MASQATI, Mohomed Sunderjee, 1945-
A MATHEMATICAL MODEL FOR OPTIMAL WASTE LOAD ALLOCATIONS.

The University of Oklahoma, Ph.D., 1974
Engineering, civil

THE UNIVERSITY OF OKLAHOMA

GRADUATE COLLEGE

A MATHEMATICAL MODEL FOR OPTIMAL WASTE LOAD ALLOCATIONS

A DISSERTATION

SUBMITTED TO THE GRADUATE FACULTY

in partial fulfillment of the requirements for the

degree of

DOCTOR OF PHILOSOPHY

BY

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Norman, Oklahoma

1974

A MATHEMATICAL MODEL FOR OPTIMAL WASTE LOAD ALLOCATIONS

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ABSTRACT

River basin planning is the first and most important step toward the water quality management of a region. Various alternatives are available to the planners. Treatment of wastes before discharge is one of the most effective methods of water pollution control. This method is easy to implement, and a water pollution control program based upon it is convenient to administer. Such a program generally consists of waste load allocations where the assimilative capacities of streams in a river basin are allocated to individual waste dischargers for the disposal of their wastes, in such a fashion as to maintain a required level of quality in the stream waters.

In the past, the quality of stream waters has been analyzed by simulating several water quality parameters. The simulation models applied are generally large and complex and require a large amount of input data. The planners need a model that is regional in nature, is capable of working with essentially inadequate data, and requires a minimum of man and computer time. Such a model was developed by Professor George Reid and others. This model is employed to analyze the quality of stream water in the Verdigris River basin.

A linear programming model was developed for making the optimal waste load allocations. The objective of the model is to minimize the total cost of waste treatment to a region. The costs of waste treatment are determined by the models developed by Shah and Reid. These models provide a basis for comparing various allocation programs in terms of the cost incurred to construct the required waste treatment facilities. The linear program of the optimal waste load allocation model is solved using the Mathematical Programming System.

A sensitivity analysis is performed by varying the value of the "judgement" parameter k_2. The changes in k_2 produce changes in the allocations of waste loads. A higher value of k_2 leads to a program of allocations with a lower degree of treatment required.

The results of the present waste load allocation program are compared to the allocations made previously under the "Comprehensive River Basin Planning" project. The results indicate that the present method of waste load allocations is superior because it takes a less amount of data and man and computer time to apply to a river basin.

ACKNOWLEDGMENTS

I wish to thank Professor George W. Reid, the chairman of the doctorate committee, for his sustained interest, effort and assistance to guide and complete this work. It has been my pleasure to work with Professor Reid. His knowledge, experience and insight have benefited me greatly in learning and working on various projects.

I also had the pleasure of working with Dr. Leale E. Streebin on the "Comprehensive River Basin Planning" project. The foundation for this dissertation was laid in that project. His support and guidance have been most essential for this work. I thank him for his most useful comments and suggestions for this dissertation.

I fondly acknowledge the friendship I have received from Dr. Arthur Bernhart, Dr. James Robertson and Dr. Antti Talvitie. I thank them for serving on the doctorate committee.

My parents have made sacrifices for me and others in the family. My success is due to them and I take pride in dedicating this work to them.

I also thank Ms. Bobbie Everidge for the wonderful job she has done on typing this dissertation.

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CHAPTER I

INTRODUCTION

A large amount of pollution of streams results from man's activities in the home, in the factory or industrial plant, and on the farm. Man's activities are the producers of waste. Human fecal matter no longer constitutes the bulk of pollution. Technology is fast advancing and new products are developed every day. Thus with the growth of industry, the sources and complexity of wastes are expected to increase.

Pollution of rivers and lakes is a major threat to the economic use of our water resources. Pollution endangers public health and esthetics. A direct result of pollution is the significant increase in the costs of subsequent utilization of polluted water. Gross organic pollution may threaten the existence of aquatic life. Several alternatives can be presented to combat pollution.

1. Treatment of water containing wastes (water treatment plants).

2. Treatment of wastes before discharge (waste treatment plants).
3. Complete elimination of a waste at the point of origin.
4. Abandoning the use of pollution causing substances (such as insecticides).
5. Abandoning the subsequent use of the water receiving a waste.

Depending on the circumstances, one or more of these alternatives are put into practice. Economics of water supply and demand play a major role in this kind of decision. If an inexhaustible supply of clean water was available, fighting water pollution would be easier; however we are faced with a relatively fixed or declining supply of water and concern is voiced by Professor George Reid (45).

> Pollution can be just as effective in reducing or eliminating a water resource as a drought or consumptive withdrawl. Thus, water quality management is the key to reuse and water available in the future for all needs. The multiple use and reuse of water results in a high degree of interaction between users (pollutors) within a common basin, making it difficult to assess individual responsibility for the ultimate or total damage to the stream.

Combating water pollution is a part of the conservation movement that embraces the totality of man's environment, focusing on pollution, population, ecology and the urban environment. A deteriorating environment has awakened the citizens to a curiosity, if not a concern, in ecology, the ecosystem, the biosphere, and the environment as a whole. Ecology is the science of the intricate web of relationships between living organisms and their living and non-living surroundings. These interdependent living and non-living parts make up ecosystems. A stream is a good example of an ecosystem.

Water pollution was the first, among different types of pollution, to draw serious concern from the federal government. U.S. Congress enacted the "Rivers and Harbors Act of 1899" which prohibited the discharge of any kind of solid refuse matter into the navigable waters of the United States. Since that early statute, the regulation of water polluting activities has become more comprehensive and sophisticated. The first comprehensive legislation on water pollution control was passed by the Congress in 1948. Additional laws have been adopted since in 1952, 1956, 1961, 1965, 1966 and 1970. In 1970, President Nixon signed into law the "National Environmental Policy Act" which created the "Council on Environmental Quality" and the "Environmental Protection Agency." The following abstract from the U.S. Congress records (58) shows the determination of the U.S. Congress and the Federal Government to eradicate water pollution.

> United States Congressional Policy contemplates the enhancement of the quality and value of our national water resources through the prevention, control, and abatement of water pollution. It is also federal policy to recognize and preserve the primary responsibilities and rights of the states in preventing and controlling water pollution through aid to them in technical research, services and financing. Grants are available to state and local governments to assist in the development of projects which demonstrate new or improved methods of controlling the discharge into any waters of untreated sewage, and development of advanced waste treatment and water purification methods.

The Federal Water Pollution Control Act Amendments of 1972 have established a time table to abate water pollution and in accordance, the Environmental Protection Agency has set guidelines to achieve best practical treatment by 1977, best available treatment by 1983, and no pollution by 1985. All of this has provided an enormous impetus to water quality management.

Purpose and Scope of the Present Study

Any attempt at water quality management on a local or fragmented basis is not only expensive but futile. Such an attempt defeats its very purpose of water pollution control. A coordinated effort on a regional basis is the key to effective water quality management. Allen Kneese recognizes three aspects of the water quality management (23):

1. Determination of the quality of water to be maintained in the waterways. (We can denote this aspect as "Goals.")
2. Devising an optimal system of technical management measures to achieve a specified pattern of water quality. This must be done within the context of overall water resources management. (We can denote this aspect as "Technology.")
3. Making optimal institutional or organizational arrangements for managing water quality. (We can denote this aspect as "Legal Management.")

All these aspects are related, forming steps in the water quality management. The choice of quality levels must depend on the costs of available, practical technology of achieving those levels, and the costs largely depend on how effective the management agency is.

Though the three aspects are interrelated, each aspect can be examined in isolation. It is an excessively huge task to study all the aspects in one study! As a matter of fact, the majority of the studies in the regional water quality management field have attempted to solve one or two of the issues involved! Any attempt at examining the whole spectrum of water quality management of a region has led to a very general discussion of the issues which is not very useful to planners who wish to solve specific problems.

The author has had the benefit of a year's experience working with the Department of Pollution Control, State of Oklahoma and Environmental Protection Agency on the "Comprehensive River Basin Planning for the State of Oklahoma" project under the guidance of Dr. Leale Streebin at the Bureau of Water and Environmental Resources Research, University of Oklahoma. The planning group felt a strong need for an approach or a planning model to deal with particularly the second aspect of water quality management within the State of Oklahoma. A search of the literature showed that little work existed that could meet the requirements of the project. The state agencies are without a comprehensive planning approach. Environmental Protection Agency is not without this need. Recently, it sent out a request for proposals to develop a management model for optimal waste load allocations (46).

This study, thus, is the result of the need felt by the State water pollution control agencies and E.P.A. It is the purpose of this study to deal with the development of an optimal system of technical management measures to achieve a stated quality of waters in a given region.

CHAPTER II

RIVER BASIN PLANNING

River and stream waters were first used for the removal and transportation of municipal wastes in southwest Asia at the dawn of history. After some 5000 years of human history and engineering analysis, we continue to use our natural waterways for the disposal of wastes. Every water course has a self-purification capacity or a waste assimilative capacity and it restores its own quality over a period of time. It is this valuable asset that man has used to his advantage. However, this self-purification potential is limited and problems occur when the wastes are increasingly discharged in excess of the waste assimilative capacity of the streams.

The self-purification process is complex and involves many physical, chemical and biological subprocesses, including sedimentation of suspended matter, coagulation of colloids, precipitation and absorption of organic and inorganic dissolved substances, and the life processes of aquatic organisms such as respiration, growth, reproduction and death. The biochemical processes in a stream polluted by wastes occur as the wastes are transported downstream. Saprophytic bacteria decompose organic molecules of large dimensions and use the energy of the reaction for growth, locomotion and reproduction. Both aerobic and anaerobic bacteria operate in the natural purification process. Aerobes require free oxygen for their life processes whereas anaerobes do not. When an ecosystem, such as a river, is overloaded with organic wastes, the supply of dissolved oxygen may be exhausted. As the anaerobic zone expands, aerobic organisms die or emigrate and waste decomposition and stabilization proceed at much slower rates. Offensive or toxic substances such as hydrogen sulfide may be produced and the entire process is retarded until more oxygen is made available.

Types of Waste Discharges (61)

The five main types of wastes discharged into receiving streams are: (1) organic; (2) microbial; (3) radioactive; (4) inorganic; and (5) thermal.

Organic wastes constitute by far the major stream pollution problem. Much of the urban and industrial wastes comprise of unstable organic matter subject to decay. The agricultural and natural pollution also contributes organic fractions and in aggregate they are large but widespread, whereas the urban industrial organic loads are concentrated.

The microbial wastes of primary concern in stream sanitation are bacteria, viruses and other pathogenic organisms. The major sources of microbial wastes are community sewerage, storm water drainage and wash waters from the urban areas. Some industries, depending on the source and kind of raw product processed, may contain microbial contaminants in their waste waters. Agricultural sources, particularly from the livestock production, are also contributors of microbes.

Radioactive wastes are generally rigidly controlled at the source. The increasing use of radioactive tracers in industry and research has increased the danger of stream pollution by the radioactive materials. Increasing replacement of the fossil fuels by the atomic fuels has posed new problems in stream sanitation.

Inorganic wastes have numerous sources: urban, industrial, agricultural and natural. Inorganic wastes are in the form of dissolved, colloidal and suspended matter. Inorganic wastes are relatively stable and do not decompose or decay. Storm drains and combined sewers usually discharge large quantities of inert, suspended matter which forms deposits in the streams. The largest portion of inorganic materials is discharged by the chemical industries. The chlorides, for example, are persistent, cumulative, and resistant to treatment. The agricultural and natural sources, although widespread, contribute large quantities of inorganics from land erosion and residual agricultural chemicals.

Heat or thermal wastes are almost entirely associated with the industrial sources, primarily the electric utility industry. The steam electric power plants require large quantities of condenser water, resulting in enormous amounts of waste heat loads. To give an example, a 4000 MW generating station would produce a thermal waste load of 17.6 billion Btu/hr at full capacity. This would require about 2 million gallons/minute of circulating condenser water.

Although the above classification of the types of wastes is essential, the most useful classification of wastes distinguishes between those that are nondegradable or conservative and those that are degradable or nonconservative.

Nondegradable wastes are usually diluted and may be changed in form, but they are not appreciably reduced in weight in the receiving water. They are mainly composed of inorganic chemicals such as chlorides, synthetic organic chemicals, and inorganic suspended solids.

Degradable wastes are reduced in weight by the biological, physical, and chemical processes which occur in natural waters. They include organic wastes of various kinds from domestic and industrial operations, bacteria, and thermal discharges.

Actually, the classification into degradable and nondegradable wastes is a simplification (23). Some types of substances, such as radioactive materials and some of the organic chemicals that appear to be inert are degradable to some degree. Viruses appear to be in the in-between category. Some radioactive materials decay rapidly while some extremely slow. Some of the synthetic organic compounds, such as alkyl benzene sulfonate (ABS) detergents, are not strictly non-degradable, but they resist attack by stream biota. On the other hand, the new detergents based on linear alkylate sulfonate (LAS) are degradable, both in streams and waste treatment plants.

Waste Assimilative Capacity Models (11)

The movement and reactions of waste materials through streams are a result of hydrodynamic transport and biological and chemical reactions by the biota, suspended materials, plant growths, and bottom sediments. These relationships can be expressed by mathematical models that reflect various inputs and outputs in an aquatic system.

Considering the oxygen balance, the general relationships for the "oxygen sag curve" are expressed as (11)

$$\frac{\partial c}{\partial t} = \varepsilon \frac{\partial^2 c}{\partial x^2} - U \frac{\partial c}{\partial x} \pm \Sigma S \qquad \text{(II-1)}$$

where
- c = concentration of dissolved oxygen
- t = time at a stationary point
- U = velocity of flow in the x direction
- ε = turbulent diffusion coefficient
- S = sources and sinks of oxygen
- x = distance downstream

The above model assumes that the concentration of any characteristic is uniform over a stream cross-section and that the area is uniform with distance.

The sources and sinks of oxygen can be listed as follows:

Sources of Oxygen

1. Dissolved oxygen in the incoming or tributary flow
2. Photosynthesis.
3. Reaeration.

Sinks of Oxygen

1. Biological oxidation of carbonaceous organic matter.
2. Biological oxidation of nitrogenous organic matter.
3. Benthal decomposition of bottom deposits.
4. Respiration of aquatic plants.
5. Immediate chemical oxygen demand.

<u>Oxygen Sources</u>

The degree of photosynthesis depends upon sunlight, temperature, mass of algae, and available nutrients. The photosynthesis exhibits a diurnal variation. Oxygen is also added to the water body by the natural reaeration process. Reaeration is primarily related to the degree of turbulence and natural mixing in the stream.

The oxygen transfer from air to the water surface is generally expressed as

$$\frac{dc}{dt} = k_2 \, (c_s - c) \qquad \text{(II-2)}$$

where k_2 = reaeration coefficient

c_s = oxygen saturation concentration

c = oxygen concentration

Oxygen Sinks

In the biological oxidation of carbonaceous organic matter, the rate of removal, k_1, is related to amount of unstabilized organics present, as

$$L = L_o e^{-k_1 t} \qquad \text{(II-3)}$$

where L = concentration of organics present at time t

L_o = concentration of organics present at time zero

t = time

k_1 relates to the removal of organics by oxidation alone. A composite coefficient relating to the sedimentation, oxidation, and volatilization can be substituted for k_1 to represent more nearly the processes taking place in a stream. The rate of oxidation of the organics in a BOD bottle is usually less than k_1 because longitudinal mixing, the presence of bottom growths, and suspended biological solids increase the reaction rate. Values of k, the BOD bottle coefficient, depend upon the characteristics of the waste. It decreases with treatment or removal of readily oxidizable organics. The range of k is generally from 0.10 to 0.60/day while k_1 may have values in excess of 20 per day!

When unoxidized nitrogen is present in the wastewater, nitrification results with the passage of time or distance downstream. Nitrifying organisms (Nitrosomonas and Nitrobacter) are sensitive to pH and function best over a pH range of 7.5 to 8.0. The rate of nitrification decreases rapidly at dissolved oxygen levels below 2.0 to 2.5 mg/L, so that at low oxygen levels in the waterbody little or nc nitrification occurs. Denitrification has been observed to occur in stretches of zero or near zero oxygen concentration.

The usual operational model for a stream nitrification process may be represented as

$$N = N_o e^{-k_n t} \tag{II-4}$$

where N = concentration of nitrogen at time t

N_o = concentration of nitrogen at time zero

k_n = nitrification coefficient

Oxygen depletion due to nitrification lags the deoxygenation from carbonaceous organics. In the effluents of secondary sewage treatment plants, the quantity of carbonaceous organics is greatly reduced, but much larger numbers of nitrifying organisms are present. Under these condition, nitrification is more rapid and may exert a significant oxygen demand.

BOD-Oxygen Sag Model

When considering streams, the turbulent diffusion, i.e., longitudinal mixing, is generally insignificant. Under steady state conditions and assuming only deoxygenation by organic matter oxidation and natural reaeration, Equation II-1 can be rewritten as

$$U \frac{dc}{dx} - k_1 L + k_2 (c_s - c) = 0 \qquad \text{(II-4)}$$

Through mathematical manipulations we obtain

$$c = c_s \frac{k_1 L_o}{k_2 - k_1} [\exp (-\frac{k_1}{U} x) - \exp (- \frac{k_2}{U} x)]$$
$$- (c_s - c) \exp (- \frac{k_2}{U} x) \qquad \text{(II-5)}$$

The critical points, D_c (critical deficit of dissolved oxygen) and t_c (time to reach critical DO level), are often of significance.

$$D_c = \frac{k_1}{k_2} L_o \cdot 10^{-k_1 t_c} \qquad \text{(II-6)}$$

$$t_c = \frac{1}{k_2 - k_1} \log \frac{k_2}{k_1} \left(1 - \frac{D_o (k_2 - k_1)}{k_1 L_o} \right) \qquad \text{(II-7)}$$

The role of photosynthesis in the dissolved oxygen balance of receiving waters is a complex one. The contribution of photosynthesis to the oxygen content is complicated by the problem of respiration. A further complicating factor with respect to organic waste and photosynthesis is the diurnal variation in dissolved oxygen. The critical dissolved oxygen deficit point is an important factor in planning or designing a waste treatment facility.

River Basin

A river or drainage basin is the entire area drained by a river or a system of connecting streams such that all streamflow originating in the area is discharged through a single outlet. Multiple channels through alluvial delta deposits constitute a single outlet. The basin is necessarily completely bounded by a "divide" which separates it from adjacent basins. The divide follows the ridge line around the basin, crossing the stream only at the outlet point. It marks the highest points between basins, but isolated peaks within a basin may reach greater elevations than any point on the divide.

Streams are commonly classed into three types (29):

1. Perennial or continuous streams.
2. Intermittent streams.
3. Ephemeral streams.

Perennial streams contain water at all times and receive their low-water flow from groundwater. Intermittent streams carry water most of the time but cease to flow occasionally because evaporation and seepage into their bed and banks exceed the available streamflow. Ephemeral streams carry water only after rains or periods of snowmelt. They are above the groundwater table at all times.

For use on the "Comprehensive River Basin Planning" project, following classification of streams was adopted:

1. Return flow - special mixing zone streams.
2. Perennial streams.

The return flow streams are defined to be those streams that are composed entirely of waste effluents during the critical flow conditions. A special mixing zone stream is defined by the Oklahoma Water Quality Standards (40) as a stream in which the combined stream and waste flow is less than four times the waste discharged to the stream. Perennial streams are the streams in which the combined stream and waste flow is greater than four times the waste discharge into them.

Stream Channel Characteristics

Streams vary radically in configuration, cross-section, depth, and hydraulic gradient. The smaller streams, tributaries and creeks generally travel through rugged terrain descending in irregular gradients. In contrast, the larger, main channel rivers that generally travel through the lower portions of the drainage basin, descend gradually in regular, flatter gradients.

In the calculation of stream assimilative capacity, three relevant channel parameters are considered reach by reach. They are: (1) the occupied channel volume; (2) the surface area; and (3) the effective depth. From these parameters are calculated two additional parameters: (1) time of travel and (2) mean velocity. It is beyond the scope of this paper to describe the practical methods of measuring these parameters in field. Interested readers should refer to "A Practical Guide to Water Quality Studies of Streams."

For use on the "Comprehensive River Basin Planning" project, Oklahoma Water Resources Board provided data on the average width and depth of streams in Oklahoma. A stream cross-section is generally very irregular and varies over the length of the stream. It is not possible to determine the cross-section reach by reach for each stream; instead a geometric shape is assumed for the ease of computations. In previous studies, a wide rectangular or triangular shape has been used usually. Other more difficult channel sections have been used. A parabolic section approximates the natural stream section best. Chow (6) states that the parabolic section is the one most used as an approximation of sections of small and medium sized natural stream channels.

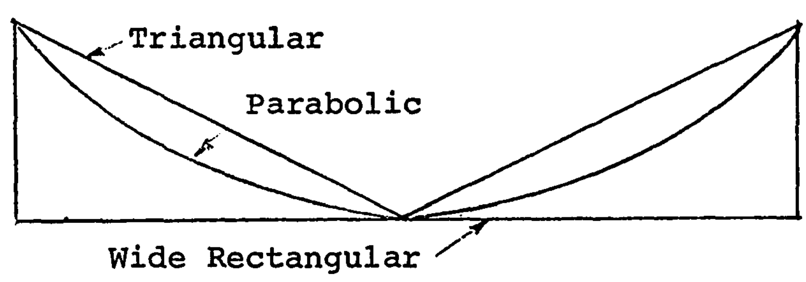


Figure II-1. Stream Channel Cross Sections.

A quadratic form of the equation is most suitable for describing the section. The general form of the equation is

$$y = a\,x^2 \tag{II-8}$$

where y = depth of the channel

x = width of the channel

a = cross sectional constant

If "H" is the depth and "d" the width of the channel, then the area of the channel section is given by:

$$A = \frac{2}{3} \cdot a \cdot d^3 \tag{II-9}$$

The hydraulic radius of the stream channel section is given by:

$$R = \frac{2}{3} \cdot a \cdot d^2 \tag{II-10}$$

Velocity of flow in a stream depends greatly upon the stream channel characteristics. Manning's model is a popular one for calculating velocity.

$$V = \frac{1.49}{n}\,[R^{2/3} \cdot S^{1/2}] \tag{II-11}$$

where S = slope of the channel bed

n = Manning's roughness coefficient

For a stream with cross sectional constant "a" and average width "d", the velocity of flow is given by

$$V = \frac{1.49}{n} [(2/3)^{2/3} \cdot a^{2/3} \cdot d^{4/3} \cdot s^{1/2}] \qquad \text{(II-12)}$$

If Q is the flow in a stream, then

$$Q = \frac{1.49}{n} [0.509 \cdot a^{5/3} \cdot d^{13/3} \cdot s^{1/2}] \qquad \text{(II-13)}$$

Time of travel between any two points is determined by dividing the distance between the two points by the average velocity of flow between the two points.

Stream Flow

Quantitative determination of runoff along the course of a stream is based on the records of streamflow. Some drainage basins are homogeneous so that runoff along the course is directly proportional to the tributary drainage area. Other drainage basins are composed of heterogeneous areas of varying flows and the estimation of streamflow is rather complex.

Critical Flow Characteristics

Where urban and industrial development relies on natural, unregulated stream runoff, the character of critical flow or low flow is the restricting element in growth potential. Any level of development carries a risk of uncertainty of severity of low flow. The levels of community and industtrial development, the required degree of wastewater treatment and downstream water quality objectives, all rely on the critical flow characteristics of a region.

Low flow characteristics at a gaging station are described by frequency curves of annual or seasonal minimum flows, by duration curves, and by base flow recession curves. Estimates of low flow characteristics at ungaged sites are generally quite inaccurate because the low flows are highly dependent on the lithology, the structure of rock formations, and on the amount of evapotranspiration, none of which has been adequately described.

Low flow is generally expressed in terms of a recurrence interval and a period of a number of days. The Environmental Protection Agency has recommended the use of a recurrence interval of two years and a period of seven days for river basin planning purposes. Riggs (49) in "Low Flow Investigations" describes the method of estimating low flows at gaging stations. This method was followed for the low flow calculations for the river basin project mentioned earlier. A model was developed to predict the low flows at ungaged sites.

$$Q_t = Q_g \cdot \left(\frac{A_t}{A_g}\right)^p \qquad \text{(II-14)}$$

where Q_t = low flow in a tributary without a gaging station

Q_g = low flow at a gaging station in appropriate neighborhood of the tributary under study

A_t = drainage area of the tributary

A_g = drainage area above the gaging station

p = low flow index

'p' is a complex function of lithology, structure of rock formations, and the amount of evapotranspiration. 'p' is also affected by the recurrence interval and the number of days. A search of the literature yielded values of p ranging from 0.2 to 1.2!

River Basin Planning

River basin planning is the first and most important step toward the water quality management of a region. The purpose of this paper is to devise a fair and scientific system of management measure for a region or a river basin. Central in the study is the abatement of water pollution with an economically optimal system of management in a river basin. The situation in a river basin can be depicted by a highly simplified diagram as follows.

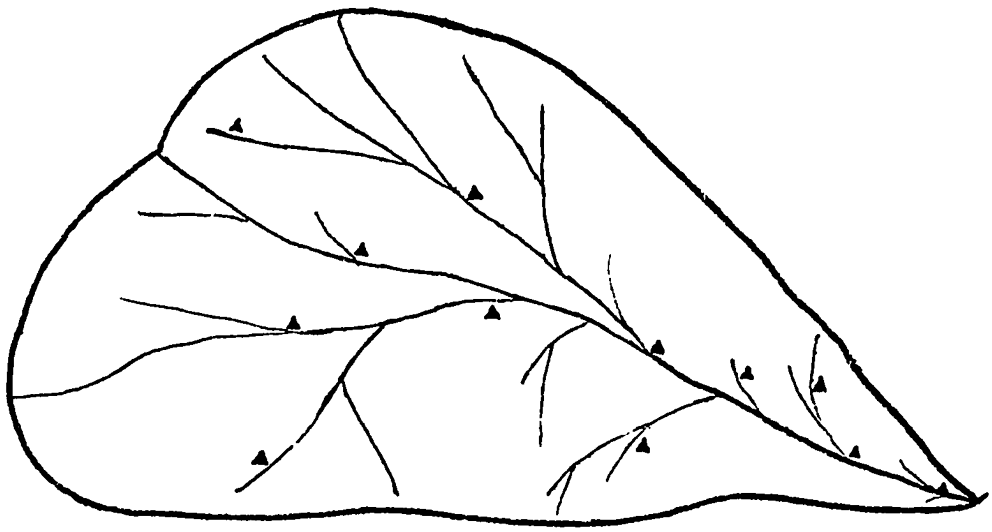


Figure II-2. A River Basin with Waste Discharges.

There are a number of industrial and municipal waste treatment plants that discharge into a river system at various points in the basin. Various alternatives are available to abate the problem of water pollution in a river basin, some of which are:

1. Reduction of waste discharge - process change.
2. Alteration of wastes after generation - waste treatment, material recovery.
3. Flow augmentation.
4. Reservoir mixing.
5. Stream reaeration.
6. Regional plants - collective treatment.
7. Effluent redistribution.
8. Regulated discharge
9. Multiple outlets from reservoirs.

Essentially, river basin planning is the analysis of these alternatives and development of a management system comprising one or more of these alternatives. The biggest challenge in the planning process arises from the lack of complete data! Not only is the available data inadequate, but there is also a factor of uncertainty involved in it. A program to collect an adequate amount of precise data is almost an impossible task! Added to this is the probabilistic nature of events in a river basin.

The Alternatives

The alternatives enumerated above fall into one of two categories (23):

1. Reduction of waste loads discharged to streams.
2. Increasing or making a more effective use of the assimilative capacities of streams.

Improving the quality of receiving waters by reducing waste loads can be accomplished in two ways:

1. Reducing the generation of wastes.
2. Modifying the residual wastes.

Methods of reducing waste generation include:

1. Changing the input raw materials.
2. Changing the production process.
3. Changing the output products.
4. Recirculation of water in plant.

Methods of reducing wastes after generation include:

1. Recovery of materials.
2. By-product innovation.
3. Waste treatment.
4. Effluent reuse - ground water recharge; waste water reclamation or renovation.

It is clear that a government planning agency has little jurisdiction over the implementation of any one of these alternatives. Individual waste discharger adopts one of the methods of reducing wastes based upon the economics of his operations. However, a regulatory agency can require an individual waste discharger to treat his wastes to a certain level before discharging them to a waterbody. Therefore, the treatment of wastes after generation shall be explored further.

Four types of waste treatment are generally recognized:

1. Preliminary treatment.
2. Primary treatment.
3. Secondary treatment.
4. Tertiary treatment, advanced treatment.

Preliminary treatments include those processes which do not significantly reduce the pollutional strength of a waste but do serve to protect or prepare the waste for subsequent treatment by altering the waste characteristics. Coarse screening, grit removal, comminution and preaeration are common preliminary treatment processes.

Primary treatments include those processes that reduce the floating and suspended solids present in wastes by mechanical means or by the action of gravity. Fine screens and sedimentation tanks are common primary treatment processes.

Secondary treatments utilize biological processes to reduce suspended and dissolved solids. Trickling filter and activated sludge processes are typical secondary treatment processes.

Prior to 1965, primary and secondary processes were used to produce a plant effluent for discharge and disposal. This was supplemented occasionally by use of additional processes such as disinfection or intermittent sand filtration.

It has become evident now that municipal and industrial wastes contain many substances that are resistant to or unaffected by the conventional waste and water treatment processes. Such substances, called refractory substances, include both organic and inorganic materials. Such wastes require advanced or tertiary treatment processes such as adsorption, electrodialysis, extraction, foaming and ion exchange.

In addition to treatments described above, a number of other processes are used for the disposal of sludges from both primary and secondary treatment processes. These include sludge concentration, digestion, filtration, drying and incineration.

Methods of increasing or making a more effective use of the assimilative capacity of streams include (23):

1. Flow augmentation.
2. Multiple outlets from reservoirs.
3. Reservoir mixing.
4. Stream reaeration.
5. Effluent redistribution - regulated discharge.

Low streamflows generally coincide with high temperatures, and this phenomenon places a great burden on the stream that receives a significant amount of waste discharges. The most common practice of increasing the streamflow during the low flow periods is by controlled releases from reservoir storage. Flow can also be increased by withdrawing water from groundwater sources or lakes and releasing it into water courses. The effectiveness of augmenting flow depends on both the type of waste and the type of receiving water involved.

The U.S. Corps of Engineers has been engaged in planning, construction and operation of a multitude of projects involving flow augmentation. The effects of reservoir storage on the water quality may be favorable and/or unfavorable. It is learnt that the bacteriological quality of the water is stabilized due to storage. This improves the quality of water impounded. However, releases from deeper parts of reservoirs are not adequate for effective dilution because the water in deeper parts of a reservoir is, often, devoid of oxygen. This phenomenon is due to the combined effect of the biochemical oxygen demand and reservoir stratification.

There are several ways to handle this situation. Multiple outlets are installed on the dam so that water can be released from different levels of a reservoir in various combinations to achieve a desired water quality. Another method is reservoir mixing which prevents stratification of the reservoir waters.

Artificial aeration of streams with either air or oxygen is an effective way to increase the assimilative capacity of streams. Mobile or fixed aerating devices can be installed in reaches of streams where needed to prevent anaerobic conditions from occurring.

Effluent redistribution is the method where effluents from a plant are taken to a distant point on a stream where stream conditions are more favorable for waste discharges, instead of a nearby point on the stream where the stream conditions do not permit the discharge of wastes. This is due to the fact that a stream may be sluggish and shallow on some upstream reaches but becomes rapid with a good amount of flow downstream. Thus, a plant located on an upstream reach may be required to pipe its wastes to a distant, downstream point for discharge.

Regulated discharge is the temporary storage of waste effluents for discharge at some more favorable flow conditions in a stream. This procedure is used by a number of food processing plants, pulp and paper industries, and petroleum refineries.

Recognizing the various alternatives available for a water quality management system and making a plan to optimally combine them is an extremely difficult task. Planning and implementation are complicated by the fractionation of decision making responsibility for various possible components of such a system. Some measures are within the purview of individual water users and waste dischargers alone, as noted earlier, while other measures fall under the jurisdiction of one or more state, regional or federal agencies. This often leads to confusion and conflict. Some measure may not be under the jurisdiction of any agency!

The operation of dams and reservoirs falls under the authority of the Corps of Engineers or sometimes some specific river dam authority. None of these is generally in charge of water pollution control. It is the experience of the author that, generally, a state water pollution control agency, engaged in the river basin planning, has only one option to control pollution of streams - that of requiring the individual waste dischargers to treat their wastes to certain levels through a waste discharge permit program!

Therefore, in this paper, for the river basin planning process, the alternative of treatment of wastes shall be adopted. It is the purpose, then, of this paper to develop a mathematical model to determine the levels of treatment for each individual waste discharger in a river basin, in such an optimal fashion that the total cost to the region for treatment of wastes is minimized.

CHAPTER III

WATER QUALITY PREDICTION MODELING

Water quality is an important consideration in the planning for a river basin. A planner needs to have at his disposal a systematized procedure for simulating water quality changes in both time and space. The quality of a watercourse changes continuously due to the various uses of water. The changes caused by one use may make the water unsuitable for another use! The relative locations of various users is also an important factor in determining the effects of water polluting substances discharged to a stream. A mathematical model of water quality changes becomes necessary to evaluate effectively the water quality patterns in a river basin.

Water Quality Data (31)

Quality of a river water is measured by a number of water quality indices. Sporadic measurements of these indices are insufficient for water quality analysis because these indices can fluctuate in their values greatly over a short time interval. Through mathematical formulations of physical, biological, and chemical processes occurring in the aquatic ecosystem, the spatial and temporal variations of water quality indices can be simulated. A continuous analysis of a river system is necessary to evaluate the fitness of water for a particular use; however for most water quality indices this task is prohibitively expensive! Thus, water quality modeling can be seen to complement data collection programs in determining the usefulness of a river system.

The simulation model that the planner wishes to use should be capable of representing changes in several parameters of water quality as they are influenced by natural and human factors impinging on the hydrologic system of a river basin. Several parameters of water quality are important and they have been modeled extensively. It is not possible to represent water quality through any one single physical, chemical or biological parameter. A number of water quality parameters must be used jointly. A basic list of parameters would include the following:

1. Dissolved Oxygen Content (DO)
2. Biochemical Oxygen Demand (BOD)
3. Chemical Oxygen Demand (COD)
4. Fecal Coliforms
5. Total Dissolved Solids (TDS)
6. Ammonia
7. Chlorophyll _a_
8. Chloride
9. Sulphate
10. Nitrate

11. Phosphate
12. Turbidity
13. Alkalinity
14. pH
15. Temperature
16. Fecal Streptococci
17. Zooplankton, etc.

Additional water quality indices such as heavy metals, pesticides, etc., may be required for special purposes.

Review of the Existing Water Quality Models

A mathematical model of a river system generally consists of a series of elements, each corresponding to a discrete stream segment. These segments are so arranged that the output from one segment or element is the input to the next. The transfer function is determined by performing a mass balance of a given water quality parameter over a time interval on a segment of the river, along the length of the river. The mathematical model is generally a partial differential equation, which is often replaced by a set of ordinary differential equations with time as the independent variable. The solution to these equations is taken as the solution to the partial differential equations at points 'dx' apart. A significant problem is to determine the closeness of the solution of the ordinary differential equation to the solution of the partial differential equation, the spacing, and the number of sections (26).

A comparison of water quality models should be based on several capabilities of the models. Desirably, a model should be capable of:

1. Simulating the desired water quality parameters.
2. Simulating an entire region or a river basin.
3. Simulating over a period of time.
4. Producing a desired output.
5. Responding to the dynamic nature of a river basin.
6. Operating at a low cost.
7. Operating with essentially inadequate data.
8. Accounting for the probabilities and uncertainties involved.
9. Minimizing the number of 'judgement' parameters.

An ideal model would have all the capabilities listed above. However, such an ideal model does not exist yet! Elaborate models of river basins tend to be complex and expensive to operate. Also, the amount of input data becomes very large. A program to collect such data would be prohibitively expensive! On the other hand, the simple models are restricted in the number of quality parameters they can simulate. Also, often the output from such models is not the desired one. With this background, following models were reviewed for this paper.

QUAL-1 Mathematical Modeling System (53), developed by the Texas Water Development Board, is designed to simulate water quality parameters, and is one of several simulation systems being developed by the Board to assist in more refined water planning and management. QUAL-1 was extensively tested for use during the initial stages of the "Comprehensive River Basin Planning" project. QUAL-1 is a fair representative of the simulation models in the water quality field. Therefore, a detailed review of this modeling system is in order.

The primary objective of the modeling system was to develop a set of interrelated water quality models capable of routing the following water quality parameters through a stream subsystem:

1. Temperature.
2. Biochemical oxygen demand (BOD) and dissolved oxygen (DO).
3. Conservative minerals.

The modelers believed that this objective would be best served by structuring separate models for each quality parameter and then coupling these models into an "integrated system" simulation package.

The basic equation describing the mass transport of conservative and nonconservative constituents, for a stream or canal segment, assuming steady state, nonuniform flow, was represented as

$$A \frac{\partial c}{\partial t} = \frac{\partial (A D_L \frac{\partial c}{\partial x})}{\partial x} - \frac{\partial (A \bar{u} c)}{\partial x} \pm A \text{ "s"} \qquad \text{(III-1)}$$

where A = cross sectional area of the stream, ft^2

c = concentration of the constituent, mg/L or temperature, °F

$\bar{u}$ = mean velocity of the stream, ft/sec

D_L = longitudinal dispersion coefficient, ft^2/sec

t = some point in time, sec

x = some point along the longitudinal axis of the stream, ft

"s" = sources or sinks of a nonconservative constituent, mg/L or temperature, °F

The first term in the above equation represents the temporal change in concentration, the second term represents the transport due to longitudinal dispersion, the third term represents the transport due to longitudinal advection, and the fourth term represents the sources or sinks if the constituent is nonconservative.

The term "dispersion" is generally used for transport associated with spatially-averaged velocity variation, as opposed to "diffusion" which is reserved for transport that is primarily associated with time averaged velocity fluctuations. Elder developed the following expression for the dispersion coefficient as

$$D_L = 5.93\ D\ u^* \qquad \text{(III-2)}$$

where D = mean depth of the channel

u^* = steady-state open channel flow, given by

$$u^* = c\ \sqrt{R\ S_e} \qquad \text{(III-3)}$$

where c = Chezy's coefficient

R = the hydraulic radius

S_e = the slope of the energy grade line

Chezy's coefficient is given by

$$c = \frac{R^{1/6}}{n} \tag{III-4}$$

where n is the Manning roughness coefficient. An expression for the slope of the energy gradient is given as

$$S_e = \left(\frac{\bar{u}\ n}{1.486\ R^{2/3}}\right)^2 \tag{III-5}$$

where symbols represent the variables as stated before.

Substituting Equations III-3, -4 and -5 into Equation III-2, and assuming R = D for a wide channel yields the expression

$$D_L = 22.6\ n\ \bar{u}\ D^{0.833} \tag{III-6}$$

Manning roughness coefficient for natural river channels varies from 0.025 to 0.030 for clean and straight channels to from 0.075 to 0.150 for very weedy, winding and overgrown channels. The dispersion coefficient varies for flumes and small streams from 3×10^{-2} to 3 ft^2/sec and for large rivers from 3 to 3×10^{2} ft^2/sec.

The most important consideration in determining the waste-assimilative capacity of a stream is its ability to maintain an adequate dissolved-oxygen concentration. The authors felt that the most accurate oxygen balance would consider all significant factors. But for their work they thought that

> many of the factors are very difficult, if not impossible to define accurately; and unless unusual conditions are present, fairly reliable predictions of the 'self-purification process' of a water body can be obtained through simulation of the simultaneous processes of reaeration (natural or artificial) and deoxygenation as measured by the biochemical oxygen demand.

The reaeration process was expressed as

$$\frac{\partial D}{\partial t} = -k_2 \ (c_s - c_t) \qquad \text{(III-7)}$$

where $D = (c_s - c_t)$ oxygen deficit, mg/L

k_2 = reaeration coefficient, 1/days

c_s = solubility of oxygen in water, mg/L

c_t = existing concentration of oxygen at time t, mg/L

The solubility of oxygen in water is primarily dependent upon temperature, pressure, and the concentration of dissolved salts. At standard pressure, the solubility of oxygen in water can be given by

$$c_s = 24.89 - 0.426T + 0.00373T^2 - 0.0000133T^3 \qquad \text{(III-8)}$$

where T is the temperature of water in °F. For elevations less than 2000 feet, the variation in the solubility of oxygen in water due to the atmospheric pressure can be given as

$$c_s' = c_s \frac{P_a}{29.92} \tag{III-9}$$

where P_a is the barometric pressure in inches of Hg.

The variation in the value of the reaeration coefficient due to the variation in temperature has been determined by Eckenfelder and O'Connor. It is given as

$$k_2^T = k_2^{20} \; (1.047)^{20-T} \tag{III-10}$$

where T is the temperature of water in °C. Authors have enumerated five different models to determine the reaeration coefficient based on stream geometry and stream characteristics. Most of these follow the general form

$$k_2 = c\frac{\bar{u}^n}{D^m} \tag{III-11}$$

where $\bar{u}$ = mean stream velocity, ft/sec

D = mean stream depth, ft

c,n,m = constants for a given stream

The rate of oxygen utilization due to biochemical oxygen demand (BOD) was expressed as a first-order bio-kinetic reaction

$$\frac{\partial L}{\partial t} = -k_1 \; L_t \tag{III-12}$$

where k_1 = BOD rate constant to base e, mg/L

L_t = concentration of BOD (ultimate), mg/L, at time t

The effects of temperature on k_1 are represented as

$$k_1^T = k_1^{20} \ (1.075)^{20-T} \qquad \text{(III-13)}$$

where T is the temperature of water in °C.

Assuming complete mixing, the authors represent the Equation III-1 for BOD-DO balance with internal mixing as

$$A \frac{\partial c}{\partial t} = \frac{\partial (AD_L \frac{\partial c}{\partial x})}{\partial x} - \frac{\partial (A\bar{u}c)}{\partial x} \pm A \text{ "}s_{DO}\text{"} \qquad \text{(III-14)}$$

where $"s_{DO}" = k_2(c_s - c) - (k_1)L$, mg/L-sec

c = concentration of dissolved oxygen in mg/L

and

$$A \frac{\partial L}{\partial t} = \frac{\partial (AD_L \frac{\partial L}{\partial x})}{\partial x} - \frac{\partial (A\bar{u}L)}{\partial x} \pm A \text{ "}s_L\text{"} \qquad \text{(III-15)}$$

where $"s_L" = -k_1L$, mg/L-sec

L = concentration of BOD (ultimate), mg/L

For conservative mineral routing, the authors felt that the concentration of a conservative mineral varies with the stream discharge, Q; thus, routing a conservative mineral requires no more than a material balance. Therefore, Equation III-1 without a source or sink term is sufficient to describe the behavior of a conservative mineral within a stream or canal system and it is given as

$$A \frac{\partial c}{\partial t} = \frac{\partial (AD_L \frac{\partial c}{\partial x})}{\partial x} - \frac{\partial (A\bar{u}c)}{\partial x} \qquad \text{(III-16)}$$

where c is the concentration of the conservative mineral in mg/L.

No analytical solution can be found to Equation III-1 under most prototype situations. Therefore the authors used a finite difference method to solve the model.

The input requirements of QUAL-1 are rather extensive. Physical properties such as the location of waste loadings and withdrawals, the location of stream or canal junctions, and the location and identification of headwater sources available for potential flow augmentation, are required. Input water quality data include biochemical oxygen demand, dissolved oxygen concentrations, temperature, and conservative mineral concentrations. Hydrologic data include headwater flows, waste discharges or withdrawals,tributary inflows, incremental flows (runnoff), and depth-velocity-discharge relationships. Reaction rates such as k_1, k_2 need to be estimated by field data or equations from literature.

The output from QUAL-1 yields a time history and spatial description of the distribution of a selected quality constituent throughout the stream or canal system of interest.

The kind of data that were available to the planning group of the "Comprehensive River Basin Planning" project were inadequate for use with QUAL-1. However, field studies were made on the Bird Creek to collect data for testing the QUAL-1 modeling system. In terms of cost, the data collection program was expensive. So was the computer time required for testing the QUAL-1. It became soon apparent that the QUAL-1 mathematical modeling system was not appropriate for the planning process due to the limitations of time, money, personnel and precise data. The need was for a different kind of model that would perform the required task within the limitations described above.

A multi-constituent mathematical model of water quality in rivers was developed by M. E. Harper (19) in 1972. This model is applicable for steady-state hydrologic conditions, although with modifications, the model can handle non-steady flows. The model represents a river system divided into a number of elements with water flowing from one element to another.

Each element is assumed to be completely mixed and for each time interval, a multiple step explicit solution is used to solve the partial differential equations describing the water quality processes. The model is capable of simulating the following parameters:

1. Temperature.
2. Dissolved oxygen.
3. Biochemical oxygen demand.
4. Conservative minerals.
5. Nitrogen-nitrate.
6. Total inorganic phosphorous, ortho phosphate.
7. Phytoplankton
8. Benthic algae, etc.

Dissolved oxygen changes are calculated by considering the following:

1. Reaeration coefficient is determined by

$$K_2^{20} = \frac{(D_m \bar{u})^{0.5}}{D^{1.5}} \qquad \text{(III-17)}$$

for streams displaying low velocities and isotropic conditions, and

$$K_2^{20} = \frac{480\ D_m^{0.5}\ S_o^{0.25}}{D^{1.25}} \times 2.31 \qquad \text{(III-18)}$$

for streams displaying high velocities and non-isotropic conditions, where

S_o = slope of the streambed

D = mean stream depth, ft

$\bar{u}$ = mean velocity, ft/day

D_m = molecular diffusion coefficient, ft^2/day, which is computed by

$$D_m = 1.91 \times 10^{-3}\ (1.037)^{T-20} \qquad \text{(III-19)}$$

Isotropic conditions are satisfied when Chezy's coefficient is greater than 17, and non-isotropic conditions exist when Chezy's coefficient has values less than 17.

A temperature correction of the type (III-10) is used to account for the variations in the K_2 due to temperature changes.

2. BOD decay is modeled as the first order bio-kinetic process

$$\frac{\partial L}{\partial t} = -K_1 L_t \qquad \text{(III-20)}$$

with a temperature correction of the type (III-13).

3. Benthal oxygen demand is computed by

$$D_B = \frac{k\ c^{0.3}}{H}\ \theta^{(T-20)} \qquad \text{(III-21)}$$

where D_B = benthal oxygen demand, mg/L

k = demand coefficient

c = dissolved oxygen concentration, mg/L

H = water depth, ft

θ = temperature correction factor

4. Algal production and respiration including phytoplankton and benthic algae are determined by a simple equation which sets the changes in dissolved oxygen equal to a factor times the rates of these two processes.

The author assumes that the conservative constituents do not react; therefore the concentration of a conservative constituent is given by

$$C_s = C_w \frac{Q_w}{Q_s + Q_w} \qquad \text{(III-22)}$$

where C_s = concentration of the conservative constituent in the stream water after discharge, mg/L

C_w = concentration of the conservative constituent in the waste,

Q_w = waste flow

Q_s = stream flow

The model emphasizes the determination of the time interval for evaluation of the equations describing the water quality processes and minimizing dispersion errors. The author developed two criteria:

1. Travel time through an element cannot be greater than time interval of simulation:

$$\Delta t = \frac{x}{u} \tag{III-23}$$

   where u is the stream velocity and x is the element length.

2. Dispersion stability is expressed by

$$\frac{D_L \Delta t}{(\Delta x)^2} < 1/2 \tag{III-24}$$

   where D_L is the dispersion coefficient. When this requirement is not met, mass is generated.

The author applied his model to the Green River, Washington. Temperature simulations were well in agreement with the observed field data; however the simulation of other quality constituents was qualitative only because of the large variations in the field data. The general response of the model indicated that the simulation of the temporal and spatial variations of the water quality parameters by the model was reasonably good.

R. V. Thomann is a well known author in the field of water quality modeling. Thomann made a study of water pollution control in the Delaware estuary (55). Early in the study, he realized that the task could be accomplished in a meaningful fashion only by modeling the entire system in a rigorous mathematical manner. He saw two basic objectives for comprehensive planning:

1. To develop an evaluation of the cause and effect relationships between the external environment (waste discharges, temperature, etc.) and the quality of water (in terms of its dissolved oxygen); and
2. To utilize a portion of this evaluation to develop further a rational approach to the attainment of various water quality goals.

The simplest systems analysis of a water body is the classical dissolved oxygen sag equation. The input to the system is the oxygen demanding material at the point of waste discharge, the output from the system is the DO deficit below saturation level, and the transformation component of the system includes reaeration, time of travel (flow velocity), and the decay of the organic matter. The classical DO sag equation representing the system in a differential equation form is

$$\frac{dc}{dt} = k_2 \; (c_s - c) - k_1 L \qquad \text{(III-25)}$$

Through manipulation, this equation can be represented as

$$u \; (\frac{dc}{dx}) + k_2 c = k_2 c_s - k_1 L \qquad \text{(III-26)}$$

where u is the velocity and x the distance. This static system is composed of the input '$k_2 c_s - k_1 L$'; the output 'c'; and the transformation given by the differential operator '$[u \; (d/dx) + k_2]$'. In qualitative terms

(Transformation)(Output) = (Input)

or

$$w \; c = f \qquad \text{(III-27)}$$

Now if the output is desired, Equation III-27 is solved as

$$c = w^{-1} \; f \qquad \text{(III-28)}$$

where w^{-1} is the reciprocal or inverse operation of differentiation, namely integration. The output response is directly proportional to the input level and the system parameters are the velocity and the reaeration coefficient. This model can be applied to an estuary as well as to a nontidal stream. The most important additional consideration is that of tidal diffusion. One useful way of considering this problem is to write a sequence of differential DO balance equations around a number of finite sections or reaches of the estuary under study. For a one-dimensional system where the sections are strung out longitudinally down the length of the stream, the equation can be written as

$$V_k \frac{dc_k}{dt} = Q_{k-1,k} \, [\xi_{k-1,k}(c_{k-1}) + (1 - \xi_{k-1,k})c_k]$$

$$- Q_{k,k-1}[\xi_{k,k-1}(c_k) + (1 - \xi_{k,k+1})c_{k+1}]$$

$$+ E_{k-1,k}(c_{k-1} - c_k) + E_{k,k+1}(c_{k+1} - c_k)$$

$$+ V_k f_k(t) \qquad \text{(III-29)}$$

where c_k = dissolved oxygen in segment k

V_k = volume of segment k

Q_{ij} = net flow from section i to section j

E_{ij} = eddy exchange (diffusion) coefficient between sections i and j

ξ_{ij} = dimensionless mixing parameter between i and j

f_k = all input sources and sinks acting in section k

Equation III-29 states that the rate of change of DO in segment k is given by the net amount of oxygen transported via the flow mechanism plus the oxygen carried into or out of the section as a result of tidal diffusion plus any actions within the segment that may increase or decrease the oxygen content. Atmospheric reaeration and biochemical oxygen demand are included in such actions.

If there are n segments in a stream or estuary, the following set of equations can be written:

$$-c_1 = \sum_{i=1}^{n} \phi_{i1}^{-1} J_i$$

$$-c_2 = \sum_{i=1}^{n} \phi_{i2}^{-1} J_i$$

$$\vdots \qquad \vdots \quad \vdots$$

$$-c_n = \sum_{i=1}^{n} \phi_{in}^{-1} J_i \qquad \text{(III-30)}$$

where c_j = DO response in segment j, mg/L

ϕ_{ij}^{-1} = transformation in input J_i to output c_j (input-output coefficients, mg/L response/ mg/L/day input)

J_i = waste load input in segment i, mg/L/day

For spatially varying fresh water inflow and diffusion rates, the transformations cannot be determined explicitly, but must be computed via high speed digital or analog techniques. The author sees three advantages of the model presented:

1. The model can compute the DO time response to any time varying input which may include, for example, a batch discharge of organic matter.
2. The model is easily extended both mathematically and computationally to more than one dimension, which allows for inclusion of tidal tributaries or embayments.

3. The steady state response due to steady state inputs is amenable to a linear programming formulation.

In the application of the model to the Delaware River, the author divided the estuary into 30 reaches, each having a length of either 10,000 or 20,000 feet. A computer program was written to handle the equations. The program could easily incorporate two dimensional aspects provided the basic information on lateral diffusivity and flow exchanges is available. The author admitted that there is a lack of this type of input prototype data.

The steady-state waste input-DO output coefficients can be computed by a relatively simple computer program. In case of the Delaware river, the author had to invert two 30 x 30 matrices and then compute the product of the inversion giving a new 30 x 30 matrix. The author felt that critical judement and consideration should be used in the interpretation of the results.

An interesting paper was written by Novotny on mathematical modeling of water quality changes in a river basin (37). Novotny represented the general equation of mass balance of flowing water as

$$\frac{\partial y}{\partial t} + D_m \frac{\partial^2 y}{\partial x^2} + \bar{u} \frac{\partial y}{\partial x} - ky = 0 \qquad \text{(III-31)}$$

where y = concentration of a substance under study

t = time

D_m = a longitudinal diffusivity coefficient

u = mean velocity of flow in x direction

k = decay coefficient

The author felt that for most streams, the effect of diffusivity may be omitted. The distribution of BOD in natural streams was expressed as

$$\frac{\partial L}{\partial t} V = -(L + \frac{\partial L}{\partial x} \Delta x)Q + LQ - j\Delta xB - (k_1 + k_3)LV \quad \text{(III-32)}$$

where Q = discharge or flow in the stream

V = volume of water = $\Delta x \cdot H \cdot B$

B = width of flow

j = flow rate of organic matter on the boundary bottom surface

k_1 = deoxygenation coefficient (laboratory bottle test)

k_3 = rate constant for BOD removal by sedimentation

H = depth of flow

The dissolved oxygen concentration model can be expressed as

$$\frac{\partial c}{\partial t} V = -(c + \frac{\partial c}{\partial x} \Delta x)Q + cQ - j\Delta xB - k_1 LV$$

$$- D_B \Delta xB + k_2(c_s - c)V \quad \text{(III-33)}$$

where c = dissolved oxygen concentration

D_B = oxygen demand by benthal deposits

k_2 = reaeration coefficient

c_s = oxygen saturation level

For conservative pollutants, the equation of mass balance becomes rather simple.

$$\frac{\partial z}{\partial t} V = -(z + \frac{\partial z}{\partial x} \Delta x)Q + zQ \qquad \text{(III-34)}$$

where z is the concentration of a conservative pollutant.

In a steady state situation, Equations III-32, III-33 and III-34 can be solved by letting $\partial y/\partial t = 0$ in Equation III-31. Equation III-32 becomes

$$\frac{dL}{dt} = -(k_1 + k_3 + j/HL)L = -\bar{k}L \qquad \text{(III-35)}$$

the ratio j/HL can be approximated by

$$j/HL \simeq \nu \frac{I^{1/4}}{H^{3/4}} \qquad \text{(III-36)}$$

thus

$$\bar{k} \simeq k_1 + k_3 + \nu\frac{I^{1/4}}{H^{3/4}} \qquad \text{(III-37)}$$

where ν = coefficient representing the ability of stream's bottom to develop biological growths

I = energy gradient in the stream

The solution to Equation III-33 becomes

$$D = \frac{L_o(\bar{k}-k_3)(e^{-\bar{k}t} - e^{-k_2t})}{k_2 - \bar{k}} + D_oe^{-k_2t} + (1-e^{-k_2t}) \frac{D_B}{k_2H} \qquad \text{(III-38)}$$

where D = deficit of dissolved oxygen in time t

D_o = deficit of DO at time zero

L_o = initial concentration of BOD in stream

The author applied his models to a dynamic situation such as exists in a natural river network. The mass balance equations were modified and called dynamic water quality model. The time step of solution Δt was set equal to 1/3 hour. The author divided the river network into sections connected by junctions. It was assumed that the self purification processes take place in the sections while waste effluents and tributaries are situated at junctions. Each section is divided into a certain number of elements and mass balance in the time step Δt is carried out in each element. The author recommended that the division of each section into elements should be finer in small streams compared to that in an estuary or a large river because the self-purification rate in small streams is greater than that in an estuary.

The dynamic water quality model for BOD was expressed as

$$\begin{aligned} \Delta L_{(i,t)} = {} & [L_{(i-1,t-\Delta t)} Q_{(i-1,t-\Delta t)} \\ & - L_{(i,t-\Delta t)} Q_{(i,t-\Delta t)}] \frac{\Delta t}{V_{(i,t)}} \\ & - (k_1 + k_3 + \nu \frac{I_i^{1/4}}{H_{(i,t)}^{3/4}}) \Delta T \, L_{(i,t-\Delta t)} \end{aligned} \qquad \text{(III-39)}$$

and the dissolved oxygen concentration changes were expressed as

$$\Delta Ac_{(i,t)} = [c_{(i-1,t-\Delta t)}Q_{(i-1,t-\Delta t)} - c_{(i,t-\Delta t)}Q_{(i,t-\Delta t)}] \frac{\Delta t}{V_{(i,t)}}$$

$$- (k_1 + \nu \frac{I_i^{1/4}}{H_{(i,t)}^{3/4}}) \Delta t\, L_{(i,t-\Delta t)}$$

$$+ [(c_{s(t)} - c_{(i,t-\Delta t)}) + k_{2(i,t)} - D_B/H_{(i,t)}]\Delta t \qquad \text{(III-40)}$$

where i is the number of element in section.

The main drawback of these models is that they require a great computer storage capacity and a great amount of central processing unit (c.p.u.) time, thus making it an expensive model to run. Also, the required input data is quite large and a collection program for input data would certainly be expensive.

Lombardo and Franz constructed a model to simulate water quality dynamics in rivers and impoundments (31, 32). This model is linked to the Hydrocomp Hydrologic Simulation Program (HSP) which was developed by the Hydrocomp, Inc., of Palo Alto, California, in 1969 to simulate the hydrologic response of a watershed. Thus, through the use of both models, the hydrologic and water quality interactions of a watershed could be simulated.

In HSP, a watershed is divided into land segments and stream reaches. The HSP system consists of three modules, LIBRARY, LANDS and CHANNEL. LIBRARY handles the input data while LANDS calculates the channel inflow volumes from input rainfall and evapotranspiration. CHANNELS calculates the flow at the end of each reach. This model is applicable to impoundments and streams not subject to tidal influence.

QUALITY is a separate module which is capable of simulating quality changes in the channel flows. A stream is represented in the model as a series of reaches. Dispersion effects are assumed to be negligible. The model consists of a set of partial differential equations which are solved by a multiple step explicit solution method.

The water quality indices simulated by the model are:

1. Temperature.
2. Biochemical oxygen demand.
3. Coliforms (total, fecal, fecal streptococci).
4. Algae - chlorophyll a.
5. Zooplankton.
6. Sediment.
7. Organic nitrogen.
8. Dissolved oxygen.
9. Total dissolved solids.
10. Conservative constituents, etc.

Dissolved oxygen was considered to be influenced by the following factors:

1. Reaeration coefficient is calculated by the method of Churchill, Elmore and Buckingham:

$$k_2^{20} = 5.026\ \bar{u}^{0.969} \times D^{-1.673} \times 2.31 \qquad \text{(III-41)}$$

where k_2^{20} = reaeration coefficient, 1/day, at 20°C

$\bar{u}$ = average velocity in the stream, ft/sec

D = average depth of the stream, ft

A temperature correction of the type (III-10) is used to account for the variations due to temperature changes.

2. BOD decay is represented by the first order biokinetic reaction as shown in Equation III-20.
3. Nitrification.
4. Denitrification.
5. Photosynthesis and algal respiration.
6. Zooplankton respiration.
7. Benthal oxygen demand is represented by the method of Fillos and Molof:

$$\text{Benthal OD} = \text{Benthal ODT}^{(1 - e^{-1.22DO})} \qquad \text{(III-42)}$$

where Benthal ODT = temperature corrected oxygen demand

DO = dissolved oxygen concentration.

Total dissolved solids and conservative constituents are assumed to be unaffected in water and are represented by a model of the type (III-22).

All three groups of coliforms are assumed to decay according to a first order decay reaction

$$\frac{dB}{dt} = -k_B \cdot B \qquad \text{(III-43)}$$

where B = concentration of bacteria-coliforms

K_B = rate constant

Rate constant for each coliform type becomes an input to the model.

The model was applied to the Green River in Washington. Temperature simulation due to the thermal wastes of a hypothetical thermal power plant was successful. Reasonable accuracy was obtained when the author simulated other quality parameters using the data collected by Harper (18).

The estuary model developed by Chen and Orlob (31) consists of a network where the links are represented by one-dimensional channels and the nodes are represented by the volume elements. The model calculates tidal velocities, discharges and elevations for an estuary from the input data. The model is capable of simulating non-steady systems in which river flows, waste discharges and tidal influences are changing.

The quality constituents simulated by the model are: temperature, toxicity, total dissolved solids, coliform, BOD, oxygen, phosphorus, alkalinity, nitrogen (ammonia, nitrite, nitrate), algae, zooplankton, fish, etc. As can be expected, the model requires a large number of input parameters. The hydrologic input data include: river flows, tide flows, waste discharges, outflows, etc. The river parameters input data include: length, width, depth, friction factor, surface area, side slopes, elevations, volumes, etc.

In the simulation of quality parameters, total dissolved solids are treated as a conservative constituent due to the difficulty of simulating all of the processes affecting total dissolved solids concentration. Though the total dissolved solids can be treated as a conservative parameter in many watersheds, the author cautions against its indiscriminate use. The validity of that assumption should be checked in each case.

Coliforms are simulated according to a first order decay reaction. In the reaeration process, the reaeration coefficient is computed by

$$k_2 = \left(\frac{DV}{H}\right)^{1/2} \qquad \text{(III-44)}$$

where D = molecular diffusivity, meters^2/sec

V = water velocity, meters/sec

H = depth of water, meters

k_2 = reaeration coefficient, meter/sec

The dissolved oxygen concentration is considered to be affected by the following factors besides reaeration: BOD, nitrification, algal growth, zooplankton and fish respiration, bacterial and benthal respiration, etc.

The nitrification process is assumed to be a first order reaction in the simulation of inorganic forms of nitrogen. The factors having effect on the nitrification are: BOD degradation, algal uptake and respiration, zooplankton excretion, and bacterial regeneration.

The solution to the mass transfer equations is obtained by a step-forward explicit technique. The model was applied to the San Francisco Bay-Delta system. The authors felt that though the trends could be simulated, the true utility of the models could not be demonstrated due to the limitations of data.

In 1971, Environmental Protection Agency published a report (20) that was prepared by the Hydroscience, Inc., of Westwood, New Jersey. This report was intended to assist and facilitate interim planning for water quality management of a river basin by presenting a general simplified methodology for the application of mathematical models to the analysis of water quality.

In the analysis of dissolved oxygen, the location and magnitude of the maximum deficit of dissolved oxygen are considered to be very essential features of the dissolved oxygen concentration profile for a river. The following equations were given for the magnitude and location of the critical DO deficit.

$$D_c = \frac{k_d}{k_a} L_o \, e^{\frac{-k_d x_c}{u}} \qquad \text{(III-45)}$$

and

$$x_c = \frac{u}{k_a - k_d} \ln \frac{k_a}{k_d} \qquad \text{(III-46)}$$

where D_c = maximum DO deficit

L_o = BOD concentration at $x = 0$

k_d = deoxygenation coefficient

k_a = reaeration coefficient

x_c = distance to location of maximum deficit

Through manipulations of Equations III-45 and III-46, the following equation was derived.

$$\frac{D_c}{L_o} = (f)^{f/(1-f)} \qquad \text{(III-47)}$$

where $f = k_a/k_d$. A practical range of f was indicated to be from 0.1 to 20.

For the purposes of planning, EPA recommended in the report that substances such as total dissolved solids, chlorides and nutrients (total nitrogen and total phosphorus) be considered as conservative substances. This classification could be extended to include any constituent which may be assumed to decay, in accordance with a single reaction, such as bacteria concentrations, radioactive matter, suspended solids, etc. Although these constituents are non-conservative, the most indicative concentration is at the outfall and hence is independent of the reaction effect.

The maximum value for this type of constituent is given by

$$c_o = \frac{W}{Q} \qquad \text{(III-48)}$$

where W = mass rate of waste discharge

Q = total freshwater flow

c_o = maximum concentration of a conservative substance

The non-conservative substances such as coliform bacteria, BOD, nutrients, etc., are governed by a first order reaction, viz.

$$c = c_o e^{-kx/u} \tag{III-49}$$

where c = concentration of a non-conservative substance

c_o = initial concentration of the substance

k = reaction coefficient

x = downstream distance

u = stream velocity.

The initial concentration was considered to be of basic importance in assessing the effectiveness of a waste treatment program for maintaining water quality standards. An interesting equation was developed for the determination of the initial concentration of a substance:

$$c_o = \frac{f_4 f_3 f_1 P_o}{f_5 DA + f_2 f_1 P_o} \tag{III-50}$$

where f_1 = population growth factor

f_2 = per capita waste flow contribution

f_3 = per capita waste load contribution

f_4 = residual fraction after treatment

f_5 = flow/drainage area

DA = drainage area

P_o = present population

Tables are presented to determine the f_1, f_2, f_3, f_4. The authors felt 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."

The technical data required for analysis fall into two categories:

1. Waste discharges:
    a. Population, growth factor.
    b. Per capita waste flows and quality: water gallons/cap-day, ultimate BOD lbs/cap-day, suspended solids lbs/cap-day, nutrients lbs/cap-day, coliform MPN/cap-day.
    c. Treatment efficiencies and residuals: marginal secondary, highrate biological, secondary with nitrification, advanced, ultimate. Percent removal of carbon, nitrogen, lbs/capita of ultimate oxygen demand remaining, etc.
2. Characteristics of drainage basin:
    a. Temperature.
    b. Natural background quality: land use, runoff effects on water quality.
    c. Fresh water flow: velocity, low flows, average annual flows.
    d. Characteristics of river channel and bed: classification of streams, surface and bed conditions.
    e. Channel geometry: average depth, surface area and volume.
    f. Dispersion coefficients.

Numerous tables, nomographs, and figures are included in the report to help the use of mathematical models for water quality analysis. It is a useful document for planning of streams and estuaries for such parameters as coliform organisms, nutrients, oxidation of carbonaceous and nitrogenous compounds, dissolved oxygen and certain dissolved solids.

The models discussed above are very good representatives of the recent efforts made in the field of water quality modeling. The literature contains a rather large number of studies made to date. It is not necessary to review all of the models developed so far because a majority of them consist of similar approaches to the ones already discussed. The model development in most cases depends on the same basic principles and methodology with slight variations. Therefore, the following studies shall be reviewed briefly, mentioning only the interesting and pertinent points of methodology and techniques employed in the models.

Tirabassi (54) employed mathematical statistics to formulate a model to predict the quality of water in rivers without a reference to the causal chemical, biological and physical relationships existing in the river flow. The approach adopted by Tirabassi is popularly known as a "black box" approach where with a known input, one tries to predict the output with a certain amount of reliability. This model can provide accurate predictive information with a minimum of time and money, if a sufficiently large data base can be made available for the river system.

Unfortunately, such a large data base is, generally, not available and the data collection, as mentioned previously, could be expensive! Thus, the "black box" may not be able to provide accurate predictions of water quality.

Tucker and Goodman (57) concentrated on stream flow routing for streams with moderate to low discharges. They designed a special flow routing procedure which focuses on the discharge, velocity and time output. These data are necessary to study the effects of unsteady flows on stream pollution. The authors noted that previously available routing methods primarily estimate water surface elevations for floods. The input data required by the model are the hydrograph data at an upstream station and discharge-velocity relationships for various reaches of the stream. An electronic digital computer processes a largely empirical program that determines the hydrograph at any specified station.

Weeter (64) incorporated diurnal photosynthetic effects in his dissolved oxygen concentration prodiction model. The model can predict the minimum and the maximum dissolved oxygen values. Weeter developed a log-log relationship between the time of travel and the reaeration coefficient and flow. The author applied his model to the Wabash river. From the available data however, he could not quantify the oxygen demand by the benthal sludge deposits, the incremental runoff from the land and the nitrification coefficient.

Mumme (36) used an interesting concept. He considered a river as a chemical reactor. The DO content of the water is compared to either the reactor product or to an excess reactant to be maximized. In order to control such a reactor, Mumme employed a mathematical model. The input to the model is a given biochemical oxygen demand loading while the output, as predicted by the model, is the DO response. The model is capable of adapting itself to changing environments. This assures, the author believes, that the model, at any point in time, shall provide an acceptably accurate representation of the actual BOD-DO relationship in a natural stream.

Lee and others (24, 25) applied forecasting techniques such as quasilinearization and invariant imbedding to the water quality modeling. Lee and Hwang considered the parameter estimation problem as a two point or multipoint boundary value problem. They used the classical least squares criterion to determine the parameters. Lee, in the other study with Erickson and Fan, used a model with axial mixing. By properly adjusting the axial diffusion coefficient, the authors could simulate streams, with the help of the model, with any degree of mixing. The authors employed second order differential equations to represent a stream system with intermediate reservoirs, and waste discharges and water intakes along the stream.

A hydro quality simulation curve was developed by Dixon, Hewdricks, and Huber (9). This model is a device for studying some practical questions that involve some measure of efficiency. Some of the aspects studied by the model are:

1. Pollution minimization by waste treatment.
2. Pollution abatement by flow regulation.
3. Maximization of water use with quality constraints.
4. Maximization of economic efficiency.
5. Development of more comprehensive laws, etc.

The authors believed that these aspects can be studied by the simulation of altered conditions, sensitivity analysis or some optimization technique.

Loucks and Lynn (33) were interested in predicting the probability distribution of minimum dissolved oxygen concentrations occurring down stream from a waste water treatment plant. They assumed that the transition probabilities for daily stream flows are described by a first-order Markov process. The authors used four models to achieve the task. Each of these four models is based upon one of the following four assumptions respectively.

1. A fixed BOD concentration exists for each daily sewage flow.
2. A range of possible BOD's exists per flow.
3. BOD's are fixed but sewage and stream flows are serially and cross correlated.
4. A range of possible BOD's exists but it is fixed and the sewage and stream flows are serially and cross correlated.

Many authors have employed interesting systems analysis techniques in the development and solution of their models. To mention a few: Novotny and Schmidtova (38) used the harmonic analysis to analyze the periodic phenomena in a stream; Bella and Dobbins (1) developed a finite difference method for the numerical analysis of the BOD-DO profiles; DiToro (8) employed the method of characteristics to a one-dimensional continuity model; Francis Hall (17) applied hysteresis-loop relationship to study the relationship between the dissolved constituents and the discharge in a stream; Fuller and Tsokos (14) used time series techniques to non-stationary water pollution data while Guymon (16) made use of a quasilinear variation principle in his model.

The Water Quality Prediction Models

A large number of the water quality models are simulation models that employ some mathematical or systems analysis techniques to simulate a stream or a system or network of streams for a certain number of water quality parameters. The review of the literature showed that, generally, these models are large and require a great amount of input data. They involve many parameters that must be estimated with precision for good results. A computer program is employed, in most cases, to solve the model. Many of these models have been reported to be expensive in operation.

For a river basin planning, the planner is generally faced with essentially inadequate data. His main source of data is the local or regional government agencies' files. For the purposes of this study, a model or models were needed that could help evaluate the water quality of a region. River basin planning is a regional planning, and this character of the planning is the most important consideration, coupled with the limitations of time, money, manpower and data.

In 1971, the Resources for the Future, Inc. published a study made for them by a group of five authors (47). These were George Reid, Wesley Eckenfelder, Leale Streebin, Robert Nelson and Oliver Love. Professor George Reid was the principal author of the group. The models developed by them were viewed to be very suitable for the purposes of this study.

Six categories of pollutants were recognized:

1. Biodegradable: organic oxygen demanding substances such as sewage and certain types of industrial wastes.
2. Bacterial: infectious agents; bacteria, viruses, parasites, etc.
3. Aggravated eutrophication: plant nutrients, particularly nitrogen and phosphorus.
4. Conserved: persistent chemicals; brines, certain toxic metallic ions, and radioactive substances that remain largely unaffected by the conventional waste treatments or natural stream recovery processes.

5. Thermal: heat from the cooling water discharges.
6. Sediments: primarily waste suspended solids incorporated in sludges.

Of prime concern to water quality management is the response of a receiving stream to the waste discharges. Three stream responses were modeled: (1) biodegradable; (2) nutritional; and (3) thermal. Persistent chemicals, sediments and bacteria can be treated as constraints. For this study, the first two responses are studied.

Development of the Models (47)

Biodegradable Model

The stream assimilative capacity can be represented in a differential form by

$$\frac{dD}{dt} = k_1 L_u - k_2 D \qquad \text{(III-51)}$$

where D = oxygen deficiency, mg/L

L_u = oxygen demand, mg/L (u refers to ultimate value)

k_1 = decomposition reaction coefficient, to base e

k_2 = reaeration reaction coefficient, to base e

t = time, days

For a steady state situation where $dD/dt = 0$,

$$k_1 L_u = k_2 D \qquad \text{(III-52)}$$

The rate of oxygen supply in mg/L per day can be expressed as

$$\text{mg/L of } O_2\text{/day} = 2.3\ k_2\ D \qquad \text{(III-53)}$$

where k_2 is reaeration coefficient to the base 10. If Q is the streamflow in million gallons per day, then

$$\text{lbs } O_2 = 2.3\ k_2 \cdot D \cdot (8.3)\ Q \qquad \text{(III-54)}$$

Of more importance than a discrete flow measurement is the volume of water effectively operative in the reoxygenation process. This volume can be visualized as a pyramid with its peak representing the source of the stream, and having a height equal to the average stream length (ℓ) in miles. The base equals the terminal flow of the stream (Q_t) expressed in million gallons per day divided by the velocity (v) expressed in miles per day. Thus the Equation III-54 appears as

$$\text{lbs } O_2/\text{day} = 2.3\ k_2 D\ \frac{Q_t}{3}\ \frac{\ell}{V}\ (8.3) \qquad \text{(III-55)}$$

All streams, in general, are composed of a main stem and one or more discrete contributing tributaries. For each basin the terminal flow is influenced by the number of reoxygenation volumes, represented by n, which is a multiplier for long systems to account for the fact that the same water may purify more than one waste discharge. The number of reoxygenation volumes or the effect of branching on aeration capacity can be represented by

$$n = \frac{R}{\ell} + \frac{x - 1}{x} \qquad \text{(III-56)}$$

where R = reach of the main stem

x = the number of discrete tributaries

Thus, a stream's capability for natural regeneration may be expressed as

$$\text{lbs of } O_2/\text{day} = 2.3\ k_2 D\ \frac{Q_t}{3}\ \frac{\ell}{V}\ n \qquad \text{(III-57)}$$

It is obviously more efficient to put in a given load in many small doses rather than one big dose; hence an adjustment needs to be made to represent the regenerative capacity of the receiving water in terms of the way the waste load is imposed. A relative effectiveness coefficient (ε) can be used to represent this adjustment. Thus,

$$\#\ \text{BOD/day} = k_2 Q_t D\ \frac{n\ell}{V}\ \left[\frac{8.3 \times 2.3}{24 \times 3}\right]\ \varepsilon \qquad \text{(III-58)}$$

the velocity V is, now, in miles per hour. ε is defined to be less than or equal to one. ε is a function of t_r and k_2 where, essentially, t_r is the time for the sag curve to recover to its original position. Hahn and Reid have developed the following expression for t_r

$$t_r = \frac{\ell(x+1)}{24V} \times \frac{1}{(\#\text{ of metropolitan areas})} \qquad \text{(III-59)}$$

The following figure shows the relationship of ε to t_r and k_2.

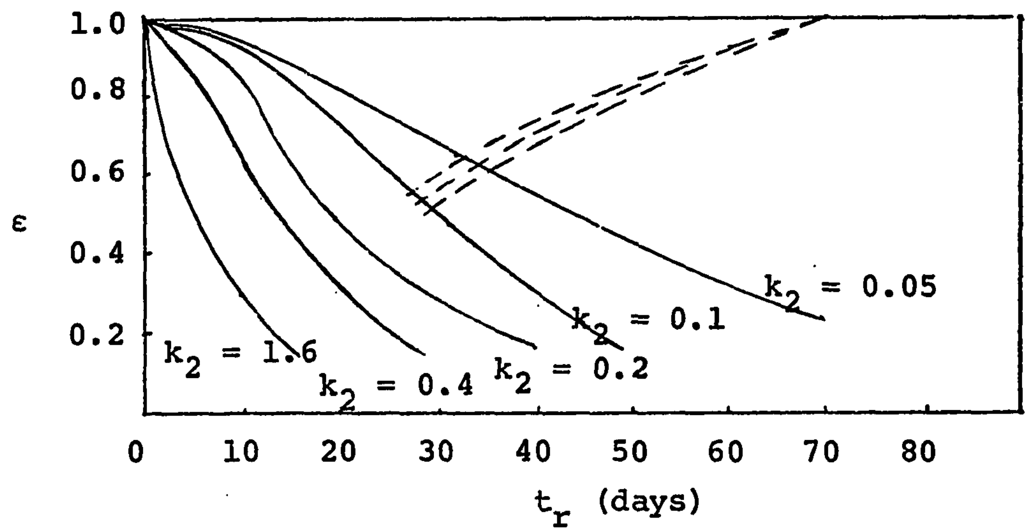


Figure III-1. Efficiency Term-Ratio of Point Loading to Uniform Loading.

If k_2 varies between 0.1 and 0.4, an analytical equation is given for ε as,

$$\varepsilon = 1 - \frac{t_r \cdot k_2^{3/2}}{10} \qquad \text{(III-60)}$$

If a population equivalent (p.e.) of 0.25 # BOD/day can be assumed, then Equation III-58 can be expressed, through manipulations, as

$$Q_{\text{point load}} = \frac{PE}{k_2 D \frac{\ell}{V} n \varepsilon} \left[\frac{250000 \times 24 \times 3}{8.3 \times 2.3}\right] \qquad \text{(III-61)}$$

where PE is population equivalents in millions.

The total required terminal flow is obtained by taking a weighted average of the flows required for point loads and those for uniform loads. Because point loading occurs primarily in metropolitan areas and uniform loading occurs in non-metropolitan areas, the weights used are the fraction of the region's population residing in the metropolitan areas (Y) and the fraction not residing in such areas (1-Y). Thus,

$$Q_t = \frac{Y}{\varepsilon}\left[\frac{PE}{Dk_2 n\frac{\ell}{V}}\right] 942{,}900 + (1-Y)\left[\frac{PE}{Dk_2 n\frac{\ell}{V}}\right] 942{,}900 \qquad \text{(III-62)}$$

or

$$Q_t = \left[\frac{Y}{\varepsilon} + (1-Y)\right] \frac{PE}{Dk_2 n\frac{\ell}{V}} (942{,}900) \qquad \text{(III-63)}$$

The DO deficit is expressed as the saturation level (c_s) minus the required quality standard for dissolved oxygen (RQS_{DO}); i.e.,

$$D = (c_s - RQS_{DO}) \qquad \text{(III-64)}$$

If PP is the fraction of the wastes discharged to streams after the waste treatment, and Q_t is the terminal flow of a stream basin and PE is the population of the basin in millions, then

$$Q_t = \left[\frac{Y}{\varepsilon} + (1-Y)\right] \frac{PE \cdot PP}{(c_s - RQS_{DO})} \cdot \frac{942{,}900}{k_2 n\frac{\ell}{V}} \qquad \text{(III-65)}$$

<u>Nutritional Model</u>

The model is developed for phosphorus. A similar approach can be used for nitrogen.

The flow required for phosphorus at an acceptable RQS_p level is

$$Q_p = \frac{\text{lbs phos/day}}{RQS_p(8.3)} \qquad \text{(III-66)}$$

where Q_p = flow required for phosphorus assimilation, mgd

RQS_p = acceptable phosphorus level, mg/L

The total phosphorus is arrived at indirectly from the amount of BOD. Raw sewage is assumed to have a BOD/P ratio of 27/1 at a per capita contribution of 0.25 lbs of BOD. On this basis, the per capita contribution of phosphorus is

$$0.25 \times \frac{1}{27} = 0.009 \text{ lbs/day}$$

The phosphorus of interest is the amount remaining after treatment. This amount in terms of population in millions (Pop) would be

$$\text{lbs of phos/day} = [(1 - TL_p)0.009(10^6)](\text{Pop}) \qquad \text{(III-67)}$$

where TL_p is phosphorus removal level. Some of this phosphorus is consumed in the biodegradation process, and this amount is related to the BOD available. By assuming a BOD to phosphorus combining ratio of 100/1, the amount of phosphorus consumed is

$$\text{lbs phos consumed/day} = \frac{0.25(1 - TL_L)}{100} (\text{Pop}) \qquad \text{(III-68)}$$

where TL_L is the BOD removal level.

Subtracting Equation III-68 from Equation III-67 gives the excessive phosphorus to be assimilated.

$$\text{Excessive phos} = (\text{Pop})[(1-TL_p)0.009 - \frac{0.25(1-TL_L)}{100}]10^6 \qquad \text{(III-69)}$$

Substituting the Equation III-69 into Equation III-66 gives the required flow.

$$Q_p = \frac{(\text{Pop})(0.009)(10^6)}{8.3(RQS_p)}[(1-TL_p)-0.27(1-TL_L)](z) \qquad \text{(III-70)}$$

z is defined to be the relative portion of the stream impounded. z is calculated by the ratio of the number of days of retention now to the number of days of retention possible when the flow is regulated to median flow.

The model is developed using nutritional ratios for domestic waste. A scale factor is necessary so that it may also be used for industrial wastes. The scale factor (F_p) is the BOD/P ratio for the industrial waste in question divided by the BOD/P ratio for municipal waste. The final form for the accelerated eutrophication or the phosphorus model is

$$Q_p = \frac{z(\text{Pop})}{F_p(RQS_p)}[(1-TL_p)-0.27(1-TL_L)]1080(TL_L) \qquad \text{(III-71)}$$

The nitrogen model is developed on the similar lines as

$$Q_N = \frac{z(\text{Pop})}{F_N(RQS_N)}[(1-TL_N)-1.44(1-TL_L)]3250(TL_L) \qquad \text{(III-72)}$$

In the nitrogen model, the raw sewage was assumed to have a BOD/N ratio of 27/3 and BOD to nitrogen combining ratio in the effluents of approximately 100/17.

CHAPTER IV

WASTE WATER TREATMENT COST ANALYSIS

Waste Water Treatment

Liquid wastes generated from the industrial and municipal sources are ultimately disposed of by discharging them into watercourses or on or beneath the surface of the ground. Disposal into streams is most common and is sometimes called the "dilution of wastes." Disposal on land is not common. Disposal beneath the ground, however, is increasing. This method of disposal is generally called ground water recharge.

Disposal of sewage by dilution includes the discharge of a raw waste or an effluent from a treatment plant into a stream or some other waterbody with a sufficient flow to assimilate the wastes properly. The degree of dilution required depends on the volume and strength of the waste and the required water quality in the receiving water. Often, in the past, untreated sewage was dumped into large streams. The amount of dilution required was estimated by the "judgement" of the people in charge. If the flow in the stream was not enough to provide adequate dilution, the sewage was treated to remove some of the putrescible matter in the waste.

Unit Operations and Processes

As mentioned earlier in the paper, four types of waste treatment are available, viz. preliminary treatment, primary treatment, secondary treatment, and tertiary or advanced treatment. They indicate the level of waste removal from an influent to a waste treatment plant. Various unit operations and processes are used to treat wastes. The following paragraphs briefly discuss the most common unit operations and processes employed by waste treatment plants. A waste treatment plant would, generally, employ one or more of these operations and processes.

In the coarse screening operation, the screens remove the larger particles floating or suspended in the wastes. The screens are cleaned mechanically. Comminutors and barminutors are used extensively in the grinding operation to grind the larger particles without first removing them from the wastes. Grit chambers are included in a treatment plant when significant quantities of sand or other inert matter are found in suspension in the wastes. Grit chambers remove the sand while the organic materials continue through the plant.

Preaeration of raw sewage is sometimes very practical to improve the removal of suspended solids in subsequent primary treatment units. Preaeration greatly increases the settling rate of the suspended solids. Preaeration is most useful when a strong sewage is to be treated and the grit removal is required.

Primary treatment of wastes involves the use of flotation and sedimentation processes. Inorganic or organic coagulants are used in the coagulation process to accelerate the settling. Adequately designed units would generally remove through these processes about 98 to 99 percent of settleable solids, 60 to 80 percent of the suspended solids and about 30 to 50 percent of oxygen demanding materials.

The waste, after the primary treatment, still contains about 20 to 40 percent of the original amount of the suspended solids and about 50 to 70 percent of the original oxygen-demanding materials. If the primary treatment is not sufficient, the wastes are subjected to the secondary treatment. The most common secondary treatment processes are the trickling filter and the activated sludge processes. Both processes are biological in nature and depend upon the availability of atmospheric oxygen and the suitable growth of microorganisms. Secondary treatment processes can be designed to provide overall plant removals of 90 to 95 percent of the suspended solids and the organic materials.

Some substances are resistant to the most modern conventional water and waste treatment processes. In such circumstances, advanced or tertiary treatment processes are employed. These processes are any physical or chemical processes that can separate a soluble material from a solvent. The common processes for advanced treatment include:

1. Adsorption.
2. Electrodialysis.
3. Emulsion separation.
4. Evaporation.
5. Extraction.
6. Foaming.
7. Freezing.
8. Hydration.
9. Ion exchange.
10. Chemical or electrochemical oxidation.

Such processes seek to remove a specific substance completely or to remove suspended solids and organic matter to a level of 99.9 percent or more.

Treatment Efficiencies (20)

For interim planning of a river basin, Environmental Protection Agency recommended the following treatment levels:

1. Marginal secondary: Conventional secondary treatment system which is overloaded, upset periodically or poorly operated.
2. High rate biological: Conventional secondary treatment system with proper operation.
3. Secondary with nitrification: Biological treatment for the removal of carbon and for nitrification.

4. Advanced: Biological or physical-chemical treatment for the removal of carbon, nitrogen, and phosphorus. Filtration and the activated carbon treatment of an effluent.
5. Ultimate: Technology not yet applied to meet this requirement on a sustained basis.

The following table adopted from Reference 20 gives the removal efficiencies of the treatment levels described above.

TABLE IV-1

ESTIMATED EFFICIENCIES OF WASTE REMOVAL
FOR VARIOUS TREATMENT LEVELS

Treatment Level	Percent Removal				Fraction of UOD Remaining f
	C*	N+	P	N	
1. Marginal Secondary	70	10	20	10	0.56
2. High Rate Biological	85	20	20	20	0.44
3. Secondary with Nitrification	90	85	20	20	0.12
4. Advanced	95	95	85	95	0.05
5. Ultimate	99	99	90	99	0.01

C* = carbonaceous BOD; N+ = oxidizable nitrogen; P = total phosphorus; N = total nitrogen; UOD = ultimate oxygen demand.

Review of Waste Water Treatment Cost Models

A number of studies have been directed toward describing the cost of municipal waste treatment. The cost is usually expressed as a function of the design flow through the plant or the design population, and the expected level of waste removal efficiency. Recognizing the need for cost data, the U.S. Public Health Service began a study of the construction costs of sewage treatment facilities. Howells and Dubois (19) made the first of such studies. They based their study on the analysis of twenty small secondary sewage treatment plants in the upper midwest. The authors considered such costs as construction, operation and maintenance costs. The costs of land, engineering, administrative and legal services were not included in the analysis. The design population of the plants studied ranged from a mere 600 to 12,500. The authors analyzed the data for the following treatment plant units and processes:

1. Primary sedimentation.
2. Activated sludge.
3. Trickling filter.
4. Secondary sedimentation.
5. Sludge digestion.
6. Sludge drying beds.

In 1960, the U.S. Public Health Service authorized another study (44) to update the previous studies made by the

U.S.P.H.S. The study evaluated the cost data for six specific types of treatment:

1. Imhoff tank.
2. Conventional primary treatment with separate sludge digestion.
3. Activated sludge.
4. Trickling filter with separate sludge digestion.
5. Trickling filter with Imhoff type treatment.
6. Stabilization ponds.

All cost data were converted into 1913 dollars using the Engineering News Record - Cost Index. In 1964, the U.S.P.H.S. conducted yet another study (43). This study summarized the cost of 1,504 sewage treatment projects constructed under the Federal Government's Construction Grants program. A series of curves were developed relating the capital construction cost of the primary and secondary waste treatment plants to the populations served by the plants, the design flows of the plants, and the design population equivalents.

Clarence Velz (63) made a study of the costs of the treatment plants. He obtained his data from the literature and the questionnaires he sent out. The objective of the study was to relate the construction cost of a plant per million gallon a day of flow (cost/MGD) to the size of the plant in terms of its flow, mgd. To estimate the total cost of a plant, the author assumed that the bid price on the construction cost was about 80 to 85 percent of the total cost, excluding the costs of land, engineering and legal fees.

The author referred all plant costs to the year 1926 as the base year of construction, adjusting with the help of the Engineering News Record - Cost Index (ENR-C Index). The author developed the following model.

$$Y = aX^b \qquad \text{(IV-1)}$$

where Y = cost of a plant per MGD of flow

X = size of the plant in terms of MGD of flow

a,b = constants

Diachishin (7) attempted to refine and update the work of Velz. He analyzed the cost data from 154 plants. The author succeeded in developing separate models for primary treatment plants and secondary treatment plants. Diachishin preferred the year 1913 as the base year of construction rather than the year 1926 as used by Velz. The construction costs were adjusted by means of the ENR-C Index.

A study of 291 projects built in Illinois between 1957 and 1968 was made by Butts and Evans (5) for the Illinois State Water Survey, in 1970. The authors employed the techniques of least squares regression analysis to relate the design population equivalent to the construction cost. The authors developed the Federal Water Pollution Control Agency's Construction Cost Index (FWPCA-C Index).

The model for estimating the construction, operation and land costs of a plant was expressed as

$$c = kP^n \qquad \text{(IV-2)}$$

where c = construction, operation and land costs

k = regression constant

P = sewage treatment plant capacity in terms of average annual load treated

n = constant

Future expansion of a plant was accounted by the model

$$c = kP^nS^m \qquad \text{(IV-3)}$$

where c = cost of new addition

k = regression constant

P = capacity of the proposed addition in terms of average annual load to be treated

S = capacity of the existing plant

n,m = constants

Wollman (65) was the first author to use a multiple regression model to estimate the operation and maintenance costs of a plant. The model was formulated as

$$Y = b_o + b_1X_1 + b_2X_2 + b_3X_3 \qquad \text{(IV-4)}$$

where Y = the annual operation and maintenance cost per daily population equivalent (PE)

X_1 = treatment level in percent of BOD removal

X_2 = percent of total waste that is industrial

X_3 = population served by the sewage system,

$b_o, b_1,$
b_2, b_3 = regression coefficients

Application of the systems analysis techniques to the preliminary design of a waste treatment plant was made by Logan and others (30). The cost data for analysis were obtained by visiting the plants. Models were developed for estimating the cost per MGD of the plant as a function of the design capacity of the plant in MGD. The unit processes of the following treatment plants were studied:

1. Primary treatment plants.
2. High rate trickling filter plants.
3. Standard rate trickling filter plants.
4. Activated sludge treatment plants.

The authors found many inconsistencies in the field data. Therefore, their analysis was based on a series of theoretical designs under ideal conditions.

Park (42) approached the problem of estimating the construction cost of a plant by considering both the hydraulic loadings and the biological loadings of the plant. The author assumed that the primary treatment plant costs can be represented by the capacity of the plant in terms of its hydraulic loading, since the hydraulic loading is an important parameter for a primary treatment plant design. However, the secondary treatment plant costs can best be represented by the capacity of the plant in terms of its organic loading. To convert the unit cost per capita to the unit cost per lb of BOD, the author assumed 0.2 lb of 5 day BOD per person per day. Similarly, to convert the unit construction cost of a primary treatment plant to the unit cost per MGD, he assumed 100 gallons per capita per day of waste flow.

Thoman and Jenkins (57) realized the regional differences in the costs of construction of the sewage treatment plants. To account for these differences in the costs, the authors partitioned the U.S. into twenty regions on a county line basis. Each of the regions corresponded to one of the twenty cities used in obtaining the U.S. Average Engineering News Records - Cost Index. They referred the costs to the year 1913 as the base year. Three models were developed for estimating the construction costs of

1. Primary treatment plants.
2. Secondary treatment plants.
3. Stabilization ponds.

The main variable in the models is the design population.

Limited data is available on construction costs of industrial waste treatment plants. An effort was made by Eckenfelder (12) to assess the construction and operation costs of several types of industrial waste treatment plants. The author could not develop any model; however, he presented graphs for estimating the construction costs.

The costs of treating industrial wastes are subject to a much greater variability than the municipal wastes. This is due to the fact that an industry has a great number of options at its disposal. These options range from different product mix to better in-plant waste control and specialized treatment techniques.

Treatment Plant Construction Cost Models

Many studies have been made to date to estimate the construction costs of treatment plants. These studies vary widely in formats. The models presented also vary in form and methodology. In 1970, Kanti Shah and George Reid made a study (48) to develop models for estimating the construction costs of waste treatment plants. These models were viewed to be most suitable for use in this study for the following reasons:

1. A large number of studies relate to a particular region and cannot account for the regional differences in the costs. The Shah and Reid models account for the regional differences by studying the whole U.S. into twenty regions.
2. The cost index used by the Shah-Reid models is the Federal Water Pollution Control Agency Sewage Treatment Plant Construction Cost Index (WPC-STP Index) which is based on information peculiar to wastewater treatment plant construction. Its usefulness for evaluating the construction costs of waste treatment plants is much greater than the ENR-C Index which has been used by many other authors.

3. Most authors have estimated unit construction costs in terms of population or the flow. The Shah-Reid models use population equivalent (PE) and the design flow as the variables. These variables, together, represent the size of a plant better than the population or the flow by itself.
4. Separate model is available for each type of treatment plant.

Development of the Models (48)

The unit costs of primary treatment plants can be related to the hydraulic loadings of the plants while the unit costs of the secondary treatment plants are related to the hydraulic and organic loadings of the plants. Population equivalent is a good measure of the organic loading of a plant. Four variables were studied to predict the costs of a plant. They are:

1. PE - population equivalent.
2. Flow - MGD.
3. BOD of the influent, mg/L.
4. Efficiency of BOD removal.

The cost was evaluated in terms of:

1. 1957-59 dollars per design PE.
2. 1957-59 dollars per MGD of design flow.

Five types of waste treatment plants were modeled. They are:

1. Primary treatment plant.
2. Waste stabilization ponds.
3. Standard rate trickling filter.
4. High rate trickling filter.
5. Activated sludge.

To account for the possible regional differences in the construction costs of these plants, the authors considered the U.S. divided into twenty different regions on a county line basis. Each region corresponded to one of the twenty cities used in obtaining the U.S. Average ENR-C Index. However, for adjusting the cost data of treatment plants obtained from various parts of the country to a common base, the WPC-STP Index was used because it is based on information peculiar to wastewater treatment plant construction. Its usefulness for evaluating the construction costs of waste treatment plants is much greater than the ENR-C Index.

The general form of the model was

$$Y = \beta_o + \beta_1 X_1 + \beta_2 X_2 + \beta_3 X_3 + \beta_4 X_4 + e \qquad \text{(IV-5)}$$

where Y = construction cost of a plant in 1957-59 dollars per design MGD or per design PE

X_1 = design PE

X_2 = design flow in MGD

X_3 = design BOD influent in mg/L

X_4 = BOD removal efficiency.

It was felt that in some situation, the linear model may not be able to represent the cost of a waste treatment plant. Therefore, along with the linear form, following non-linear forms of the model were tested.

$$\text{Ln } Y = b_o + b_1X_1 + b_2X_2 + b_3X_3 + b_4X_4 \tag{IV-6}$$

$$\frac{1}{\text{Ln } Y} = b_o + b_1 \text{ Ln } X_1 + b_2 \text{ Ln } X_2 + b_3 \text{ Ln } X_3 + b_4 \text{ Ln } X_4 \tag{IV-7}$$

$$\text{Ln } Y = b_o + b_1 \text{ Ln } X_1 + b_2 \text{ Ln } X_2 + b_3 \text{ Ln } X_3 + b_4 \text{ Ln } X_4 \tag{IV-8}$$

$$\frac{1}{Y} = b_o + b_1X_1 + b_2X_2 + b_3X_3 + b_4X_4 \tag{IV-9}$$

The sample size of the data analyzed was 563. The sample size for each type of plant modeled was as follows:

1. Primary municipal - 102.
2. Stabilization ponds - 157.
3. Standard rate trickling filter - 67.
4. High rate trickling filter - 122.
5. Activated sludge - 115.

A modified stepwise regression procedure was used to analyze the data. Dummy variables were used to quantify different types of secondary plants.

The variables, X_3 and X_4, the influent BOD and the BOD removal efficiency, were found to be "not significant" statistically, in the estimation of the construction costs of the waste treatment plants studied. The models developed are:

1. Primary treatment plants:

$$\text{Ln } Y'' = 12.42 + 0.3852\ X_2 \qquad \text{(IV-10)}$$

where Y" is the construction cost per design MGD, in 1957-59 dollars.

2. Waste stabilization ponds:

$$\frac{1}{\text{Ln } Y''} = 0.1291 - 0.0044\ \text{Ln } X_1 + 0.0073\ \text{Ln } X_2 \qquad \text{(IV-11)}$$

$$\frac{1}{Y'} = 0.0511 + 0.0001\ X_1 - 0.0640\ X_2 \qquad \text{(IV-12)}$$

where Y' is the construction cost per design PE in 1957-59 dollars.

3. Standard rate trickling filter:

$$\text{Ln } Y'' = 7.90 + 0.5007\ \text{Ln } X_1 - 0.9568\ \text{Ln } X_2 \qquad \text{(IV-13)}$$

4. High rate trickling filter:

$$\text{Ln } Y'' = 9.39 + 0.3557\ \text{Ln } X_1 - 0.6443\ \text{Ln } X_2 \qquad \text{(IV-14)}$$

$$\text{Ln } Y' = 9.39 - 0.6443\ \text{Ln } X_1 + 0.3557\ \text{Ln } X_2 \qquad \text{(IV-15)}$$

5. Activated sludge treatment plants:

$$\text{Ln } Y'' = 8.53 + 0.4610 \text{ Ln } X_1 - 0.7375 \text{ Ln } X_2 \qquad \text{(IV-16)}$$

$$\text{Ln } Y' = 8.53 - 0.5389 \text{ Ln } X_1 + 0.2634 \text{ Ln } X_2 \qquad \text{(IV-17)}$$

Limited data were available to the authors on industrial waste treatment plants for treating petroleum, chemical, or pulp and paper wastes. Due to this limitation, the authors could not compute a regression equation with precision. A total sample of 25 primary treatment plants and 26 secondary treatment plants was available. The models based upon this sample were developed as primary treatment plants:

$$\text{Ln } Y''_p = 12.93509 - 0.09734 \text{ Ln } X_2 - 2.09333\ D_1 - 0.22875\ D_2 \qquad \text{(IV-18)}$$

as secondary treatment plants:

$$\text{Ln } Y''_s = 11.99740 - 0.54917 \text{ Ln } X_2 + 0.20309 \text{ Ln } X_3 - 0.10770\ D_1 - 0.10804\ D_2 \qquad \text{(IV-19)}$$

where Y_p'' = construction cost per design MGD of primary industrial waste treatment plants in 1957-59 dollars

Y_s'' = construction cost per design MGD of secondary industrial waste treatment plants in 1957-59 dollars

X_2 = design flow in MGD

X_3 = design influent BOD in mg/L

$D_1 = 0$, $D_2 = 0$ for petroleum wastes

$D_1 = 1$, $D_2 = 0$ for pulp and paper wastes

$D_1 = 0$, $D_2 = 1$ for chemical wastes.

CHAPTER V

OPTIMAL WATER QUALITY MANAGEMENT MODEL

Is water pollution control worth what it costs? The answer to this question has not been given completely, but there is a great amount of concern among the general public and the federal and local governments to control water pollution in spite of the costs involved. State and regional water quality management agencies are charged with the responsibility of preserving and improving the quality of waters under their jurisdiction. A water pollution control program undertaken by an agency should be legally equitable, technically feasible and economically efficient.

In the water quality management planning of a river basin, the planners are faced with the main task of determining and allocating the waste assimilative capacities of the streams in the river basin among the waste dischargers who dispose of their liquid wastes by diluting them in the stream waters. Determination of waste assimilative capacity of a stream has been described previously. Allocation of the waste assimilative capacities of the streams among the waste dischargers involves determining the amount of wastes that each discharger can release into the stream water for dilution and disposal without violating the water quality standards of the stream. In most cases, a discharger would be required to treat his wastes before disposing them. The planners will have to determine the levels of treatment for the waste dischargers such that the water quality objectives of the river basin plans are met. This planning process is sometimes called the "allocation of waste loads." The aim of the planners is the "optimal" allocation of the waste loads which is a system of allocating the waste loads to the waste dischargers at a total minimum cost to the river basin. The cost incurred to an individual discharger varies with the method of allocation used. Planners in the past have generally used three methods of allocation of waste loads. They are:

1. Uniform treatment method.
2. Cost minimization method.
3. Zoned treatment method.

The uniform treatment method: The uniform treatment method requires each discharger to treat his wastes to the same level before discharging them to the river waters. This is a commonly used method in the present water quality management programs. The most important advantage of this method is the ease of administration of the program. However, this advantage is outweighed by the disadvantages of being economically inefficient and inequitable. The assimilative capacity of the stream waters is not fully utilized because discharges at non-critical points are required to treat the wastes to the same level as the ones at critical points. Such an inefficient use of the streams' capacity is expensive! Also, under such a program, no consideration is given to the differences in the costs of waste treatment even though each source of waste is required to treat the wastes to the same level.

Cost minimization method: Under this method, treatment levels for dischargers are determined in such a manner that the overall cost to the river basin for the treatment of wastes is minimized while maintaining a desired quality of water in the river basin. Under this program, no unnecessary treatment is required at a source. This method is equitable in the sense that a source does not have to increase the level of treatment of its wastes if the discharge of the wastes do not lower the quality of the water in the stream. However, this method can be inequitable. Suppose that two waste treatment plants are located close to each other along a river and that both are discharging essentially the same type of wastes. The marginal effects of the wastes on the river system can be expected to be almost same. Now, if the treatment costs at one plant are lower than those at the other, the plant with the lower treatment cost would be required, under this plan, to treat its wastes to a higher degree of removal than the other plant. In some situations, the plant with a higher cost of waste treatment may not be required to treat its wastes. Because of the unequal cost burdens, this method is difficult to implement.

Zoned treatment method: This method is a compromise between the uniform treatment method and the cost minimization method. A river basin is divided into a number of zones. Dischargers in a zone are required to treat their wastes to the same level of treatment. The level of treatment for each zone is determined in such a manner that the overall cost to the river basin is minimized, while maintaining the desired quality of water in the river basin. If only one zone is established, then the method reduces to the uniform treatment method. If zones are established for each source, the method reduces to the cost minimization method.

This method is equitable in that waste sources located near one another are treated similarly. There are no rigid criteria for establishing the zones. Zones can be established on a categorical basis instead of a geographical basis. Under such a zoning method, all the plants under one category treat their wastes to the same level. For example, all paper mills can be placed in one category while all municipal plants can be placed in another category and so on. The planners should form zones based upon the needs of the water quality management of the river basin.

This method has the advantage of being almost as easy to administer as the uniform treatment method. Also, the requirement of same level of treatment for sources located in the same zone leads to a more equitable allocation of waste loads.

The objective of an optimal water quality management program allocating waste loads to the dischargers is either to maximize the benefits of or to minimize the costs of water pollution control program. There are numerous difficulties associated with the measurements of benefits of a water quality improvement program. The social and political factors involved are not quantifiable, nor are they fully comprehensible. However, the engineering estimates of the costs to achieve various levels of water quality control can generally be made with a fair degree of accuracy (56). The optimality criterion is then to determine the degree of waste removal at various waste sources so as to attain a specific level of dissolved oxygen in a river basin for a minimum amount of regional expenditure. Dissolved oxygen is used as an indicator of water quality when non-conservative parameters such as BOD, nutrients, etc., are considered.

A water quality management plan has to work within certain constraints that determine the alternatives available to implement it. In systems terminology, the constraints determine the boundaries of the system of technical management measures adopted by the planners. These constraints are discussed below.

1. The general public has come to a stage of awareness about the potential hazards of various kinds of pollution to the environment that it is inconceivable to permit a source of liquid wastes to discharge the wastes to a

stream or any other waterbody without any prior treatment of the wastes. Also, to preserve the quality of waters, no waste discharger should be permitted to lower the level of treatment presently employed by him. Many authors have advanced the requirement of at least a primary treatment of liquid wastes prior to discharge in a waterbody. Therefore, a river basin planning authority can formulate a constraint of requiring a source of wastes to treat its wastes to a level which is greater than the primary treatment level and is equal to or greater than the present level of waste treatment at the source.

2. The presence of fecal coliform bacteria generally indicates fresh and possibly dangerous pollution. An analysis for total coliforms would indicate the presence of feces of human and warm-blooded animals, and coliforms associated with vegetation, soils, sewage, storm water drainage, surface water runoff and others. The parameter of the bacterial pollution is a coliform most probable number (MPN) or a bacterial count per 100 ml of sample. The Oklahoma's water quality standards (40) state:

   > Bacteria: In areas designated as G_1 (recreation, primary body contact) or A (public and private water supply), bacteria of the fecal coliform group shall not exceed a monthly geometric mean of 200/100 ml,...
   > In areas designated G_2 (secondary body contact) bacteria of the fecal coliform group shall not exceed a monthly geometric mean of 1000/100 ml...

   Besides fecal coliform bacteria, many other bacteria and microorganisms such as viruses may be present in stream water. A water pollution control program should include measures to control the number of such microorganisms below a safe limit. The Oklahoma Water Quality Standards do not mention such microorganisms. The best constraint would be that no viruses or other pathogenic bacteria be discharged to a stream.

3. Toxic substances are a group of substances that are potentially very harmful to aquatic life even in small quantities. The Oklahoma Water Quality Standards state:

> Toxic substances: Toxic substances shall not be present in such quantities as to cause the waters to be toxic to human, animal, plant or aquatic life, nor detrimental to any beneficial use including continued ingestion by livestock or continued use for irrigation...

Much work has been done in the area of toxic limits of various elements; however, no universally accepted standards exist. Environmental Protection Agency has adopted a list of standards for toxic substances (60). The Oklahoma Standards state:

> ...[T]he toxic limit shall not exceed one tenth of the 96 hour median tolerance limit for the most sensitive species common to the stream. In the absence of information on the most sensitive species, the concentration shall not exceed one tenth of the 96 hour median tolerance limit to <u>Pimephales</u> <u>promelas</u> (Fathead Minnow) and/or <u>Lepomis</u> <u>Macrochirus</u> (Blue Gill).

About synergistic effects,

> ...The following materials may have synergistic effects: ammonia, cadmium, hexavalent chromium, trivalent chromium, copper, cyanide, lead, mercury, nickel, selenium, silver, and zinc. These substances shall not be present in sufficient concentration to allow the cumulative relationship value to exceed the

numerical value of one. The cumulative relationship value (CRV) is defined as

$$CRV = \frac{c_a}{L_a} + \frac{c_b}{L_b} + \ldots + \frac{c_n}{L_n}$$

where c_a, c_b, ..., c_n are the measured concentrations in the streams and L_a, L_b, ..., L_n are respective maximum permissible concentrations if each constiuent were present alone.

4. Stream standards and effluent standards are an important part of the water quality management of a river basin. Stream standards are based on a system of stream classification based upon the intended use of the water. Most stream standards relate to fecal coliform, pH, dissolved oxygen, phenolic compounds, and temperature. Effluents standards are used either to restrict the amount of pollutants discharged into a surface waterbody or to specify the degree of treatment required before discharge. In-stream standards and effluent standards are used as constraints in the water quality management programs.

Dissolved oxygen concentration is one of the most important parameters of water quality. The Oklahoma Standards state:

> Dissolved Oxygen: The dissolved oxygen concentration shall not be less than 5 mg/L for all warm waters, and 6 mg/L for those waters designated as small-mouth bass or trout fisheries. ... The dissolved oxygen shall not be less than 2 mg/L within the mixing zone. ... The dissolved oxygen content of a return flow stream shall not be less than 2 mg/L.

For minerals like chlorides, sulfates and total dissolved solids, the Oklahoma Standards state:

Minerals: For chlorides, sulfates and total dissolved solids the arithmetic mean of the concentrations of the samples taken for a year at any point shall not exceed one standard deviation greater than the arithmetic mean of the historical data generated at that point. Not more than one in twenty samples randomly collected shall exceed two standard deviations greater than the arithmetic mean of the historical data generated at that point.

The stream standards are most important in a waste load allocation program. The effluent standards should be treated as upper-limit constraints. Industrial waste discharges are guided by the interim guidelines set by the Environmental Protection Agency's Office of Permit Programs (61).

Water Quality Management Modeling

Linear and non-linear programming techniques have been used to formulate the water quality management models. Some authors have also used the dynamic programming, integer programming or some stochastic techniques to obtain "optimal" solutions of their models. A water quality management model generally consists of one or more submodels or components to analyze the following aspects of a river basin planning

1. Water quality analysis.
2. Economic-cost analysis.
3. Hydrology and other basin characteristics.
4. Optimality-constraints, solutions, etc.

Under the water quality modeling, many water quality management models have been reviewed mainly from the water quality modeling standpoint. These models will not be reviewed here again; instead, a very recent study will be reviewed. This study is a typical representative of the recent trends and efforts in the field of water quality management modeling.

Study of a Water Quality Management Model

Figure V-1 is a schematic diagram of the regional residuals management model developed by Spofford, Russell and Kelly (52). This model is deterministic and steady state. Only one season is considered for study at a time. The management model, as previously developed by Spofford (51), has three major components:

1. A linear programming industry model that relates the inputs and outputs of various production processes and consumption activities at specified locations within a region. This model also considers the amounts and types of residuals generated by the production of each product, the costs of transforming these residuals from one form to another, the costs of transporting these residuals from one place to another within the region, and the cost of any final discharge-related activity such as landfill operations.
2. Physical, chemical, and biological models of the natural world. These are referred to as the "environmental models."

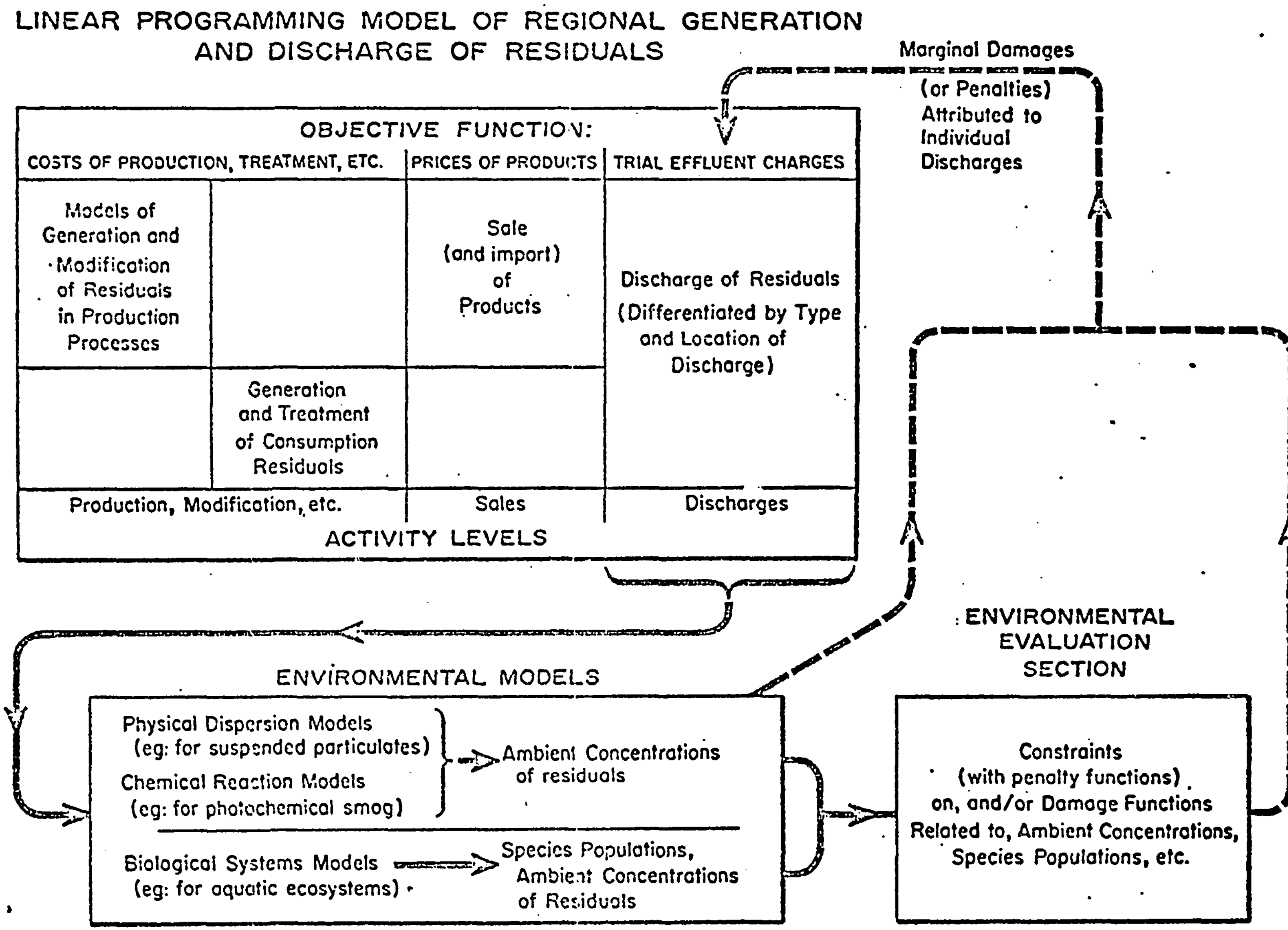


Figure V-2. SCHEMATIC DIAGRAM OF THE REGIONAL RESIDUALS MANAGEMENT MODEL (52).

3. Environmental evaluation section: The ambient concentrations of residuals and population sizes of species which are predicted by the environmental models, are either translated into damages or are compared to ambient quality standards.

This management model is static from an economic point of view. The model includes two management alternatives: (1) No treatment, and (2) On-site treatment. The model, however, does not consider the alternatives such as low flow augmentation, instream aeration, and regional sewage treatment. The model has a non-linear simulation sub-model of the natural world within its optimization framework.

The optimization technique used by the authors is a form of the gradient search method of non-linear programming and involves iterating through the system of three sub-models, namely residuals generation and discharge sub-model, environmental sub-model, and environmental evaluation sub-model. The iterative process as described by the authors is useful in understanding the working of the model.

> At iteration k, the generation and discharge sub-model, which is structured as a linear programming problem, is solved using a set of effluent charges which is based on the state of the natural world on the $(k-1)^{th}$ iteration. The resulting discharges are passed to the environmental sub-models which transform them into information on ambient concentrations and species populations. These data on the resulting state of the natural world are then compared to exogenously specified standards of environmental quality. Penalty functions are used to reflect the solution's failure in meeting these standards, marginal penalties associated with each discharge of each type of residual are computed and returned to the generation and discharge

model as prices on residuals discharges for the $(k+1)^{st}$ iteration. When all the constraints are met (within some predetermined tolerance) and no further improvement in the objective function is possible, successive sets of both discharges and effluent charges will be the same, and the algorithm has found an optimum.

The authors admit that the major problem in handling the model is its size as related both to round-off error in matrix inversion and to computer time required for solution.

The non-linear sub-model of the aquatic ecosystem is capable of including as endogenous variables the sizes of species populations and the concentrations of certain materials. The approach to modeling the ecosystem is based on the principle of mass continuity. The components of the ecosystem are grouped in classes or "compartments" according to their function; each class is represented in the model by an endogenous, or state, variable. Eleven endogenous variables are designated for the model, namely nitrogen, phosphorus, turbidity (suspended solids), organic material, algae, bacteria, fish, zooplankton, dissolved oxygen, toxics, and heat (temperature). The exogenous variables are: turn-over rate (or advective flow), and the inputs of the eleven abovementioned variables. The material flows among the compartments within a given reach of a river are shown in the Figure V-2. The outputs of the model are densities of fish biomass, algal densities, and the dissolved oxygen levels.

The mathematical description of material transfers among the compartments is based on empirical formulations, each compartment requiring a separate differential equation

Notation:

N = nitrogen
P = phosphorus
A = algae
B = bacteria
H = zooplankton
F = fish
L = organic matter
O = oxygen

Note: The three remaining endogenous variables, heat (temperature), toxics, and suspended solids, are assumed to affect the rates of material transfers among these compartments.

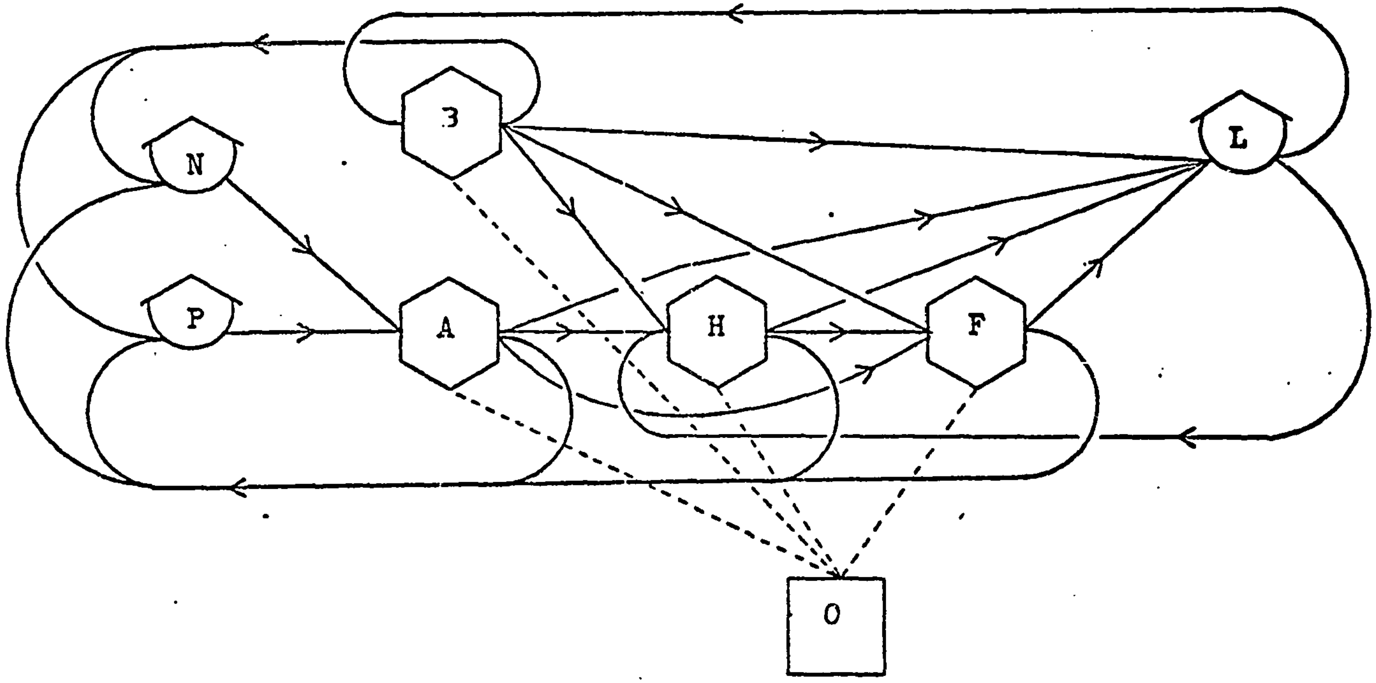


Figure V-2. Diagram of materials flows among compartments within a single reach. (52)

to describe mass continuity. A steady state solution and an environmental response matrix of the differential equation set are required. The authors suggest two ways to obtain steady state solutions: (1) the simultaneous simulation of a non-linear differential equation set, and (2) the simultaneous solution of a set of non-linear algebraic equations. Neither of these, however, guarantees a stable point equilibrium. Even when a steady state solution can be found, an additional problem is that there may be more than one stable point equilibrium. The problem with obtaining a response matrix is the computations involved in obtaining the inverses of the matrices for each state of the natural world. Thus, it becomes a problem of the computer time.

The authors admit that the non-linear representations of the ecosystem increase the complexity and the costs, but they feel that the non-linear representation adds to the realism and the predictive capability of the model.

The Water Quality Management Model

The model developed by Spofford, Russell and Kelly is an elaborate model. As a matter of fact, a majority of the available water quality models are elaborate. They are large, require a large amount of data and are expensive to operate in terms of computer time and man time. The needs of the river basin planning, however, dictate the use of a model that is capable of working with essentially inadequate data with a minimum of cost. Such a model would not be mathematically exact in the sense that model would be a simplification of the natural world. The equations will be rather empirical in nature, instead of being the exact relationships of the complex physical, chemical, and biological processes of the natural world. Planning authorities have realized through their experience that the so called "exact" mathematical models too often defeat the purpose of planning because they are not capable of operating on the inadequate and inexact data available to the planners. A regional model that is "macroscopic" in nature (in terms of data requirements) provides solutions that can be very valuable in planning.

To develop a regional water quality model, consider a river basin with a terminal flow Q_t and having a number of waste discharges throughout the basin. The objective of the model is to determine the level of treatment required of each source of wastes in the basin in such a fashion that the total cost to the region is minimized while obtaining a desired level of water quality in the river system.

In mathematical terms:

$$\text{Minimize } c = \sum_{i=1}^{n} c_i \quad i = 1, 2, \ldots, n$$

where c_i is the cost of treatment to the waste discharger i. c_i is a function of the level of treatment e_i required at the source i; i.e.,

$$c_i = f(e_i)$$

The level of treatment should be such that the required level of quality in the river system is achieved.

The model as it stands cannot be solved. The cost function is non-linear and unknown. Use is made of the quality models and the cost models to formulate the optimal allocation model. Also, the constraints are expressed in mathematical terms.

The quality model determines the level of waste removal for the entire river system. Let this be represented as E. If the waste load at a source i is represented as w_i, then

$$W = \sum_{i=1}^{n} w_i$$

The total amount of wastes to be removed is

$$W \times E$$

If the required treatment at a source i is e_i then the total waste removal is

$$\sum_{i=1}^{n} w_i \times e_i$$

The cost models determine the cost of a waste treatment plant in terms of population equivalent and flow. For a given city or town or an industry, the population equivalent and flow requirements are determined from other data. To achieve a certain level of waste removal, a certain type of plant must be installed. The cost models then determine the costs of the proposed plant. Thus, to determine the cost of waste treatment, the type of plant required should be known. And the type of plant depends upon the degree of treatment required. The cost models do not provide a direct relationship between the levels of treatment and the costs of treatment. However, if we assume that the cost of treatment is higher for a higher degree of treatment and lower for a lower degree of treatment, then we can proceed as follows.

The objective function is represented as

$$\text{Minimize} \quad \sum_{i=1}^{n} e_i$$

subject to

$$\sum_{i=1}^{n} w_i \times e_i \geq W \times E$$

Other constraints can be formulated and the problem is written as

$$\text{Minimize} \quad \sum_{i=1}^{n} e_i$$

subject to

$$\sum_{i=1}^{n} w_i \times e_i \geq W \times E$$

$$e_i > e_{pi}$$

$$e_i > e_{minimum}$$

where e_{pi} = the present level of treatment at the source i

$e_{minimum}$ = the minimum level of treatment required of all sources

The solution to this problem is not necessarily the minimum cost solution. Generally, a number of "optimal" solutions of the above problem are obtained. Each solution then must be examined to determine the costs of the treatments. The solution that has the minimum cost is the solution sought after.

The above formulation is a general one. The next chapter shows how the optimal allocation model can be applied to a specific river basin. Generally, in a specific case, numerical data are substituted and depending upon the situation, more constraints can be added. Also, the allocation model can be applied a number of times if the river basin is divided into divisions for planning purposes.

The working of the water quality management model is shown by Figure V-3. For river basin planning, the management model requires the following data:

1. Streams:
    a. Names and locations of the major streams.
    b. Names and locations of the tributaries.
    c. Lengths of all streams.
    d. Average velocity of flow in the major streams.
    e. Stream flows (gaging stations)
    f. Low flows in the streams.
    g. Slope of stream beds, etc.

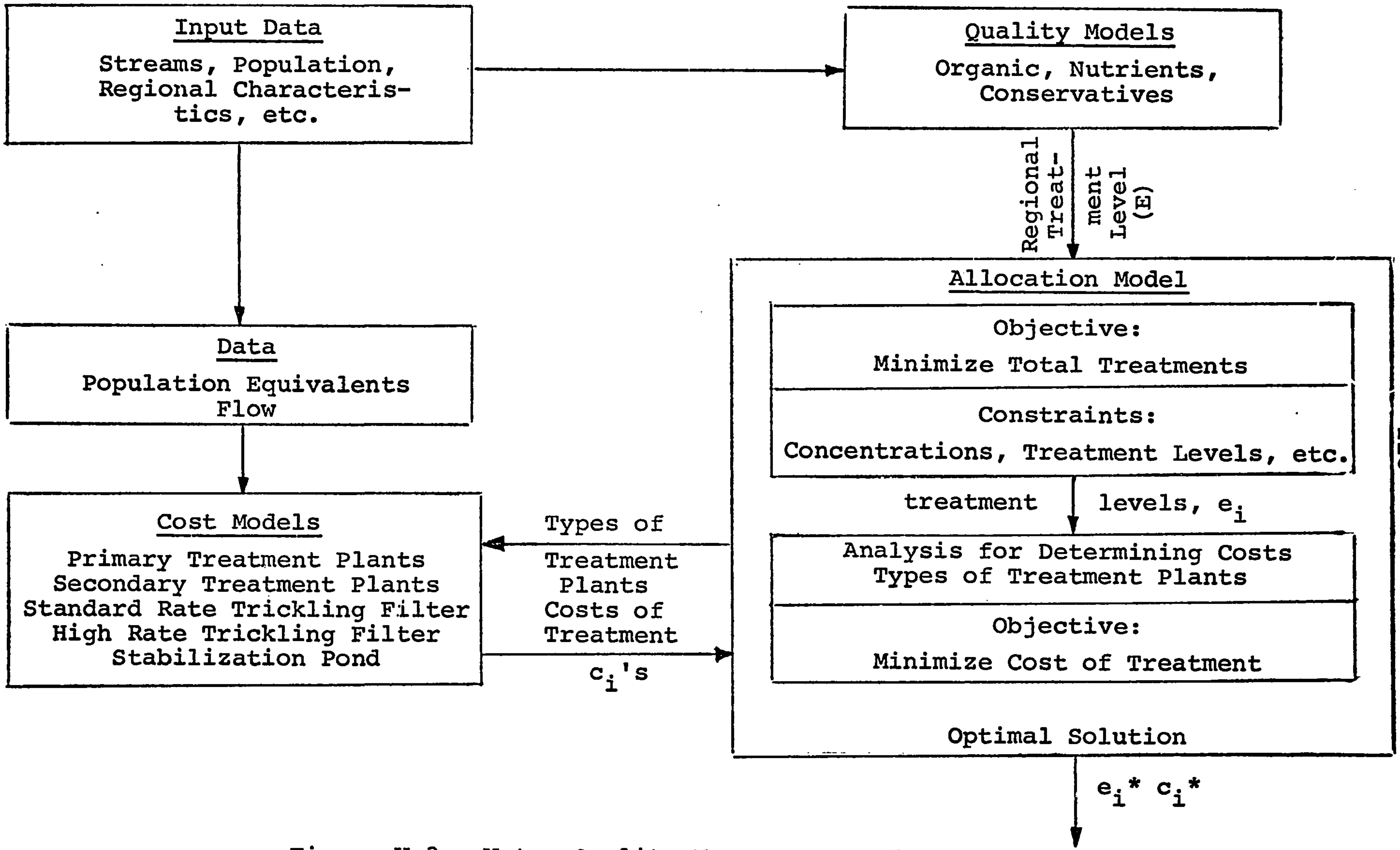


Figure V-3. Water Quality Management Model

2. Population:

   a. Population of the river basin.

   b. Populations of counties, subdivisions and municipalities in the river basin.

3. Geography - Geology:

   a. Topographical maps of the basin.

   b. Maps of streams in the basin, drainage areas of the major streams.

   c. Map of the basin with counties, subdivisions and municipalities.

   d. Map of the locations of the waste dischargers.

4. Waste Loads:

   a. Name and location of the waste discharger.

   b. Name of the receiving stream, the point of discharge.

   c. Design data of the plant--flow, PE, etc.

   d. Influent and effluent analysis.

   e. S.I.C. code number and discharge permit number in case of an industrial plant.

   f. Type of treatment employed--screening, primary clarifier, trickling filter, activated sludge, stabilization pond, aerated lagoon, Imhoff tank, etc.

5. Water Quality:

   a. Historical water quality in a stream if available.

   b. Data from the sampling stations.

   c. Required levels of quality parameters.

6. Other Regional Characteristics:

   a. Reaeration coefficients.

   b. Efficiency term ratios.

   c. Number of reoxygenation volumes, etc.

The water quality management model presented here has several advantages. They are:

1. The model is regional in nature, applies to a whole region.
2. The data requirements are macroscopic or regional.
3. The model is extremely simple in its construction and working.
4. The underlying concepts are very simple.
5. The model avoids elaborate and complicated simulations generally found in other models.
6. The waste load allocation model is linear, thus linear programming can be employed with ease.
7. The cost models are simple to work with.
8. The output of the allocation model is in terms of the levels of treatment required for each source of wastes.
9. The data required for the models can be obtained with considerable ease from state and local agencies' files or federal agencies' files or from the literature.

CHAPTER VI

A STUDY OF THE VERDIGRIS RIVER BASIN

The river basin selected for the application of the model developed in the preceding chapters is the part of the Verdigris River basin that is in Oklahoma. The river basin is approximately 4,290 square miles in area and lies in the northeast corner of Oklahoma. The surface waters of this river basin consist of three major stream systems and two major reservoirs. The stream systems are:

1. The Verdigris River system.
2. The Caney River system.
3. The Bird Creek system.

The reservoirs are: (1) Hulah Lake; (2) Oologah Lake. The Caney River and the Bird Creek are the major tributaries of the main stem Verdigris in the basin. The map of the basin is shown in Figure VI-1.

The Verdigris River (41)

The Verdigris River originates in Chase County, Kansas and flows southward into Oklahoma. It joins the Arkansas River in Oklahoma near Muskogee. The Verdigris drains an area of approximately 8,303 square miles of which 4,290

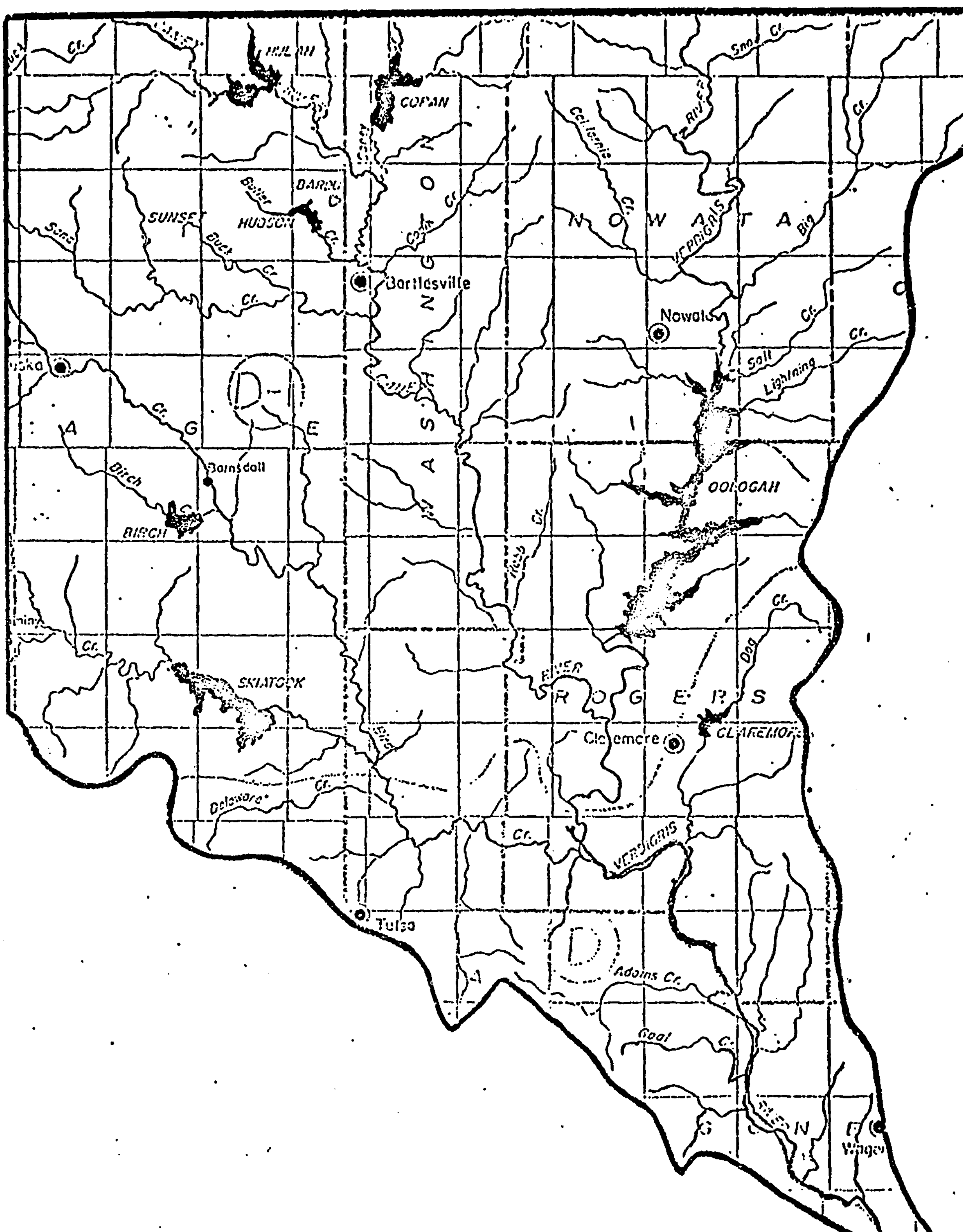


Figure VI-1. The Verdigris River Basin.

square miles lie in Oklahoma. In the past, U.S.G.S. and U.S. Army Corps of Engineers operated four gaging stations on the Oklahoma reach of the Verdigris. The gaging station at Inola reported an annual average flow of 2,915,000 acre-feet for a 25 year period ending in 1969. The following table lists the tributaries to the Verdigris.

TABLE VI-1

TRIBUTARIES TO THE VERDIGRIS

Caney River	Plumb Creek
Bird Creek	Talala Creek
Opossum Creek	Blue Creek
Snow Creek	Four-Mile Creek
Hickory Creek	Sweetwater Creek
Cedar Creek	Spunky Creek
California Creek	Adams Creek
Big Creek	Bull Creek
Salt Creek	Coal Creek
Double Creek	Gar Creek
Mappen Creek (Lightning Creek)	Billy Creek

The Caney River (41)

The Caney River flows from Kansas into Washington County, Oklahoma, and proceeds southwards to meet the Verdigris northwest of Claremore. The Caney River drains an area of approximately 2,111 square miles of which 1,616 square miles lie in Oklahoma. The U.S.G.S. gaging station near Ramona reported an annual discharge of 623,000 acre-feet for a 27 year period ending in 1969. The following table lists the tributaries to the Caney River.

TABLE VI-2

TRIBUTARIES TO THE CANEY

Buck Creek	Coon Creek
Pond Creek	Mission Creek
Turkey Creek	Little Caney River
Hickory Creek	Sand Creek
Fish Creek	Rabb Creek
Hog Shooter Creek	Saunder Creek
Curl Creek	Horsepen Creek
Double Creek	

The Bird Creek (41)

The Bird Creek originates near Personia in Osage County, Oklahoma, and travels southeastward to meet the Verdigris 6 miles south of Claremore. The Bird Creek drains an area of approximately 1,147 square miles all of which lie in Oklahoma. The Bird Creek has two continuous gaging stations located at Avant and near Sperry. The station near Sperry gaged an average annual flow of 318,100 acre-feet for 31 year period ending in 1969, with periods of no flow at times in the years of 1939, 1954, 1964 and 1966. The following table lists the tributaries to the Bird Creek.

TABLE VI-3

TRIBUTARIES TO THE BIRD CREEK

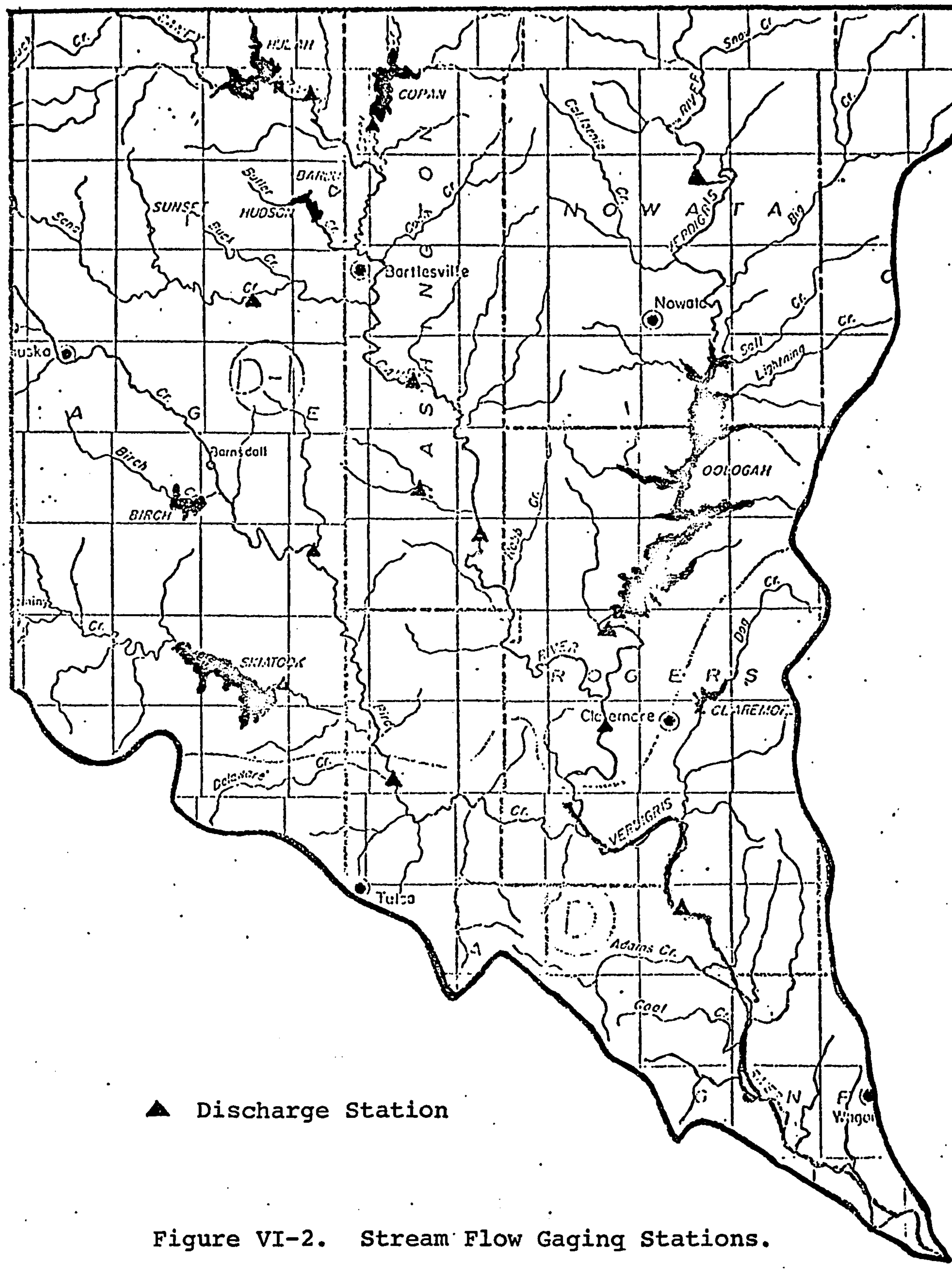
Middle Bird Creek	Charley Creek
Clear Creek	Hominy Creek
Nelagoney Creek	Delaware Creek
Choteau Creek	Flat Rock Creek
Birch Creek	Ranch Creek
Dogthresher Creek	Mingo Creek
Candy Creek	Elm Creek
Marshall Creek	

Figure VI-2 shows the stream flow gaging stations in the basin Figures VI-3, VI-4 and VI-5 show the daily discharge duration curves and monthly and yearly discharges for the stream flow gaging stations number 7-1710, 7-1765, and 7-1786, respectively.

Many of the tributaries listed above in Tables VI-1, VI-2 and VI-3 have their own tributaries. Some of them have second order and third order tributaries.

Hulah Lake (41)

Hulah Lake was built by U.S. Army Corps of Engineers and is located in northern Osage County about ten miles northwest of Bartlesville. Hulah Lake provides flood protection for about 57,000 acres of land in the Caney River valley downstream from the dam. It reduces flooding along the Verdigris River downstream from the mouth of the Caney River, and also aids indirectly in water control along the Arkansas River. The lake supplies approximately 11 million gallons a day of water supply to Bartlesville. The reservoir impounds the Caney River with a drainage area of 732 square miles. The maximum storage capacity of the reservoir is 2.926×10^5 acre-feet and the average yield of the lake is 16.9 million gallons per day. The reservoir serves the purposes of: (1) flood control; (2) water supply; (3) recreation; (4) fish and wild life; and (5) water quality control.



▲ Discharge Station

Figure VI-2. Stream Flow Gaging Stations.

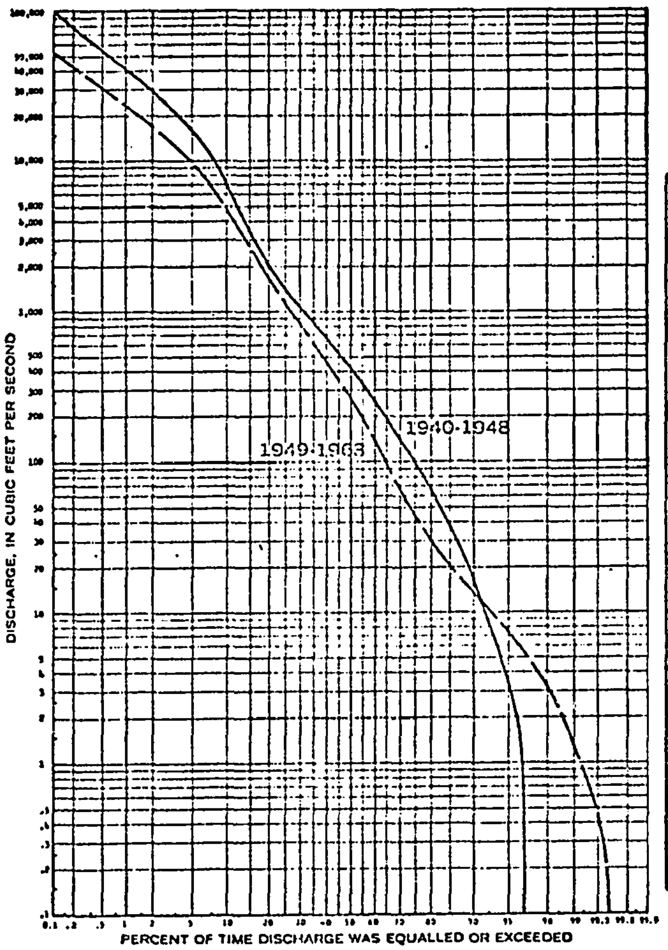


Figure VI-3.

SURFACE WATER RECORDS AND DURATION CURVE OF DAILY DISCHARGE

STREAM GAGING STATION 7-1710 VERDIGRIS RIVER NEAR LENAPAH

LOCATION—Lat. 36°51'05", long. 95°35'06", at downstream side of county road bridge, 2.8 miles east of Lenapah, and at mile 144.6.

DRAINAGE AREA—3,639 sq. mi.

AVERAGE DISCHARGE—9 years (water years 1940-48), 2,772 cfs; 20 years (water years 1949-68), 1,767 cfs; 30 years (water years 1939-68), 2,022 cfs (1,464,000 ac.-ft. per year).

DAILY DISCHARGE—1940-48: maximum, 134,000 cfs; median, 460 cfs; minimum, no flow. 1949-68: maximum, 114,000 cfs; median, 255 cfs; minimum, no flow.

REMARKS—Flow regulated since 1949 by reservoirs in Kansas (Fall River, 1949; Toronto 1960; Elk City, 1966), which have a combined drainage area of 1,949 sq. mi.

MONTHLY AND YEARLY DISCHARGE, IN ACRE-FEET

Water Year	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	The Year
1955	75,240	1,920	1,490	18,800	12,220	17,690	7,310	164,600	63,810	5,230	334	4,020	372,700
1956	76,880	419	470	669	876	328	591	3,060	28,240	9,190	222	26	121,600
1957	127	863	558	408	3,740	3,180	128,500	627,200	926,100	64,530	5,120	9,520	1,770,000
1958	19,170	42,450	14,510	22,470	43,890	627,900	287,000	158,500	61,760	384,600	20,150	71,640	1,754,000
1959	6,850	28,750	10,610	13,770	27,950	33,110	109,300	183,100	24,320	568,000	61,630	32,950	1,110,000
1960	652,700	38,840	38,780	81,730	133,000	255,400	219,700	235,900	157,500	11,630	59,720	10,570	1,934,000
1961	57,140	91,160	101,700	36,170	63,440	210,900	315,500	1,526,000	179,100	154,100	49,810	717,000	3,432,000
1962	323,400	634,000	249,500	132,200	169,400	131,000	49,080	15,220	120,600	28,130	3,280	262,200	2,118,000
1963	125,900	23,530	36,510	93,550	21,260	121,600	17,520	15,720	32,870	5,910	5,020	1,450	500,900
1964	1,420	1,610	1,310	1,200	1,550	1,390	45,990	14,760	130,800	1,850	7,230	4,770	213,900
1965	944	282,000	92,980	42,110	24,630	88,560	506,700	63,220	531,700	21,580	3,730	207,400	1,866,000
1966	11,930	2,730	10,270	12,140	22,890	33,190	21,950	63,740	37,890	3,000	14,790	1,820	236,400
1967	1,300	1,340	1,800	3,960	2,710	2,000	17,650	29,900	239,400	213,100	38,230	162,200	715,600
1968	147,900	166,100	45,830	41,250	28,920	79,650	146,000	167,500	127,000	82,560	191,500	9,810	1,234,000
1969	80,770	292,600	165,200	150,600	114,500	293,200	350,500	399,700	743,700	190,500	7,610	46,810	2,836,000

Figure VI-4.

SURFACE WATER RECORDS AND DURATION CURVE OF DAILY DISCHARGE

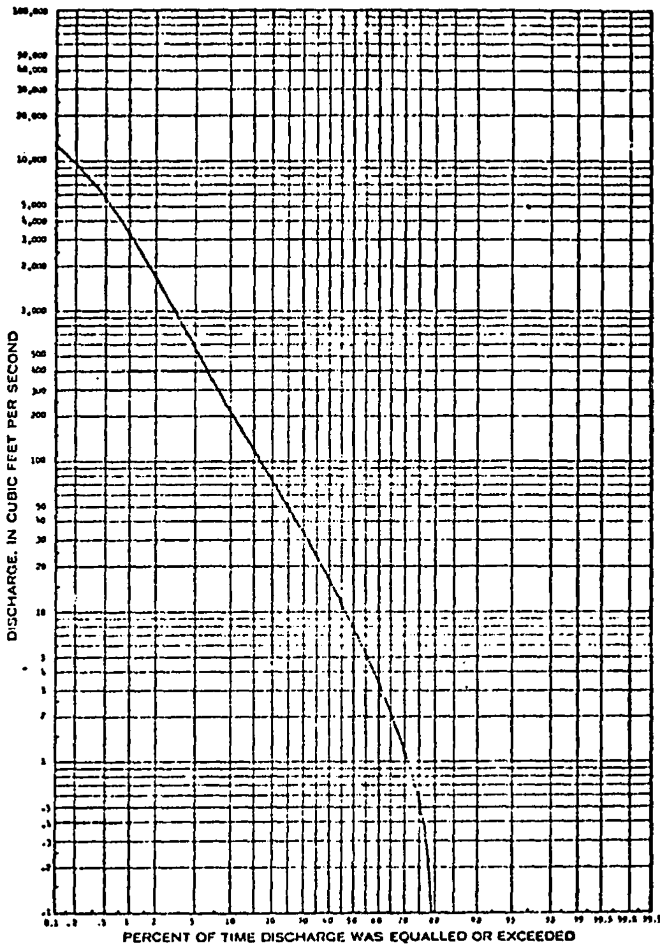
STREAM GAGING STATION 7-1765 BIRD CREEK AT AVANT

LOCATION—Lat. 36°29'11", long. 96°03'45", at downstream side of county road bridge at Avant, 1.5 miles upstream from Candy Creek, and at mile 54.2.

DRAINAGE AREA—364 sq. mi.

AVERAGE DISCHARGE—23 years (water years 1946-68), 159 cfs (115,100 ac.-ft. per year).

DAILY DISCHARGE—maximum, 20,800 cfs; median, 7.1 cfs; minimum, no flow.



MONTHLY AND YEARLY DISCHARGE, IN ACRE-FEET

Water Year	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	The Year
1955	1,660	63	51	263	640	7,330	2,420	57,630	3,600	32	87	35	73,810
1956	3,660	0	0	0	2	0	0	0	330	0	0	0	3,990
1957	0	0	0	0	0	672	42,430	139,400	157,600	10,410	6,160	5,500	362,200
1958	365	707	251	1,040	340	31,050	12,700	3,770	164	5,230	689	1,120	57,430
1959	6	429	32	36	167	2,580	1,660	39,760	5,650	51,150	361	3,210	105,000
1960	117,400	2,360	16,260	7,040	12,170	20,730	18,290	38,540	5,660	361	1,430	69	240,300
1961	721	1,070	4,280	382	809	8,180	5,260	75,940	17,630	33,700	54,240	94,340	296,600
1962	[illegible]	56,450	21,870	4,200	3,920	10,510	10,100	998	11,680	1,190	997	34,820	167,500
1963	[illegible]	2,160	1,030	5,660	659	5,760	2,360	2,020	63	27	49	0	22,550
1964	[illegible]	0	7	102	113	164	9,730	2,430	2,450	101	9,240	2,070	26,410
1965	81	9,790	3,930	9,090	4,870	4,400	39,040	14,550	3,770	3,040	0	13,850	106,400
1966	45	0	243	212	259	2,240	134	6,830	2,650	21	1,380	4,990	19,000
1967	4	0	1	327	155	43	5,050	11,740	12,480	26,280	1,840	9,900	67,810
1968	[illegible]	2,330	960	5,630	3,710	44,060	35,410	14,150	9,370	1,340	234	0	126,500
1969	[illegible]	12,360	10,930	4,640	11,970	18,690	21,370	2,790	79,130	3,450	33	4,270	170,100

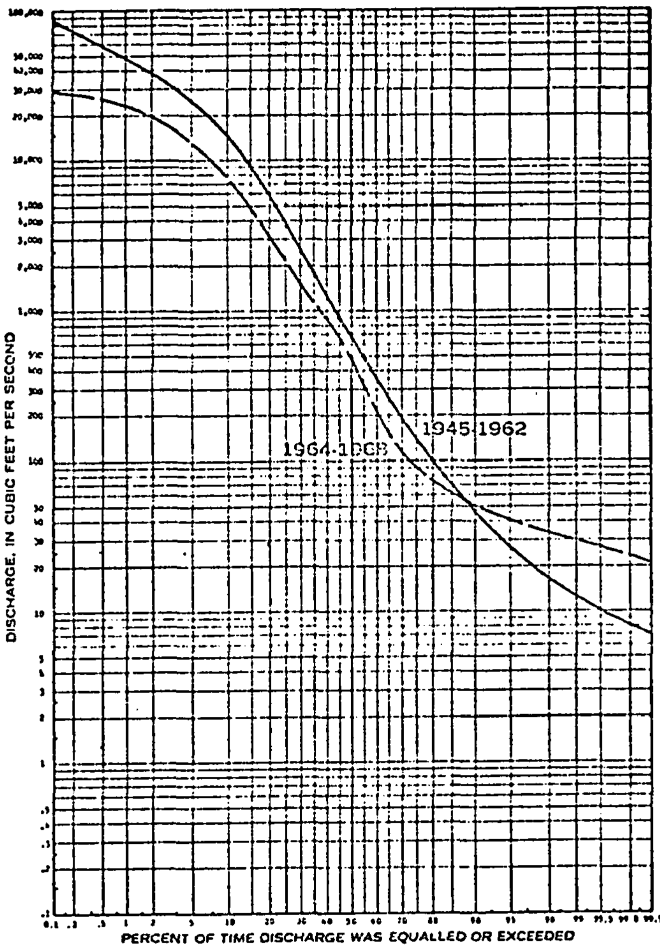


Figure VI-5.

SURFACE WATER RECORDS AND DURATION CURVE OF DAILY DISCHARGE

STREAM GAGING STATION 7-1786 VERDIGRIS RIVER NEAR INOLA

LOCATION—Lat. 36°03'45", long. 95°37'10", at downstream side of bridge on State Highway 33, 6 miles west of Inola, and at mile 48.8.

DRAINAGE AREA—7,911 sq. mi.

AVERAGE DISCHARGE—18 years (water years 1945-62), 4,483 cfs; 5 years (water years 1964-68), 2,325 cfs; 24 years (water years 1945-68), 3,893 cfs (2,822,000 ac.-ft. per year).

DAILY DISCHARGE—1945-62: maximum, 114,000 cfs; median, 670 cfs; minimum, 6.2 cfs. 1964-68: maximum, 28,200 cfs; median, 560 cfs; minimum, 19 cfs.

REMARKS—*Flow partly regulated since May 1963 by* Oologah Reservoir, 41.5 miles upstream. The lake has a drainage area of 4,339 sq. mi. and was filled principally during 1963.

MONTHLY AND YEARLY DISCHARGE, IN THOUSANDS OF ACRE-FEET

Water Year	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	The Year
1955	103	9	4	35	29	87	24	396	167	14	3	4	875
1956	128	2	2	2	3	2	8	22	34	10	1	1	215
1957	1	2	3	3	9	15	581	1,764	2,463	402	19	26	5,288
1958	21	54	24	42	58	1,070	590	263	67	500	52	81	2,872
1959	12	35	16	20	49	91	161	430	77	1,117	199	93	2,298
1960	1,969	117	202	200	277	551	440	703	272	72	89	19	4,911
1961	43	131	171	35	105	295	573	2,562	505	543	401	1,431	6,795
1962	595	1,189	536	208	250	300	180	45	208	46	10	462	4,079
1963	246	46	70	175	36	227	48	20	5	20	4	3	900
1964	2	3	3	3	4	7	224	37	204	10	82	42	621
1965	5	401	162	130	57	144	985	181	623	55	5	339	3,087
1966	23	4	8	17	50	75	45	108	150	5	32	53	570
1967	4	3	20	14	5	4	96	121	345	526	90	194	1,422
1968	192	251	71	129	157	488	542	239	294	104	228	27	2,722
1969	109	420	355	276	291	473	739	453	1,303	553	38	88	5,098

Oologah Lake (41)

Oologah Lake is located approximately two miles southeast of the town of Oologah in Rogers County and impounds the Verdigris River with a drainage area of 4,339 square miles. It has a maximum storage capacity of 1.519×10^6 acre-feet. The dam provides flood protection to approximately 93,000 acres downstream from the dam. The lake provides water supply to Tulsa, Collinsville, and Claremore. It also supplies water to several rural water districts and a large privately owned utility company. The average yield of the lake is 154 million gallons a day. The reservoir serves the purposes of: (1) flood control; (2) water supply; (3) recreation; (4) fish and wild life; and (5) navigation.

Five more lakes in the basin have been authorized for construction by the Congress. The U.S. Army Corps of Engineers Tulsa District, have the responsibility of preconstruction planning. The following table summarizes the information on these lakes (Table VI-4, page 126).

There are a number of municipal lakes whose primary purpose is water supply. Table VI-5 gives the information on these lakes (page 127). These lakes impound a total of 2577 acres of surface water in the basin with a storage capacity of 42,661 acre-feet.

Oklahoma has been divided into seven basins for planning purposes. The Verdigris River basin under study here is a part of the Middle Arkansas River Basin. Figure VI-6 shows the seven planning basins of Oklahoma.

TABLE VI-4

AUTHORIZED SURFACE WATER DEVELOPMENT (41)

Site Name	Stream	Drainage Area (sq mi)	Flood Control (ac-ft)	Conservation and Sediment Storage (ac-ft)	Maximum Storage (ac-ft)	Conservation Pool Area (acres)	Yield (mgd)	Purpose**
Birch	Birch	66	39,000	19,200	58,200	1,137	6.0	FC-WS-R-FW-WQC
Candy	Candy	43	30,700	41,000	72,400	2,120	8.0	FC-WS-R-FW-WQC
Copan	Little Caney	505	184,300	46,000	230,300	4,850	19.0	FC-WS-R-FW-WQC
Sand	Sand	137	51,700	39,300	91,000	1,940	13.0	FC-WS-R-FW-WQC
Skiatook	Hominy	354	182,300	331,200	513,500	10,540	76.0	FC-WS-R-FW-WQC
TOTAL		1105	488,000	476,600	965,400	20,587	122.0	

*Statistics given are for ultimate development, taken from Reference 41.

**FC - Flood Control; WS - Water Supply; R - Recreation; FW - Fish and Wildlife; P - Power (hydroelectric); N - Navigation; and WQC - Water Quality Control.

Abbreviations used: sq mi = square miles; ac-ft = acre-feet; and mgd = million gallons per day.

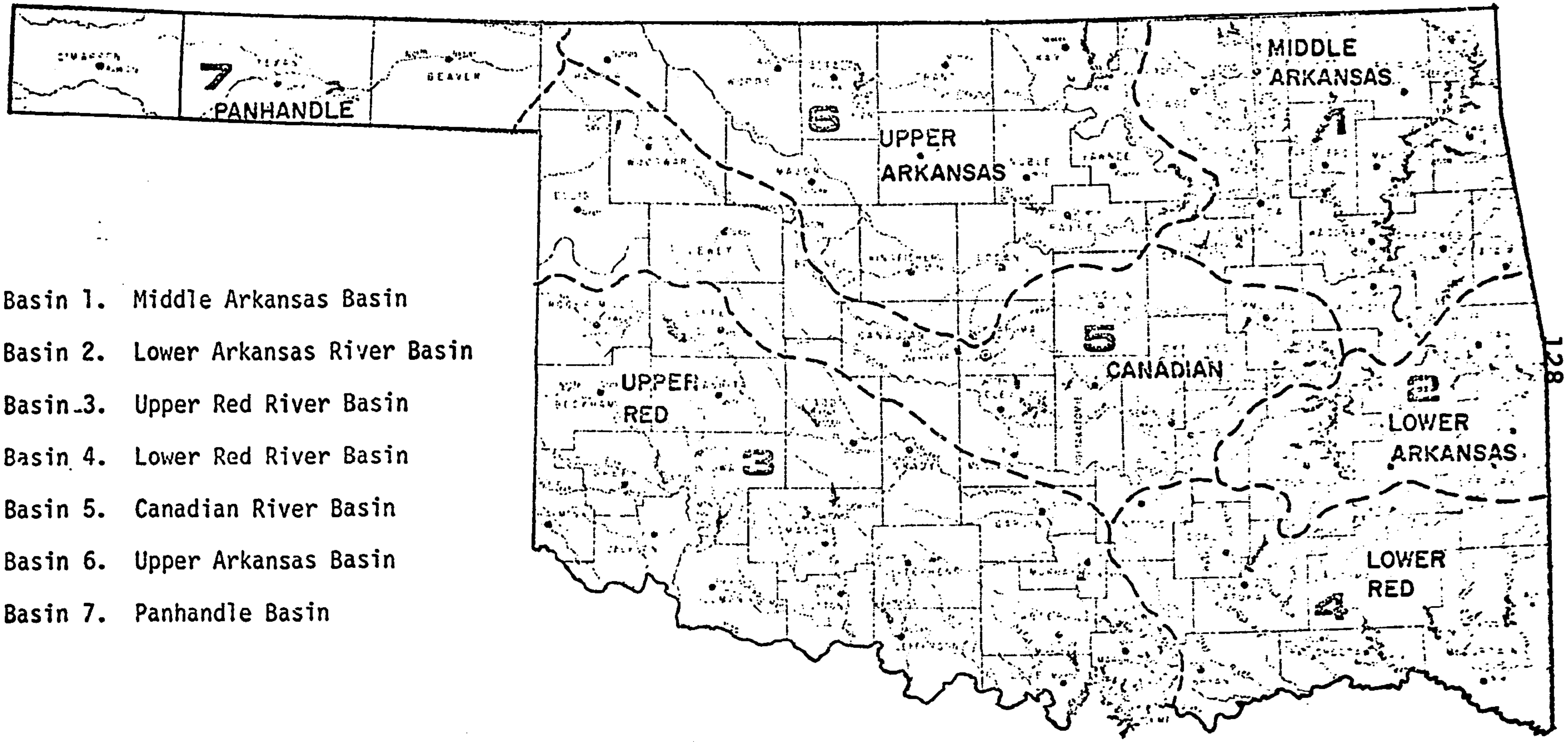
TABLE VI-5

MUNICIPAL LAKES
(Adopted from Reference 41)

	Name of the Lake	City Supplied	Lake Area (acres)	Conservation Storage (acre-feet)
(1)	Ramona	Ramona	14	70
(2)	Hudson	Bartlesville	335	5,300
(3)	Pawhuska	Pawhuska	95	2,850
(4)	Waxhoma	Barnsdall	140	2,000
(5)	Hominy	Hominy	200	5,000
(6)	Blue Stem	Pawhuska	800	17,000
(7)	Claremore	Claremore	431	2,586
(8)	Yahola	Tulsa	425	7,000
(9)	Chelsea #1	Chelsea	14	210
(10)	Chelsea #2	Chelsea	25	100
(11)	Nowata	Nowata	65	200
(12)	Hominy City	Hominy	18	270
(13)	Ochelata	Ochelata	15	75

The upper part of the Verdigris River basin, containing upper reaches of the Verdigris, Caney and Bird Creek, is mostly composed of low rolling hills with a vegetation cover of scrub oak in the eastern parts and native grass sod in the western part. The lower regions of the Verdigris River basin contains the highly urbanized Tulsa Standard Metropolitan Statistical Area (41).

The basin contains a stretch of the McClellan-Kerr Navigation System. The channel turns upstream on the Verdigris from Muskogee to a distance of 50 miles to Catoosa.



Basin 1. Middle Arkansas Basin

Basin 2. Lower Arkansas River Basin

Basin 3. Upper Red River Basin

Basin 4. Lower Red River Basin

Basin 5. Canadian River Basin

Basin 6. Upper Arkansas Basin

Basin 7. Panhandle Basin

Figure VI-6. Water Quality Planning Basins.

Climate (41)

The climate of the Verdigris River basin is generally moist and sub-humid. Spring and autumn seasons are mild with warm days and cool nights. In summer, days are long and hot while, in winter, the days are comparatively mild with occasional periods of extreme cold.

Temperature variations across the basin are slight. The mean annual temperature is 60°F. Average daily maximum temperatures average around 44°F in January and 92°F in July. The daily minimum temperatures average 26°F in January to 69°F in July.

The average annual precipitation varies from 34 inches in western Osage County to 40 inches in northwestern Nowata County. Precipitation is more general and widespread during the autumn and winter months while it is showery and scattered during spring and summer months. Mean annual snowfall in the region is from 8 to 10 inches and is fairly well distributed over the three winter months. The average annual lake evaporation is reported to be 50 inches in western Craig County to 55 inches in western Osage County. It may be of interest to note that the average annual runoff of the basin is approximately 7.5 inches.

Water Use

Tables VI-6 and VI-7 are adopted from Reference 41.

TABLE VI-6

WATER USE

County	Municipal, Industrial and Recreational (acre-feet)	Irrigation		Total (acre-feet)
		(acre-feet)	(acres land)	
Nowata	882	0	0	882
Osage	17,680	505	590	18185
Rogers	4,338	1322	1005	5660
Tulsa	8,074	259	104	8333
Wagoner	2,956	641	846	3597
Washington	857	1406	1103	2263

TABLE VI-7

PUBLIC WATER SUPPLIES

Town	Capacity (mgd)	Per Capita Use (gal)	Average Use (mgd)	Source
Barnsdall	0.72	229	0.36	Barnsdall Lake
Bartlesville	7.32	161	4.78	Hudson & Hulah Lakes
Chelsea	0.16	93	0.15	Chelsea Lakes #1 and #2
Claremore	4.0	92	0.84	Claremore Lake
Collinsville	--	175	0.53	Verdigris River
Hominy	--	91	0.21	Hominy Lake
Nowata	4.47	120	0.44	Verdigris River
Owasso	--	113	0.39	Purshased from Tulsa
Pawhuska	3.00	226	0.96	Pawhuska & Bluestem Lakes
Tulsa	--	150	49.86	Spavinaw & Eucha Lakes
Wagoner	2.59	81	0.40	Fort Gibson Lake

Water Quality (41)

Factors affecting the quality of water:

1. Geology. The kinds and amounts of minerals dissolved depend upon the availability of soluble minerals in the rock formations. Moderate to low mineral content along with calcium and bicarbonate are the distinguishing quality characteristics of the water of streams originating in the basin and not affected significantly by man's activities. The calcium and bicarbonate characteristics are probably due primarily to the solution of calcium carbonate from the sedimentary rocks. Calcium carbonate is a common material in sandstone and is the predominant mineral in limestone.
2. Man's activities. The production of oil, livestock, and the operation of municipal and industrial waste disposal plants contribute dissolved and suspended material in water. The poor quality of Bird Creek probably can be partially blamed to the activities of man in the Tulsa Standard Metropolitan Statistical Area.
3. Stream flow. Variable streamflow causes variable quality in the stream waters. Quality values are significantly high during periods of low flow. Rainfall of high intensity results in stream water, consisting primarily of overland flow, low in dissolved mineral content but high in suspended material.

Stream Water Quality

Figure VI-7 shows the locations of water quality sampling stations in the basin. There are two kinds of stations: (1) daily; and (2) periodic; but several of the stations are both daily and periodic. Table VI-8 adopted from Reference 41 shows the maximum and minimum quality values for streams for several parameters. The table shows that upper Caney and its tributaries and some of the smaller tributaries of the Verdigris River contain low to moderate amounts of dissolved solids. The water of these streams is usually of the calcium bicarbonate type and moderately hard to hard. In contrast, the middle and lower parts of the Caney River, Verdigris River, Bird Creek, and a number of their tributaries have more mineralized water. The lower part of Bird Creek, upper Hominy Creek, Caney River, and some of their tributaries have concentrations of dissolved solids of 1000 mg/L or more at times.

Table VI-9 adopted from Oklahoma Water Quality Standards (40) shows the yearly mean standard for chloride, sulfate and total dissolved solids at several important points in the basin.

The quality of water determines to a great extent the beneficial use of the water made. Oklahoma water quality standards designate the beneficial uses as listed in Table VI-10, below. Limitations placed on streams are listed in Table VI-11.

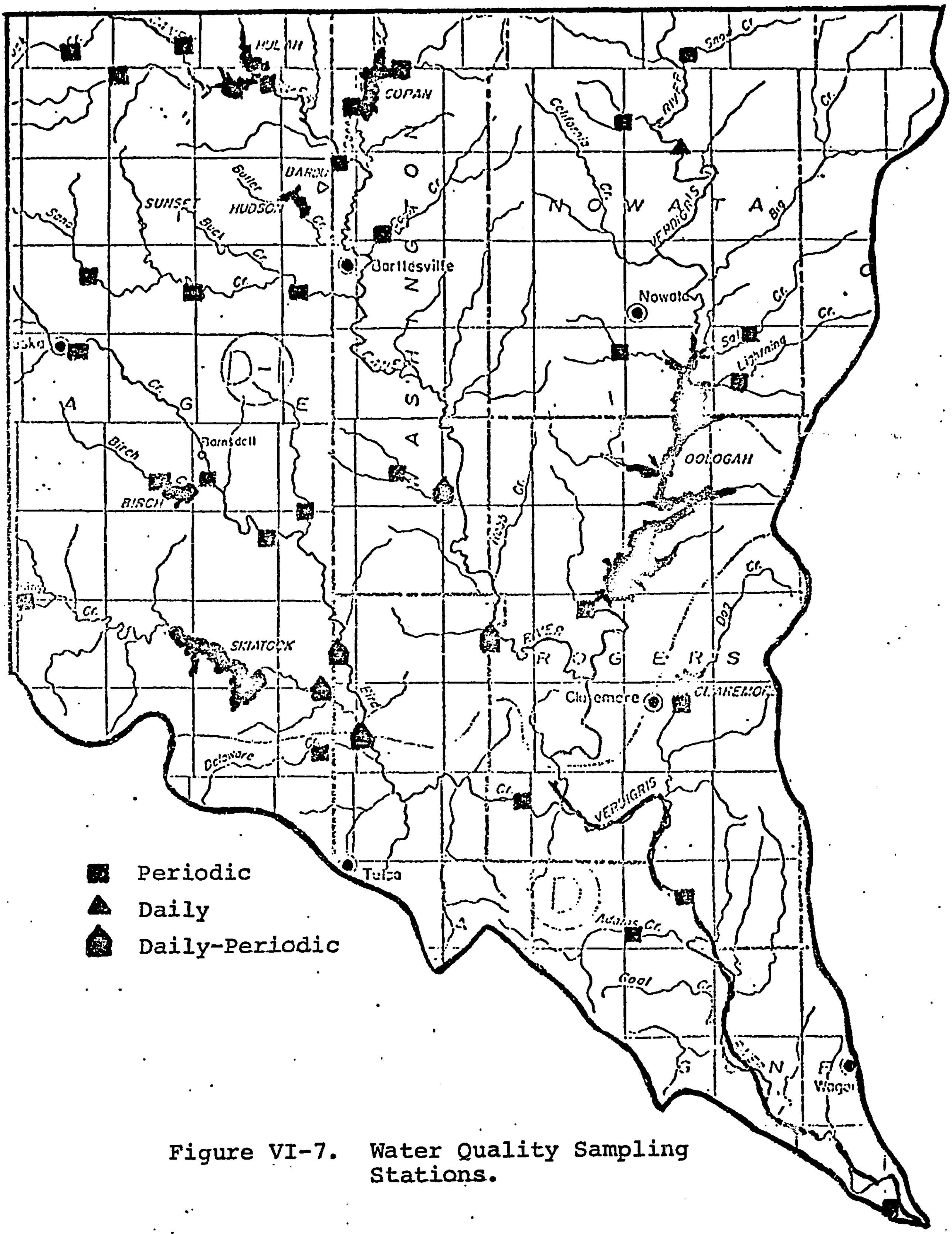


Figure VI-7. Water Quality Sampling Stations.

TABLE VI-8

MAXIMUM AND MINIMUM QUALITY VALUES FOR STREAMS
(Results in Parts per Million)

Creek	Sulfate (SO_4)		Chloride (Cl)		Nitrate (NO_3)		Dissolved Solids (Residue at 180°C)	
	Max	Min	Max	Min	Max	Min	Max	Min
Possum Creek near Lenapah	54.0	45.0	118	70.0	1.0	0.1	43	314
Snow Creek near Lenapah	37.0	33.0	34	8.0	0.2	0.1	337	302
Verdigris R. near Lenapah	150.0	10.0	375	15.0	16.0	0.0	937	114
Big Creek near Nowata	60.0	19.0	860	5.0	1.8	0.1	1,210	201
Salt Creek near Alluwe	34.0	10.0	1,240	4.0	1.9	0.0	2,480	154
Lightning Ck. near Alluwe	155.0	67.0	3,960	28.0	2.8	0.2	7,190	328
Verdigris R. near Oologah	55.0	37.0	252	58.0	1.9	0.2	694	348
Buck Creek near Boulanger	44.0	35.0	43	20.0	1.4	0.2	341	313
Caney Creek near Boulanger	47.0	29.0	170	14.0	10.0	0.6	732	314
Pond Creek near Boulanger	31.0	12.0	16	3.5	--	--	269	139
Caney River near Hulah	34.0	28.0	24	16.0	1.2	0.1	292	261
Cotton Creek near Copan	89.0	23.0	2,650	104.0	1.3	0.1	5,650	294
Little Caney R. below Cotton Creek near Copan	35.0	30.0	680	34.0	4.0	0.0	1,550	117
Coon Creek near Dewey	115.0	66.0	468	170.0	5.9	0.2	1,190	586
Caney R. near Bartlesville	89.0	14.0	367	44.0	26.0	0.1	931	225
Sand Creek near Pawhuska	34.0	12.0	16	6.2	0.8	0.1	300	184
Caney River at Ochalata	64.0	8.2	415	8.0	13.0	0.0	1,150	127
Double Creek Subwatershed No. 5 near Ramona	25.0	22.0	152	7.0	--	--	--	--
Caney River near Ramona	112.0	8.0	610	11.0	18.0	0.0	1,380	90
Caney River near Collinsville	91.0	16.0	305	22.0	3.7	0.1	1,000	132
Bird Creek near Pawhuska	75.0	19.0	242	16.0	3.7	0.0	678	239

TABLE VI-8--Continued

Creek	Sulfate (SO_4)		Chloride (Cl)		Nitrate (NO_3)		Dissolved Solids (Residue at 180°C)	
	Max	Min	Max	Min	Max	Min	Max	Min
Bird Creek near Barnsdall	182.0	14.0	225	18.0	2.8	0.2	810	202
Birch Creek near Barnsdall	45.0	8.0	196	26.0	3.6	0.2	468	150
Bird Creek near Avant	36.0	5.8	515	19.0	1.2	0.1	1,230	68
Candy Creek near Avant	52.0	13.0	305	14.0	3.5	0.0	770	129
Bird Creek near Skiatook	49.0	16.0	212	34.0	1.0	0.0	740	173
Hominy Creek near Hominy	59.0	6.8	1,460	105.0	3.2	0.1	2,710	307
Hominy Creek near Skiatook	55.0	9.0	605	32.0	4.0	0.2	1,250	72
Bird Creek near Sperry	43.0	5.4	1,910	20.0	6.4	0.1	3,590	76
Delaware Creek near Sperry	123.0	25.0	2,400	290.0	2.2	0.2	3,890	616
Bird Creek near Catoosa	135.0	8.0	265	4.0	52.1	0.1	740	114
Dog Creek near Claremore	129.0	23.0	64	6.0	2.0	0.4	348	127
Verdigris R. near Inola	116.0	8.3	1,700	12.0	36.0	0.1	3,060	91

TABLE VI-9

YEARLY MEAN STANDARDS

Sample Point Name	Chloride	Sulfate	TDS
Verdigris River near Oologah	82	167	475
Little Caney River near Copan	344	48	881
Caney River near Ramona	191	42	586
Bird Creek near Sperry	273	38	707
Bird Creek near Catoosa	152	54	519
Verdigris River near Inola	80	45	367

TABLE VI-10

BENEFICIAL USES

Code	Beneficial Use
A	Public and private water supplies
B	Emergency public and private water supplies
C1	Fish and wildlife propagation
C2	Fish and wildlife propagation to the extent allowed by specifically stated water quality parameters
D	Agriculture (ncludes livestock watering and irrigation)
E	Hydroelectric power
F1	Industrial and municipal cooling water
F2	Receiving, transporting and/or assimilation of adequately treated waste
G1	Recreation, primary body contact (includes recreational uses where the human body may come in direct contact with the water to the point of complete body submergence)
G2	Recreation, secondary body contact (includes recreational uses, such as fishing, wading and boating, where ingestion of water is not probable)
H	Nagivation
I	Aesthetics
J	Small-mouth bass fishery excluding lake waters
K	Trout fishery (put-and-take)

TABLE VI-11

LIMITATIONS

Code	Limitation
(a)	All streams and reservoir designated (a) are protected by prohibition of any future discharge of pollutants.
(b)	All streams designated (b) are return flow streams for which special water quality standards have been established.
(c)	Streams or stream systems in which advanced waste treatment of all waste discharges is required are designated (c).

The streams in the basin have been designated beneficial uses and limitations by the Oklahoma Standards. The following table is adopted from there.

TABLE VI-12

STREAM DESIGNATIONS

Stream	Beneficial Uses	Limitation
Verdigris River	A Cl D E Fl F2 Gl G2 H I	
Coal Creek	A Cl D Fl F2 Gl G2 I	
Adams Creek	A Cl D Fl F2 Gl G2 I	
Bird Creek	A Cl D Fl F2 Gl G2 I	
Mingo Creek	A Cl D Fl F2 Gl G2 I	
Belaware Creek	A Cl D Fl F2 Gl G2 I	
Hominy Creek	A Cl D Fl F2 Gl G2 I	
Candy Creek	A Cl D Fl F2 Gl G2 I	
Birch Creek	C2 D Fl F2 G2 I	
Caney River	A Cl D Fl F2 Gl G2 I	
Rabb Creek	A Cl D Fl F2 Gl G2 I	
Sand Creek	A Cl D Fl F2 Gl G2 I	
Buck Creek	A Cl D Fl F2 Gl G2 I	
Coon Creek	A Cl D Fl F2 Gl G2 I	
Caney Creek	A Cl D Fl F2 Gl G2 I	
Pond Creek	A Cl D Fl F2 Gl G2 I	
Buck Creek	A Cl D Fl F2 Gl G2 I	

TABLE VI-12--Continued

Stream	Beneficial Uses	Limitation
Blue Creek	A C1 D F1 F2 G1 G2 I	
Lightning Creek	A C1 D F1 F2 G1 G2 I	
Salt Creek	A C1 D F1 F2 G1 G2 I	
Big Creek	A C1 D F1 F2 G1 G2 I	
California Creek	A C1 D F1 F2 G1 G2 I	
Unnamed tributary*	C2 D F1 F2 G2 I	b
Snow Creek	A C1 D F1 F2 G1 G2 I	

*Return flow, city of Delaware.

Basin Population

Population data is very important for our model. The population data used were obtained from the 1970 census of population compiled by the Bureau of the Census, United States Department of Commerce.

Basin Geography

The river basin is not formed on the basis of existing political boundaries such as state lines or county lines. However, the census data are generally given for existing political entities such as state, county, city, town, etc. Therefore, the geography of the basin is important in deciding the population of the basin. The basin contains areas from seven counties, the first two of which are completely within the basin.

1. Washington
2. Nowata
3. Osage

4. Rogers

5. Wagoner

6. Tulsa

7. Craig

There are twenty-four municipalities within the basin with some kind of waste treatment facility.

TABLE VI-13

LIST OF MUNICIPALITIES

Town	County	Town	County
1. Avant	Osage	13. Nowata	Nowata
2. Barnsdall	Osage	14. Ochelata	Washington
3. Bartlesville	Washington	15. Owasso	Tulsa
4. Catoosa	Rogers	16. Pawhuska	Osage
5. Chelsea	Rogers	17. Ramona	Washington
6. Claremore	Rogers	18. Rolling Hills	Wagnore
7. Collinsville	Tulsa	19. Skiatook	Tulsa
8. Copan	Washington	20. S. Coffeyville	Nowata
9. Delaware	Nowata	21. Sperry	Tulsa
10. Dewey	Washington	22. Tulsa	Tulsa
11. Hominy	Osage	23. Wagoner	Wagnore
12. Inola	Rogers	24. Wynona	Osage

The population of a town or city is an important factor in deciding the type of treatment plant required for the community. The following table lists the population of each municipality (Table VI-14, page 140).

The total population of the basin is determined by the populations of the subdivisions contained in the basin. The Bureau of the Census (4) compiles the population of each subdivision of each county in the state. A county subdivision

TABLE VI-14

POPULATION DATA

Town	Population	Town	Population
1. Avant	439	13. Nowata	3679
2. Barnsdall	1579	14. Ochelata	330
3. Bartlesville	29638	15. Owasso	3491
4. Catoosa	970	16. Pawhuska	4238
5. Chelsea	1622	17. Ramona	600
6. Claremore	9084	18. Rolling Hills	--
7. Collinsville	3009	19. Skiatook	2930
8. Copan	558	20. S. Coffeyville	649
9. Delaware	534	21. Sperry	1123
10. Dewey	3958	22. Tulsa	330409*
11. Hominy	2274	23. Wagoner	4959
12. Inola	948	24. Wynona	1239

*Represents the urban population according to Ref. 4.

is a fairly small area and it is possible with the help of appropriate maps to determine the county-subdivisions contained in the basin and thus the total population of the basin. The total population of the basin was found to be 467,971. There is one metropolitan area, viz. the Tulsa Standard Metropolitan Statistical Area. The urban population of this SMSA is 330,409 (4).

Waste Discharge Inventories

The data on Municipal Discharge Inventory was obtained from the files of the Oklahoma State Health Department. Table VI-15 lists the inventory for municipal dischargers in the basin. The Industrial Waste Discharge Inventory was provided by the Oklahoma Water Resources Board. Table VI-16 lists the

TABLE VI-15

MUNICIPAL DISCHARGE INVENTORY

DISCHARGER Receiving Stream	Flow (MGD) De-sign	Flow (MGD) Oper.	Param-eter	Quantity of Discharge mg/L Inf.	mg/L Eff.	lbs/day Inf.	lbs/day Eff.	BOD re* (%)	pH	Type of Treatment
1 AVANT (Osage) Bird Creek		.006	BOD	679	578	34	28.9	15	--	Primary Clar.
2 BARNSDALL (Osage) Bird Creek	.12	.107	BOD		7		6.2		6.8	Lagoon
			TSS		78		69.6			
3 BARTLESVILLE-HILLCREST (Washington) Caney River	.072	.051	BOD	103	16	43.8	6.8	84	7.8	Aerated Lagoon
			TSS	148	8	63	3.4			
4 BARTLESVILLE-1 (Washington) Caney River	2.18	2.7	BOD	173	19	3896	428	89	7.8	Activ. sludge
			TSS	162	31	3648	698			Aerated Lagoon
5 BARTLESVILLE-2 (Washington) Caney River	.57	.24	BOD	142	14	284	28	90	7.3	Aerated Lagoon
			TSS	204	30	408	60			
6 CATOOSA (Rogers) Spunky Creek	.155	.056	BOD	232	18	108	8.4	92	8.4	Lagoon
			TSS	18	47	8.4	22			
7 CHELSEA (Rogers) Total Retention	.17	.13								
8 CLAREMORE (Rogers) Dog Creek	2.11	1.28	BOD	275	95	2936	1014	65	7.5	Trick. Filter
			TSS	224	128	2391	1366			
9 COLLINSVILLE (Tulsa) Total Retention	.55	.3								

*re = Removal Efficienty.

TABLE VI-15--Continued

DISCHARGER Receiving Stream	Flow (MGD) De-sign	Flow (MGD) Oper.	Param-eter	Quantity of Discharge mg/L Inf.	Quantity of Discharge mg/L Eff.	Quantity of Discharge lbs/day Inf.	Quantity of Discharge lbs/day Eff.	BOD re (%)	pH	Type of Treatment
10 COPAN (Washington)	.067	.041	BOD		49		16.8		8.7	Lagoon
Little Caney River			TSS		22		7.5			
11 DELAWARE (Nowata)	.165	.035								
Total Retention										
12 DEWEY (Washington)	.75	.388	BOD	199	27	644	87.4	86	7.5	Aerated Lagoon
Four Mile Creek			TSS	156	47	505	152			
13 HOMINY (Osage)	.305	.18	BOD	209	50	314	75	76	6.7	Primary Clar.
Penn Creek			TSS		252		278			Trick. Filter
14 INOLA (Rogers)	.057	.069	BOD	358	50	206	29	86	9.5	Lagoon
Pea Creek			TSS	444	388	256	223			
15 NOWARA (Nowata)	.319	.296	BOD	256	19	632	47	92	9.2	Primary Clar.
Western Br. Creek			TSS	222	10	548	24.7			Trick. Filter
16 OCHELATA	.109	.04								
(Washington)										
Total Retention										
17 OWASSO (Tulsa)	.7	.33								
Total Retention										
18 PAWHUSKA (Osage)	1.039	.308	BOD	121	24	311	61.6	80	8.5	Aerated Lagoon
Bird Creek			TSS	120	76	308	195			
19 RAMONA (Washington)	.049	.042								
Total Retention										
20 ROLLING HILLS	.353		BOD	233	20			91	6.7	Activ. Sludge
(Wagoner)			TSS	88	28					
Spunky Creek										

TABLE VI-15--Continued

DISCHARGER Receiving Stream	Flow (MGD) De-sign	Flow (MGD) Oper.	Param-eter	Quantity of Discharge mg/L Inf.	mg/L Eff.	lbs/day Inf.	lbs/day Eff.	BOD re (%)	pH	Type of Treatment
21 SKIATOOK (Tulsa) Total Retention	.52	.232								
22 SOUTH COFFEYVILLE (Nowata) Verdigris River	.14	.044	BOD		20		7.3		9.2	Lagoon
			TSS		100		36.7			
23 SPERRY (Tulsa) Hominy Creek	.146	.112	BOD		48		44.8		7.4	Primary Clar. Trick. Filter Imhoff Tank
			TSS		36		33.6			
24 TULSA (Tulsa) N.S. Bird Creek	11.	10.	BOD	127	22	10592	1835	83	7.2	Primary Clar.
			TSS	55	15	4587	1251			
25 TULSA FLAT ROCK (Tulsa) Bird Creek	4.	5.1	BOD	134	38	5700	1617	72	7.4	Activ. Sludge Aerated Lagoon
			TSS	76	69	3233	2934			
26 TULSA COAL (Tulsa) Bird Creek	4.	3.5	BOD	243	15	7093	438	94	7.4	Primary Clar. Trick. Filter
			TSS	230	38	6813	1109			
27 TULSA ROSE DEW (Tulsa) Spunky Creek	.02	.016	BOD		81		11		7.8	Lagoon
			TSS		108		14.1			
28 WAGONER (Wagoner) Coal Creek	.4	.75	BOD	205	15	1711	94	93	6.9	Lagoon
			TSS	182	85	1138	532			
29 WYNONA (Osage) Tributary to Birch Creek	.048	.034	BOD		40		11.3		8.2	Primary Clar. Lagoon
			TSS		130		36.9			

TABLE VI-16

INDUSTRIAL DISCHARGE INVENTORY

Discharger	Receiving Stream	Flow (MGD)	Parameter	Quantity Discharged mg/L	Quantity Discharged lbs/day	Remarks
1 Peabody Coal Co	Spencer Crk Oologah Res	0.05	COD	44	18.4	Treatment consists of settling basins. Discharge made up of coal mining & crushed wastes plus storm runoff. There is evidence of seepage from basins.
			Oil & Grease	3.0	1.25	
			SO_4	485	202	
			Total Solids	1644	686	
2 Tulsa Rendering Co. Collinsville	Black Jack C Penn Creek Caney River	0.05	BOD	40	16.7	Treatment consists of skimming & settling 2 anaerobic lagoons. Effluent from final aerobic lagoon used for irrigation
			NH_3	3.2	1.33	
3 Phillips Petroleum Research Bartlesville	Eliza Creek Sand Creek Caney River	0.06	BOD	1	.05	
			COD	20	10	
			P	.16	.08	
			Total Solids	236	118	
4 National Zinc Co. Bartlesville	Eliza Creek Caney River	0.5	Cd	.26	1.08	Treatment handled by settling & oxidation lagoon, settling and neutralization lagoon, and final settling lagoon.
			Cr	.4	1.67	
			Pb	1.0	4.17	
			Tot. Slds	3850	16054	
			Zn	3.4	14.18	
5 Public Service Co. NE Station Tulsa	Verdigris R.	1.21	NH_3	.6	6.05	There are 4 0.016 mg capacity holding tanks & 1 0.04 mg capacity neutralization basin. Treatment is primarily for 1.2 MGD cooling water
			Total P	1.6	16.2	
			TDS	5660	57117	

TABLE VI-16--Continued

Discharger	Receiving Stream	Flow (MGD)	Parameter	Quantity Discharged mg/L	Quantity Discharged lbs/day	Remarks
6 Chandler Materials Co. Tulsa	Bird Creek	0.0002	NH_4-N	2.8	0.047	Surface drainage autoclave blowdown
			Oil & Grease	3.6	0.06	
			Phenols	27	.045	
7 Kaiser Aluminum & Chemical Co Tulsa	No Name Crk Mingo Creek Bird Creek	0.01	Oil & Grease	8	.67	Using stabilization ponds (Lagoons).
			Ba^{++}	14	1.17	
8 Byron Jackson, Div of Borg-Warner Corp Tulsa	Mingo Creek Bird Creek Verdigris R	0.00124	BOD	6	0.06	Boiler blowdown cooling water & storm water. No treatment
			COD	19	0.196	
9 Black, Sivalls & Bryson Tulsa	Mingo Creek Bird Creek Verdigris R	0.018	BOD	1.4	0.21	Two discharge points. Spent acid is treated by limestone filtration & pH neutralization. No treatment for cooling water.
			COD	6	0.9	
			NH_3	1.96	.29	
			Oil & Grease	50	.75	
			P	4.6	.69	
10 Leland Equipment Co. Tulsa	Mingo Creek Bird Creek Verdigris R	0.00024	BOD	4	.008	Treatment consists of grease trap, where sludge in hauled to Spartan dump.
			Oil & Grease	6.2	.012	
11 Hathaway Ind. Tulsa	Mingo Creek Bird Creek Verdigris R	0.003	CN	180	4.5	Treatment consists of chlorine, $NaCO_3$, septic tank & sand filter.
			Pb	2.72	.068	
			NH_4-N	166	4.15	
			pH	2.0	2.0	
12 American Airlines, Inc. Tulsa	Mingo Creek Bird Creek Verdigris R	0.05	Total P	14.7	6.13	Two discharge points: #1 Mingo Creek, #2 Bird Creek No treatment. Injection well?
			Oil & Grease	11	4.59	

TABLE VI-16--Continued

Discharger	Receiving Stream	Flow (MGD)	Parameter	Quantity Discharged mg/L	Quantity Discharged lbs/day	Remarks
13 Fram Corp Tulsa	No Name Crk	0.0394	BOD	74	24.3	Uses a filter process to remove solids and oil
	Bird Creek		COD	203	66.7	
	Verdigris R		Oil & Grease	8.4	2.76	
			Cr	.15	.05	
14 Lake Country Beverage, Inc Tulsa	Mingo Creek	0.0008	BOD	355	2.37	Discharges are entirely cooling & floor wash water. No treatment.
	Bird Creek		COD	3668	24.47	
	Verdigris R		Oil & Grease	37.5	.25	
			pH	7.8	7.8	
15 North American Rockwell Corp Tulsa	Mingo Creek	0.05	Oil & Grease	572	238	No treatment for surface discharge of 0.05 MGD. An additional 0.07 MGD plating waste is disposed into an injection well.
	Bird Creek			3.1	1.29	
	Verdigris R					
16 McDonnell-Douglas Corp. (Douglas Aircraft) Tulsa	Mingo Creek	0.0393	#2BOD	30	9.8	McD-D Corp has 4 discharging points into Mingo Creek. Points #1, #2 & #4 receive no treatment. Point #3 receives chlorination & reduction clarifier, recarbonation, & super chlorination.
	Bird Creek		#2COD	63	19.8	
	Verdigris R		$\#2NO_3$	5.26	1.72	
		0.078	#3BOD	50	32.5	
			#3COD	89	57.9	
		0	#3F	2.9	1.89	
			$\#3NO_3$	5.8	3.77	
		0.344	#4BOD	30	86.1	
			#4COD	58	166.4	
			#4F	2.05	5.88	
		0.00167	#1			

TABLE VI-16--Continued

Discharger	Receiving Stream	Flow (MGD)	Parameter	Quantity Discharged mg/L	Quantity Discharged lbs/day	Remarks
17 Dowell, a div of Dow Chemical Co. Tulsa	Flatrock Cr Bird Creek Verdigris R	1.2×10^{-4}	#1BOD	7	.007	Discharge occurs at 2 pts. Pt #1 receives the effluent from the equipment testing area. This effluent is treated by one grease trap & 2 settling tanks. Pt #2 effluent originates in equipment washing area &iis treated by 2 separator tanks to remove oil & grease.
			#1COD	180	.18	
			#1 Oil& Grease	1.6	.0006	
		2×10^{-4}	#2BOD	66	.02	
			#2COD	600	1.0	
			#2 Oil& Grease	111	.19	
			#2 Total Solids	686	1.14	
18 Bumper Service of Tulsa, Inc Tulsa	Coal Creek Mingo Creek Bird Creek Verdigris R	0.09	Cr	2.2	.165	Chrome and Nickel are precipitated by pH adjustment. The cyanide is recirculated thru system, they are presently developing a new treatment process to reduce discharge.
			CN	0.12	0.09	
			Ni	8.4	6.4	
19 Tulsa Chrome Plating Co. Tulsa	Spunky Crk Verdigris R	0.02	Cr	1.7	0.03	Treatment is a batch process (0.002 MG/batch). The CrO_4 is precipitated & the sludge is disposed of by US Pollution Control. The supernatant is pumped into holding ponds & treated with sodium disulfite. pH 12.
			SO_4	165	2.75	
			COD	71.8	1.19	

TABLE VI-16--Continued

Discharger	Receiving Stream	Flow (MGD)	Parameter	Quantity Discharged mg/L	Quantity Discharged lbs/day	Remarks
20 Yuba Heat Transfer Div Tulsa	No Name Crk Bird Creek Verdigris R	0.00743	BOD	20	2.5	Treatment consists of a sludge & oil separator from which the grit & silt is placed into an on-site land fill, and the liquid is pumped into an oxidation lagoon along with domestic wastes.
			COD	124	15.5	
			Oil & Grease	4.6	.58	

inventory for the industrial dischargers in the basin. The Figure VI-8 shows the locations of the dischargers in the basin.

Formulation of the Problem

With the above information, a waste load allocation program can be devised for the river basin. The analysis shall be restricted to the carbonaceous-BOD loads only as the data on the nitrogen and phosphorus were not available. The problem can be formulated as follows:

$$\text{Minimize} \quad \sum_{i=1}^{n} e_i \qquad i = 1, 2, \ldots, n$$

subject to

$$\sum_{i=1}^{n} cBOD_i \times e_i \geq CBOD \times E$$

$$e_i \geq e_{pi}$$

$$e_i \geq 0.35$$

where e_i = level of treatment required at the source i

$cBOD_i$ = carbonaceous BOD loading of the source i

$CBOD = \sum_{i=1}^{n} cBOD_i$ = total carbonaceous load in the basin

E = regional waste removal level for carbonaceous BOD load

e_{pi} = the present level of treatment at source i

0.35 = the primary treatment level of cBOD removal

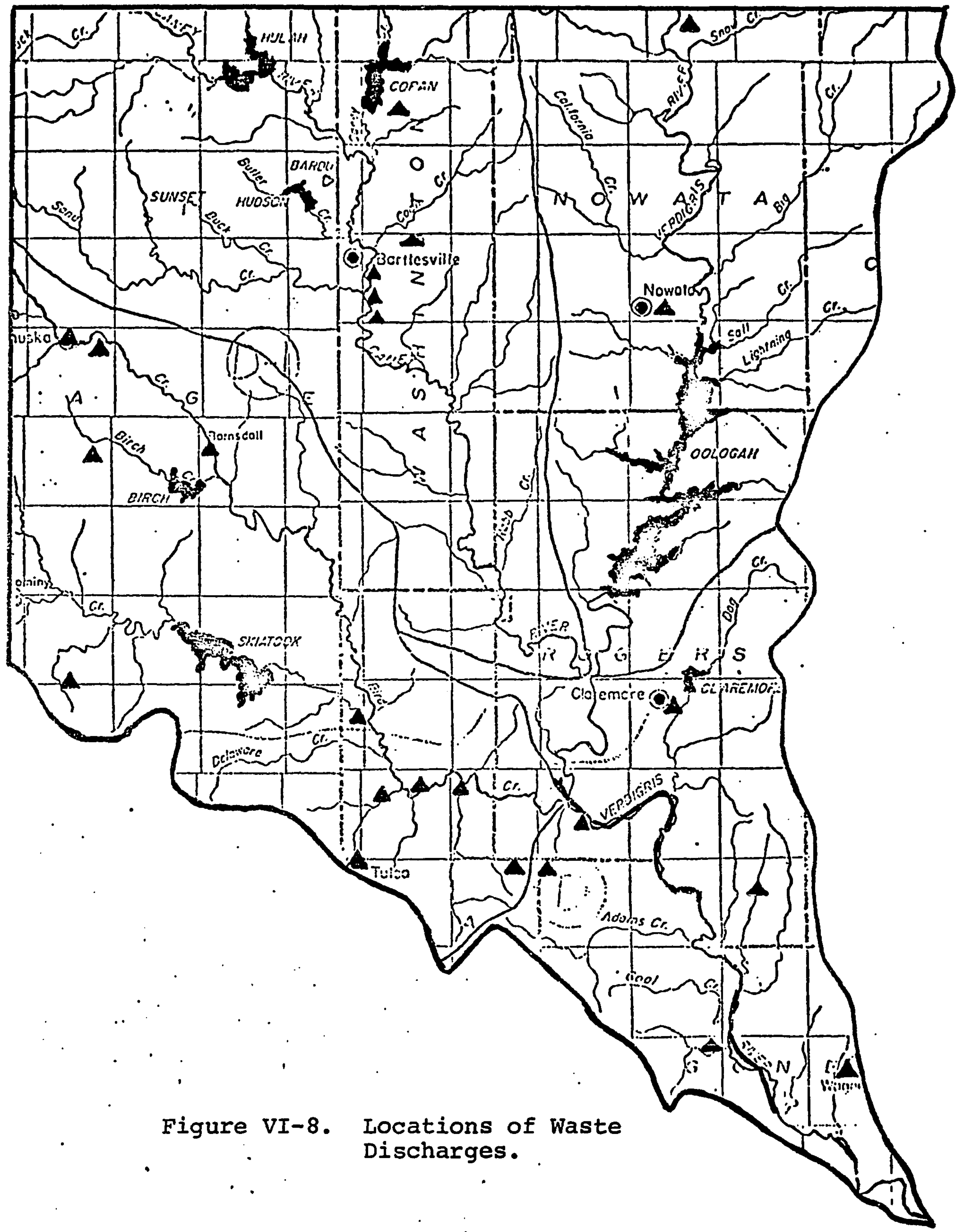


Figure VI-8. Locations of Waste Discharges.

<u>Determination of E</u>

The water quality model for the biodegradable waste is

$$Q_t = [\frac{Y}{\varepsilon} + (1-Y)] \frac{PE(PP)}{(c_s - RQS_{DO})} \times \frac{942,900}{k_2 \frac{n\ell}{V}}$$

The value of flows in the streams are usually obtained from the U.S.G.S. gaging station data. Of importance in our analysis are the critical flows or low flows. Seven days - two years low flows are used. The low flows are computed from the U.S.G.S. average minimum flows data by the method described by Riggs (49). The following table lists the low flows thus obtained.

TABLE VI-17

LOW FLOWS

	Stream	CFS	MGD
(1)	The Bird Creek @ Sperry	2.0	1.293
(2)	The Caney River @ Bartlesville	1.2	0.776
(3)	The Verdigris @ Claremore	16.5	10.664
(4)	The Verdigris @ confluence with the Arkansas	19.95	12.894

The critical oxygen deficit values are obtained from the Oklahoma Water Quality Standards which specify the critical temperature and critical dissolved oxygen values for each type of stream. Table VI-18 lists the pertinent data.

TABLE VI-18

CRITICAL OXYGEN DEFICIT VALUES

Type of Stream	Critical Temp. °F	c_s (mg/L)	RQS_{DO} (mg/L)	c_s-RQS_{DO} (mg/L)
(1) Return flow - special mixing zone	90	7.4	2.0	5.4
(2) Warm water	90	7.4	5.0	2.4
(3) Small mouth bass	84	7.8	6.0	1.8
(4) Trout	68	9.2	6.0	3.2

The value of k_2 varies from 0.1 to 0.8 in Oklahoma (39). The other reaction coefficient of interest, k_1, is found to vary from 0.1 to 0.18 in the Oklahoma streams. The Department of Pollution Control, the State of Oklahoma, has used the self-purification constant "f" = k_2/k_1 in their planning. The value of "f" used for the region containing the Verdigris River basin is 4.0, corrected for the temperature. This narrows the choice of a value for k_2 to 0.4 to 0.72.

The stream river-miles data are very important. The stream lengths are obtained by measuring them on topographical maps. The data obtained are listed in Table VI-19.

<u>Determination of the Costs</u>

Since the type of the treatment must be known besides the population equivalent and flow data, to determine the

TABLE VI-19

STREAM DATA

Stream System	Length of the Main Stem (miles)	Number of Tributaries	Total Stream Length (miles)
(1) The Bird Creek	84.0	15	299.5
(2) The Caney River	94.0	15	362.7
(3) The Verdigris River	148.5	22	1151.3

cost of a treatment plant, the required level of treatment at a source, viz. e_i must be translated to the type of waste treatment plant required at the source. This is achieved by the following table.

TABLE VI-20

DETERMINING THE TYPE OF WASTE TREATMENT PLANT REQUIRED

e_i	Required Treatment Level	Type of Treatment Plant Required
≤ 0.35	Primary	Primary
0.35 to 0.70	Marginal secondary	Standard rate trickling filter
0.70 to 0.85	High rate biological	High rate trickling filter
0.85 to 0.95	Secondary with nitrification	Activated sludge
0.95 to 0.99	Advanced	Activated sludge
≥ 0.99	Ultimate	--

The following tables, Tables VI-21 and VI-22, contain the results of the computations for optimal allocations of waste loads made with the help of the model.

TABLE VI-21

MUNICIPAL WASTE LOAD ALLOCATIONS

Municipal Plant	Influent BOD Load		BOD Removal Efficiency Percent		Effluent - BOD		Suspended Solids (in stream)	
	MGD	lbs/day	Pres.	Req.	mg/L	lbs/day	mg/L	lbs/day
(1) Avant (Osage)	0.006	34	15	35	218.2	11.90	30	10.51
(2) Barnsdall (Osage)	0.107	?	?	--	--	--	30	37.53
(3) Bartlesville-Hillcrest (Washington)	0.051	43.8	84	84	79.35	36.79	30	60.05
(4) Bartlesville-1 (Washington)	2.70	3896.0	89	89	17.46	428.56	30	650.52
(5) Bartlesville-2 (Washington)	0.24	284.0	90	90	13.01	28.4	30	82.57
(6) Catoosa (Rogers)	0.056	108.0	92	92	16.97	8.64	30	14.01
(7) Chelsea (Rogers) Total Retention	0.13	--	--	--	--	--	30	--
(8) Claremore (Rogers)	1.28	2936.0	65	85	37.85	440.40	30	335.27
(9) Collinsville (Tulsa) Total Retention	0.30	--	--	--	--	--	30	--
(10) Copan (Washington)	0.041	?	?	?	?	?	30	13.76
(11) Delaware (Nowata) Total Retention	0.035	--	--	--	--	--	30	--
(12) Dewey (Washington)	0.388	644.0	86	86	25.56	90.16	30	97.58
(13) Hominy (Osage)	0.18	314.0	76	76	46.05	75.36	30	55.04
(14) Inola (Rogers)	0.069	206.0	86	86	45.98	28.84	30	25.02
(15) Nowata (Nowata)	0.296	632.0	92	92	18.79	50.56	30	80.06

TABLE VI-21--Continued

Municipal Plant	Influent BOD Load		BOD Removal Efficiency Percent		Effluent - BOD		Suspended Solids (in stream)	
	MGD	lbs/day	Pres.	Req.	mg/L	lbs/day	mg/L	lbs/day
(16) Ochelata (Washington) Total Retention	0.04	--	--	--	--	--	30	--
(17) Owasso (Tulsa) Total Retention	0.33	--	--	--	--	--	30	--
(18) Pawhuska (Osage)	0.308	311	80	80	22.21	62.20	30	259.95
(19) Ramona (Washington) Total Retention	0.042	--	--	--	--	--	30	--
(20) Rolling Hills (Wagoner)	?	?	91	91	?	?	30	?
(21) Skiatook (Tulsa) Total Retention	0.232	--	--	--	--	--	30	--
(22) South Coffeyville (Nowata)	0.044	?	?	?	?	?	30	14.01
(23) Sperry (Tulsa)	0.112	?	?	?	?	?	30	325.26
(24) Tulsa N.S. (Tulsa)	10.0	10592	83	97	3.50	317.76	30	3352.65
(25) Tulsa Flat Rock (Tulsa)	5.1	5700	72	97	3.69	171.00	30	1951.56
(26) Tulsa Coal (Tulsa)	3.5	7093	94	97	6.69	212.79	30	1025.82
(27) Tulsa Rose Dew (Tulsa)	0.016	?	?	?	?	?	30	60.05
(28) Wagoner (Wagoner)	0.75	1711	93	93	17.57	119.77	30	217.67
(29) Wynona (Osage)	0.034	?	?	?	?	?	30	13.01

TABLE VI-22

INDUSTRIAL WASTE LOAD ALLOCATIONS

Industrial Discharger	Influent BOD Load		BOD Removal Efficiency Percent		Effluent - BOD		Suspended Solids (effluent)	
	MGD	lbs/day	Pres.	Req.	mg/L	lbs/day	mg/L	lbs/day
(1) Peabody Coal Co. St. Louis, MO	0.05	--	--	--	--	--	30	13.64
(2) Tulsa Rendering Co. Collinsville	0.05	16.7	?	35	23.89	10.86	30	13.64
(3) Phillips Petroleum Research Bartlesville	0.06	.05	?	0.0	0.09	0.05	30	16.37
(4) National Zinc Co. Bartlesville	0.5	--	--	--	--	--	30	136.36
(5) Public Service Co. Tulsa	1.21	--	--	--	--	--	30	330.0
(6) Chandler Materials Co., Tulsa	0.0002	--	--	--	--	--	30	0.05
(7) Kaiser Aluminum & Chemical Corp Tulsa	0.01	--	--	--	--	--	30	2.7
(8) Byron Jackson Tulsa	0.00124	0.06	?	0.0	5.32	0.06	30	0.34
(9) Black, Sivals & Bryson, Tulsa	0.018	0.21	?	0.0	1.28	0.21	30	4.90
(10) Leland Equipment Co Tulsa	0.00024	0.008	?	0.0	3.67	.0080	30	0.065
(11) Hathaway Ind. Tulsa	0.003	--	--	--	--	--	30	0.82
(12) American Airlines Tulsa	0.05	--	--	--	--	--	30	13.64

TABLE VI-22--Continued

Industrial Discharger	Influent BOD Load		BOD Removal Efficiency Percent		Effluent - BOD		Suspended Solids (effluent)	
	MGD	lbs/day	Pres.	Req.	mg/L	lbs/day	mg/L	lbs/day
(13) Fram Corp, Tulsa	0.0394	24.3	?	35	44.11	15.8	30	10.75
(14) Lake Country Beverage Beverage, Inc. Tulsa	0.0008	2.37	?	35	211.75	1.54	30	0.22
(15) North American Rockwell Corp. Tulsa	0.05	--	--	--	--	--	30	13.64
(16) McDonnell Douglas Corp, Tulsa	0.0393	9.8	?	35	17.83	6.37	30	10.75
	0.078	32.5	?	35	29.8	21.13	30	21.27
	0.344	86.1	?	35	17.9	55.97	30	93.82
	0.00167	--	--	--	--	--	30	0.46
(17) Dowell, Tulsa	0.00012	0.007	?	0.0	6.42	0.007	30	0.03
(18) Bumper Service of Tulsa, Inc. Tulsa	0.09	--	--	--	--	--	30	24.55
(19) Tulsa Chrome Plating Co., Tulsa	0.02	--	--	--	--	--	30	5.45
(20) Yuba Heat Transfer Division, Tulsa	0.00743	2.5	?	35	24.13	1.63	30	2.03

CHAPTER VII

SUMMARY AND CONCLUSIONS

Water pollution control is of great concern to the government and the general public. Therefore, the management of water quality has become very important. River basin planning is an important aspect of the water quality management of a region. Many authors have attempted to develop mathematical models to predict and manage the quality of waters in streams and rivers. More than 50 models were reviewed for this dissertation. Several of these more significant models were studied in detail and compared on the basis of data requirements, quality parameters, outputs and computational ease. The table VII-1 summarizes the information on these models.

Reid models were selected for water quality prediction for this dissertation. These models were viewed to be very suitable because they are simple in construction, inexpensive to use and require only a minimum amount of input data. These models were originally developed for predicting the flow augmentation requirements given the waste removal levels and other data. These models were modified here to predict

TABLE VII-1

COMPARISON OF THE WATER QUALITY MODELS

MODEL	QUALITY PARAMETERS	DATA REQUIREMENTS	OUTPUT	COMMENTS
QUAL-I	temperature, BOD-DO conservative minerals	Waste load inventories, stream data, head water sources data, flows, velocities, BOD,DO, T, k_1, k_2, etc.	time history and spatial description of a parameter.	can be applied to a region, moderately expensive in terms of data collection and operation
Harper	temperature, BOD-DO, conservative minerals, nutrients, etc.	stream velocity, slope, depth, flow, BOD, DO, waste flows, mineral concentrations, etc.	simulation of a quality parameter.	applicable to a stream. can be applied to a region. moderate cost.
Thomann	DO, BOD	DO, flows, diffusion coefficients, mixing parameter, etc.	DO time response to a varying input of a waste discharge.	applicable to a "estuary" and a non tidal stream. high speed digital computer reqd. for solution.
Novotny	DO, BOD	flows, velocities, diffusivity coefficient, depth, width, energy gradient in the stream, etc.	DO time response to a waste load.	requires great computer storage capacity, applicable to a stream network.

TABLE VII-1 (Continued)

COMPARISON OF THE WATER QUALITY MODELS

MODEL	QUALITY PARAMETERS	DATA REQUIREMENTS	OUTPUT	COMMENTS
Lombardo & Franz	temperature, BOD, DO, coliforms, sediments, nutrients, TDS, etc.	mean velocity, depth, concentrations of BOD, DO, coliforms, minerals, etc. flows, reaction rate constants, etc.	simulations of the hydrologic and water quality characteristics of a stream or a lake.	applicable to a stream or a lake. moderately expensive.
Chen & Orlob	temperature, BOD, DO, nutrients, toxicity, zooplankton, fish, TDS, coliform, etc.	river flows, waste discharges, tidal flows, outflows, lengths, width, depth, friction factor, surface area, side slopes, etc.	tidal velocities, discharges, elevations in an estuary. simulations of a quality parameter.	applicable to an estuary or a lake. moderately expensive.
Reid & others	temperature, BOD, nutrients, conservative minerals.	population data, length, velocity, k_2, flows, phosporous and nitrogen removal levels, BOD removal level, etc.	flow reqd. for augmentation or given a flow the treatment levels.	applicable to a region as a whole. very inexpensive to use.

the waste removal level for a region given the available low flows, population of the basin, etc.

The Mathematical Programming System (MPS/360) developed by IBM was used to solve the linear optimal allocation model. The MPS is available on IBM 360/370 systems to solve linear programs. Generally, the optimal allocation problem consists of a large number of constraints and variables. Each variable represents a source of waste in the river basin. The constraints consist of the upper and lower limits of treatment at each source and the water quality requirements. Such a model is very tedious for hand calculations. The MPS is very useful for solving the model.

The water quality model for the carbonaceous BOD (CBOD) contains one "judgment" parameter namely k_2, the reaeration coefficient. As stated in the previous chapter, the value of k_2 for the Verdigris River basin was determined to be in the range from 0.4 to 0.72. A conservative value of 0.5 was used for the basin. It is interesting to note that the "optimal" allocations of waste loads made using the value of $k_2 = 0.5$ remain essentially the same for values of k_2 from 0.50 to 0.60. If a value of 0.4 is assumed for k_2, then the allocation to Bartlesville-1 plant is affected. The required degree of BOD removal is raised from the present level of 89 percent to 96 percent. If the value of k_2 is assumed to be 0.72, then the allocation to Tulsa Flat Rock plant is affected. The required degree of treatment for CBOD removal is lowered from the 97 percent required under the present allocations to 90 percent.

The efficiency term ratio, ε, was found to be very close to unity. This indicates that the velocities in the streams and tributaries and the distribution of the point loads are such that the river system recovers from the effects of the point loads.

The cost models determine the costs in terms of 1957-59 dollars and are useful for comparing costs under various allocation programs. It should be remembered that the actual estimations of costs should be based upon further data from the recent constructions of sewage and waste treatment plants.

For the purposes of the "Comprehensive River Basin Planning" project, the waste load allocations were made by determining the assimilative capacities of the streams at the points of discharge. It is interesting to compare the results obtained under the two different allocation programs. For the ease of comparison, the waste loads are grouped into three classes: (1) concentrated loads; (2) medium loads; and (3) light loads.

In case of the concentrated loads such as Tulsa North Side, Tulsa Flat Rock and Tulsa Coal plants, the required degree of treatment is higher under the present allocation program. The only exception is the Claremore plant where the required degree of treatment is lower than the one determined under the previous program.

In case of the medium loads such as Bartlesville-2, Catoosa, Nowata, Pawhuska and other plants, the CBOD removal requirement is higher under the present program, while in the case of Dewey, Hominy and Inola it is slightly lower than that under the previous program.

For the light loads, such as industrial loads (for CBOD only) and others, either the minimal treatment, namely 35 percent CBOD removal, or no treatment is required under the present allocation program. It was considered unrealistic to require a discharger to treat his wastes if his CBOD load was less than 1 lb/day. The allocations made here compare fairly well with the allocations made previously. In some cases, the allocations are slightly higher while in other cases, they are slightly lower.

One obvious conclusion is that the method of waste load allocations presented here is superior to the methods used previously by other authors. The present method of the model is simple, quick and less expensive to use. Also, the data requirement of the model is less than that of the other models.

The models presented here should be very valuable to a river basin planning agency that is engaged in the interim planning for the basin. The models provide a quick waste load allocation program for water pollution control at relatively little cost.

It should be realized the model developed here is a macroscopic model and the results obtained are to be used for interim planning purposes. The field data available, generally, do not permit a very fine analysis, nor is such an analysis warranted. However, if precise data were available and if a finer study was required, models presented in Chapter 2 can be utilized for the purpose. The appendix shows sample calculations. It is apparent that the computations involved are very simple.

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APPENDIX

PROCEDURE AND THE SAMPLE CALCULATIONS

The Biodegradable model is

$$Q_t = \left[\frac{Y}{\varepsilon} + (1-Y)\right] \frac{PE\ (PP)}{(C_s - RQS_{DO})} \times \frac{942,900}{k_2 \frac{nL}{V}}$$

(Equation III-65 pg. 72)

This model is modified as follows

$$E = 1 - \frac{Q_t \cdot (C_s - RQS_{DO}) \cdot k_2 \cdot n \cdot L}{PE \cdot V \cdot (942,900)} \times \frac{1}{\left[\frac{Y}{\varepsilon} + (1-Y)\right]}$$

$E = 1 - PP$

The data used are

Q_t = 12.894 mgd. (table VI-17)

PE = 4,67,971 x 10^{-6} (page 140)

L = 1151.3 miles (table VI-19)

n = 1.06 (equation III-56)

V = 1.36 miles/hour (U.S.G.S. data)

k_2 = 0.50 (judgment parameter)

ε = 1.00 (equation III-59 & figure III-1)

Y = 0.70 (page 140)

$C_s - RQS_{DO}$ = 2.4 mg/L (table VI-18)

substituting these data and solving we get

E = 0.94.

Twenty eight waste dischargers reported their BOD loads. Tables VI-15 and VI-16 contain the data for these sources of wastes. The total BOD loading in the basin W is obtained as

$$W = \sum_{i=1}^{28} cBOD_i$$

$$= 34{,}679.425 \text{ lbs/day}$$

The allocation problem is formulated as a linear program as follows.

$$\text{Minimize } \sum_{i=1}^{28} e_i$$

subject to

Quality constraints

$$\sum_{i=1}^{28} cBOD_i \times e_i \geq 34679.425 \times 0.94$$

Policy constraints

$$e_i \geq e_{pi}$$

$$e_i \geq 0.85 \text{ if } cBOD_i \geq 1000 \text{ lbs/day}$$

$$e_i \geq 0.35 \text{ if } cBOD_i \geq 1 \text{ lb./day}$$

$$e_i = e_{pi} \text{ if } cBOD_i \leq 1 \text{ lb./day}$$

$$e_i \leq 0.97$$

$$e_i \geq 0$$

This linear problem is solved with the help of the Mathematical Programming System (MPS-360). The results of the optimal solution are presented in tables VI-21 & 22. The costs of the optimal solution are determined as follows.

<u>Municipal Treatment Plants</u>:

(1) Avant (Osage)

Pop. served 439

Present treatment level 15% BOD removal.

The design flow for the plant is 0.006 mgd.

The required level of BOD removal is 35%. A primary treatment plant would be required. The cost of this plant is determined as follows.

$$\text{Ln } Y'' = 12.42 + 0.3853\ x_2$$

$$X_2 = 0.006 \text{ mgd.}$$

$$\therefore Y'' = 2{,}500{,}00 \text{ \$/mgd.}$$

total cost:

$$\frac{2{,}50{,}000}{0.8} \times \frac{0.006}{1}$$

= 1875 $ 1957-59 dollars

(2) Claremore (Rogers)

Pop. served = 9084

Design flow = 2.11 mgd.

Present treatment level 65%

Required BOD removal level 85%

A high rate trickling filter is recommended.

The cost of this plant is determined as follows.

$$\text{Ln } Y'' = 9.39 + 0.3557 \text{ Ln}X_1 - 0.6443 \text{ Ln}X_2$$

$$X_1 = 9084$$

$$X_2 = 2.11 \text{ mgd.}$$

$$\therefore Y'' = 4{,}23{,}000 \text{ \$/mgd.}$$

the total cost

$$\frac{4{,}23{,}000}{0.8} \times \frac{2.11}{1}$$

= 1,1,18,000 \$ 1957-59 dollars

(3) Tulsa (Tulsa)

Three plants. Total pop. served = 330409

(i) Tulsa coal plant.

Design flow 4 mgd.

Required BOD removal 97%

An activated sludge treatment plant is recommended.

The cost of this plant is determined as follows.

$$\text{Ln } Y'' = 8.53 + 0.4610 \ln X_1 - 0.7375 \text{ Ln} X_2$$

X_1 = 80,000 (est.)

X_2 = 4 mgd.

Y'' = 3,36,000 \$/mgd.

the total cost:

$$\frac{3{,}36{,}000}{0.8} \times \frac{4}{1}$$

= 1,680,000 \$ 1957-59 dollars

(ii) Tulsa Flat Rock

Design flow is 5.1 mgd.

Required BOD removal level is 97%

An activated sludge treatment plant is recommended.

The cost of this plant is determined as follows.

$$\text{Ln } Y'' = 8.53 + 0.4610 \text{ Ln} X_1 - 0.7375 \text{ Ln} X_2$$

X_1 = 90,000 (est.)

X_2 = 5.1 mgd.

Y'' = 2,90,000 \$/mgd.

the total cost:

$$\frac{2,90,000}{0.8} \times \frac{5.1}{1}$$

$= 1,8,50,000$ \$ 1957-59 dollars

(iii) Tulsa North Side:

Design flow is 11.0 mgd.

Required BOD removal level is 97%

An activated sludge treatment plant is recommended.

The cost of this plant is determined as follows.

$$\text{Ln } Y'' = 8.53 + 0.4610 \text{ Ln}X_1 - 0.7375 \text{ Ln}X_2$$

$X_1 = 1,60,000$ (est.)

$X_2 = 11.0$ mgd.

$Y'' = 2,13,000$ \$/mgd.

the total cost:

$$\frac{2,13,000}{0.8} \times \frac{11}{1}$$

$= 2,9,22,000$ \$ 1957-59 dollars.

the total cost to Tulsa is:

$2,9,22,000 + 1,8,50,000 + 1,6,80,000$

$= 6,4,52,000$ \$ 1957-59 dollars

Industrial Treatment Plants

Five industries are required to treat their wastes for 35% BOD removal. Industrial primary treatment plants are recommended. The model used is:

$$\text{Ln } Y_p'' = 12.93509 - 0.09734 \text{ Ln}X_2 - 0.22875$$

(1) Tulsa Rendering Co.

Design flow = 0.05 mgd.

$Y_p'' = 4{,}36{,}000$ \$/mgd.

the total cost:

$$\frac{4{,}36{,}000}{0.8} \times \frac{0.05}{1}$$

= 27,220 \$ 1957-59 dollars.

(2) Farm Corp.

Design flow = 0.0394 mgd.

$Y_p'' = 4{,}45{,}000$ \$/mgd.

the total cost:

$$\frac{4{,}45{,}000}{0.8} \times \frac{0.0394}{1}$$

= 22100 \$ 1957-59 dollars

(3) Lake Country Beverage, Inc.

Design flow = 0.0008 mgd.

$Y_p'' = 8{,}15{,}000$ \$/mgd.

the total cost:

$$\frac{8{,}15{,}000}{0.8} \times \frac{0.0008}{1}$$

= 815 \$ 1957-59 dollars

(4) McDonnell Douglas Corp.

Design flow = 0.4613 mgd.

$Y_p'' = 3{,}60{,}000$ \$/mgd.

the total cost:

$$\frac{3{,}60{,}000}{0.8} \times \frac{0.4613}{1}$$

= 2,08,000 \$ 1957-59 dollars

(5) Yuba Heat Transfer Division

Design flow = 0.00743 mgd.

$Y_p'' = 5{,}20{,}000$ \$/mgd.

the total cost:

$$\frac{5{,}20{,}000}{0.8} \times \frac{0.00743}{1}$$

= 4820 \$ 1957-59 dollars

the total cost to the river basin is:

\$<u>7,834,830</u> \$ in 1957-59 dollars.