as from combined sewer overflows.

A wide variety of planning problems can be analyzed by use of steady-state mathematical models which can provide the necessary spatial detail for important water quality variables. Certain phenomena can achieve steady-state conditions within a short time interval and as such, can be modeled with relative ease. Examples of the phenomena which can be modeled on a steady-state basis are bacterial dieoff, dissolved oxygen concentrations, and nutrient distribution and recycle. These steady-state representations are particularly useful because of the ease of model operation and ability to respond rapidly and relatively inexpensively to specific planning questions.

1. Hydrology and Climatology

The hydrology of the basin or metropolitan region, in particular the freshwater flow, is of considerable importance in mathematical modeling. This parameter determines not only the dilution which the wastewater receives, but also the velocity at which the waste moves downstream. The flow also affects some of the reaction coefficients. In the determination of the total flow in the river, any net flow introduced by waste sources must also be included. For up-basin regions and small streams, this flow can be significant.

The determination of the water temperature characteristics of the river sets the level of the reaction coefficients in any model related to bacterial or higher order biological activity. Water temperature together with drought flows generally establish the most critical design conditions for determining the adequacy of waste treatment schemes.

Section III-C2 presents some technical data on river flows and temperature that may be used for simplified mathematical models.

2. Hydrodynamics

The hydrodynamic properties of a body of water, for example, velocity, tidal characteristics, and turbulent diffusion, form the basic transport mechanisms which classify the body of water into one of several generic categories to be discussed below. The degree of detailed hydrodynamic information that is required is strongly dependent on the time and space scale of the problem under consideration.

River velocities can often be related to river flows by a log-log relationship (see Section III-C2). If information is available which correlates velocity with flow (or depth with flow), this information can form a basis for predicting the velocity regime in a river under different drought-flow conditions.

Tidal velocities can often be obtained from the U.S. Coast and Geodetic Survey Tide and Current Tables, or from direct measurement. The net river flow in estuarine analysis also forms an important input into the mathematical model of estuarine systems. Flow records are often available for estuarine tributaries that would allow one to construct the net river flow regime at the head end of an estuary and downstream along its length. Flows due to incremental drainage area accretions can be readily estimated with data from upstream reaches.

For large lakes and coastal waters, the hydrodynamic situation becomes increasingly more complex. Density stratification further adds to the difficulty of specifying the hydrodynamic circulation. For lakes, therefore, the hydrodynamic equations must be considered in determining water movements and subsequent pollutant distributions.

C. Model Classification of Natural Systems

The classification of natural water systems for water quality analysis is based primarily on the number of spatial dimensions which must be considered and on the mixing characteristics of the body of water.

1. Streams and Rivers

The simplest situation is a one-dimensional flowing stream or river where the mixing characteristics are such that the dispersion of the mass of material can be neglected in comparison to the flow. In this case, the river flow is the major mass transport mechanism. This simplification is significant in terms of computational complexity and the amount of information required for water quality analysis. The fundamental equation that governs the transport of material in a non-dispersive system is given by:

$$\frac{\partial c}{\partial t} = - \frac{1}{A} \frac{\partial}{\partial x} (Qc) - Kc$$

$$c = c_o(t) \ @ \ x = 0 \qquad \text{(II-1)}$$

where:

c = water quality variable

t = time

x = distance downstream

A = cross-sectional area

Q = river flow

K = first order decay coefficient

For a complete specification of the waste material one requires the intial concentration, the reaction rate, and the river flow and cross-sectional areas. For some variables, there may be a coupling effect where the solution of one equation feeds forward into a second equation and acts as an input. For example, the interaction between the biochemical oxygen demand and the dissolved oxygen is represented by a coupled set of equations. If one is interested in tracking nitrogen components through a nitrification regime in a river system, three or four or more equations may be required, all of which interact through reaction kinetics.

2. Estuaries

An estuary is defined here as that portion of a coastal river where the tidal action from the ocean is a significant hydrodynamic parameter. There are two broad sections of estuaries, the tidal river portion where the water body ebbs and floods but is entirely freshwater; and the lower estuarine portion where, in addition to ebbing and flooding of the tide, a significant incursion of sea salts occurs. One or two spatial dimensions, (e.g. the longitudinal and vertical dimensions) may be of importance in estuaries. The primary difference, however, between estuaries and the one-dimensional river flow situation is the dispersive mass transport due to the tidal mixing occasioned by tidal flow reversals. This forms an important transport phenomena in addition to the net freshwater flow through the estuary and, as such, must be included specifically in the analysis.

Several methods are available to directly evaluate the dispersion coefficient (see Section III-C2). The selection of the method for the evaluation of the dispersion coefficient is in part, determined by the time and funds available and by the specific time and space scale associated with the water quality problem of concern. Care must be excercised in calculation of the dispersional characteristics of the estuary to insure that the dispersion coefficient calculated is consistent with the modeling effort in which it is to be employed for projection of water quality. This is particularly significant in that the dispersion coefficient is related to the time and space scale over which it is measured.

3. Lakes and Reservoirs

Lakes and reservoirs can involve either two or three spatial dimensions. The flow regime in these bodies of water can be quite complex since there is usually no dominant mechanism which determines the advective flow and mixing in contrast to the case of estuaries and rivers. The stratification which can occur due to the absence of intense advective or mixing forces, complicates the distribution of water quality constituents in a vertical direction. Thus, lakes and reservoirs can encompass a broad spectrum of complexity starting with essentially completely mixed bodies to highly stratified complex situations.

A number of attempts have been made to define the hydrodynamic regime associated with lakes, reservoirs and impoundments. In general, the mixing, turbulence and advection are due to winds, seiches, and density differences. From a practical planning standpoint, two options are open to modeling lakes and impoundments. On the one hand, it may be possible to apply some of the refined mathematical techniques which have been developed to evaluate the hydraulic regime. Alternatively, it may be possible and practical, depending on the water quality problem being addressed, to employ observed data and field measurements as an adequate assessment for the hydrodynamic circulations. As an example, it is possible to obtain data on the thermal stratification within the lake or impoundment and accept this as the basis for segmentation of a model of the lake. In addition, it is possible to inject dye into various areas of the lake and determine dispersion, mixing, and circulation patterns from an observation of the transport of dye or other constituents within the lake.

4. Coastal Waters

Coastal waters encompassing tidal embayments and near shore coastal waters can require two or three dimensional analyses. The techniques available for evaluation of the hydraulic regime in terms of circulation pattern, dispersional coefficients et.al., are essentially similar to those available for evaluation of these phenomena in lakes and in estuaries. Once again, the particular water quality problem being addressed will dictate the most effective method of developing an adequate understanding of the hydraulic circulation and mixing patterns.

D. Identification and Quantification of Present and Projected Inputs

After the phenomena which are responsible for the mass transport of significant water quality variables are identified, the next step is to identify the location of and quantify the magnitude of present and projected waste inputs. The inputs or sources of pollutional substances which should be considered in the modeling effort for a fully developed water quality plan include:

1. Treatment installations: (point sources)
    - a) Municipal plants
    - b) Industrial plants
    - c) Storm overflow treatment plants
    - d) Agricultural

2) Storm overflow related sources and runoff:
    - a) Municipal and Industrial
        - 1) Combined sewers
        - 2) Separate sewers
        - 3) Surface drainage

b) Surburban, partially developed, and Agricultural
   1) Combined and Separate systems
   2) Land and surface drainage
   3) Ground water

3) Background and other sources:
   a) Bottom Deposits (e.g., utilization of material such as oxygen; release of material such as ammonia; sink of material such as solids)
   b) Faulty collection facilities (e.g., regulators, tide gates)
   c) Drainage from land sources. (e.g., swamps, small streams, creeks, septic tank overflows)
   d) Residual influences from upstream sources. (e.g., treatment plants, algae growths, impoundments, et.al.)

4) Other major sources of polluting substances which may be unique to a specific problem or area.

Each of the waste sources indicated above must be characterized in terms of the specific water quality problems to be investigated in the planning effort. Characterization of waste sources must be considered (see Table II-1), and the spatial and temporal distribution of these constituents from each of the waste inputs should be determined. As an example, average values for the quantity of waste introduced into a body of water may be employed if the water quality problem is being addressed on a steady-state basis. If however, the water quality problem is being addressed on a non-steady-state basis with a seasonal time scale, the waste sources should be characterized in terms of their variability on the seasonal time scale.

E. Specification of Kinetic Interactions

The interrelationships of the variables implicated in the water quality problem must be specified in order to continue the model construction. Variables can be grouped into the following classifications:

a) Conservative - non-interacting (examples are salinity, total dissolved solids, total phosphorus, total nitrogen)

b) Non-conservative - first order kinetic approximations (BOD, coliform bacteria)

c) Sequential reacting variables - first order kinetics appear applicable for some studies (BOD-DO; Ammonia-Nitrite-Nitrate-DO)

d) Sequential acting variables with feedback interaction - (Nitrification as a sequential reaction with algae utilization providing the feedback interaction through death of algae)

e) Complex interreacting systems - non-linear kinetics (Phytoplankton growth, nutrient utilization, sedimentation and sediment exchange)

Figure II-1 is a reaction diagram indicating the kinetic interrelationships for a number of sequential reactions, which have significance in water quality management. In particular, the BOD-DO reactions are indicated together with the Ammonia-Nitrite-Nitrate sequential reaction system. For the

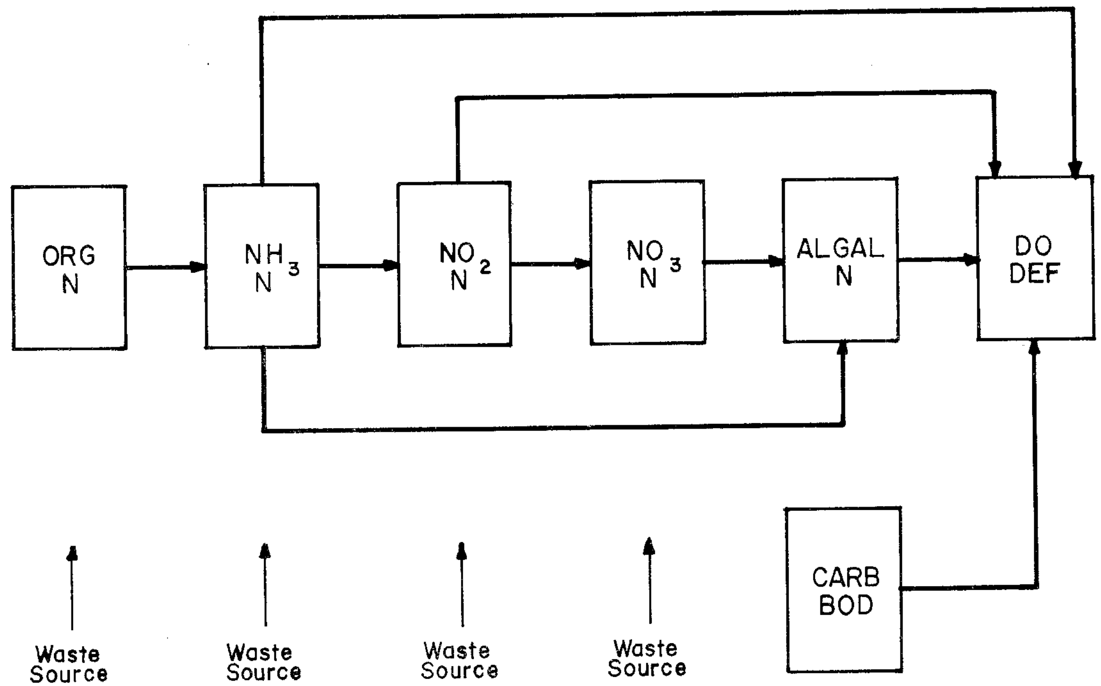

FIGURE II-1

SCHEMATIC OF THE
DISSOLVED OXYGEN AND NITROGEN SYSTEM

Nitrogen cycle, the growth of algae utilizes nitrogen and subsequent cell death returns the nitrogen in the organic form for subsequent feedback into the nitrogen system. Figure II-2 indicates a kinetic pathway diagram for the interacting system which can represent phytoplankton growth, multiple nutrient utilization, predation, and control of algae growth by physical factors such as light penetration. These kinetic diagrams are applicable to each physical section of a water quality system.

While the kinetic specifications indicated above are not all inclusive, they deal with a substantial number of constituents and water quality problems. It should by pointed out that a lack of knowledge of the significant mechanisms is one of the most important impediments to understanding and subsequent modeling of many water quality problems.

The range of values or the reaction coefficients employed in the kinetics specifications, should be compared to the range of literature values available on the same reaction sequence. If the modeling effort requires values of the reaction coefficients, which are substantially different from those found under controlled laboratory conditions, the model output should be carefully reviewed to determine the reasons for any descrepancies.

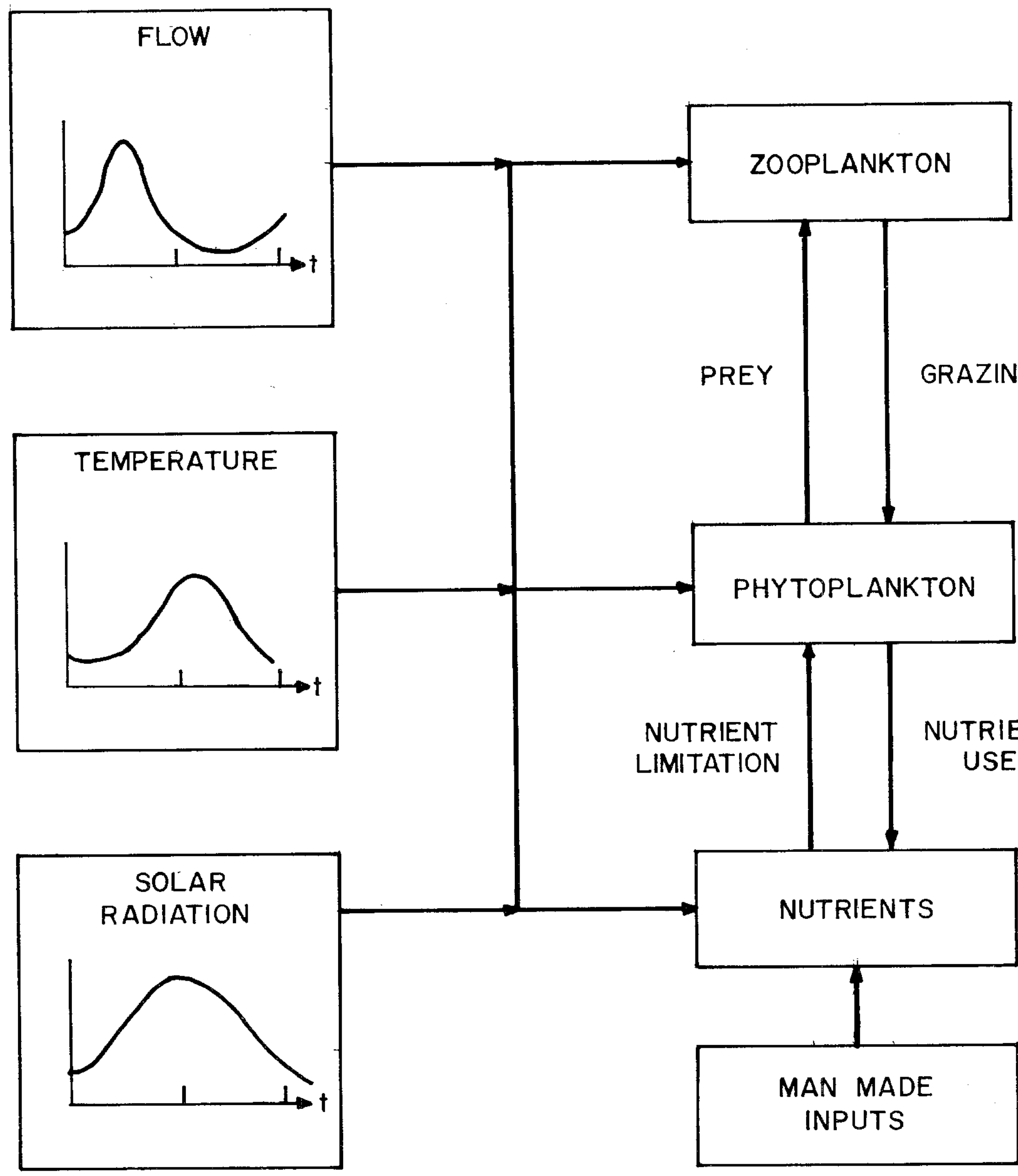

FIGURE II-2

INTERACTIONS: ENVIRONMENTAL VARIABLES AND THE ZOOPLANKTON, PHYTOPLANKTON AND NUTRIENT SYSTEMS

F. Verification Analysis

The model coefficients as, for example, reaction and dispersion coefficients, which represent the particular characteristics of the body of water being considered are evaluated based on a set of observed concentrations. The interplay of the calculated model output and the available data produces a set of the unknown coefficients which describe the various physical, chemical, and biological processes involved. In addition to the water quality variables of interest, it is often prudent to investigate the distribution of other variables for which adequate data is available, as an additional verification of the model. For example, in estuaries, the salinity distribution is usually considered even if no water quality significance is attached to salinity. By comparing the mathematical model output against the observed salinity distribution, the hydrodynamic and dispersive parameters which are employed in the model can be evaluated. A major portion of the effort in constructing a mathematical model of water quality for a body of water is devoted to verification and establishment of the veracity of the model. Although this is a difficult and time-consuming task, it is the only way to establish the necessary validity which renders the model useful for planning purposes. The procedure usually followed is:

a) Examine model output, using the coefficients established and compare to a different set of data. (Higher flow, lower temperature, different loading conditions).

b) Check goodness of fit of the observed data to computed values.

c) Re-evaluate any changes in coefficients and repeat the procedure.

d) The result is a model that should resonably reproduce observed data with an internally consistent set of coefficients.

The above verification procedure is usually quite adequate for steady-state water quality models. When considering non-steady-state, time dependent water quality models, several additional steps in the verification are usually required. With respect to hydrodynamic modeling it is desirable to compare hydrodynamic model output with observed data for both stage and velocity patterns. Some water quality systems are most sensitive to these velocity fluctuations and patterns. With regard to non-steady-state water quality models, it is prudent to carry out verification procedures for these models by comparisons of calculated and observed data sequentially obtained at t=0, t=1, t=2, ... t=n.

Particular effort should be devoted to insure that model verification includes examination of observed data which were collected in accordance with the critical time and space scale of the water quality problem being examined. As an example, if seasonal treatment is being examined, employing a non-steady-state water quality model, model verification should proceed through the season which is critical to the water quality problem.

1. Sensitivity to System Parameter Verification

During the verification procedure, it is usually the case that certain parameters of the model have a more drastic effect on the model predictions than other parameters. From the point of view of data collection and data analysis, it is critical to know which mechanisms are the most important in the sense that the system is the most sensitive to the magnitude of the coefficients which describe these mechanisms. For the verified model as determined above, the coefficients should be varied in a systematic way to determine system response. For example, BOD-DO system response is linear to loads but non-linear to reaction coefficients. System response sensitivity must be obtained by trial and error.

G. Analysis of Control Alternatives

The primary utility of mathematical models that have been verified is in analysis and evaluation of environmental control procedures to achieve specific water quality objectives. Thus, the results of applying the verified mathematical model can indicate the degree of treatment required at a specified treatment plant and can indicate the constituents which should be removed to meet water quality objectives. As an example, from a dissolved oxygen standpoint, it is possible to trade off very high removals of carbonaceous oxygen demanding material (95% to 98% removal of BOD - which might require carbon adsorption) with nitrification of the effluents on either a continuing or seasonal basis.

The mathematical models for water quality can be employed in an attempt to assess changes, which would occur in the future, in the type of water quality problem to be encountered in a given area. As an example, it is conceivable that an existing dissolved oxygen problem may be replaced, after installation of adequate treatment, by a significant problem in dissolved oxygen from diurnal swings created by oxygen production and respiration of plankton algae. An analysis of the nutrients, light penetration, hydrodynamics, and other characteristics of the system might indicate the potential for algae problems.

The modeling effort should be considered as a continually evolving process which interfaces with data collection and observation of environmental conditions and with the construction and operation of facilities. For example, preliminary modeling can provide information for initial decisions on plant location and construction. Following the installation and operation of treatment facilities, additional observed data can be collected on the receiving water and a comparison made of the observed conditions and those projected by the modeling effort. The model can then be refined and if necessary, directed towards other water quality problems and the procedure repeated, with the modeling effort indicating the need for construction of facilities. The modeling effort is therefore consistent with the viable ongoing aspect of the planning process.

H. Summary Discussion

There are classes of water quality problems which are amenable to a straightforward analysis. Varying degrees of prototype complexity can then be built into the planning effort in a sequential fashion. Examples are found in streams and estuaries which are characterized by relatively simple geometry and which have water quality problems associated with conservative constituents or water quality variables, such as dissolved oxygen. On the other hand, when the stream or estuary geometry is highly complex or requires a two-dimensional analysis, modeling efforts become significnatly more difficult and are usually not amenable to simplified models. In addition, as the water quality problems shift from phenomena which are well understood, such as nitrogenous oxidation, dissolved oxygen, TDS, and simple bacteriological systems to more complex problems associated with eutrophication, algal growth, and accumulation in the food chain, it is usually necessary to start with a relatively complex water quality modeling effort. The implication of this complex water quality modeling effort is a substantial increase in the cost required for a rational problem analysis and the cost associated with the collection of observed information necessary for verification of the water quality model.

With respect to lakes and impoundments, it is usually the case that detailed and complex water quality models are required to address problems characteristic of these bodies of water. There are instances in lakes and impoundments where the water quality problems of concern are of a very localized nature around waste discharges or in the vicinity of the entrances of streams. In these instances, it may be practical to consider a simplified steady-state analysis. However, where the problem in lakes and impoundments is that of present or future eutrophication, it will be necessary to develop a significantly complex modeling effort to address water quality problems in a rational manner.

III. PRELIMINARY MATHEMATICAL MODELING FOR INTERIM PLANNING

A. Conditions and Assumptions

The preceding section has outlined the components of mathematical modeling conducted as part of detailed planning activities. For certain situations, however, it may be necessary to forego a detailed construction of a mathematical model of the water body. Rather, a preliminary "first-trial" approximation of system responses and subsequent required treatment level suffices. A preliminary analysis is justified on the grounds that either the problem context is such as to preclude a detailed time consuming and expensive modeling effort or it is desired to conduct "screening" analyses which will indicate areas where more detailed work may be required.

One reason for constructing a mathematical model is to provide information on the adequacy of proposed treatment measures for achieving water quality standards as discussed in Section II-G above. However, the output from a preliminary mathematical model analysis should <u>never</u> be interpreted as a certainty but as a guide to be used in conjunction with the analysts' engineering judgement.

The stream and estuary models discussed below, generally proceed from analyses based on the minimum available information to those analyses based on detailed data on waste discharges, river geometry or reaction coefficients. Thus, the analyses for dissolved oxygen responses in streams can be carried out by knowing only the design population, drainage area, maximum water temperature and depth of the stream. In the absence of data on the depth, a qualitative description of the river can be used. It should, of course, be recognized that the degree of assurance in the validity of the outcome of the analysis increases as the amount of information and data increases. The objective in all of the analyses discussed below is to present preliminary models that can draw on all of the data the analyst has at hand. In the absence of certain key input information, alternate input data are supplied to the analyst. (Refer to Section III-C -- Technical Data Required for Analysis.)

As with all preliminary or simplified analyses, it is important to recognize and understand the conditions and assumptions which underlie the approach. In the analyses which follow, several such key assumptions have been made, including:

1. Temporal steady-state, i.e., all system parameters and inputs are assumed constant in time;

2. Constant spatial system parameters, i.e., characteristics such as river flow, deoxygenation and reaeration coefficients, and tidal dispersion, are assumed constant with distance along the flow axis.

3. Only point waste load inputs are included.

4. Uncertainties in the analyses of various constituents are incorporated to some degree in a "background" or natural quality.

5. The point waste load inputs represent residual waste from a range of discrete, as opposed to continuous, treatment levels.

The user of the simplified models should recognize the consequences of each of these assumptions in interpreting the results of the analysis. The temporal and spatial steady-state assumptions (#1 and #2) mean that no information will be obtained on the time-variable behavior of the water body. Thus, for example, if there is a significant diurnal variation in waste load, the preliminary analysis will not be able to describe the resulting diurnal variation in water quality. However, for many problem situations, primary interest centers on the worst or critical conditions. In the diurnal waste load example just mentioned, the analyst may choose to input the maximum daily load in the steady-state model recognizing that the water quality responses will be improved at the lower daily levels of the waste load. The spatial steady-state means that changes in cross-sectional area or depth, for example, are not incorporated directly. The analyst must use some judgement in choosing representative depths, geometry, or river flows for use in the preliminary mathematical model. The estimate of the average water depth under the low flow condition (such as the seven day-ten year low flow) should be estimated for the analysis.

The third assumption is usually well justified, since the majority of water quality problems result from point source waste discharges from municipal and industrial inputs. Distributed waste sources such as associated with land or urban runoff are not included directly, although their effects are allowed for in the background concentration. If the analyst has additional information on the order of magnitude of a distributed waste load, it can be incorporated as a series of point sources distributed at constant intervals along the length of the river. The multiple source analysis is then used.

For dissolved oxygen, other sources and sinks of DO are incorporated in a "background" DO deficit. Uncertainties in input information and analysis structure are therefore reflected in the water quality response as a "factor of safety" rather than at the waste source itself. Algal photosynthesis and respiration, benthal oxygen demand and distributed BOD sources are some of the oxygen sources and sinks that are considered to be embodied in the background deficit. Experience has indicated that for most problem contexts, a background dissolved oxygen deficit of about 1.0 mg/l is the usual case. In the absence of any data on dissolved oxygen in the river or estuary, this background deficit is suggested.

B. Mathematical Models

1. Background

This section describes the basic theory of mathematical modeling of one-dimensional water bodies. The equations are presented for the spectrum of situations extending from the stream to the large river, the tidal river and the saline estuary. The section therefore establishes the theoretical basis for the simplified models to be discussed in Parts IV and V.

Any natural body of water may be viewed as a mathematical system, composed of a number of complex interacting subsystems. The system receives, on the one hand, a series of external inputs such as rainfall, solar radiation, runoff, and winds which interact with the water body and its drainage basin to determine the natural background quality of the water. On the other hand, the system is subjected to a variety of man-made effects such as wastewater discharges, water diversions, and runoff from urban and land developments which also influence water quality. The response of the system to each of these inputs is the spatial and temporal distribution of the concentration of various substances which affect water use. Such substances include dissolved and suspended solids, various chemicals, dissolved oxygen, nutrients (nitrogen and phosphorus), bacteria and algae concentrations.

The system is composed of a number of elements, with physical characteristics and corresponding mathematical descriptions. Physically, the concentration of these substances is determined by the dispersion and advection characteristics of the water body and by the various physical, chemical, biological, or radiological reactions which affect the substance. Mathematically, the system is described by a set of partial differential equations, with variable coefficients, each term of which corresponds to one of the basic characteristics.

Fundamental to the analysis of the problem is the continuity equation. The point form of this equation in one-dimensional form is:

$$\frac{\partial c}{\partial t} = \frac{\partial}{\partial x} \text{ (flux) } \pm \sum S \qquad \text{(III-1)}$$

in which:

c	=	concentration of water quality variable
t	=	time
flux	=	$E\frac{\partial c}{\partial x} - Uc$
S	=	sources and sinks of the substance, c
U	=	velocity
E	=	dispersion coefficient

The continuity equation expresses a relationship between the flux of mass and the sources and sinks of mass. The flux is determined by the hydrodynamics of the system, which is related to the hydrology, meterology, and geomorphology of the areas. The term Uc is the flux due to advection by the fluid containing the mass. The term $E\frac{\partial c}{\partial x}$ is the flux commonly ascribed to dispersion in the "x" direction. The flux due to dispersion is assumed to be proportional to the gradient concentration in the direction of decreasing concentration. Any mass of pollutional substance is transferred by this mechanism from a zone of high concentration to one of low concentration. Many factors contribute to the total spread of mass:

turbulent diffusion, velocity gradients, tidal effects, and density differences.

While the flux term defines the material moving in and out of an elemental volume, the term S represents the sum of the various sources and sinks of material within the volume. Characteristic sources and sinks are reactions of a physical, chemical, or biological nature which occur in natural waters. Many of these reactions may be represented by first-order kinetics, i.e., the rate of the reaction is proportional to the concentration of the substance. The rate coefficient is usually identified as K. Although not necessarily fundamentally correct, this kinetic order is a practical approximation to many reactions in natural waters.

2. Steady-State Analysis

A form of Equation (III-1) which is appropriate for the one-dimensional analysis of a non-conservative substance in streams, rivers, and estuaries, is:

$$0 = E\frac{d^2c}{dx^2} - U\frac{dc}{dx} - Kc \qquad \text{(III-2)}$$

The equation is expressed in the steady-state form, which is characteristic of many regions of the country during late summer or early fall. At this time, when the freshwater flow is low and the temperature high, the most severe water quality con ditions usually occur. The reaction coefficient, K, is descriptive of the particular substance under consideration. The velocity, U, is due to the freshwater discharge. The tidal velocity is not included in Equation (III-2), implying that water quality conditions are at either high or low water slack, or some suitable tidal average which defines the limits of quality variation at a point in space.

The coefficient, E, refers to the longitudinal dispersion. It is most significant in the saline portion of the estuary where a number of factors contribute to the intrusion of the salt up the estuary. The concentration of other substances, which are of concern in water quality in estuaries, is affected in a manner similar to that of the salt. In the tidal, but non-saline sections of the river, the dispersion — although not as pronounced as in the saline section — is still a significant factor in the analysis of water quality. Upstream of the tidal influence, the effect of longitudinal mixing is much less and in many cases may be disregarded, particularly in the upstream freshwater tributaries and feeder streams.

The solution of Equation (III-2) for a continuous rate of waste discharge of W into a river system with a flow rate of Q is:

$$c = c_o e^{gx} \qquad x \leqslant 0 \tag{III-3a}$$

$$c = c_o e^{jx} \qquad x \geqslant 0 \tag{III-3b}$$

in which:

$$j = \frac{U - \sqrt{U^2 + 4KE}}{2E}$$

$$g = \frac{U + \sqrt{U^2 + 4KE}}{2E}$$

and

$$c_o = \frac{W}{A\sqrt{U^2 + 4KE}}$$

These equations may be re-expressed in two alternate forms. In one case, the advective or freshwater component of the flux is emphasized and the initial concentration and the exponents are:

$$c_o = \frac{W}{AU\sqrt{1 + 4KE/U^2}} = \frac{W}{Qm} \tag{III-4a}$$

$$m = \sqrt{1 + \frac{4\ KE}{U^2}}$$

$$Q = AU$$

$$g = \frac{U}{2E}\,[1 + m] \qquad \text{(III-4b)}$$

$$j = \frac{U}{2E}\,[1 - m]$$

In the second case, the dispersive component usually associated with tidal mixing is predominant:

$$c_o = \frac{W}{A\sqrt{4KE}\,\sqrt{1 + U^2/4KE}} = \frac{W}{Rp} \qquad \text{(III-5a)}$$

$$p = \sqrt{1 + U^2/4KE}$$

$$R = A\sqrt{4KE}$$

$$g = \frac{U}{2E} + p\frac{\sqrt{K}}{\sqrt{E}}$$

$$j = \frac{U}{2E} - p\frac{\sqrt{K}}{\sqrt{E}} \qquad \text{(III-5b)}$$

Equations (III-4a) and (III-4b) highlight the advective flux; the product UA is the freshwater flux, Q. As one moves upstream from the tidal river into the freshwater stream, the dispersion coefficient decreases, the velocity becomes more pronounced, and the parameter m approaches one, and the initial concentration equals W/Q. On the other hand, as one moves downstream into the saline tidal stretches of the river, the dispersion is more significant, the freshwater velocity less, the parameter "m" approaches infinity, the parameter p approaches one, representing the equations of the dispersive system, as shown by Equations (III-5a) and (III-5b). The term, $R = A\sqrt{4KE}$, which has the same units as the freshwater flow, may be regarded as the dispersive flux of a non-conservative substance.

It is therefore both convenient and practical to classify one-dimensional river systems into three categories: on one end of the spectrum the freshwater stream, in which the advective component is the significant flux, and on the other, the estuary, in which the dispersion is usually predominant. The intermediate range is identified as the river, which may be either tidal or non-tidal. The appropriate differential equations for a non-conservative substance in each of these systems are as follows:

Stream: $$0 = U\frac{dc}{dx} - Kc \qquad \text{(III-6)}$$

River (Tidal + non-tidal) $$0 = E\frac{d^2c}{dx^2} - U\frac{dc}{dx} - Kc \qquad \text{(III-7)}$$

Estuary: $$0 = E\frac{d^2c}{dx^2} - Kc \qquad \text{(III-8)}$$

It is possible to delineate quantitatively between these systems by means of a dimensionless analysis of Equation (III-2), which may readily be converted to the following form:

$$0 = n\frac{d^2\Gamma}{d\S^2} - \frac{d\Gamma}{d\S} - \Gamma \qquad \text{(III-9)}$$

In which:

$$\Gamma = \frac{c}{c_o}$$

$$\S = \frac{Kx}{U}$$

$$n = \frac{KE}{U^2}$$

The solution of Equation (III-9) is similar to that of Equation (III-2):

$$\Gamma = e^{\left(\frac{1 - \sqrt{1 + 4n}}{2n}\right)\S} \qquad \text{(III-10)}$$

The significant dimensionless number is $n = KE/U^2$. One limit, $n \rightarrow \infty$, is characteristic of dispersive systems or tidal systems: high dispersion with zero advection. More specifically $n \rightarrow \infty$ represents a system in which the tidal dispersion is so great by contrast to the freshwater advection that the latter may be neglected. The other limit, n=0, characterizes advection systems or freshwater streams, in which the advection component is the significant flux term. Intermediate between these limits lie many rivers in which both factors are important. As an approximation, the following criterion may be used to delineate the limits of the freshwater advective-dispersive system. Identifying "a" as the fractional error between a freshwater advective model and a tidal dispersion model, the limits of the system, as established by n are:

$$a > n > \frac{1}{a} \qquad \text{(III-11)}$$

Values of n less than "a" are characteristic of advective system and values of n greater than 1/a are indicative of dispersive systems. For example, if the maximum permissible error is to be no greater than 10% (a = 0.1), the limits of "n" for the tidal river are 0.1 to 10, with the stream classified by $0 \leqslant n \leqslant 0.1$ and an estuary for $10 \geqslant n \geqslant \infty$.

3. <u>Coupled Reactions - Dissolved Oxygen</u>

The above discussion refers to substances which singularly decay in accordance with a first-order reaction or at least may be approximated by this kinetic expression, e.g., coliform bacterial die-away, or oxidation of organic matter.

In considering substances such as dissolved oxygen, however, the sequence of two reactions must be taken into account. In the following analysis, the photosynthetic contribution is assumed to be balanced by the respiration by the aquatic plants. If allowance is to be made for this factor, it conservatively should be incorporated on the negative side, i.e., as a deficit, based on those periods during the year when death and respiration are greater than the photosynthetic contribution.

The appropriate differential equation for the dissolved oxygen deficit analysis is:

$$0 = E\frac{d^2D}{dx^2} - U\frac{dD}{dx} + K_d L_o e^{J_d x} - K_a D \qquad \text{(III-12)}$$

whose solution is:

$$D = \frac{K_d L_o}{K_a - K_d} [e^{J_d x} - \frac{m_d}{m_a} e^{J_a x}] \qquad \text{(III-13)}$$

in which:

D	=	dissolved oxygen deficit
L_o	=	biochemical oxygen demand at x=0
K_d	=	deoxygenation coefficient

$$K_a = \text{reaeration coefficient}$$

$$m_d = \sqrt{1 + \frac{4K_dE}{U^2}}$$

$$m_a = \sqrt{1 + \frac{4K_aE}{U^2}}$$

$$J_d = \frac{U}{2E}\ [1 \pm m_d]$$

$$J_a = \frac{U}{2E}[1 \pm m_a]$$

$$L_o = \frac{W}{A\sqrt{U^2 + 4K_dE}}$$

The deficit, as defined by Equation (III-13) produces a typical sag in the dissolved oxygen concentration. The minimum concentration or maximum deficit is classically identified as the critical point, whose location is defined by:

$$X_c = \frac{1}{J_d - J_a} \ln \left(\frac{J_a}{J_d} \cdot \frac{m_d}{m_a}\right) \tag{III-14}$$

Substitution of Equation (III-14) in Equation (III-13) and simplification yields:

$$\frac{D_c}{L_o} = \frac{(\alpha_j - 1)}{\alpha_j(\Phi - 1)}\left[\frac{\alpha_j}{\alpha_m}\right]^{\frac{1}{1 - \alpha_j}} \tag{III-15}$$

in which:

$$\Phi = \frac{K_a}{K_d}$$

$$\alpha_m = \frac{\sqrt{1 + 4\Phi n_d}}{\sqrt{1 + 4n_d}}$$

$$\alpha_j = \frac{1 - \sqrt{1 + 4\Phi n_d}}{1 - \sqrt{1 + 4n_d}}.$$

Thus, the ratio of the critical deficit to the biochemical demand at x=0, is defined by two dimensionless parameters:

$$\Phi = \frac{K_a}{K_d}$$

$$n_d = \frac{K_d E}{U^2}$$

Equation (III-15) is shown graphically in Figure (III-1) which plots the ratio D_c/L_o against the parameter $\frac{K_d E}{U^2}$ for various values of Φ. It represents the spectrum of one-dimensional systems: freshwater streams to saline estuaries, with the intermediate range of rivers, which may be either tidal or non-tidal. Delineation between these systems is shown by the approximation suggested previously, i.e., $n < 0.1$

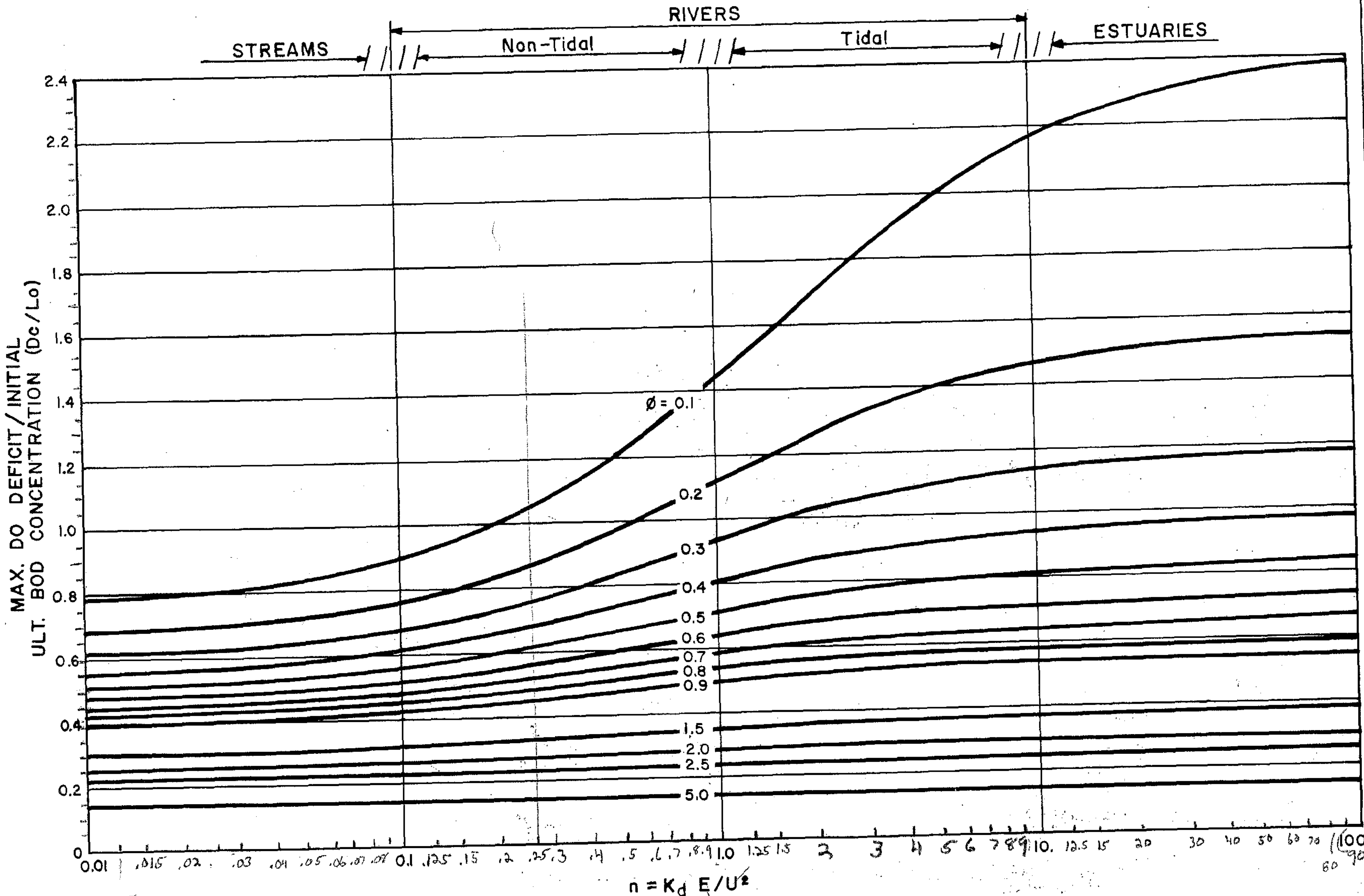

FIGURE III-1
DISSOLVED OXYGEN RESPONSE AS A FUNCTION OF $K_d\ E/U^2$

defines a freshwater advective system, in which the dispersion may be neglected and $n > 10$, a saline dispersive system, in which the advection may be neglected. The former applies to the upstream branches and tributaries of large river systems and the latter to the tidal saline mouths at the ocean.

It is emphasized that the lines of demarcation are approximate and depend on the degree of accuracy desired and on the relative magnitude of the four coefficients: the two flux coefficients, U and E and the two reactive coefficients, K_d and K_a. Thus, a stretch of river may, during the high flow season, be classified as a stream (high U) but in the low flow period, as a river (low U). This is shown in Figure III-1. Further, the same stretch for a given hydrodynamic condition may be classified differently in accordance with the reaction coefficients: highly reactive substances (high K) tend to the estuarine model, while slowly reacting (low K) substances lend themselves more appropriately to the stream model. In any case, when applied to the dissolved oxygen analysis during the steady-state low flow period, the functional plot with the delineations may be accepted as reasonable approximations, at least to establish an order of magnitude for the vast majority of river systems in this country.

C. Technical Data Required for Analysis

This section consists of a technical description of the various factors which are of importance in a water quality analysis of streams, rivers, and estuaries. Those factors which determine the quantity of waste flow discharged from the treatment plant to the river are reviewed. In addition, the geophysical characteristics of the drainage area, and the receiving water body, are discussed, which, in conjunction with the waste flow discharged, establishes the concentration of the particular substance under consideration. The quantity of waste depends on the size of the population, the per capita water usage and waste quantities and the removal efficiency of the particular waste treatment process. Ranges of efficiency are readily established for the secondary and advanced levels of waste treatment.

The geophysical characteristics include the temperature, the natural background water quality, the structure and shape of the drainage system and the flow and dispersion of the river and the estuary. The freshwater flow is a function of the rainfall, and the characteristics of the drainage basin which may be generalized for different sections of the country. The dispersional features, which are determined by the tidal parameters and geomorphology of the particular estuary are somewhat more difficult to generalize. Some guidelines can be given which are useful for preliminary analysis.

1. Waste Discharges

a. Population

The design population for a particular community is established by the assumed or given growth rate of the area and a pre-determined design period. Projected growth rates vary for the different regions of the country and reflect primarily the economic projections of the areas. The nature of the community (urban, suburban, rural) the extent and type and diversity of industrialization are also conditions which affect the rate of population growth. This factor is particularly important because relatively small differences in growth rate may result in population and waste flow differentials sufficient to require a change in the level of waste treatment for a particular community. Table III-1 presents a range and a suggested value for different rates of growth which may be anticipated throughout the country.*

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* Appendix B contains some growth projections for various regions of the country considering metropolitan and rural areas.

TABLE III-1

POPULATION GROWTH FACTORS

Area Description	Growth Factor (f_1)	% Increase Range
Established Static	1.10	0 - 25
Low to Moderate	1.50	25 - 75
Moderate to High	2.00	75 - 125
Rapidly Developing	2.50	125 - 375

Thus, the design population is simply the present population multipled by the growth factor applicable to the community or region:

$$P_d = f_1 P_o \tag{III-16}$$

b. Per Capita Waste Flows and Quality

The per capita waste flows and waste quality from municipalities varies in accordance with many of the same factors which affect population growth: the nature of the community, the extent of industrialization, and the characteristics of the region. In addition, the economic status of the area and hydrological features of the region are contributing factors in different ways to the quantity and strength of per capita contributions. Table III-2 presents some guidelines for per capita waste flows and the more common waste constituents.

TABLE III-2

PER CAPITA WASTE FLOWS AND QUALITY

	$\frac{\text{gallons}}{\text{cap-day}}$	Ultimate BOD $\left(\frac{\text{pounds}}{\text{cap-day}}\right)$ C	N	Suspended Solids $\left(\frac{\text{pounds}}{\text{cap-day}}\right)$	Nutrients $\left(\frac{\text{pounds}}{\text{cap-day}}\right)$ N	P	Coliform $\left(\frac{\text{MPN}}{\text{cap-day}}\right)$ $\times 10^9$
Low	100	0.12	0.07	0.15	0.015	0.003	50
Average	175	0.25	0.15	0.30	0.035	0.006	100
High	300	0.40	0.28	0.50	0.055	0.010	150

A "typical" municipal waste falls between the first two categories with a daily per capita flow of about 125 gallons and an ultimate carbonaceous and nitrogenous BOD of about 0.20 and 0.12 pounds per capita day, respectively. The higher per capita waste flows represent large amounts of infiltration or industrial flow. The waste flow and mass rate of discharge are therefore:

$$q = f_2 P_d = f_2 f_1 P_o \qquad \text{(III-17)}$$

in which:

q = wastewater flow (usually million gallons/day, MGD)

f_2 = per capita flow contribution (gallons/capita-day)

Also;

$$I = f_3 P_d = f_3 f_1 P_o \qquad \text{(III-18)}$$

in which:

I = mass rate of waste constituent = influent to treatment plant (pounds/day)

f_3 = per capita contribution (pounds/capita-day)

c. Treatment Efficiencies and Residuals

For purposes of this report, treatment levels have been categorized into groups which may be qualitatively described as follows:

1. Marginal Secondary

Conventional secondary treatment systems which are overloaded - upset periodically or poorly operated (range $\pm$ 25% carbonaceous BOD residual).

2. **High Rate Biological**

Conventional secondary treatment systems with proper operation (range ± 15% carbonaceous BOD residual).

3. **Secondary with Nitrification**

Biological treatment for removal of carbon and for nitrification (range ± 10% carbonaceous BOD residual).

4. **Advanced**

Biological or physical chemical treatment for carbon, nitrogen, and phosphorus removal. Filtration and activated carbon treatment of the effluent (range ± 5% carbonaceous BOD residual).

5. **Ultimate**

Technology not yet applied to meet this requirement on a sustained basis.

With these discrete levels and the following assumptions, guidelines can be developed for the overall efficiency of treatment. These efficiency guidelines are summarized in Table III-3 for ultimate oxygen demand, and Table III-4 for phosphorus and total nitrogen. The assumptions used for Tables III-3 and III-4 are: a) 125 gallons/capita/day, b) 0.25 pounds ultimate carbonaceous BOD/capita-day, c) 0.20 pounds ultimate nitrogenous BOD/capita-day, d) 0.025 pounds PO_4/capita-day, e) .042 pounds nitrogen/capita-day.

TABLE III-3

ESTIMATED EFFICIENCY OF TREATMENT LEVELS
ULTIMATE OXYGEN DEMAND

Treatment Level	% Removal		#/capita/UOD remaining			f_4 fraction UOD remaining
	C*	N+	C*	N+	total	
1. Marginal secondary treatment	70	10	.075	.18	.255	.56
2. High Rate Biological Treatment	85	20	.037	.16	.197	.44
3. Secondary treatment with nitrification	90	85	.025	.030	.055	.12
4. Advanced treatment	95	95	.013	.01	.023	.5
5. Ultimate treatment	99	99	.0025	.002	.0045	.1

C* - Carbonaceous BOD

N+ - oxidizable nitrogen

f_4 - residual fraction after treatment

TABLE III-4

ESTIMATED EFFICIENCY OF TREATMENT LEVELS
PHOSPHORUS AND TOTAL NITROGEN

Treatment Level	Phosphorus			Total Nitrogen		
	fraction removed	residual f_4	# PO_4 capita/day remaining	fraction removed	residual f_4	# N capita/day remaining
1. Marginal secondary treatment	.20	.80	.02	.10	.90	.038
2. High Rate Biological	.20	.80	.02	.20	.80	.034
3. Secondary Treatment With Nitrification	.20	.80	.02	.20	.80	.034
4. Advanced Treatment	.85	.15	.0037	.95	.05	.002
5. Ultimate Treatment	.90	.10	.0025	.99	.01	.0004

The efficiency guidelines in Tables III-3 and III-4 may be used to estimate the discharge waste load. More generally:

$$W = f_4 I = f_4 f_3 f_1 P_o \qquad \text{(III-19)}$$

in which:

W = mass rate of waste material discharged to the receiving water (pounds/day)

f_4 = residual fraction after treatment (Tables III-3 and III-4) = 1 - e

where

e = net efficiency of waste removal

2. Characteristics of Drainage Basin

a. Temperature

The most critical water quality conditions usually occur during the middle to late summer or early fall when the temperature is high. The temperature affects the solubility of many substances as well as the rate coefficients of many reactions. Temperature ranges of both waste and receiving water should therefore be determined from measurements in any particular case. In the absence of such data, Figure III-2 may be used to assign a design temperature condition. Temperature is also an important variable in dissolved oxygen analyses since, together with the salinity, it determines the saturation value of DO. Figure III-3 shows the variation of the saturation value with temperature.

b. Natural Background Quality

The natural environment and runoff determine to a great extent the background water quality, i.e., the quality of the water before it has been significantly affected by man's activities. Natural waters may vary appreciably in quality, depending on the characteristics of land and its usage, and the rainfall and runoff patterns of the area. Such information may be available from historical data or river surveys; if not, Table III-5 presents reasonable guidelines for some of the major constituents.

TABLE III-5

BACKGROUND WATER QUALITY

	D.O. Deficit (mg/l)	MPN/100 ml
Low	0.5	10
Moderate	1.0	100
High	2.0	1,000

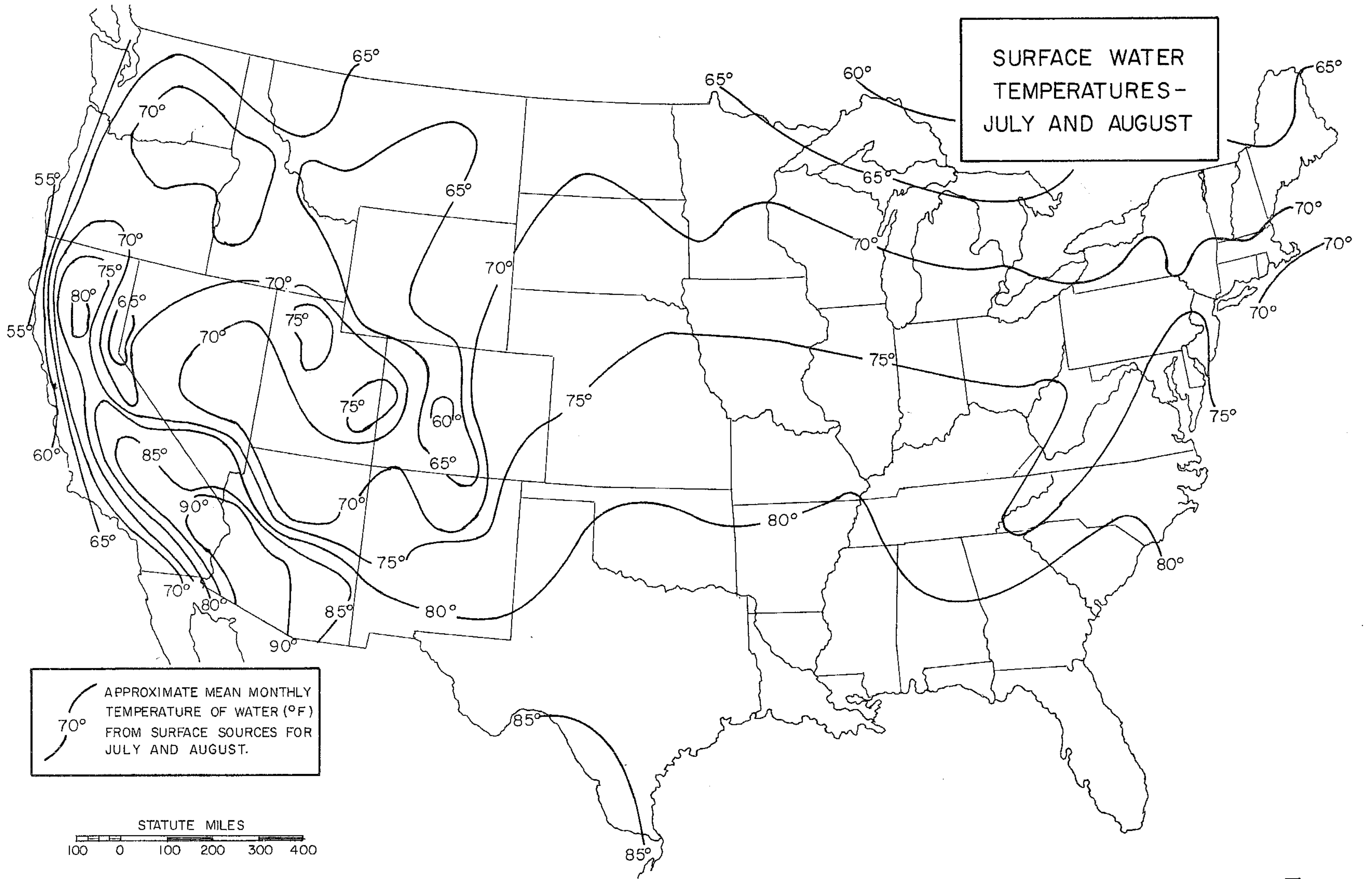

SURFACE WATER TEMPERATURES - JULY AND AUGUST
APPROXIMATE MEAN MONTHLY TEMPERATURE OF WATER (°F) FROM SURFACE SOURCES FOR JULY AND AUGUST.
STATUTE MILES
100 0 100 200 300 400
55°
60°
65°
70°
75°
80°
85°
90°

FIGURE III-2

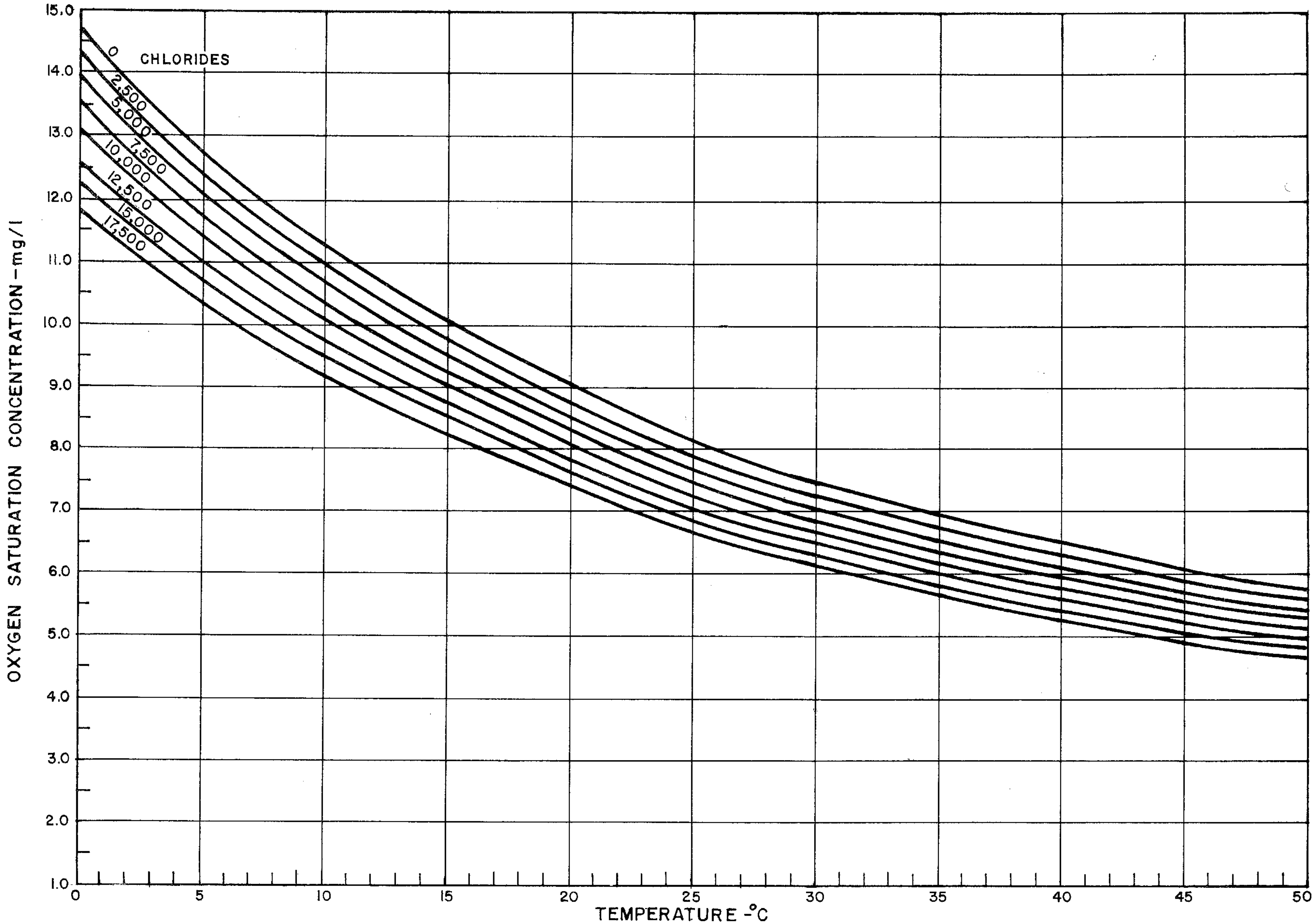


FIGURE III-3

SATURATION-TEMPERATURE-CHLORIDE RELATIONSHIPS

The high values of dissolved oxygen deficit and coliform are indicative of highly organic and swampy areas while the lower values are associated with the less productive mineral regions. The suspended solids concentration may vary considerably, both spatially and temporally being virtually absent in some rivers, to values of thousands of mg/l in others. In the preliminary models discussed in the next section, one must incorporate any natural background water quality plus the uncertainties in water quality responses due to the use of simplified models.

c. <u>Freshwater Flow</u>

The freshwater flow provides not only dilution of the waste flow but also determines the velocity of the stream, which, in turn, establishes the spatial influence of the waste discharge. Furthermore, this parameter affects many of the reaction coefficients. The flow varies considerably over the year and from year to year. The low flow period usually occurs in late summer - early fall and in conjunction with the still high temperature that is characteristic of that time of the year, produce the most severe water quality conditions. For purposes of the simplified analyses described in this report, the design frequency is assumed to be in the order of a

10 or possibly 20 year drought of the minimum daily, weekly or monthly flow. In many areas of the country, statistical analyses of low flows have been performed by either federal or state agencies (usually the latter in conjunction with the U.S. Geological Survey). The drought flow may therefore be readily available for the particular stream in question.

If such an analysis is not available, the low flow may be estimated as follows: in the majority of cases, the annual average flow is tabulated in the USGS flow records. The ratio of both the mean drought to the average annual flow and the design drought to the mean drought generally falls within the range of 0.1 to 0.5, thus the ratio of the design drought to average annual has an approximate range of 0.01 to 0.25. Design drought is interpreted as above, i.e., 10-20 year return period for weekly to monthly minimum flows. Common values of 0.05 to 0.10 apply to unregulated and uncontrolled rivers along most of the eastern coast, in the northwest and certain sections of the midwest. The ephemeral streams of the southwest have zero natural flow during drought periods. The wastewater discharge therefore constitutes the only flow in the channel. In these areas, under severe drought conditions, this water may evaporate or infiltrate into the ground water, whose elevation is below the river channel bed.

Furthermore, if the average annual flow is not available, it may be estimated from records of adjacent or nearby drainage basins of similar characteristics, and adjusted by the drainage areas of the respective streams. Typical values for average annual flows and the seven day-ten year low flows in cfs/sq.mi. are presented in Table III-6 for several states, together with average estimates of the percent ratio of the seven day-ten year low flow to the annual average flow. The local office of the U.S.G.S. may also be consulted for estimation of low flows.

TABLE III-6

TYPICAL LOW FLOW DATA

State	Annual Flow (cfs/Mi^2)	7 Day-10 Year Low Flow (cfs/Mi^2)	Average Ratio (%) 7 Day-10 Year / Annual Flow
Iowa	.407	.0155	2.5
Missouri	.81	.0615	4.6
Tennessee	1.58	.1400	3.5
Virginia	1.0	.0468	2.5
Montana	1.170	.118	6.7
Wyoming	.935	.118	7.7
Michigan	.765	.186	6.7

Note: In most cases, the local office of the United States Geological Survey is able to provide flow records for rivers within their jurisdiction. These records usually provide information sufficient to establish flow conditions within the context of the model.

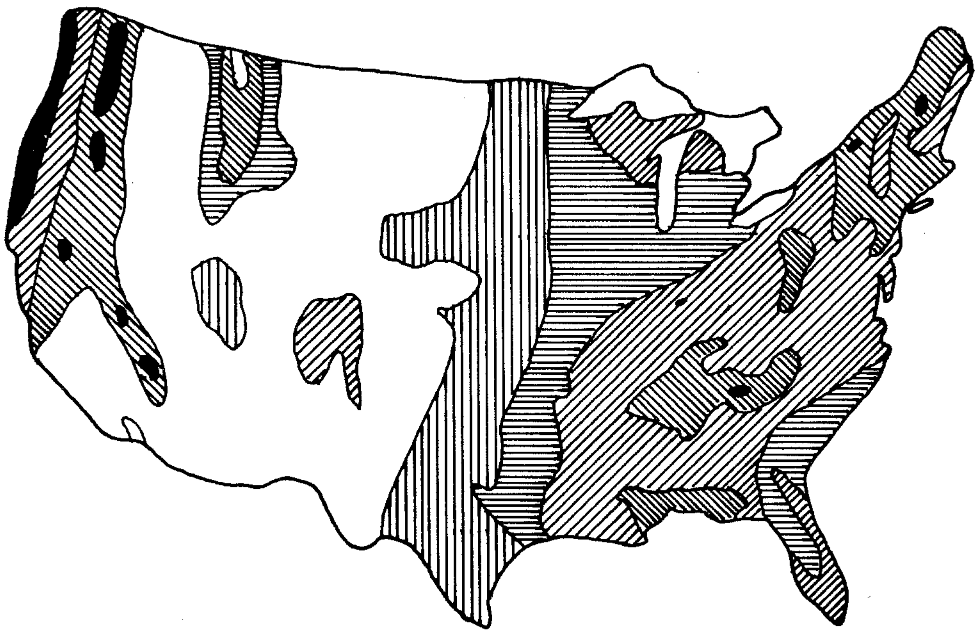
Figure III-4, abstracted from USGS publications, is included to indicate orders of magnitude of annual average flows in various parts of the country. The flow expressed in cubic feet per second per square mile of drainage area is multiplied by the drainage area at the location of the wastewater discharge to obtain the flow in the river. As indicated above, for ephemeral streams or those in the arid or semi-arid regions of the country, the stream flow is the wastewater flow. The total flow of the stream in cfs at the point of discharge is therefore:

$$Q = f_5 DA + 1.54\ f_2 f_1 P_o \qquad \text{(III-20)}$$

where f_5 is cubic feet per second per square mile of drainage area, DA in square miles. The factor 1.54 converts MGD to cfs.

Chart A is a nomograph which can be used to estimate the total river flow for a given drainage area, population and runoff coefficient.

The drought flow used in the water quality analysis, may be specified by low flow regulation. Furthermore, in the analysis, account must be taken of the source of water supply, whether ground or surface; if the latter whether from the same or different river and appropriate allowance made for this flow.



EXPLANATION
IN CUBIC FEET
PER SEC. PER SQUARE MILE

<.08

.8 to 1.6

.08 to .4

1.6 to 3.1

.4 to .8

3.1 to 6.2

FIGURE III-4

AVERAGE RUNOFF IN THE CONTERMINOUS UNITED STATES

REF. U.S.DEPT. OF INTERIOR

d. Characteristics of River Channel and Bed

A qualitative description of the river is helpful in evaluating many of the factors which affect water quality. This description should classify and describe the river with respect to its location in the basin and the nature of its flow and in particular, the nature of the channel bed and water surface, if possible. With respect to the first category, the stream should be identified as follows: a) the small, fingertip creeks which are the upstream limits, b) the feeders and c) tributaries to major drainage outlets. This classification could roughly be identified with drainage areas of a) 50 square miles or less for the upstream limits, b) 50 - 500 square miles for the feeder streams and c) 500 - 5000 square miles for tributaries. The fourth category of 5000 - 50,000 miles covers most of the major drainage systems in the country. (There are approximately 10 rivers in this country with drainage areas larger than 50,000 square miles.) An approximate estimate of the slope of the river (common range of 0.1 - 1.0 - 10 feet/1000 feet with limits of 0.1 and 100) is desirable, and may be readily obtained from contour maps of the area. A physical description of the river and the basin with some geological information will also be of value.

A visual inspection of the river is most helpful in defining the surface and bed conditions. The latter should be described at least with respect to conditions ranging from a fixed stable, rocky bed to unstable alluvial sandy, silty beds. The stable rock bed is conducive to relatively high oxidation coefficients. If the water depth is of the same order as the average diameter of rocks or the roughness elements, high aeration rates also exist. The movable silty bed has less opportunity to develop a stable benthic population, and with lower roughness, both the oxidation and the aeration coefficient are of lower values. The surface conditions may be estimated as smooth glassy, unruffled reflective appearance (low oxygen transfer) to the turbulent, wavy, broken, white water appearance (high oxygen transfer).

e. Channel Geometry

The most important dimension of the river geometry required for the water quality analysis is the average depth at design drought flows. It represents the ratio of the surface area to the volume, over the length of river which is affected by the waste discharge. This dimension is most significant with respect to estimating many of the coefficients in the physical, chemical and biological reactions. If information cannot be obtained or is not available, approximate estimates should at least be made in line with the following water classications: creeks, 1 foot or less, and 1 to 2 feet; streams, 2 to 5 feet and 5 to 10 feet; rivers, 10 to 30 feet or greater. Assuming the depth is estimated or measured at a particular flow condition, the depth at the design frequency flow may be approximated by the following relation:

$$H \sim Q^m \tag{III-21}$$

where Q is the river flow and m is a coefficient characteristic of the area. The value of the exponent m, varies from 0.4 to 0.6 and applies on the average to free flowing streams of approximate rectangular cross-section, unaffected by backwater conditions.

Equation III-21 indicates that the flow and depth are related by a straight line on log-log paper. Plotting the measured depth and flow on log-log paper and drawing a straight line through this point at a slope given by m, permits the estimation of the depth at the drought flow.

If the stretch of the river under investigation falls within the backwater curve from a reservoir or constriction, the exponent in the above is reduced in proportion to the proximity to the control point. As a limit, in the immediate zone of a reservoir, the depth variation may be regarded as insignificant over a large range of flows ($m = 0$).

Although velocity data are not usually available, a reasonable effort should be made to estimate this parameter, particularly for the water quality analysis of multiple sources. If the velocity is known at one flow, it may be estimated for other flow conditions by a relation identical to that of the depth for free flowing streams. Thus, for channels which are approximately rectangular in cross-section:

$$U \sim Q^{1 - m} \tag{III-22}$$

where:

U = stream velocity

Similar arguments concerning flow through pool areas lead to the limit of linear correlation of velocity inflow (i.e., an exponent of 1.0 for the velocity-flow relation).

Estimates of depth, tidal velocity, and net velocity can usually be made more readily in the case of estuaries from U.S. Coast and Geodetic Survey maps or U.S. Corps of Engineers Navigation maps. Mean tidal depth may be approximated from the data on soundings and widths may be directly scaled, the product of which establishes the cross-sectional area. The net downstream velocity is then given by the freshwater flow (as discussed in c. above) divided by the mean tidal cross-sectional area. Estimates should also be made of the maximum or average tidal currents which are usually available from similar sources. These estimates aid in determining the degree of tidal mixing. Again, exact values of the tidal velocity are not always necessary, but rather ranges of the following orders may suffice: a) relatively slow, constricted estuaries, 0.5 to 1.0 fps; b) moderate free flowing, 1 to 2 fps, and c) rapid, highly tidal, 2 to 3 fps and greater. The approximate ranges of salinity should be specified. In mg/l chlorides, the following ranges are useful: a) greater than 10,000 mg/l, b) moderate 1,000 to 10,000 mg/l; c) low salinity, less than 1,000 mg/l. In coastal plain estuaries of the east coast, salinities extend into the river channel between 50 and 100 miles under low flow conditions.

f. Dispersion Coefficients

Although dispersion is evident in some freshwater streams, it may usually be neglected in a water quality analysis of the type covered in this report. However, in tidal bodies, this effect must be incorporated in the analysis. The inclusion of dispersion is in essence accounting for the mixing and translation of the tides. Equation (III-2) shows the basic equation that incorporates the dispersion effect through the coefficient, E, usually in square miles/day. The effect is sufficiently significant in many estuaries as to render the effect of freshwater flow insignificant. The range of conditions to be expected in natural water bodies is discussed in detail in Section III-B above. The classifications given in Equations III-6, III-7, and III-8 are particularly appropriate. Further, the mixing effect is a function of the density characteristics associated with the heavier saline water from the ocean by contrast to the lighter freshwater from the rivers. Thus, the magnitude of the dispersion coefficient is relatively large in the vicinity of the mouth of the estuary where both salinity and tidal effects are great. It decreases in the upstream direction with decreasing salinity and tides. It is further reduced in the non-saline but tidal section of the estuary but is still of sufficient magnitude to be taken into account in conjunction with the freshwater flow. Typical values are shown in Figure III-5. Table C-2 in Appendix C shows a variety of dispersion coefficients as estimated for different tidal rivers and estuaries.

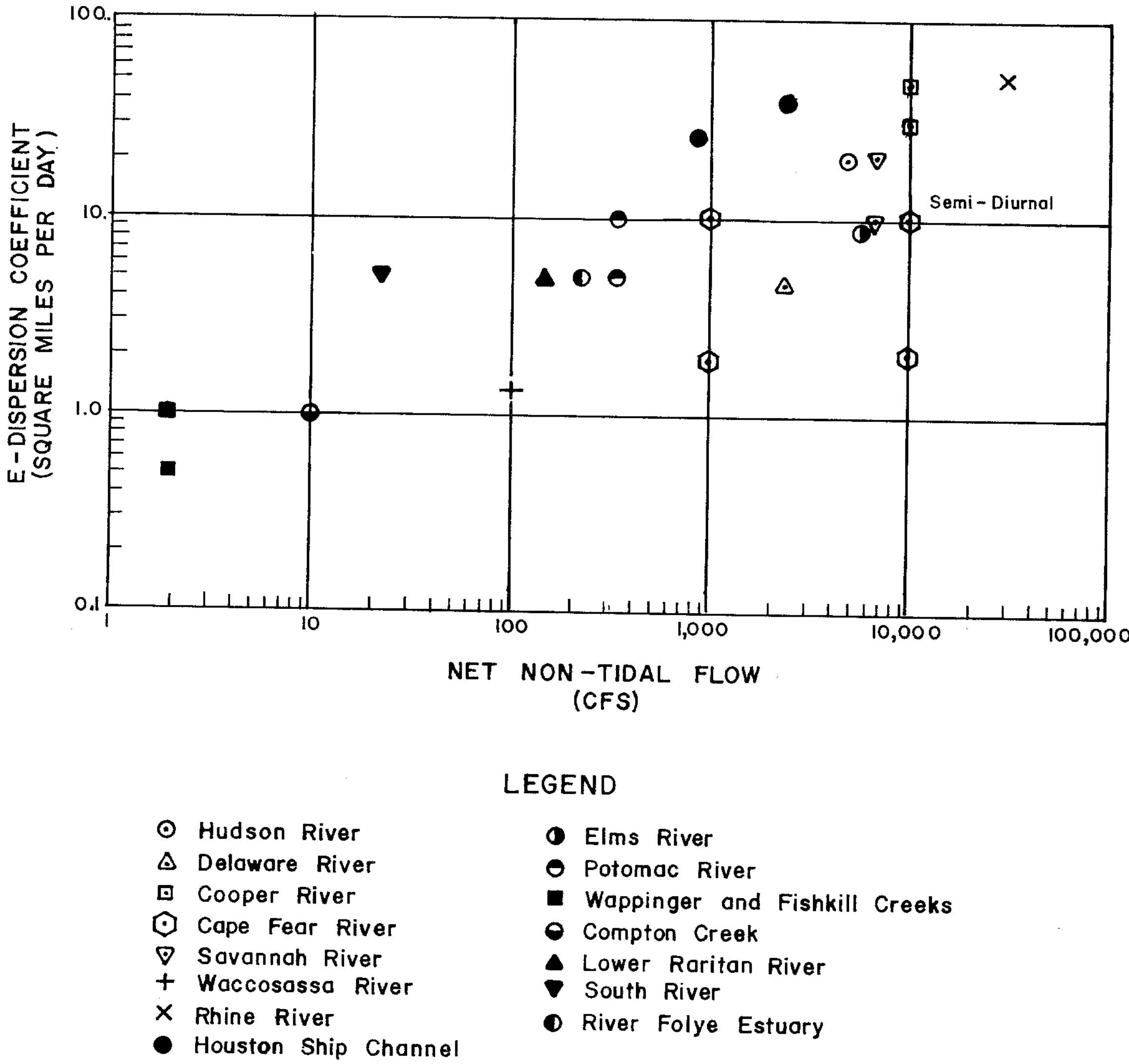


FIGURE III-5

DISPERSION COEFFICIENT FOR DIFFERENT TIDAL RIVERS AND ESTUARIES

IV. PRELIMINARY MATHEMATICAL MODELS - STREAMS AND RIVERS

A. Outline of Models

Water quality analysis of streams may be classified in accordance with the reactive nature of the constituents in wastewaters: conservative and non-conservative. In addition, in accordance with the previous discussion on mathematical models (Section III-B), dispersion effects are assumed to be insignificant for the case of streams and rivers.

1. Conservative Substances

For the purpose of this project, conservative substances may include total dissolved solids, chlorides, and nutrients (total nitrogen and total phosphorus). This classification may also apply to any constituent which may be assumed to decay, in accordance with a single reaction: bacteria concentrations, radioactive matter; suspended solids, certain chemical parameters, and oxygen demanding material. Although these constituents are obviously non-conservative, the most indicative concentration is at the outfall and hence is independent of the reaction effect. In this sense, these latter variables can be regarded as conservative. This is explored more fully below in Section B.

The maximum value for this type of constituent is simply the mass rate of waste discharge divided by the total freshwater flow:

$$C_o = \frac{W}{Q} \tag{IV-1}$$

This is the model used in single waste source analysis, discussed below in Section IV-B.

In the analysis of water quality for multiple sources the conservative variable approach offers a simplified method of computation requiring minimal information. For slowly reacting substances ($K \leq 0.2$/day), it may be sufficiently accurate to add responses such as given by Equation IV-1 to indicate the order of magnitude. The total concentration in the river is simply the arithmetic addition of the individual effects plus the background value, as shown in Figure IV-1.

The total concentration can be compared to the water quality standard and, if satisfactory, no further refinement in the analysis is required. If the comparison is marginal, it may usually be accepted since the analysis is on the conservative side.

If the analysis however, indicates that the total concentration in the river exceeds the standard it may be necessary to account for both the tributary inflow from ground

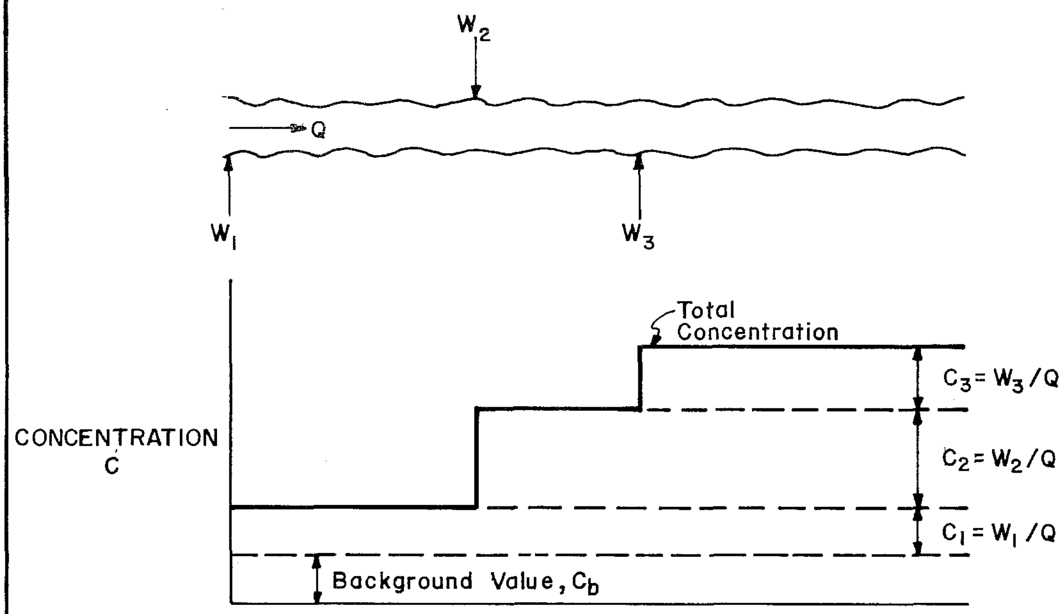


FIGURE IV-1

SUPERPOSITION OF MULTIPLE WASTE SOURCES OF CONSERVATIVE SUBSTANCES

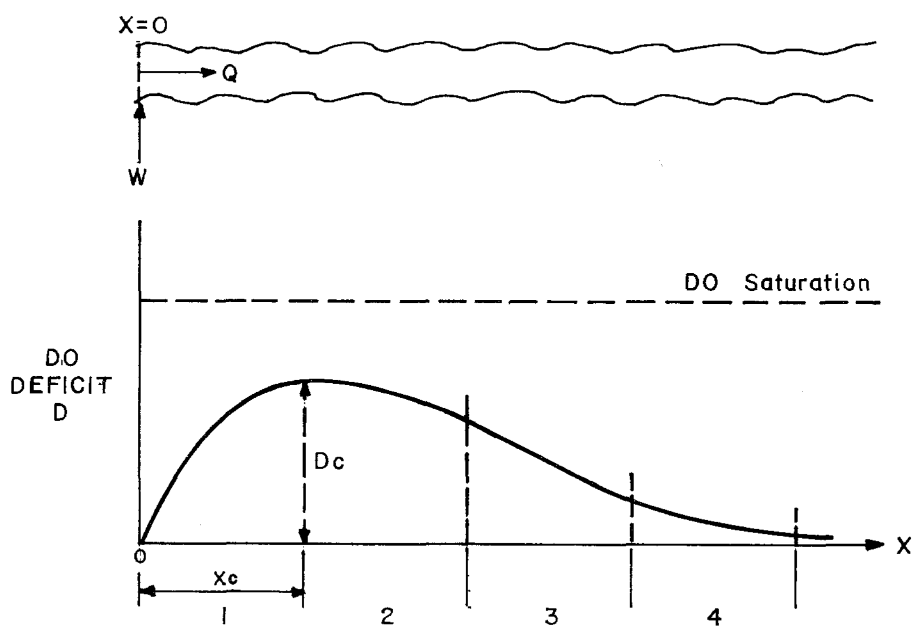


FIGURE IV-2

TYPICAL DO DEFICIT PROFILE

water if it exists and the decay of any non-conservative substance. Both of these factors would reduce the concentration and provide a more realistic analysis. However, this approach requires more information on tributary inflow, reaction coefficient, and velocity. The latter is sometimes difficult to assign, since velocity data are usually not available, particularly at low flow conditions.

2. Non-conservative Substances

In some cases it may be desirable or necessary to account for the decay of certain substances, particularly in those areas where multiple sources are present. The governing differential equation is given in Equation III-6. The concentration of a substance which is characterized by a singular reaction is therefore given by the solution to Equation III-6:

$$c = c_o e^{Kx/U} \tag{IV-2}$$

in which:

c_o = $\frac{W}{Q}$ concentration in the stream at $x = 0$

K = reaction coefficient (per day)

U = stream velocity

x = downstream distance

Table IV-1 contains ranges of values for reaction coeficients in freshwater streams.

TABLE IV-1

RANGE OF VALUES OF REACTION COEFFICIENTS IN STREAMS

Substance	K(per day)
Coliform Bacteria	1 - 3
BOD	0.2 - 2.0
Nutrients	0.1 - 1.0

The coefficients in Table IV-1 are for water temperatures in the 20°-25°C range. Conversion to other temperatures can be made by:

$$K_T = K_{20}(1.047)^{T - 20}$$

where K_T is the reaction coefficient at temperature, T (°C), and K_{20} is the reaction coefficient at 20°C.

As indicated in the preceding section, for those substances which decay at a relatively slow reaction ($K \leq 0.2$/day), the conservative analysis should be sufficient. This approach by-passes the necessity of determining the stream velocity. If the more refined analysis is required, the values above provide a practical range to be used.

3. Dissolved Oxygen Analysis

The concentration profile of dissolved oxygen downstream from a waste discharge is the result of a consecutive reaction: the first which is primarily the oxidative reaction of the residual organic matter, and the second which is the reaeration replacing the deficit caused by the first reaction. Consider a single source of wastewater discharging at a rate of W into a stream with a freshwater flow of Q. The outfall is located at x = 0 downstream from which a typical dissolved oxygen deficit profile results, as shown in Figure IV-2. The DO deficit is given by the saturation value of oxygen, C_s, minus the dissolved oxygen, i.e.:

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

where:

C = dissolved oxygen.

The preliminary model for dissolved oxygen deficit is given by differential Equation III-12, which for negligible dispersion is:

$$0 = U\frac{dD}{dx} + K_d L_o e^{-\frac{K_d x}{U}} - K_a D \tag{IV-4}$$

The equation of the deficit profile is given by the solution of this equation as:

$$D = \frac{K_d L_o}{K_a - K_d} [e^{-\frac{K_d x}{U}} - e^{-\frac{K_a x}{U}}] \qquad \text{(IV-5)}$$

This is a special case of Equation III-13.

The magnitude and location of maximum deficit are the essential features of the profile. For initial deficit of zero, the magnitude and location of the maximum deficit are given in the usual fashion as:

$$D_c = \frac{K_d}{K_a} L_o e^{-\frac{K_d x_c}{U}} \qquad \text{(IV-6)}$$

$$x_c = \frac{U}{K_a - K_d} \ln \frac{K_a}{K_d} \qquad \text{(IV-7)}$$

where:

D_c	=	maximum DO deficit
L_o	=	BOD concentration = $\frac{W}{Q}$ at $x = 0$
K_d	=	deoxygenation or deaeration coefficient
K_a	=	reaeration coefficient (also expressed as K_2)

x_c = distance to location of maximum deficit

Substituting Equation (IV-7) into (IV-6) and rearranging terms, there results:

$$\frac{D_c}{L_o} = [\Phi]^{\frac{\Phi}{1 - \Phi}} \qquad \text{(IV-8)}$$

in which:

$$\Phi = \frac{K_a}{K_d}$$

It is readily apparent that the essential features of the profile are uniquely determined by the dimensionless parameter, Φ, the ratio of the reaeration to deoxygenation coefficient, which is called an assimilation ratio. A practical range of Φ is from 0.1 to 20, for which the ratio, D_c/L_o takes on the values shown in Figure IV-3. The solid line is the actual relation, as indicated by Equation (IV-8) and the dashed line is a graphical approximation that is used below with other empirical relationships.

Equation (IV-8) forms the basis for the dissolved oxygen response in streams due to a single waste source. This analysis is discussed in Section IV-C. A detailed discription of the reaction coefficients in dissolved oxygen, especially the assimilation ratio, Φ is given in Technical Appendix A.

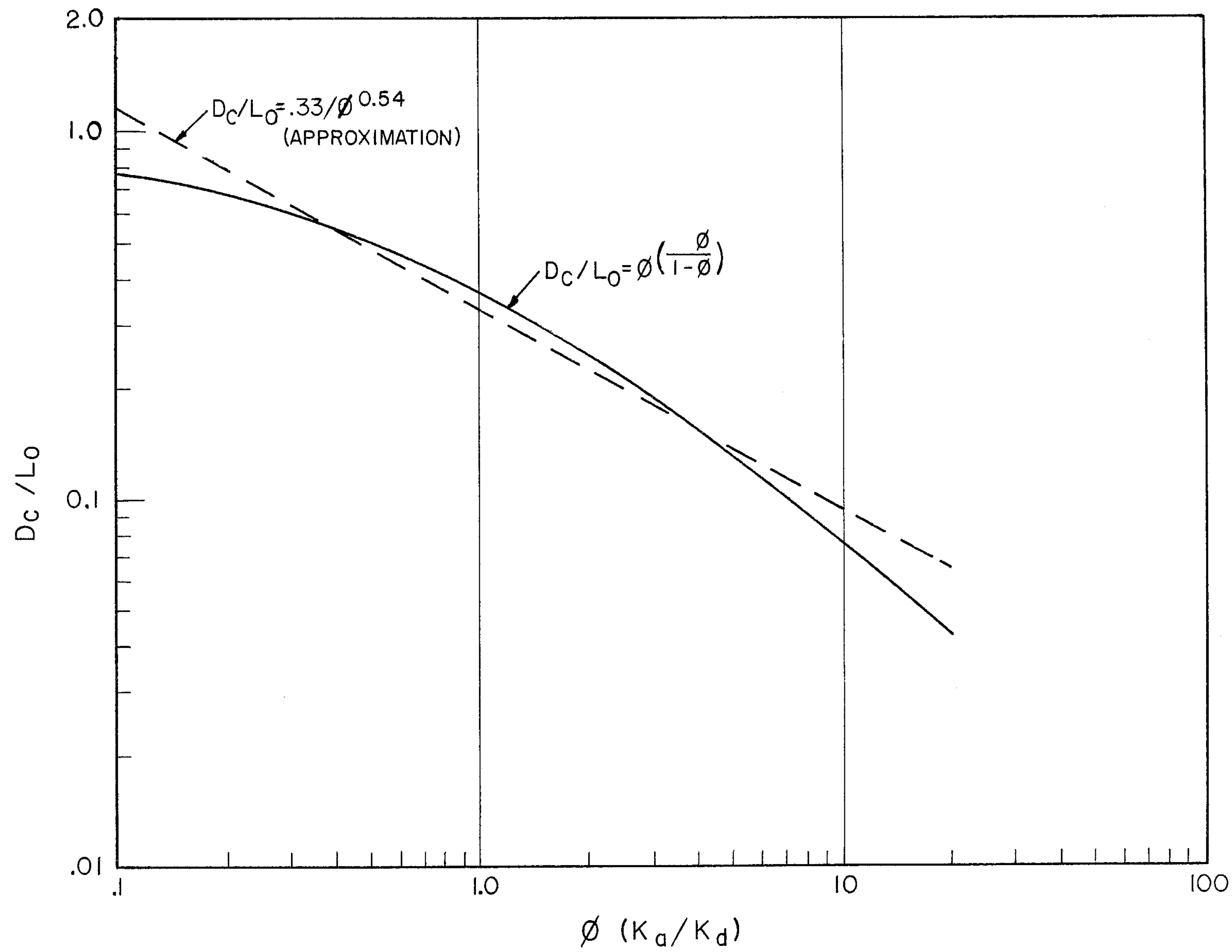


FIGURE IV-3

RATIO D_C/L_O AS A FUNCTION OF ASSIMILATION RATIO, ϕ

For multiple source analyses, the stream may be envisioned as a series of n segments each x_c in length, over which the profile extends. Equation (IV-5) may then be reexpressed in terms of $n \cdot x_c$ distances as:

$$\frac{D_n}{L_o} = \frac{\Phi^{\frac{n}{1 - \Phi}}(1 - \Phi^{-n})}{\Phi - 1} \qquad \text{(IV-9)}$$

This subdivision of the DO deficit profile is shown in Figure IV-2. Equations (IV-5) and (IV-9) form the basis for the multiple source DO analyses in streams (Section IV-D).

B. Single Waste Sources - Importance of Initial Concentration

The concentration in the stream or estuary at the location of the waste discharge is referred to as the initial concentration. In the case of many constituents, such as nutrients, bacteria, and refractory organics, this concentration provides a basis for a direct comparison with water quality standards, since the maximum value occurs at the outfall — the initial concentration. Figure IV-4a shows this effect. In other cases, such as the oxygen demanding material (BOD) the initial concentration is responsible for the magnitude of drop in the dissolved oxygen downstream from the waste

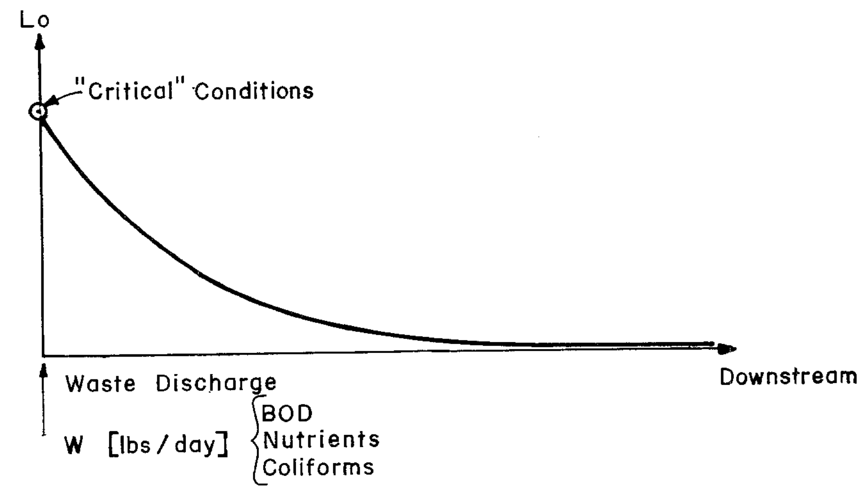


FIGURE IV - 4(A)

"CRITICAL" CONDITIONS OCCUR AT OUTFALL

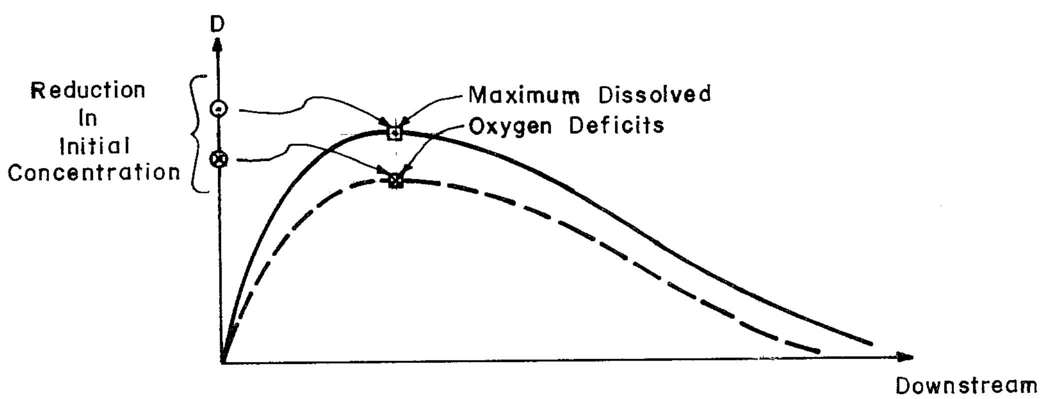


FIGURE IV - 4(B)

FOR DO, REDUCTION IN INITIAL CONCENTRATION REDUCES DEFICIT

input as shown in Figure IV-4b. In any case, the initial concentration is of basic importance in assessing the effectiveness of a waste treatment program for maintaining water quality standards.

The initial concentration is readily determined as simply the mass rate of discharge divided by the appropriate dilution flow. The rate of discharge is a function of the design population and the treatment efficiency (see Section III-Cl). The dilution flow depends on the nature of the receiving water body, either stream or estuary. Thus, the initial concentration has the general form:

$$c_o = \frac{W}{Q} \tag{IV-10}$$

Equation (III-19) can be substituted for W. For the stream, Q is evaluated by Equation (III-20), and therefore:

$$c_o = \frac{f_4 f_3 f_1 P_o}{f_5 DA + f_2 f_1 P_o} \tag{IV-11}$$

where it should be recalled:

f_1 = population growth factor (Table III-1)

f_2 = per capita waste flow contribution (Table III-2)

f_3 = per capita waste load contribution (Table III-2)

f_4 = residual fraction after treatment (Tables III-3 and III-4)

f_5 = flow/square mile drainage area (Figure III-4)

DA = drainage area

P_o = present population

Note that in this form, the initial concentration depends on the primary variables of present population and drainage area which presumably are the two basic items that are known with "certainty". Guidelines for the "f" factors in Equation (IV-11) are given in previous sections. Of course, if any of the factors have been estimated from other data, those estimates should be used. In any analysis of water quality response, the analyst should be aware of the sensitivity of the results to each of the factors given in Equation (IV-11). Different conclusions may be drawn depending on the assumptions made relative to each of the factors. This is illustrated below by an example.

1. Example

Assume an application for a construction grant for a waste plant has been submitted which contains the following information:

12,000 = present population

24,000 = design population

Biological treatment plant of the high rate trickling filter type is proposed for financial support. An analysis is to be made of the effects of the proposed plant on the water quality of the stream.

The additional information which must be gathered, if not submitted with the application, is:

<u>Waste Discharge</u>

1. Per capita waste load contribution = 0.40 pounds/day total oxygen demand (carbonaceous plus nitrogenous), Table III-2

2. Influent to plant = 9,600 pounds/day at design population

3. Residual fraction remaining = 0.30, Table III-3 and considering that trickling filters may provide nitrification slightly greater than 20%

4. Effluent mass rate = 0.3 x 9,600 = 2,880 pounds/day total oxygen demand

5. Per capita waste flow contribution = 100 gallons/day, Table III-2

6. Effluent discharge rate = 2.4 MGD at design population

<u>River Dilution Flow</u>

1. Drainage area = 120 square miles, estimates from U.S.G.S. gaging stations

2. Drought flow in cfs/square mile = 0.1, Table III-6

3. Stream drought flow = 12 cfs

Total Flow

1. $Q = 2.4 + 12 \times 0.65 \ (\frac{MGD}{cfs}) = 10.2 \ MGD$

a) Initial BOD Concentration

$$L_o = \frac{2{,}880}{10.2 \times 8.34} =$$

$$\left[\frac{\text{pounds/day}}{\text{MGD} \times 8.34 \frac{\text{lbs}}{\text{MG}}/\text{mg/l}}\right]$$

= 35 mg/l total oxygen demand (BOD)

This example proceeded in a step-wise fashion to indicate the significance of each factor. The concentration could be arrived at directly by using Equation (IV-11):

$$f_1 = \frac{24{,}000}{12{,}000} = 2.0$$

$$f_2 = 100 \text{ gallons/capita/day}$$

$$f_3 = 0.40 \text{ pounds/capita day}$$

$$f_4 = 0.30$$

$$f_5 = 0.1 \text{ cfs/sm.}$$

$$L_o = \frac{(0.30 \times 0.40 \times 2.0 \times 12{,}000)\,(\text{pounds/day})}{8.34(0.1 \times 120 \times 0.65(\frac{MGD}{cfs}) + 100 \times 2.0 \times 12{,}000 \times 10^{-6}(\frac{MGD}{gal}))} =$$

$$\frac{2,880 \text{ pounds/day}}{8.34\ (7.8 + 2.4)} = 35 \text{ mg/l}$$

Depending on the characteristics of the receiving stream, the dissolved oxygen computation will indicate whether or not this is satisfactory (i.e., maintaining water quality standards).

Assume that the calculation of the dissolved oxygen concentration indicates that the DO standard is violated and in order to correct the violation the initial BOD concentration should be 23 mg/l instead of 35 mg/l. The determination of the violation of the DO standard may be done by the analysis given in the next section. See, for example, Chart. B.

b) Sensitivity Analysis

After completing an analysis such as just given, the analyst should always review the computations to determine the sensitivity of the conclusion to the various assumptions that were made. The reviewing engineer, therefore, has a number of analysis options open. In the context of the previous example, these options are:

1. Increase overall treatment requirements from 70% to approximately 80%. (i.e., 0.80 = 1.00 - 0.30 x 23/35.)

2. Review population study original growth factor assumed = 2.0. If a growth factor = 1.3 is stipulated, the proposed plant is satisfactory (i.e., 1.3 = 2.0 x 23/35.)

3. Check per capita contribution of 0.40 pounds of oxygen demand per capita day which was assumed; a value of 0.26 provides a satisfactory condition (i.e., 0.26 = 0.40 x 23/35).

4. The drought flow assumption of 0.1 cfs/sq. mi. may be too conservative; a value of 0.15 cfs/sq.mi. would provide sufficient dilution (i.e., 0.15 = 0.10 x 35/23), to meet requirements. Chart A can be used here to test the sensitivity of the total river flow to change in the runoff coefficient.

As seen from the example, there are generally four significant factors, which, in the absence of detailed data and study may be assigned rather arbitrarily, based on the judgment of the engineer. In addition, for dissolved oxygen, an additional factor which assesses the reaeration capacity of the stream must be assigned (Section III B-3). A review of a more detailed analysis of one of these factors may permit the acceptance of present plans, as indicated above. For example, an average municipal per capita contribution of 0.32 pounds/capita day, a drought flow of 0.12 cfs/sm and the original growth factor maintain satisfactory conditions:

$$35 \text{ mg/l} \times \frac{0.32}{0.40} \times \frac{0.10}{0.12} = 23 \text{ mg/l}$$

Other combinations of revised factors may also prove feasible.

The point then is to recognize the sensitivity of the final result to each of the input data items and carefully analyze the effects of each of the assumptions that are made.

c. Initial Coliform Concentration

The coliform count in the river is determined as follows: assume the average value of 80×10^9 MPN per capita day (Table III-2) and a removal of 99.9% assuming chlorination (residual of 0.001),

$$\text{MPN} = \frac{(0.001 \times 80 \times 10^9 \times 2 \times 12{,}000)\ (\text{MPN/day})}{(10.2 \times 10^6 \times 3{,}785\ (\frac{\text{ml}}{\text{gal}})\ (\text{ML/day})} =$$

$$= 50\ \text{MPN/ml}$$

This concentration should generally be quite acceptable but one must recognize the possible addition of natural background coliform counts (see Table III-5) and other sources of bacterial contamination in metropolitan areas, such as combined sewer overflows and urban drainage.

d. Initial Nutrient Concentration

The nutrient discharge may be of significance and should be checked. For the total nitrogen assuming an average of 0.034 pounds/nitrogen/capita day and a removal of 20% with a residual factor of 0.80, (Table III-4) the initial stream concentration is:

$$N_o = \frac{(0.80 \times 0.035 \times 2.0 \times 12{,}000)\ \text{pounds/day}}{8.34 \times 10.2\ (\text{MGD})} =$$

$$= 8\ \text{mg/l total nitrogen}$$

During the low flow period on relatively small drainage areas, this condition may be typical, i.e., high concentration of nitrogen indicative of potential algal problems. Since standards have not been rigorously established for this constituent, the above computation should be employed to indicate a potential problem rather than a strict criterion for additional treatment. It may be appropriate to indicate, that additional treatment may be required if the eutrophication potential is realized in the future.

C. Single Waste Sources - Dissolved Oxygen Analysis

The estimation of treatment adequacy to meet dissolved oxygen standards for a single waste source in a stream or river can be summarized in a nomograph as given by Chart B. This chart affords the analyst the opportunity to test the efficiency of several treatment levels in meeting dissolved oxygen standards. Several points should be noted. As indicated previously, (Section IIIC-1(c)), several discrete levels of treatment are considered, (as shown in Table III-3), rather than a continuous spectrum of treatment efficiency. This level of accuracy is sufficient since the major purpose of the analysis is to estimate required treatment levels and not to estimate the detailed water quality response of the stream. Figure B-1 therefore permits estimation of the total oxygen demand in pounds/day (including the oxygen demanding equivalent of any oxidizable nitrogen) that would be estimated to be discharged from the given design population and treatment level. The discharged load shown in Figure B-1 makes use of Table III-3.

Figure B-2 requires the total river flow (see Chart A and Figure III-4) which includes the stream flow above the discharge plus the flow for the waste source itself, (Equation III-20). Given the total oxygen demand from Figure B-1 and the river flow, Figure B-2 is used to estimate the initial concentration of the total oxygen demand in mg/l resulting after dilution in the river, (Equation IV-1). A zero upstream BOD concentration is assumed.

Figure B-3 incorporates the fundamental reaeration capacity of the stream using several different data levels. Information on the stream can be at one or all of the following levels:

a) Qualitative description, e.g., shallow, main drainage rivers, impounded rivers, etc.

b) A measurement of the depth of the river, a key variable (see Section IIIC-2e)

c) Estimate of Φ, the ratio of the re-aeration coefficient, K_a and the deoxygenation coefficient, K_d.

The analyst, given the previous description of the stream, then estimates the maximum DO deficit (mg/l) using Figure B-3. Note that a zero initial DO deficit has been assumed. Any such "background" DO deficit is incorporated at the conclusion of the analysis. The construction of the Figure basically draws on Equation IV-5. A detailed discussion of the background of Figure B-3 is given in Technical Appendix A.

Figure B-4 incorporates the variation of the DO saturation level with maximum water temperature (see Figure III-3). With the maximum DO deficit, the analyst proceeds through Figure B-4 with maximum water temperature to estimate the minimum dissolved oxygen (mg/l). It is at this point that a "background" DO deficit can be included. In the absence of any data, a background DO deficit of 1.0 mg/l is suggested. Indeed, it is recommended that the DO background deficit of 1.0 mg/l be considered as a minimum level and only if firm data and other anlayses are available, should a background level less than 1 mg/l be used.

The rationale for incorporating a constant background DO deficit rests on two grounds a) computational simplicity and b) existence of other sources and sinks of dissolved oxygen. The theory indicates that an initial DO deficit decays exponentially at a rate given by the reaeration rate and the stream velocity. As such, it appears in the expression for the maximum DO deficit, an undesirable trait. Countering the decrease in initial DO deficit is the existence of other phenomena which affect DO, including benthal oxygen demanding material, algal photosynthesis and respiration, incremental additions of oxygen demanding loads from incremented drainage, among others. It was judged that a desirable procedure would be to incorporate all of these effects in a single constant background DO deficit.

The output from the "first pass" through Chart B therefore provides a first estimate of the minimum dissolved oxygen which is then compared to the standard. If the dissolved oxygen for the given level of treatment is less than the standard, the analyst can reverse the procedure by starting at the required DO standard and continuing through Figures B-4, B-3, and B-2 using the same water temperature, river reaeration capacity and total river flow. One now enters Figure

B-1 with the required total oxygen demand mass discharge. For the fixed design population, the next highest level of treatment is chosen which will equate or exceed the required pounds/day discharge. As with the single variables discussed in the previous section (BOD, nutrients), a sensitivity analysis (see IV-B-1b) should be conducted to determine the effect each step and assumption has on the conclusion.

1. Example - Use of Chart B

Design Population - 51,000 - Proposed Treatment High Rate Biological Treatment

DO Standard - 4 mg/l

River - Intermediate Channel, depth 5-10 feet.

Drainage Area at Discharge Location - 1,700 mi^2

Estimated Drought Flow Rate: .05 cfs/mi^2

Maximum Water Temperature - 30°C

Chart A: Figure A-1 @ 1,700 mi^2 and .05 cfs/mi^2 gives 85 cfs.

Figure A-2 @ 51,000 population gives total drought flow of 95 cfs.

Chart B: Figure B-1, Design Population - 51,000 @ High Rate treatment gives effluent load of 10,000/lb/day ultimate BOD Figure B-2, Drought flow of 95 cfs and 10,000/lb/day gives in-stream oxygen demand concentration (L_o) of about 20 mg/l.

Figure B-3, using intermediate channel drainage river depth 5-10 feet and concentration from Figure B-2, gives maximum DO deficit of about 5.9 mg/l.

Figure B-4, at water temperature of 30°C, 6.9 mg/l maximum deficit and using 1 mg/l background DO deficit, gives minimum dissolved oxygen of 0.5 mg/l.

Proposed high rate biological treatment scheme may not, therefore, achieve the objective.

The analysis procedure can now be reversed and a minimum DO of 4 mg/l @ 1 mg/l background deficit is chosen. Figure B-4 shows for 30°C, that a maximum deficit of about 2.4 mg/l is allowable.

Figure B-3 with 2.4 mg/l deficit and 5-10 feet depth river gives allowable in-stream concentration of about 8 mg/l.

Figure B-2 at the drought flow of 95 cfs gives an allowable 4,000 lb/day discharge.

Figure B-1 @ 4,000 lb/day discharge load, shows that for 51,000 design population, one should consider the necessity for standard rate biological treatment including nitrification of oxidizable nitrogen.

D. Multiple Waste Sources - Dissolved Oxygen Analysis

If more than one waste source influences the dissolved oxygen of a specific region, one must recognize the overlapping effects of such discharges in a multiple waste source analysis. The additional data needed for the analysis is information on the stream velocity. Data on the individual waste sources and characteristics of the river as described in the preceeding sections are also required. However, a simplified dissolved oxygen analysis is possible that does not require information on stream velocities. Two approaches to the multiple waste source DO problem in streams are therefore presented. The first approach makes use of the stream velocity and the "influence lines" for water quality. This analytical approach couples a nomograph with a tabular listing and summary of the individual and total response. The second approach draws on several empirical relationships and approximations and is applied specifically to the dissolved oxygen distributions in streams. In this latter approach, the single source stream nomograph (Chart B) is used together with a mathematical approximation to the DO equation. Graphical display of the individual effects of the plants followed by graphical superposition is employed to estimate the total response.

1. Analytical Approach Using Stream Velocity

Chart C shows the technique for computing the individual dissolved oxygen response from each of a series of waste sources. The information from the single source analysis supplemented by the estimate of the stream velocity, is required. The analyst should examine the length of the river under consideration and spot all waste sources of concern. The assumptions underlying Chart C are:

A) All stream parameters, e.g., flow, velocity, reaction coefficients are constant over the length of the river under study.

B) No major tributaries enter throughout the reach.

C) Upstream (above the first waste source) concentrations of BOD and DO deficit are zero.

Table IV-2 is intended to accompany Chart C. Each waste source is numbered beginning at one for the first source. The total cumulative downstream distance in miles is entered in column one of Table IV-2. With the design population and treatment level, Figure B-1 of Chart B provides an estimate of the discharge load in pounds/day. This estimate is entered in column four for each waste source. As shown in Table IV-2 columns five include the individual DO deficit response (mg/l) for each waste input at the beginning of each reach. These responses require knowledge of the parameters, K_a/U and Φ, the assimilation ratio.

Figure C-1 is entered with the mile point beginning at x = 0 at the first waste discharge location. With

TABLE IV-2

DISSOLVED OXYGEN - MULTIPLE WASTE SOURCES

River or Stream ____________________ Total Drought River Flow
Date ____________________ (Chart (B) ____________________ cfs
Design Year ____________________ Drought Velocity ____________________ fps
Deoxygenation coefficient ________ /day K/U ____________________
= ____________________ Analyst ____________________

(1) Total Down-Stream Distance (miles)	(2) Waste Source No.	(3) River Reach No.	(4) Input (lb/day) (Chart B)	(5) DO Deficit Response (mg/l) for input in beginning of Reach No. (Chart C)					(6) Subtotal DO Def. Response (mg/l)
				1	2	3	4	5	
0	1	1	W_1	0					$D_{1,j}$
				$D_{1,1}$					$D_{1,j}$
				$D_{1,2}$					$D_{1,j}$
X_1		1		$D_{1,3}$					$D_{1,j}$
	2	2	W_2	$D_{1,4}$	0				$D_{1,j} + D_{2,j}$
				$D_{1,5}$	$D_{2,1}$				$D_{1,j} + D_{2,j}$
				$D_{1,6}$	$D_{2,2}$				$D_{1,j} + D_{2,j}$
$X_1 + X_2$		2		$D_{1,7}$	$D_{2,3}$				$D_{1,j} + D_{2,j}$
	3	3	W_3	$D_{1,8}$	$D_{2,4}$	0			$D_{1,j}+D_{3,j}+D_{4,j}$
				$D_{1,9}$	$D_{2,5}$	$D_{3,1}$			$D_{1,j}+D_{2,j}+D_{3,j}$
$X_1+X_2+X_3$		3		$D_{1,10}$	$D_{2,6}$	$D_{3,2}$			$D_{1,j}+D_{2,j}+D_{3,j}$

*First subscript each waste source number, second subscript indicates milepoint, e.g., $D_{1,6}$ is DO deficit due to waste source #1 at 6th milepoint location downstream from entrance of Source #1.

TABLE IV-2
(continued)

DISSOLVED OXYGEN - MULTIPLE WASTE SOURCE

(1) Total Down-Stream Distance (miles)	(7) Background Deficit (mg/l)	(8) Total DO Deficit Response (mg/l) (6) + (7)	(9) DO Saturation (mg/l)	(10) DO (mg/l) (9)-(8)	(11) DO Standard (mg/l)	(12) DO Standard Comparison (mg/l) (10) - (11)
0	B	$D_{1',j} + B$	C_s			
	B	$D_{1',j} + B$	C_s			
	B	$D_{1',j} + B$	C_s			
X_1	B	$D_{1',j} + B$	C_s			
	B	$D_{1',j}+D_{2',j}+B$	C_s			
	B	$D_{1',j}+D_{2',j}+B$	C_s			
	B	$D_{1',j}+D_{2',j}+B$	C_s			
X_1+X_2	B	$D_{1',j}+D_{2',j}+B$	C_s			
	B	$D_{1',j}+D_{2',j}+D_{3',j}+B$	C_s			
	B	$D_{1',j}+D_{2',j}+D_{3',j}+B$	C_s			
$X_1+X_2+X_3$	B	$D_{1',j}+D_{2',j}+D_{3',j}+B$	C_s			

* Note comments on Part I - Table IV-2.

knowledge of K/U, this distance is converted to a dimensionless length scale, $x^* = K_d X/U$. Note that as with the single source analysis, approximations are possible even if the BOD decay coefficient, K_d, is not known. However, some estimate of the velocity should be at hand. This estimate should correspond to the velocity occurring at the drought flow conditions. Section IIIC-2 provides some guidelines.

Figure C-2 presents a series of DO deficit "influence graphs" for different values of Φ and X^*. The Figure also indicates the estimated DO deficit profile if only a qualitative description of the stream is available. The output from Figure C-2 has units mg/l DO deficit per mg/l ultimate BOD at the waste outfall.

After exiting from Figure C-2, the analyst proceeds to convert the DO response to mg/l DO deficit per 1,000 pounds per day waste input by Figure C-3. This requires an estimate of the drought flow which can be obtained from Chart A or other sources. The final step in determining the deficit response is given by Figure C-4 which converts the unit response to the deficit response due to the actual discharge given by column (4) of Table IV-2.

The use of Chart C is repeated for the first waste source at a series of conveniently chosen distances downstream.

Each DO deficit at the downstream locations is entered in the first sub-column of column (5) and represents the DO deficit profile due to the waste source of the beginning of reach #1.

The entire procedure is now repeated for the second waste source taking care to again begin use of Figure C-1 at $x = 0$ miles. However, when the deficit responses are entered in column (5), 2nd reach, they are entered beginning at x_1 miles, the distance of the first reach. This is shown in Table IV-2 by the notation D_2j in columns (5).

Each waste source is treated in a similar fashion until all DO deficit responses have been obtained for all of the sources. Column (6) is now computed by adding across the system responses given in the individual responses of column (5). The DO deficit at each mile point due to all waste sources is thus given in Column (6). As indicated, the individual responses due to each waste source, when added at each mile point provide an estimate of the total deficit at that mile point.

The background DO deficit is entered in column (7) and added to the sub-total deficit profile in column (6) to give the total response in column (8). If data on the background deficit are not available, 1 mg/l is suggested as a reasonable estimate. The saturation value of dissolved oxygen under the prevailing temperature conditions is entered in column (9). Figure III-3 shows the DO saturation values under a range of temperature conditions.

Column (10) is the estimated dissolved oxygen at each location and is given by subtracting the total deficit in column (8) from the saturation value of DO given in Column (9), which therefore represents the dissolved oxygen profile estimated to occur in the design year for the treatment levels specified for each waste source.

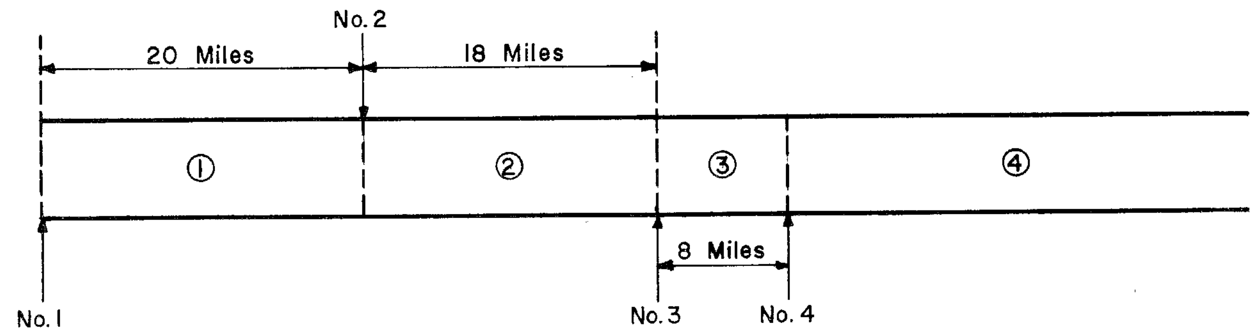
The DO standard is entered in column (11) at each location along the length of the river. Reference should be made to appropriate objectives specified by state and Federal agencies.

Finally, column (12) indicates those areas of the river length where the standards are estimated to be violated under the conditions of the analysis. Column (12) is obtained by subtracting the DO standard given in column (11) from the DO profile given in column (10). All positive values indicate DO concentrations greater than required and show conformance with the required DO level. Negative values, i.e., values of DO less than the standard indicate standard violation and require a re-analysis procedure.

For the latter case of a standard violation, a suggested first iteration is to increase the level of treatment of waste sources upstream from the region of violation. The increased treatment should be to the next discrete level of treatment as shown in Tables III-3 and III-4 and in Figure B-1 or as given by other data. A new multiple source analysis with the increased levels of treatment is then carried out for the river stretch under violation and a standard comparison is again computed. The waste treatment increase procedure is then repeated until compliance (i.e., all positive values in column (12) are achieved.

a. Illustrative Example

Table IV-3 and Figure IV-5 provide the basic input data used in the following example which illustrates the use of the multiple stream source analysis. As shown, four waste sources are considered over about a 50 mile stretch of the river. Four reaches of the river are therefore defined as shown in Figure IV-5. Table IV-4 indicates the numerical analysis of the estimated DO profile due to the four waste sources. Column (4) was obtained from Figure B-1. Column (5) was obtained from Chart C using a K_d/U of 0.8/mi. Note how each individual DO deficit response begins at the location of the outfall. It can also be seen that actually each DO deficit response profile is in direct proportion to the first



RIVER CHARACTERISTICS: Drought Flow - 770 CFS
Drought Velocity - 0.25 FPS
Depth - 5 - 10 Feet
Estimated Reaeration Ratio Ø - 1.5

FIGURE IV - 5

ILLUSTRATIVE EXAMPLE
MULTIPLE WASTE SOURES IN STREAMS

TABLE IV-3

ILLUSTRATIVE EXAMPLE
MULTIPLE WASTE SOURCE IN RIVERS

Waste Source	River Mile Location	Design Population	First Estimate of Treatment Level	Revised Treatment Levels to Meet D.O. Standard
#1	0	65,000	HR–BIO[1]	STD–RT–BIO[2]
#2	20	240,000	HR–BIO	STD–RT–BIO
#3	38	50,000	HR–BIO	STD–RT–BIO
#4	46	37,000	HR–BIO	STD–RT–BIO

[1] HR–BIO: High Rate Activated Sludge (see Figure B–1 and Table III-3).

[2] STD–RT–BIO: Standard Rate Biological Treatment, with nitrification.

TABLE IV-4

ILLUSTRATIVE EXAMPLE
DISSOLVED OXYGEN - MULTIPLE WASTE SOURCE

(1) Total Down-Stream Distance (miles)	(2) Waste Source No.	(3) River Reach No.	(4) Input (lb/day)	(5) DO Deficit Response (mg/l) for input in beginning of Reach No. (Chart C) 1	2	3	4	(6) Subtotal DO Def. Response (mg/l)
0	1	1	13,000	0				0
5		1		0.8				0.8
10		1		0.9				0.9
15		1		0.9				0.9
20	2	2	48,000	0.7	0			0.7
25		2		0.5	2.8			3.2
30		2		0.4	3.4			3.8
35		2		0.3	3.2			3.5
38	3	3	10,000	0.2	2.8	0		3.0
40		3		0.2	2.6	0.3		3.1
46	4	4	7,500	0.1	1.9	0.7	0	2.7
50		4		0.1	1.5	0.7	0.4	2.7
55		4		0.1	1.1	0.6	0.5	2.3
60		4		-	0.8	0.5	0.5	1.8
70		4		-	0.4	0.3	0.3	1.0
80		4		-	0.2	0.1	0.2	0.5
90		4		-	0.1	0.1	0.1	0.3

TABLE IV-4
(continued)

ILLUSTRATIVE EXAMPLE
DISSOLVED OXYGEN - MULTIPLE WASTE SOURCE

(1) Total Down-Stream Distance (miles)	(7) Background Deficit (mg/l)	(8) Total DO Deficit Response (mg/l)	(9) DO Saturation (mg/l)	(10) DO (mg/l)	(11) DO Standard (mg/l)	(12) DO Standard Comparison (mg/l)
0	1.0	1.0	7.4	6.4	5.0	+ 1.4
5	1.0	1.8	7.4	5.6	5.0	+ 0.6
10	1.0	1.9	7.4	5.5	5.0	+ 0.5
15	1.0	1.9	7.4	5.5	5.0	+ 0.5
20	1.0	1.7	7.4	5.7	5.0	+ 0.7
25	1.0	4.2	7.4	3.2	5.0	- 1.8
30	1.0	4.8	7.4	2.6	5.0	- 2.4
35	1.0	4.5	7.4	2.9	5.0	- 2.1
38	1.0	4.0	7.4	3.4	5.0	- 1.6
40	1.0	4.1	7.4	3.3	5.0	- 1.7
46	1.0	3.7	7.4	3.7	5.0	- 1.3
50	1.0	3.7	7.4	3.7	5.0	- 1.3
55	1.0	3.3	7.4	4.1	5.0	- 0.9
60	1.0	2.8	7.4	4.6	5.0	- 0.4
70	1.0	2.0	7.4	5.4	5.0	+ 0.4
80	1.0	1.5	7.4	5.9	5.0	+ 0.9
90	1.0	1.3	7.4	6.1	5.0	+ 1.1

profile. The proportionality is given by the ratios of the input waste loads. Column (10) shows the estimated DO profile and indicates a spatial minimum of about 3.6 mg/l. Column (12) shows the comparison of this profile to an assumed DO standard of 5.0 mg/l. As indicated, the standards are violated from about mile 25 to mile 60. Inspection of the table indicates that a major source of the violation is waste source #2, although each of the sources contributes significant quantities to the total profile. In order to meet the required standard, some increase in treatment may be required.

If all four sources are considered to provide standard rate biological treatment with nitrification, the new set of input waste levels is shown in Table IV-5. As shown in that Table, the increased treatment level of all four sources results in meeting the DO standard of 5 mg/l at all locations downstream. It should be noted that a uniform increase of treatment at significant sources is not the only combination that will achieve the required standard. For purposes of the preliminary analysis, however, further refinement in trading off treatment levels between discharges is not warranted.

2. Approximate Graphical Procedure

Simplification of the above approach is possible. This procedure requires the analyst to sketch in graphical

TABLE IV-5

ILLUSTRATIVE EXAMPLE
DISSOLVED OXYGEN - MULTIPLE WASTE SOURCE

First Iteration on Treatment Levels
To Achieve DO Standard

(1) Total Down-Stream Distance (miles)	(2) Waste Source No.	(3) River Reach No.	(4) Input (lb/day)	(5) DO Deficit Response (mg/l) for input in beginning of Reach No. (Chart C) 1	2	3	4	(6) Subtotal DO Def. Response (mg/l)
0	1	1	3,500	0				0
5				.2				0.2
10				0.3				0.3
15				0.2				0.2
20	2	2	13,000	0.2	0			0.2
25				0.1	0.8			0.9
30				0.1	0.9			1.0
35				0.1	0.9			1.0
38	3	3	3,000	0.1	0.8	0		0.9
40				0.1	0.7	0.1		0.9
46	4	4	2,000	-	0.5	0.2	0	0.7
50				-	0.4	0.2	0.1	0.7
55				-	0.3	0.2	0.1	0.6
60				-	0.2	0.1	0.1	0.4
70				-	0.1	0.1	0.1	0.3
80				-	-	-	-	-
90				-	-	-	-	-

TABLE IV-5
(continued)

ILLUSTRATIVE EXAMPLE
DISSOLVED OXYGEN - MULTIPLE WASTE SOURCE

First Iteration on Treatment Levels
To Achieve DO Standard

(1) Total Down-Stream Distance (miles)	(7) Background Deficit (mg/l)	(8) Total DO Deficit Response (mg/l)	(9) DO Saturation (mg/l)	(10) DO (mg/l)	(11) DO Standard (mg/l)	(12) DO Standard Comparison (mg/l)
0	1.0	1.0	7.4	6.4	5.0	+ 1.4
5	1.0	1.2	7.4	6.2	5.0	+ 1.2
10	1.0	1.3	7.4	6.1	5.0	+ 1.1
15	1.0	1.2	7.4	6.2	5.0	+ 1.2
20	1.0	1.2	7.4	6.2	5.0	+ 1.2
25	1.0	1.9	7.4	5.5	5.0	+ 0.5
30	1.0	2.0	7.4	5.4	5.0	+ 0.4
35	1.0	2.0	7.4	5.4	5.0	+ 0.4
38	1.0	1.9	7.4	5.5	5.0	+ 0.5
40	1.0	1.9	7.4	5.5	5.0	+ 0.5
46	1.0	1.7	7.4	5.7	5.0	+ 0.7
50	1.0	1.7	7.4	5.7	5.0	+ 0.7
55	1.0	1.6	7.4	5.8	5.0	+ 0.8
60	1.0	1.4	7.4	6.0	5.0	+ 1.0
70	1.0	1.3	7.4	6.1	5.0	+ 1.1
80	1.0	1.0	7.4	6.4	5.0	+ 1.4
90	1.0	1.0	7.4	6.4	5.0	+ 1.4

form the major features of the DO deficit profile due to each waste source. Each profile is then graphically superimposed to provide the total deficit profile. This profile is then subtracted from the saturation value of DO. The resulting dissolved oxygen profile is then compared to standards and stream reaches where violations occur are graphically noted. An iteration on the treatment level of each source is then performed, as above, until the required standard is met. A major advantage of the procedure is that it does not require a specification of the stream velocity provided low flow velocity is not greater than about 2 fps.

In order to sketch the individual DO deficit profiles, it is necessary to determine the location of the maximum deficit and other pertinent points, sufficient to approximate the overall profile.

A formula for the location of the maximum deficit may be developed by expressing Equation IV-7 in terms of K_d and K_a and relating these parameters to the average depth and velocity. The final functional form is:

$$x_c \simeq 3H^{5/4} \cdot U^{1/3} \tag{IV-12}$$

in which:

x_c = distance to maximum deficit from the waste discharge - miles

H = average depth in feet

U = stream velocity at drought flow - feet per second

Since a significant number of streams and rivers have low flow velocities in the order of 1 fps, the velocity term may be dropped for the accuracy required for this project. Therefore: $x_c \simeq 3H^{5/4}$. The distance to the maximum deficit is then only a function of the depth. This holds for velocities within the range of 0.5 to 1.7 fps. The coordinates of the critical location are thus specified by Equations IV-8 and IV-12, in which both D_c and x_c are only functions of the average depth, H.

a. Definition of Total Profile

In addition, to the coordinate of the maximum dissolved oxygen deficit, three additional points are desirable in order to describe the total profile. The ratio of the deficit at any point to that at the critical location is defined by division of Equation IV-9 by IV-8:

$$\frac{D_n}{D_c} = \frac{\Phi^{\frac{n - \Phi}{1 - \Phi}}[1 - \Phi^{-n}]}{\Phi - 1} \qquad \text{(IV-14)}$$

The coordinates at two additional points are readily identified by Equation IV-13. At $x/x_c = 1/2$ and 2, the magnitude of the ratio of D_n/D_c is approximately 0.85 for a wide range of Φ, as shown in Figure IV-6(A). Beyond $2x_c$ the difference between the exact and approximate profiles becomes increasingly divergent, the rate at which each approaches zero being a function of Φ.

In order to estimate the location at which the deficit may be assumed to be zero, the area under the deficit profile was defined by series of triangles and quadrilaterals shown in Figure IV-6(B). The sum of the individual areas was equated to the area determined by integrating Equation IV-5 from zero to infinity. In this equality the only unknown is the distance from $x/x_c = 2$ to the location at which the profile intersects the x axis. The total number of x_c segments to define the deficit profile is two plus this number and equals:

$$n \simeq 4 + 0.6\Phi \qquad \text{for } \Phi > 1 \qquad \text{(IV-14a)}$$

$$n \simeq 4 + \frac{0.6}{\Phi} \qquad \text{for } \Phi < 1 \qquad \text{(IV-14b)}$$

Since Φ is related to the average depth H, (see Technical Appendix A):

$$n \simeq 4 + \frac{9}{H} \qquad \text{for } H < 10 \qquad \text{(IV-15a)}$$

$$n \simeq 5 \qquad \text{for } H > 10 \qquad \text{(IV-15b)}$$

In summary, the total DO deficit profile may be readily approximated by the following steps:

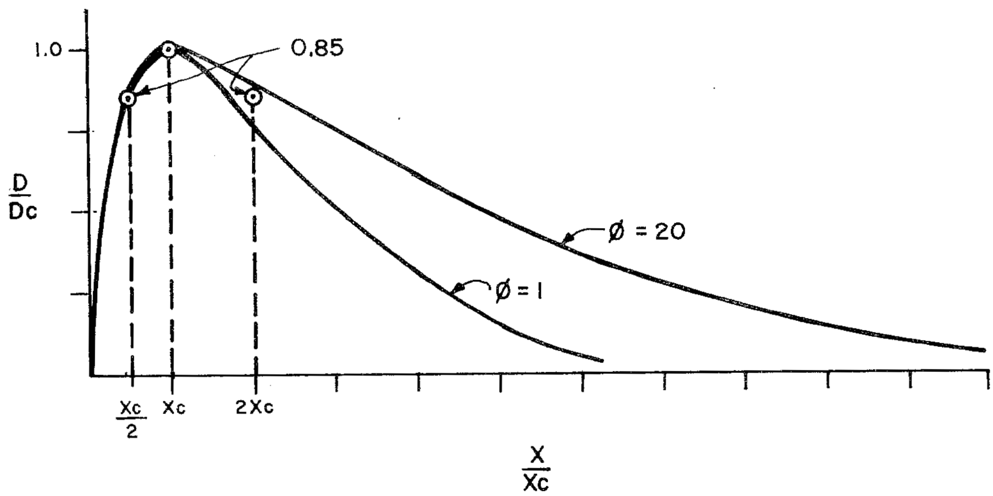


FIGURE IV-6(A)

DIMENSIONLESS DEFICIT PROFILE
FOR RANGE OF Ø

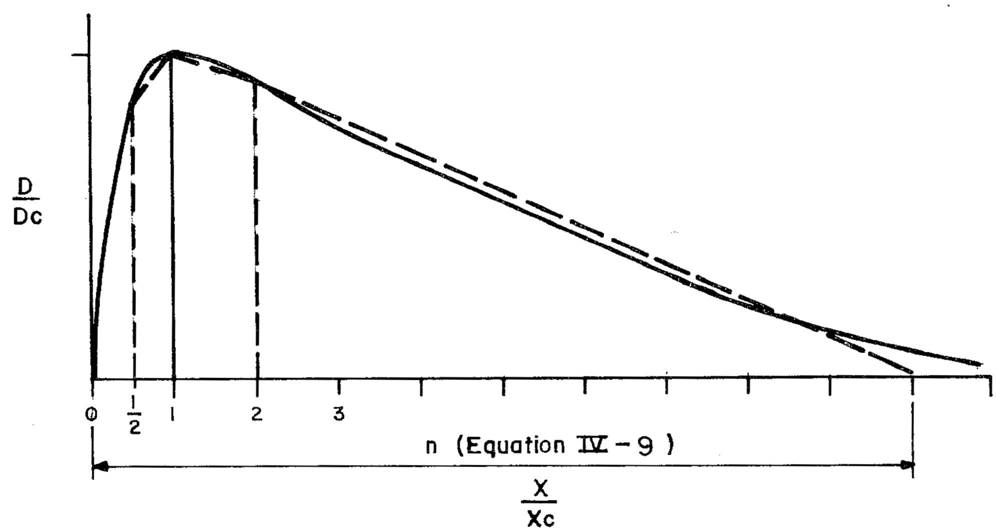
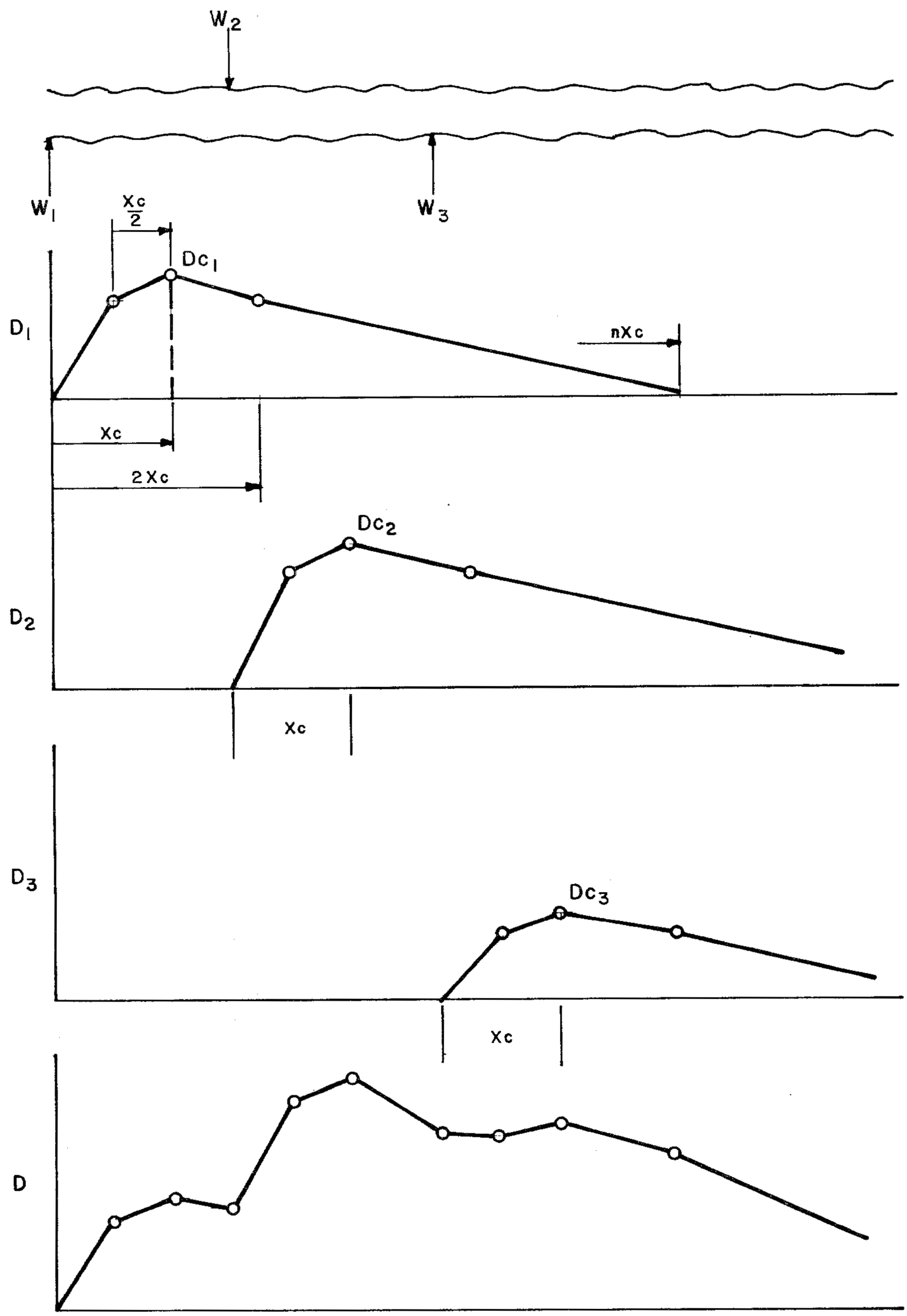


FIGURE IV-6(B)

GENERALIZED GEOMETRIC APPROXIMATION

1. L_o (the initial concentration for BOD) is estimated by the procedure for a single source, i.e., from W and Q. (Equation IV-1) Figures B-1 and B-2 of Chart B permit this estimation.
2. For the given computed L_o or that estimated from Figure B-2, D_c is read from Figure B-3.
3. Knowing the depth from which D_c is determined, x_c is computed from Equation (IV-12).
4. At $x = \frac{x_c}{2}$ amd $2x_c$, $D = 0.85\ D_c$.
5. At $x = nx_c$ in which n is determined from Equation (IV-14) or (IV-15), $D = 0$.
6. Each profile is plotted with $x = 0$ at its individual discharge location by steps 1 - 5.
7. The total profile is simply the arithmetic addition of the individual profiles as shown in Figure IV-7.
8. A constant background deficit should be assumed and added to the total profile above.



TOTAL DEFICIT PROFILE DUE TO W_1, W_2, W_3

FIGURE IV-7

GRAPHICAL SUPERPOSITION OF INDIVIDUAL DO PROFILES

V. PRELIMINARY MATHEMATICAL MODELS - TIDAL RIVERS AND ESTUARIES

A. Outline of Models

For purposes of this report, a tidal river is defined as that portion of a water body that is subject to reversals of current direction but does not include estuaries where the effects of freshwater runoff may be small. Thus, tidal rivers that may oscillate in velocity direction due to causes other than astronomical tides are included in the analysis. An example of the latter case is the flow oscillations in the tributaries of the Great Lakes caused by wind produced seiches. Estuaries are those water bodies that are dominated by tidal dispersion and have negligible net freshwater flow.

In general, as indicated in Section III-B the breakdown between the tidal river and the estuary is given in terms of the estuarine number, $n = K_d E/U^2$, thus for:

$n \approx 1-10$ are considered as tidal rivers

$n > 10$ are considered as estuaries

Guidelines are presented throughout Part V to aid the analyst in estimating water quality responses for both situations. The appropriate general equation for the tidal river and estary situation must include the effects of tidal mixing occasioned by the effect of current reversals. This mixing effect is introduced in the model through a tidal dispersion coefficient. Section IIIB-2 discusses the importance of the dispersion effect and sets the stage for the simplified model. The differential equation for a non-conservative variable such as BOD is given by Equation III-2.

Water quality analyses of tidal rivers may be classified in a manner similar to that of streams; in accordance with the reactive nature of constituents in the waste waters, which may be either conservative or non-conservative.

1. Conservative Substances

The analysis for conservative substances is identical to that for streams. The concentration is simply the mass rate of waste discharge divided by the freshwater flow:

$$c_o = \frac{W}{Q} \tag{V-1}$$

It can be seen that this equation results from Equation III-4a for conservative substances, K = 0.

Equation V-1 may be applied to substances such as total dissolved solids, nutrients and other material which decay at such slow rates that they may be regarded as conservative.

Furthermore, as an approximation, substances which decay at more rapid rates such as bacteria, may also be assumed to fall in this classification. The concentrations calculated by Equation V-1 for such substances, are obviously on the conservative side, since they are the probable maximum values in the estuary which may be anticipated from a particular source.

If more accurate estimates are desirable, the concentration should be calculated in accordance with the procedure for non-conservative substances.

2. Non-conservative Substances

Many substances decay in accordance with a single reaction or at least for practical engineering purposes may be assumed to decay in this fashion. As discussed previously, the reaction is assumed to be first order with a coefficient, K.

The concentration of these substances is given by Equation III-3 repeated here as:

$$c_1 = c_o e^{gx} \qquad x \leqslant 0 \qquad \text{(V-2a)}$$

$$c_2 = c_o e^{jx} \qquad x \geqslant 0 \qquad \text{(V-2b)}$$

in which:

$$c_o = \frac{W}{Qm}; \; m = \sqrt{1 + 4KE/U^2}$$

$$g = \frac{U}{2E}[1 + m], \; j = \frac{U}{2E}(1 - m)$$

As with the stream analysis it should be noted that the following assumptions have been made:

a) steady-state

b) constant coefficients exist, i.e., flow, cross-sectional area, reaction kinetics and dispersion characteristics are all constant along the length of the estuary under study.

c) point waste sources only are considered

The concentration, C_o, is at the location of the waste discharge (x = 0) at mean tide and represents the initial concentration. In Equation V-2 the subscript, 1, identifies the upstream segment and the subscript, 2, the downstream segment. The profile oscillates from high to low tide.

In the tidal river case, it should be noted that the dilution consist of both the freshwater flow and the effects of tidal dispersion. This is shown in Equation V-2 where:

$$c_o = \frac{W}{Qm} = \frac{W}{Q'} = \frac{f_4 f_3 f_1 P_o}{(f_5 DA + f_2 f_1 P_o)m} \qquad \text{(V-3)}$$

where Q' is the effective tidal dilution flow which for positive dispersion is greater than Q. As discussed in Section

III B-3, values of $n = KE/U^2$ for tidal rivers range from 1 - 10 (Figure III-1). Thus m ranges from about 1.4 - 3.3.

In accordance with the discussion in Section III B-3, if the estuarine number $n = KE/U^2$ is sufficiently large (>10), the freshwater flow may be neglected. This has been designated as the estuary case. The solution for this situation is given by Equation III-5a for $P \simeq 1$ repeated here as:

$$c = c_o e^{gx} \qquad x \leqslant 0 \tag{V-4a}$$

$$c = c_o e^{jx} \qquad x \geqslant 0 \tag{V-4b}$$

where:

$$c_o = \frac{W}{2A\sqrt{KE}} = \frac{W}{R} \tag{V-4c}$$

$$g = \frac{\sqrt{K}}{\sqrt{E}}, \quad j = -\frac{\sqrt{K}}{\sqrt{E}}$$

Note that the initial concentration now depends on a "dilution" flow R, which incorporates the cross-sectional area, reaction and dispersion coefficients.

The relationships indicated in Equation V-2 and V-4 are based on a constant cross-sectional area, A. Since most estuaries vary in cross-section along the axis of flow this area must be estimated as the average over which the profile at mean tide extends. For highly reactive substances (K > 2/day) this distance may be in the order of 10 or 20 miles while for moderately reacting material $K < 0.5$/day it may be as much as 50 miles. The difficulty in assigning a realistic average area over such distances is evident from a casual inspection of a geographic map of the coastal area of the United States. A common physical feature of the topography not taken into account by the above model is the number of tributaries which feed many estuaries and the delta network which characterizes many estuarine mouths. Obviously, a more complete mathematical description of the estuarine structure is required for such situations. In spite of these difficulties, at least, some engineering approximation may be made and the error introduced is invariably on the conservative side.

Table V-1 presents ranges of values for reaction coefficient in tidal rivers (and estuaries) for the pertinent substances.

TABLE V-1

FIRST ORDER RANGE OF VALUES FOR REACTION COEFFICIENTS TIDAL RIVERS AND ESTUARIES

Substance	K-per day
Coliform	2 - 4
BOD	0.2 - 0.5
Nutrients	0.1 - 0.25 or conservative (K = 0)

3. Dissolved Oxygen Analysis

The analysis for dissolved oxygen proceeds in accordance with the discussion in Section III-B entitled "Mathematical Models". The waste discharge causes a drop in the dissolved oxygen concentration with a subsequent rise further downstream. The tidal river profile is therefore similar to that of the stream. Due to the tidal action, however, the deficit in dissolved oxygen is translated upstream and the associated dispersion flattens the profile. The tidal river profile is therefore projected further upstream and downstream by contrast to the stream profile. The equation of the dissolved oxygen deficit is given by Equation III-13. The location of the maximum deficit and the value of the maximum deficit are given by Equations III-14 and III-15 respectively.

As may be seen from these equations the profile is determined by the ratio, $\Phi = \frac{K_a}{K_d}$ and also the parameter $n = \frac{K_d E}{U^2}$. The following sections relate to a discussion of these factors.

Equation III-13 forms the basis for the multiple waste source analysis discussed below and a variation of that equation is used to construct the single waste source nomograph discussed in Section V-B.

4. Reaction Coefficients

As in the case of the freshwater stream, the surface transfer coefficient, K_L, is a more fundamental expression of reaeration phenomenon. It is related to the volumetric transfer coefficient by the depth.

$$K_a = \frac{K_L}{H} \tag{V-3}$$

where K_L is the surface transfer coefficient [ft/day], H is the average depth at mean tide [ft.] and K_a is the reaeration coefficient [1/day].

The transfer coefficient is a function of the velocity and depth of flow. In the tidal river and estuarine case, the pertinent velocity is the average tidal current. The ranges of transfer and reaeration coefficients which may be encountered in estuary are presented in Table V-2.

TABLE V-2

RANGE OF TRANSFER AND REAERATION COEFFICIENTS ESTIMATED FOR TIDAL RIVERS AND ESTUARIES (K_L in ft/day, K_a in 1/day)

Mean Tidal Depth (ft)	Average Tidal Velocity (fps) 1		1-2		2	
	K_L	K_a	K_L	K_a	K_L	K_a
< 10	4	0.5	5.5	0.6	7	0.8
10 - 20	3	0.2	4.5	0.3	6	0.4
20 - 30	2.5	0.1	3.5	0.14	5	0.2
> 30	2	0.06	2.5	0.08	4	0.12

The probable range of K_L is between 3 - 6 feet/day with limits from 2 to a possible 10 feet per day for a shallow estuary with high tidal velocity. The reaeration coefficient varies more because of the depth effect.

Anticipating the effect of treatment on the oxidation in the natural estuarine environment, the range of the deoxygenation or deaeration coefficient, K_d, may be from 0.2 - 0.5 with a probable average in the order of 0.3, (See Table V-1). This order further assumes that the estuary is no shallower than about 5 feet.

The assimilation ratio, Φ, may readily be tabulated from the above data and is summarized in Table V-3 for different conditions.

TABLE V-3

TABULATION OF ASSIMILATION RATIO - Φ
TIDAL RIVERS AND ESTUARIES

Reaeration Coefficient	K_d			
K_a (1/day)	0.2	0.3	0.4	0.5
0.08	0.4	0.27	0.20	0.16
0.15	0.75	0.50	0.38	0.30
0.30	1.5	1.0	0.75	0.60
0.60	3.0	2.0	1.5	1.2

Tables V-2 and V-3 indicate that the deeper main channel estuaries have Φ values from 0.2 to 0.8, while the shallower tidal tributaries are in the range 0.8 to 3.0. The lower limit of each of these ranges indicates the more restrictted tidal bodies of lower velocity, higher temperatures, and effluents from less advanced degrees of treatment, while the upper limit describes the free flowing, higher velocity estuary, and more advanced treatment in more moderate temperature regions of the country.

5. Estuarine Number

In addition to the assimilation ratio, Φ, the estuarine number, n, is the additional specification which characterizes water quality in tidal rivers and estuaries. As indicated in the Section IIIC-2f, the practical range of the dispersion coefficient is from 1 to 20 (mi^2/day). The upper limit describes the highly saline, high tidal velocity estuarine stretches in the vicinity of the mouth, while the lower limit applies to the upstream, non-saline, low tidal sections of the estuary. The dispersion coefficient, E, with the advection velocity, U, gives a sufficient hydrodynamic definition for each estuary. The advection is that associated with the freshwater flow and is determined by dividing the flow, Q, by the average cross-sectional area, A. This coefficient may therefore vary over a wide range due to the number of geophysical and hydrological factors which affect it, not only within the estuary itself, but also by the characteristics of the drainage basin. The velocity ranges from 0.1 - 10 miles per day. The lower value is frequently insignificant and may be dropped from the analysis without significant error. Neglecting the velocity implies use of the estuary model with zero freshwater flow. A practical lower limit for the velocity is probably in the order of 0.5 mi/day and an upper limit may be 4 mi/day. Within this range the tidal river case applies. The estuarine number, n, is tabulated from this range of advection velocity and a practical range of dispersion coefficient in Table V-4.

TABLE V-4

RANGE OF ESTUARINE NUMBER, $K_d E/U^2$
FOR TIDAL RIVERS
$K_d = 0.3$/day

Tidal Dispersion (sq.mi/day)	Advective velocity - mi/day 0.5	1.0	2.0	4.0
2	2.4	0.6	0.15	0.04
5	6.0	1.5	0.38	0.10
10	12.0	3.0	0.75	0.19
20	24.0	6.0	1.5	0.75

In view of the fact that the oxidation coefficient, K_d, may vary from 0.2 to 0.5 per day, the values indicated in the above table may vary by 50% or more. On the basis of the analysis given in Section III-B and the preceeding data, a generalized range of n = 1 - 10 was assigned to tidal rivers.

A summary of the above tabulations with approximate physical descriptions of the types of tidal rivers and estuaries is presented in Table V-5.

TABLE V-5

CLASSIFICATION OF TIDAL RIVERS AND ESTUARIES

	Assimilation Ratio (Φ)		KE [mi^2/day^2]		Estuary Number "n"	
Description	Average Value	Range	Aver. Value	Range	Average Value	Range
Large, deep, main channel in vicinity of mouth	0.3	0.1-0.5	10	5 -20	15	5 -30
Moderate navigation channel, upstream from mouth saline, large tidal tributaries	0.5	0.2-1.0	3	2 - 5	5	2 -10
Minimum navigation upstream, smaller saline or nonsaline tidal tributaries	1.0	0.5-2.0	1.5	1 - 2	2	0.5- 5
tidal tributaries, shallow and nonsaline	2.0	1.0-3.0	.5	.2-1	1	0.2- 2

A tabulation of a number of estuaries on which water quality analysis were performed are presented in Table C-2 in Appendix C.

B. SINGLE WASTE SOURCE

1. BOD, Coliform Bacteria, Nutrients

For single waste sources, the analysis for water quality responses depends on determination of the intial concentration. The situation is similar to that of streams as shown in Figure IV-4 and discussed in Section IV-B. Thus, the initial concentration represents the maximum expected concentration and forms a conservative basis for comparison to standards. Equations V-1 and V-3 and V-4c provide the means for computating the initial concentration at the outfall. The primary difference between the tidal river analysis and the stream analysis lies in estimating the effective dilution flow, "Q" = Qm or the estuary dilution flow $R = 2A\sqrt{KE}$.

2. Dissolved Oxygen

A nomograph has been constructed for the analysis of the dissolved oxygen response due to a single waste source in a tidal river. The nomograph is in two parts - Charts D and E. This diversion is necessary because of the appearance of flow, cross-sectional area and tidal dispersion characteristics in the DO relationship for tidal rivers.

The analyst begins on Chart D with estimates of the cross-sectional area, net non-tidal flow (freshwater flow), the assimilation ratio, and the parameter KE (miles/day)^2. Estimates of the area are generally available from U.S. Coast & Geodetic Survey Charts. Freshwater flow data can be obtained from USGS surface water records or drainage area information (see Chart A and Figure III-4). The waste flow, if significant, should be added to the freshwater flow. Chart A may again be utilized knowing the population served. The parameter KE is the product of the deoxygenation coefficient and the dispersion coefficient. The deoxygenation coefficient for BOD is about 0.2 -0.5/day (See Table V-1) and a good estimate in the absence of any other data is 0.3/day. The dispersion coefficient is somewhat more difficulat to evaluate. Section III-C-2f and the preceeding section, V-A-5, discuss this parameter. Particular attention is directed to Table V-5. Figure D-1 of Chart D gives a qualitative description of the range of KE in different sections of a tidal river. Table C–2 in Appendix C lists dispersion coefficients and the product KE for a variety of actual estuarine situations. This Table can also be used as a guide in the absence of observed data.

Entering Chart D with the cross-sectional area, the analyst proceeds in Figure D-1 to the value of KE. Proceeding to Figure D-2 to the left, the net non-tidal flow is introduced and the "effective dilution flow" is estimated at the botton of Figure D-2. This value is required in Figure E-2 of Chart E. This effective dilution flow can be seen in Equation V-2 where the maximum BOD concentration is given at the outfall as the initial concentration. For zero net non-tidal flow, the estuary case, the effective dilution flow is given only in terms of the dispersion (see Equation V-4c).

For tidal rivers, where net non-tidal flow is significant, the analyst must also proceed to Figure D-3 from Figure D-1, where the net non-tidal flow is again required. The output from Figure D-3 is the estuary number $n = KE/U^2$. This number is then used in Figure D-4 together with the assimilation ratio, Φ, to determine the "correction factor", which is needed in Chart E-4. This factor is the ratio of the maximum DO deficit for a tidal river, with significant net non-tidal flow, to the maximum deficit for an estuary when the net non-tidal flow is zero.

The assimilation ratio can be estimated from the qualitative information given in Table V-5 or if the information is available from Tables V-2 and V-3.

Two points should be noted in Chart D. First, if the net non-tidal flow is essentially zero, the estuary case, the analyst need only deal with Figure D-1 and D-2. It will be seen that Figure D-3 and D-4 will result in a correction factor of one. Secondly, if a non-interacting variable such as coliform bacteria or total nitrogen is being considered then the analyst again need only use Figure D-1 and D-2. The output from Figure D-2 is then used in computing the initial concentration. The latter Figures D-3 and D-4 are thus used only when dissolved oxygen is being analyzed.

Considering now Chart E, the procedure is similar to that used for single waste sources in streams and rivers. Figure E-1 is identical to that used for streams (Figure B-1) The analyst can begin with the design population and treatment level to estimate the effluent ultimate oxygen demanding load. Figure E-2 together with the "effective dilution flow" from Chart D then permits estimation of the value of the BOD at the outfall. From Figure E-2, the analyst proceeds to E-3 which, with the estimate of the assimilation ratio, Φ results in an estimate of the maximum DO deficit, if no net non-tidal flow were present. Figure E-4 corrects this assumption by utilization of the "correction factor" previously determined from Chart D. Figure E-5 converts the maximum DO deficit to a minimum dissolved oxygen depending on the temperature and salinity concentrations. The output from Figure E-5 is therefore the minimum DO to be expected in the estuary due to the population and treatment level specified by Figure E-1. In the absence of observed data, a background DO deficit of 1 mg/l is suggested. If the minimum DO does not meet the required standards, then a reverse procedure can be followed. The required minimum DO is used in Figure E-5 and keeping all values as before, the procedure is reversed and a new level of treatment in Figure E-1 is estimated.

a. Illustrative Example

Consider the following basic data for a single waste source located along a tidal river:

Design Population - 51,000
Proposed Treatment - High Rate Biological
DO Standard - 5.0 mg/l
Fresh Water Drought Flow - 750 cfs
Average Tidal Velocity, 1 - 2 feet/second
Mean Tide Depth - 15 feet
Cross-sectional Area - 7500 ft^2
Chloride 18,000 mg/l
Temperature = 25°C

An examination of the geographic location of the river indicates that it is upstream from the mouth, still navigable by small craft vessels and in the saline zone.

Using Tables V-2, V-3, and V-5, it is estimated that the assimilation ratio is about 1.0 and the value of KE is about 1.5 mi^2/day^2.

From Figure D-1, at the cross-sectional area of 7500 ft^2 and KE = 1.5, to Figure D-2 at a net non-tidal flow of 750 cfs yield an effective dilution flow of about 1400 cfs. Note that the waste flow was not added to the net non-tidal flow in this case bacause of its relatively insignificant magnitude when compared to the freshwater flow. Also, Figure D-3 gives an estuary number of about 0.6 and from Figure D-4 at $\Phi = 1.0$, a correction factor of 0.9.

Proceeding to Chart E, Figure E-1 gives an effluent load at the design population of about 10,000 lbs/day. Using the effective dilution flow of 1400 cfs (from Figure D-2). Figure E-2 shows a maximum BOD concentration of about 1.3 mg/l. Figure E-3, for an assimilation ratio of 1.0, gives a deficit of about 0.6 mg/l. Figure E-4 at a correction factor of 0.9, together with Figure E-5, at 18,000 mg/l chlorides and 25°C and a background DO deficit of 1.0 mg/l, yields an estimated minimum DO of about 5.0 mg/l. The proposed treatment is therefore adequate within the limits of the analysis.

C. Multiple Waste Sources - Dissolved Oxygen

The procedure that is suggested for the analysis of dissolved oxygen in estuaries with multiple waste sources is similar to that suggested for streams (Section IV). For estuaries however, the introduction of tidal dispersion does not permit construction of a readily usable nomograph as for streams (See Chart C). The fundamental task in multiple source analysis for estuaries is to determine the individual DO deficit profile resulting from each waste source. These profiles can then be superimposed to determine the total deficit. For streams, it is possible to describe the "influence lines" of DO deficit in a nomograph (Chart C) or to graphically approximate the stream DO profile as discussed in Section IV. For estuaries, the display of the "influence lines" is more complicated due to tidal dispersion. Tables have therefore been prepared which provides the analyst with the unit responses of DO deficit for a wide class of tidal rivers and estuaries.

The multiple waste source procedures for estuaries uses Equation III-13 as its basis. As indicated in the discussion of that equation, the response ratio, D/L_o is given by two dimensionless numbers:

$$\Phi = K_a/K_d$$

And:

$$n = K_d E/U^2$$

for the dimensionless distance, $x^* = K_d x/u$. Therefore, for various values of x^* beginning at $x^* = 0$ at the outfall, the ratio of the DO deficit to a unit initial concentration of ultimate BOD can be tabulated (by Equation III-13) for different values of Φ and n. It should be recalled (Equation III-4a) that:

$$L_o = \frac{W}{Qm} = \frac{W}{Q'}$$

where $m = \sqrt{1 + 4n}$ and Q' is the effective dilution flow. Chart D can be used to obtain the effective dilution flow (Figure D-1 to Figure D-2) and also can be used to estimate the parameter n. With the effective dilution flow, Figures E-1 and E-2 of Chart E permit estimation of the intial BOD concentrations at the outfall, L_o. Appendix D, Tables of DO Response for Tidal Rivers and Estuaries, lists the mg/l DO deficit per mg/l BOD at the outfall over a range of Φ, n and x^*. The columns of each of the tables therefore represents the DO deficit profile per unit of BOD for a fixed Φ and n. The actual DO profile is then given by multipling the unit responses by the total L_o as determined from Figure E-2.

A Table similar to Table IV-2 is useful in arraying the total response. Alternately, the individual deficit responses can be graphically plotted and then superimposed, as in the graphical stream analysis (see, for example, Figure IV-7).

The procedure therefore can be summarized as follows:

1) All pertinent waste sources along the estuary are listed and their estimated effluent loads (lbs/day) and flows (cfs) are obtained (e.g., Figure E-1).

2) With the system parameters, (cross-sectional area, KE and Q) Figure D-1 and D-2 provide the effective dilution flow.

3) Figure E-2 provides the ultimate BOD at the outfall.

4) The Tables of Appendix D provide the unit DO deficit response for Φ and n over the distance x*.

5) Multiplication of the responses by L_o as determined from (3), gives the DO deficit profile due to the first sources.

6) Procedure (1) to (5) is repeated for each waste source.

7) The individual DO deficit profiles are added to provide the total DO deficit due to all sources.

8) A constant background deficit may be assumed (1 mg/l) and added to the total deficit profile above.

9) The total deficit profile is subtracted from the saturation value of DO to determine the actual dissolved oxygen profile under the design conditions.

10) A comparison of this estimated DO profile and DO standard can then be made to determine any regions of violations.

11) If standards are violated, increase treatment levels of all or some combination of sources to next discrete level and repeat procedure.

APPENDIX A

DISCUSSION OF REACTION COEFFICIENTS FOR DISSOLVED OXYGEN ANALYSIS IN STREAMS

TECHNICAL APPENDIX A

DISCUSSION OF REACTION COEFFICIENTS FOR DISSOLVED OXYGEN ANALYSIS IN STREAMS

1. AERATION

The transfer of oxygen or any sparingly soluble gas from the atmosphere to water is essentially a surface controlled phenomenon, the resistance to transfer being localized at the air-liquid interface. It is more appropriate therefore to express the transfer in terms of the surface coefficient K_L rather than the volumetric coefficient K_a. The relation between the two is:

$$K_a = K_L \cdot \frac{A}{V} = \frac{K_L}{H} \qquad \text{(A-1)}$$

in which:

K_a	=	reaeration coefficient (per day)
K_L	=	surface transfer coefficient (feet/day)
A	=	surface area of river reach
V	=	volume of river reach
H	=	average depth

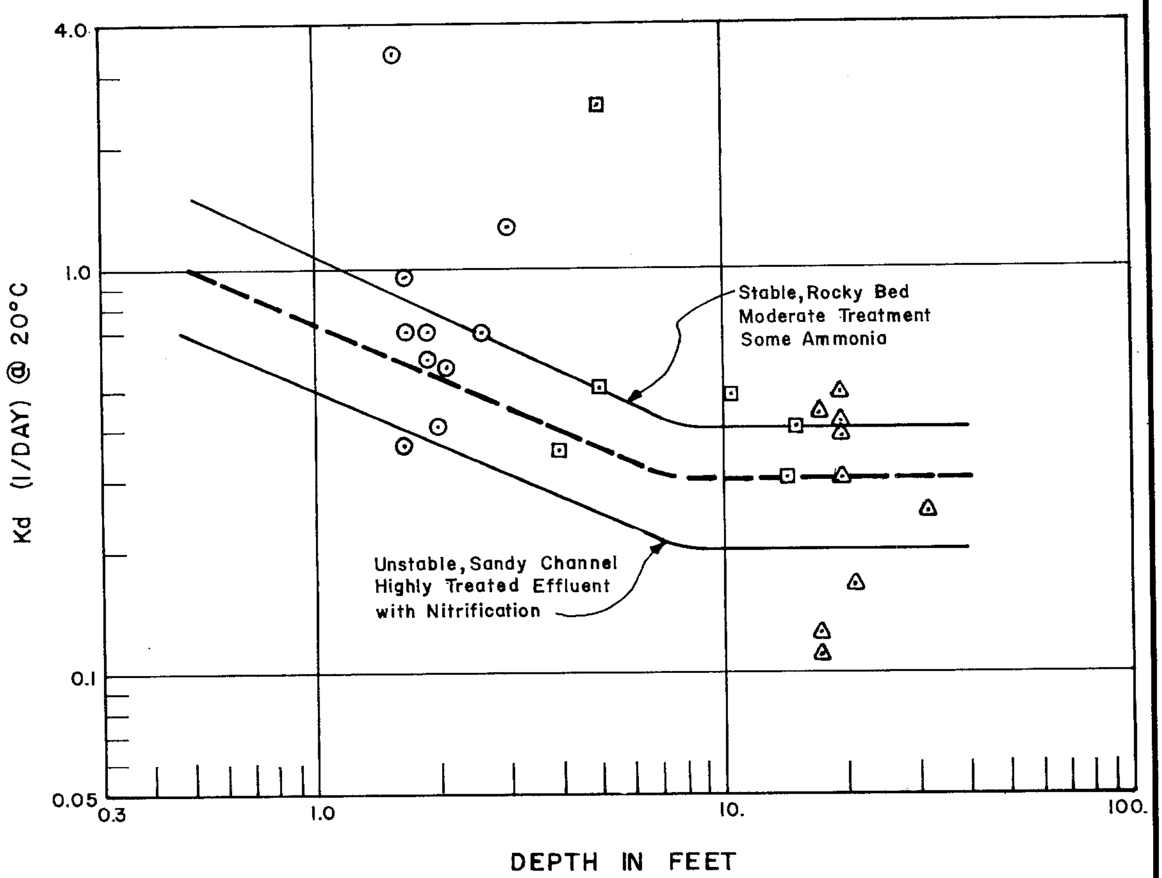
It has been shown both theoretically and experimentally that these coefficients are directly proportional to the velocity of the stream, raised to some power "a" and inversely reated to the average depth of the stream to a power "b". The experimental values have been derived from both field and laboratory tests. Only the field data, abstracted from the work of the TVA group and of the Stevenage Laboratory in England, are used in the subsequent sections of this Appendix.

A plot of the transfer coefficient versus the average depth of the stream is shown in Figure A-1 for various ranges of velocity. For each range of line of correlation has been sketched in graphically. The English Data for K_L greater than 40 were not weighed in the fit. The general pattern of lines substantiate the general functional relationship between the transfer and velocity and depth. Values in the upper range may be limited by laminar flow conditions ($H < 1.0$ ft) and are therefore affected primarily by the velocity and independent of the depth. In the lower range, a minimum value in the order of 2 feet per day is an approximate limit.

The indicated correlations have been converted to the reaeration coefficient, K_a, by means of Equation A-1, and are presented in the Figure A-2 for the three ranges of velocities which may be expected during low flow conditions. The three curves are also indicative of the range of water surface conditions, and river bed characteristics. Broken turbulent water surfaces, usually associated with rough steep channels are described in the upper curve, while the lower one is reppresented of unbroken water surfaces, flat slopes and smooth channels.

2. DEAERATION COEFFICIENT

A similar correlation has been crudely developed for the deaeration or deoxygenation coefficient K_d and average depth, H. The rationale behind this correlation lies in fact that the greater the wetted perimeter to cross-section, which is equivalent to small depth, the greater the contact with the biological file in stream bed, which is the one most important factor in natural oxidative processes. The tendency for this relation to hold is greater for the rocky stream bed rather than a silty bed. However the general trend appears reasonable up to a depth of about 5 to 10 feet. Data surveys, reported in the literature and conducted by Hydroscience, are plotted in Figure A-3. Although bed conditions were not always described with great precision, sufficient information was usually available to justify the descriptive terms associated with correlations. The upper line refers to a stable



LEGEND

⊙ Shallow Streams (1-3 Ft.)
⊡ Medium Streams (3-15 Ft)
△ Deep Rivers (> 15 Ft)

FIGURE A-3

DEOXYGENATION COEFFICIENT (Kd) AS
A FUNCTION OF DEPTH

rocky channel bed, with benthal communities similar to those found in trickling filters. The lower line is descriptive of unstable channels with biological sparse benthal communities.

Furthermore the nature of the residual organic matter in the effluent from the treatment plant is a determining factor. A large portion of the material is probably residual bacterial cells, since most of the complex organic matter in the raw waste has been converted in the biological treatment processes. Thus the lower limit represents highly treated, well oxidized effluents with efficient secondary sedimentation of the active bacterial populations. The residues are therefore highly stable organically with low rates of oxidation with respect to both carbon and nitrogen. The upper limit is representative of wastes with more residual organic material, some of which is capable of relatively rapid oxidation. The upper limit is also indicative of higher values of ammonia, typical of high rate biological systems, in which nitrification is minimal. The middle curve is therefore characteristic of effluents with some but not total nitrification. Assuming the lower and middle curves will be most representative of effluents from treatment plants to be constructed, they have been transfered to Figure A-3 to indicate the limits which may be anticipated in the near future.

3. ASSIMILATION RATIOS

The fundamental ratio which is most indicative of the river capacity is the ratio of the reaeration coefficient (K_a) and the deoxygenation coefficient (K_d). Inspection of Figures A–2 and A–3 show that the upper limits of each of these parameters usually co-exist in a given stream, since the hydraulic and geophysical characteristics are similar. The ratio Φ (K_a/K_d) is plotted in Figure A-4 versus the average depth, the correlations being developed directly from those in Figure A-2 and A-3. The heavy solid line of the figure represents the average case. The practical range and probable limits are also indicated. If actual data is available from stream surveys to determine Φ, this information should obviously be employed in the DO analysis and Figure A-4 may be by-passed. If however, the engineer has no data with which to estimate the individual coefficients, the ratio may be estimated simply from the depth. Further information on the nature of the stream and the anticipated treatment enables the analyst to make a further refinement within the range indicated. If no information is available, the probable average correlation is the practical choice.

A final note of caution is introduced by reminding the analyst that the numerical values of assimulation ratio

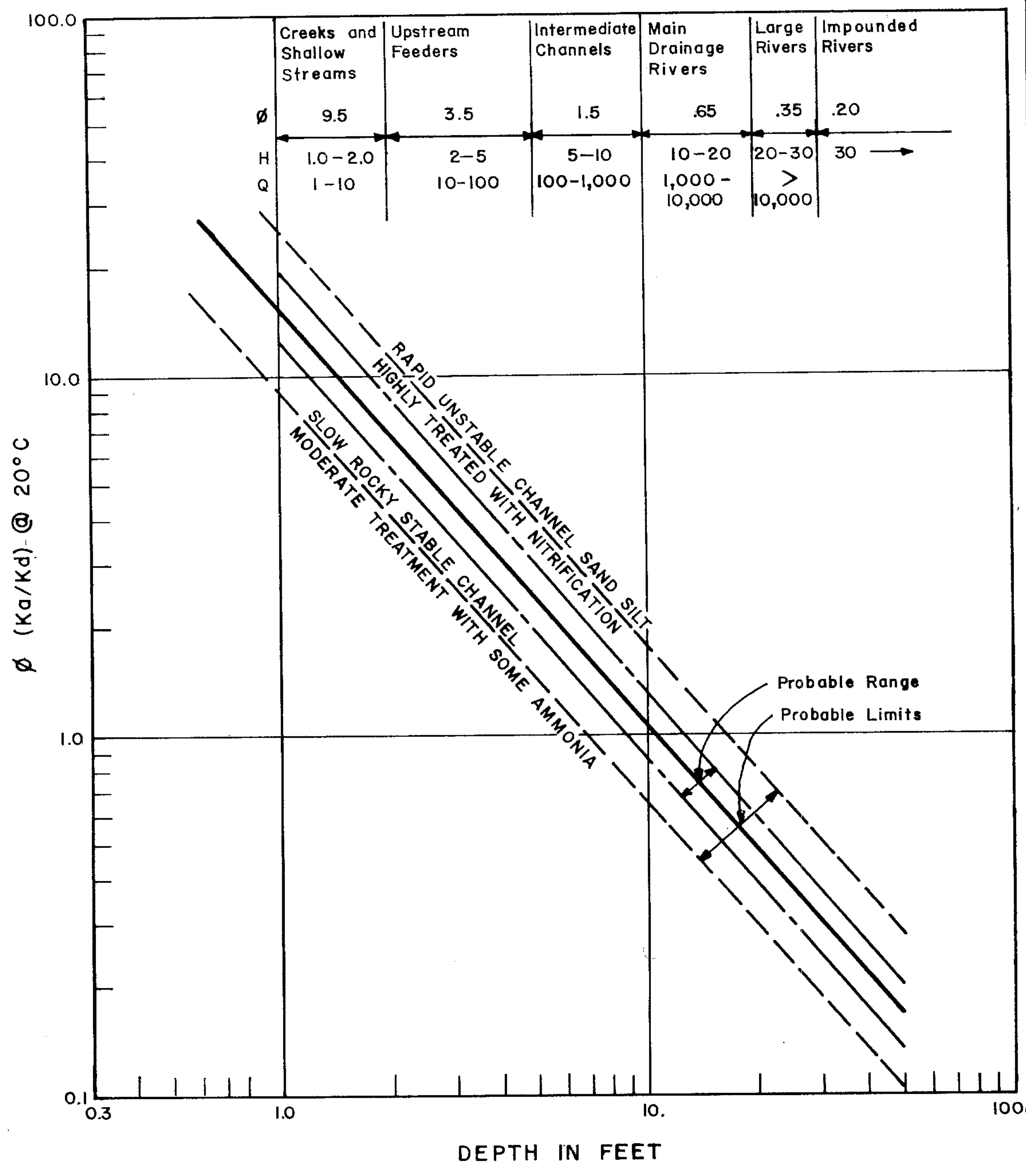


FIGURE A-4

ASSIMILATION RATIO (Ø) AS A FUNCTION OF DEPTH

Φ, suggested in this report are characteristic of moderate to well-treated effluents in a stream with low to moderate velocities. The correlation shown in this figure may therefore not be representative of existing conditions in many streams receiving wastes from overloaded or poorly operated treatment plants. For wastewaters receiving only primary or less treatment, the likelihood or correlation is even less.

The final step is the conversion of the ratio Φ, to (D_c/L_o), indicated in Figure IV-3 of the main report. The relation between D_c/L_o and the average depth, H, with the limits as indicated, is shown in Figure A-5. The relation presented in Figure A-4, is the basis of the nomographs presented in the stream analysis section IV-C and IV-D.

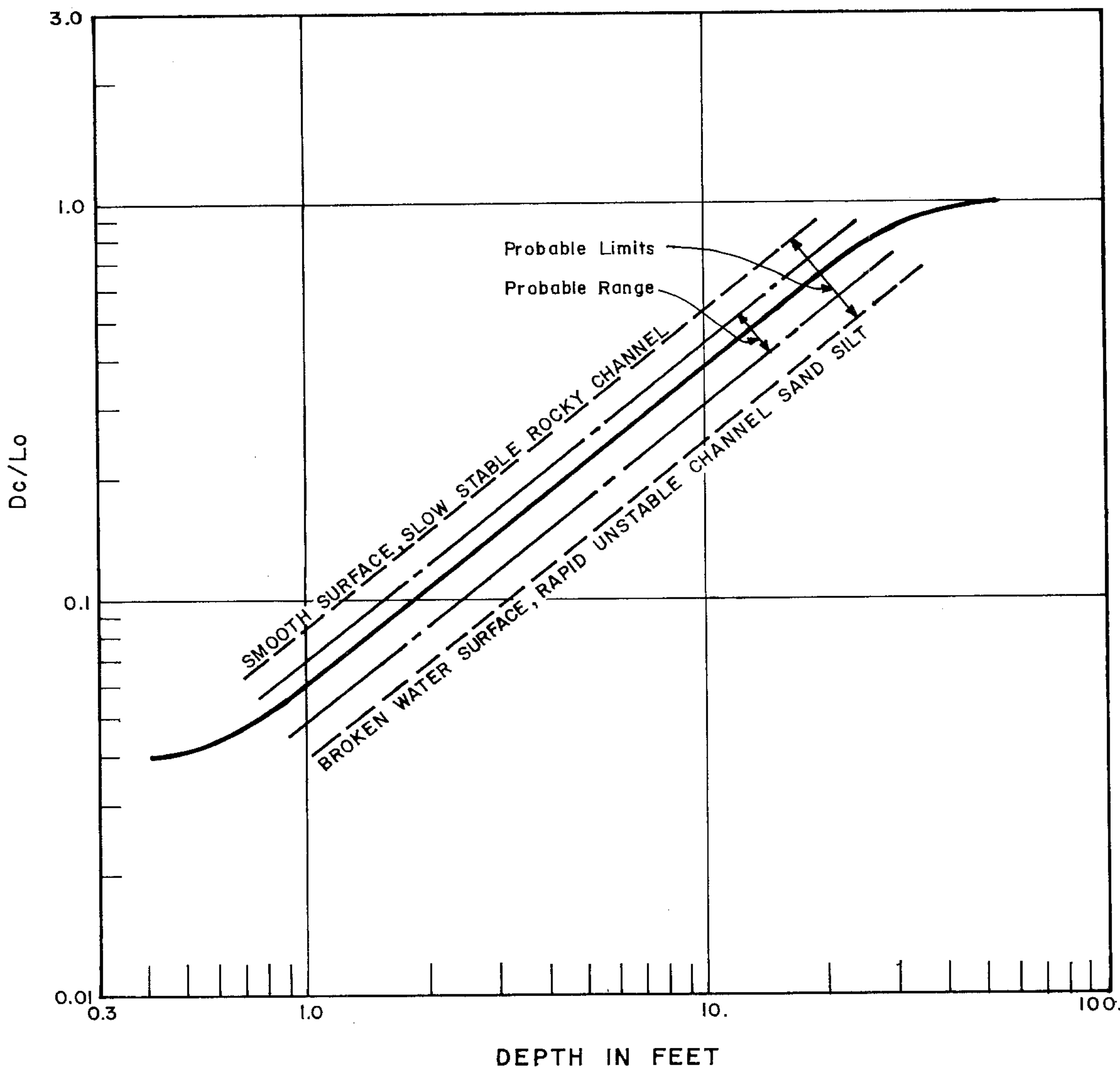


FIGURE A-5

RATIO Dc/Lo AS
A FUNCTION OF DEPTH

APPENDIX B

TABLES OF POPULATION ESTIMATES

POPULATION ESTIMATES FOR STANDARD METROPOLITAN STATISTICAL AREAS

Metropolitan Area	Average Annual Percent Change
Abilene, Texas	1.4
Akron, Ohio	0.8
Albany-Schenectady-Troy, New York	0.7
Albuquerque, New Mexico	2.0
Allentown-Bethlehem-Easton, Pennsylvania	0.5
Altoona, Pennsylvania	-0.4
Amarillo, Texas	2.0
Ann Arbor, Michigan	1.8
Asheville, North Carolina	1.0
Atlanta, Georgia	2.1
Atlantic City, New Jersey	1.2
Augusta, Georgia-South Carolina	0.5
Austin, Texas	2.0
Bakersfield, California	1.6
Baltimore, Maryland	1.0
Baton Rouge, Louisiana	1.8
Bay City, Michigan	0.4
Beaumont-Port Arthur, Texas	0.6
Binghamton, New York	0.6
Birmingham, Alabama	0.1
Boston-Lawrence-Haverhill-Lowell, Mass.[1]	0.4
Bridgeport-Stamford-Norwalk, Connecticut[2]	1.6
Brockton, Massachusetts[3]	2.0
Buffalo, New York	2.0

1 Data shown for Massachusetts State Economic Area C. For SMSA's 1960 population was: Boston 2,595,481; Lawrence-Haverhill, 199,136; Lowell, 164,243.

2 Data shown for Connecticut State Economic Area A. For SMSA's, 1960 population was Bridgeport, 337,983; Stamford, 178,409; Norwalk, 96,756.

3 Data shown for Massachusetts State Economic Area D. For Brockton SMSA, 1960 population was 149,458.

POPULATION ESTIMATES FOR STANDARD METROPOLITAN STATISTICAL AREAS

Metropolitan Area	Average Annual Percent Change
Canton, Ohio	0.5
Cedar Rapids, Iowa	1.0
Charleston, South Carolina	1.1
Charleston, West Virginia	-0.7
Charlotte, North Carolina	1.8
Chattanooga, Tennessee-Georgia	0.4
Chicago, Illinois	0.9
Cincinnati, Ohio-Kentucky	0.5
Cleveland, Ohio	0.4
Colorado Springs, Colorado	0.9
Columbia, South Carolina	1.5
Columbus, Georgia-Alabama	0.3
Columbus, Ohio	1.5
Corpus Christi, Texas	0.4
Dallas, Texas	2.1
Davenport-Rock Island-Moline, Iowa-Illinois	0.8
Dayton, Ohio	1.1
Decatur, Illinois	0.6
Denver, Colorado	2.0
Des Moines, Iowa	0.2
Detroit, Michigan	0.5
Duluth-Superior, Minnesota-Wisconsin	-0.5
Durham, North Carolina	1.3
El Paso, Texas	1.4
Erie, Pennsylvania	(Z)
Eugene, Oregon	1.9
Evansville, Indiana-Kentucky	-0.3
Flint, Michigan	0.9
Fort Lauderdale-Hollywood, Florida	4.3

(Z) Less than 500 or 0.05 per cent.

POPULATION ESTIMATES FOR STANDARD METROPOLITAN STATISTICAL AREAS

Metropolitan Area	Average Annual Percent Change
Fort Wayne, Indiana	1.3
Fort Worth, Texas	1.3
Fresno, California	1.9
Galveston-Texas City, Texas	1.2
Gary-Hammond-East Chicago, Indiana	0.8
Grand Rapids, Michigan	0.9
Greensboro-High Point, North Carolina	1.4
Greenville, South Carolina	0.8
Hamilton-Middletown, Ohio	1.0
Harrisburg, Pennsylvania	0.6
Hartford-New Britain, Connecticut[1]	1.4
Honolulu, Hawaii	1.2
Houston, Texas	2.1
Huntington-Ashland, West Virginia-Ky.-Ohio	2.1
Huntsville, Alabama	4.3
Indianapolis, Indiana	0.7
Jackson, Michigan	0.8
Jacksonville, Florida	1.4
Jersey City, New Jersey	-0.2
Johnstown, Pennsylvania	-0.9
Kalamazoo, Michigan	1.4
Kansas City, Missouri-Kansas	0.7
Kenosha, Wisconsin	1.6
Knoxville, Tennessee	0.4
Lake Charles, Louisiana	0.3
Lancaster, Pennsylvania	0.6
Lansing, Michigan	1.5

1 Data shown for Connecticut State Economic Area C. For SMSA's 1960 population was: Hartford, 549,249; New Britain, 129,397.

POPULATION ESTIMATES FOR STANDARD METROPOLITAN STATISTICAL AREAS

Metropolitan Area	Average Annual Percent Change
Las Vegas, Nevada	3.9
Lexington, Kentucky	2.2
Lima, Ohio	0.9
Lincoln, Nebraska	0.6
Little Rock-North Little Rock, Arkansas	1.4
Lorain-Elyria, Ohio	1.6
Los Angeles-Long Beach, California	2.3
Louisville, Kentucky-Indiana	0.6
Lubbock, Texas	2.1
Lynchburg, Virginia	1.5
Macon, Georgia	1.2
Madison, Wisconsin	2.0
Manchester, New Hampshire[1]	1.6
Memphis, Tennessee-Arkansas	1.1
Miami, Florida	2.0
Milwaukee, Wisconsin	0.6
Minneapolis-St. Paul, Minnesota	1.2
Mobile, Alabama	1.1
Monroe, Louisiana	1.4
Montgomery, Alabama	0.3
Muncie, Indiana	0.8
Nashville, Tennessee	1.3
New Bedford-Fall River, Massachusetts[2]	0.2
New Haven-Waterbury-Meriden, Connecticut	1.0

1 Data shown for New Hampshire State Economic Area A. For Manchester SMSA, population was 102,861.

2 Data shown for Massachusetts State Economic Area E. For SMSA;s 1960 population was: New Bedford, 143,176; Fall River, 138,156.

POPULATION ESTIMATES FOR STANDARD METROPOLITAN STATISTICAL AREAS

Metropolitan Area	Average Annual Percent Change
New Orleans, Louisiana	1.3
New York, New York	0.6
Newark, New Jersey	1.0
Newport News-Hampton, Virginia	1.2
Norfolk-Portsmouth, Virginia	0.7
Odgen, Utah	1.2
Oklahoma City, Oklahoma	1.4
Omaha, Nebraska-Iowa	1.3
Orlando, Florida	2.7
Paterson-Clifton-Passaic, New Jersey	1.3
Pensacola, Florida	1.4
Peoria, Illinois	0.7
Philadelphia, Pennsylvania-New Jersey	0.9
Phoenix, Arizona	3.0
Pittsburgh, Pennsylvania	-0.3
Pittsfield, Massachusetts[1]	0.2
Portland, Maine[2]	0.6
Portland, Oregon-Washington	1.1
Providence-Pawtucket-Warwick, Rhode Island[3]	0.2
Pueblo, Colorado	0.3
Racine, Wisconsin	1.5
Raleigh, North Carolina	1.8
Reading Pennsylvania	0.2

1 Data shown for Massachusetts State Economic Area F. For Pittsfield SMSA, 1960 population was 76,772.

2 Data shown for Maine State Economic Area A. For Portland SMSA, 1960 population was 139,122.

3 Data shown for Rhode Island State Economic Area A. For Providence-Pawtucket-Warwick SMSA, 1960 population was 821,101.

POPULATION ESTIMATES FOR STANDARD METROPOLITAN STATISTICAL AREAS

Metropolitan Area	Average Annual Percent Change
Richmond, Virginia	1.7
Roanoke, Virginia	1.5
Rochester, New York	1.3
Rockford, Illinois	1.2
Sacramento, California	2.8
Saginaw, Michigan	1.0
St. Louis, Missouri-Illinois	0.7
Salt Lake City, Utah	1.8
San Antonio, Texas	1.1
San Bernardino-Riverside-Ontario, Calif.	3.1
San Diego, California	1.9
San Francisco-Oakland, California	1.6
San Jose, California	4.2
Santa Barbara, California	4.3
Savannah, Georgia	0.2
Scranton, Pennsylvania	-1.1
Seattle, Washington	1.2
Shreveport, Louisiana	0.4
Sioux City, Iowa	-0.9
South Bend, Indiana	0.1
Spokane, Washington	-0.2
Springfield, Illinois	0.4
Springfield, Missouri	1.6
Springfield, Ohio	1.2
Springfield-Chicopee-Holyoke, Massachusetts[1]	0.6
Stockton, California	1.6
Syracuse, New York	1.3
Tacoma, Washington	0.9

1 Data shown for Massachusetts State Economic Area A. For Springfield-Chicopee-Holyoke, SMSA 1960 population was 493,999.

POPULATION ESTIMATES FOR STANDARD METROPOLITAN STATISTICAL AREAS

Metropolitan Area	Average Annual Percent Change
Tampa-St. Petersburg, Florida	2.5
Terre Haute, Indiana	(Z)
Toledo, Ohio	0.3
Topeka, Kansas	0.6
Trenton, New Jersey	1.3
Tucson, Arizona	2.2
Tulsa, Oklahoma	0.4
Tuscaloosa, Alabama	1.3
Utica-Rome, New York	0.8
Waco, Texas	0.5
Washington, D.C., Maryland-Virginia[1]	2.3
Waterloo, Iowa	0.3
West Palm Beach, Florida	3.0
Wheeling, West Virginia-Ohio	-0.4
Wichita, Kansas	-0.1
Wichita Falls, Texas	1.1
Wilkes-Barre-Hazelton, Pennsylvania	-0.6
Wilmington, Delaware-New Jersey	1.7
Winston-Salem, North Carolina	1.4
Worcester-Fitchburg-Leominster, Massachusetts[2]	0.4
York, Pennsylvania	0.6
Youngstown-Warren, Ohio	0.4

(Z) Less than 500 or 0.05 per cent.

1 Adjusted to exclude 12,520 erroneously reported in Fairfax County.

2 Date shown for Massachusetts State Economic Area B. For SMSA's, 1960 population was: Worchester, 328,898; Fitchburg-Leominster, 90,158.

POPULATION INFORMATION FOR NON-METROPOLITAN COUNTIES

F.W.Q.A. Regional Office or State	Median Percentage Change in Population for 1960—1970 Largest Towns		
	<10,000 population	10,000 to 25,000	25,000 to 50,000
Chicago Region	1.0	5.0	11.0
Atlanta Region	0	6.5	12.0
Philadelphia Region	-1.0	0.5	9.0
New York Region	5.5	8.0	14.0
North East Boston Region	3.0	11.0	17.0
Kansas City Region	-8.0	2.0	10.0
Dallas Region	- .5	3.5	10.0
Denver Region	-10.0	0.5	18.5
San Francisco Region	11.5	2.5	--
Seattle Region	-2.8	4.3	17.0
Oregon	1.7	2.1	30.8
Washington	1.6	9.3	15.7
Idaho	-6.9	9.1	9.1

APPENDIX C

REPRESENTATIVE LISTING OF STREAM AND ESTUARINE PARAMETERS

TABLE C-1

STREAMS AND RIVERS

River Name	Category	Depth (ft.)	Area (ft^2)	Flow (cfs)	Velocity (fps)	K_L (ft/day)	K_a @20°C (1/day)	K_d @20°C (1/day)	Φ (K_a/K_d) (Dimension-less)	K_a/U (1/mi)	Reference
Grand River	Shallow	1.9	320.0	295.	0.92		4.5	0.59	7.6	.0392	3
Clinton R.	Shallow	1.58	44.6	33.	0.72		5.9	3.37	1.7	.286	3
Truckee R.	Shallow	1.67	150.	180.	1.20		5.6	0.36	15.5	.0184	3
		1.67	150.	195.	1.30		5.7	0.36	15.8	.0170	
		1.67	150.	271.	1.81		6.6	0.96	6.8	.0325	
Flint River	Shallow	2.1	210.	134.	0.64		3.5	0.56	6.3	.0535	3
		2.6	210.	174.	0.83		3.9	0.63	5.7	.0509	
		2.6	400.	174.	0.44		3.1	0.69	3.0	.0960	
		1.7	290.	204.	0.73		5.0	0.69	7.3	.0579	
		1.9	400.	204.	0.51		2.2	0.69	3.2	.0926	
Jackson R. (Virginia)	Shallow	3	365.	100.	0.27		4.1	1.25	3.28	.284	40,41,44
N. Branch Potomac (Md., W.Va.)	Shallow	2	100.	100.	1.0		9.0	0.40	22.5	.0245	45

TABLE C–1
(continued)

STREAMS AND RIVERS

River Name	Category	Depth (ft.)	Area (ft^2)	Flow (cfs)	Velocity (fps)	K_L (ft/day)	K_a @20°C (1/day)	K_d @20°C (1/day)	Φ (K_a/K_d) (Dimension-less)	K_a/U (1/mi)	Reference
Clarion R.	Shallow	1		1 - 10.				3			2
		1.9			0.55	9.	2.26				
South River		1-2		35.				2			
Ivel River (England)	Shallow	1.21		4.86	0.14	5.58	2.35				1
		1.51		4.15	0.14	4.64	2.06				
		1.09		3.87	0.13	6.83	3.20				
		1.50		15.40	0.37	6.45	2.37				
		1.08		4.86	0.16	9.75	4.57				
		0.38		4.15	0.15	4.00	2.09				
		1.12		3.87	0.13	2.55	1.18				
		1.46		15.40	0.38	8.65	3.18				
		1.31		15.40	0.47	14.70	6.18				
		2.44		10.07	0.23	4.16	0.90				
		2.03		10.07	0.22	6.35	1.66				
Lark River (England)	Shallow	1.74		10.94	0.28	2.48	0.78				1
		1.47		10.94	0.37	6.04	2.12				
		1.82		36.20	0.50	5.10	1.41				
		2.41		36.20	0.43	1.48	0.31				

TABLE C-1
(continued)

STREAMS AND RIVERS

River Name	Category	Depth (ft.)	Area (ft^2)	Flow (cfs)	Velocity (fps)	K_L (ft/day)	K_a @20°C (1/day)	K_d @20°C (1/day)	Φ (K_a/K_d) (Dimen-sion-less)	K_a/U (1/mi)	Reference
Derwent R. (England)		0.72		21.60	1.37	41.90	31.80				1
		0.89		21.60	1.19	39.70	24.53				
		0.85		21.60	1.07	37.80	34.57				
Black Beth River (England)	Shallow	0.40		2.70	0.44	19.30	25.59				1
		0.40		2.70	0.56	20.90	28.34				
		0.39		2.70	0.63	21.00	22.80				
		0.60		17.70	1.83	54.80	49.17				
		0.69		17.70	1.81	39.40	30.77				
		1.00		17.70	1.54	34.20	18.46				
St. Sunday's Beck (Eng.)	Shallow	0.82		19.10	1.07	32.40	21.05				1
		0.78		19.10	1.27	23.70	16.06				
Yewdale Beck (England)	Shallow	0.64		5.10	0.46	14.40	12.04				1
		0.48		5.10	0.60	19.20	30.32				
		0.72		17.30	1.16	25.30	18.90				
		0.66		17.30	1.31	24.80	20.25				
		0.67		17.30	1.30	21.10	17.09				
		0.69		17.30	1.25	24.60	19.16				
Elk River	Shallow	0.9			0.97	12	5.84				2

TABLE C-1
(continued)

STREAMS AND RIVERS

River Name	Category	Depth (ft.)	Area (ft^2)	Flow (cfs)	Velocity (fps)	K_L (ft/day)	K_a @20°C (1/day)	K_d @20°C (1/day)	Φ (K_a/K_d) (Dimen-sion-less)	K_a/U (1/mi)	Reference
Mohawk River	Shallow	3	143.				.07	.23		.0025	28, 29
Mohawk River (New York)	Medium	15	3800.	800.	.21		20.	.40		.107	
North Branch Susquehanna	Medium	4	1700.	1000.	0.60		1.5	0.35	4.28	.0356	30
New River (Virginia)	Medium	5	1720.	1200.	0.70		1.04	2.5 0.5	0.41 2.1	.229 .0437	15, 16
Wabash	Medium	5-7		1000-5000				.40			
Clinch R.	Medium	3.27		3300.	3.07	17.1	2.27				2
		5.09		4500	3.69	16.2	1.44				
		4.42		3190	3.10	10.0	.98				
		6.14		5890	2.68	7.0	.50				
		5.65		5910	2.78	9.5	.74				
		7.17		5930	2.64	18.7	1.13				
Holston	Medium	11.41		10,385	2.92	7.4	.28				2
		2.12		3230	2.47	16.4	3.36				
		2.93		6400	3.44	18.9	2.79				
		4.54		14,085	4.65	16.3	1.57				
		9.50		10,440	3.94	9.7	.46				
		6.29		6540	2.51	5.6	.39				

TABLE C–1
(continued)

STREAMS AND RIVERS

River Name	Category	Depth (ft.)	Area (ft^2)	Flow (cfs)	Velocity (fps)	K_L (ft/day)	K_a @20°C (1/day)	K_d @20°C (1/day)	Φ (K_a/K_d) (Dimension-less)	K_a/U (1/mi)	Reference
Holston R.	Medium	7.52		10,500	3.15	4.7	.27				2
		7.07		10,500	3.30	3.0	.55				
		5.44		5590	3.11	6.8	.54				
		8.06		11,930	4.28	11.2	.60				
		3.98		952	2.73	11.2	1.25				
Fr. Broad	Medium	9.38		12,010	2.41	5.9	.27				2
		10.19		17,120	3.06	5.3	.23				
		3.29		4,105	2.40	14.2	1.88				
		4.74		8775	3.46	9.2	.84				
		5.72		12,455	4.02	8.1	.88				
		6.98		17,270	4.52	14.7	.91				
		4.29		4150	1.85	9.3	1.00				
		6.01		8775	2.75	1.6	.55				
		7.16		12,455	3.23	14.5	.98				
		9.49		17,270	3.71	4.9	.25				
Wautaga R.	Medium	3.42		3112	5.0	43.8	5.6				2
Hiwassee	Medium	3.02		1145	3.05	11.9	1.7				2
		2.83		1145	3.91	21.0	3.2				
Ohio River	Deep	32	43,000	6000	.14		.06	0.25	.24	.109	4,46, 47

TABLE C-1
(continued)

STREAMS AND RIVERS

River Name	Category	Depth (ft.)	Area (ft^2)	Flow (cfs)	Velocity (fps)	K_L (ft/day)	K_a @20°C (1/day)	K_d @20°C (1/day)	ϕ (K_a/K_d) (Dimension-less)	K_a/U (1/mi)	Reference
Upper Hudson (Troy, N.Y. - Saugerties)	Deep	17.5	6,000	3000	0.5		.34	.125	2.7	.0153	30
		21.0	6,750	4500	1.5		.34	.165	2.1	.0067	
Lower Sacramento R	Deep	15-20	8,000	10,000	1.5		.28	.40	.70	.017	33
							.15		.37		
							.24		.60		
Upper James River (Va.)	Deep	15.5	11,500	1800	0.16		.15	.48	0.31	.184	38,39
			14,500	2600	0.18		.14	.30	0.47	.102	
			14,500	9000	0.63		.12	.31	0.39	.045	
			14,000	7500	0.53		.13	.41	0.32	.047	
			14,500	4500	0.31		.22	.39	0.56	.077	
			13,500	3800	0.28		.24	.38	0.63	.083	
			15,000	1350	0.13		.22	.43	0.51	.202	
Illinois R.	Deep	10-12		8000				.07			2
		9.2			1.37	6	.225				
		9.0			1.57	7	.269				
		8.9			1.63	7	.224				

TABLE C-2

TIDAL RIVERS AND ESTUARIES

Name	Depth (ft.)	Area $(ft)^2$	Net non-Tidal Flow (cfs)	Advective Vel. (fps)	Dispersion Coef. $(\frac{mi^2}{day})$	K_d @20°C $(\frac{1}{day})$	K_a @20°C $(\frac{1}{day})$	Φ (K_a/K_d) (Dimension less)	K_dE $(\frac{mi^2}{day^2})$	K_dE/U^2 (Dimensionless)	Reference
Delaware R. Estuary	25	20,000 75,000 150,000 270,000	2,500	.12 .033 .016 .009	5	0.3	0.17	0.97	1.5	0.39 5.1 2.2 69.	20,22 35
New York Harbor											
Hudson River	35	135,000	5,000	0.037	20	0.25	0.09	0.36	5	13.3	23-27
East River	40	80,000	0	0	10	0.23	0.08	0.35	2.3		32
Cooper River (S. Carolina)	40	40,000	10,000	0.25	30 ↓ 50	0.30	0.08	0.27	9 15	0.54 0.90	43
Savannah River (Ga., S.C.)	10 ↓ 28	10,000 ↓ 40,000	7,000	0.7 0.17	10 ↓ 20	0.30	0.1 ↓ 0.65	0.33 2. 2	3. 6.	0.02 0.78	14

TABLE C-2
(continued)

TIDAL RIVERS AND ESTUARIES

Name	Depth (ft.)	Area $(ft)^2$	Net non-Tidal Flow (cfs)	Advective Vel. (fps)	Dispersion Coef. $(\frac{mi^2}{day})$	K_d @20°C $(\frac{1}{day})$	K_a @20°C $(\frac{1}{day})$	Φ (K_a/K_d) (Dimension less)	K_dE $(\frac{mi^2}{day^2})$	K_dE/U^2 (Dimension-less)	Reference
Lower Raritan River (N.J.)	14 17.5	3,200 5,000	150	.047 .029	5	0.20	0.32 0.18	1.6 0.9	1. 1.	1.7 4.5	17,18
South River (N.J.)	12	2,500	23	0.01	5	0.20	0.40	2.0	1.	37.4	24
Houston Ship Channel (Tex)	25	17,500 25,000	900 2,600	.05 .10	27 40	0.25	0.10	0.40	6.8 10.	9.9 3.7	34
Cape Fear R. Estuary (N.C.)	9.7 20 13.0	2,100 9,700 33,000	1,000	.48 .10 .03	2 ↓ 10	0.23	0.3 0.1 0.37	1.3 0.44 1.6	.46 2.3	.007 .17 9.6	19
Potomac River	10 ↓ 25	100,000 ↓ 1,700,000	550	.006 .0003	1.0 ↓ 10	0.47	0.38	0.79	.47 ↓ 4.7	4.9 >1,000	36
Compton Creek (New Jersey)	14.5 10.5	1,000 790	10	.010 .013	1.0	0.23	0.3 0.48	0.77 0.48	.23	8.6 5.1	25

TABLE C-2
(continued)

TIDAL RIVERS AND ESTUARIES

Name	Depth (ft.)	Area $(ft)^2$	Net non-Tidal Flow (cfs)	Advective Vel. (fps)	Dispersion Coef. $(\frac{mi^2}{day})$	K_d @20°C $(\frac{1}{day})$	K_a @20°C $(\frac{1}{day})$	Φ (K_a/K_d) (Dimension less)	K_dE $(\frac{mi^2}{day^2})$	K_dE/U^2 (Dimension-less)	Reference
Wappinger & Fishkill Cks. (New York)	9	500	2	.004	0.5	0.30	0.25	0.83	0.15	35	
	4	2,000		.001	1.0				0.30	112	
River Foyle Estuary (N. Ireland)	10	10,000	250	.025	5.	0.30	0.29	0.97	1.5	9.0	
	15	15,000		.017	5.		0.22	0.73	1.5	19.4	
	25	20,000		.013	5.		0.10	0.33	1.5	33.2	
	5	40,000		.006	5.		1.16	3.8	1.5	156.	

Note: Other tidal rivers and estuaries used; Rhine, Waccosassa, Rhine Uaguina, Elms.

APPENDIX D

TABLES OF DO DEFICIT RESPONSE FOR TIDAL RIVERS AND ESTUARIES

MULTIPLE SOURCE - TIDAL RIVERS AND ESTUARIES
MG/L DO DEFICIT PER MG/L ULT. BOD INPUT

$\Phi = 0.10$

K_dE/U^2

K_dX/U	0.01	0.05	0.10	0.20	0.30	0.40
40.0	0.021	0.023	0.025	0.028	0.032	0.035
38.0	0.025	0.027	0.030	0.035	0.039	0.043
36.0	0.031	0.034	0.036	0.042	0.047	0.052
34.0	0.038	0.041	0.044	0.051	0.057	0.063
32.0	0.046	0.050	0.054	0.062	0.070	0.076
30.0	0.056	0.061	0.066	0.076	0.084	0.093
28.0	0.069	0.074	0.081	0.092	0.103	0.112
26.0	0.084	0.091	0.098	0.112	0.125	0.136
24.0	0.103	0.111	0.120	0.136	0.151	0.165
22.0	0.126	0.135	0.146	0.166	0.184	0.200
20.0	0.153	0.165	0.178	0.202	0.223	0.242
18.0	0.187	0.201	0.217	0.245	0.271	0.294
16.0	0.229	0.245	0.264	0.299	0.329	0.356
14.0	0.279	0.299	0.322	0.363	0.400	0.432
12.0	0.341	0.365	0.393	0.442	0.485	0.524
10.0	0.416	0.445	0.479	0.538	0.589	0.635
8.0	0.508	0.543	0.583	0.653	0.714	0.768
6.0	0.618	0.660	0.707	0.790	0.860	0.922
4.0	0.737	0.785	0.839	0.932	1.011	1.080
2.0	0.773	0.823	0.880	0.978	1.060	1.132
0.0	0.020	0.094	0.178	0.323	0.446	0.552
-0.1	0.000	0.025	0.097	0.243	0.370	0.482
-0.2	0.000	0.005	0.046	0.173	0.299	0.411
-0.3	0.000	0.001	0.020	0.119	0.235	0.346
-0.4	0.000	0.000	0.009	0.080	0.183	0.288
-0.5	0.000	0.000	0.004	0.053	0.140	0.237
-0.6	0.000	0.000	0.001	0.034	0.106	0.194
-0.7	0.000	0.000	0.001	0.022	0.080	0.157
-0.8	0.000	0.000	0.000	0.014	0.060	0.127
-0.9	0.000	0.000	0.000	0.009	0.044	0.102
-1.0	0.000	0.000	0.000	0.006	0.033	0.082
-1.1	0.000	0.000	0.000	0.003	0.024	0.065
-1.2	0.000	0.000	0.000	0.002	0.018	0.052
-1.3	0.000	0.000	0.000	0.001	0.013	0.041
-1.4	0.000	0.000	0.000	0.001	0.009	0.032
-1.5	0.000	0.000	0.000	0.001	0.007	0.026

MULTIPLE SOURCE - TIDAL RIVERS AND ESTUARIES
MG/L DO DEFICIT PER MG/L ULT. BOD INPUT

$\Phi = 0.10$

K_dE/U^2

K_dX/U	0.50	0.60	0.80	1.00	2.00	3.00
50.0	0.015	0.016	0.019	0.022	0.035	0.048
48.0	0.018	0.020	0.023	0.026	0.041	0.057
46.0	0.022	0.024	0.027	0.031	0.049	0.066
44.0	0.026	0.029	0.033	0.037	0.058	0.078
42.0	0.032	0.035	0.040	0.045	0.069	0.092
40.0	0.039	0.042	0.048	0.054	0.082	0.108
38.0	0.047	0.050	0.058	0.065	0.097	0.127
36.0	0.057	0.061	0.069	0.078	0.115	0.149
34.0	0.068	0.074	0.084	0.093	0.136	0.175
32.0	0.083	0.089	0.101	0.112	0.162	0.205
30.0	0.100	0.108	0.121	0.134	0.192	0.241
28.0	0.121	0.130	0.146	0.162	0.227	0.283
26.0	0.147	0.157	0.176	0.194	0.270	0.333
24.0	0.178	0.190	0.212	0.233	0.320	0.391
22.0	0.215	0.229	0.256	0.280	0.379	0.459
20.0	0.260	0.277	0.308	0.336	0.450	0.539
18.0	0.315	0.335	0.371	0.404	0.534	0.633
16.0	0.382	0.405	0.447	0.485	0.633	0.743
14.0	0.462	0.489	0.538	0.582	0.751	0.872
12.0	0.559	0.591	0.648	0.699	0.889	1.021
10.0	0.676	0.713	0.780	0.838	1.050	1.193
8.0	0.816	0.859	0.935	1.001	1.234	1.384
6.0	0.977	1.026	1.112	1.185	1.433	1.584
4.0	1.140	1.193	1.285	1.362	1.615	1.761
2.0	1.195	1.250	1.346	1.425	1.686	1.833
0.0	0.646	0.729	0.871	0.989	1.373	1.590
-1.0	0.144	0.212	0.352	0.485	0.975	1.270
-2.0	0.022	0.045	0.110	0.193	0.620	0.941
-3.0	0.003	0.008	0.032	0.071	0.374	0.669
-4.0	0.000	0.002	0.009	0.025	0.219	0.464
-5.0	0.000	0.000	0.002	0.009	0.126	0.317
-6.0	0.000	0.000	0.001	0.003	0.071	0.214
-7.0	0.000	0.000	0.000	0.001	0.040	0.144
-8.0	0.000	0.000	0.000	0.000	0.023	0.096
-9.0	0.000	0.000	0.000	0.000	0.013	0.064
-10.0	0.000	0.000	0.000	0.000	0.007	0.043

MULTIPLE SOURCE - TIDAL RIVERS AND ESTUARIES
MG/L DO DEFICIT PER MG/L ULT. BOD INPUT

Φ = 0.10

K_dE/U^2

K_dX/U	4.00	5.00	6.00	7.00	8.00	10.00
69.0	0.014	0.019	0.024	0.029	0.034	0.045
66.0	0.018	0.023	0.029	0.035	0.041	0.054
63.0	0.023	0.029	0.036	0.043	0.050	0.065
60.0	0.029	0.036	0.044	0.052	0.061	0.078
57.0	0.036	0.045	0.055	0.064	0.074	0.094
54.0	0.046	0.056	0.068	0.079	0.090	0.113
51.0	0.057	0.070	0.083	0.097	0.110	0.136
48.0	0.072	0.088	0.103	0.118	0.134	0.164
45.0	0.091	0.109	0.127	0.145	0.163	0.197
42.0	0.114	0.136	0.157	0.178	0.198	0.237
39.0	0.143	0.169	0.194	0.218	0.241	0.286
36.0	0.181	0.211	0.240	0.267	0.294	0.344
33.0	0.227	0.263	0.296	0.327	0.358	0.414
30.0	0.286	0.327	0.365	0.401	0.435	0.498
27.0	0.360	0.407	0.451	0.492	0.530	0.599
24.0	0.452	0.507	0.557	0.602	0.644	0.720
21.0	0.569	0.631	0.687	0.737	0.783	0.865
18.0	0.715	0.785	0.847	0.902	0.951	1.037
15.0	0.898	0.975	1.042	1.100	1.151	1.240
12.0	1.123	1.206	1.275	1.334	1.386	1.472
9.0	1.393	1.477	1.545	1.601	1.649	1.727
6.0	1.688	1.765	1.825	1.874	1.914	1.976
3.0	1.914	1.981	2.031	2.070	2.102	2.149
0.0	1.730	1.829	1.902	1.958	2.003	2.071
-2.0	1.170	1.338	1.467	1.568	1.649	1.772
-4.0	0.684	0.866	1.017	1.142	1.247	1.413
-6.0	0.377	0.532	0.672	0.795	0.904	1.085
-8.0	0.202	0.318	0.432	0.540	0.640	0.814
-10.0	0.107	0.187	0.274	0.362	0.447	0.603
-12.0	0.056	0.109	0.173	0.240	0.309	0.443
-14.0	0.029	0.064	0.108	0.159	0.213	0.324
-16.0	0.015	0.037	0.068	0.105	0.146	0.236
-18.0	0.008	0.021	0.042	0.069	0.100	0.171
-20.0	0.004	0.012	0.026	0.045	0.069	0.124
-22.0	0.002	0.007	0.016	0.030	0.047	0.090
-24.0	0.001	0.004	0.010	0.020	0.032	0.065
-26.0	0.001	0.002	0.006	0.013	0.022	0.047

MULTIPLE SOURCE - TIDAL RIVERS AND ESTUARIES
MG/L DO DEFICIT PER MG/L ULT. BOD INPUT

$\Phi = 0.10$ K_dE/U^2

K_dX/U	20.00	30.00	40.00	50.00	70.00	100.00
160.0	0.001	0.003	0.007	0.011	0.023	0.046
152.0	0.002	0.005	0.009	0.015	0.030	0.057
144.0	0.002	0.007	0.012	0.020	0.038	0.071
136.0	0.004	0.009	0.017	0.026	0.049	0.088
128.0	0.006	0.013	0.023	0.035	0.063	0.109
120.0	0.008	0.018	0.032	0.047	0.081	0.136
112.0	0.012	0.026	0.043	0.062	0.104	0.169
104.0	0.018	0.037	0.059	0.083	0.133	0.209
96.0	0.027	0.052	0.081	0.110	0.171	0.260
88.0	0.041	0.074	0.110	0.147	0.220	0.322
80.0	0.061	0.105	0.151	0.196	0.282	0.400
72.0	0.091	0.149	0.206	0.261	0.362	0.496
64.0	0.136	0.210	0.281	0.347	0.465	0.614
56.0	0.203	0.298	0.384	0.462	0.597	0.760
48.0	0.302	0.421	0.524	0.614	0.764	0.939
40.0	0.451	0.595	0.714	0.814	0.976	1.155
32.0	0.671	0.839	0.970	1.076	1.239	1.411
24.0	0.995	1.175	1.306	1.408	1.556	1.704
16.0	1.452	1.615	1.724	1.803	1.912	2.013
8.0	2.010	2.102	2.157	2.194	2.241	2.280
0.0	2.222	2.279	2.308	2.326	2.348	2.364
-8.0	1.347	1.610	1.766	1.869	1.999	2.105
-16.0	0.653	0.947	1.155	1.309	1.521	1.715
-24.0	0.300	0.528	0.717	0.871	1.105	1.340
-32.0	0.136	0.289	0.436	0.567	0.784	1.025
-40.0	0.061	0.157	0.263	0.366	0.551	0.774
-48.0	0.027	0.085	0.158	0.235	0.385	0.581
-56.0	0.012	0.046	0.095	0.151	0.268	0.434
-64.0	0.006	0.025	0.057	0.096	0.186	0.324
-72.0	0.002	0.013	0.034	0.062	0.130	0.241
-80.0	0.001	0.007	0.020	0.039	0.090	0.180
-88.0	0.001	0.004	0.012	0.025	0.062	0.134
-96.0	0.000	0.002	0.007	0.016	0.043	0.099
-104.0	0.000	0.001	0.004	0.010	0.030	0.074
-112.0	0.000	0.001	0.003	0.007	0.021	0.055
-120.0	0.000	0.000	0.002	0.004	0.015	0.041

MULTIPLE SOURCE - TIDAL RIVERS AND ESTUARIES
MG/L DO DEFICIT PER MG/L ULT. BOD INPUT

$\Phi = 0.20$

K_dE/U^2

K_dX/U	0.01	0.05	0.10	0.20	0.30	0.40
20.0	0.023	0.026	0.028	0.033	0.038	0.042
19.0	0.029	0.031	0.034	0.040	0.046	0.051
18.0	0.035	0.038	0.042	0.049	0.055	0.062
17.0	0.043	0.046	0.051	0.059	0.067	0.074
16.0	0.052	0.056	0.062	0.071	0.081	0.089
15.0	0.064	0.069	0.075	0.087	0.097	0.108
14.0	0.078	0.084	0.091	0.105	0.118	0.129
13.0	0.095	0.102	0.111	0.127	0.142	0.156
12.0	0.116	0.125	0.135	0.154	0.172	0.188
11.0	0.141	0.152	0.164	0.187	0.207	0.226
10.0	0.172	0.185	0.200	0.227	0.250	0.272
9.0	0.210	0.226	0.243	0.275	0.302	0.327
8.0	0.257	0.275	0.295	0.332	0.364	0.393
7.0	0.313	0.334	0.358	0.401	0.438	0.471
6.0	0.380	0.405	0.434	0.483	0.525	0.562
5.0	0.459	0.488	0.521	0.577	0.624	0.664
4.0	0.548	0.581	0.617	0.680	0.731	0.775
3.0	0.634	0.670	0.710	0.777	0.832	0.878
2.0	0.679	0.718	0.761	0.833	0.891	0.939
1.0	0.576	0.620	0.670	0.752	0.819	0.875
0.0	0.020	0.093	0.173	0.307	0.415	0.504
-0.1	0.000	0.024	0.094	0.230	0.344	0.439
-0.2	0.000	0.005	0.044	0.164	0.277	0.375
-0.3	0.000	0.001	0.020	0.112	0.218	0.314
-0.4	0.000	0.000	0.008	0.075	0.168	0.261
-0.5	0.000	0.000	0.003	0.049	0.128	0.214
-0.6	0.000	0.000	0.001	0.032	0.097	0.174
-0.7	0.000	0.000	0.001	0.020	0.072	0.141
-0.8	0.000	0.000	0.000	0.013	0.054	0.113
-0.9	0.000	0.000	0.000	0.008	0.040	0.090
-1.0	0.000	0.000	0.000	0.005	0.029	0.072
-1.1	0.000	0.000	0.000	0.003	0.021	0.057
-1.2	0.000	0.000	0.000	0.002	0.016	0.045
-1.3	0.000	0.000	0.000	0.001	0.011	0.035
-1.4	0.000	0.000	0.000	0.001	0.008	0.028
-1.5	0.000	0.000	0.000	0.000	0.006	0.022

MULTIPLE SOURCE - TIDAL RIVERS AND ESTUARIES
MG/L DO DEFICIT PER MG/L ULT. BOD INPUT

$\Phi = 0.20$

$K_d E/U^2$

$K_d X/U$	0.50	0.60	0.80	1.00	2.00	3.00
28.0	0.011	0.012	0.015	0.017	0.032	0.048
27.0	0.013	0.015	0.018	0.021	0.037	0.055
26.0	0.016	0.017	0.021	0.025	0.043	0.063
25.0	0.019	0.021	0.025	0.029	0.051	0.073
24.0	0.023	0.025	0.030	0.035	0.059	0.084
23.0	0.027	0.030	0.035	0.041	0.069	0.096
22.0	0.032	0.036	0.042	0.049	0.080	0.111
21.0	0.039	0.043	0.050	0.058	0.093	0.127
20.0	0.047	0.051	0.060	0.068	0.109	0.146
19.0	0.056	0.061	0.071	0.081	0.127	0.169
18.0	0.068	0.074	0.085	0.096	0.148	0.194
17.0	0.081	0.088	0.101	0.114	0.172	0.223
16.0	0.098	0.106	0.121	0.135	0.200	0.256
15.0	0.117	0.126	0.144	0.161	0.233	0.295
14.0	0.141	0.151	0.172	0.190	0.272	0.338
13.0	0.169	0.181	0.204	0.226	0.316	0.388
12.0	0.203	0.217	0.243	0.267	0.367	0.445
11.0	0.243	0.260	0.290	0.317	0.426	0.510
10.0	0.292	0.311	0.344	0.375	0.495	0.583
9.0	0.350	0.371	0.409	0.443	0.572	0.664
8.0	0.419	0.443	0.485	0.522	0.660	0.755
7.0	0.500	0.527	0.573	0.614	0.759	0.853
6.0	0.594	0.623	0.674	0.717	0.866	0.959
5.0	0.700	0.731	0.785	0.830	0.979	1.067
4.0	0.812	0.845	0.901	0.946	1.091	1.172
3.0	0.917	0.951	1.007	1.052	1.190	1.263
2.0	0.979	1.014	1.072	1.117	1.252	1.320
1.0	0.922	0.963	1.030	1.082	1.237	1.314
0.0	0.580	0.645	0.750	0.833	1.076	1.194
-1.0	0.125	0.182	0.295	0.398	0.750	0.941
-2.0	0.018	0.036	0.088	0.151	0.461	0.678
-3.0	0.002	0.006	0.024	0.052	0.266	0.465
-4.0	0.000	0.001	0.006	0.017	0.148	0.309
-5.0	0.000	0.000	0.002	0.006	0.080	0.202
-6.0	0.000	0.000	0.000	0.002	0.043	0.130
-7.0	0.000	0.000	0.000	0.001	0.023	0.083
-8.0	0.000	0.000	0.000	0.000	0.012	0.052

MULTIPLE SOURCE - TIDAL RIVERS AND ESTUARIES
MG/L DO DEFICIT PER MG/L ULT. BOD INPUT

$\Phi = 0.20$ $K_d E/U^2$

$K_d X/U$

	4.00	5.00	6.00	7.00	8.00	10.00
46.0	0.006	0.009	0.012	0.015	0.019	0.027
44.0	0.008	0.011	0.015	0.019	0.023	0.033
42.0	0.010	0.014	0.019	0.024	0.029	0.040
40.0	0.013	0.018	0.024	0.030	0.036	0.049
38.0	0.017	0.023	0.030	0.037	0.044	0.060
36.0	0.022	0.030	0.038	0.046	0.055	0.073
34.0	0.029	0.038	0.048	0.058	0.068	0.089
32.0	0.038	0.049	0.061	0.073	0.084	0.109
30.0	0.049	0.063	0.077	0.091	0.105	0.132
28.0	0.064	0.080	0.097	0.113	0.130	0.162
26.0	0.083	0.103	0.123	0.142	0.161	0.197
24.0	0.108	0.132	0.155	0.177	0.199	0.240
22.0	0.140	0.168	0.195	0.221	0.246	0.292
20.0	0.182	0.215	0.247	0.276	0.304	0.355
18.0	0.236	0.275	0.311	0.344	0.375	0.431
16.0	0.306	0.350	0.391	0.428	0.462	0.522
14.0	0.396	0.446	0.490	0.530	0.566	0.629
12.0	0.510	0.564	0.612	0.654	0.691	0.755
10.0	0.652	0.709	0.758	0.800	0.836	0.898
8.0	0.826	0.882	0.928	0.967	1.000	1.055
6.0	1.025	1.075	1.114	1.147	1.174	1.217
4.0	1.226	1.264	1.293	1.316	1.335	1.364
2.0	1.362	1.390	1.410	1.426	1.438	1.456
0.0	1.265	1.312	1.345	1.370	1.390	1.418
-2.0	0.826	0.932	1.011	1.072	1.120	1.192
-4.0	0.451	0.568	0.664	0.743	0.810	0.914
-6.0	0.229	0.324	0.410	0.487	0.554	0.668
-8.0	0.112	0.178	0.245	0.308	0.368	0.474
-10.0	0.054	0.096	0.143	0.192	0.240	0.330
-12.0	0.025	0.051	0.083	0.118	0.154	0.227
-14.0	0.012	0.027	0.048	0.072	0.098	0.155
-16.0	0.006	0.014	0.027	0.044	0.062	0.105
-18.0	0.003	0.008	0.015	0.026	0.040	0.071
-20.0	0.001	0.004	0.009	0.016	0.025	0.048
-22.0	0.001	0.002	0.005	0.010	0.016	0.032
-24.0	0.000	0.001	0.003	0.006	0.010	0.022

MULTIPLE SOURCE - TIDAL RIVERS AND ESTUARIES
MG/L DO DEFICIT PER MG/L ULT. BOD INPUT

$\Phi = 0.20$

$K_d E/U^2$

$K_d X/U$	20.00	30.00	40.00	50.00	70.00	100.00
100.0	0.001	0.003	0.007	0.012	0.026	0.051
95.0	0.002	0.005	0.010	0.016	0.033	0.062
90.0	0.002	0.007	0.013	0.021	0.041	0.076
85.0	0.004	0.010	0.018	0.028	0.052	0.092
80.0	0.005	0.013	0.024	0.037	0.066	0.113
75.0	0.008	0.019	0.032	0.048	0.083	0.137
70.0	0.012	0.026	0.043	0.063	0.104	0.168
65.0	0.017	0.036	0.058	0.082	0.132	0.204
60.0	0.025	0.050	0.078	0.108	0.166	0.248
55.0	0.037	0.070	0.105	0.141	0.209	0.301
50.0	0.055	0.098	0.141	0.184	0.263	0.366
45.0	0.081	0.136	0.190	0.240	0.330	0.442
40.0	0.120	0.189	0.254	0.312	0.413	0.534
35.0	0.176	0.263	0.339	0.405	0.515	0.641
30.0	0.259	0.364	0.450	0.523	0.639	0.766
25.0	0.379	0.500	0.594	0.671	0.787	0.907
20.0	0.550	0.680	0.776	0.850	0.957	1.063
15.0	0.784	0.909	0.995	1.057	1.145	1.226
10.0	1.081	1.176	1.236	1.278	1.333	1.382
5.0	1.387	1.427	1.450	1.466	1.485	1.500
0.0	1.479	1.500	1.511	1.518	1.525	1.531
-5.0	1.080	1.208	1.280	1.326	1.382	1.427
-10.0	0.655	0.842	0.962	1.046	1.155	1.250
-15.0	0.370	0.551	0.684	0.783	0.924	1.055
-20.0	0.202	0.349	0.471	0.570	0.719	0.870
-25.0	0.109	0.217	0.318	0.407	0.551	0.707
-30.0	0.058	0.134	0.213	0.287	0.416	0.567
-35.0	0.031	0.082	0.141	0.201	0.313	0.452
-40.0	0.016	0.050	0.093	0.140	0.233	0.358
-45.0	0.009	0.030	0.062	0.098	0.174	0.282
-50.0	0.005	0.018	0.041	0.068	0.129	0.222
-55.0	0.002	0.011	0.027	0.047	0.095	0.174
-60.0	0.001	0.007	0.018	0.032	0.071	0.136
-65.0	0.001	0.004	0.011	0.022	0.052	0.106
-70.0	0.000	0.003	0.008	0.016	0.038	0.083
-75.0	0.000	0.002	0.005	0.011	0.028	0.065
-80.0	0.000	0.001	0.003	0.007	0.021	0.051

MULTIPLE SOURCE - TIDAL RIVERS AND ESTUARIES
MG/L DO DEFICIT PER MG/L ULT. BOD INPUT

$\Phi = 0.30$ K_dE/U^2

K_dX/U

	0.01	0.05	0.10	0.20	0.30	0.40
15.0	0.016	0.018	0.020	0.024	0.028	0.033
14.5	0.019	0.021	0.023	0.028	0.033	0.037
14.0	0.022	0.024	0.027	0.032	0.038	0.043
13.5	0.026	0.028	0.031	0.037	0.043	0.049
13.0	0.030	0.033	0.036	0.043	0.050	0.056
12.5	0.034	0.038	0.042	0.049	0.057	0.064
12.0	0.040	0.044	0.048	0.057	0.065	0.073
11.5	0.046	0.051	0.056	0.066	0.075	0.084
11.0	0.054	0.059	0.065	0.076	0.086	0.096
10.5	0.063	0.068	0.075	0.087	0.099	0.110
10.0	0.073	0.079	0.086	0.100	0.113	0.126
9.5	0.084	0.091	0.100	0.116	0.130	0.144
9.0	0.098	0.106	0.115	0.133	0.149	0.164
8.5	0.114	0.123	0.133	0.153	0.171	0.188
8.0	0.132	0.142	0.154	0.176	0.196	0.214
7.5	0.153	0.164	0.178	0.202	0.224	0.244
7.0	0.177	0.190	0.205	0.232	0.256	0.278
6.5	0.205	0.220	0.236	0.266	0.293	0.316
6.0	0.237	0.253	0.272	0.305	0.333	0.359
5.5	0.273	0.292	0.312	0.348	0.379	0.406
5.0	0.314	0.335	0.357	0.396	0.429	0.458
4.5	0.360	0.382	0.407	0.449	0.484	0.514
4.0	0.411	0.434	0.461	0.506	0.543	0.574
3.5	0.464	0.490	0.518	0.565	0.604	0.636
3.0	0.517	0.545	0.575	0.624	0.664	0.697
2.5	0.565	0.594	0.626	0.678	0.718	0.752
2.0	0.599	0.630	0.663	0.717	0.759	0.793
1.5	0.601	0.634	0.670	0.728	0.772	0.809
1.0	0.543	0.581	0.622	0.688	0.739	0.780
0.5	0.376	0.425	0.477	0.561	0.627	0.680
0.0	0.020	0.091	0.169	0.293	0.388	0.465
-0.2	0.000	0.005	0.043	0.155	0.258	0.344
-0.4	0.000	0.000	0.008	0.071	0.156	0.238
-0.6	0.000	0.000	0.001	0.030	0.089	0.158
-0.8	0.000	0.000	0.000	0.012	0.049	0.102
-1.0	0.000	0.000	0.000	0.005	0.026	0.064
-1.2	0.000	0.000	0.000	0.002	0.014	0.040

MULTIPLE SOURCE - TIDAL RIVERS AND ESTUARIES
MG/L DO DEFICIT PER MG/L ULT. BOD INPUT

$\Phi = 0.30$

$K_d E/U^2$

$K_d X/U$	0.50	0.60	0.80	1.00	2.00	3.00
20.0	0.010	0.011	0.014	0.017	0.034	0.053
19.0	0.013	0.014	0.018	0.022	0.042	0.064
18.0	0.017	0.019	0.023	0.028	0.052	0.077
17.0	0.022	0.024	0.030	0.035	0.064	0.093
16.0	0.028	0.032	0.038	0.045	0.079	0.112
15.0	0.037	0.041	0.049	0.057	0.097	0.135
14.0	0.048	0.053	0.063	0.073	0.120	0.163
13.0	0.062	0.069	0.081	0.093	0.148	0.196
12.0	0.081	0.089	0.104	0.118	0.181	0.236
11.0	0.106	0.115	0.133	0.149	0.222	0.282
10.0	0.137	0.149	0.170	0.189	0.272	0.338
9.0	0.178	0.192	0.217	0.239	0.332	0.403
8.0	0.231	0.247	0.275	0.301	0.404	0.478
7.0	0.298	0.316	0.349	0.378	0.488	0.563
6.0	0.381	0.402	0.439	0.470	0.584	0.659
5.0	0.483	0.506	0.545	0.578	0.692	0.762
4.0	0.602	0.625	0.666	0.699	0.806	0.868
3.0	0.725	0.749	0.788	0.820	0.915	0.967
2.0	0.821	0.845	0.884	0.913	0.999	1.040
1.0	0.814	0.842	0.887	0.921	1.016	1.059
0.0	0.528	0.580	0.663	0.725	0.896	0.973
-0.5	0.266	0.330	0.437	0.521	0.762	0.875
-1.0	0.110	0.159	0.254	0.339	0.616	0.759
-1.5	0.042	0.071	0.139	0.208	0.481	0.642
-2.0	0.015	0.030	0.073	0.124	0.367	0.534
-2.5	0.005	0.012	0.037	0.072	0.276	0.438
-3.0	0.002	0.005	0.019	0.041	0.204	0.356
-3.5	0.001	0.002	0.009	0.023	0.150	0.286
-4.0	0.000	0.001	0.004	0.013	0.109	0.229
-4.5	0.000	0.000	0.002	0.007	0.079	0.182
-5.0	0.000	0.000	0.001	0.004	0.057	0.144
-5.5	0.000	0.000	0.001	0.002	0.041	0.113
-6.0	0.000	0.000	0.000	0.001	0.029	0.089
-6.5	0.000	0.000	0.000	0.001	0.021	0.070
-7.0	0.000	0.000	0.000	0.000	0.015	0.055
-7.5	0.000	0.000	0.000	0.000	0.010	0.043
-8.0	0.000	0.000	0.000	0.000	0.007	0.033

MULTIPLE SOURCE - TIDAL RIVERS AND ESTUARIES
MG/L DO DEFICIT PER MG/L ULT. BOD INPUT

$\Phi = 0.30$

$K_d E/U^2$

$K_d X/U$	4.00	5.00	6.00	7.00	8.00	10.00
30.0	0.012	0.018	0.024	0.030	0.037	0.050
28.0	0.018	0.025	0.032	0.040	0.048	0.065
26.0	0.025	0.034	0.044	0.054	0.064	0.084
24.0	0.036	0.047	0.060	0.072	0.084	0.109
22.0	0.051	0.066	0.081	0.096	0.111	0.141
20.0	0.072	0.091	0.110	0.128	0.146	0.181
18.0	0.102	0.126	0.149	0.171	0.192	0.232
16.0	0.144	0.173	0.201	0.227	0.252	0.297
14.0	0.202	0.238	0.270	0.300	0.328	0.377
12.0	0.283	0.324	0.361	0.394	0.424	0.476
10.0	0.392	0.438	0.477	0.511	0.542	0.594
8.0	0.535	0.582	0.621	0.654	0.683	0.730
6.0	0.713	0.755	0.789	0.817	0.841	0.879
4.0	0.910	0.940	0.964	0.982	0.998	1.022
2.0	1.066	1.083	1.095	1.104	1.112	1.123
0.0	1.017	1.046	1.066	1.081	1.092	1.108
-1.0	0.844	0.901	0.941	0.971	0.995	1.029
-2.0	0.646	0.726	0.785	0.830	0.866	0.919
-3.0	0.472	0.561	0.631	0.686	0.732	0.801
-4.0	0.335	0.422	0.495	0.555	0.605	0.685
-5.0	0.232	0.312	0.381	0.441	0.493	0.578
-6.0	0.159	0.227	0.290	0.347	0.397	0.482
-7.0	0.108	0.164	0.219	0.270	0.317	0.399
-8.0	0.072	0.117	0.164	0.209	0.251	0.328
-9.0	0.048	0.084	0.122	0.160	0.198	0.268
-10.0	0.032	0.059	0.090	0.123	0.155	0.218
-11.0	0.021	0.042	0.066	0.093	0.121	0.177
-12.0	0.014	0.029	0.049	0.071	0.095	0.143
-13.0	0.009	0.021	0.036	0.054	0.073	0.116
-14.0	0.006	0.014	0.026	0.041	0.057	0.093
-15.0	0.004	0.010	0.019	0.031	0.044	0.075
-16.0	0.003	0.007	0.014	0.023	0.034	0.060
-17.0	0.002	0.005	0.010	0.017	0.026	0.048
-18.0	0.001	0.003	0.007	0.013	0.020	0.038
-19.0	0.001	0.002	0.005	0.010	0.016	0.031
-20.0	0.000	0.002	0.004	0.007	0.012	0.024

MULTIPLE SOURCE - TIDAL RIVERS AND ESTUARIES
MG/L DO DEFICIT PER MG/L ULT. BOD INPUT

$\Phi = 0.30$ $K_d E/U^2$

$K_d X/U$	20.00	30.00	40.00	50.00	70.00	100.00
100.0	0.000	0.001	0.001	0.003	0.007	0.017
95.0	0.000	0.001	0.002	0.004	0.010	0.022
90.0	0.000	0.001	0.003	0.006	0.013	0.029
85.0	0.001	0.002	0.004	0.008	0.018	0.037
80.0	0.001	0.003	0.006	0.011	0.024	0.047
75.0	0.001	0.004	0.009	0.016	0.031	0.060
70.0	0.002	0.007	0.014	0.022	0.042	0.077
65.0	0.004	0.010	0.020	0.031	0.056	0.098
60.0	0.006	0.016	0.029	0.043	0.075	0.125
55.0	0.010	0.024	0.041	0.060	0.100	0.159
50.0	0.017	0.037	0.060	0.084	0.133	0.201
45.0	0.028	0.056	0.087	0.117	0.176	0.254
40.0	0.047	0.085	0.125	0.163	0.232	0.320
35.0	0.076	0.129	0.179	0.225	0.305	0.401
30.0	0.124	0.194	0.255	0.309	0.398	0.498
25.0	0.201	0.289	0.360	0.420	0.513	0.613
20.0	0.322	0.424	0.501	0.562	0.652	0.744
15.0	0.503	0.608	0.681	0.736	0.813	0.886
10.0	0.753	0.838	0.892	0.930	0.981	1.026
5.0	1.034	1.071	1.092	1.106	1.123	1.138
0.0	1.143	1.155	1.161	1.165	1.169	1.172
-5.0	0.805	0.906	0.963	1.000	1.046	1.082
-10.0	0.456	0.600	0.695	0.761	0.850	0.928
-15.0	0.237	0.369	0.468	0.545	0.656	0.762
-20.0	0.118	0.218	0.304	0.377	0.490	0.609
-25.0	0.058	0.125	0.193	0.254	0.359	0.477
-30.0	0.028	0.071	0.120	0.169	0.259	0.369
-35.0	0.013	0.040	0.075	0.112	0.185	0.282
-40.0	0.006	0.023	0.046	0.073	0.131	0.215
-45.0	0.003	0.013	0.028	0.048	0.093	0.162
-50.0	0.001	0.007	0.017	0.031	0.065	0.122
-55.0	0.001	0.004	0.010	0.020	0.046	0.092
-60.0	0.000	0.002	0.006	0.013	0.032	0.068
-65.0	0.000	0.001	0.004	0.008	0.022	0.051
-70.0	0.000	0.001	0.002	0.005	0.015	0.038
-75.0	0.000	0.000	0.001	0.003	0.011	0.028

MULTIPLE SOURCE - TIDAL RIVERS AND ESTUARIES
MG/L DO DEFICIT PER MG/L ULT. BOD INPUT

$\Phi = 0.50$

$K_d E/U^2$

$K_d X/U$	0.01	0.05	0.10	0.20	0.30	0.40
10.0	0.014	0.016	0.018	0.023	0.028	0.033
9.5	0.018	0.020	0.023	0.029	0.034	0.040
9.0	0.023	0.025	0.029	0.036	0.043	0.049
8.5	0.029	0.032	0.037	0.045	0.053	0.061
8.0	0.037	0.041	0.046	0.056	0.065	0.075
7.5	0.047	0.052	0.058	0.070	0.081	0.091
7.0	0.060	0.066	0.073	0.087	0.100	0.112
6.5	0.076	0.083	0.092	0.108	0.122	0.136
6.0	0.097	0.105	0.115	0.133	0.150	0.165
5.5	0.122	0.132	0.144	0.164	0.183	0.200
5.0	0.154	0.165	0.178	0.202	0.222	0.241
4.5	0.192	0.205	0.220	0.246	0.268	0.288
4.0	0.238	0.253	0.269	0.297	0.321	0.342
3.5	0.291	0.308	0.326	0.356	0.381	0.402
3.0	0.351	0.369	0.388	0.420	0.445	0.466
2.5	0.414	0.433	0.453	0.485	0.511	0.531
2.0	0.471	0.491	0.512	0.545	0.570	0.591
1.5	0.505	0.527	0.550	0.585	0.612	0.632
1.0	0.485	0.512	0.540	0.583	0.614	0.638
0.5	0.356	0.396	0.437	0.499	0.544	0.578
0.0	0.020	0.089	0.160	0.268	0.345	0.404
-0.1	0.000	0.023	0.086	0.200	0.286	0.351
-0.2	0.000	0.004	0.040	0.141	0.228	0.298
-0.3	0.000	0.001	0.018	0.096	0.178	0.248
-0.4	0.000	0.000	0.007	0.063	0.136	0.204
-0.5	0.000	0.000	0.003	0.041	0.103	0.165
-0.6	0.000	0.000	0.001	0.026	0.077	0.133
-0.7	0.000	0.000	0.000	0.016	0.057	0.106
-0.8	0.000	0.000	0.000	0.010	0.042	0.084
-0.9	0.000	0.000	0.000	0.006	0.030	0.067
-1.0	0.000	0.000	0.000	0.004	0.022	0.052
-1.1	0.000	0.000	0.000	0.002	0.016	0.041
-1.2	0.000	0.000	0.000	0.001	0.011	0.032
-1.3	0.000	0.000	0.000	0.001	0.008	0.025
-1.4	0.000	0.000	0.000	0.001	0.006	0.019
-1.5	0.000	0.000	0.000	0.000	0.004	0.015

MULTIPLE SOURCE - TIDAL RIVERS AND ESTUARIES
MG/L DO DEFICIT PER MG/L ULT. BOD INPUT

$\Phi = 0.50$

$K_d E/U^2$

$K_d X/U$	0.50	0.60	0.80	1.00	2.00	3.00
15.0	0.005	0.006	0.008	0.010	0.025	0.042
14.5	0.006	0.007	0.010	0.013	0.029	0.047
14.0	0.007	0.009	0.012	0.015	0.034	0.054
13.5	0.009	0.011	0.014	0.018	0.039	0.061
13.0	0.011	0.013	0.017	0.022	0.045	0.070
12.5	0.014	0.016	0.021	0.026	0.053	0.080
12.0	0.017	0.019	0.025	0.031	0.061	0.090
11.5	0.020	0.024	0.030	0.037	0.070	0.103
11.0	0.025	0.029	0.036	0.044	0.081	0.117
10.5	0.031	0.035	0.044	0.052	0.094	0.132
10.0	0.038	0.043	0.052	0.062	0.109	0.149
9.5	0.046	0.052	0.063	0.074	0.125	0.169
9.0	0.056	0.063	0.076	0.088	0.144	0.191
8.5	0.068	0.076	0.091	0.105	0.166	0.215
8.0	0.083	0.092	0.108	0.124	0.190	0.242
7.5	0.101	0.111	0.129	0.146	0.217	0.271
7.0	0.123	0.134	0.154	0.173	0.248	0.304
6.5	0.149	0.161	0.183	0.203	0.282	0.339
6.0	0.179	0.193	0.217	0.238	0.321	0.378
5.5	0.215	0.230	0.255	0.278	0.363	0.419
5.0	0.257	0.273	0.300	0.323	0.408	0.464
4.5	0.306	0.322	0.350	0.373	0.457	0.510
4.0	0.360	0.377	0.405	0.428	0.509	0.558
3.5	0.420	0.437	0.464	0.487	0.562	0.606
3.0	0.485	0.500	0.527	0.548	0.616	0.653
2.5	0.549	0.564	0.588	0.607	0.666	0.698
2.0	0.607	0.621	0.643	0.661	0.711	0.736
1.5	0.649	0.663	0.684	0.700	0.743	0.764
1.0	0.657	0.672	0.696	0.713	0.757	0.776
0.5	0.604	0.626	0.658	0.682	0.742	0.767
0.0	0.449	0.486	0.542	0.582	0.683	0.726
-1.0	0.089	0.127	0.199	0.262	0.459	0.556
-2.0	0.011	0.022	0.053	0.089	0.261	0.378
-3.0	0.001	0.003	0.012	0.027	0.137	0.240
-4.0	0.000	0.000	0.003	0.008	0.069	0.147
-5.0	0.000	0.000	0.001	0.002	0.034	0.088
-6.0	0.000	0.000	0.000	0.001	0.016	0.051

MULTIPLE SOURCE - TIDAL RIVERS AND ESTUARIES
MG/L DO DEFICIT PER MG/L ULT. BOD INPUT

$\Phi = 0.50$

$K_d E/U^2$

$K_d X/U$

	4.00	5.00	6.00	7.00	8.00	10.00
22.0	0.011	0.016	0.022	0.028	0.035	0.049
21.0	0.014	0.020	0.027	0.035	0.042	0.058
20.0	0.018	0.025	0.034	0.042	0.051	0.069
19.0	0.023	0.032	0.041	0.051	0.061	0.081
18.0	0.029	0.040	0.051	0.062	0.073	0.096
17.0	0.037	0.049	0.062	0.075	0.088	0.113
16.0	0.046	0.061	0.076	0.091	0.105	0.133
15.0	0.059	0.076	0.093	0.110	0.126	0.156
14.0	0.075	0.095	0.114	0.132	0.150	0.182
13.0	0.094	0.117	0.139	0.159	0.178	0.213
12.0	0.118	0.144	0.168	0.190	0.211	0.247
11.0	0.148	0.177	0.203	0.227	0.249	0.287
10.0	0.185	0.217	0.245	0.270	0.293	0.332
9.0	0.230	0.264	0.293	0.319	0.342	0.382
8.0	0.284	0.319	0.349	0.375	0.398	0.436
7.0	0.348	0.383	0.413	0.438	0.460	0.496
6.0	0.421	0.455	0.483	0.506	0.526	0.559
5.0	0.504	0.534	0.559	0.579	0.596	0.623
4.0	0.592	0.617	0.636	0.652	0.666	0.686
3.0	0.678	0.696	0.710	0.721	0.730	0.743
2.0	0.751	0.762	0.770	0.776	0.781	0.788
1.0	0.787	0.794	0.799	0.803	0.806	0.810
0.0	0.749	0.763	0.774	0.781	0.787	0.795
-1.0	0.613	0.650	0.676	0.696	0.711	0.733
-2.0	0.456	0.511	0.552	0.583	0.608	0.645
-3.0	0.320	0.382	0.431	0.470	0.502	0.551
-4.0	0.218	0.277	0.327	0.368	0.404	0.460
-5.0	0.144	0.197	0.243	0.283	0.319	0.378
-6.0	0.094	0.137	0.178	0.215	0.249	0.307
-7.0	0.060	0.094	0.129	0.161	0.192	0.246
-8.0	0.038	0.064	0.092	0.120	0.146	0.196
-9.0	0.024	0.044	0.065	0.088	0.111	0.155
-10.0	0.015	0.029	0.046	0.065	0.084	0.122
-11.0	0.009	0.020	0.033	0.047	0.063	0.096
-12.0	0.006	0.013	0.023	0.034	0.047	0.075
-13.0	0.004	0.009	0.016	0.025	0.035	0.058
-14.0	0.002	0.006	0.011	0.018	0.026	0.045

MULTIPLE SOURCE - TIDAL RIVERS AND ESTUARIES
MG/L DO DEFICIT PER MG/L ULT. BOD INPUT

$\Phi = 0.50$ $K_d E/U^2$

$K_d X/U$

	20.00	30.00	40.00	50.00	70.00	100.00
57.0	0.001	0.004	0.009	0.015	0.030	0.057
54.0	0.002	0.006	0.012	0.020	0.038	0.069
51.0	0.003	0.008	0.016	0.026	0.047	0.082
48.0	0.004	0.011	0.021	0.033	0.059	0.099
45.0	0.006	0.016	0.029	0.043	0.073	0.118
42.0	0.009	0.022	0.038	0.055	0.090	0.141
39.0	0.014	0.031	0.050	0.071	0.112	0.167
36.0	0.020	0.042	0.067	0.091	0.138	0.198
33.0	0.030	0.058	0.088	0.117	0.169	0.235
30.0	0.044	0.080	0.115	0.148	0.206	0.276
27.0	0.064	0.109	0.151	0.188	0.251	0.324
24.0	0.093	0.148	0.196	0.237	0.304	0.377
21.0	0.135	0.199	0.252	0.296	0.365	0.437
18.0	0.192	0.266	0.322	0.367	0.434	0.502
15.0	0.271	0.349	0.406	0.449	0.511	0.571
12.0	0.374	0.451	0.503	0.541	0.594	0.643
9.0	0.503	0.568	0.609	0.639	0.677	0.712
6.0	0.648	0.690	0.715	0.732	0.754	0.772
3.0	0.777	0.791	0.799	0.804	0.810	0.815
0.0	0.811	0.817	0.820	0.821	0.823	0.825
-3.0	0.669	0.716	0.741	0.757	0.776	0.791
-6.0	0.480	0.565	0.616	0.649	0.692	0.727
-9.0	0.321	0.421	0.487	0.533	0.596	0.650
-12.0	0.205	0.302	0.373	0.426	0.500	0.570
-15.0	0.128	0.212	0.279	0.333	0.413	0.492
-18.0	0.078	0.146	0.205	0.256	0.336	0.419
-21.0	0.047	0.099	0.149	0.195	0.270	0.354
-24.0	0.028	0.067	0.107	0.147	0.215	0.297
-27.0	0.017	0.044	0.077	0.110	0.171	0.247
-30.0	0.010	0.029	0.055	0.081	0.134	0.205
-33.0	0.006	0.019	0.039	0.060	0.105	0.169
-36.0	0.003	0.013	0.027	0.044	0.082	0.138
-39.0	0.002	0.008	0.019	0.033	0.064	0.113
-42.0	0.001	0.005	0.013	0.024	0.050	0.092
-45.0	0.001	0.004	0.009	0.017	0.038	0.075
-48.0	0.000	0.002	0.006	0.013	0.030	0.061
-51.0	0.000	0.002	0.004	0.009	0.023	0.050

MULTIPLE SOURCE - TIDAL RIVERS AND ESTUARIES
MG/L DO DEFICIT PER MG/L ULT. BOD INPUT

$\Phi = 0.70$ $K_d E/U^2$

$K_d X/U$	0.01	0.05	0.10	0.20	0.30	0.40
10.0	0.003	0.004	0.005	0.006	0.009	0.011
9.5	0.004	0.005	0.006	0.009	0.011	0.014
9.0	0.006	0.007	0.009	0.012	0.015	0.018
8.5	0.008	0.010	0.012	0.016	0.020	0.024
8.0	0.012	0.014	0.016	0.021	0.026	0.031
7.5	0.016	0.019	0.022	0.028	0.034	0.041
7.0	0.023	0.026	0.030	0.037	0.045	0.053
6.5	0.031	0.035	0.040	0.050	0.059	0.068
6.0	0.043	0.048	0.054	0.066	0.077	0.087
5.5	0.059	0.065	0.072	0.086	0.099	0.111
5.0	0.080	0.088	0.096	0.113	0.127	0.141
4.5	0.108	0.117	0.127	0.146	0.163	0.178
4.0	0.144	0.155	0.167	0.187	0.206	0.222
3.5	0.190	0.202	0.215	0.238	0.257	0.273
3.0	0.246	0.258	0.272	0.296	0.316	0.332
2.5	0.309	0.323	0.337	0.361	0.380	0.396
2.0	0.375	0.389	0.404	0.427	0.445	0.460
1.5	0.427	0.442	0.458	0.482	0.499	0.513
1.0	0.435	0.454	0.474	0.502	0.522	0.537
0.5	0.337	0.369	0.402	0.448	0.480	0.503
0.0	0.019	0.087	0.153	0.247	0.312	0.358
-0.1	0.000	0.022	0.082	0.184	0.257	0.311
-0.2	0.000	0.004	0.038	0.129	0.205	0.263
-0.3	0.000	0.001	0.016	0.087	0.159	0.218
-0.4	0.000	0.000	0.007	0.057	0.121	0.178
-0.5	0.000	0.000	0.003	0.037	0.091	0.144
-0.6	0.000	0.000	0.001	0.023	0.067	0.115
-0.7	0.000	0.000	0.000	0.015	0.049	0.091
-0.8	0.000	0.000	0.000	0.009	0.036	0.072
-0.9	0.000	0.000	0.000	0.006	0.026	0.057
-1.0	0.000	0.000	0.000	0.003	0.019	0.044
-1.1	0.000	0.000	0.000	0.002	0.013	0.034
-1.2	0.000	0.000	0.000	0.001	0.009	0.027
-1.3	0.000	0.000	0.000	0.001	0.007	0.020
-1.4	0.000	0.000	0.000	0.000	0.005	0.016
-1.5	0.000	0.000	0.000	0.000	0.003	0.012

MULTIPLE SOURCE - TIDAL RIVERS AND ESTUARIES
MG/L DO DEFICIT PER MG/L ULT. BOD INPUT

$\Phi = 0.70$

$K_d E/U^2$

$K_d X/U$	0.50	0.60	0.80	1.00	2.00	3.00
12.0	0.005	0.006	0.008	0.011	0.027	0.045
11.5	0.006	0.007	0.010	0.014	0.032	0.052
11.0	0.008	0.009	0.013	0.017	0.038	0.061
10.5	0.010	0.012	0.017	0.021	0.046	0.071
10.0	0.013	0.016	0.021	0.026	0.055	0.082
9.5	0.017	0.020	0.026	0.033	0.065	0.095
9.0	0.022	0.026	0.033	0.041	0.077	0.110
8.5	0.028	0.033	0.041	0.050	0.091	0.127
8.0	0.037	0.042	0.052	0.062	0.108	0.146
7.5	0.047	0.053	0.065	0.076	0.127	0.168
7.0	0.060	0.067	0.081	0.094	0.149	0.193
6.5	0.076	0.085	0.100	0.115	0.175	0.220
6.0	0.097	0.106	0.124	0.140	0.204	0.250
5.5	0.122	0.133	0.152	0.170	0.237	0.284
5.0	0.153	0.165	0.186	0.205	0.274	0.321
4.5	0.191	0.204	0.226	0.245	0.315	0.360
4.0	0.236	0.249	0.272	0.291	0.359	0.402
3.5	0.288	0.301	0.324	0.343	0.407	0.446
3.0	0.347	0.359	0.381	0.399	0.456	0.489
2.5	0.410	0.421	0.441	0.456	0.505	0.532
2.0	0.472	0.482	0.498	0.511	0.550	0.570
1.5	0.523	0.532	0.546	0.557	0.587	0.601
1.0	0.549	0.558	0.572	0.582	0.608	0.619
0.5	0.520	0.534	0.554	0.569	0.603	0.617
0.0	0.393	0.421	0.462	0.490	0.559	0.587
-0.5	0.191	0.232	0.297	0.345	0.470	0.523
-1.0	0.074	0.105	0.164	0.214	0.369	0.444
-1.5	0.026	0.044	0.084	0.124	0.277	0.365
-2.0	0.009	0.017	0.041	0.069	0.202	0.293
-2.5	0.003	0.007	0.019	0.037	0.145	0.231
-3.0	0.001	0.002	0.009	0.020	0.102	0.180
-3.5	0.000	0.001	0.004	0.010	0.071	0.139
-4.0	0.000	0.000	0.002	0.005	0.049	0.106
-4.5	0.000	0.000	0.001	0.003	0.033	0.080
-5.0	0.000	0.000	0.000	0.001	0.022	0.061
-5.5	0.000	0.000	0.000	0.001	0.015	0.045
-6.0	0.000	0.000	0.000	0.000	0.010	0.034

MULTIPLE SOURCE - TIDAL RIVERS AND ESTUARIES
MG/L DO DEFICIT PER MG/L ULT. BOD INPUT

$\Phi = 0.70$ $K_d E/U^2$

$K_d X/U$	20.00	30.00	40.00	50.00	70.00	100.00
54.0	0.001	0.002	0.005	0.009	0.018	0.037
51.0	0.001	0.003	0.007	0.012	0.024	0.045
48.0	0.001	0.004	0.009	0.016	0.031	0.056
45.0	0.002	0.007	0.013	0.021	0.039	0.069
42.0	0.003	0.010	0.018	0.028	0.050	0.084
39.0	0.005	0.014	0.025	0.038	0.064	0.103
36.0	0.008	0.020	0.035	0.050	0.082	0.125
33.0	0.013	0.030	0.048	0.067	0.103	0.151
30.0	0.021	0.043	0.066	0.088	0.130	0.182
27.0	0.033	0.061	0.090	0.116	0.163	0.219
24.0	0.051	0.087	0.121	0.152	0.203	0.261
21.0	0.078	0.123	0.163	0.196	0.250	0.309
18.0	0.117	0.172	0.215	0.251	0.306	0.362
15.0	0.175	0.236	0.282	0.317	0.369	0.421
12.0	0.255	0.317	0.361	0.394	0.439	0.482
9.0	0.359	0.415	0.452	0.477	0.512	0.543
6.0	0.484	0.522	0.545	0.561	0.581	0.598
3.0	0.602	0.615	0.622	0.627	0.633	0.638
0.0	0.640	0.644	0.645	0.647	0.648	0.649
-3.0	0.518	0.557	0.577	0.591	0.606	0.619
-6.0	0.358	0.427	0.469	0.497	0.533	0.563
-9.0	0.229	0.308	0.361	0.399	0.450	0.496
-12.0	0.140	0.213	0.268	0.310	0.370	0.428
-15.0	0.083	0.143	0.193	0.235	0.298	0.362
-18.0	0.048	0.094	0.137	0.175	0.236	0.303
-21.0	0.027	0.061	0.096	0.129	0.185	0.250
-24.0	0.015	0.039	0.067	0.094	0.144	0.205
-27.0	0.008	0.025	0.046	0.068	0.111	0.167
-30.0	0.005	0.016	0.031	0.049	0.085	0.135
-33.0	0.003	0.010	0.021	0.035	0.064	0.109
-36.0	0.001	0.006	0.014	0.025	0.049	0.087
-39.0	0.001	0.004	0.010	0.017	0.037	0.069
-42.0	0.000	0.002	0.006	0.012	0.028	0.055
-45.0	0.000	0.001	0.004	0.009	0.021	0.044
-48.0	0.000	0.001	0.003	0.006	0.015	0.035
-51.0	0.000	0.001	0.002	0.004	0.011	0.027

MULTIPLE SOURCE - TIDAL RIVERS AND ESTUARIES
MG/L DO DEFICIT PER MG/L ULT. BOD INPUT

$\Phi = 0.70$

K_dE/U^2

K_dX/U	4.00	5.00	6.00	7.00	8.00	10.00
20.0	0.006	0.010	0.014	0.019	0.024	0.034
19.0	0.009	0.013	0.018	0.024	0.030	0.042
18.0	0.011	0.017	0.023	0.030	0.037	0.051
17.0	0.015	0.022	0.030	0.038	0.046	0.062
16.0	0.020	0.029	0.038	0.047	0.056	0.074
15.0	0.027	0.038	0.048	0.059	0.069	0.090
14.0	0.036	0.049	0.061	0.073	0.085	0.108
13.0	0.048	0.063	0.077	0.091	0.105	0.130
12.0	0.063	0.080	0.097	0.113	0.128	0.155
11.0	0.082	0.103	0.121	0.139	0.155	0.185
10.0	0.107	0.130	0.151	0.170	0.188	0.219
9.0	0.139	0.165	0.187	0.208	0.226	0.258
8.0	0.179	0.206	0.230	0.252	0.270	0.302
7.0	0.227	0.256	0.281	0.302	0.320	0.351
6.0	0.286	0.315	0.339	0.359	0.376	0.404
5.0	0.355	0.382	0.404	0.421	0.436	0.461
4.0	0.432	0.455	0.472	0.487	0.499	0.518
3.0	0.512	0.528	0.540	0.550	0.558	0.571
2.0	0.583	0.592	0.599	0.604	0.608	0.614
1.0	0.625	0.630	0.632	0.635	0.636	0.639
0.0	0.601	0.611	0.617	0.622	0.625	0.630
-1.0	0.487	0.515	0.535	0.550	0.562	0.578
-2.0	0.354	0.397	0.429	0.454	0.474	0.503
-3.0	0.242	0.290	0.328	0.358	0.384	0.423
-4.0	0.159	0.204	0.242	0.275	0.302	0.347
-5.0	0.102	0.141	0.175	0.206	0.234	0.279
-6.0	0.064	0.095	0.125	0.152	0.178	0.222
-7.0	0.040	0.063	0.087	0.111	0.134	0.174
-8.0	0.024	0.042	0.061	0.080	0.099	0.136
-9.0	0.015	0.027	0.042	0.057	0.073	0.105
-10.0	0.009	0.018	0.029	0.041	0.054	0.080
-11.0	0.005	0.011	0.019	0.029	0.039	0.061
-12.0	0.003	0.007	0.013	0.020	0.028	0.047
-13.0	0.002	0.005	0.009	0.014	0.021	0.035
-14.0	0.001	0.003	0.006	0.010	0.015	0.027
-15.0	0.001	0.002	0.004	0.007	0.011	0.020

MULTIPLE SOURCE - TIDAL RIVERS AND ESTUARIES
MG/L DO DEFICIT PER MG/L ULT. BOD INPUT

$\Phi = 0.90$ K_dE/U^2

K_dX/U	0.01	0.05	0.10	0.20	0.30	0.40
6.0	0.021	0.024	0.028	0.036	0.044	0.051
5.8	0.025	0.028	0.033	0.041	0.050	0.058
5.6	0.029	0.033	0.038	0.047	0.056	0.064
5.4	0.033	0.038	0.043	0.053	0.063	0.072
5.2	0.039	0.044	0.049	0.061	0.071	0.081
5.0	0.045	0.050	0.057	0.069	0.080	0.090
4.8	0.052	0.058	0.065	0.078	0.090	0.101
4.6	0.060	0.067	0.074	0.088	0.101	0.112
4.4	0.070	0.076	0.084	0.099	0.113	0.125
4.2	0.080	0.088	0.096	0.112	0.126	0.138
4.0	0.092	0.100	0.109	0.126	0.140	0.154
3.8	0.106	0.114	0.124	0.141	0.156	0.170
3.6	0.121	0.130	0.140	0.158	0.174	0.187
3.4	0.138	0.147	0.158	0.176	0.192	0.207
3.2	0.156	0.166	0.177	0.196	0.213	0.227
3.0	0.177	0.187	0.198	0.218	0.234	0.248
2.8	0.199	0.210	0.221	0.241	0.257	0.271
2.6	0.223	0.234	0.246	0.265	0.281	0.295
2.4	0.249	0.260	0.271	0.291	0.307	0.320
2.2	0.276	0.286	0.298	0.317	0.332	0.345
2.0	0.303	0.313	0.325	0.343	0.358	0.370
1.8	0.329	0.340	0.351	0.369	0.383	0.394
1.6	0.353	0.364	0.376	0.393	0.406	0.416
1.4	0.374	0.385	0.397	0.414	0.427	0.436
1.2	0.388	0.400	0.412	0.430	0.443	0.452
1.0	0.391	0.405	0.419	0.439	0.452	0.462
0.8	0.379	0.397	0.414	0.437	0.453	0.464
0.6	0.346	0.369	0.391	0.421	0.441	0.455
0.4	0.283	0.315	0.345	0.386	0.413	0.432
0.2	0.179	0.224	0.267	0.325	0.363	0.389
-0.0	0.019	0.084	0.146	0.230	0.284	0.323
-0.2	0.000	0.004	0.036	0.119	0.186	0.236
-0.4	0.000	0.000	0.006	0.052	0.109	0.159
-0.6	0.000	0.000	0.001	0.021	0.060	0.102
-0.8	0.000	0.000	0.000	0.008	0.031	0.063
-1.0	0.000	0.000	0.000	0.003	0.016	0.038

MULTIPLE SOURCE - TIDAL RIVERS AND ESTUARIES
MG/L DO DEFICIT PER MG/L ULT. BOD INPUT

$\Phi = 0.90$ $K_d E/U^2$

$K_d X/U$	0.50	0.60	0.80	1.00	2.00	3.00
12.0	0.002	0.002	0.003	0.005	0.014	0.026
11.5	0.002	0.003	0.004	0.006	0.018	0.031
11.0	0.003	0.004	0.006	0.008	0.021	0.037
10.5	0.004	0.005	0.008	0.010	0.026	0.043
10.0	0.006	0.007	0.010	0.013	0.032	0.051
9.5	0.008	0.009	0.013	0.017	0.039	0.061
9.0	0.010	0.013	0.017	0.022	0.047	0.071
8.5	0.014	0.017	0.022	0.028	0.057	0.084
8.0	0.019	0.022	0.029	0.036	0.069	0.098
7.5	0.025	0.029	0.037	0.045	0.083	0.115
7.0	0.033	0.038	0.048	0.058	0.100	0.134
6.5	0.044	0.050	0.062	0.073	0.119	0.156
6.0	0.058	0.065	0.079	0.091	0.142	0.181
5.5	0.077	0.085	0.100	0.114	0.169	0.208
5.0	0.100	0.109	0.126	0.141	0.199	0.239
4.5	0.129	0.139	0.158	0.174	0.233	0.273
4.0	0.165	0.176	0.196	0.212	0.272	0.310
3.5	0.209	0.221	0.240	0.257	0.314	0.349
3.0	0.261	0.272	0.291	0.306	0.358	0.389
2.5	0.319	0.329	0.346	0.360	0.404	0.428
2.0	0.380	0.388	0.402	0.413	0.447	0.466
1.5	0.435	0.442	0.453	0.461	0.485	0.497
1.0	0.470	0.476	0.485	0.492	0.509	0.517
0.5	0.457	0.467	0.480	0.489	0.511	0.520
0.0	0.351	0.373	0.404	0.426	0.476	0.496
-0.5	0.168	0.203	0.257	0.297	0.398	0.440
-1.0	0.064	0.090	0.139	0.181	0.309	0.371
-1.5	0.022	0.036	0.069	0.103	0.229	0.301
-2.0	0.007	0.014	0.033	0.056	0.165	0.239
-2.5	0.002	0.005	0.015	0.030	0.116	0.186
-3.0	0.001	0.002	0.007	0.015	0.080	0.143
-3.5	0.000	0.001	0.003	0.008	0.055	0.109
-4.0	0.000	0.000	0.001	0.004	0.037	0.082
-4.5	0.000	0.000	0.001	0.002	0.025	0.061
-5.0	0.000	0.000	0.000	0.001	0.016	0.045

MULTIPLE SOURCE - TIDAL RIVERS AND ESTUARIES
MG/L DO DEFICIT PER MG/L ULT. BOD INPUT

$\Phi = 0.90$

$K_d E/U^2$

$K_d X/U$	4.00	5.00	6.00	7.00	8.00	10.00
21.0	0.002	0.004	0.006	0.008	0.010	0.016
20.0	0.003	0.005	0.007	0.010	0.013	0.020
19.0	0.004	0.007	0.010	0.013	0.017	0.025
18.0	0.006	0.009	0.013	0.017	0.022	0.031
17.0	0.008	0.012	0.017	0.022	0.028	0.039
16.0	0.011	0.016	0.022	0.028	0.035	0.048
15.0	0.015	0.022	0.029	0.036	0.044	0.059
14.0	0.021	0.029	0.038	0.047	0.055	0.072
13.0	0.028	0.039	0.049	0.059	0.069	0.088
12.0	0.038	0.051	0.063	0.075	0.087	0.108
11.0	0.052	0.067	0.081	0.095	0.108	0.131
10.0	0.070	0.088	0.104	0.119	0.133	0.158
9.0	0.094	0.114	0.132	0.148	0.163	0.190
8.0	0.124	0.146	0.166	0.184	0.199	0.226
7.0	0.163	0.187	0.208	0.226	0.241	0.268
6.0	0.211	0.236	0.256	0.274	0.289	0.314
5.0	0.269	0.293	0.312	0.328	0.342	0.364
4.0	0.337	0.357	0.373	0.387	0.398	0.415
3.0	0.409	0.424	0.436	0.445	0.453	0.465
2.0	0.477	0.486	0.492	0.497	0.501	0.506
1.0	0.522	0.525	0.527	0.529	0.530	0.532
0.0	0.507	0.513	0.518	0.521	0.523	0.527
-1.0	0.406	0.430	0.446	0.458	0.468	0.481
-2.0	0.290	0.325	0.352	0.373	0.390	0.415
-3.0	0.193	0.233	0.264	0.290	0.311	0.344
-4.0	0.124	0.160	0.192	0.218	0.241	0.278
-5.0	0.077	0.108	0.136	0.161	0.183	0.221
-6.0	0.047	0.071	0.094	0.116	0.137	0.172
-7.0	0.028	0.046	0.065	0.083	0.101	0.133
-8.0	0.017	0.030	0.044	0.059	0.073	0.102
-9.0	0.010	0.019	0.029	0.041	0.053	0.077
-10.0	0.006	0.012	0.020	0.029	0.038	0.058
-11.0	0.003	0.007	0.013	0.020	0.027	0.044
-12.0	0.002	0.005	0.009	0.014	0.019	0.032
-13.0	0.001	0.003	0.006	0.009	0.014	0.024
-14.0	0.001	0.002	0.004	0.006	0.010	0.018

MULTIPLE SOURCE - TIDAL RIVERS AND ESTUARIES
MG/L DO DEFICIT PER MG/L ULT. BOD INPUT

Φ = 0.90

$K_d E/U^2$

$K_d X/U$	20.00	30.00	40.00	50.00	70.00	100.00
57.0	0.000	0.001	0.002	0.003	0.008	0.018
54.0	0.000	0.001	0.002	0.005	0.011	0.023
51.0	0.000	0.001	0.003	0.006	0.014	0.029
48.0	0.001	0.002	0.005	0.009	0.019	0.036
45.0	0.001	0.003	0.007	0.012	0.024	0.045
42.0	0.002	0.005	0.010	0.017	0.032	0.057
39.0	0.003	0.008	0.015	0.023	0.042	0.070
36.0	0.004	0.012	0.021	0.032	0.054	0.087
33.0	0.007	0.018	0.030	0.044	0.071	0.108
30.0	0.012	0.027	0.043	0.059	0.091	0.132
27.0	0.020	0.039	0.060	0.080	0.116	0.161
24.0	0.032	0.058	0.084	0.107	0.148	0.196
21.0	0.051	0.085	0.116	0.143	0.187	0.236
18.0	0.080	0.123	0.158	0.187	0.233	0.281
15.0	0.124	0.174	0.212	0.242	0.287	0.332
12.0	0.189	0.241	0.279	0.307	0.348	0.386
9.0	0.276	0.326	0.358	0.381	0.412	0.441
6.0	0.385	0.420	0.442	0.456	0.475	0.491
3.0	0.494	0.507	0.514	0.518	0.524	0.528
0.0	0.534	0.536	0.537	0.538	0.539	0.539
-3.0	0.425	0.458	0.476	0.488	0.502	0.513
-6.0	0.285	0.344	0.380	0.405	0.436	0.462
-9.0	0.176	0.241	0.286	0.318	0.363	0.403
-12.0	0.103	0.162	0.207	0.242	0.293	0.342
-15.0	0.059	0.105	0.146	0.179	0.231	0.285
-18.0	0.033	0.067	0.101	0.130	0.180	0.235
-21.0	0.018	0.042	0.068	0.094	0.138	0.191
-24.0	0.010	0.026	0.046	0.066	0.105	0.154
-27.0	0.005	0.016	0.031	0.047	0.079	0.123
-30.0	0.003	0.010	0.020	0.033	0.059	0.098
-33.0	0.001	0.006	0.013	0.023	0.044	0.077
-36.0	0.001	0.004	0.009	0.016	0.033	0.061
-39.0	0.000	0.002	0.006	0.011	0.024	0.048
-42.0	0.000	0.001	0.004	0.007	0.018	0.037
-45.0	0.000	0.001	0.002	0.005	0.013	0.029
-48.0	0.000	0.000	0.002	0.003	0.009	0.022

MULTIPLE SOURCE - TIDAL RIVERS AND ESTUARIES
MG/L DO DEFICIT PER MG/L ULT. BOD INPUT

$\Phi = 1.50$

K_dE/U^2

K_dX/U	0.01	0.05	0.10	0.20	0.30	0.40
5.0	0.013	0.015	0.018	0.024	0.029	0.035
4.8	0.016	0.018	0.021	0.028	0.034	0.040
4.6	0.019	0.022	0.025	0.032	0.039	0.045
4.4	0.023	0.026	0.030	0.038	0.045	0.052
4.2	0.027	0.031	0.035	0.044	0.052	0.059
4.0	0.033	0.037	0.042	0.051	0.060	0.068
3.8	0.039	0.044	0.049	0.060	0.069	0.077
3.6	0.047	0.052	0.058	0.069	0.079	0.088
3.4	0.056	0.062	0.068	0.080	0.090	0.100
3.2	0.067	0.073	0.080	0.092	0.103	0.113
3.0	0.079	0.085	0.093	0.106	0.117	0.127
2.8	0.093	0.100	0.108	0.122	0.133	0.143
2.6	0.110	0.117	0.125	0.139	0.151	0.161
2.4	0.129	0.136	0.144	0.158	0.169	0.179
2.2	0.150	0.157	0.165	0.179	0.190	0.199
2.0	0.173	0.180	0.188	0.201	0.212	0.220
1.8	0.198	0.205	0.212	0.224	0.234	0.242
1.6	0.224	0.230	0.237	0.248	0.257	0.264
1.4	0.250	0.256	0.262	0.272	0.279	0.285
1.2	0.273	0.279	0.284	0.293	0.300	0.305
1.0	0.291	0.297	0.303	0.311	0.317	0.321
0.8	0.299	0.306	0.313	0.322	0.328	0.332
0.6	0.288	0.299	0.309	0.322	0.329	0.335
0.4	0.249	0.268	0.285	0.305	0.318	0.326
0.2	0.167	0.201	0.230	0.265	0.286	0.300
-0.0	0.019	0.078	0.129	0.191	0.227	0.251
-0.1	0.000	0.020	0.068	0.141	0.187	0.217
-0.2	0.000	0.004	0.031	0.098	0.147	0.182
-0.3	0.000	0.001	0.013	0.064	0.112	0.149
-0.4	0.000	0.000	0.005	0.041	0.084	0.120
-0.5	0.000	0.000	0.002	0.026	0.061	0.095
-0.6	0.000	0.000	0.001	0.016	0.045	0.075
-0.7	0.000	0.000	0.000	0.010	0.032	0.058
-0.8	0.000	0.000	0.000	0.006	0.023	0.045
-0.9	0.000	0.000	0.000	0.004	0.016	0.035
-1.0	0.000	0.000	0.000	0.002	0.011	0.026

MULTIPLE SOURCE - TIDAL RIVERS AND ESTUARIES
MG/L DO DEFICIT PER MG/L ULT. BOD INPUT

$\Phi = 1.50$ K_dE/U^2

K_dX/U

	0.50	0.60	0.80	1.00	2.00	3.00
8.0	0.005	0.006	0.009	0.012	0.028	0.043
7.5	0.007	0.009	0.012	0.016	0.034	0.052
7.0	0.010	0.012	0.017	0.021	0.043	0.063
6.5	0.015	0.017	0.023	0.028	0.053	0.075
6.0	0.020	0.024	0.030	0.037	0.066	0.090
5.5	0.029	0.033	0.041	0.049	0.082	0.107
5.0	0.040	0.045	0.054	0.063	0.100	0.127
4.5	0.055	0.061	0.072	0.082	0.122	0.150
4.0	0.075	0.082	0.095	0.106	0.148	0.176
3.5	0.102	0.110	0.123	0.135	0.177	0.204
3.0	0.136	0.144	0.158	0.170	0.210	0.235
2.5	0.179	0.186	0.200	0.211	0.246	0.267
2.0	0.228	0.235	0.246	0.255	0.283	0.299
1.5	0.281	0.285	0.293	0.299	0.318	0.328
1.0	0.325	0.328	0.332	0.336	0.346	0.350
0.5	0.336	0.340	0.345	0.348	0.356	0.359
0.0	0.268	0.281	0.298	0.310	0.336	0.346
-0.2	0.207	0.227	0.255	0.273	0.316	0.332
-0.4	0.149	0.172	0.207	0.232	0.291	0.314
-0.6	0.102	0.126	0.163	0.191	0.264	0.294
-0.8	0.068	0.089	0.126	0.155	0.236	0.273
-1.0	0.044	0.062	0.095	0.123	0.210	0.251
-1.2	0.028	0.042	0.071	0.097	0.184	0.230
-1.4	0.018	0.029	0.053	0.076	0.161	0.209
-1.6	0.011	0.019	0.038	0.059	0.140	0.189
-1.8	0.007	0.013	0.028	0.045	0.121	0.171
-2.0	0.004	0.008	0.020	0.035	0.104	0.154
-2.2	0.003	0.005	0.015	0.026	0.089	0.138
-2.4	0.002	0.004	0.010	0.020	0.076	0.123
-2.6	0.001	0.002	0.007	0.015	0.065	0.110
-2.8	0.001	0.002	0.005	0.011	0.055	0.097
-3.0	0.000	0.001	0.004	0.008	0.047	0.086
-3.2	0.000	0.001	0.003	0.006	0.040	0.077
-3.4	0.000	0.000	0.002	0.005	0.034	0.068
-3.6	0.000	0.000	0.001	0.004	0.028	0.060
-3.8	0.000	0.000	0.001	0.003	0.024	0.053
-4.0	0.000	0.000	0.001	0.002	0.020	0.046

MULTIPLE SOURCE - TIDAL RIVERS AND ESTUARIES
MG/L DO DEFICIT PER MG/L ULT. BOD INPUT

$\Phi = 1.50$ $K_d E/U^2$

$K_d X/U$	4.00	5.00	6.00	7.00	8.00	10.00
15.0	0.005	0.008	0.011	0.014	0.017	0.025
14.0	0.007	0.011	0.014	0.019	0.023	0.032
13.0	0.010	0.015	0.020	0.025	0.030	0.040
12.0	0.014	0.020	0.026	0.033	0.039	0.051
11.0	0.021	0.028	0.036	0.043	0.050	0.064
10.0	0.029	0.039	0.048	0.056	0.064	0.080
9.0	0.041	0.053	0.063	0.073	0.083	0.099
8.0	0.058	0.071	0.083	0.095	0.105	0.123
7.0	0.080	0.096	0.109	0.121	0.132	0.151
6.0	0.110	0.127	0.141	0.154	0.165	0.183
5.0	0.149	0.166	0.180	0.192	0.202	0.219
4.0	0.196	0.212	0.225	0.236	0.245	0.259
3.0	0.252	0.265	0.274	0.282	0.289	0.299
2.0	0.309	0.317	0.322	0.326	0.330	0.335
1.0	0.353	0.355	0.357	0.358	0.359	0.360
0.0	0.351	0.354	0.356	0.358	0.359	0.360
-0.5	0.319	0.328	0.334	0.338	0.342	0.347
-1.0	0.275	0.291	0.302	0.310	0.317	0.326
-1.5	0.230	0.251	0.266	0.278	0.287	0.301
-2.0	0.188	0.212	0.231	0.245	0.257	0.275
-2.5	0.150	0.177	0.197	0.214	0.227	0.248
-3.0	0.119	0.145	0.166	0.184	0.198	0.221
-3.5	0.093	0.118	0.139	0.157	0.172	0.197
-4.0	0.072	0.095	0.116	0.133	0.148	0.173
-4.5	0.056	0.077	0.095	0.112	0.127	0.152
-5.0	0.043	0.061	0.078	0.094	0.108	0.133
-5.5	0.032	0.048	0.064	0.078	0.092	0.116
-6.0	0.025	0.038	0.052	0.065	0.078	0.100
-6.5	0.019	0.030	0.042	0.054	0.066	0.087
-7.0	0.014	0.024	0.034	0.045	0.055	0.075
-7.5	0.010	0.018	0.027	0.037	0.046	0.064
-8.0	0.008	0.014	0.022	0.030	0.039	0.055
-8.5	0.006	0.011	0.018	0.025	0.032	0.047
-9.0	0.004	0.009	0.014	0.020	0.027	0.040
-9.5	0.003	0.007	0.011	0.017	0.022	0.034
-10.0	0.002	0.005	0.009	0.013	0.018	0.029

MULTIPLE SOURCE - TIDAL RIVERS AND ESTUARIES
MG/L DO DEFICIT PER MG/L ULT. BOD INPUT

$\Phi = 1.50$ K_dE/U^2

K_dX/U	20.00	30.00	40.00	50.00	70.00	100.00
36.0	0.001	0.004	0.008	0.013	0.024	0.041
34.0	0.002	0.006	0.011	0.016	0.029	0.049
32.0	0.003	0.008	0.014	0.021	0.036	0.057
30.0	0.004	0.010	0.018	0.026	0.043	0.067
28.0	0.006	0.014	0.023	0.033	0.053	0.079
26.0	0.009	0.019	0.030	0.042	0.064	0.092
24.0	0.012	0.025	0.039	0.052	0.077	0.107
22.0	0.018	0.034	0.050	0.065	0.092	0.124
20.0	0.026	0.046	0.064	0.081	0.110	0.143
18.0	0.036	0.061	0.082	0.100	0.131	0.164
16.0	0.052	0.080	0.104	0.123	0.155	0.188
14.0	0.072	0.104	0.130	0.150	0.181	0.213
12.0	0.100	0.135	0.161	0.182	0.211	0.240
10.0	0.136	0.173	0.198	0.217	0.243	0.268
8.0	0.182	0.216	0.239	0.255	0.277	0.297
6.0	0.237	0.265	0.282	0.294	0.310	0.323
4.0	0.297	0.314	0.324	0.331	0.339	0.346
2.0	0.348	0.353	0.356	0.358	0.360	0.362
0.0	0.364	0.365	0.365	0.366	0.366	0.366
-2.0	0.315	0.331	0.339	0.344	0.350	0.355
-4.0	0.243	0.275	0.293	0.305	0.320	0.333
-6.0	0.176	0.217	0.243	0.261	0.284	0.305
-8.0	0.122	0.166	0.196	0.217	0.247	0.274
-10.0	0.083	0.124	0.154	0.178	0.211	0.243
-12.0	0.055	0.091	0.120	0.143	0.178	0.213
-14.0	0.036	0.066	0.092	0.114	0.149	0.185
-16.0	0.023	0.047	0.069	0.090	0.123	0.160
-18.0	0.015	0.033	0.052	0.070	0.101	0.137
-20.0	0.009	0.023	0.039	0.054	0.083	0.117
-22.0	0.006	0.016	0.029	0.042	0.067	0.099
-24.0	0.004	0.011	0.021	0.032	0.054	0.084
-26.0	0.002	0.008	0.016	0.025	0.044	0.071
-28.0	0.001	0.005	0.012	0.019	0.035	0.059
-30.0	0.001	0.004	0.008	0.014	0.028	0.050
-32.0	0.001	0.003	0.006	0.011	0.023	0.042
-34.0	0.000	0.002	0.004	0.008	0.018	0.035

MULTIPLE SOURCE - TIDAL RIVERS AND ESTUARIES
MG/L DO DEFICIT PER MG/L ULT. BOD INPUT

$\Phi = 2.00$

$K_d E/U^2$

$K_d X/U$	0.01	0.05	0.10	0.20	0.30	0.40
5.0	0.007	0.008	0.010	0.014	0.017	0.021
4.8	0.009	0.010	0.012	0.016	0.020	0.024
4.6	0.010	0.012	0.014	0.019	0.023	0.028
4.4	0.013	0.015	0.017	0.022	0.027	0.032
4.2	0.015	0.018	0.021	0.026	0.032	0.037
4.0	0.019	0.021	0.025	0.031	0.037	0.043
3.8	0.023	0.026	0.029	0.036	0.043	0.049
3.6	0.027	0.031	0.035	0.043	0.050	0.057
3.4	0.033	0.037	0.042	0.050	0.058	0.065
3.2	0.040	0.045	0.050	0.059	0.067	0.074
3.0	0.049	0.053	0.059	0.069	0.077	0.085
2.8	0.058	0.064	0.070	0.080	0.089	0.097
2.6	0.070	0.076	0.082	0.093	0.102	0.111
2.4	0.084	0.090	0.096	0.108	0.117	0.125
2.2	0.100	0.106	0.113	0.124	0.134	0.142
2.0	0.119	0.125	0.131	0.142	0.151	0.159
1.8	0.140	0.145	0.152	0.162	0.171	0.178
1.6	0.163	0.168	0.174	0.183	0.191	0.197
1.4	0.187	0.192	0.197	0.205	0.212	0.217
1.2	0.212	0.215	0.220	0.226	0.232	0.236
1.0	0.233	0.237	0.240	0.246	0.250	0.253
0.8	0.249	0.252	0.256	0.261	0.264	0.267
0.6	0.250	0.256	0.261	0.267	0.271	0.274
0.4	0.225	0.238	0.248	0.260	0.266	0.271
0.2	0.157	0.184	0.206	0.230	0.244	0.253
-0.0	0.019	0.074	0.118	0.168	0.196	0.213
-0.1	0.000	0.019	0.062	0.124	0.160	0.184
-0.2	0.000	0.003	0.028	0.085	0.125	0.153
-0.3	0.000	0.001	0.012	0.055	0.095	0.125
-0.4	0.000	0.000	0.005	0.035	0.070	0.100
-0.5	0.000	0.000	0.002	0.022	0.051	0.078
-0.6	0.000	0.000	0.001	0.013	0.037	0.061
-0.7	0.000	0.000	0.000	0.008	0.026	0.047
-0.8	0.000	0.000	0.000	0.005	0.018	0.036
-0.9	0.000	0.000	0.000	0.003	0.013	0.027
-1.0	0.000	0.000	0.000	0.002	0.009	0.021

MULTIPLE SOURCE - TIDAL RIVERS AND ESTUARIES
MG/L DO DEFICIT PER MG/L ULT. BOD INPUT

$\Phi = 2.00$

$K_d E/U^2$

$K_d X/U$	0.50	0.60	0.80	1.00	2.00	3.00
8.0	0.003	0.004	0.005	0.007	0.017	0.028
7.5	0.004	0.005	0.007	0.009	0.021	0.034
7.0	0.006	0.007	0.010	0.013	0.027	0.041
6.5	0.008	0.010	0.013	0.017	0.034	0.050
6.0	0.012	0.014	0.018	0.023	0.043	0.061
5.5	0.017	0.020	0.025	0.030	0.054	0.073
5.0	0.024	0.028	0.034	0.040	0.067	0.088
4.5	0.034	0.038	0.046	0.054	0.084	0.106
4.0	0.048	0.053	0.062	0.071	0.103	0.126
3.5	0.067	0.073	0.083	0.092	0.126	0.149
3.0	0.092	0.099	0.110	0.119	0.153	0.174
2.5	0.125	0.132	0.143	0.152	0.183	0.201
2.0	0.166	0.172	0.182	0.190	0.215	0.229
1.5	0.212	0.217	0.224	0.229	0.247	0.256
1.0	0.256	0.258	0.262	0.265	0.273	0.278
0.5	0.276	0.278	0.280	0.282	0.286	0.288
0.0	0.225	0.234	0.247	0.255	0.272	0.279
-0.2	0.174	0.189	0.210	0.224	0.256	0.267
-0.4	0.123	0.142	0.169	0.189	0.234	0.252
-0.6	0.083	0.102	0.132	0.154	0.211	0.235
-0.8	0.054	0.071	0.100	0.124	0.188	0.217
-1.0	0.035	0.049	0.075	0.097	0.166	0.199
-1.2	0.022	0.033	0.055	0.076	0.145	0.181
-1.4	0.013	0.022	0.040	0.058	0.125	0.164
-1.6	0.008	0.014	0.029	0.045	0.108	0.147
-1.8	0.005	0.009	0.021	0.034	0.093	0.132
-2.0	0.003	0.006	0.015	0.026	0.079	0.118
-2.2	0.002	0.004	0.011	0.019	0.067	0.105
-2.4	0.001	0.003	0.007	0.014	0.057	0.093
-2.6	0.001	0.002	0.005	0.011	0.048	0.082
-2.8	0.000	0.001	0.004	0.008	0.041	0.073
-3.0	0.000	0.001	0.003	0.006	0.034	0.064
-3.2	0.000	0.000	0.002	0.004	0.029	0.056
-3.4	0.000	0.000	0.001	0.003	0.024	0.049
-3.6	0.000	0.000	0.001	0.002	0.020	0.043
-3.8	0.000	0.000	0.001	0.002	0.017	0.038
-4.0	0.000	0.000	0.000	0.001	0.014	0.033

MULTIPLE SOURCE - TIDAL RIVERS AND ESTUARIES
MG/L DO DEFICIT PER MG/L ULT. BOD INPUT

$\Phi = 2.00$ K_dE/U^2

K_dX/U	4.00	5.00	6.00	7.00	8.00	10.00
10.0	0.018	0.025	0.031	0.037	0.043	0.054
9.5	0.022	0.029	0.036	0.043	0.049	0.061
9.0	0.026	0.034	0.042	0.049	0.056	0.068
8.5	0.032	0.040	0.049	0.056	0.064	0.077
8.0	0.038	0.047	0.056	0.065	0.072	0.086
7.5	0.045	0.056	0.065	0.074	0.082	0.096
7.0	0.054	0.065	0.075	0.085	0.093	0.108
6.5	0.064	0.076	0.087	0.096	0.105	0.120
6.0	0.076	0.089	0.100	0.110	0.118	0.133
5.5	0.089	0.103	0.114	0.124	0.133	0.147
5.0	0.105	0.119	0.130	0.140	0.149	0.163
4.5	0.123	0.137	0.148	0.157	0.166	0.179
4.0	0.143	0.156	0.167	0.176	0.183	0.196
3.5	0.165	0.177	0.187	0.195	0.202	0.213
3.0	0.189	0.200	0.209	0.215	0.221	0.230
2.5	0.214	0.223	0.230	0.236	0.240	0.247
2.0	0.239	0.246	0.251	0.255	0.258	0.263
1.5	0.262	0.266	0.269	0.272	0.273	0.276
1.0	0.280	0.282	0.283	0.284	0.285	0.286
0.5	0.289	0.290	0.290	0.291	0.291	0.291
0.0	0.282	0.284	0.286	0.287	0.287	0.289
-0.5	0.255	0.262	0.267	0.270	0.273	0.277
-1.0	0.218	0.231	0.240	0.246	0.252	0.259
-1.5	0.180	0.197	0.210	0.219	0.227	0.238
-2.0	0.145	0.165	0.180	0.191	0.201	0.215
-2.5	0.114	0.135	0.152	0.165	0.176	0.193
-3.0	0.089	0.110	0.126	0.140	0.152	0.171
-3.5	0.069	0.088	0.105	0.119	0.131	0.150
-4.0	0.053	0.070	0.086	0.099	0.111	0.131
-4.5	0.040	0.056	0.070	0.083	0.094	0.114
-5.0	0.030	0.044	0.057	0.069	0.080	0.099
-5.5	0.023	0.034	0.046	0.057	0.067	0.085
-6.0	0.017	0.027	0.037	0.047	0.056	0.073
-6.5	0.013	0.021	0.029	0.038	0.047	0.063
-7.0	0.009	0.016	0.023	0.031	0.039	0.053

MULTIPLE SOURCE - TIDAL RIVERS AND ESTUARIES
MG/L DO DEFICIT PER MG/L ULT. BOD INPUT

$\Phi = 2.00$

$K_d E/U^2$

$K_d X/U$	20.00	30.00	40.00	50.00	70.00	100.00
36.0	0.001	0.002	0.005	0.008	0.015	0.027
34.0	0.001	0.003	0.006	0.010	0.019	0.033
32.0	0.002	0.005	0.009	0.013	0.023	0.039
30.0	0.002	0.006	0.011	0.017	0.029	0.046
28.0	0.003	0.009	0.015	0.022	0.035	0.054
26.0	0.005	0.012	0.019	0.027	0.043	0.064
24.0	0.008	0.016	0.026	0.035	0.053	0.075
22.0	0.011	0.022	0.033	0.044	0.064	0.088
20.0	0.016	0.030	0.044	0.056	0.078	0.103
18.0	0.024	0.041	0.056	0.070	0.093	0.120
16.0	0.034	0.055	0.073	0.088	0.112	0.139
14.0	0.049	0.073	0.093	0.109	0.134	0.159
12.0	0.069	0.097	0.117	0.134	0.158	0.182
10.0	0.097	0.126	0.147	0.162	0.184	0.206
8.0	0.133	0.161	0.180	0.194	0.213	0.230
6.0	0.178	0.202	0.217	0.227	0.241	0.253
4.0	0.229	0.244	0.253	0.260	0.267	0.274
2.0	0.275	0.280	0.283	0.284	0.287	0.288
0.0	0.291	0.291	0.292	0.292	0.292	0.292
-2.0	0.249	0.262	0.269	0.273	0.278	0.283
-4.0	0.187	0.214	0.229	0.240	0.252	0.263
-6.0	0.132	0.165	0.187	0.202	0.221	0.238
-8.0	0.089	0.124	0.148	0.165	0.190	0.212
-10.0	0.059	0.090	0.114	0.133	0.160	0.186
-12.0	0.038	0.065	0.087	0.105	0.133	0.161
-14.0	0.024	0.046	0.065	0.082	0.109	0.139
-16.0	0.015	0.032	0.049	0.064	0.089	0.118
-18.0	0.010	0.022	0.036	0.049	0.072	0.100
-20.0	0.006	0.015	0.026	0.037	0.058	0.084
-22.0	0.004	0.011	0.019	0.029	0.047	0.071
-24.0	0.002	0.007	0.014	0.022	0.037	0.059
-26.0	0.001	0.005	0.010	0.016	0.030	0.049
-28.0	0.001	0.003	0.007	0.012	0.024	0.041
-30.0	0.001	0.002	0.005	0.009	0.019	0.034
-32.0	0.000	0.002	0.004	0.007	0.015	0.028
-34.0	0.000	0.001	0.003	0.005	0.012	0.023

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