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WATER QUALITY EVALUATION OF REGIONALIZED WASTEWATER SYSTEMS

by

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CHAPTER I

INTRODUCTION

There are many testimonies to the history of man's control of water from its reflection in the cultures of past and present civilizations to archaeological records of dams, ditches and aqueducts. For thousands of years, water quantity control has been practiced on as large a scale as the daming and diversion of rivers while water quality control was seldom exercised on a scale above altering the physical characteristics of the contents of an urn. However, this observation does not come as a surprise. The shortage of water in quantity is more obvious than the shortage of water in quality, particularly given the use constraints of early societies.

Major advancements in water quality control were not to take place until the devastation of crowding, filth, and plagues made obvious the shortage of water in quality. Two concurrent events took place - the bacteriological contamination of water supplies causing the spread of water-borne diseases to reach epidemic proportions and the overloading of water courses with wastes which rendered their waters septic and noxious. Treatment of raw water supplies and treatment of wastewaters became necessary. The view of the water-use cycle as an engineering system was rarely seen: the connection between treatment of wastewater discharged upstream and treatment of raw water downstream was vaguely understood but no attempt was made to specifically evaluate the alternatives provided by variations of the degree of treatment before discharge and the degree of treatment before consumption.

It was not until almost a century later, while water quantity management grew from local, single-purpose projects to take on basin-wide proportions of multi-purpose, maximum use projects, that the systems perspective of water quality management was achieved and the tools necessary for the solution of large-scale systems problems began to appear. Because water quality is functionally dependent on water quantity, a merging of these hitherto distinct disciplines took place. The practice of the concept of cost effectiveness that evolved from water quantity managment along with the still controversial cost-benefit analysis technique was carried over to water quality management. The complications that arose from the inability to quantify intangibles in water quantity management are even more prevalent in water quality management today. At the time of writing, the most recent guidelines for water quality management planning in the United States are those of the U.S. Environmental Protection Agency (1971). The guidelines require projects to be conceived in regional plans and evaluated through water quality models of the regions' water courses to be eligible for Federal funding. These guidelines are to assure the effectiveness of wastewater management plans of local authorities in meeting water quality standards prescribed largely by the Federal government.

The question is not one of the cost effectiveness of a given set of standards in light of alternative plans to meet those standards, but rather the cost effectivenss of one plan relative to another in achieving given water quality standards. For this reason, the optimization of system plans to meet given standards is, in fact, a suboptimization and, for the purposes of this study, this situation will be regarded as an institutional constraint.

1.1 Public Investment in Regional Wastewater Management

Whatever the procedures for evaluating wastewater management projects and allocating public funds, there is no question that massive investments will be required to support these projects. This observation is supported by many recent studies. A study by the ASCE Urban Water Resources Council (1970) for the Office Water Resources Research revealed that

> About one-tenth of expenditures by local governments today are for water and wastewater services. Over the next several years, total direct and indirect public expenditures for construction and operation of urban water resource facilities (most of it borne by local levels of government) is expected to be on the order of $15 billion per year.

A preview of the forthcoming National Water Commission Report suggests waiving the user-pay requirement for wastewater treatment plants while the current construction program in underway. The commission's estimated cost for municipal and industrial wastewater treatment to achieve present water quality standards is $210 billion (Civil Engineering, 1973). Another study undertaken by the Business and Defense Service Administration (1967) estimates that $37.4 billion will be needed for the collection and treatment of municipal wastes by 1980. Similar estimates by the U.S. Public Health Service are $700 million annually during the seventies and by the Federal Water Pollution Control Administration (now the Water Quality Office of the EPA) are $26-$29 billion over the coming five years.

Although these estimates have considerable variance, they all indicate a massive commitment to public spending in the area of water management. The magnitude of these investments dictate a scrupulous examination of alternatives in metropolitan water management. Implicit in this is the knowledge of system performance and the economics of alternative systems.

1.2 Regional Wastewater Management Plans

Associated with every variable factor in wastewater management there are alternatives. A joint report by the NAS-NRC (1966) delineates these alternatives. They are given below with examples.

Alternatives of objective: A water course may be used for dilution of sewage or preserved for its scenic and recreational values.

Engineering alternatives: Wastewater treatment may be accomplished through the activated sludge process or through trickling filtration.

Management alternatives: Recreation may be achieved by maintaining clean water courses or by building swimming pools.

Institutional alternatives: A water system may be managed by a local government, a basin agency, a state agency, or a federal agency.

Timing and size alternatives: A wastewater treatment operation may be part of a staged construction plan to meet anticipated increases in wastewater flows.

The overall objective of the manager in resource development is to maximize the welfare of the people in the region of development. Translation of this broad objective into development plans is complicated by the range of choice in alternative ways of achieving the objective. Is it better to have the public pay higher prices for manufactured goods so that the industry producing these goods may treat the waste it discharges to a local waterway in order to improve the water quality which would allow the public to recreate in the waterway or does a cheaper good justify degraded water quality? If the economic value of the recreation is greater than the increased price of goods, then the public welfare would be increased and the plan should be undertaken. In the case of plans which produce intangible benefits that defy economic quantification, how are alternative plans to be evaluated? There is, of course, no single answer to the evaluation method but nevertheless a rational evaluation must take place.

A simplifying procedure to remedy this complication is to adopt a set of water quality standards and to compare alternatives by their ability to meet these standards and the public investment required for instituting each alternative plan. Given the maintenance of a specific set of water quality standards at least cost as the objective, the alternatives encountered in this study will be primarily of the management type.

Maintaining a specific set of water quality standards becomes the objective and there exist many alternative means to realize this objective. Households in a community generate wastes which can be handled at the household in low density developments or removed by a sewerage system and handled at a centralized site in higher density developments. The character of the waste itself may be altered both at the household level through regulations regarding such practices as the use of garbage grinders and the levying of water prices and at the sewerage system level through the use of separate sewer systems.

After wastewater has been collected from a metropolitan region, its disposal is necessary. If direct discharge to a local waterway is prohibited by its impact on the quality of the receiving waters, then the wastewater must be treated before discharge or transported from the region for disposal in another waterway or in or on the ground. These are all management alternatives to the problem solution.

This study does not initially proclaim that a sewerage - sewage treatment system is the overall best system for a region, although it is the most likely alternative combination today, but given that a sewerage - sewage treatment system is to be employed, this study will attempt to produce information regarding the optimality conditions of the size and location of collection and treatment facilities.

1.3 Constraints on the Treatment System Plan

The most frequent problem confronting metropolitan wastewater management is the treatment of wastes generated in the region. In most instances the region is serviced by wastewater collection networks and the decisions are how many plants are to service the region, where should these plants be located, and what treatment processes should be employed by each plant. These decisions are constrained by the characteristics of the region.

If the region is already sewered, then the number and location of treatment plants is constrained by the size and drainage pattern of the region's collection networks. There exist alternatives for enlarging the collection system through the use of interceptor sewers but disaggregating the system is not likely to be feasible. If the region or parts of the region are not sewered, then the only constraints are those of the site (geography, geology, etc.), the technology of sewerage (velocity ranges, pipe sizes, etc.), and the morphology of the area (density, total population, etc.).

Metropolitan regions are confronted with treating their wastewaters because the waterways receiving these wastes are being degraded. The water quality of the receiving waters is not at an acceptable level. Thus, the characteristics of the receiving waters are also constraints on the regional treatment system plan. The flows, flow variations, velocity, channel characteristics and physio-chemical characteristics of the receiving waters will influence the resultant water quality from wastewater discharges. These characteristics will also influence the number, size, and location of treatment plants entertained by alternative plans.

Other constraints on the system plan are present in the sociological characteristics of the region which may prohibit treatment plans to be located at sites which are best from the points of view of both system economics and resultant water quality. There may also exist administrative constraints which require certain plant sizes for competent operation. The economics of system scale also constrain the number, size and location of treatment plants.

This study will consider the constraints presented by the receiving waters primarily and by the economics of the service secondarily. The remainder of this study is directed to these two areas.

1.4 Regional Morphology

The morphology of a region will affect the economics of wastewater management plans and consequently the decisions regarding the selection of alternative plans. Two different morphologic conditions will be studied: the fully developed metropolitan area where the entire area is serviced by collection systems and the partially developed region where there exist isolated communities which are fully serviced while the intermedia around the communities require no collection service. A distinction will be made between existing and planned developments in this study.

The fully developed metropolitan region is composed of many sanitary districts sharing common boundaries. In an existing development, a decision must be made regarding the number of these districts to be joined in their contribution to a treatment facility while in a planned development an additional decision is presented in the determination of size and shape of districts. The decisions encountered in the existing development involve tradeoffs between interceptor and plant costs in conjunction with the effects of alternative plans on water quality. For example, many sanitary districts may be aggregated through interceptor systems with the cost of these interceptors offset by the economy of scale characteristic of treatment facilities. In such a situation, however, the cost of degraded water quality attributable to this aggregation may present an unbalance in the above costs; the savings in cost of any service plan must be traded-off with the cost of environmental resources consumed by the plan. The planned development allows for the selection of sanitary district size and shape and the optimal system plan is given recourse to aversion of costs incurred by large collection systems subject to a diseconomy of scale.

The partially developed region is composed of a number of dispersed communities each with a service system composed of a small number of sanitary districts. The alternatives offered to system plans are the various degrees of aggregation through interceptors. The decision regarding alternate plans involve the same considerations: the cost of interceptors, the scale economies of treatment facilities, and the impact of alternative plans on receiving water quality. The distinction between existing and planned or future developments is of lesser importance for the partially developed region since the separate collection systems are of a size not subject to the diseconomy of network scale.

Thus, the morphology of the region under study must be taken into consideration for developing appropriate study plans. The degree of development and the time staging of developments will act in unison to require special consideration from the study plan. In the case of a fully developed region, an existing development will require a study with emphasis on interceptor costs while a proposed development will require a study with emphasis on collection network costs. The partially developed region demands special attention to interceptor costs for both existing and proposed developments. Both fully and partially developed regions in both cases of existing and proposed developments require consideration of treatment plant costs and impact on water quality.

1.5 The Optimal Degree of System Centralization

Metropolitan wastewater management has been demonstrated to be an area of large public investment. It is also an area where there exists some doubt of the wisdom of the current management practice of highly centralized regional systems with regard to both economic and water quality considerations.

From the point of view of system economics, it is generally accepted that there exist economies of scale for the construction and operation of wastewater treatment facilities. Recent studies have indicated that there exist diseconomies of scale for wastewater collection networks (Dajani, 1971). The map of cost functions for wastewater collection and treatment combined have service areas with points of minimum average cost (Adams, et al, 1971). The minimum cost service areas increase with increased population densities, but they are generally small for densities commonly experienced in the U.S.A. today. This observation demonstrates the necessity of investigation into noncentralized regional wastewater plans.

There are two reasons for considering decentralized systems from a water quality point of view. The first is that the facilities of a multiple plant regional system will be spatially distributed along the receiving waters, resulting in the contributory wasteloads being more uniformly distributed in the receiving waters in comparison to the concentrated waste loading of the highly centralized system. Secondly, the facilities of the multiple plant system produce effluents whose qualities are only weakly correlated; that is, the facilities do not consistently produce the same quality effluents simultaneously (Adams and Gemmell, 1973). Because the fluctuating operating efficiencies of various facilities of a region are not in phase (that statistically the facilities operate independently), the receiving waters would not experience the sharp waste loading peaks that might result from a highly centralized system.

Because there is some doubt as to the wisdom of the planning practice concerning highly centralized regional wastewater systems and because metropolitan wastewater management is an area demanding massive investment of public funds, there is need for research treating the optimal degree of centralization of wastewater treatment facilities.

1.6 Objectives and Organization of the Study

The objectives of this study are primarily to determine the water quality effects of the size, number, and location of wastewater discharge sites and the variability of these wastewater loads, and, secondarily, to assess the importance of these effects in relation to the economics of water quality management systems for different conditions of regional morphology. The study plan outlines the strategy for accomplishing these objectives.

The first step in the study was to develop an appropriate water quality model for the evaluation of alternative wastewater management plans. The model was programmed in deterministic form with a selected set of nominal values for the model parameters. This set of values reflects a particular regional setting which may be used as a basis for comparison with other settings. This programmed model was then run with individual parameter values varying over their full physical ranges. From this sensitivity analysis, the model response to parameter variability was sensed.

Subsequently, the deterministic water quality effects of the location, size, and number of wastewater discharge points were assessed. A series of stream systems were investigated, each with 1, 2, 4, 8, 16 and 32 point discharges. The stream systems varied in length from 64 to 384 miles in increments of 64 miles with the same quantity of waste being discharged in each system. For each state of aggregation, equal distances between points of outfall were assumed. Thus, each stream system depicts a different condition of regional morphology; the shorter stream system lengths depicting more metropolitan regions and the longer stream systems depicting more rural regions. The water quality implications of different degrees of wastewater collection, treatment, and discharge consolidation were assessed for different morphological conditions of the region. The effect of stream size on the wastewater dilution ratio was determined by repeating the above experiment for a series of different stream sizes. In each of the above experiments, the tributary flow to the stream was attributed only to wastewater discharges. A condition was hypothesized whereby the streamflow would increase with length in accordance with a runoff function, and the water quality effects of system centralization were determined for this stream condition allowing for an evaluation of the original assumption of an unaugmented stream.

In preparation for the next step in the research which was to assess the effects of parameter variability on the system, a study of the performance of wastewater treatment facilities was undertaken. The performance of both individual plants and regionally related plant groups were studied. The purpose of studying individual plants was to examine and describe the nature of plant performance variability while the purpose of studying regionally related groups of plants was to determine the degree of dependence in performance among plants of the group. From this information on plant performance variability, it was possible to develop stochastic models to describe this variability.

The water quality model was reprogrammed with subroutines to describe the performance models which would accommodate the phenomena of parameter variability. The stochastic subroutines for the exogenous (input) variables and the endogenous (stream) variables were added to the simulation program individually which allowed for a parameter-by-parameter analysis of the system response to variability. With subroutines incorporated for all time variable parameters, a series of simulation runs were executed for all combinations of plant number, stream system length, and average dilution ratios. The output from the simulation model was in terms of the dissolved oxygen frequency response of the stream. With this information, it was possible to evaluate the stochastic performance of alternative regional wastewater management systems. The temporal framework of these system simulations was that of a 7 consecutive day, 10 year low flow period. Subsequently, a yearly simulation was conducted in order to compare the yearly frequency of achieving water quality goals with the frequency of achieving these goals during the critical season.

Finally, an economic evaluation of alternative regional wastewater management systems was undertaken. Cost functions for various components of these systems (collection networks, interceptor piping, pumping stations, forcemains, and treatment plants) were studied and with these cost functions the costs of alternative systems were estimated for different conditions of regional morphology.

At this stage it was then possible to compare alternative regional wastewater management plans not only in terms of water quality resulting from the size, number, and location of regional treatment plants and the frequency distribution of the water quality response, but also in terms of the cost of constructing and operating these alternative systems.

The computer programs employed in this study were written in the FORTRAN IV language and executed on the CDC 6400 system at Northwestern University's Vogelback Computing Center. (For program listings and sample outputs see Adams, 1963.)

CHAPTER II

LITERATURE REVIEW

DETERMINISTIC WATER QUALITY MODELS

Since dissolved oxygen was selected as the water quality parameter to be studied, it is appropriate to review models which have been proposed to describe the oxygen consumption and uptake in nontidal running waters. When an oxygen demanding waste is introduced into a stream, it is biochemically oxidized. Accompanying this deoxygenation is a reoxygenation by the addition of oxygen molecules from plant photosynethesis and the atmosphere to the stream water. Depending on the relative rates of these deoxygenation and reoxygenation processes, the quantity of the wasteload, and the dilution of the waste, the magnitude and location of the minimum dissolved oxygen (DO) concentration will vary accordingly.

2.1 Streeter and Phelps Model

The earliest attempt to describe this phenomenon led to the classical equations of H. W. Streeter and E. B. Phelps (1925). According to their theory, the dissolved oxygen concentration in a stream is governed by two reactions: (a) the oxygen is depleted by the respiration of bacteria stabilizing organic matter, and (b) the oxygen is replenished by adsorption from the atmosphere at the water surface. Both of these reactions were assumed to obey first order kinetics. By definition the biochemical oxygen demand (BOD) is increased at the same rate that the DO is depleted. The BOD and DO are decreased at a rate proportional to the remaining BOD, and the DO is increased at a rate proportional to the oxygen deficit (OD = c_s - c, where c_s is the DO saturation concentration and c is the DO concentration), which results in the following differential equations:

$$\frac{dL}{dt} = e^{-k_1 L} \tag{2.1}$$

and

$$\frac{dD}{dt} = e^{-k_2 D} + k_1 L \tag{2.2}$$

where t is the time of the reaction, L is the BOD concentration at time t, D is the oxygen deficit at time t, k_1 is the deoxygenation rate constant, and k_2 is the reaeration rate constant. The solutions to these equations are given by

$$L = L_o e^{-k_1 t} \tag{2.3}$$

and

$$D = \frac{k_1 L_o}{k_2 - k_1} (e^{-k_1 t} - e^{-k_2 t}) + D_o e^{-k_2 t} \tag{2.4}$$

where $L = L_o$ and $D = D_o$ at time $t = 0$ and e is the base of natural logarithms.

To apply these equations, it is necessary to specify temporal conditions. These conditions are usually established by assumptions of the stream's hydraulic regime. This model assumes that the flow is uniform and steady and that there is no longitudinal dispersion.

2.2 Dobbins' Model

The theory of Streeter and Pehlps was modified and extended to take into account various additional sources and sinks of oxygen by W. E. Dobbins (1954). In addition to bacterial oxidation and atmospheric reaeration, Dobbins delineates the following processes which may take place in any given reach of stream:

1. The removal of BOD by sedimentation or adsorption;

2. The addition of BOD along the reach by the scour of bottom deposits or by the diffusion of partially decomposed organic products from the benthal layer into the water above;

3. The addition of BOD along the reach by local runoff;

4. The removal of oxygen from the water by diffusion into the benthal layer to satisfy the oxygen demand in the aerobic zone of this layer;

5. The removal of oxygen from the water by purging action of gases rising from the benthal layer;

6. The addition of oxygen by the photosynthetic action of plankton and fixed plants;

7. The removal of oxygen by the respiration of plankton and fixed plants;

8. The continuous redistribution of both the BOD and oxygen by the effect of longitudinal dispersion.

Dobbins incorporates the above processes into equations for the BOD and OD profiles of streams. The assumptions used in this analysis are the following:

1. The stream flow is steady and uniform;

2. The process for the reach as a whole is a steady-state process;

3. The removal of BOD by both the bacterial oxidation and the sedimentation or adsorption or both are first order reactions, the rates of removal at any section being proportional to the amount present;

4. The removal of oxygen by the benthal demand and by plant respiration, the addition of oxygen by photosynthesis, and the addition of BOD from the benthal layer or the local runoff are all uniform along the reach;

5. The BOD and oxygen are uniformly distributed over each cross section.

The differential equations for the BOD and OD profiles are as follows:

$$D_L \frac{d^2L}{dx^2} - V \frac{dL}{dx} - (k_1 + k_3) L + R = 0 \qquad (2.5)$$

and

$$D_L \frac{d^2t}{dt^2} + k_2 D - V \frac{dc}{dx} - k_1 L - A = 0 \qquad (2.6)$$

in which D_L is the coefficient of longitudinal dispersion, x is the distance downstream, V is the average velocity, k_3 is the rate constant for BOD removal by sedimentation or adsorption or both (if positive) or BOD addition by resuspension (if negative), R is the rate of addition of BOD along the reach by local runoff, c is the DO concentration ($D = c_s - c$, where c_s is the DO saturation level), and A is the net rate of removal of oxygen by the benthal demand and the effect of plants. The solutions of these equations are given by

$$L = L_o e^{mx} + \frac{R}{k_1 + k_3} (1 - e^{mx}) \tag{2.7}$$

and

$$D = \frac{k_1 (L_o - \frac{R}{k_1 + k_3})(e^{mx} - e^{rx})}{k_2 - (k_1 + k_3)} + D_o e^{rx}$$

$$+ (\frac{A}{k_2} + \frac{k_1 R}{k_2 (k_1 + k_3)}) (1 - e^{rx}) \tag{2.8}$$

where

$$m = \frac{V - (V^2 + 4(k_1 + k_3) D_L)^{\frac{1}{2}}}{2D_L} \tag{2.9}$$

and

$$r = \frac{V - (V^2 + 4k_2 D_L)^{\frac{1}{2}}}{2D_L} \tag{2.10}$$

If the effect of dispersion is negligible and may be neglected, equations (2.7) and (2.8) may be written as follows:

$$L = L_o e^{-(k_1+k_3)t} + \frac{R}{k_1 + k_3} (1 - e^{-(k_1+k_3)t}) \tag{2.11}$$

and

$$D = \frac{k_1 (L_o - \frac{R}{k_1 + k_3})(e^{-(k_1+k_3)t} - e^{k_2 t})}{k_2 - (k_1 + k_3)} + D_o e^{-k_2 t}$$

$$+ (\frac{A}{k_2} + \frac{k_1 R}{k_2 (k_1 + k_3)})(1 - e^{-k_2 t}) \tag{2.12}$$

The application of these equations must be made in accordance with the assumptions stated above. The uniformity assumptions can normally be satisfied by selecting short reaches in the analysis.

2.3 Other Models

The equations for the BOD and OD profiles given by (2.7, 2.8) and (2.11, 2.12) are valid for application to streams where the hydraulic regimes are steady and uniform and the stream water quality and input wastewater quality and quantity are constant, with or without dispersion. Several investigators have solved these equations for conditions other than the above. Since these solutions are analytic, the variable input functions are deterministic. Although the values of the input functions are different at different times, these values are absolutely determined; they are not probabilistic. In cases where stochastic models were employed to describe input functions, the models were either totally theoretic or conceived in a special purpose application.

WATER QUALITY MANAGEMENT MODELS

Given the broad objective of maintaining a set of water quality standards for a region's waterways, there is a wide spectrum of management alternatives for accomplishing this objective. A review of the literature reveals many endeavors to explore these alternatives. Typically, these alternatives are treated individually for a specific case study; thus an attempt to tradeoff alternatives is thwarted by the lack of commonality in the base conditions or settings of these studies. The alternatives offered by a regional wastewater management problem may include streamflow regulation by on-stream storage, degrees of wastewater treatment, effluent storage, in-stream aeration, wastewater reuse, and the size, number, and location of plants. These regional wastewater management problems are usually formulated as optimization problems, the objectives of which are to minimize the system cost subject to treatment, inventory, and water quality constraints. Some representative examples of studies in this realm are discussed.

2.4 Optimization and Simulation Models

One of the earliest efforts in exploring the impact of management alternatives on wastewater treatment plant design practice was put forth by Montgomery and Lynn (1964). A digital computer simulation model was developed to represent a wastewater treatment system that included the possibilities of employing effluent storage and low flow augmentation. The objective of the study was to provide insight to the operation of the system employing these management alternatives. The system performance was evaluated by a water quality model in terms of the frequency with which an assigned limiting value of the allowable critical deficit in the stream was exceeded. Four physical alternatives was evaluated: (a) treatment plant - stream, (b) treatment plant - augmentation reservoir - stream, (c) treatment plant - effluent storage - stream, and (d) treatment plant - augmentation reservoir - effluent storage - stream. Alternative systems (b) and (d) substantially improved the performance of alternative (a) while alternative (b) only marginally improved the performance of alternative (a). Although some of the operational characteristics of the real system were oversimplified and an economic evaluation of the systems was not undertaken, this study unveiled an inherently useful approach to regional wastewater management planning.

Another cognate work presented by Sobel (1965) was the mathematical programming formulation of the degree of treatment required by plants along a watercourse to meet water quality standards at least cost. This problem was mathematically formulated as a linear programming problem; however, a numerical application was not presented. Concurrently, Deininger (1965) and Kerri (1966) formulated essentially the same problem as a linear programming problem. Numerical solutions to hypothetical problems were provided.

Loucks (1965) reworked the problem of Montgomery's by describing the stochastic processes as first order Markov chains. Values for the processes were drawn from transition probability matrices. This procedure allowed for serial and cross correlation of variable parameter values. Loucks concluded from the study that it may be economically advantageous to employ a storage basin for treatment plant effluents, particularly in cases of discharge to a small stream.

Recognizing the limitations of approximating nonlinear functions in the linear programming formulation of Sobel and Deininger, Liebman (1965, 1966) proposed a solution to the problem by dynamic programming. The problem was solved for a simplified example based on data from the Wilamette River. The Deininger problem was restated by ReVelle et al (1967, 1968) for a hypothetical example and by Anderson and Day (1968) for the Miami River. This problem was subsequently generalized by Clough and Bayer (1968) to include both short-run and long-run objective function and dual water quality constraints (DO and BOD). Shih (1970) also solved Deininger's problem by dynamic programming but included the costs of water treatment for reuse in the problem.

Thomann and Sobel (1964) formulated the same linear programming problem for a tidal stream, and later Thomann and Marks (1966) applied it to the Delaware River Estuary. Smith and Morris (1969) also reported the results of this same study of the Delaware Estuary.

SIZE, NUMBER, AND LOCATION OF TREATMENT PLANTS

The regional wastewater management alternatives given by the size, number, and location of treatment plants have received a limited amount of attention. This section is devoted to a discussion of published research considering these alternatives. Again, the problem is formulated as an optimization problem with the objective of minimizing the cost of system components with or without water quality constraints.

2.5 Some Qualitative Comments

The earliest printed reference addressed to the concept of centralized sewage treatment works appeared in 1936 (Lewin, 1936). This paper merely enumerated the pros and cons of treatment at a single central site as opposed to treatment at a number of smaller sites. The advantages of a centralized plant were stated as:

1. Transfer of responsibility for proper sewage disposal from individual authorities to a central board.
2. Liberation of valuable land occupied by numerous small plants.

3. Equalization of sewage quality and strength resulting from mixing sewage and wastes from larger population groups.

4. Easier supervisory control of one works by River Conservators.

5. Provision of employment to Consulting Engineers and contractors on a grand scale.

6. Extension of scope and specialized workers in sewage disposal.

The disadvantages of centralized plants are stated as:

1. The detrimental effect upon streams of the discharge of even good effluents of large volume at one point.

2. The large capital cost of construction both of the sewerage system and the disposal works.

3. The collection of huge quantities of sludge to be disposed of in one locality.

4. Magnification of troubles when breaks in operating efficiency occur.

5. Reduction of employment of skilled and unskilled labor.

The author suggested that more extensive research was needed to simplify sewage treatment methods before "the 'Centralization' idea can become the 'Master Method' of sewage purification."

More recently, Busch (1971) summarized the pros and cons of centralized treatment systems in the following points:

1. Reaction rate differences between various industrial wastes and between industrial and municipal wastes mean that the contributor of a rapidly degradable waste must pay for a disproportionate amount of residence time. Furthermore, disparities in organic removal rates frequently result in finely divided bacterial cell residue difficult to settle at conventional clarification rates.

2. For new combined systems financed by public bonds and built to serve industry, industrial contributors must sign long-term contracts commensurate with revenue bond terms, usually 20 years or more

3. Regional (centralized) systems served by gravity sewers must accommodate a disproportionately greater amount of infiltration because of longer collection lines.

4. Regional (centralized) systems built to replace existing minicipal plants having outstanding bond issues must be supported by new taxes while taxpayers continue to pay for abandoned facilities.

5. Most municipal wastes do not require biological treatment because the collection system is a self-seeded, plug-flow reactor, and soluble carbon conversion usually has been accomplished in the sewer. The longer the residence time in the sewer the more likely this conversion is to be complete. Therefore, regional (centralized) municipal systems are less likely to require a biological process than are smaller plants.

6. A mass balance will show, within the constraints of DO solubility, that a point source discharge of high volume must be of much higher quality than the same volume discharge at more numerous points if stream quality is not to be degraded.

Busch goes on to say that "none of the expensive studies carried out to define maximum stream assimilative capacity has ever recommended that the critical case is the <u>minimum</u> assimilative capacity and that this can be calculated in a straight-forward fashion. In other words, maximum assimilative capacity must involve probabilitic analysis and must establish confidence limits."

It is clear from these comments that an assessment of the water quality implications of alternative regional wastewater management plans, in conjunction with their economic impact, must be made in order to properly evaluate these plans.

2.6 Some Quantitative Examples

The greater part of the relatively small number of studies conducted on regional wastewater treatment have been concerned only with the economics of system components of alternative plans involving various degrees of wastewater centralization. A series of waste sources is identified, and the problem is to determine the economically optimal state of aggregation of these sources. The cost functions considered are those of interceptor sewers, pumping stations, forcemains, and treatment facilities.

Since the cost functions are concave, a general solution for minimum cost is difficult to obtain. Deininger and Su (1971) approximated these cost functions by quadratics and formulated a quadratic programming problem. Because there exists more than one local minimum, the solution to this problem is not necessarily a global minimum. Their procedure was to formulate a related linear program and then to apply Murty's ranking extreme point approach to obtain an optimal solution to the original problem. This method was applied to a hypothetical region of seven wastewater sources in which the economically optimal solution was a single plant system.

A solution of Converse (1972) to essentially the same problem as that of Deininger and Su is based on an extension of dynamic programming in which the solution is embedded in both the upstream and downstream conditions. An example was made of the Merrimack River Basin in which 12 wastewater sources were identified. The optimal solution was a four-plant system. The author notes that "the system cost is not strongly affected by the number of plants." For example, the cost of a ten-plant system is only 10 percent over the cost of the optimal four plant system.

Employing a piecewise linear approximation to the cost functions, Wanielista and Bauer (1972) solved the problem of Deininger and Su with an integer programming formulation. An 11 point wastewater source system in the Little Econ River Basin was used as an example for the problem. The results of the integer programming problem specified a minimum cost system consisting of two treatment plants. It was noted that an 11 plant system would have a cost of 10 percent more than that of the minimum.

Yao (1973) used the Connecticut River Basin as an example to demonstrate the water quality impact of regional wastewater treatment. Seven waste sources were identified on the branched river system. A deterministic Streeter-Phelps formulation was employed as the water quality model with the note: "A deterministic model could serve the present purposes better by demonstrating the effects of regionalization and other alternatives on stream quality in terms of simple figures. It is, however, important to keep in mind the limitations of using a simplified approach to a complex problem." Two alternatives were explored: (a) secondary treatment at all waste sources, and (b) secondary treatment at one centralized site. The one-plant system resulted in higher overall DO levels because the decentralized system discharged into tributaries with flows of approximately 5 percent of the main stem into which the one-plant system discharged.

Mendiratta and Davidson (1972) treated a problem similar to that of Yao, with the exception that instead of several waste treatment plants, a single hypothetical plant with a distributed outfall system was employed. Since this analysis was also deterministic, the receiving water perceives no difference between these two systems. The authors note that: "The state-space formulation of the biochemical transport model in this study is restricted to one-dimensional, steady state, deterministic modeling. Models based on higher space dimensions and time are totally unwarranted at this stage of application." The purpose of the analysis was to formulate and solve the general nonlinear problem of specifying the minimum degree of treatment, the fraction of the effluent to be distributed, and the continuous discharge pattern for an individual waste discharger subject to a set of dual water quality constraints (BOD and DO). The analysis employed Pontryagin's minimum principle in conjunction with penalty function criteria and a first order gradient search. The authors concluded that the savings in degree of treatment required by distributed outfall systems may range from significantly to marginally below the cost of the outfall system. Because the problem was formulated with a single plant, the length of stream over which the effluent was distributed was necessarily small.

CHAPTER III

THE WATER QUALITY MODEL

The structural elements of the receiving water quality model are subject to the water quality criteria of concern to the wastewater system planner. The criteria employed in any water quality system study are a function of the nature of the study and the characteristics of the study region's water uses. However, dissolved oxygen has long been recognized as a prime indicator of water quality and usually finds its way into water quality studies. Since no other parameter depicts instream water quality as well as dissolved oxygen, its concentration governs stream ecology and many water uses, and since the standards for other pollutants usually found in wastewaters are often met if the dissolved oxygen standard is met, the water quality model will be concerned primarily with this parameter.

3.1 Description of Model

For the purposes of this study, a two reaction model as given by equations (2.3) and (2.4) was adopted. Consideration of other possible reactions as given by the model in equations (2.7) and (2.8) is inappropriate at this time since the physical setting is a hypothetical one and an assignment of these reactions would be arbitrary. Upon verification of the hypothesis regarding system behavior stated earlier, the extension of this study to more complicated models would be appropriate.

This study is concerned with the minimum DO concentration occuring in the stream. This condition exists when the oxygen demand rate equals the reaeration rate or when

$$\frac{dD}{dt} = 0$$

Differentiating equation (2.4) and equating to zero, the time corresponding to the minimum DO (or equivalently, the maximum OD) may be solved, which is given by

$$t_{min} = \frac{1}{k_2 - k_1} \ln \left[\frac{k_2}{k_1} \left(1 - \frac{k_2 - k_1}{k_1 L_o} D_o\right) \right] \tag{3.1}$$

and the maximum deficit is given by

$$D_{min} = \frac{k_1 L_o}{k_2 e^{k_1 t_{min}}} \tag{3.2}$$

The values of D_o, L_o, and T_o (stream water temperature) are derived by mass/energy balances on the system as follows

$$D_o = D_{s_T} - \frac{Q_r D_r + Q_w D_w}{Q_r + Q_w} , \tag{3.3}$$

$$L_o = \frac{Q_r L_r + Q_w L_w}{Q_r + Q_w} , \tag{3.4}$$

and

$$T_o = \frac{Q_r T_r + Q_w T_w}{Q_r + Q_w} , \tag{3.5}$$

where D_{s_T} is the dissolved oxygen saturation concentration at temperature T; Q_r, D_r, L_r, and T_r are the flow, DO concentration, BOD concentration, and temperature of the stream prior to the waste input, respectively; and Q_w, D_w, L_w, and T_w are the flow, DO concentration, BOD concentration, and temperature of the waste stream, respectively.

The sag equation as given by equation (2.4) is capable of predicting the DO deficit at any time of flow downstream due to a single point of waste discharge. This formulation must be modified to accommodate multiple discharges. The stream system may be decomposed by lateral sections into reaches on the basis of either similarity of channel characteristics within a reach or location of points of outfall or both. The variables in the sag equation may then be viewed as subscripted variables with subscripts corresponding to reaches (see Figure 3.1). Since it is assumed

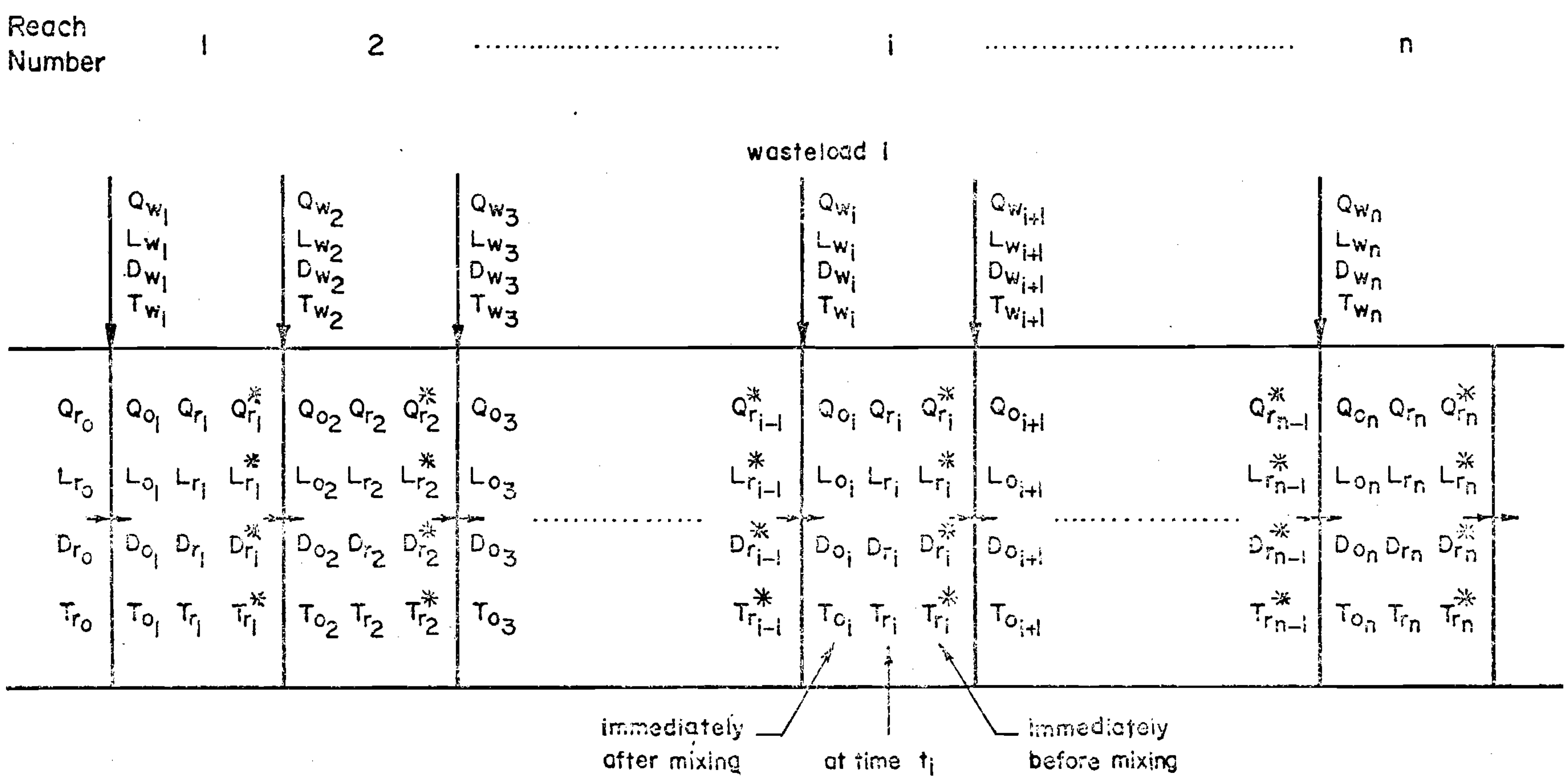


Figure 3.1

Schematic Diagram for Model Notation

that flow and heat transfers do not occur within a reach, the model for multiple outfalls may be written as

$$D_i = \frac{k_{1_i}}{k_{2_i} - k_{1_i}} L_{o_i} \left(e^{-k_{1_i} t_i} - e^{-k_{2_i} t_i}\right) + D_{o_i} e^{-k_{2_i} t_i} \tag{3.6}$$

where

$$L_{o_i} = \frac{Q_{r_{i-1}} L^*_{r_{i-1}} + Q_{w_i} W_{w_i}}{Q_{r_{i-1}} \quad Q_{w_i}} , \tag{3.7}$$

$$L_{r_i} = L_{o_i} e^{-k_{1_i} t_i} , \tag{3.8}$$

$$D_{o_i} = D_{s_T} - \frac{Q_{r_{i-1}} D^*_{r_{i-1}} + Q_{w_i} D_{w_i}}{Q_{r_{i-1}} + Q_{w_i}} , \tag{3.9}$$

$$D_{r_i} = D_{s_{T_i}} - \left[\frac{k_{1_i} L_{o_i}}{k_{2_i} - k_{1_i}} \left(e^{-k_{1_i} t_i} - e^{-k_{2_i} t_i} \right) + D_{o_i} e^{-k_{2_i} t_i} \right] , \tag{3.10}$$

$$Q_{r_i} = Q_{r_{i-1}} + Q_{w_i} \tag{3.11}$$

$$T_i = \frac{Q_{r_{i-1}} T_{r_{i-1}} + Q_{w_i} T_{w_i}}{Q_{r_{i-1}} + Q_{w_i}} \tag{3.12}$$

$$Q_o = Q_{r_i} = Q^*_{r_i} \tag{3.13}$$

and $\quad T_{o_i} = T_i = T^*_{r_i}$ (3.14)

The asterisk superscripts (*) denote parameter values at the end of each reach and t_i is the time of travel in reach i. By substitution, the subscripted form of the DO sag equation becomes

$$D_i = \frac{k_i}{k_{2_i} - k_{1_i}} \left(\frac{Q_{r_{i-1}} L_{o_{i-1}} e^{-k_{1_{i-1}} t^*_{i-1}} + Q_{w_i} L_{w_i}}{Q_{r_i}} \right) \left(e^{-k_{1_i} t_i} - e^{-k_{2_i} t_i} \right)$$

$$+ \left[D_{s_{T_i}} - \left\{ \frac{Q_{r_{i-1}} \left(D_{s_{T_{i-1}}} - \frac{k_{1_{i-1}} L_{o_{i-1}}}{k_{2_{i-1}} - k_{1_{i-1}}} \left(e^{-k_{1_{i-1}} t^*_{i-1}} - e^{-k_{2_{i-1}} t^*_{i-1}} \right) + D_{o_{i-1}} e^{-k_{2_{i-1}} t^*_{i-1}} \right) + Q_{w_i} D_{w_i}}{Q_{r_i}} \right\} \right] e^{-k_{1_i} t_i} \tag{3.15}$$

3.2 Analytic Input Functions

In equation (3.15) the value of the reaeration rate constant, k_2, is a function of the hydraulic properties of the stream which are in turn a function of the streamflow. Thus, it is necessary to derive these functional relationships.

The shape of the channel section is of importance only in its effect on the stage-discharge relationship. For the sake of providing an analytic function for stage-discharge, a rectangular channel section was assumed. The steady-state analysis allowed the use of a uniform flow, open channel formulation. Such a formulation is that of Manning (Chow, 1959) and is given by

$$Q = \frac{1.486}{n} AR^{2/3} S^{1/2} \tag{3.16}$$

where Q is the flow in cfs, n is the Manning number (a measure of the channel roughness), A is the cross sectional area of the channel in sq. ft., R is the hydraulic radius in feet, and S the channel slope. Because the channel section is rectangular, the following is evident

$$A = XH^2 \quad , \tag{3.17}$$

$$R = \frac{XH}{2 + X} \quad , \tag{3.18}$$

where X = B/H, and H and B are the depth of flow in feet and width of the channel in feet, respectively. Substituting equations (3.17) and (3.18) in equation (3.16) and solving for H yields

$$H = \left(\frac{(2 + X)^{2/3} \; nQ}{1.486 \; X^{5/3} \; S^{1/2}} \right)^{3/8} \tag{3.19}$$

By the continuity equation

$$V = \frac{Q}{A} \tag{3.20}$$

where V is the mean stream velocity in fps. Substituting equation (3.17) in equation (3.20) gives

$$V = \frac{Q}{XH^2} \quad \text{in fps} \tag{3.21}$$

or

$$V = \frac{3600 \; Q}{XH^2} \quad \text{in fph.} \tag{3.22}$$

> The reaeration capacity of a flowing stream depends primarily upon the prevailing degree of turbulence of the stream. Oxygen transfer from the atmosphere into the water can take place only at the air-water interface that exists at the stream surface, and this interface is constantly and randomly changing (being replaced or renewed) due to turbulent mixing of the flowing water. Hence, for any specific degree of oxygen depletion, the rate at which oxygen can be gained by the flowing water is directly proportional

> to the rate at which the water surface is being replaced from below by turbulent mixing. Turbulence is a very complex process, and is not as yet susceptible to independent measurement or evaluation. As a result, we have had no independent means of knowing the rate of water surface renewal in a natural stream, and it has therefore not been possible to evaluate reaeration capacity in terms of stream turbulence. It has thus been necessary over the years to attempt to evaluate stream reaeration capacity by the indirect oxygen balance method of Streeter and Phelps (Tsivoglou and Wallace, 1972).

The explanations of stream reaeration in terms of its hydraulic properties was originally proposed by Streeter and Phelps based on data collected on the Ohio River (1925). The proposed model combined the concepts of molecular diffusion and turbulence. An earlier model developed by Black and Phelps (1911) demonstrated that when the reaeration process is governed solely by molecular diffusion, the reaeration rate coefficient is inversely proportional to the square of the depth.

It was assumed that under uniform physical conditions, turbulence may be expressed as a power function of velocity, and combining this with molecular diffusivity, they postulated the following model for the reaction rate:

$$k_2 = \frac{cV^n}{H^2} \tag{3.23}$$

where the constants c and n define the stream type in terms of the fixed physical conditions, such as slope, character of the bottom, depth, shape, etc., V is the average velocity, and H is the depth. The constants c and n where indirectly evaluated from reaches of the Ohio River and varied from 0.23 to 131 and from 0.57 to 5.40, for c and n, respectively. These ranges of values suggested the need for further study of the reaeration rate function.

O'Connor and Dobbins (1956) developed models for the reaeration rate constant of the idealized oxygen sag equation based on some theoretical concepts of fluid turbulence. Equations were developed for the cases of nonisotropic and isotropic turbulence given by

$$k_2 = \frac{1100\ D^{1/2}\ S^{1/4}}{H^{5/4}} \qquad \text{(nonisotropic turbulence)} \qquad (3.24)$$

and $$k_2 = \frac{D^{1/2}\ V^{1/2}}{H^{3/2}} \qquad \text{(isotropic turbulence)} \qquad (3.25)$$

where D is the coefficient of molecular diffusivity (= 0.8×10^{-4} sq ft/hr for oxygen through an aqueous film).

Tsivoglou and Wallace (1972) comprehensively reviewed studies on the reaeration process and noted that with few exceptions the proposed predictive models for the reaeration rate constant basically followed the form proposed by Streeter and Phelps in 1925. The exceptions are the Isaacs-Maag model (1969) which contains an additional empirical coefficient for channel shape, the Krenkel-Orlob model (1962) which employs a longitudinal mixing coefficient, and the Thackston-Krenkel model (1969) which involves the Froude number. An additional formulation is given by Camp (1963) which replaces the mean velocity (V) with the mean temporal velocity gradient (G).

Tsivoglou and Wallace recommended the use of the following formulation:

$$k_2 = c\ \left(\frac{\Delta h}{t_f}\right) \qquad (3.26)$$

where c is a constant of proportionality called the "escape coefficient" with units of length, Δh is the change in water surface elevation from the beginning to the end of the reach, and t_f is the time of flow through the reach. The escape coefficient is given by

$$c = ab \qquad (3.27)$$

or $$c = \frac{0.693}{(\Delta h)_{1/2}} \qquad (3.28)$$

where a depicts the physical molecular properties of the diffusing gas (oxygen) and the quality of the water, b represents the mixing characteristics and hydraulic properties of the stream, and $(\Delta h)_{1/2}$ is the "half height" or the water surface elevation change required for the downstream deficit to take the value of half the upstream deficit. The approximate limits of the escape coefficient are from 0.030/ft for highly polluted waters (up to 30 mg/l 5 day BOD) to 0.085/ft for lightly polluted waters (down to 2 mg/l 5 day BOD) at 25^{o} C.

It is recognized that each model presented for k_2 has been validated for some specific application but that all models lack the necessary generality to be applied universally, with the possible exception of the Tsivoglou-Wallace model which requires much more specific information. Unfortunately, the effect of pollution on the escape coefficient has not been adequately characterized for the general use of this model either. Based on the extensive data of Churchill, et al (1962), a model of the Streeter-Phelps form was selected for use in this study; namely that of O'Connor and Dobbins given by equation (3.25) for isotropic turbulence.

Substituting H and V from equation (3.19) and (3.22), respectively, in equation (3.25), the following expression is obtained for the reaeration rate constant:

$$K_2 = \frac{8.16\ X^{17/16}\ S^{15/32}}{(2+X)^{5/8}\ n^{15/16}\ Q^{7/16}} \qquad (3.29)$$

where K_2 is the coefficient in base 10 (days^{-1}, 20^{o}C).

The DO saturation value is a function of temperature as given in the relationship

$$D_{s_T} = 14.652 - 0.41022\ T + 7.9910 \times 10^{-3}\ T^2 - 7.7774 \times 10^{-5}\ T^3 \qquad (3.30)$$

in which D_{s_T} is the DO saturation level in mg/l at temperature T (Committee on Sanitary Engineering Research, 1960).

The deoxygenation and reaeration rate constants are also functions of temperature given by the following relationships

$$K_{1_T} = K_{1_{20^oC}}\ \theta^{T-20} \qquad (3.31)$$

$$K_{2_T} = K_{2_{20^oC}} \, \emptyset^{T-20} \tag{3.32}$$

for any temperature T in degrees centigrade. The values for θ and $\emptyset$ are 1.047 (Gotaas, 1948) and 1.0241 (Committee on Sanitary Engineering Research, 1961), respectively.

Given the preceding series of analytic functions, the minimum dissolved oxygen levels in each reach of an n reach system may be computed using the algorithm presented in Figure 3.2.

3.3 Nominal Values for Model Parameters

The regional wastewater management problem of assessing the impact of treatment plant centralization on water quality was approached in both a deterministic and a stochastic manner. The deterministic treatment involves a water quality model with constant parameter values. These values are viewed as the nominal values of the system, and this section will describe the values selected and the rationale behind their selection.

A regional hydraulic population equivalent of one million people was chosen because it is of sufficient size to offer water quality problems. It is also a common enough size to be representative of many regions. At 170 gpcpd, the total wastewater flow (QW) generated by the region is 263 cfs.

The selection of a nominal value for streamflow (QRO) may be considered to be somewhat less arbitrary than other nominal values. The primary flow condition to which this study is directed is the average over seven consecutive days which occurs once in 10 years. If the length of the section of stream under study is relatively short, that section may be assumed anywhere along the length of the stream: the upstream end with a relatively small flow or the downstream end with a relatively large flow. However, for relatively long study reaches, the physical basin must be considered. The streamflows will be a function of the physical basin and the drainage area tributary to different gauging points of the stream. This information is presented in Figure 3.3 for Illinois streams (Lara, 1970). An inspection of Figure 3.3 indicates a lack of any strong

REACH LENGTH = SYSTEM LENGTH/NUMBER OF PLANTS

$$Q_r(i) = Q_r(0) + Q_w(i) \quad , i = 1$$

$$Q_r(i) = Q_r(i-1) + Q_w(i) \quad , i = 2, 3, \ldots, n$$

$$H(i) = (n(2 + X)^{2/3} Q_r(i)/1.486\ X^{5/3}\ S^{1/2})^{3/8}$$

$$V(i) = .0284\ Q_r(i)/X(H(i))^2$$

$$t^*(i) = (\text{REACH LENGTH})/V(i)$$

$$T(i) = (Q_w(i)\ T_w(i) + Q_r(0)\ T_r\ (0))/Q_r(i), \ i = 1$$

$$T(i) = (Q_w(i)\ T_w(i) + Q_r(i-1)\ T_r(i-1))/Q_r(i), \ i = 2, 3, \ldots, n$$

$$K_{1_T}(i) = K_{1_{20^\circ C}}(i)\ (1.047)^{T(i)-20}$$

$$K_{2_{20^\circ C}}(i) = 0.0939\ (V(i))^{\frac{1}{2}}/H(i))^{3/2}$$

$$K_{2_T}\ (i) = K_{2_{20^\circ C}}\ (i)(1.0241)^{T(i)-20}$$

$$L_o(i) = (Q_w(i)\ L_w(i) + Q_r\ (0)\ L_r(0))/Q_r(i), \quad i = 1$$

$$L_o(i) = \left(Q_w(i)\ L_w(i) + Q_r(i-1)\ L_o(i-1)\ 10^{-K_{1_T}(i-1)\ t^*(i-1)}\right)/Q_r(i), \ i-2, \ldots, n$$

(cont.)

Figure 3.2. Algorithm for Computing Dr_{min} in Each Reach of an n Plant System

$$D_{s_T}(i) = 14.652 - 0.41022\ T(i) + 7.9910 \times 10^{-3}\ T^2(i) - 7.7774 \times 10^{-5}\ T^3(i)$$

$$D_r^*(i) = D_{s_T}(i) - \left[\left(K_{1_T}(i)/(K_{2_T}(i) - K_{1_T}(i))\right) L_o(i) \left(10^{-K_{1_T}(i)\ t^*(i)} - 10^{-K_{2_T}(i)\ t^*(i)}\right) + D_o(i)\ 10^{-K_{2_T}(i)\ t^*(i)}\right]$$

$$D_o(i) = D_{s_T}(i) - \left((Q_w(i)\ D_w(i) + Q_r(0)\ D_r(0))\ /Q_r(i)\right) \quad , i = 1$$

$$D_o(i) = D_{s_T}(i) - \left((Q_w(i)\ D_w(i) + Q_r(i-1)\ D_r^*(i-1))\ /Q_r(i)\right) , \ i=2,\ldots,n$$

CHECK: $K_{1_T}(i)\ L_o(i) \leq K_{2_T}(i)\ D_o(i)$

YES $\longrightarrow D_{r_{min}}(i) = D_{s_T}(i) - D_o(i)$

NO $\longrightarrow t_{min}(i) = \left(K_{2_T}(i) - K_{1_T}(i)\right)^{-1} \log \left[\dfrac{K_{2_T}(i)}{K_{1_T}(i)} \left(1 - \dfrac{K_{2_T}(i) - K_{1_T}(i)}{K_{1_T}(i)\quad L_o(i)}\right) D_o(i)\right]$

CHECK $t_{min}(i) > t^*(i)$

YES $\longrightarrow D_{r_{min}}(i) = D_r^*(i)$

NO $\longrightarrow D_{r_{min}}(i) = D_{s_T}(i) - \dfrac{K_{1_T}(i)\ L_o(i)}{K_{2_T}(i)\quad 10^{K_{1_T}(i)\ t_{min}(i)}}$

Figure 3.2 (continued)

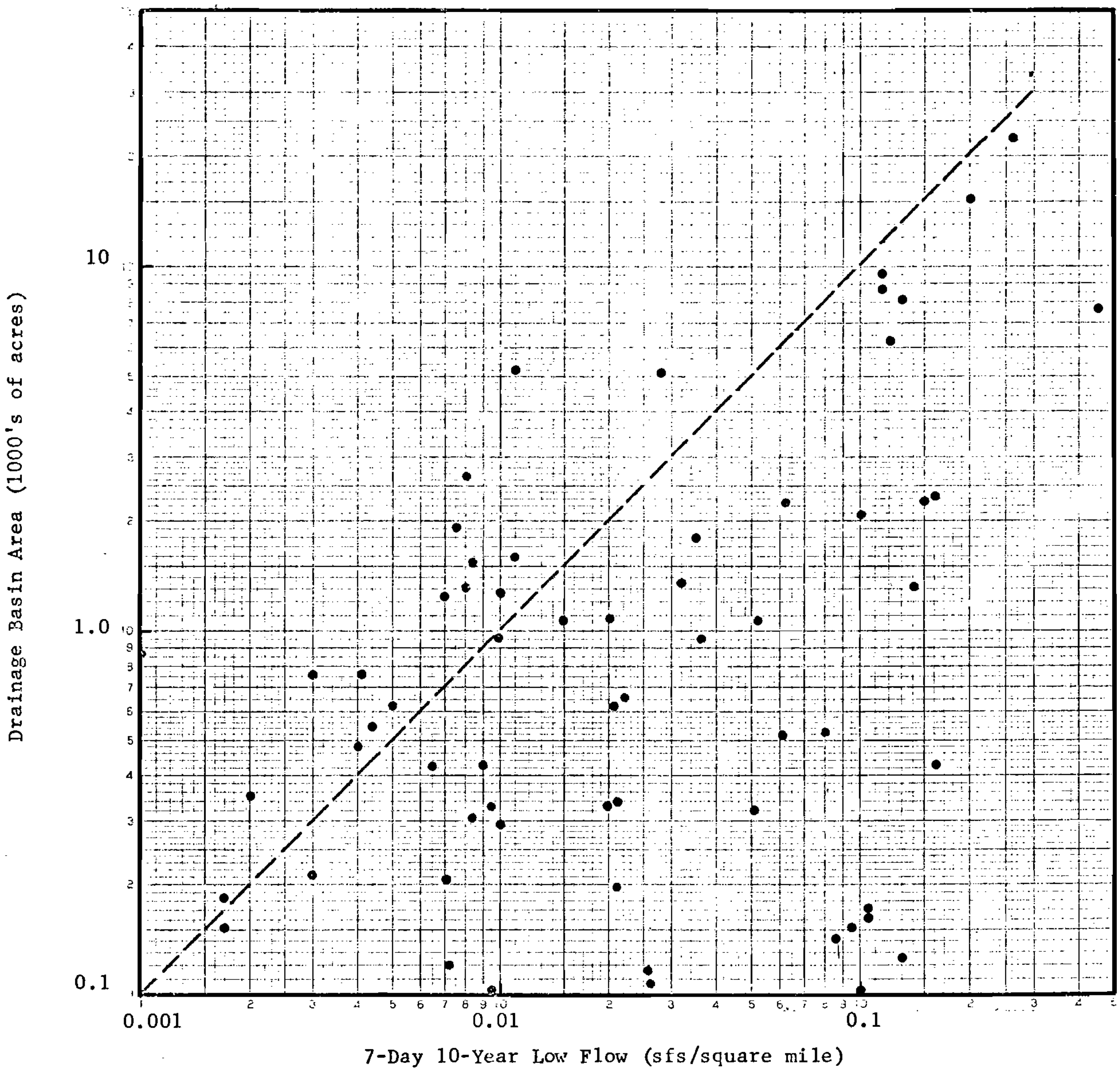


Figure 3.3

Low-Flow - Basin Area Relationships
for Illinois Streams

relationship between the magnitude of the low flow and the drainage basin area; the envelope of sample points is very large. These flows are given in Table 3.1 for some specific river basins in Illinois. For basin length-to-width ratios of 2, 3, and 4, the tributary drainage basin areas for different stream lengths were computed. The 7 day-10 year unit low flows corresponding to these basin areas were determined from Figure 3.3, and their respective low flows were computed. Both upper and lower limits of flow values were selected. This information is presented in Table 3.2. Since this derivation assumed the stream lengths to be upstream reaches, these flows may be construed to be representative of lower flow limits for given lengths and length-to-width ratios. Figure 3.4 indicates the relationship between average population density and basin area for various total populations, and Table 3.3 presents some information concerning population density and size of places in the U.S.A.

From the above information, it is possible to select a streamflow representative of system size and the population contained in the system. A streamflow for the study region's watercourse of 527 cfs was chosen. This value is in accordance with the information presented above. This nominal streamflow results in a dilution ratio of 2.1 which is small enough to require secondary wastewater treatment in Illinois.

A nominal stream length of 64 miles was selected as being representative of a metropolitan region with a population equivalent of one million people. Although the nominal number of plants (N) is 1, a following section of this study considers plant systems with up to 32 plants in which case the space between plants would be in 2 mile increments, a limiting condition below which spacing could not be considered.

The stream channel geometry was assumed to be rectangular. The channel geometry is arbitrary in as much as it was only an input point to the model for the velocity and the stage-discharge relationship. Thus, the channel width (B) is equal to a constant (X) multiplied by the depth (H).

Table 3.1

Streamflow Data for Illinois River Basins

Basin and Gauging Station	Basin Area (sq mi)	Average Flows (cfs)	Unit Low Flow (cfs/sq mi)	7 Day-10 Yr Low Flow (cfs)
Rock River				
@ Rockton	6,290	3,550	.12	755
@ Oregon	8,120	4,204	.12	960
@ Como	8,700	5,141	.11	970
@ Joslin	9,520	5,380	.11	1,047
Kankakee River				
@ Momence	2,340	1,830	.16	293
@ Wilmington	5,250	3,880	.08	258
Des Plaines River				
@ Gurnee	230	128	<.0001	
@ Des Plaines	359	215	.0006	.24
Du Page River				
@ Sherwood	325	220	.05	16
Fox River				
@ Algonquin	1,364	761	.032	24
@ Dayton	2,570	1,491	.061	91
Illinois River				
@ Marseilles	7,640	11,200	.45	5,040
@ Kingston Mines	15,200	13,700	.20	2,740
@ Meredosa	25,300	20,050	.16	3,210

Source: Low Flow Frequencies of Illinois Streams, Division of Waterways, State of Illinois, 1970.

Table 3.2

Streamflow Data for Basin Characteristics

Stream Length	2/1					3/1					4/1				
	A_d[1]	q_{min}[2]	q_{max}	Q_{min}[3]	Q_{max}	A_d	q_{min}	q_{max}	Q_{min}	Q_{max}	A_d	q_{min}	q_{max}	Q_{min}	Q_{max}
64	2,050	.001	.25	2.1	512	1,360	.001	.15	.20	204	1,025	.001	.15	1.0	154
128	8,200	.03	.4	246	3,280	5,460	.01	.30	55	1,638	4,100	.001	.30	4.1	1,230
192	18,450	.15	.5	2,750	9,225	12,300	.08	.50	980	6,150	9,220	.04	.40	369	3,690
256	32,800	.2	.6	6,560	19,680	21,800	.2	.5	4,360	10,900	16,400	.10	.5	1,640	5,200
320	51,200	.2	.6	10,240	30,720	34,200	.2	.6	6,840	20,520	25,600	.2	.6	5,120	13,250
384	73,700	.2	.6	14,740	44,220	49,200	.2	.6	5,840	29,520	36,900	.2	.6	7,380	22,140

Basin Length-to-Width Ratio

[1] A_d = Drainage Area in sq. mi.

[2] q = Unit Low Flow in cfs/sq. mi.

[3] Q = Low Flow in cfs

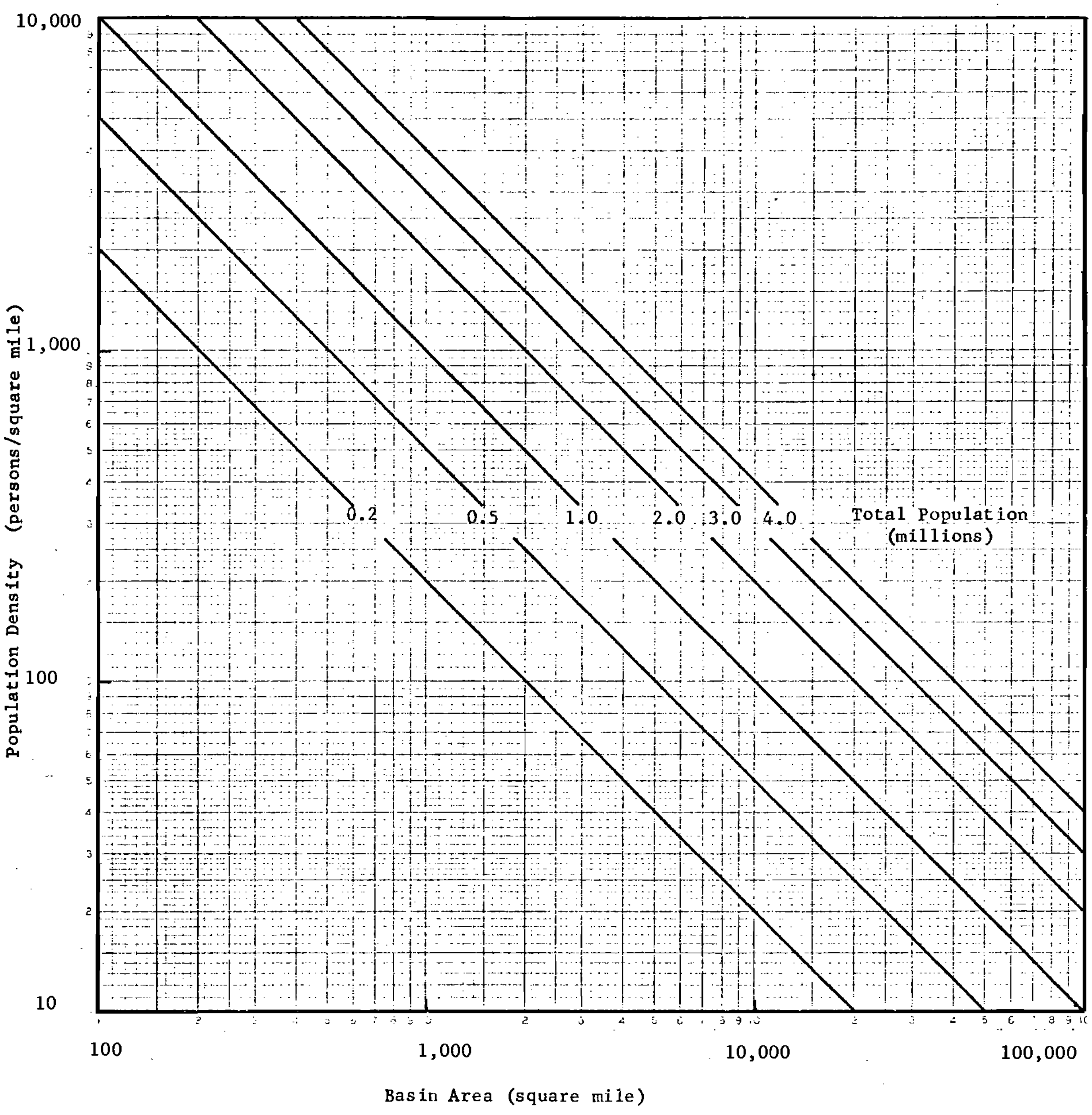


Figure 3.4

Basin Area - Population Density Relationship

Table 3.3

Population and Density in Groups of Places
Classified According to Size

Area	Population	Land Area in Square Miles	Population Per Square Mile of Land Area	Population Per Acre of Land Area
Places of 1,000,000 or more	17,484,059	1,261	13,865	21.7
Places of 500,000 to 1,000,000	11,110,991	1,888	5,885	9.2
Places of 250,000 to 500,000	10,765,881	2,401	4,484	7.0
Places of 100,000 to 250,000	11,652,426	2,728	4,271	6.7
Places of 50,000 to 100,000	13,835,902	3,539	3,910	6.1
Places of 25,000 to 50,000	14,950,612	5,319	2,811	4.4
Places of 10,000 to 25,000	17,568,286	6,939	2,532	4.0
Places of 5,000 to 10,000	9,779,714	5,005	1,954	3.1
Places of 2,500 to 5,000	7,580,028	5,242	1,446	2.3
Urban-fringe areas	10,540,851	5,917	1,781	2.8
Rural territory	54,054,425	3,508,736	15	0.023
United States	179,323,175	3,548,974	51	0.080

Source: U.S. Department of Commerce, Bureau of the Census, 1960; Census of Population, Volume 1: Characteristics of the Population, Part A: Number of Inhabitants.

The nominal value of the constant is 5. The channel properties, roughness and slope, are also arbitrary since they only affect the velocity (R) which is reflected in the time of travel and the reaeration coefficient and have no import in themselves. A Manning number (MN) of 0.060 and a slope (S) of 0.001 ft/ft were chosen because they provided reasonable times of travel (40.82 mi/da = 2.5 fps) and reaeration coefficients (0.40/day, base 10, 20°C). The resulting depth of flow and width of channel were 7.96 ft and 39.79 ft, respectively.

The nominal values of 21.5°C (70.63°F) and 1.68°C (62.25°F) were chosen for the temperature of the wastewater (TW) and the receiving water (TRO), respectively. These values were calculated for the 244th day of the year (September 1st) from the deterministic forcing functions of the stochastic temperature models presented in Chapter VI. These temperatures are consistent with the time of the low-flow condition determined by the stochastic daily streamflow model, also presented in Chapter VI. The resultant river water temperature (T) after dilution is 18.36°C.

The input BOD (LW) was assumed to have a nominal value of 30 mg/l which is representative of the ultimate carbonaceous BOD from a conventional secondary treatment plant. The initial stream BOD (LRO) before wastewater discharge was designated a nominal value of 3 mg/l which is typical of an unpolluted natural water.

The selected nominal values for temperature resulted in a DO saturation (DSAT) value of 9.24 mg/l. The initial stream DO (DRO) was assumed to be 90 percent of saturation or 8.32 mg/l. The DO of the wastewater (DW) was assumed to be 4.0 mg/l, an arbitrary level.

The selection of nominal values for the deoxygenation and reaeration rate constants demands particular scrutiny. From Table 3.4 it can be seen that generally typical values do not exist. Values were chosen such that they would fall within an acceptable range and produce reasonable results with other nominal values incorporated into the model. The nominal value for $K_{1_{20^\circ C}}$ is 0.20/day (0.186/day @ 18.36°C) and that for K_2 is 0.40/day (0.381/day @ 18.36°C).

Table 3.4

Deoxygenation and Reaeration Rate Constants
at 20°C Base 10

(Base e = Base 10 x 2.3)

K_1	Remarks	K_2	Remarks
.09 - .24	Sewage (Gotaas)	.115	Quiescent water (Adeney)
Avg. .10			
.04 - .30	Stream (Gotaas)	.05 - .96	Gotaas
.04 - .30	Sewage (Kothandaraman)	.17 - .30	Large streams (Fair & Geyer)
Avg. .10			
.04 - .13	Delaware River	.09 - 4.0	Eckenfelder and O'Connor
.05 - .24	Sacramento River	.26	Average (Kothandaraman)
Avg. .13			
		.05 - .30	Delaware River
.02 - .14	Ohio River		
Avg. .07		2.57	Illinois River (cold, turbulent)
		.05 - .51	Holston River (large, moderate velocity)
		Avg. .27	

The resulting minimum DO (DRMIN) is 5.57 mg/l which occurred at 44.14 miles downstream of the wastewater discharge and required 1.08 days (TMIN) to be reached.

3.4 Model Sensitivity to Nominal Parameter Values

A range of values was selected for each of the parameters previously described (MN, S, X, QRO, LRO, DRO, TRO, K_1, LW, TW, and QW). The DO sag model was run over each range individually to assess the response of the model to a change in the nominal value of a particular parameter. The minimum stream DO resulting from parameter values was calculated as a percent deviation from the nominal minimum stream DO

$$\frac{Dr_{min} - Dr_{min(nominal)}}{Dr_{min(nominal)}} \times 100\% \tag{3.48}$$

The nominal values of parameters and their tested ranges are presented in Table 3.5. Before these results are discussed, a few general comments should be made.

(a) If $K_1L_o < K_2D_o$, then $Dr_{min} = Ds_T - D_o$.

Thus, the boundary condition causes a discontinuity in the system. This discontinuity can be identified when TMIN = 0.0.

(b) Since the nominal Dr_{min} is closer to the saturation level than zero, the negative deviations are potentially higher than the positive deviations.

(c) The sensitivity of the stream is measured in terms of the nominal values. Thus, it is dependent on the nominal values.

The Manning number, a measure of channel roughness, was varied across its entire physical range. The range of stream response was + 23.6 to - 19.5% from the nominal. At high Manning numbers the velocity is low and so, in turn, is K_2, thus Dr_{min} is reduced. At low Manning numbers the velocity is so high that K_2 is raised to the point where $K_2D_o > K_1L_o$ and $Dr_{min} = Ds_T - D_o$.

Table 3.5

Nominal Values and Ranges of Parameters

Parameter	Nominal Value	Range min	Range max
*Manning number (MN)	0.060	0.01	0.10
*Slope (S)	0.001 ft/ft	0.0001	0.0100
*Width/depth ratio (X)	5	1	50
*Streamflow (QRO)	527 cfs	10	30,000
Stream BOD (LRO)	3 mg/l	0	50
Stream DO (DRO)	90% of SAT	20	100
Stream temperature (TRO)	16.8°C	0	36
Deoxygenation coefficient ($K_{1_{20^\circ C}}$)	0.20 BASE 10	.01	.50
Wastewater BOD (LW)	30 mg/l	0	200
Wastewater DO (DW)	4 mg/l	0	10
Wastewater temperature (TW)	21.5°C	5	30
*Wastewater flow (QW)	263 cfs	0	1,000

*Denotes arbitrary nominal values selected in such a way as to obtain reasonable values for velocity, K_2, and Dr_{min}.

The same phenomenon occurs when channel slope is varied, although even lower velocities are induced. The range of stream response was + 23.6 to -47.6 percent from the nominal.

With an increase in breadth-to-width ratio the stream velocity and depth are decreased. Both of these actions increase the value of K_2. Again, the above phenomenon occurs with a range of stream responses of + 23.6 to -52.5 percent from the nominal.

As the streamflow is increased, a convex response curve results in an optimum streamflow. Although velocity is increased with increasing flow, depth is increased as well. The net effect is a decrease in K_2. After the optimum discharge is reached, the decrease in K_2 overrides the dilution effect of increased streamflow and Dr_{min} decreases. This phenomenon is a function of the stage-discharge curve and in application would probably not exist because of the movement of high flows in a flood plain as well as the channel proper. The stream DO response ranged from + 33.0 to -63.4 percent from the nominal.

The addition of stream BOD simply reduces the stream DO until a limit of zero is reached. The range of stream response is + 8.1 to -100 percent of the nominal. The variation in stream DO has an analogously simple response with the range of deviations from + 4.0 percent at 100 percent saturation to -53.9 percent at 20 percent saturation.

The greatest effect of varying stream temperature is in varying the DO saturation values and thus stream DO which is a percentage of saturation. An increase in temperature causes a decrease in Dr_{min} through a decrease in DRO and an increase in K_1. The range of stream response is + 48.9 percent at $0^{\circ}C$ to -42.6 percent from the nominal at $36^{\circ}C$.

With an increase in the value of K_1 (the rate at which the waste is biochemically oxidized), the minimum stream dissolved oxygen concentration is decreased. At values below about 0.09 ($20^{\circ}C$, base 10), $K_1L_o < K_2D_o$ and $Dr_{min} = D_{s_T} - D_o$. The range of stream response is + 23.6 (at $K_{1_{20^{\circ}C}} = 0.1$) to 39.7 percent (at $K_{1_{20^{\circ}C}} = 0.50$) from the nominal.

The addition of input BOD reduces the stream DO to zero at the lower end of the response range. At the upper end of the response range, $K_1L_o < K_2D_o$ and $Dr_{min} = D_{s_T} - D_o$. Thus, the range of stream response is + 23.6 to -100 percent from the nominal.

Varying the input DO over its entire physical range produced little response from the stream. The variation of stream response is + 10.8 to -11.4 percent from the nominal Dr_{min}.

As was the case with stream temperature, the greatest effect of varying wastewater temperature is in varying DO saturation values. Since stream DO is expressed as a percentage of saturation, it varies directly with temperature. The result is a range in stream response of + 22.4 to -10.4 percent from the nominal.

An increase in wastewater flow increases both depth and velocity with a net result of decreasing K_2. The increase in volume is also an increase in mass loading, thus L_o is increased. The effect of reducing K_2 and increasing L_o is to continuously decrease Dr_{min}. The range in stream responses is + 53.8 to -71.4 percent from the nominal Dr_{min}.

CHAPTER IV

DETERMINISTIC ANALYSIS

The water quality model described in the previous chapter was employed to examine the effects of size, number, and location of wastewater treatment plant discharges on receiving waters of different sizes; that is, stream sizes providing different dilution ratios. Six different stream system lengths were explored: 64, 128, 192, 256, 320, and 384 miles. For each stream system length, six different plant systems were examined (1, 2, 4, 8, 16, and 32 plants) in a deterministic analysis. The configurations of these plant systems are illustrated in Figure 4.1. It was assumed that the treatment plant discharges of each system were equally spaced. The length of stream between discharge points defined a reach; thus, the stream system length, s, and the number of plants, n, specify the distance between plants (or reach length) as s/n and the number of reaches as n + 1. Additionally, an extra reach following that of the last load point was considered to be of variable length. This physical arrangement is fundamental to all experiments with the water quality model.

For each combination of stream system length and number of plants in the system, the computer model was run for a series of seven dilution ratios (1/1, 2/1, 4/1, 10/1, 20/1, 40/1, and 80/1). The dilution ratio is defined as the ratio of the initial stream flow and the total wastewater flow generated by the region. In the above experiments, it was assumed that tributary streamflow was due solely to wastewater discharges. An additional experiment was performed in which the streamflow was increased with length in accordance with a specified area-runoff function. The results of these analyses are discussed below.

4.1 Effect of Size, Number, and Location of Plants

The control or base system for this water quality evaluation involves a stream system 64 miles in length with one centralized regional wastewater treatment plant and a dilution ratio of 2/1. The water quality effects of disaggregating the region's wastewater through a multiple point discharge scheme is now examined. For each case in which the model

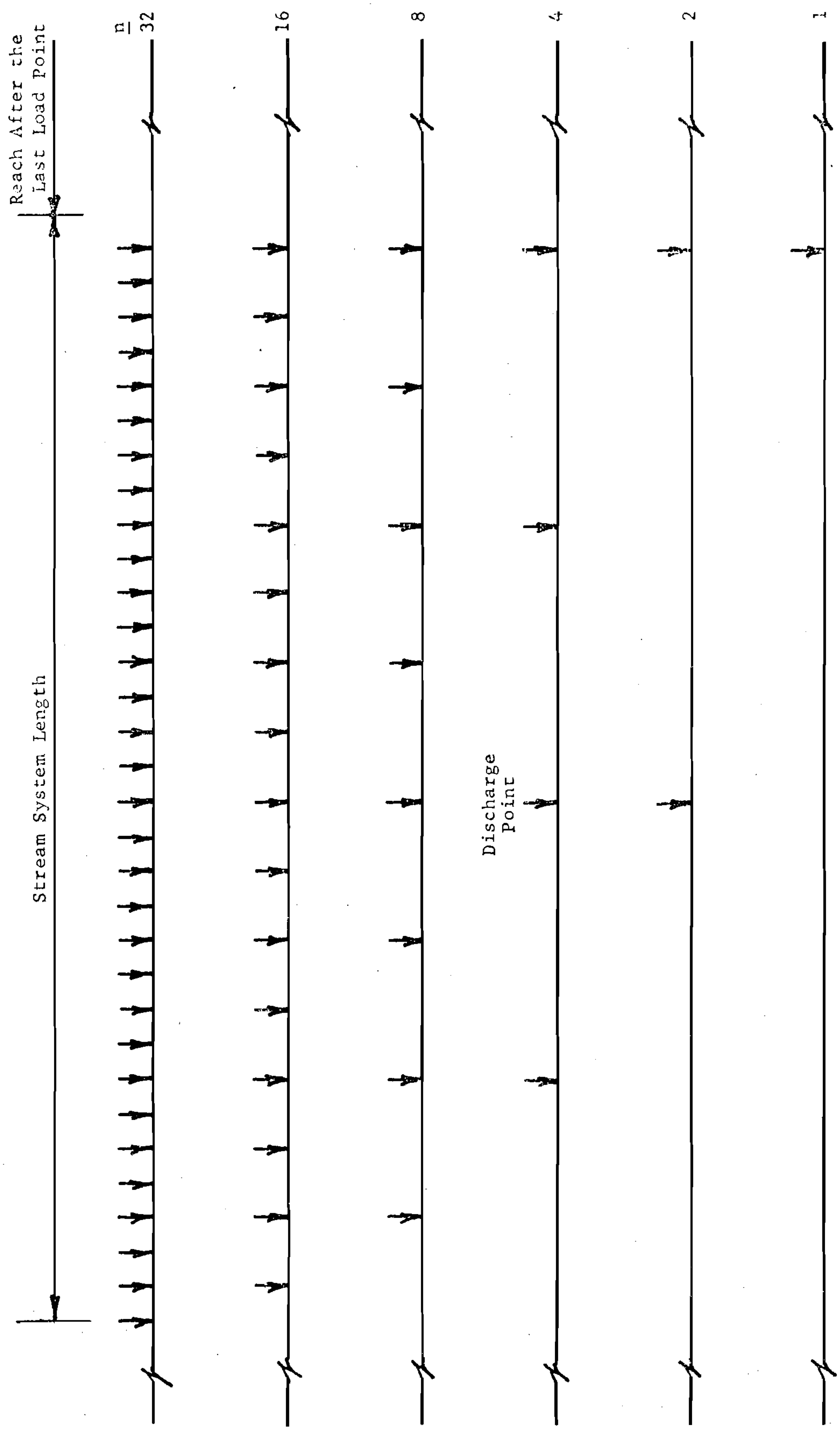


Figure 4.1
Regional Wastewater System Configurations

was run, the numerical model parameter values were equal to those in the base system with the exception of wastewater flow which was equally divided among the plants serving the region. Thus, the flow from each plant in an n plant system was of the same strength and was equal to QW/n where QW is the wastewater flow in the single plant system of the control condition.

For each computer run, each reach in the system was searched for its minimum dissolved oxygen concentration, and the minimum DO of all reaches was determined. These results are summarized in Table 4.1 from which it is evident that disaggregating the region's wastewater from a single discharge point to 32 uniformly spaced discharge points results in a 0.29 mg/l improvement in the minimum dissolved oxygen concentration. Furthermore, plant systems with greater than about 8 plants result in a negligible water quality improvement over that number of plants.

As the distance between discharge points increases or as the stream system length increases, there is a greater travel time between wastewater discharge points and a greater opportunity for self-purification of the water course. In order to quantify the degree of water quality improvement attributable to increased distance between discharges, the water quality model was run for a series of stream system lengths, each for a series of numbers of plants in the system. The shortest system length (64 miles) is viewed as being representative of a metropolitan region while the longer system lengths are viewed as being representative of more rural regions. For each combination of number of plants in the system and stream system length, the Dr_{min} was calculated for each reach and the absolute Dr_{min} for the entire system was determined. A summary of these calculations is presented in Table 4.2.

An examination of these results indicates a dramatic water quality improvement as the distance between plants is increased. This may be seen from Figure 4.2 in terms of the number of plants in the system or equivalently from Figure 4.3 in terms of the distance between plants. Figure 4.2 indicates that for a given stream system length, water quality improvement is experienced by additional numbers of plants in the system. However, the water quality improvements achieved by systems with greater

Table 4.1

Effect of Location, Size, and Number of Plants on Water Quality

Number of Plants	Reach Length (miles)	Critical Reach	DR_{min} (mg/l)	$^*\Delta DR_{min}$ (mg/l)
1	64	1	5.56	0
2	32	2	5.75	0.19
4	16	5	5.81	0.25
8	8	9	5.84	0.28
16	4	17	5.85	0.29
32	2	33	5.85	0.29

$^*\Delta DR_{min} = DR_{min}$ (n Plant) - DR_{min} (1 Plant)

TABLE 4.2

Effect of System Length and Number of Plants on Water Quality

System Length (Miles)	64		128		192		256		320		384	
Number of Plants	DR_{min}[1]	DIST[2]	DR_{min}	DIST	DR_{min}	DIST	DR_{min}	DIST	DR_{min}	DIST	DR_{min}	DIST
1	5.56		5.56		5.56		5.56		5.56		5.56	
2	5.75	32	6.10	64	6.49	96	6.73	128	6.73	160	6.73	192
4	5.81	16	6.27	32	6.75	48	7.13	64	7.31	80	7.44	96
8	5.84	8	6.32	16	6.83	24	7.24	32	7.50	40	7.65	48
16	5.85	4	6.35	8	6.85	12	7.28	16	7.57	20	7.74	24
32	5.85	2	6.36	4	6.87	6	7.30	8	7.60	10	7.78	12

[1] DR_{min} = minimum stream dissolved oxygen concentration (mg/l)

[2] DIST = distance between plants (miles)

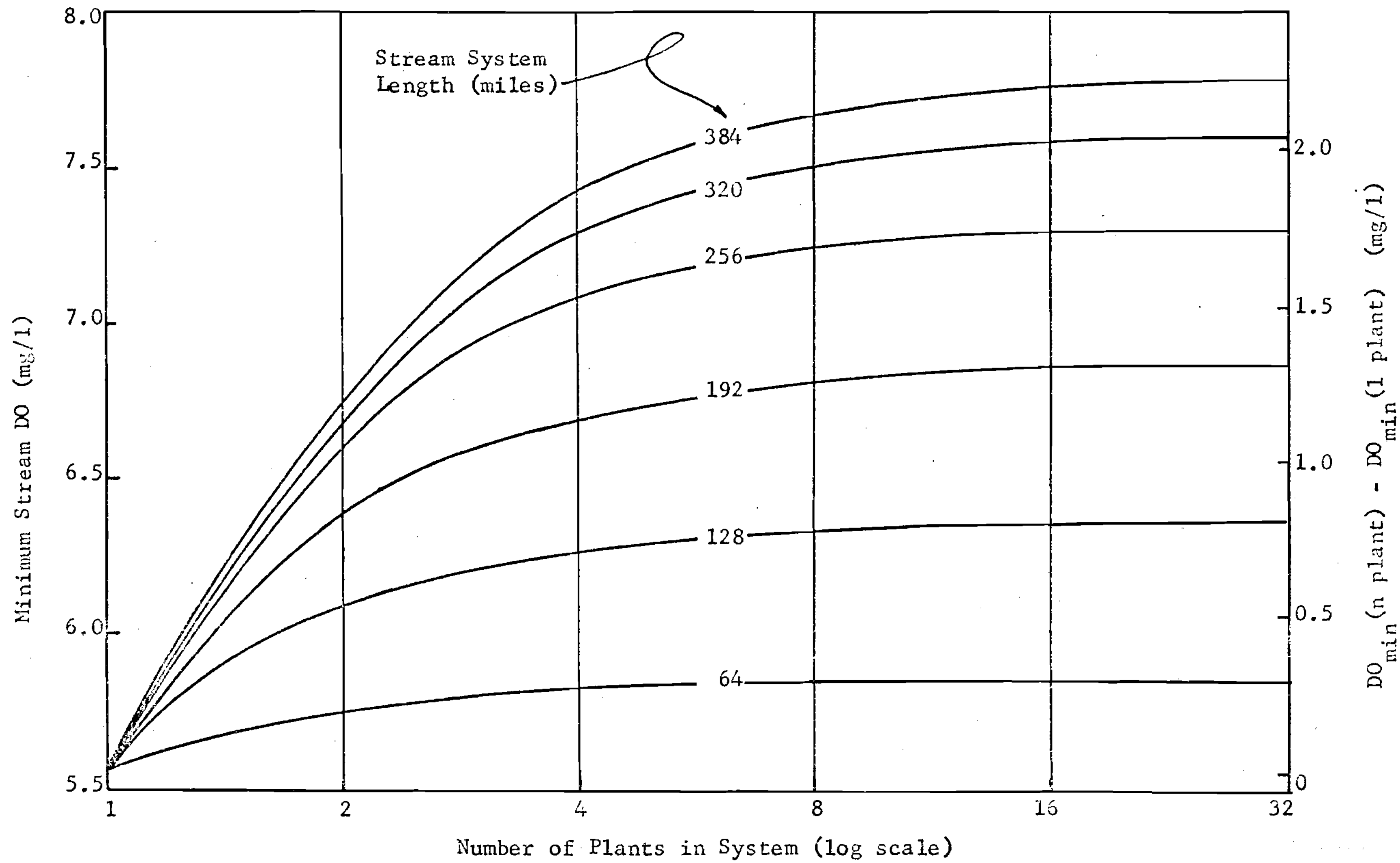


Figure 4.2

Effects of the Number of Plants in the System on Water Quality for Various System Lengths

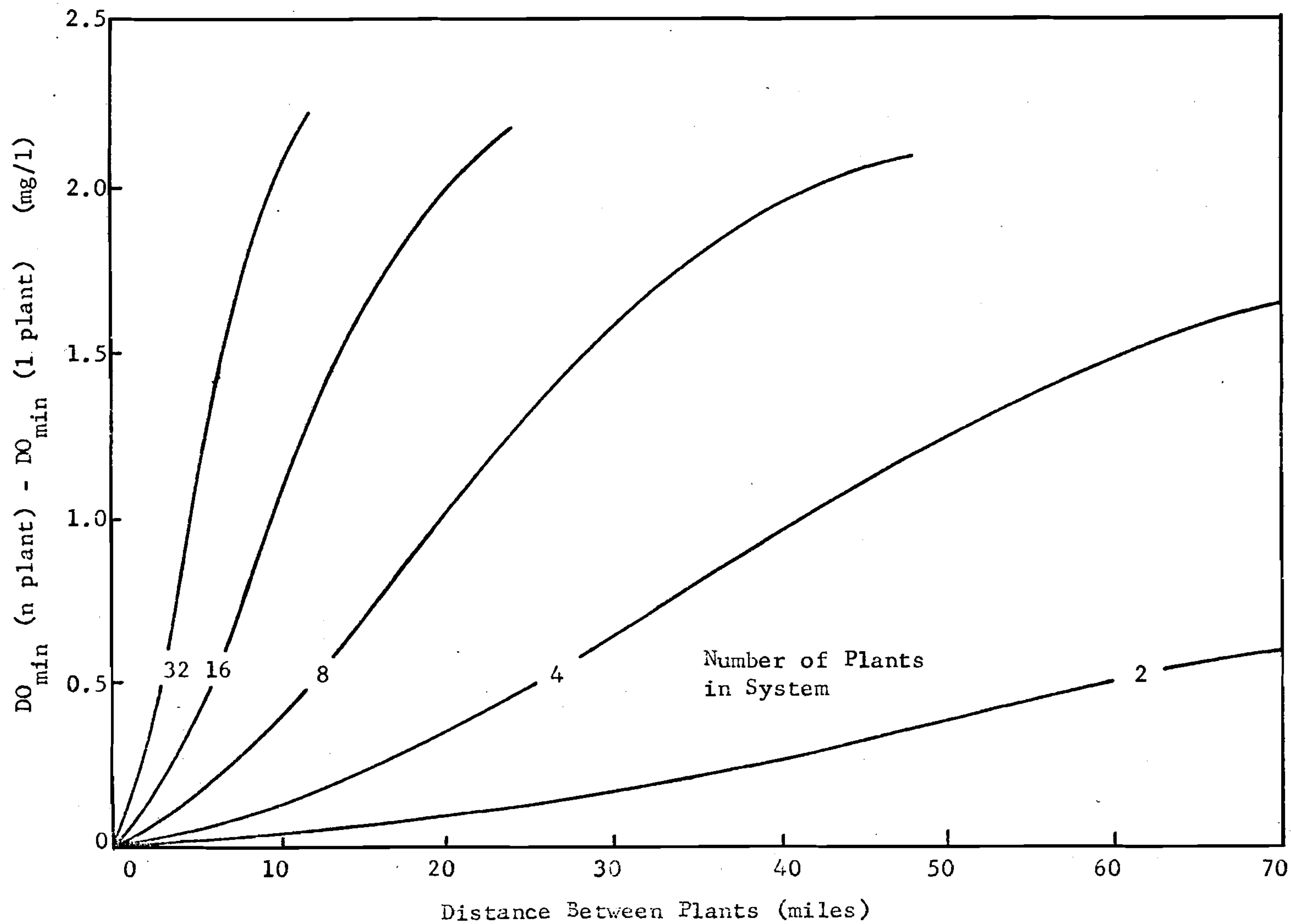


Figure 4.3

Effects of Distance Between Plants on Water Quality for Various Numbers of Plants than about 8 plants are only marginally greater than that achieved by an 8 plant system. Furthermore, as the stream system length is increased, substantial water quality improvements are experienced. Similarly, Figure 4.3 indicates that plant systems with large numbers of plants experience greater water quality improvement from small increments in distance between plants than do systems with small numbers of plants.

4.2 Location and the Effect of Dilution

It was noted in the previous chapter that although a nominal dilution ratio of 2/1 was stipulated, the dilution ratio is a function of the region's physical environment: the low flow-drainage area function, length of stream under consideration, relative location of this length of stream, and the basin geometry. With this in mind, it would be useful to develop information concerning the effect of the dilution ratio on the water quality response to regional wastewater system centralization. An experiment was performed which employed the previously described water quality model to assess these effects. The model runs of the previous section for each combination of stream system length and number of plants in the system were repeated for a series of dilution ratios: 1/1, 2/1, 4/1, 10/1, 20/1, 40/1, and 80/1. The output from these computer runs is summarized in Tables 4.3 and 4.4.

Figure 4.4 presents plots of the water quality improvement due to an n plant system, ΔDO_{min} (defined as the difference between DO_{min} of an n plant system and DO_{min} of a 1 plant system), for the number of plants in the system. A family of curves is presented for a variety of dilution ratios for the nominal 64 mile stream system. It is evident from Figure 4.4 that the water quality improvement due to the disaggregation of plants is greatest at small dilution ratios and becomes negligible at higher dilution ratios. Again, a breakoff in the increase in water quality improvement is evident at an aggregation state of about 8 plants, beyond which the improvement over an 8 plant system is only marginal. The water quality improvements for a 32 plant system, $\Delta DO_{min}(32)$, at a 1/1 dilution ratio are 0.5 and 2.9 mg/1 for the 64 and 384 mile systems, respectively.

Table 4.3

Effect of System Length, Number of Plants, and Dilution Ratio on Minimum DO Concentration (in mg/1)

Dilution Ratio	Stream Length (miles)	Number of Plants in System 1	2	4	8	16	32
1	64	4.73	5.06	5.16	5.19	5.20	5.21
	128	4.73	5.60	5.82	5.89	5.91	5.92
	192	4.73	6.09	6.43	6.52	6.55	6.57
	256	4.73	6.14	6.85	6.99	7.03	7.05
	320	4.73	6.14	7.06	7.30	7.36	7.38
	384	4.73	6.14	7.11	7.49	7.58	7.62
2	64	5.56	5.75	5.81	5.84	5.85	5.85
	128	5.56	6.10	6.27	6.32	6.35	6.36
	192	5.56	6.49	6.75	6.83	6.85	6.87
	256	5.56	6.73	7.13	7.24	7.28	7.30
	320	5.56	6.73	7.31	7.50	7.57	7.60
	384	5.56	6.73	7.44	7.65	7.74	7.78
3	64	6.27	6.36	6.40	6.42	6.45	6.44
	128	6.27	6.56	6.71	6.73	6.75	6.76
	192	6.27	6.82	7.02	7.09	7.12	7.14
	256	6.27	7.10	7.31	7.42	7.47	7.49
	320	6.27	7.12	7.44	7.59	7.65	7.68
	384	6.27	7.12	7.55	7.67	7.74	7.78
10	64	6.86	6.90	6.92	6.92	6.93	6.93
	128	6.86	6.97	7.03	7.06	7.07	7.08
	192	6.86	7.08	7.20	7.25	7.28	7.30
	256	6.86	7.23	7.37	7.44	7.47	7.49
	320	6.86	7.35	7.46	7.53	7.56	7.58
	384	6.86	7.35	7.50	7.58	7.62	7.64
20	64	7.06	7.07	7.08	7.09	7.09	7.09
	128	7.06	7.10	7.13	7.14	7.15	7.15
	192	7.06	7.15	7.21	7.24	7.25	7.26
	256	7.06	7.22	7.31	7.35	7.37	7.38
	320	7.06	7.30	7.37	7.41	7.43	7.45
	384	7.06	7.40	7.41	7.45	7.47	7.49

continued

Table 4.3 (cont.)

Effect of System Length, Number of Plants, and Dilution Ratio on Minimum DO Concentration (in mg/l)

Dilution Ratio	Stream Length (miles)	Number of Plants in System 1	2	4	8	16	32
40	64	7.10	7.10	7.10	7.11	7.11	7.11
	128	7.10	7.11	7.12	7.13	7.13	7.13
	192	7.10	7.13	7.15	7.16	7.17	7.17
	256	7.10	7.15	7.19	7.20	7.23	7.23
	320	7.10	7.18	7.24	7.26	7.27	7.28
	384	7.10	7.22	7.27	7.29	7.30	7.31
80	64	7.03	7.03	7.03	7.03	7.03	7.03
	128	7.03	7.03	7.03	7.04	7.04	7.04
	192	7.03	7.04	7.04	7.05	7.05	7.05
	256	7.03	7.04	7.06	7.06	7.07	7.07
	320	7.03	7.05	7.08	7.09	7.10	7.10
	384	7.03	7.07	7.10	7.11	7.12	7.12

Table 4.4

Effect of System Length, Number of Plants, and Dilution Ratio on Location of Critical Deficit (in miles from the first load point)

Dilution Ratio	Stream Length (miles)	Number of Plants in System 1	2	4	8	16	32
1	64	34.5	56.1	65.5	69.9	72.0	73.1
	128	34.5	86.4	108	117	122	124
	192	34.5	120	156	171	180	186
	256	34.5	26.7	142	164	192	208
	320	34.5	26.7	96.6	126	180	200
	384	34.5	26.7	20.8	104	168	180
2	64	44.2	63.0	71.8	76.0	78.0	79.0
	128	44.2	90.1	110	120	124	126
	192	44.2	122	156	171	180	186
	256	44.2	37.2	143	195	224	240
	320	44.2	37.2	97.5	125	180	200
	384	44.2	37.2	31.7	102	96.0	132
4	64	59.0	74.5	82.1	85.9	87.7	88.7
	128	59.0	96.7	115	124	128	130
	192	59.0	125	156	170	180	186
	256	59.0	157	141	161	176	182
	320	59.0	52.6	98.2	122	120	120
	384	59.0	52.6	122	100	96	96
10	64	89.9	100	105	108	109	110
	128	89.9	115	128	134	137	139
	192	89.9	133	157	169	180	186
	256	89.9	156	141	150	144	144
	320	89.9	84.5	109	123	120	130
	384	89.9	84.5	107	108	120	120
20	64	125	132	136	137	138	139
	128	125	139	149	154	156	157
	192	125	152	168	176	180	186
	256	125	166	155	164	176	176
	320	125	184	168	160	160	160
	384	125	121	108	149	145	156

continued

Table 4.4 (cont.)

Effect of System Length, Number of Plants, and Dilution Ratio on Location of Critical Deficit (in miles from the first load point)

Dilution Ratio	Stream Length (miles)	Number of Plants in System 1	2	4	8	16	32
40	64	176	180	181	182	183	184
	128	176	185	189	192	193	194
	192	176	191	199	204	206	207
	256	176	198	212	224	224	216
	320	176	207	197	204	201	210
	384	176	218	205	198	195	204
80	64	246	248	249	250	250	250
	128	246	251	253	254	255	256
	192	246	254	258	260	261	262
	256	246	256	256	268	270	271
	320	246	262	271	280	280	280
	384	246	267	288	264	267	276

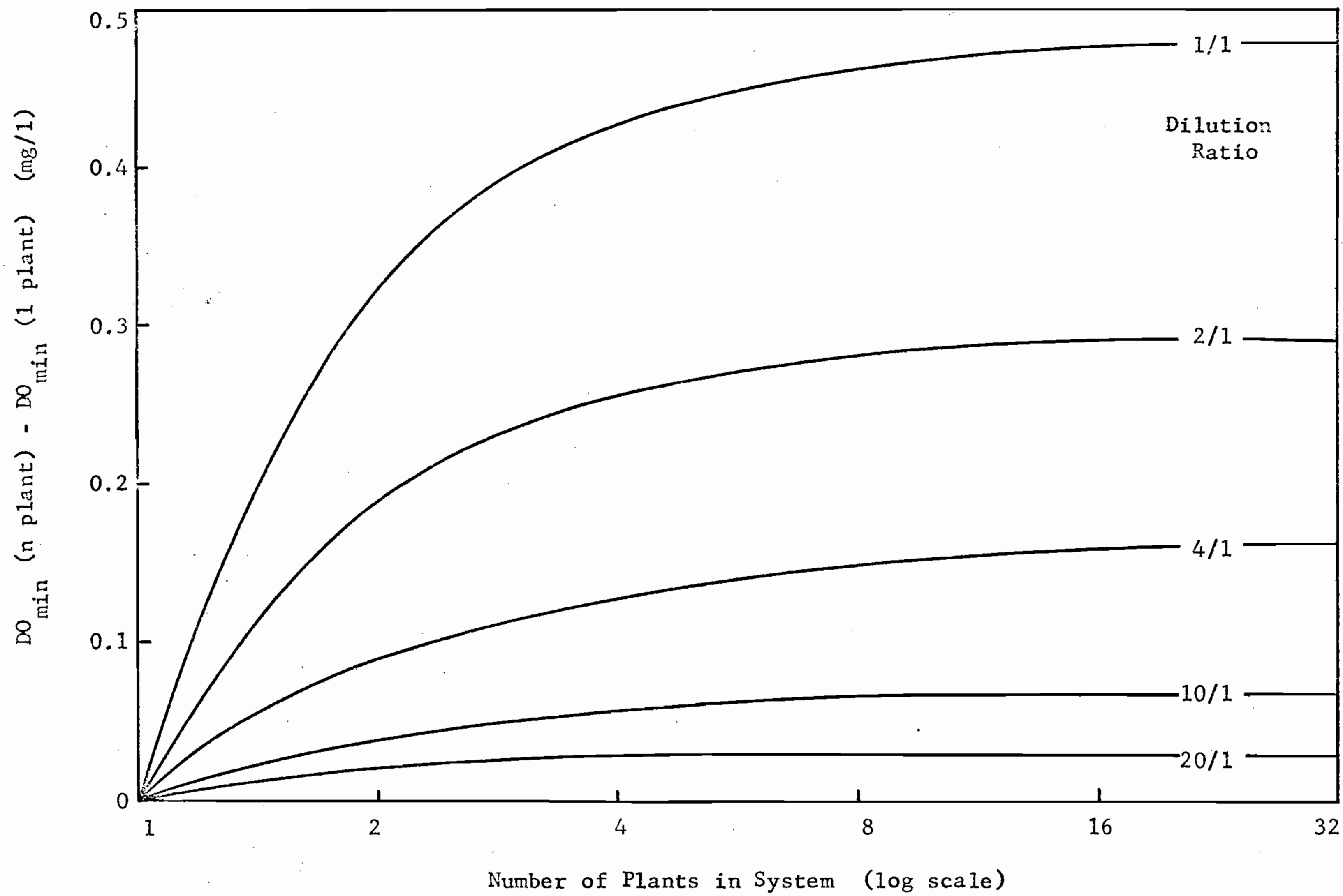


Figure 4.4

Effect of the Number of Plants in the System on Water Quality for Various Dilution Ratios

Figure 4.5 presents plots of DO_{min} and ΔDO_{min} versus dilution ratio for systems of various plant numbers and a stream system length of 384 miles. For each system of a fixed number of plants, there is a dilution ratio at which DO_{min} is maximized. Beyond that dilution ratio, the increased flow and depth of flow affect K_2 more significantly than L_o, resulting in a steadily decreasing DO_{min}. As the number of plants in the system increases, the dilution ratio at which DO_{min} is maximized decreases.

Figures 4.6, 4.7, 4.8, 4.9, and 4.10 present plots of DO_{min} and ΔDO_{min} versus dilution ratio for 32, 16, 8, 4, and 2 plant systems, respectively. Each figure presents a family of curves depicting various system lengths or, equivalently, distance between plants in the system. These plots serve to reinforce previous observations: (a) as n is increased, ΔDO_{min} is increased at a decreasing rate such that at a disaggregation state of 8 plants, the increase in ΔDO_{min} due to further disaggregation is negligible, (b) as the dilution ratio is increased, DO_{min} reaches a maximum for a given stream length and steadily decreases with increased dilution ratios, (c) for a given disaggregation state, ΔDO_{min} increases with an increase in distances between plants at a decreasing rate, and (d) the dilution ratio at which DO_{min} is maximized decreases with an increase in distance between plants.

4.3 Additional Effects of Augmented Streamflow

Since the previous experiment assumed that the flow throughout the length of the system is augmented only by wastewater contributions, the validity of this assumption is now examined by an analysis of stream systems in which the flow increases with length. This experiment assumes that there exists a 2/1 dilution ratio at the centroid of wastewater contributions and the flow is augmented by the following relationship:

$$q = 10^{-5} A \qquad (4.1)$$

in which q is the runoff per unit acre (cfs/sq mi) and A is the drainage area (sq mi). Assuming a drainage basin with a 3/1 length-to-width ratio, the drainage area may be expressed as a function of length as follows:

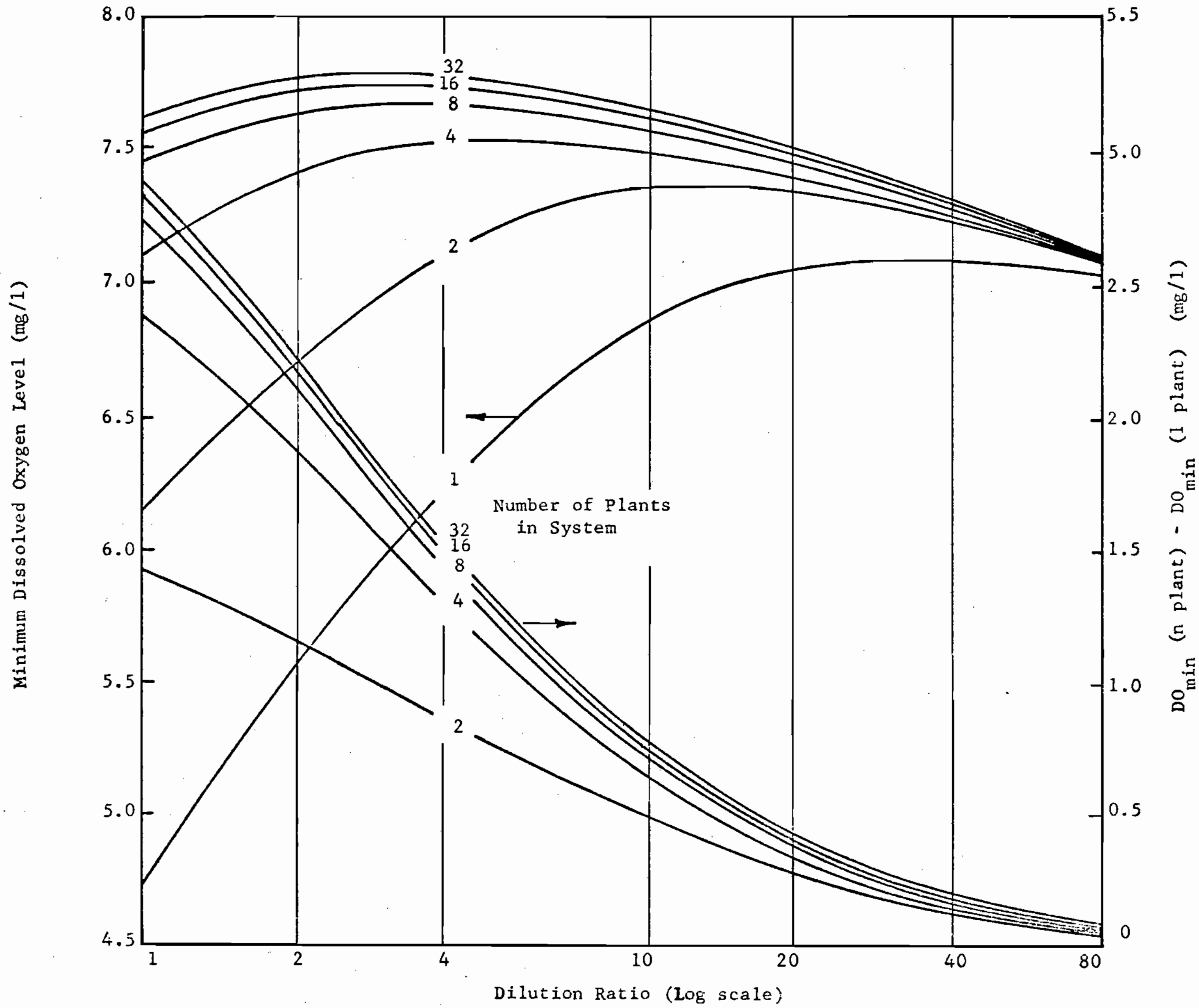


Figure 4.5

Effect of Dilution Ratio on Water Quality for
Various Number of Plants - 384 Mile System

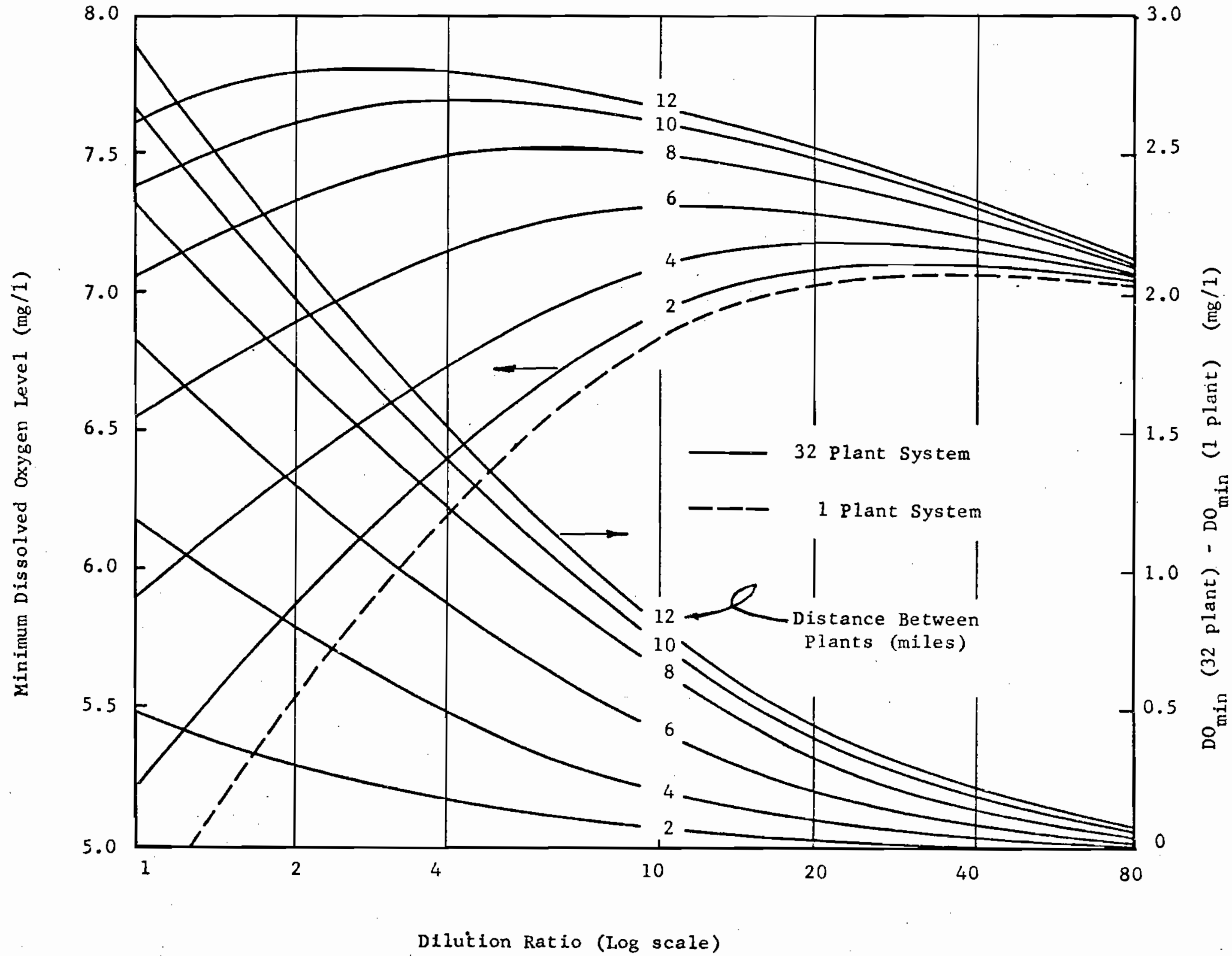


Figure 4.6

Effect of Dilution Ratio on Water Quality for Various Distances Between Plants - 32 Plant System

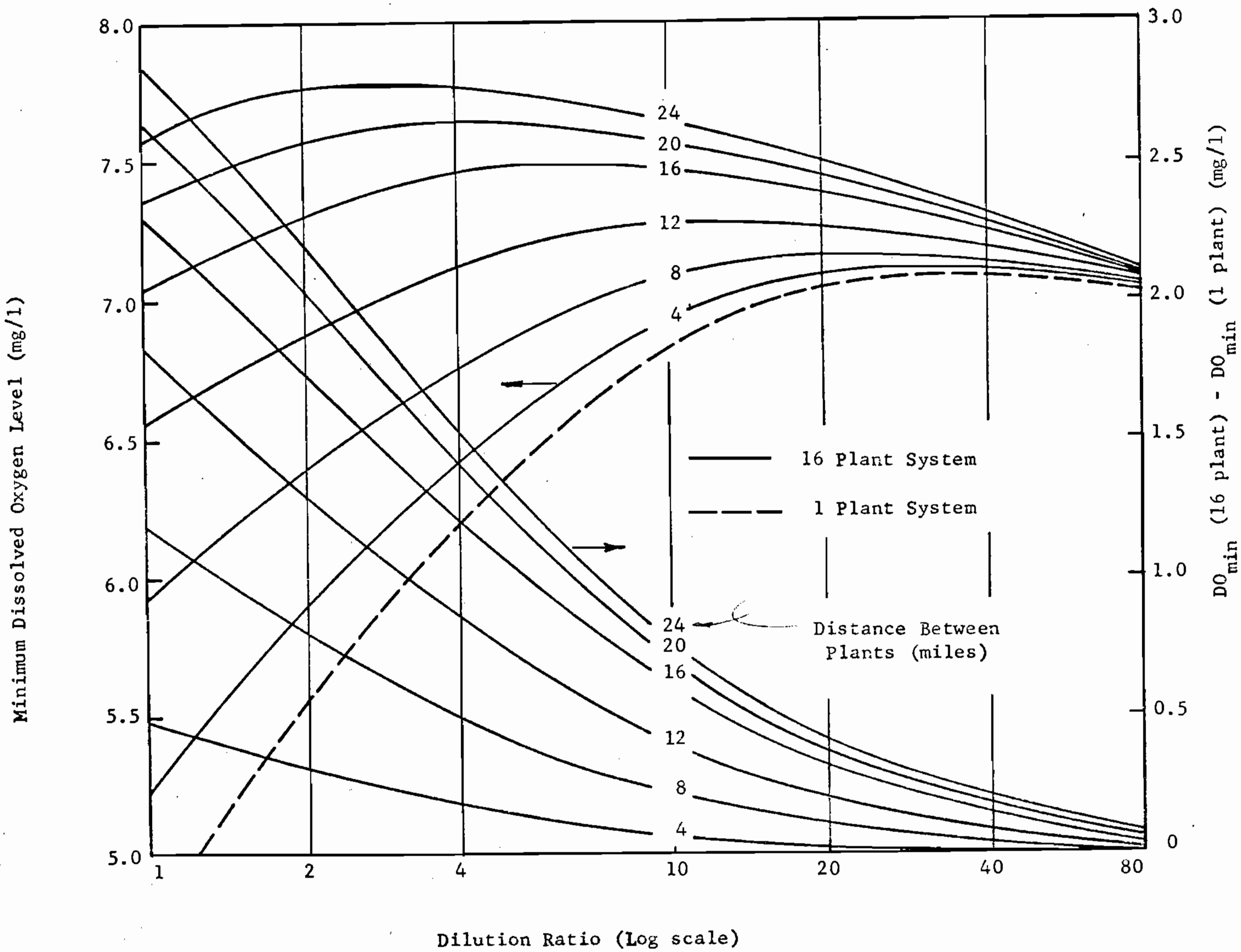


Figure 4.7

Effect of Dilution Ratio on Water Quality for Various Distances Between Plants - 16 Plant System

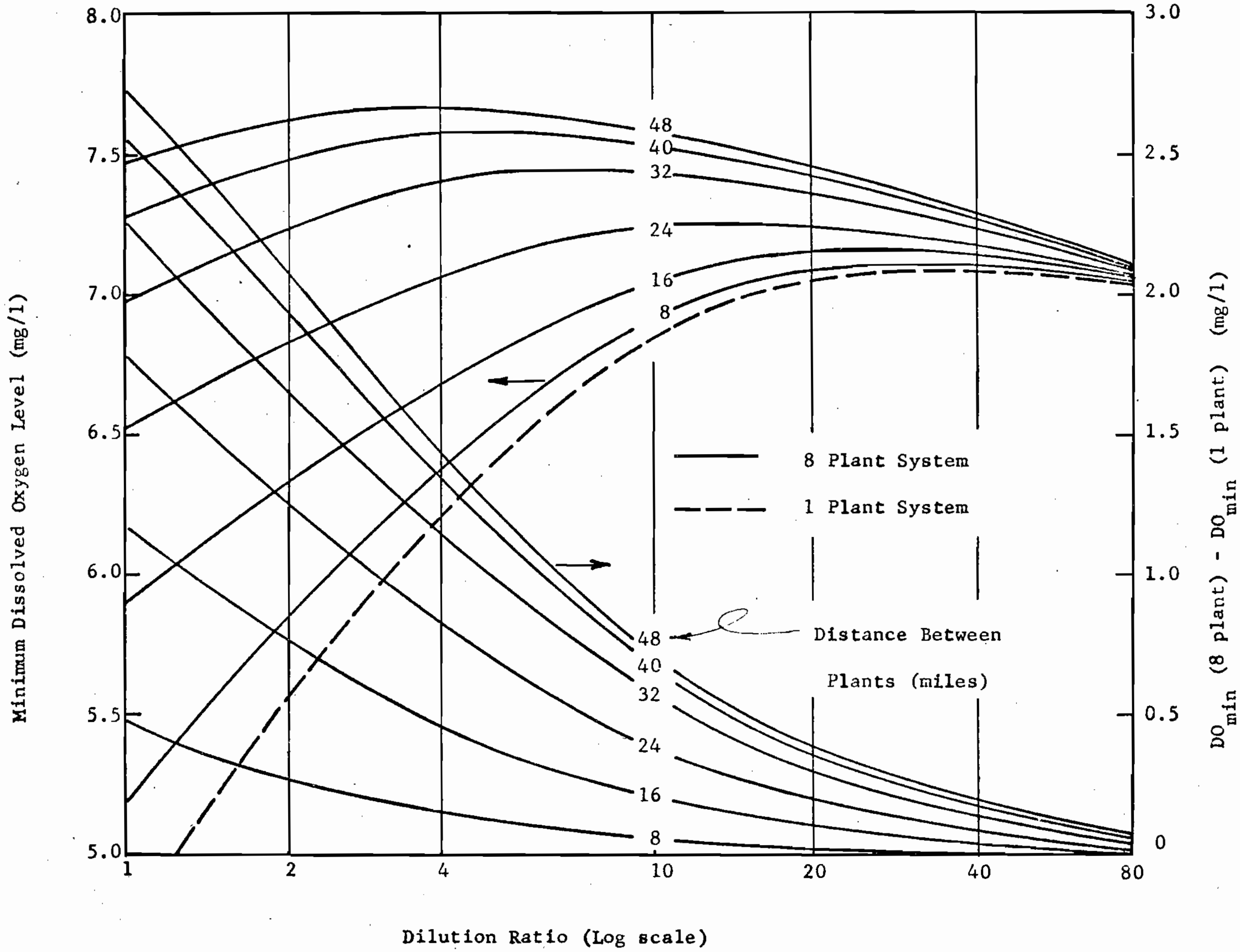


Figure 4.8

Effect of Dilution Ratio on Water Quality for Various Distances Between Plants - 8 Plant System

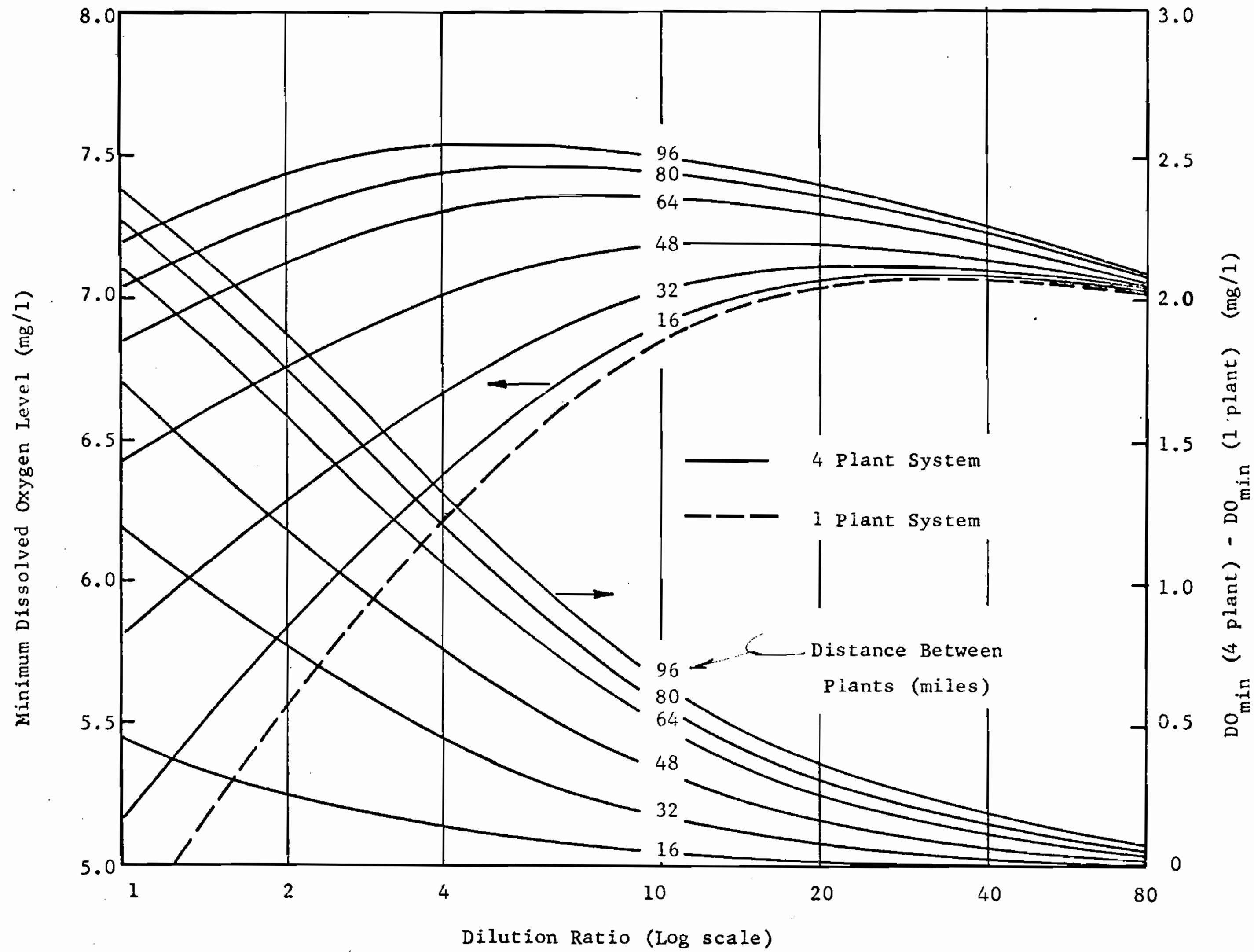


Figure 4.9

Effect of Dilution Ratio on Water Quality for Various Distances Between Plants - 4 Plant System

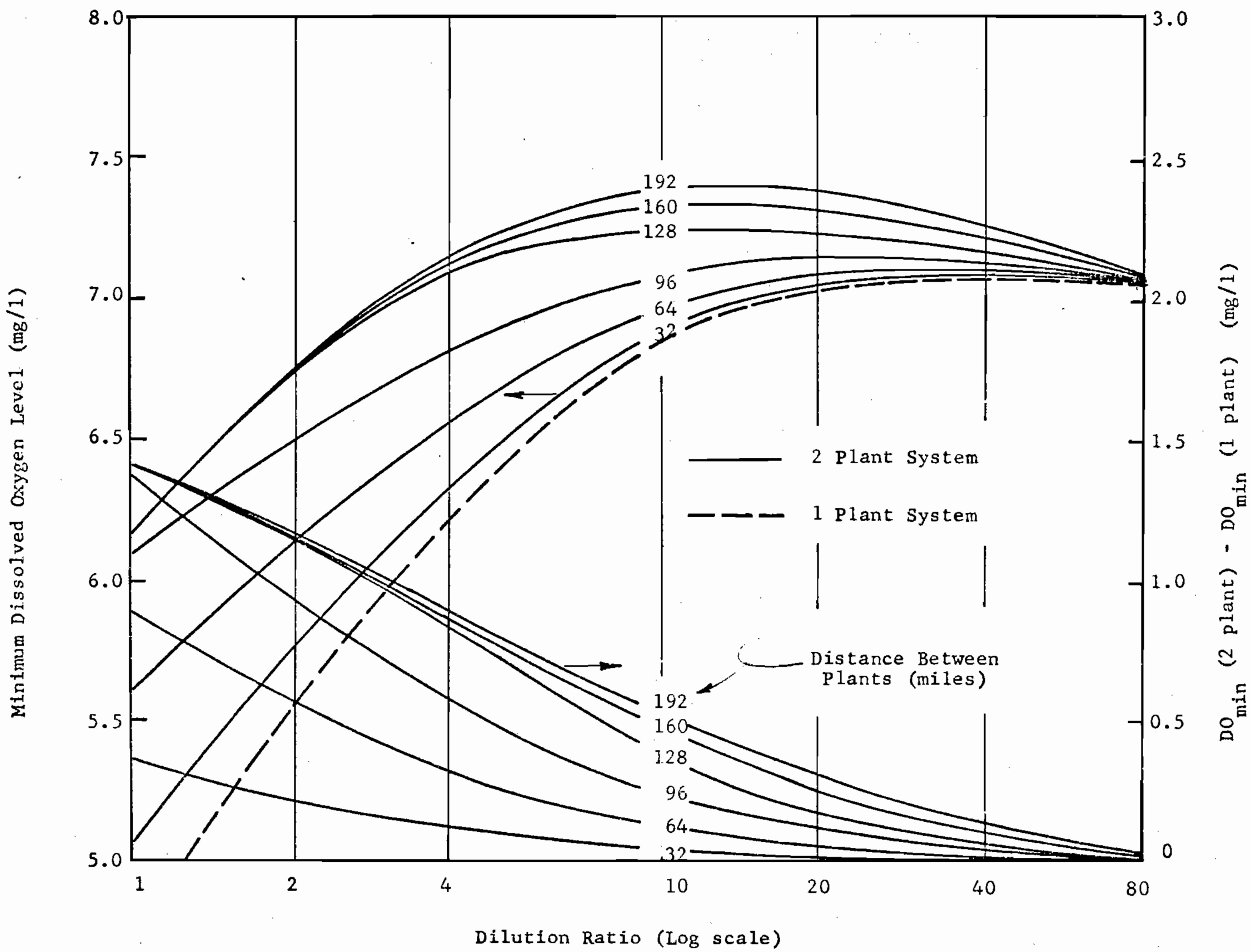


Figure 4.10

Effect of Dilution Ratio on Water Quality for Various Distances Between Plants - 2 Plant System

$$A = L^2/3 \tag{4.2}$$

in which L is the stream length in miles. Combining equations (4.1) and (4.2), the following relationship for streamflow is obtained:

$$Q = qA = 10^{-5} A^2 = 10^{-5} L^4/9 \quad . \tag{4.3}$$

in which Q is the streamflow in cfs.

The experiment was performed for each system of 32, 16, 8, 4, 2, and 1 plants on stream system lengths of 64, 128, 192, and 256 miles. The results of these experiments are summarized in Table 4.5. Plots of DO_{min} and ΔDO_{min} with both the number of plants in the system for various stream system lengths and the distance between plants for various plant number systems are presented in Figures 4.11 and 4.12 respectively. Through a comparison of Figures 4.11 and 4.12 with Figures 4.2 and 4.3, certain conclusions regarding the effect of incrementally augmented streamflow on the previous results may be drawn. Generally, the values of DO_{min} for a particular aggregation state are larger for the shorter stream system lengths (up to 192 miles) and smaller for the longer stream system lengths (256 miles and longer). The exception is the aggregation state of 2 plants in which the value of DO_{min} is uniformly larger in the augmented stream for even long stream lengths. Figure 4.12 indicates that for any aggregation state, there is a distance between plants at which DO_{min} is maximized in comparison to Figure 4.3 which indicates monotonically increasing DO_{min} levels. An explanation for this observation is given. Because the nominal dilution ratio of 2/1 was assumed at the centroid of wasteloads, the upstream reaches of long stream systems must have small streamflows in order to meet the constraint of equation (4.3). Thus, for long stream systems with large numbers of plants, the condition exists where the upstream plants discharge to reaches with small streamflows which inevitably results in the critical DO level occurring in those reaches.

4.4 Summary of Results

For a given stream size, or dilution ratio, and a given stream length, an increase in the disaggregation state of wastewater treatment

Table 4.5

Effect of System Length and Number of Plants on Water Quality for a Stream with Augmented Flow

System Length (miles)	Minimum Dissolved Oxygen Levels (mg/1) and ΔDO_{min} (mg/1) for Number of Plants in System					
	1	2	4	8	16	32
64	5.85	6.03 0.18	6.09 0.24	6.11 0.26	6.12 0.27	6.13 0.28
128	5.85	6.73 0.88	6.85 1.00	6.91 1.06	6.94 1.09	6.95 1.10
192	5.85	6.73 0.88	7.07 1.22	7.18 1.33	7.22 1.37	7.23 1.38
256	5.85	6.73 0.88	6.39 0.54	6.51 0.66	6.95 1.10	7.09 1.24

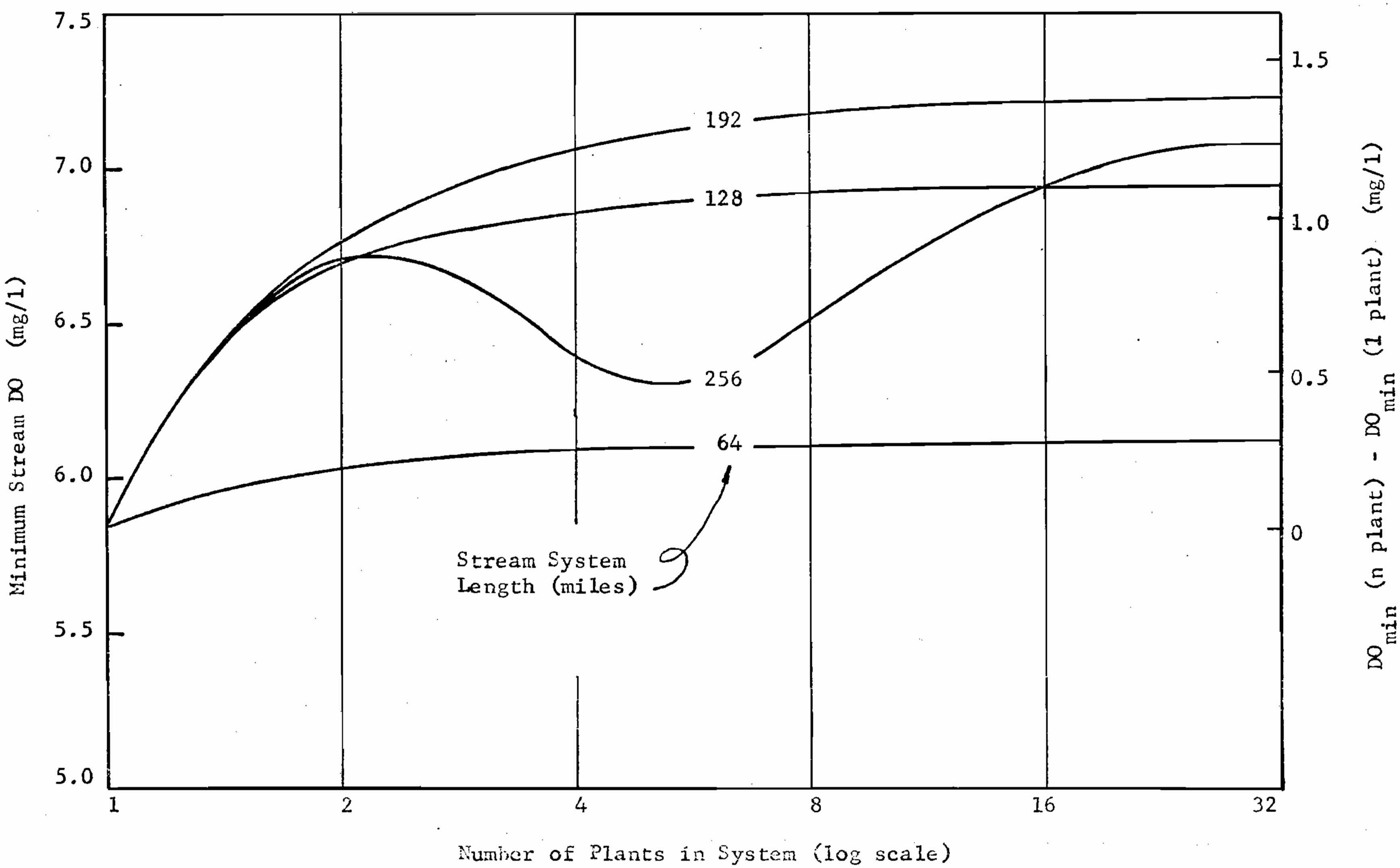


Figure 4.11

Effect of Number of Plants in the System on Water Quality for Various System Lengths of a Stream with Augmented Flow

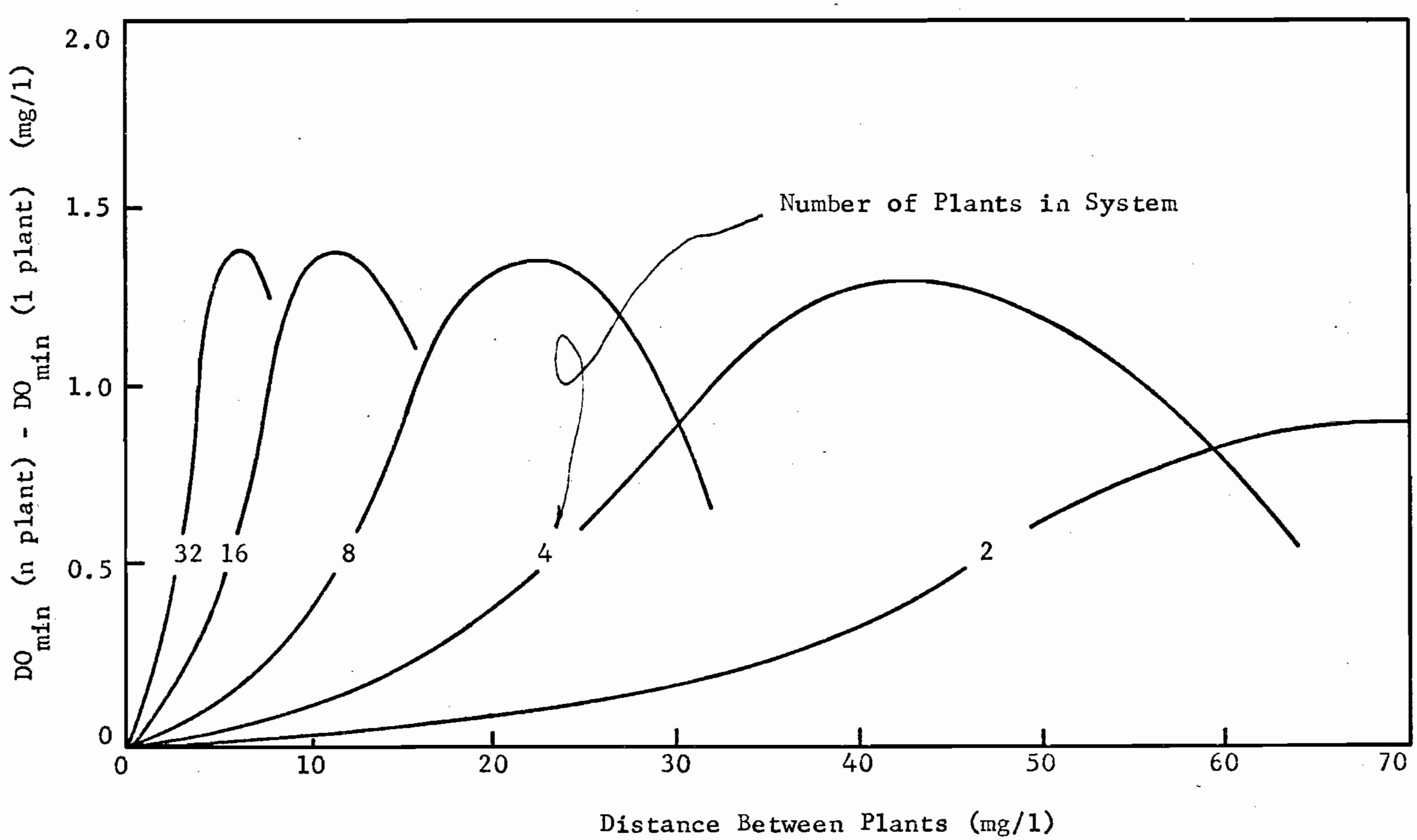


Figure 4.12

Effect of Distance Between Plants on Water Quality for Various Numbers of Plants for a Stream with Augmented Flow plants results in an improved water quality. This water quality improvement is negligible for short stream system lengths typical of metropolitan regions while it is considerable for longer stream system lengths indicative of more rural regions. The magnitude of the improvement may be greater than a 2 mg/l increase in the minimum stream dissolved oxygen level. It is noted that the greater part of the water quality improvement due to decentralization is achieved by a disaggregation state of approximately 8 plants. Further increases in disaggregation state result in only marginal improvements over an 8 plant system.

The water quality improvement resulting from a decentralized regional wastewater treatment system is also a function of the dilution ratio. For a given stream system length, the water quality improvement due to a fixed disaggregation state increases with a decrease in the dilution ratio. Thus, it is concluded that decentralization is more beneficial in cases with relatively small streamflows. Furthermore, for a given stream system length and a fixed number of plants in the system, there exists a dilution ratio at which DO_{min} is maximized. Beyond this dilution ratio, DO_{min} steadily decreases as previously explained.

The sensitivity of these experiments to the natural stream condition of augmented flow was assessed. It is concluded that although the values of DO_{min} are generally larger for short stream systems and smaller for longer stream systems, there exists the same general increase in water quality with an increase in disaggregated state.

CHAPTER V

WASTEWATER TREATMENT PLANT PERFORMANCE

A study of wastewater treatment plant performance was undertaken to provide the background for the development of the stochastic input models presented in the following chapter. The performance of individual plants and plant groups were studied. The purpose of studying individual plant performances was to examine and describe the nature of plant performance variability. This was accomplished through an examination of time series of plant influent and effluent variables. Among the components of performance variability studied were trend, cyclical variation, and random variation. Trend was studied by linearly regressing each of the influent and effluent parameters against time, and cyclical variation was determined by a periodic regression of each variable against time. The runs test and serial correlation were employed to study the random variation of the time series. The frequency distribution of parameters were compared to some selected theoretical distributions and the significance of fit was determined by the Kolmogorov-Smirnov one-sample test.

The purpose of studying regionally related groups of plants was to determine the degree of dependence in performance, between plants in pairs and among all plants as a group, in order to detect any underlying regional factors which may commonly influence the performance of plants within the region. This dependence was assessed through the use of simple and multiple correlation techniques of both the parametric and nonparametric type. Simple correlation was used to determine the dependence in performance between pairs of plants in a group, and multiple correlation was used to determine the dependence among all plants in a group. Simple linear correlation was measured by the parametric Pearson product moment correlation coefficient and simple rank correlation was measured by the non-parametric Kendall rank correlation coefficient. Multiple linear correlation was measured by the parametric multiple correlation coefficient and multiple rank correlation was measured by the non-parametric Kendall coefficient of concordance.

5.1 Data Description

The data used in the analyses were taken from activated sludge plants in the Chicago region. All plants are operated by the Metropolitan Sanitary District of Greater Chicago. By name they are Northside, Calumet, West Southwest, Hanover Park, and Hazelcrest, and they will be referred to as Plant 1, Plant 2, Plant 3, Plant 4, and Plant 5, respectively. The variables studied for each of these plants are the following: influent biochemical oxygen demand (BOD), effluent BOD, influent suspended solids (SS), effluent SS, effluent dissolved oxygen (DO), and discharge (Q). Although effluent variables as indices of performance were primarily of interest, influent variables were also examined. The influent and effluent quality variables were measured in milligrams per litre (mg/l) and discharge was measured in millions of gallons per day (MGD).

One calendar year of data was analyzed (January 1, 1969 - December 31, 1969). The samples are daily composite measurements; thus, the maximum sample size for each variable was 365. Unfortunately, there were occurrences of missing data. Two sets of data based on this original set was analyzed. If one or more plants had missing data on a particular day, then that day was deleted from the sequence for that particular variable to form the reduced data set. A supplemented data set was formed by filling in values wherever missing by interpolation. The supplemented data set has a sample size of 365 for all variables. The following are the sample sizes obtained for the reduced data set: influent BOD - 348, effluent BOD - 356, influent SS - 354, effluent SS - 362, effluent DO - 186, and discharge - 364. The following analyses were then conducted on these data sets.

The analyses for trend were performed on yearly sequences of daily data of both the reduced and supplemented forms while the analyses for cyclical variation were performed on yearly sequences of daily data of only the reduced data form. The analyses for randomness were run on various lengths of sequences of reduced daily data ranging from one month to one year, for each variable, for each plant. Because of the relatively frequent occurrence of missing data in the case of effluent DO, the tests for randomness were not considered for this variable.

Autocorrelation coefficients with multiple lags were calculated for yearly sequences of supplemented data. The distribution tests treated yearly sequences of the reduced data set. The analyses for dependence were also performed on yearly sequences of daily data of the reduced form.

Table 5.1 presents a statistical summary of the time series analyzed, based on yearly sequences of daily data of the reduced form.

5.2 Performance of Individual Plants

The time variation of treatment plant performance is subject to a diversity of forces. There may be both random and nonrandom factors present. Among the nonrandom factors which may be present are persistence or trend and cyclical movement. Around this nonrandom movement in time may be superimposed random variations. In examining and characterizing plant performance variability, it is profitable to decompose the variation into its most fundamental components.

Analysis for Trend

Trend in the performance data was studied through linear regressions of the variables against time. The regression model employed is of the following form:

$$x_i = \alpha + \beta i + \epsilon_i \tag{5.1}$$

where x_i is the value of the performance variable at time i, i is the time of the observation on the variable, α is the intercept of the regression model, β is the slope of the regression model, and ϵ represents the random departure from the regression model at time i. Regression curves were fitted by the method of least squares. That is, the regression coefficients α and β are estimated in a way that minimizes the sum of squares of deviations from the regression line given by

$$S = \sum_{i-1}^{n} \epsilon_1^2 = \sum_{i-1}^{n} (x_i - \alpha - \beta i)^2 \tag{5.2}$$

where n is the number of observations on x (Draper and Smith, 1966). The least squares estimates of α and β are a and b, respectively.

Table 5.1
Statistical Characteristics of Yearly Series
January 1969 - December 1969

Plant Number	Mean	Std.Dev.	Maximum	Minimum	Range
#1					
Influent BOD (mg/l)	145	59.9	394	41	353
Effluent BOD (mg/l)	12	11	115	3	112
Influent SS (mg/l)	165	96	660	32	628
Effluent SS (mg/l)	15	23	212	4	208
Effluent DO (mg/l)	8.1	0.98	10.5	5.8	4.7
Discharge (MGD)	317	46.5	407	244	163
#2					
Influent BOD (mg/l)	132	43.2	243	41	202
Effluent BOD (mg/l)	19	8.5	99	7	92
Influent SS (mg/l)	179	74	584	59	525
Effluent SS (mg/l)	18	9.2	51	4	47
Effluent DO (mg/l)	3.7	1.7	7.7	0.0	7.7
Discharge (MGD)	178	37.5	295	127	168
#3					
Influent BOD (mg/l)	362	216	1,920	104	1,816
Effluent BOD (mg/l)	15	6.0	46	3	43
Influent SS (mg/l)	826	874	10,322	189	10,133
Effluent SS (mg/l)	21	13	194	5	189
Effluent DO (mg/l)	7.4	1.5	11.0	2.6	8.4
Discharge (MGD)	843	144	1,244	603	641
#4					
Influent BOD (mg/l)	133	55.4	419	25	394
Effluent BOD (mg/l)	11	6.1	40	0	40
Influent SS (mg/l)	98	102	1,072	14	1,058
Effluent SS (mg/l)	10	8.1	80	1	79
Effluent DO (mg/l)	1.6	1.3	6.6	0.2	6.4
Discharge (MGD)	1.90	0.53	4.74	1.12	3.62
#5					
Influent BOD (mg/l)	153	151	1,315	31	1,284
Effluent BOD (mg/l)	16	14	188	2	186
Influent SS (mg/l)	189	383	3,926	19	3,907
Effluent SS (mg/l)	24	19	225	1	224
Effluent DO (mg/l)	3.5	1.5	8.1	0.7	7.4
Discharge (MGD)	0.71	0.20	1.34	0.10	1.24

Tables 5.2 and 5.3 present a summary of results from the regression analyses for the reduced and supplemented data sets, respectively. The results obtained for both data sets are similar. Inspection of the regression coefficients indicates that the influence of trend is small. A comparison of the average regression coefficients of all plants for each variable reveals that the influent variables have positive regression coefficients while the effluent variables have negative coefficients are very much larger than the absolute values of effluent variable coefficients. The null hypothesis that the regression coefficient β is zero can be tested, against the alternate hypothesis that the coefficient is not zero, through the use of the "t" or "F" statistic. In all but 8 of the 30 cases, the null hypothesis is rejected at the 95 percent confidence level. Although the regression coefficients are significantly non-zero, they are still of small magnitude.

Analysis for Cyclical Movement

The presence of cyclical variation in the performance data was determined through periodic regressions of the variables against time. The regression model employed is of the following form:

$$x_i = \bar{x} + \sum_{j=1}^{m} \left(\alpha_j \cos\left(\frac{2\pi i}{n}\right) + \beta_j \sin\left(\frac{2\pi ij}{n}\right)\right) + \epsilon_i \qquad (5.3)$$

where $\bar{x}$ is the mean value of the variable, j is the harmonic number, m is the number of harmonics (9 in this case), i is the time of the observation, n is the fundamental period (365 for a yearly sequence of daily observations), and α_j and β_j are harmonic coefficients.

As in the linear regressions, the periodic regression curves were fitted to the supplemented data set by the method of least squares for a series of nine harmonics with periods of n/1, n/2, ..., n/9 where n is the length of the data record in days (365 in this case). From the periodic regression coefficients it is possible to calculate the percentage of the total variance (V_j) accounted for by the j^{th} harmonic as given by:

Table 5.2

Linear Regression Results for Trend Analysis
(Based on Reduced Data Set)

Variables & Plant No.		Mean	Intercept (a)	Regression Coef. (b)	t Value Calculated[1]	F Value Calculated[2]	Average b for All Plants
Infl. BOD	1	144	108	0.209	6.95	48.3	
	2	132	97	0.199	9.73	94.7	
	3	362	226	0.781	7.25	52.6	
	4	133	126	0.043	1.46	2.12	
	5	153	171	-0.103	-1.28	1.64	0.267
Effl. BOD	1	12	7.3	0.029	5.15	26.5	
	2	19	19	-0.002	-0.48	0.23	
	3	15	16	-0.008	-2.59	6.70	
	4	11	12	-0.008	-2.46	6.03	
	5	16	23	-0.035	-5.07	25.7	-0.003
Infl. SS	1	165	89	0.429	9.69	93.8	
	2	179	148	0.172	4.61	21.3	
	3	826	311	2.90	6.78	46.0	
	4	98	87	0.063	1.18	1.39	
	5	189	308	-0.674	-3.43	11.8	0.578
Effl. SS	1	16	5.7	0.054	4.96	24.6	
	2	18	29	-0.061	4.96	334	
	3	21	20	0.005	0.74	0.55	
	4	10	7.4	0.017	4.38	19.1	
	5	24	31	-0.039	-4.17	17.4	-0.005
Effl. DO	1	8.1	8.4	-0.004	-2.98	8.87	
	2	3.7	2.7	0.011	4.96	24.6	
	3	7.4	8.2	-0.009	-4.64	21.5	
	4	1.6	2.7	-0.012	-7.67	58.8	
	5	3.5	3.7	-0.001	-0.66	0.44	-0.003
Discharge	1	316.7	339.6	-0.126	-5.65	31.9	
	2	177.6	199.1	-0.118	-6.65	44.3	
	3	842.8	850.0	-0.040	-0.55	0.30	
	4	1.90	1.88	0.000	0.33	0.11	
	5	0.71	0.84	-0.001	-7.92	62.8	-0.059

[1] t critical at 5 percent significance is approximately 2.0

[2] F critical at 5 percent significance is approximately 3.8

Reject H_o : $\beta = 0$ against H_1 : $\beta \neq 0$ for $|t \text{ calculated}|$ (or F) > t critical (or F) with probability of Type I error <5 percent.

Table 5.3

Linear Regression Results for Trend Analysis
(Based on Supplemented Data Set)

Variables & Plant No.		Mean	Intercept (a)	Regression Coef. (b)	t Value Calculated[1]	F Value Calculated[2]	Average b for All Plants
Infl. BOD	1	142	107	0.194	6.98	48.7	
	2	131	95.8	0.191	10.1	102	
	3	360	224	0.744	7.50	56.3	
	4	133	125	0.041	1.48	2.18	
	5	152	169	-0.094	-1.28	1.64	0.215
Effl. BOD	1	12.4	7.25	0.028	5.24	27.5	
	2	18.6	18.0	-0.002	-0.491	0.241	
	3	15.0	16.3	-0.007	-2.36	5.58	
	4	10.6	12.0	-0.008	-2.51	6.30	
	5	16.4	22.6	-0.034	-5.17	26.7	-0.005
Infl. SS	1	163	87.4	0.414	9.87	97.3	
	2	177	145	0.174	0.035	24.1	
	3	819	312	2.77	6.86	47.1	
	4	97.4	85.6	0.064	1.29	1.65	
	5	187	304	-0.637	-3.44	11.8	0.557
Effl. SS	1	15.5	5.67	0.054	4.97	24.7	
	2	18.2	29.2	-0.060	-18.4	337	
	3	20.8	19.9	0.005	0.719	0.517	
	4	10.5	7.30	0.017	4.46	19.9	
	5	23.5	30.6	-0.039	-4.18	17.5	-0.006
Discharge	1	316.7	339.7	-0.125	-5.66	32.0	
	2	177.5	199.1	-0.118	-6.69	44.7	
	3	842.8	850.3	-0.040	-0.565	0.320	
	4	1.90	1.88	0.000	0.302	0.091	
	5	0.71	0.84	-0.001	-7.96	63.3	-0.057

[1] t critical at 5 percent significance is approximately 2.0

[2] F critical at 5 percent significance is approximately 3.8

Reject H_o: $\beta = 0$ against H_1: $\beta \neq 0$ for $|$ t calculated $|$ (or F) $>$ t critical (or F) with probability of Type I error <5 percent

$$V_j = \begin{cases} \dfrac{a_j^2 + b_j^2}{2s^2} \times 100\% \;, & j < \dfrac{n}{2} \\[2ex] \dfrac{a_j^2 + b_j^2}{s^2} \times 100\% \;, & j = \dfrac{n}{2} \end{cases}$$

where a_j and b_j are the sample estimates of α_j and β_j, respectively, and s^2 is the sample variance (Thomann, 1970).

The percent of the total variance explained by each harmonic for each variable and plant was calculated. A relatively small percentage of the variance was explained by the harmonics for all variables except discharge. Furthermore, in most cases the first harmonic accounted for the greatest portion of the variance explained. Subsequent harmonics explain very little of the variance encountered, except in the case of discharge where the fourth harmonic as well as the first account for most of the variance. Tables 5.4 to 5.8 present this information for the variables analyzed.

Analysis for Randomness

Since so little trend was detected in the data records and so much of the system variance is unexplained by the harmonic functions fitted to the data, it is of interest to test the raw data records for random behavior without the removal of the nonrandom components detected above. Two different techniques for studying randomness were applied to the reduced data sets for various lengths ranging from one month to a year. The techniques employed were serial correlation and the test for runs above and below the median.

The method of the runs test is relatively simple. The median value of a sequence of observations is determined, and each value in the sequence is labeled "+" if it lies above the median or "-" if it lies below the median. A "run" is defined as an unbroken series of the same

Table 5.4

Variance Accounted for by Harmonics
(as a Percent of the Total Variance)

Influent BOD

Harmonic Number	Plant 1	Plant 2	Plant 3	Plant 4	Plant 5	
1	20.08	24.42	13.73	2.77	9.30	
2	2.39	8.05	13.53	0.22	2.71	
3	3.31	1.86	1.95	5.51	0.45	
4	9.38	4.01	1.62	3.81	1.35	
5	2.69	1.37	0.42	8.91	1.41	
6	0.53	0.10	0.08	0.98	2.51	
7	1.16	0.04	0.14	0.72	1.18	
8	0.26	0.46	0.40	0.84	0.75	
9	0.30	0.12	0.67	0.94	0.14	
Total	48.10	40.42	32.53	24.70	19.79	avg = 33.11

Table 5.5

Variance Accounted for by Harmonics
(as a Percent of the Total Variance)

Effluent BOD

Harmonic Number	Plant 1	Plant 2	Plant 3	Plant 4	Plant 5	
1	10.91	9.11	7.94	25.36	12.35	
2	2.04	7.91	1.65	4.82	5.68	
3	0.26	4.57	9.97	0.44	0.05	
4	0.58	0.69	5.21	3.19	0.82	
5	0.83	0.53	2.97	2.72	1.15	
6	0.02	1.21	0.01	1.25	4.76	
7	0.40	0.20	3.65	1.10	1.83	
8	1.26	0.79	0.65	0.06	0.04	
9	1.10	0.21	1.34	1.29	0.10	
Total	17.41	25.22	33.40	40.22	26.78	avg = 28.61

Table 5.6

Variance Accounted for by Harmonics
(as a Percent of the Total Variance)

Influent SS

Harmonic Number	Plant 1	Plant 2	Plant 3	Plant 4	Plant 5	
1	20.90	11.94	14.76	3.12	7.08	
2	4.38	4.93	8.19	0.99	2.85	
3	2.53	7.94	2.28	0.70	0.96	
4	8.09	0.41	0.44	0.06	1.14	
5	1.71	0.15	0.01	2.81	2.10	
6	0.23	0.30	0.04	0.97	3.06	
7	2.41	1.33	0.13	0.19	1.92	
8	0.42	0.48	0.20	0.06	0.68	
9	0.42	0.22	0.48	0.17	0.18	
Total	41.10	27.69	26.54	9.06	19.96	avg = 24.87

Table 5.7

Variance Accounted for by Harmonics
(as a Percent of the Total Variance)

Effluent SS

Harmonic Number	Plant 1	Plant 2	Plant 3	Plant 4	Plant 5	
1	9.07	39.58	2.27	13.48	7.93	
2	2.26	10.64	0.30	4.82	1.75	
3	0.12	5.86	0.35	0.87	0.43	
4	0.58	8.22	3.87	0.22	1.33	
5	0.13	0.58	4.48	0.24	3.13	
6	0.08	0.94	1.45	0.65	0.53	
7	0.05	0.35	0.73	2.96	2.76	
8	0.83	0.05	0.10	3.83	0.75	
9	0.62	0.89	0.36	3.11	0.28	
Total	13.74	67.10	13.91	30.18	18.67	avg = 28.72

Table 5.8

Variance Accounted for by Harmonics
(as a Percent of the Total Variance)

Discharge

Harmonic Number	Plant 1	Plant 2	Plant 3	Plant 4	Plant 5	
1	22.32	29.26	29.22	4.63	21.25	
2	0.76	0.25	3.16	10.29	5.00	
3	0.12	0.26	0.36	2.85	2.12	
4	14.06	9.82	8.69	12.26	17.12	
5	8.11	3.09	2.97	2.22	2.12	
6	4.39	5.13	3.44	2.01	6.25	
7	2.64	1.71	0.34	4.72	0.25	
8	3.31	0.43	1.64	1.03	1.25	
9	0.12	0.38	0.31	4.00	0.62	
Total	55.84	50.32	50.13	44.02	56.00	avg = 15.22

sign. Concern is with the number of runs rather than just the length of runs. Through a study of run behavior in random process, randomness tests based on runs may be derived (Hoel, 1962). A simple statistic to apply is the total number of runs. Intuitively, too few or too many runs in the sequence would imply a bias. The distribution of this statistic has been determined and may be used in testing data for randomness (Swed and Eisenhart, 1948).

Caution must be exercised in applying the runs test. It is possible not to discover certain types of nonrandomness of a cyclical nature; however, tests based on observations equally spaced in time and sequences of variable lengths are less likely to be deceived. For data that possess cyclical features, a test based on correlation may be more effective than the runs test to discover such features. A test for randomness based on serial correlation is included for a comparison of results with the runs test.

Serial correlation (or autocorrelation) coefficients with lag 1 were calculated for each sequence of each variable-plant combination. The circular definition of the serial correlation coefficient is given by:

$$ {}_{\ell}R_n = \frac{x_1 x_{\ell+1} + x_2 x_{\ell+2} + \ldots + x_n x_{\ell} - \left(\sum_{i=1}^{n} x_i \right)^2 / n}{\sum_{i=1}^{n} x_i^2 - \left(\sum_{i=1}^{n} x_i \right)^2 / n} \qquad (5.5) $$

where ℓ is the lag and n is the sample size (Wald and Wolfowitz, 1945). The distribution of the serial correlation coefficient may be used to test the significance of the sample coefficients (Anderson, 1948).

The null hypothesis that the sequence was random is tested against the alternative hypothesis that the sequence is nonrandom. The null hypothesis was rejected at the 95 percent confidence level in the runs test and at the 98 percent confidence level in the serial correlation test. The results at these confidence levels are quite comparable which indicates that the runs test provided for a somewhat more stringent test for randomness than did serial correlation in these cases.

The results of the runs tests and the serial correlation tests are presented in Tables 5.9 and 5.10, respectively. These tables give the results of tests performed on 1, 2, and 4 month sequences. An inspection of these results indicates a pattern: the longer the sequence, the less likely the sequence is to be random. Additionally, the monthly sequences for influent and effluent quality variables exhibit a greater degree of randomness than does discharge. This observation is reasonable, since high positive fluctuations in flow would experience some damping from flow regulation at the treatment plant. These tests were also applied to yearly records which resulted in a uniform rejection of the null hypothesis of randomness.

While the influence of nonrandom components of performance variability appears to be small, it is demonstrated to be enough of a force to cause a rejection of the hypothesis of purely random variability in long term records. For records of shorter terms, this influence is demonstrated to be less significant. Thus, for long term records the hypothesis of randomness is frequently rejected while for short term records the hypothesis is relatively infrequently rejected. When the data records are frequently rejected as being composed of purely random variation, it is believed that the records still contain a substantial random component. This conclusion is based on an inspection of the magnitude of serial correlation coefficients. When they are significantly non-zero, they are relatively small.

The runs test was also applied to sequences of monthly means for each of the variables. The test failed to reject the null hypothesis of randomness in virtually all cases.

In addition to serial correlation coefficients of lag 1, coefficients with lags 2, 3, ..., 20 were calculated on the supplemented data sets for sequences of a year in length for each variable and each plant studied. Plots of the values of these coefficients against their lag number are known as autocorrelograms. Such plots are useful for studying random behavior. Generally, the lag 2 coefficients are much smaller than lag 1 coefficients. An implication of this may be that composite sampling

Table 5.9

Runs Test Results for Sequences of Variable Length

Variable	Length of Sequence (months)	Total # of Sequences	# of Random Sequ. Plant Number 1	2	3	4	5	Average
Influent BOD	1	12	4	7	9	9	7	7
	2	6	1	2	4	0	2	2
	4	3	0	1	0	0	0	0
Effluent BOD	1	12	10	8	8	8	5	8
	2	6	1	1	3	1	1	2
	4	3	0	0	0	0	0	0
Influent SS	1	12	5	8	9	7	10	8
	2	6	1	3	2	2	2	2
	4	3	0	1	0	1	1	1
Effluent SS	1	12	9	9	6	10	8	8
	2	6	2	4	1	2	2	2
	4	3	1	1	0	0	0	0
Discharge	1	12	5	5	7	4	1	4
	2	6	0	1	0	0	0	0
	4	3	0	1	0	0	0	0

Null hypothesis of randomness requested at 95 percent confidence level.

Table 5.10

Test for Randomness by Serial Correlation for Sequences of Variable Length

Variable	Length of Sequence (months)	Total # of Sequences	# of Random Sequ. Plant Number 1	2	3	4	5	Average
Influent BOD	1	12	7	8	10	7	7	8
	2	6	0	1	4	3	3	2
	4	3	0	1	1	1	1	1
Effluent BOD	1	12	11	10	6	4	8	8
	2	6	3	5	3	1	2	3
	4	3	1	0	0	0	1	0
Influent SS	1	12	8	6	9	11	9	9
	2	6	0	3	3	5	4	3
	4	3	0	1	1	2	2	1
Effluent SS	1	12	10	8	4	10	8	8
	2	6	3	2	1	2	2	2
	4	3	1	0	1	0	2	1
Discharge	1	12	2	3	6	2	0	3
	2	6	0	0	0	0	0	0
	4	3	0	0	0	0	0	0

Null hypothesis of randomness rejected at 98 percent confidence level.

tends to disguise the randomness of the variables and that such tests for randomness should be based on coefficients with larger lag numbers. The autocorrelograms may be used to study cyclical variation of smaller periods. Inspection of the plots did not reveal any obvious cycles (Adams and Gemmell, 1973).

Histograms of the reduced data records were machine plotted and inspected for the general forms of the frequency distribution of the variables. None of the forms of the classical theoretical distributions obviously matched the plots. Generally, the histograms were left-skewed. The reduced data recores were fitted to the uniform, normal, and Poisson distributions and the goodness of the fit was tested by the Kolmogorov-Smirnov one-sample test (Siegel, 1956). The data did not significantly fit any of these theoretical distributions.

At this point it appears appropriate to discuss the application of the above randomness tests to normalized data; that is, data from which the nonrandom components are removed. This approach was not taken for two reasons. Firstly, it was expected that tests on normalized data would fail to reject the null hypothesis of randomness as others have found (Wallace and Zollman, 1971). Secondly, inferential statements based on tests from modified data make it necessary to predict the components removed from the raw data in order to apply such statements to conditions other than those tested. The intention of the study was not at this level. Rather, the intention was to make more generally applicable statements concerning performance variability.

5.3 Performance of Plant Groups

As previously discussed, the dependence or independence of the simultaneous performance of regionally related treatment plants is of interest as an indication of underlying regional factors commonly affecting plant performance. Dependence in performance was measured through the use of correlation techniques. The plants were first taken in pairs for which simple linear and simple rank correlation coefficients were calculated for each variable. Next, all plants were considered together and multiple linear and multiple rank correlation coefficients were calculated for the regional group of plants for each variable.

Simple Linear and Simple Rank Correlation

Generally, correlation is a study of two or more variables simultaneously to see how they are related. For two variables, x and y, n pairs of measurements are taken, such as (x_1,y_1), (x_2,y_2), ..., (x_n,y_n). A graph of these points in the x,y plane is known as a scatter diagram and may reveal a relationship between the two variables. The relationship is independent of the choice of origin and the scale of measurement for x and y, if instead of plotting, x_i, y_i, simple standard units are plotted, i.e., $u_i = (x_i - \bar{x})/s_x$ and $v_i = (y_i - \bar{y})/s_y$, where $\bar{x}$ and $\bar{y}$ are the sample means of x and y, respectively, and s_x and s_y are the sample standard deviations of x and y, respectively.

The correlation coefficient is defined as:

$$r = \sum_{i=1}^{n} \frac{(x_i - \bar{x})(y_i - \bar{y})}{n\, s_x\, s_y} \tag{5.6}$$

There are two properties of r:

(1) $-1 \leq r \leq 1$

(2) $r = \pm 1$ if and only if all points lie on a straight line.

Thus the magnitude of r determines the strength of the relationship while the sign of r relates whether y tends to increase or decrease, with x. It should be noted that r is a useful measure of the strength of the relationship between two variables only when the variables are linearly related.

In problems involving linear correlation, the value of r may be conceived as the first sample value in a sequence of sample values r_1, r_2, ..., that is obtained if repeated sets of similar data are obtained. These sets of data may be thought of as having been obtained by drawing n random samples from some population.

The population being sampled can be described by the frequency function $f(x,y)$ of the two variables x and y. The function $f(x,y)$ contains a parameter ρ whose value measures the extent to which x and y are linearly related in the probability sense. Then r may be used to estimate the value of ρ. The parameter ρ is called theoretical, or population correlation coefficient.

Theoretically, it is possible to derive the frequency function of the random variable r, if x and y are assumed to possess a normal frequency function. The frequency function of r depends only on the parameters ρ and n (Hoel, 1962).

If the normality assumptions are satisfied, it is possible to use the sample value of r to test for independence in the following way. If the variables are independent, the regression curves are horizontal and vertical straight lines, thus implying the parameter ρ is equal to zero. If r is close to zero, there is not sufficient reason to doubt the independence. If r is far from zero, the hypothesis that the two variables are independent is rejected.

In light of the preceding tests on the distribution of performance variables, the above normality assumptions may be questioned, and it is desirable to test for independence by nonparametric or distribution-free methods. Such methods make no assumptions about the underlying distributions of the populations from which samples are drawn. A useful nonparametric measure of correlation is the Kendall rank correlation coefficient.

The rationale of the Kendall rank correlation coefficient, τ, is the following. Consider two sets of plant performance data - two sequences of daily data, one for each plant. The observations from one plant, say x, are ranked in terms of magnitude of observations. The sequence of days is reordered to conform to the ranked sequence of x, and the observations of the other plant, y, are reordered to conform to the reordered sequence of days. Then M is defined for this reordered sequence of y by:

$$M = Nc - Nd \tag{5.7}$$

where Nc = the number of times y_i is followed by a larger y, for all i, and Nd = the number of times y_i is followed by a smaller y, for all i. The coefficient τ is defined by:

$$\tau = M/\tfrac{1}{2}n(n-1), \; -1 \leq \tau \leq +1 \tag{5.8}$$

where n is the number of observations. In the case that two or more observations of either x or y are tied, the tied observations are given the average of the ranks they would have received if there were no ties. The formula for τ then becomes:

$$\tau = M/\left((\tfrac{1}{2}n(n-1) - Tx)^{\frac{1}{2}}(\tfrac{1}{2}n(n-1) - Ty)^{\frac{1}{2}} \right) \tag{5.9}$$

where $Tx = \frac{1}{2}\Sigma t(t-1)$, t = the number of tied observations in each group of ties on x, (5.10)

and $Ty = \frac{1}{2}\Sigma t(t-1)$, t = the number of tied observations in each group of ties on y. (5.11)

The asymptotic distribution of τ under the null hypothesis that τ = is normal with

$$\text{mean} = \mu_\tau = 0$$

and standard deviation $= \sigma_\tau = \left(\dfrac{2(2n+5)}{9n(n-1)}\right)^{\frac{1}{2}}$. (5.12)

Thus,

$$z = \frac{\tau - \mu_\tau}{\sigma_\tau} = \frac{\tau}{\left(\dfrac{2(2n+5)}{9n(n-1)}\right)^{\frac{1}{2}}} \tag{5.13}$$

is approximately normally distributed with zero mean and unit variance (Siegel, 1956). The significance of τ may then be found from the normal distribution. If the null hypothesis of $\tau = 0$ is not rejected, then there is not sufficient reason to doubt the independence of performance of plants. If the alternative hypothesis of $\tau \neq 0$ is not rejected, then it cannot be said reasonably that the plants perform independently of one another.

Both the Pearson product moment and the Kendall rank correlation coefficients were calculated from the reduced data sets for all plant pairs for each variable. The rank correlation coefficients are used for the statistical test of independence while the more familiar produce moment correlation coefficients are included for their use in interpreting the strength of the relationships. Tables 5.11 to 5.16 present correlation coefficient matrices for each of the variables examined. The null hypothesis is that the individual treatment plants perform independently of each other ($\tau = 0$), the alternate hypothesis being dependent performance among plants ($\tau \neq 0$). Included with the values of coefficients are their significance. The significance is a probability figure; it denotes the probability that a process with $\tau = 0$ would produce a coefficient of the cited value. If the significance is smaller than some preselected level, the null hypothesis is rejected. Generally, the significance of coefficients is small, indicating that the correlation is not zero (a rejection of the null hypothesis of independent performance). It is noted that the coefficient values for discharge are much larger than those for all other variables, and the rejection of the null hypothesis is much stronger in this case. Although the coefficient values are significantly non-zero in most cases, the degree of dependence of performance variables is still weak while the degree of dependence of discharge among plants is quite strong.

Multiple Linear and Multiple Rank Correlation

The multiple correlation coefficient is an index of the degree of relationship between one dependent variable and several independent variables, in combination. The coefficient measures the degree to which variations of the dependent variable are related to the combined action of the other factors. The significance of the multiple correlation coefficient is clear if all the independent variables are viewed as constituting a single independent series. It is then seen to be a measure of the relationship between the dependent variable and the independent series, which is what the simple correlation coefficient is in the case of two variables. In the case of the multiple coefficient, the independent series has several component elements; however, this does not alter the funda-

Table 5.11

Correlation Coefficients
(Value of Coefficients & Significance)

Influent BOD

Product Moment

Plant Number	1	2	3	4	
2	.5021 .0000				
3	.2311 .0000	.2073 .0000			
4	.2330 .0000	.1562 .0017	.0005 .4961		
5	.1216 .0114	.0550 .1525	-.0198 .3557	.0317 .2770	avg = 0.1519

Kendall Rank

Plant Number	1	2	3	4	
2	.3930 .0000				
3	.1918 .0000	.2212 .0000			
4	.2408 .0000	.1723 .0000	.0812 .0019		
5	.2576 .0000	.2472 .0000	.1102 .0011	.1657 .0000	avg = 0.2081

Table 5.12

Correlation Coefficients
(Value of Coefficients & Significance)

Effluent BOD

Product Moment

Plant Number	1	2	3	4	
2	.3216 .0000				
3	.1040 .0247	.0915 .0419			
4	.2156 .0000	.2232 .0000	-.2110 .0000		
5	.0475 .1851	.1530 .0019	-.0319 .2735	.2769 .0000	avg = 0.1190

Kendall Rank

Plant Number	1	2	3	4	
2	.2432 .0000				
3	-.1233 .0003	.0472 .0919			
4	.3003 .0000	.1985 .0000	-.1746 .0000		
5	.0641 .0356	.2290 .0000	-.0300 .1993	.1966 .0000	avg = 0.0951

Table 5.13

Correlation Coefficients
(Value of Coefficients & Significance)

Influent SS

Product Moment

Plant Number	1	2	3	4	
2	.4147 .0000				
3	.2300 .0000	.1682 .0007			
4	.0163 .3796	.0592 .1327	-.0061 .4542		
5	.0011 .4915	-.0103 .4231	-.0553 .1490	-.0702 .0951	avg = 0.0748

Kendall Rank

Plant Number	1	2	3	4	
2	.3309 .0000				
3	.2541 .0000	.1779 .0000			
4	.1399 .0000	.0784 .0139	.0872 .0072		
5	.0894 .0060	.0436 .1104	-.0298 .2017	.0199 .2881	avg = 0.1191

Table 5.14

Correslation Coefficients
(Value of Coefficients & Significance)

Effluent SS

Product Moment

Plant Number	1	2	3	4	
2	-.1003 .0280				
3	.4042 .0000	-.0179 .3666			
4	.1522 .0018	-.1880 .0002	.0391 .2288		
5	-.0864 .0499	.2295 .0000	-.0456 .1931	-.0685 .0963	avg = 0.0318

Kendall Rank

Plant Number	1	2	3	4	
2	-.0630 .0000				
3	.0777 .0136	.0594 .0458			
4	.1785 .0000	-.2751 .0000	.0245 .2340		
5	-.0724 .0198	.2784 .0000	.0218 .2676	-.1762 .0000	avg = 0.0054

Table 5.15

Correlation Coefficients
(Value of Coefficients & Significance)

Effluent DO

Product Moment

Plant Number	1	2	3	4	
2	.1193 .0515				
3	.3485 .0000	.0315 .3338			
4	.1402 .0275	-.0658 .1849	.2628 .0001		
5	-.3409 .0000	-.0383 .3008	-.2764 .0001	.0776 .1449	avg = 0.0259

Kendall Rank

Plant Number	1	2	3	4	
2	.0795 .0537				
3	.2488 .0000	.0269 .2929			
4	.0905 .0334	-.0898 .0343	.1876 .0001		
5	-.2440 .0000	-.0391 .2139	-.1673 .0003	.1160 .0094	avg = 0.0209

Table 5.16

Correlation Coefficients
(Value of Coefficients & Significance)

Discharge

Product Moment

Plant Number	1	2	3	4	
2	.8202 .0000				
3	.7846 .0000	.7837 .0000			
4	.5482 .0000	.5357 .0000	.4124 .0000		
5	.6150 .0000	.7091 .0000	.4883 .0000	.4836 .0000	avg = 0.6181

Kendall Rank

Plant Number	1	2	3	4	
2	.6583 .0000				
3	.5990 .0000	.5599 .0000			
4	.3423 .0000	.3231 .0000	.2582 .0000		
5	.4458 .0000	.4909 .0000	.3241 .0000	.3449 .0000	avg = 0.4346

mental nature of the coefficient.

The value of this coefficient depends upon the relationship between the standard error of estimate and the standard deviation of the dependent variable. It is given by:

$$R_{1.23\cdots\cdots} = (1 - s^2_{1.23\cdots\cdots}/s^2_1)^{\frac{1}{2}} \tag{5.14}$$

where $s_{1.23\cdots\cdots}$ is the standard error of estimate and s_1 is the standard deviation of the dependent variable. The subscript to the left of the period relates to the dependent variable, while those to the right relate to the independent variables. It should be noted that no positive or negative sign is attached to R because the correlation could be positive for some of the independent variables, and negative for others.

The value of the multiple correlation coefficient varies not only with the degree of correlation and the size of the sample, but also with the number of independent variables and the choice of the dependent variable.

For a test of the significance of R, the following procedure may be followed. If the true multiple correlation coefficient is zero, then the statistic

$$T = \left(\frac{R^2}{1 - R^2}\right)\left(\frac{n - k}{k - 1}\right) \tag{5.15}$$

where n is the sample size and k-1 is the number of independent variables, has an F distribution with (k-1, n-k) degrees of freedom. Thus, to test $H_o:R^2 = 0$ against $H_1:R^2 > 0$, reject H_o for:

$$T > F_{\alpha;k-1,n-1} \tag{5.16}$$

where α is the significance of the rejection (Anderson, 1958). It is assumed that the observations of the variables are drawn from populations with multivariate normal distributions. Since these normality assumptions may not hold in light of the preceding distribution tests, it is preferable to test the significance of multiple correlation by non-parametric methods.

As the multiple linear correlation coefficient is analogous to the simple linear correlation coefficient, the Kendall coefficient of concordance is analogous to the Kendall rank correlation coefficient. Instead of two sets of rankings as in the two dimensional rank correlation coefficient, there are k sets of rankings of n observations. The Kendall coefficient of concordance is then given by:

$$W = \frac{S}{\frac{1}{12}k^2(n^3-n)} \quad , \quad S = \sum \left(R_j - \frac{\sum_{j=1}^{n} R_j}{n} \right)^2 \tag{5.17}$$

where S is the sum of squares of the observed deviations from the mean of R_j; R_j is the sum of ranks for ranked observation j, j=1, 2, ..., n; k is the number of sets of rankings (number of plants); and n is the number of observations.

In the case of tied observations, the same procedure used for the Kendall rank correlation coefficient is applied here. The correction factor is

$$T = \frac{\Sigma\,(t^3-t)}{12} \tag{5.18}$$

where t is the number of observations in a group tied for a given rank and Σ directs the summation over all groups of ties within any one of the k rankings. With the above correction for ties, the Kendall coefficient of concordance becomes

$$W = \frac{S}{\frac{1}{12}k^2(n^3-n) - k\sum_{T} T} \tag{5.19}$$

where $\sum_{T}$ directs the summation of the values of T for all of the k rankings.

Under the null hypothesis of independence, the asymptotic distribution of the statistic

$$\chi^2 = k(n-1)\, W \tag{5.20}$$

is distributed as chi square with n-1 degrees of freedom. The significance of W may then be tested through the use of this statistic. The null hypothesis that the k sets of ranked observations are independent is rejected if

$$k(n-1)\, W > \chi^2_{\alpha, n-1}$$

where α is the significance of the rejection (Siegel, 1956).

Both the multiple linear correlation coefficient and the Kendall coefficient of concordance were calculated from the reduced data sets of each variable in the analysis. The calculated values of the 2, 3, 4, and 5 dimensional multiple linear correlation coefficients are presented in Tables 5.17, 5.18, 5.19, and 5.20, respectively. The calculated values of the coefficients of concordance are presented in Table 5.21. The conclusions drawn from these results are similar to those drawn from the results of the two dimensional correlation analysis. There is a fairly consistent rejection of the null hypothesis of independence. Although the correlation is non-zero, it is still a weak correlation except in the case of discharge where the correlation is quite strong.

5.4 Summary of Wastewater Treatment Plant Performance

The analyses of individual plant performance revealed several interesting results. The analysis for trend indicated that there was little linear persistence in the data records. The regression coefficients calculated for effluent quality variables were very slightly negative while those calculated for influent quality variables were positive with larger absolute values. Generally, the trend found in records a year in length

Table 5.17

Two-Dimensional Multiple Correlation Coefficients

Dependent Variable	Influent BOD	Effluent BOD	Influent SS	Effluent SS	Effluent DO	Discharge
Plant 1	$R_{1.2} = 0.502$	$R_{1.2} = 0.318$	$R_{1.2} = 0.415$	$R_{1.3} = 0.404$	$R_{1.3} = 0.349$	$R_{1.2} = 0.820$
Plant 2	$R_{2.1} = 0.502$	$R_{2.1} = 0.318$	$R_{2.1} = 0.415$	$R_{2.5} = 0.230$	$R_{2.1} = 0.119$	$R_{2.1} = 0.820$
Plant 3	$R_{3.1} = 0.231$	$R_{3.4} = 0.213$	$R_{3.1} = 0.230$	$R_{3.1} = 0.404$	$R_{3.1} = 0.349$	$R_{3.1} = 0.785$
Plant 4	$R_{4.1} = 0.233$	$R_{4.5} = 0.278$	$R_{4.5} = 0.070$	$R_{4.2} = 0.188$	$R_{4.3} = 0.263$	$R_{4.1} = 0.548$
Plant 5	$R_{5.1} = 0.122$	$R_{5.4} = 0.278$	$R_{5.4} = 0.070$	$R_{5.2} = 0.230$	$R_{5.1} = 0.341$	$R_{5.2} = 0.709$
Degrees of Freedom (n-1)	347	355	353	361	185	363
$R_{5\%}$ significance	0.105	0.104	0.105	0.103	0.144	0.103

Reject null hypothesis that $R = 0$ for $R_{tabulated} > R_{critical}$ at 95 percent confidence of rejection.

Table 5.18

Three-Dimensional Multiple Correlation Coefficients

Dependent Variable	Influent BOD	Effluent BOD	Influent SS	Effluent SS	Effluent DO	Discharge
Plant 1	$R_{1.24} = 0.526$	$R_{1.24} = 0.352$	$R_{1.23} = 0.445$	$R_{1.34} = 0.427$	$R_{1.35} = 0.432$	$R_{1.23} = 0.851$
Plant 2	$R_{2.13} = 0.511$	$R_{2.14} = 0.352$	$R_{2.13} = 0.421$	$R_{2.54} = 0.287$	$R_{2.14} = 0.146$	$R_{2.15} = 0.860$
Plant 3	$R_{3.12} = 0.254$	$R_{3.41} = 0.262$	$R_{3.12} = 0.244$	$R_{3.14} = 0.405$	$R_{3.14} = 0.410$	$R_{3.12} = 0.822$
Plant 4	$R_{4.13} = 0.239$	$R_{4.51} = 0.345$	$R_{4.52} = 0.091$	$R_{4.21} = 0.231$	$R_{4.35} = 0.306$	$R_{4.15} = 0.579$
Plant 5	$R_{5.13} = 0.131$	$R_{5.42} = 0.293$	$R_{5.43} = 0.238$	$R_{5.21} = 0.238$	$R_{5.13} = 0.380$	$R_{5.24} = 0.720$
Degrees of Freedom (n-2)	346	354	352	360	184	362
$R_{5\%}$ significance	0.131	0.130	0.130	0.129	0.180	0.129

Reject null hypothesis that R = 0 for $R_{tabulated} > R_{critical}$ at 95 percent confidence of rejection.

Table 5.19

Four-Dimensional Multiple Correlation Coefficients

Dependent Variable	Influent BOD	Effluent BOD	Influent SS	Effluent SS	Effluent DO	Discharge
Plant 1	$R_{1.243}=0.543$	$R_{1.243}=0.370$	$R_{1.235}=0.446$	$R_{1.342}=0.432$	$R_{1.352}=0.443$	$R_{1.234}=0.862$
Plant 2	$R_{2.134}=0.513$	$R_{2.143}=0.366$	$R_{2.134}=0.425$	$R_{2.541}=0.293$	$R_{2.145}=0.146$	$R_{2.153}=0.889$
Plant 3	$R_{3.124}=0.261$	$R_{3.412}=0.283$	$R_{3.125}=0.250$	$R_{3.142}=0.405$	$R_{3.145}=0.456$	$R_{3.125}=0.830$
Plant 4	$R_{4.132}=0.245$	$R_{4.513}=0.413$	$R_{4.523}=0.094$	$R_{4.213}=0.232$	$R_{4.351}=0.321$	$R_{4.152}=0.584$
Plant 5	$R_{5.132}=0.131$	$R_{5.421}=0.296$	$R_{5.431}=0.091$	$R_{5.214}=0.233$	$R_{5.134}=0.416$	$R_{5.243}=0.728$
Degrees of Freedom (n-3)	345	353	351	359	183	361
$R_{5\%\ significance}$	0.150	0.148	0.149	0.147	0.205	0.147

Reject null hypothesis that R = 0 for $R_{tabulated} > R_{critical}$ at 95 percent confidence of rejection.

Table 5.20

Five-Dimensional Multiple Correlation Coefficients

Dependent Variable	Influent BOD	Effluent BOD	Influent SS	Effluent SS	Effluent DO	Discharge
Plant 1	$R_{1.2435}=0.551$	$R_{1.2435}=0.372$	$R_{1.2354}=0.446$	$R_{1.3425}=0.434$	$R_{1.3524}=0.454$	$R_{1.2345}=0.864$
Plant 2	$R_{2.1345}=0.513$	$R_{2.1435}=0.378$	$R_{2.1345}=0.425$	$R_{2.5413}=0.294$	$R_{2.1453}=0.147$	$R_{2.1534}=0.890$
Plant 3	$R_{3.1245}=0.265$	$R_{3.4125}=0.283$	$R_{3.1254}=0.250$	$R_{3.1425}=0.406$	$R_{3.1452}=0.456$	$R_{3.1254}=0.832$
Plant 4	$R_{4.1325}=0.245$	$R_{4.5132}=0.435$	$R_{4.5231}=0.094$	$R_{4.2135}=0.232$	$R_{4.3512}=0.331$	$R_{4.1523}=0.588$
Plant 5	$R_{5.1324}=0.131$	$R_{5.4213}=0.296$	$R_{5.4312}=0.091$	$R_{5.2143}=0.239$	$R_{5.1342}=0.417$	$R_{5.2431}=0.732$
Degrees of Freedom (n-4)	344	352	350	358	182	360
$R_{5\%}$ significance	0.166	0.163	0.164	0.163	0.228	0.162

Reject null hypothesis that R = 0 for $R_{tabulated} > R_{critical}$ at 95 percent confidence of rejection.

Table 5.21

Kendall Coefficients of Concordance

	Influent BOD	Effluent BOD	Influent SS	Effluent SS	Effluent DO	Discharge
Value of Coefficient	0.442	0.307	0.340	0.204	0.226	0.678
Significance Level (probability figures)	0.000	0.000	0.000	0.384	0.112	0.000

was small. It should be borne in mind that a year is a short period for a trend analysis. A more intensive analysis of trend should incorporate longer data records with monthly (or equivalent period) means to reduce both the sample size and the sample variance.

The analysis for cyclical variation indicated that only the first harmonic accounted for a significant portion of the explained variance for influent and effluent quality variables. Discharge showed more prominent first and fourth harmonics. More variance was explained by the harmonic functions in the case of discharge than in the cases of influent and effluent quality variables. Generally, much of the system variability was left unexplained by the harmonic functions fitted to the data records.

Since so much variance was left unexplained by the nonrandom components analyzed, the raw data records were tested for random behavior. The shorter sequences proved to behave more randomly than longer sequences, indicating the influence of nonrandom forces in long term records. The variability of discharge proved to be much less random than did the variability of influent and effluent quality variables. Although tests on yearly sequences rejected the hypothesis of randomness, sequences of monthly means were accepted as being random. There was a frequent rejection of the hypothesis of purely random variation particularly for the longer sequences, but it is believed that the variation is still considerably random in all variables with the exception of discharge. Inspection of autocorrelograms indicated a consistent decrease in the serial correlation from the first to the second lag. A possible interpretation is that daily composite measurements used as observations affect tests for randomness of time series. The autocorrelograms did not reveal any obvious short term cyclical movement.

Tests for goodness of fit of data records to theoretical frequency distributions proved that the data did not significantly fit the uniform, normal or Poisson distributions. Generally, the empirical frequency distributions were left-skewed but otherwise irregular.

The two analyses of the combined performance of plants employed simple and multiple correlation techniques which produced comparable results. Although the performance variables were rejected as being perfectly independent from plant to plant, there was only a weak correlation among influent and effluent quality variables of plants. Discharge exhibited a much stronger correlation among plants. This indicates that the discharge of one plant is dependent to some degree on the discharge of other plants in the region while quality variables are fairly independent from plant to plant in a region.

CHAPTER VI

STOCHASTIC INPUT MODELS

A previous chapter examined the water quality impact due to the degree of wastewater treatment centralization assuming the wastewater and stream water quality and quantity parameters to be constant. Attention is now directed to the water quality impact due to system centralization when these parameters act in a stochastic manner. Before assessing the effects of variable wasteloads and stream conditions, it is necessary to develop stochastic models to generate values for these variables. The purpose of this chapter is to develop such models for the following variables: wastewater flow, wastewater BOD concentration, wastewater DO concentration, wastewater temperature, streamflow, stream BOD concentration, stream DO concentration, stream temperature, deoxygenation rate constant, and the reaeration rate constant.

6.1 Models for Wastewater Flow, BOD Concentration, and DO Concentration

The study of treatment plant performance variability presented in the previous chapter represents a sample of very small and very large plants. To supplement this sample, the data from mid-range sized plants studied by Thomann (1964) were employed. This was the only other study of long term performance variability found in the literature. The coefficients of variation, C_v (= σ/μ, where σ = sample standard deviation and μ = sample mean), were calculated from these data for the parameters of concern and are presented in Table 6.1.

Plots of these coefficients of variation against mean discharge, as indicator of plant size, are presented in Figure 6.1 to 6.4. An inspection of Figure 6.1 indicates that the coefficient of variation of effluent BOD (EBOD) is inversely proportional to plant size but the relationship is not clearly defined by the data. A regression equation of the form

$$C_v = a(\bar{Q})^b \tag{6.1}$$

in which $\bar{Q}$ is the mean plant flow and a and b are constants, was fitted

Table 6.1

Coefficients of Variation of Performance Parameters

Plant	Influent BOD (mg/1)			Effluent BOD (mg/1)			Discharge (MGD)			Removal Ratio		
Number	μ	σ	C_v	μ	σ	C_v	μ	σ	C_v	μ	σ	C_v
Adams												
3	362	216	.60	15	6.0	.40	843	41.5	.055	.948	.030	.047
1	145	59.9	.41	12	11	.90	317	37.5	.12	.913	.043	.090
2	132	43.2	.33	19	8.5	.45	178	144	.81	.845	.076	.031
4	133	55.4	.42	11	6.1	.55	1.90	0.53	.28	.965	.052	.056
5	153	151	.99	16	14	.88	0.71	0.20	.28	.849	.141	.166
Thomann												
1	183	47.6	.26	23	12	.50	2.5	0.53	.21			
2	125	33.8	.27	14	11	.82	6.5	2.08	.32			
3	275	74.3	.27	35	37	1.07	17.5	2.98	.17			
4	453	118	.26	59	47	.79	60.6	11.5	.19			
5	180	45.0	.25				100.7	15.1	.15			
6	260	78.0	.30				102.2	21.5	.21			
7	163	48.9	.30				107.1	28.9	.27			
8	215	36.6	.17	57	13	.22	140.2	19.6	.14			

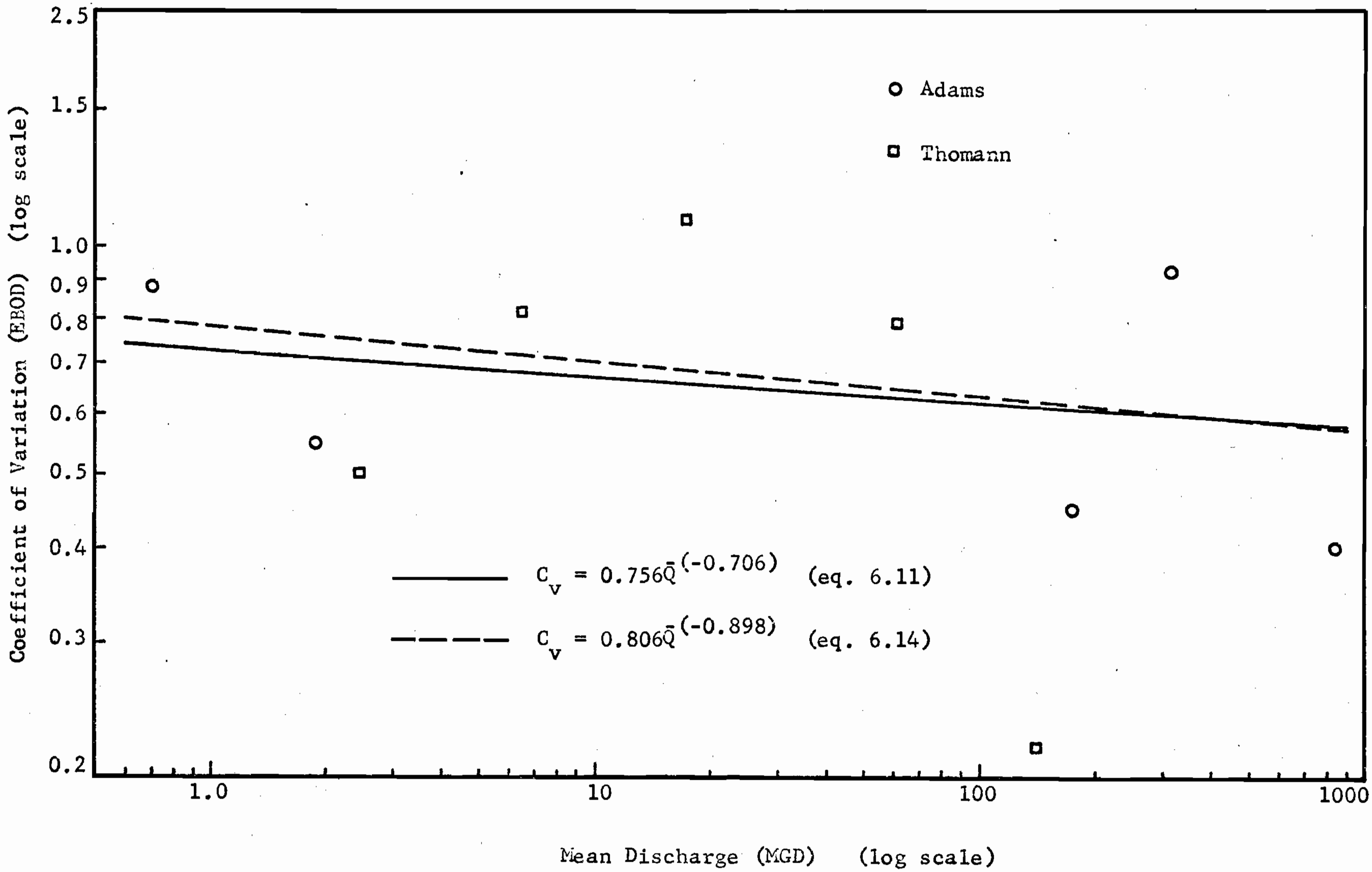


Figure 6.1

Coefficient of Variation - Plant Size Relationships
Effluent BOD

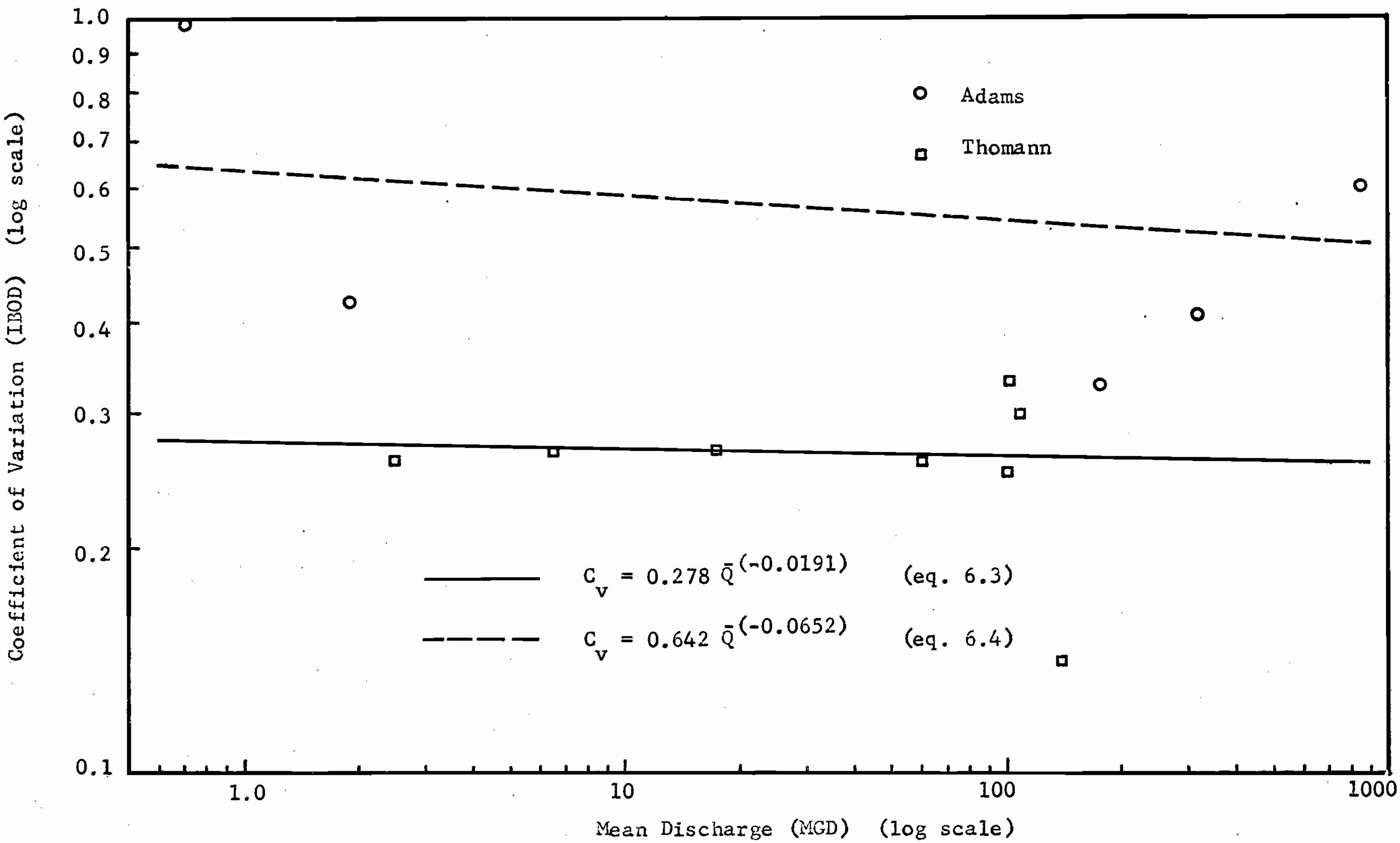


Figure 6.2

Coefficient of Variation - Plant Size Relationships
Influent BOD

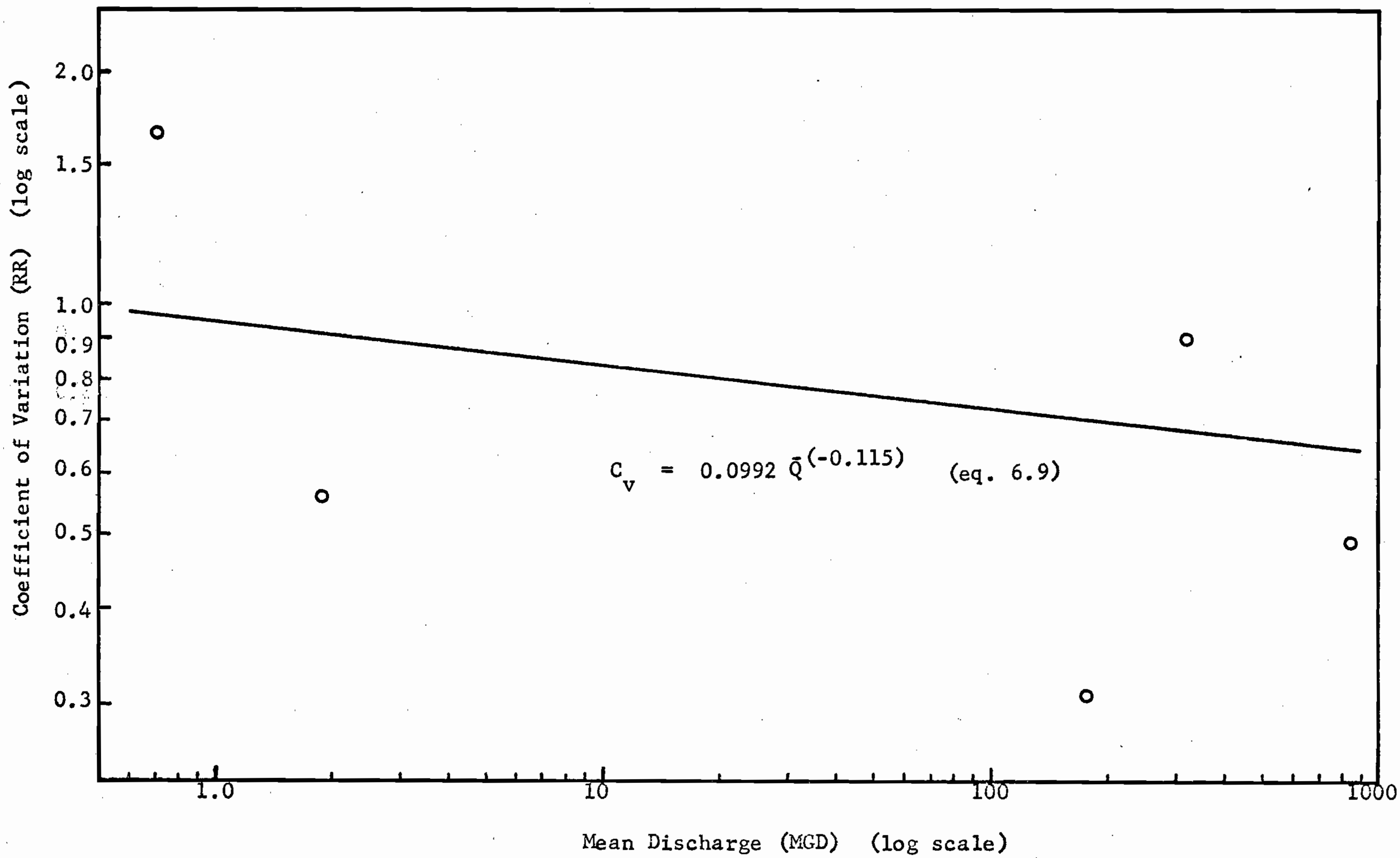


Figure 6.3

Coefficient of Variation - Plant Size Relationships
Removal Ratio

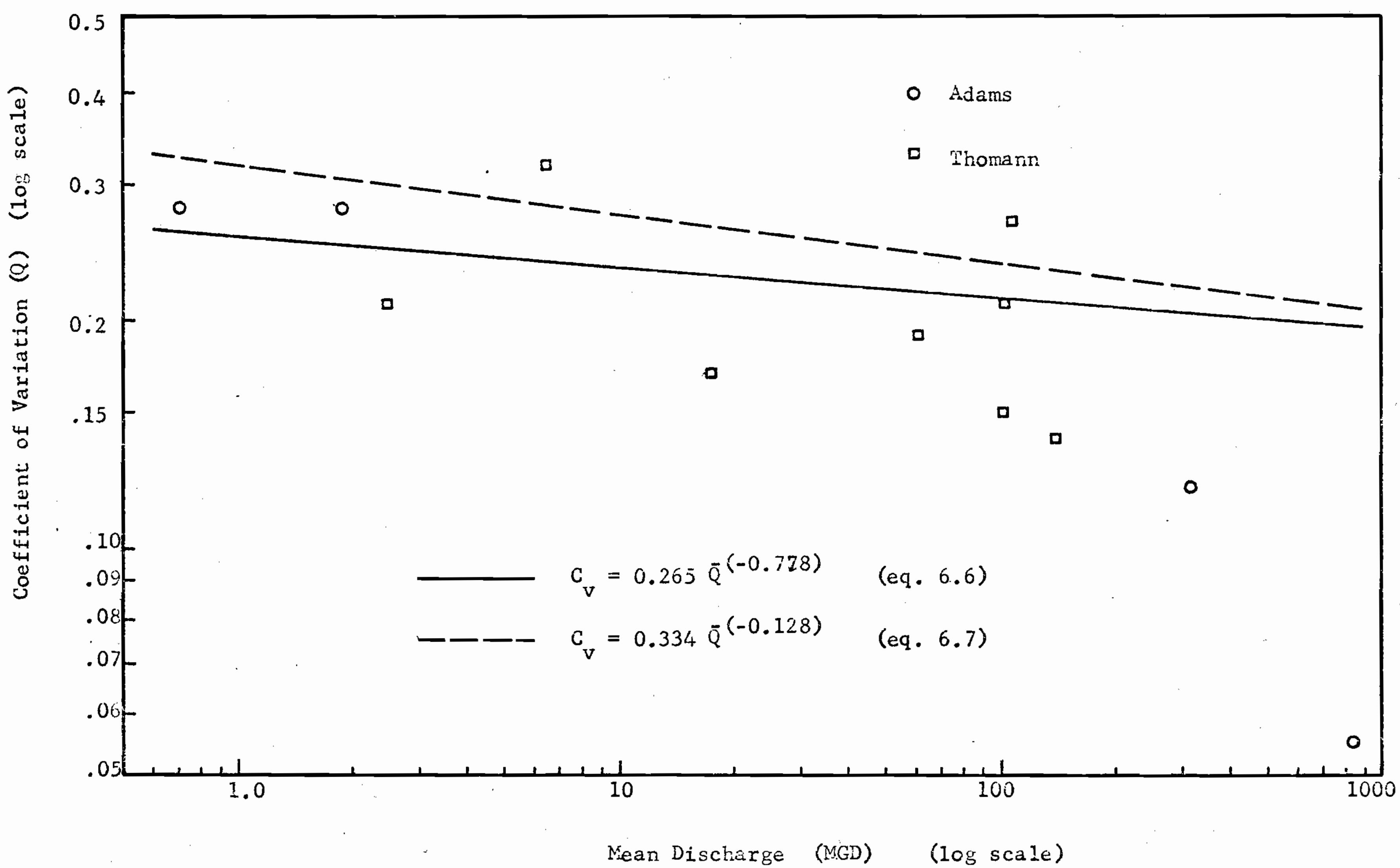


Figure 6.4

Coefficient of Variation - Plant Size Relationships
Discharge

to the data. Such an equation could be used to predict the coefficient of variation of the effluent BOD concentration knowing the mean plant flow. Considering the lack of fit of this regression ($R^2 = 0.124$), it was deemed appropriate to generate values of EBOD by an alternative method, regress this generated data, and compare the resulting regression equation with the above equation.

Thomann's data for influent BOD (IBOD) depicted a fairly clearly defined relationship. Further, the plants from which his data are taken are more representative of plant sizes treated by this study (see Figure 6.4). If the removal ratio, RR(= (IBOD - EBOD/IBOD), is known, the EBOD may be calculated from the relationship

$$\text{EBOD} = \text{IBOD}\ (I - \text{RR}) \tag{6.2}$$

Unfortunately, Thomann did not calculate removal ratios from his data. Figure 6.3 presents this information for the author's data. The coefficients of variation of IBOD and RR were regressed against plant size using functions of the form of equation (6.1). From these regression equations which are presented in Table 6.2, it is possible to compute the variance of IBOD and RR knowing their means and the plant size ($\sigma^2 = (\mu C_v)^2$, $C_v = a(\bar{Q})^b$).

If IBOD and RR are assumed to be normally distributed, EBOD values may be simulated in the following manner:

$$C_{v_{IBOD}} = a_1(\bar{Q})^{b_1} \tag{6.2a}$$

$$C_{v_{RR}} = a_2(\bar{Q})^{b_2} \tag{6.2b}$$

$$\sigma_{IBOD} = \mu_{IBOD} C_{v_{IBOD}} \tag{6.2c}$$

$$\sigma_{RR} = \mu_{RR}\ C_{v_{RR}} \tag{6.2d}$$

$$\text{IBOD} \longrightarrow \text{NID}\ (\mu_{IBOD}, \sigma_{IBOD}) \tag{6.2e}$$

$$RR \longrightarrow NID\ (\mu_{RR}, \sigma_{RR}) \tag{6.2f}$$

$$EBOD = IBOD\ (1 - RR) \tag{6.2}$$

For a series of 14 plant sizes, 200 simulations of EBOD were then executed for each size. The sample mean, variance, and coefficient of variation were calculated for each set of simulations. The coefficient of variation was regressed against plant size using power functions of the form of equation (6.1). These regression equations are presented in Table 6.2. A comparison of the regression equations for the simulated and the actual EBOD data indicates a good agreement; for example, equations (6.11) and (6.14). Thus, equations (6.2) and equations (6.3) and (6.9) with μ_{IBOD} = 200 mg/1 and μ_{RR} = 0.85 were selected as the model for generating values of EBOD (LW).

From the study of the performance of regionally related wastewater treatment plant groups, it was determined that the daily discharges of treatment plants were fairly highly correlated. Since the degree of correlation among plants was found to be moderately to highly correlated but not perfectly correlated, a technique is presented to generate flows with a specified degree of correlation in which an analytic function of the form

$$q_i = f(i) + v_i, \tag{6.18}$$

where q_i is the value of the function at plant i, f(i) is the deterministic response function, and v_i is a random component, is posited. A linear autoregressive relation may be posited (Fiering, 1967) as follows:

$$q_i = \beta_o + \beta_1 q_{i-1} + \beta_2 q_{i-2} + \ldots \beta_m q_{i-m} + \omega_i \tag{6.19}$$

where the β_i are autoregressive coefficients and the ω_i are independent random error terms. Since the objective is to generate flows of one plant knowing only the flow from the next plant upstream, let m = 1, so that

Table 6.2

Size-Variability Regression Relationships

Parameter	Sample Source	Sample Size	R^2	Regression Equation	
IBOD	Thomann	8	.022	$C_v = 0.278\bar{Q}^{(-0.01905)}$	(6.3)
	Adams (arithmetic)	5	.237	$C_v = 0.642\bar{Q}^{(-0.06516)}$	(6.4)
	Adams (geometric)	5	.284	$\sigma_g = 1.667\bar{Q}_g^{(-0.01831)}$	(6.5)
Discharge	Thomann	8	.175	$C_v = 0.265\bar{Q}^{(-0.07783)}$	(6.6)
	Adams (arithmetic)	5	.167	$C_v = 0.334\bar{Q}^{(-0.12842)}$	(6.7)
	Adams (geometric)	5	.887	$\sigma_g = 1.304\bar{Q}_g^{(-0.01622)}$	(6.8)
Removal Ratio $\left(\frac{I-E}{I}\right)$	Adams (arithmetic)	5	.328	$C_v = 0.0992\bar{Q}^{(-0.1148)}$	(6.9)
	Adams (geometric)	5	.465	$\sigma_g = 1.162\bar{Q}_g^{(-0.01622)}$	(6.10)
EBOD	Thomann & Adams	10	.124	$C_v = 0.756\bar{Q}^{(-0.07058)}$	(6.11)
	Adams (arithmetic)	5	.638	$C_v = 0.722\bar{Q}^{(-0.0498)}$	(6.12)
	Adams (geometric)	5	.919	$\sigma_g = 1.852\bar{Q}_g^{(-0.03206)}$	(6.13)
EBOD (simulated)	From Eq. 6.3 & 6.9:				
	μ_I = 200 mg/1, μ_{RR} = 0.85	14	.908	$C_v = 0.806\bar{Q}^{(-0.08984)}$	(6.14)
	μ_I = 200 mg/1, μ_{RR} = 0.90	14	.890	$C_v = 1.321\bar{Q}^{(-0.10636)}$	(6.15)
	From Eq. 6.4 & 6.9:				
	μ_I = 200 mg/1, μ_{RR} = 0.85	14	.919	$C_v = 1.137\bar{Q}^{(-0.09719)}$	(6.16)
	μ_I = 200 mg/1, μ_{RR} = 0.90	14	.886	$C_v = 1.698\bar{Q}^{(-0.11732)}$	(6.17)

$$q_i = \beta_o + \beta_1 q_{i-1} + \omega_i. \tag{6.20}$$

If the error term is neglected, and if q_i and q_{i-1} are derived from a bivariate normal population with equal means (μ) and standard deviations (σ), the regression function of q_i on q_{i-1} is linear and homoscedastic (of constant variance); the conditional expectation is

$$E(q_i/q_{i-1}) = \mu + \rho(q_{i-1} - \mu) \tag{6.21}$$

in which ρ is the correlation coefficient and the variance is

$$V(q_i/q_{i-1}) = \sigma^2(1 - \rho) \tag{6.22}$$

which is independent of q_{i-1}. This suggests a form for estimating the parameters β_o, β_1, and ω_i. Consider:

$$q_i = \mu + \rho(q_{i-1} - \mu) + \epsilon_i\sigma(1 - \rho^2)^{\frac{1}{2}} \tag{6.23}$$

where ϵ_i is a normal random deviate. The parameters are seen to be

$$\beta_o = \mu(1 - \rho) \tag{6.24}$$

$$\beta_1 = \rho \tag{6.25}$$

$$\omega_i = \epsilon_i\sigma(1 - \rho^2)^{\frac{1}{2}} \tag{6.26}$$

Then the first plant flow, q_i, in a series of plant flows in a system is generated by the original method; it is randomly drawn from a population distribution $N(\mu, \mu C_v)$ where $C_v = 0.265\mu^{-0.07783}$ (from equation (6.6)). Subsequent flows in the system are determined by the equation

$$q_i = \mu + (q_{i-1} - \mu) + \epsilon_i\sigma(1 - \rho^2)^{\frac{1}{2}} \tag{6.27}$$

in which μ = (total regional wastewater flow)/(number of plants in system), ρ is determined from performance data (for example, Table 5.21), $\sigma = \mu C_v$, and the ϵ_i are drawn from a population distribution N(0,1). This technique preserves the expected value and variance that were previously established and accommodates the correlation that was revealed by the performance studies.

From the examination of the dissolved oxygen concentrations of effluents from Chicago area treatment plants, the data analyzed were decidedly nonrandom and varied within a small range. It was concluded that the wastewater DO concentration is dependent on the operating characteristics and design of the plant (DO level maintained in the activated sludge aeration tank, retention time in secondary clarifiers, availability of post aeration, etc.) rather than influent variability. This research assumes discharges from identically operated plants in the system, operated by a single regional authority. Thus, the design and operation of the region's plants would be internally congruent. For these reasons, a constant effluent DO is assumed for all plants in the region.

6.2 Model for Wastewater Temperature

In order to establish a method for generating values for wastewater temperature, temperature data were collected from 8 Chicago area treatment plants. A statistical summary of these data is given in Table 6.3 and a representative plot of a yearly series of temperature data is presented in Figure 6.5. A model for this series is posited by the following function

$$Y_t = f(t) + v_t \tag{6.28}$$

in which Y_t is the recorded observation at each of n equally spaced intervals of time t, f(t) is the deterministic response function, and v_t is an error term.

Inspection of Figure 6.5 indicates a strong single periodic movement in the data given by the function

Table 6.3

Statistical Characteristics of Temperature Series (in Degrees Fahrenheit)

Plant Number	Plant Name	Plant Type	Plant Size (MGD)	Air		Influent		Effluent	
				Mean	Std.Dev.	Mean	Std.Dev.	Mean	Std.Dev.
1	Northside	Act. Sldg.	300					59.1	8.7
2	Calumet	Act. Sldg.	200	49.3	19.6	64.5	8.0		
3	W. Southwest	Act. Sldg.	900			70.5	7.2	66.5	9.0
4	Hanover Park	Act. Sldg.	2	46.4	20.1	60.4	5.3		
5	Hazelcrest	Act. Sldg.	.7	49.3	21.1	56.1	6.0		
6	E. Chgo.Hts.	Trk. Fltr.	3.5	49.0	20.4	57.3	5.4	56.7	6.8
7	Lemont	Trk. Fltr.	.6	48.3	22.3	59.7	6.1	57.3	8.7
8	Orlando Park	Trk. Fltr.	.8	56.1	22.1	59.2	6.5	57.8	8.4

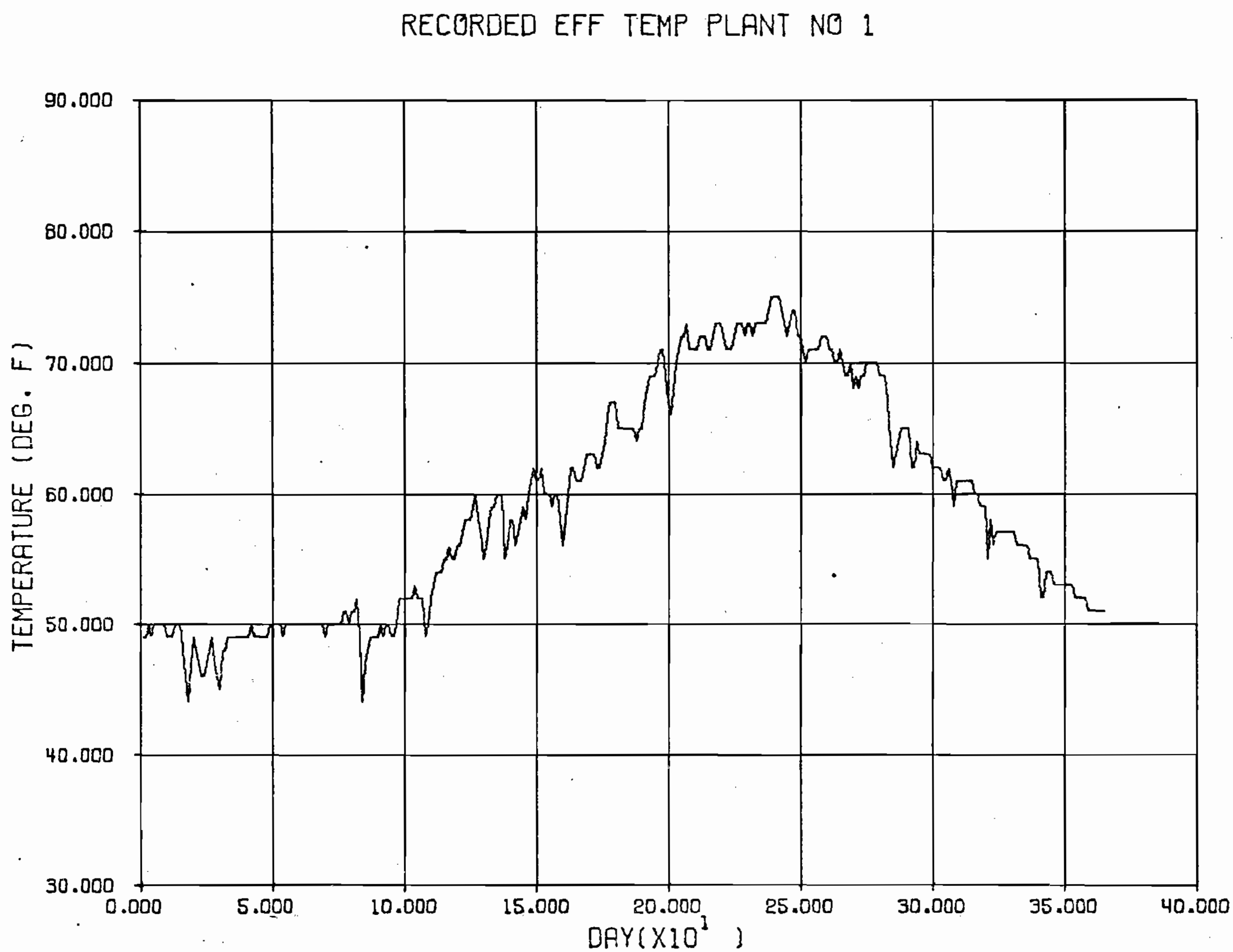


Figure 6.5

Recorded Effluent Temperature - Plant 1

$$f(t) = \alpha_o + \alpha_1 \cos(\omega t - \alpha_2) \qquad (6.29)$$

in which α_o is a constant (mean) value of the response, α_1 is the amplitude of the cyclic response, and α_2 is the phase angle measured from $t = 1$ (in radians), and $\omega = 2\pi/365$ radians. Regression models of the form of equation (6.29) were fitted to these temperature series and the results of these periodic regressions are given in Table 6.4. A typical regression line fitted to the recorded temperature data is presented in Figure 6.6. The variance explained by the regression equations for effluent wastewater temperature is generally over 90 percent of the total sample variance suggesting that equation (6.29) is a very good coice for the deterministic response function.

The Arima (autoregressive, integrated, moving average) family of models is proposed for the stochastic counterpart of the model. An individual model of this family may be of order p, autoregressive, and/or of order 1, moving average, and may operate in the d^{th} difference of the data trace (Box and Jenkins, 1970). Thus, the broad classification of the parametric time series, the p, d, q models, is given by

$$\phi_p (B)(1 - B)^d z_t = \theta_q(B)a_t \qquad (6.30)$$

where $\phi_p(B) = (1 - \phi_1 B - \phi_2 B^2 - \ldots - \phi B^p)$, $\theta_q(B) = (1 - \theta_1 B - \theta_2 B^2 - \ldots - \theta_q B^q)$, and the ϕ's and θ's are constants associated with the order p autoregressive and the order q moving average, respectively, $z_t = f(t) - y_t$ is the t^{th} data trace, and $a_t \longrightarrow$ NID $(0,\sigma^2)$. B is the backwards operator such that $By_t = y_{t-1}$, $B^2 y_t = y_{t-2}$, etc.

The p, d, q classification and its associated parameters must be identified. A plot of the residuals, z_t, given by

$$z_t = y_t - f(t) = y_t - \left[\alpha_o + \alpha_1 \cos (\omega t - \alpha_2) \right] \qquad (6.31)$$

is presented in Figure 6.7. Although this plot appears random, the autocorrelation function (acf) of z_t presented in Figure 6.8 indicates that

Table 6.4

Periodic Regression Results for Temperature Series

Temperature & Plant Number		Mean (α_o)	Variance	A	B	α_1	α_2	Variance Expl.	Variance Expl.(%)
Air	2	49.3	383	-23.5	-8.9	25.1	3.5044	315	82.2
	4	46.4	404	-23.5	-8.8	25.1	3.4981	316	78.3
	5	49.3	445	-25.3	-9.6	27.0	3.5035	366	82.2
	6	49.0	409	-24.8	-7.4	25.9	3.4318	335	81.8
	7	48.3	499	-24.0	-8.0	28.2	3.4304	397	79.6
	8	56.1	490	-26.1	-8.5	27.5	3.4571	378	77.0
Influent	2	64.5	63.8	- 6.62	-8.90	11.1	4.0731	51.5	80.6
	3	70.5	51.3	- 7.08	-5.84	9.18	3.8313	42.1	82.2
	4	60.4	28.4	- 4.22	-5.82	7.19	4.0850	25.8	90.9
	5	56.1	35.9	- 4.94	-6.41	8.09	4.0556	32.7	91.2
	6	57.3	29.1	- 4.21	-5.98	7.32	4.0929	26.8	92.1
	7	59.7	37.3	- 5.81	-5.77	8.19	3.9228	33.5	89.8
	8	59.2	42.0	- 5.26	-6.87	8.65	4.0594	37.4	89.2
Effluent	1	59.1	74.8	- 8.08	-8.75	11.9	3.9670	71.0	94.9
	3	66.5	81.4	- 9.46	-7.94	12.4	3.8401	76.3	93.8
	6	56.7	46.5	- 6.87	-6.03	9.14	3.8623	41.7	89.7
	7	57.3	76.4	- 9.66	-6.73	11.8	3.7503	69.3	90.7
	8	57.8	71.1	- 7.83	-8.18	11.3	3.9492	64.2	90.3

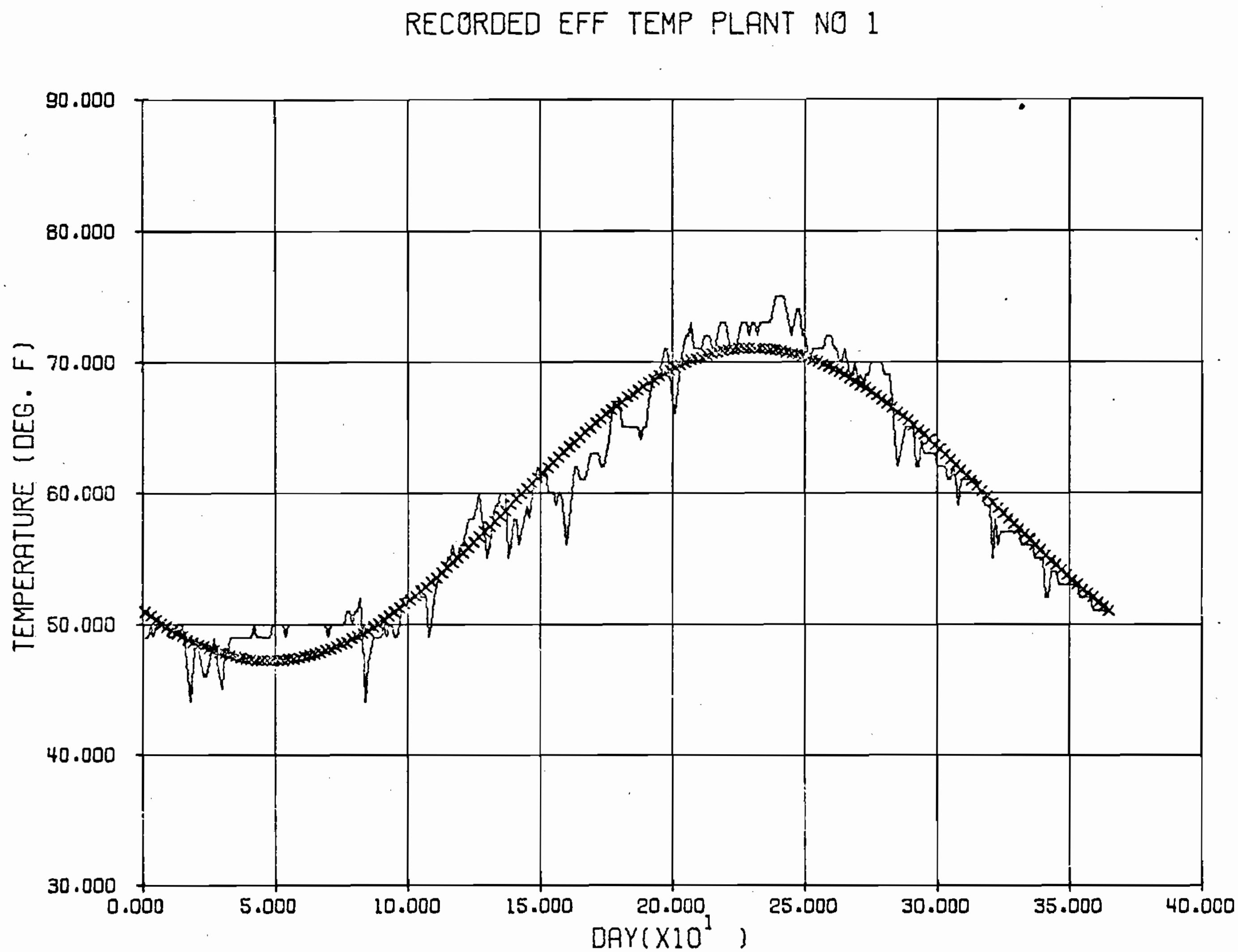


Figure 6.6
Regression Model Fitted to Temperature Series

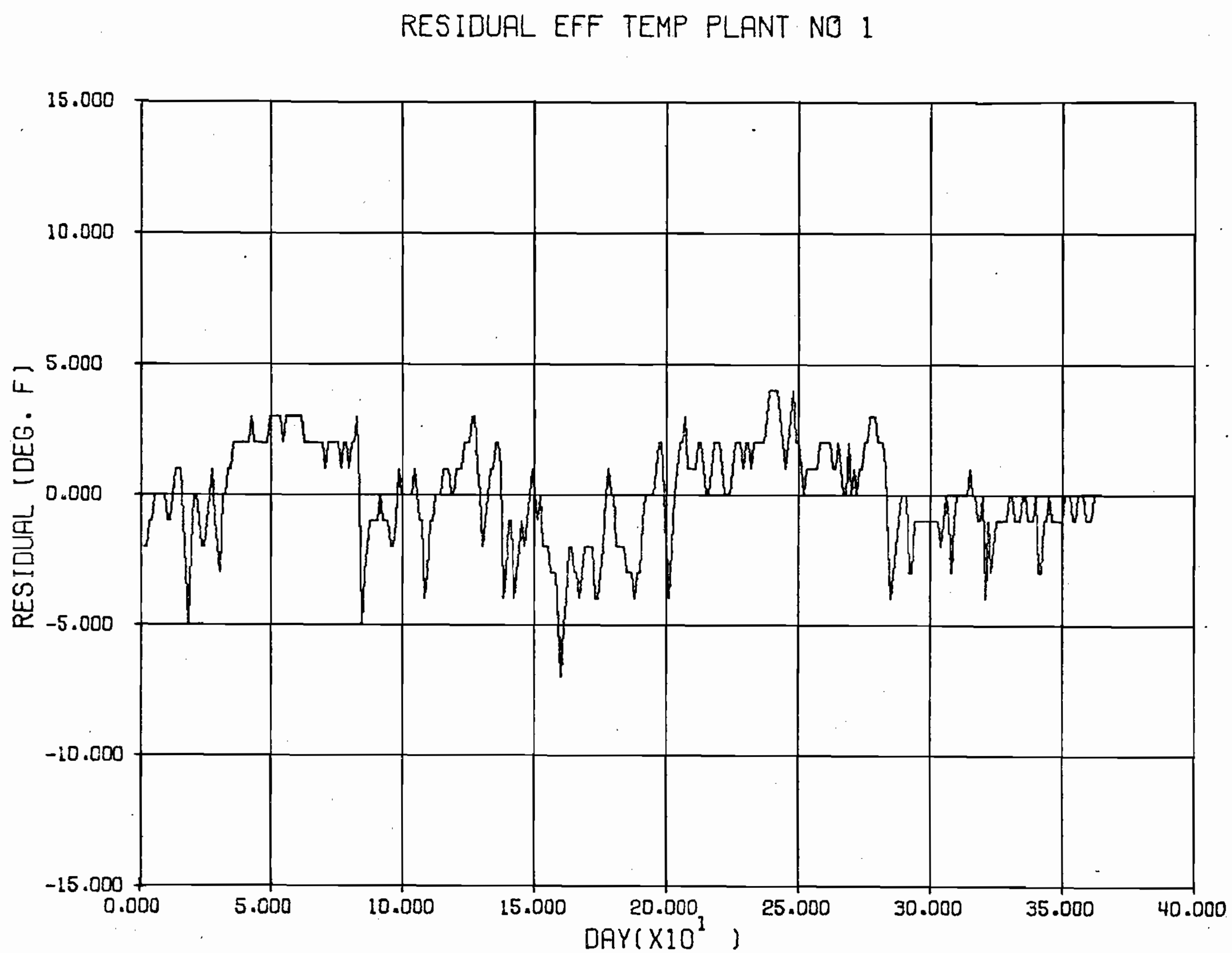


Figure 6.7

Residuals from Periodic Regression Model for Temperature

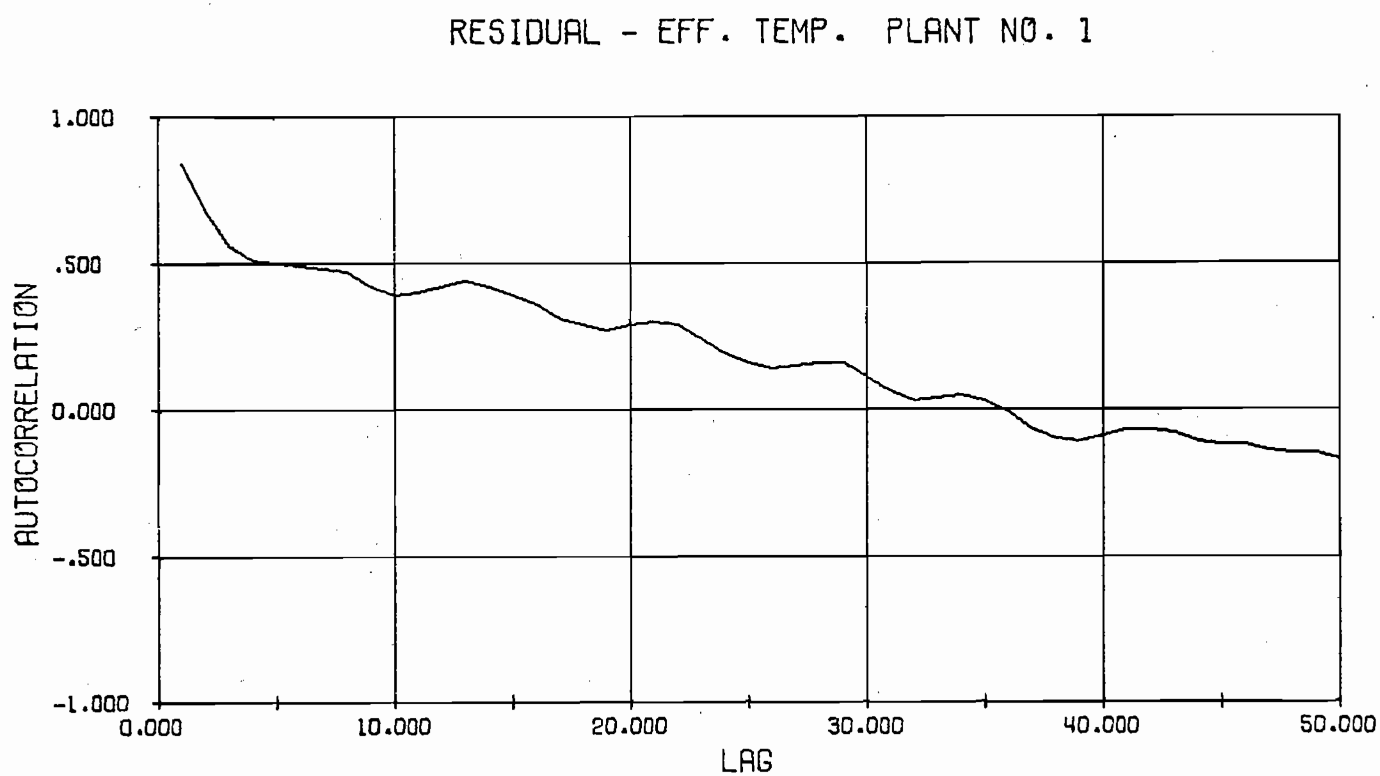


Figure 6.8

Autocorrelation Function for Temperature Residuals the observations on z_t are not independent of each other. Another residual a_t is introduced and is defined by

$$a_t = \frac{(1 - \phi B)}{(1 - \theta B)} z_t \tag{6.32}$$

The acf of a_t is presented in Figure 6.9. Since the autocorrelation coefficients for all lags are close to zero and the acf has no discernable trend or cycle, this model was selected as being appropriate for predicting effluent temperature. This model is autoregressive of order 1 and moving average of order 1 and is given by

$$y_t = f(t) + \left(\frac{1 - \theta B}{1 - \phi B} \right) a_t \tag{6.33}$$

where $$f(t) = \alpha_o + \alpha_1 \cos (\omega t + \alpha_2) \tag{6.34}$$

and ϕ and θ are determined from the relationships

$$\rho_1 = \frac{(1 - \theta\phi)(\phi - \theta)}{1 + \theta^2 - 2\phi\theta} \tag{6.35}$$

and

$$\rho_2 = \rho_1 \phi \tag{6.36}$$

in which ρ_1 and ρ_2 are the autocorrelation coefficients for z_t of lags 1 and 2, respectively. Values for ϕ and θ are determined from acf's as in Figure 6.8 and are presented in Table 6.5. The values for α_o(59.1), α_1(11.9), α_2(3.9670), ϕ(0.80), and θ(-0.12) are estimated from the data for Plant 1. Substituting these values into equation (6.33) results in the following effluent temperature model

$$y_t(^oF) = 11.8 + 0.80 y_{t-1} + 11.9 \cos(0.017214t - 3.9670)$$
$$-9.5 \cos(0.017214(t-1) - 3.9670) + a_t + 0.12 a_{t-1} \tag{6.37}$$

in which $a_t \longrightarrow \text{NID}\ (0, 1.069)$.

Table 6.5

ARMA Model Parameters for Temperature Series

Plant Number	Air					Influent					Effluent				
	ρ_1	ρ_2	ϕ	θ	s	ρ_1	ρ_2	ϕ	θ	s	ρ_1	ρ_2	ϕ	θ	s
1											.841	.678	.806	-.120	1.069
2	.671	.317	.472	-.388	5.980	.724	.551	.716	.078	1.077					
3						.707	.425	.601	-.217	2.154	.775	.537	.693	-.211	1.428
4	.550	.230	.509	-.059	7.800	.695	.522	.751	.109	1.205					
5	.633	.302	.477	-.268	6.792	.620	.447	.721	.166	1.385					
6	.594	.227	.382	-.342	7.006	.617	.458	.742	.206	1.177	.635	.320	.504	-.224	1.687
7	.571	.275	.482	-.133	8.243	.640	.486	.759	.206	1.493	.640	.400	.625	-.025	2.055
8	.599	.306	.511	-.138	8.439	.657	.498	.758	.180	1.615	.692	.503	.727	.067	1.917

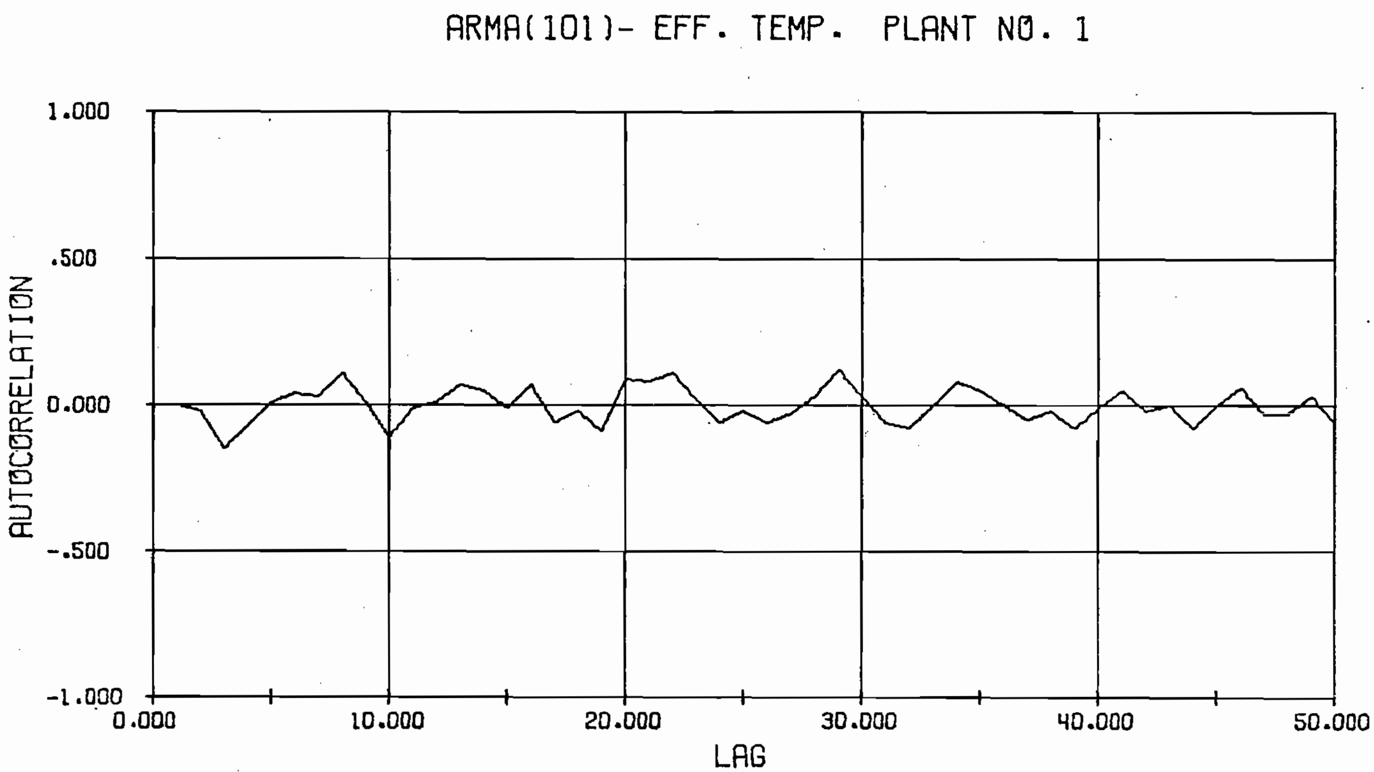


Figure 6.9
Autocorrelation Function for ARMA Model

This model for effluent wastewater temperature given by equation (6.37) was used to simulate a yearly sequence of temperatures. The simulated temperature series is presented in Figure 6.10. A comparison of Figures 6.5 and 6.10 suggest that equation (6.37) is a sufficiently good model for generating wastewater temperatures.

6.3 Models for Stream Flow, BOD and DO Concentration, and Temperature

A review of the literature revealed several attempts to construct predictive models for daily streamflows (Halter and Miller, 1967; Beard, 1967; Quimpo, 1968; Kottegota, 1972; McMichael and Hunter, 1972). Most of these models are not adequate because the serial correlations between previous flows are not incorporated in the generator. It is necessary to preserve annual effects such as spring floods and summer droughts; thus, the model should be locked into a repeating time scale. On the other hand, the model should be constructed in such a way that tomorrow's flow is not generated as a consequence of a thunderstorm that occurred on the same day last year.

On these grounds, the most appropriate model for this study is that of McMichael and Hunter (1972) which was developed and tested with data from the Ohio River. At the onset of the analysis, it is recognized that the variance of the response σ_Q^2 was associated with the mean of flow Q_t; a large variance accompanying a large flow. The variance was proportional to the mean. To provide a fixed variance over the range of flows, the natural logarithm of daily discharge is used ($q_t = \ln Q_t$). Resort was taken to a parametric time series model of the previously described form

$$q_t = f(t) + v_t \tag{6.38}$$

The model chosen for the deterministic response function is

$$f(t) = \alpha_o + \alpha_1 \cos[\,(2\pi t/365) + \alpha_2\,] \tag{6.39}$$

The estimated autocorrelation functions of the residuals (q_t - f(t)) indicated that both autoregressive and moving average terms were required

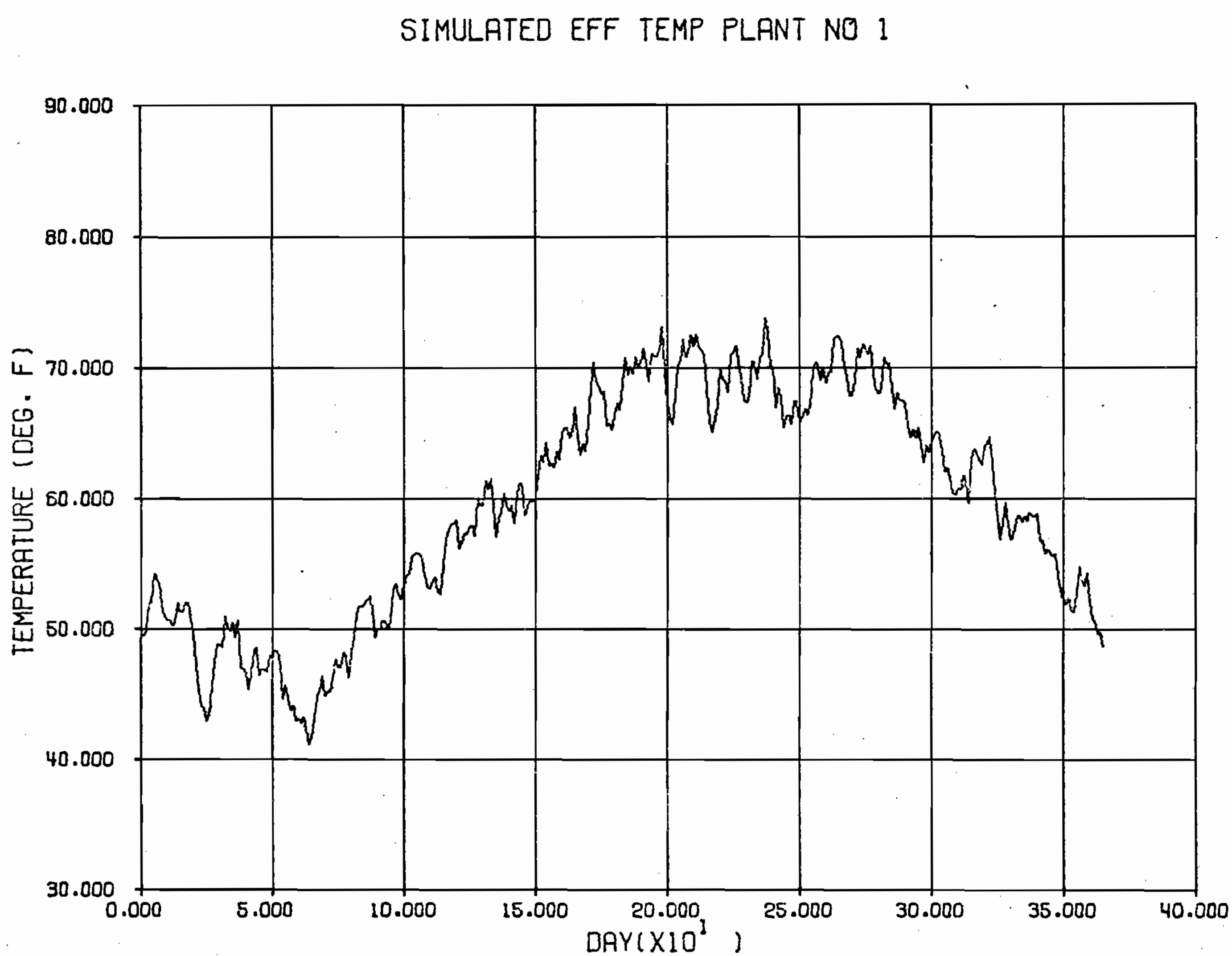


Figure 6.10

Simulated Effluent Temperature

in the stochastic model. The following (p, d, q) - (1, 0, 1) model was employed:

$$q_t = \alpha_o + \alpha_1 \cos\left(\frac{2\pi t}{365} + \alpha_2\right) + \left(\frac{1 - \theta B}{1 - \phi B}\right) a_t \tag{6.40}$$

where $a_t \longrightarrow$ NID $(0, \sigma^2)$. Thus, the forecast function made at time t is given by

$$q_t = \phi q_{t-1} + f(t) - \phi f(t-1) + a_t - \theta a_{t-1} \tag{6.41}$$

or

$$q_t = (1-\phi) \ln M + \phi q_t + A \cos (\omega t - \alpha) - \phi A \cos (\omega(t-1) - \alpha) + a_t - \theta a_{t-1} \tag{6.42}$$

For the purposes of this study, the following parameter values are employed:

$\ln M = 0.5$, $A = 0.5$, $\phi = 0.95$, $\theta = -0.52$, and $a_t \longrightarrow$ NID (0, 0.17).

The model used to generate values for stream temperature was also developed and tested on Ohio River temperature by McMichael and Hunter (1972). The model is (p, d, q) - (1, 0, 0) or of first order autoregressive and is given by

$$y_t(^{o}F) = 5.066 + 0.915\, y_{t-1} + 22.4 \cos (0.017214t + 0.54258) - 20.5 \cos (0.017214(t-1) + 0.54258) + a_t \tag{6.43}$$

where $a_t \longrightarrow$ NID (0, 1.4) and the initial temperature (at t =1) = 40°F.

The objective of the simulation experiment is to determine Dr_{min} variability as a result of input and system variability. The initial stream DO (DRO) is not directly varied in this experiment because it is essentially the same parameter that is being measured. A different experimental time scale would warrant a function for DRO variability on a phenomenological basis. Inasmuch as DRO is a percentage of the saturation DO, it will vary indirectly with TRO.

The same difficulty is reached in assigning a function for LRO variability. Since the assumption of an unpolluted water upstream of the discharge point(s) is made, LRO will vary only within a small range. Further, it has been demonstrated that variations of 100 percent from the nominal value caused a deviation of only 10 percent from the Dr_{min} response. Since this study is not specifically concerned with the upstream BOD distribution, the assumption of its form is arbitrary. The distribution of LRO is assumed to be NID (μ, σ) with μ = 3.0 mg/l, the nominal value, and σ = 1.5 mg/l (C_v = 0.5) with the distribution truncated at 0 and 6 mg/l, providing a maximum deviation of 100 percent from the nominal value.

6.4 Models for the Coefficients of Deoxygenation and Reaeration

The variation of K_1 has been studied by Kothandaraman (1968), for its randomness and distribution of random variation on the basis of 83 observations of K_1 from the Ohio River. Randomness was tested by the Runs Up and Down test and the hypothesis that the observations form a random sample was not rejected. The hypothesis that the sample was drawn from a normal distribution was tested by the Kolmogorov-Smirnov one-sample test and was not rejected at the 95 percent confidence level. The values of the distribution parameters are μ = 0.173 (20^{o}C, base e) and σ = 0.077 with C_v = 0.38. This coefficient of variation is used with the nominal value of K_1 as the mean, and values are generated independently from a normal distribution with these parameter values.

The variations in the geometry and the stream channel characteristics are of interest inasmuch as they vary the reaeration rate constant. Rather than arbitrarily assigning stochastic models to these parameters, a stochastic model for K_2 based on mean values of these parameters is proposed. Kothandaraman devised a technique for generating stochastic values of K_2 based on the mean values of parameters affecting K_2. From a series of regression analyses on reaeration measurements on TVA streams (Churchill, 1962), the following relationship was developed

$$k_{2\,(20^oC,\ \text{base e})} = 5.827\ V^{0.924}\ H^{-1.705} \qquad (6.44)$$

Recognizing that the residual variance was high (36.8 percent of the estimated k_2), a study of the distribution of residual error was undertaken. The hypothesis that the residual error was normally distributed ($N(0, .368\ k_2)$) was not rejected by the Kolmogorov-Smirnov one-sample test. Values for K_2 are generated by estimating K_2 from V and H as described by equation (3.25) and adding this normally distributed random error.

CHAPTER VII
STOCHASTIC ANALYSIS

It has been demonstrated that the distribution of nonvariable wastewater sources over their receiving waters has the effect of significantly improving the receiving water quality when the distance between outfalls is large and the dilution ratio is small. The research objective at this point is to examine the water quality impact of variable wasteloads under variable stream conditions. In simulating the behavior of the system, it must be given a temporal framework. Initially, the system was simulated in the framework of the lowest average 7 consecutive day flow occurring once in ten years, a common flow condition employed by water quality standards. The input models developed in the previous chapter were included in the water quality simulation model one-by-one with the minimum dissolved oxygen frequency response of the receiving waters determined after each set of simulation runs. All input models with the exception of the daily streamflow generator were employed at this point, and experiments were then conducted with stream systems of different lengths and dilution ratios. Subsequently, the daily streamflow model was incorporated to simulate yearly conditions rather than the specific low flow season. These simulations are concerned with the day-to-day variability of the system, and each simulation is that of a steady-state system.

Before discussing the results of these simulations, an explanation is presented for the hypothesis of improved water quality low-frequency response due to disaggregated regional wastewater systems. Consider the following theorem (Meyer, 1965):

Let X be a random variable with expectation $E(X) = \mu$ and variance $V(X) = \sigma^2$

and let $\bar{X}$ be the sample mean of a random sample of size n.

Then (a) $E(\bar{X}) = \mu$ (7.1a)

(b) $V(\bar{X}) = \sigma^2/n$ (7.1b)

(c) for large n, $(\bar{X}-\mu)(\sigma/\sqrt{n}) \sim N(0,1)$ (7.1c)

For the purposes of this discussion, consider a simplistic case where only the effluent BOD concentration is variable. To establish the variance of the system,

let $(\sigma)^2$ = the variance of the plant, and
let $(\sigma^*)^2$ = the variance of the system.

From the regression relation relationships developed in the previous chapter,

$$\sigma_i = \mu_i \, a \left(\frac{Q}{i}\right)^{-b} \tag{7.2}$$

where i is the number of plants in the system and Q is the total wastewater discharged by the system. For i = 1 and i = n, equation (7.2) becomes

$$\sigma_1 = \mu_1 \, a \left(\frac{Q}{i}\right)^{-b} \quad , \quad i = 1 \tag{7.3}$$

and

$$\sigma_n = \mu_n \, a \left(\frac{Q}{n}\right)^{-b} \quad , \quad i = n \tag{7.4}$$

If $\mu_i = \mu_n$, then the division of equations (7.3) and (7.4) results in the solution

$$\sigma_n = \sigma_1 n^b \tag{7.5}$$

and

$$\sigma_n^* = \sigma_n / \sqrt{n} = \sigma_1 n^b / \sqrt{n} \tag{7.6}$$

by the above theorem.

From equation (6.12), b = 0.05 and if n = 32

$$\sigma_{32}^* = \sigma_1 / (32^{(0.50 - 0.05)} = 0.21 \, \sigma_1 = 0.21 \sigma_1^* \tag{7.7}$$

This simple example demonstrates that the variance of the 32 plant system may be considerably less than that of a single plant system.

7.1 Monte Carlo Methods

The simulation of engineering systems often requires the generation of sequences of random numbers according to certain probability distributions. A simple example may involve a discrete probability distribution function in which the sequence is drawn from n different numbers each occurring with equal probability. This sequence may be obtained through successive spins of a roulette wheel or rolls of a die, hence the name Monte Carlo.

Monte Carlo methods comprise that branch of experimental mathematics which is concerned with experiments on random numbers (Hammersley and Handscomb, 1964). Problems handled by Monte Carlo methods are of two types: probabilistic or deterministic depending on whether or not they are directly concerned with the behavior and outcome of random processes. In the case of a probabilistic problem, the simplest Monte Carlo approach is to observe random numbers chosen in such a way that they directly simulate the physical random process of the original problem, and to infer the desired solution from the behavior of these random numbers.

One of the main strengths of theoretical mathematics is its concern with abstraction and generality: one can write symbolic expressions or formal equations which abstract the essence of a problem and reveal its underlying structure. However, this same strength carries with it an inherent weakness: the more general and formal its language, the less is theory ready to provide a numerical solution to a particular application. The idea behind the Monte Carlo approach to deterministic problems is to exploit this strength of theoretical mathematics while avoiding its associated weakness by replacing theory with experiment whenever the former falters (Hammersley and Handscomb, 1964). Monte Carlo methods have been applied to completely deterministic problems such as the inversion of matrices, solution of partial differential equations, location of extrema, and numerical integration. In these cases, the deterministic problem is replaced by a probability model such that the expected value of the answer to the probability model is the solution of the deterministic problem.

A Monte Carlo simulation requires sequences of random numbers which are drawn from a distribution that is generally nonuniform. Methods for directly generating random numbers with a particular distribution are not usually available. However, methods do exist for generating random numbers with a uniform distribution. Almost all methods for generating nonuniform distributions are based on the principle of transforming a uniformly distributed sequence of random numbers into the required sequence.

There are many physical processes that may be considered for generating sequences of uniformly distributed random numbers. However, the input of the sequences to digital computers may be cumbersome. Consequently, mathematical methods for generating "pseudo-random" numbers have been developed to facilitate digital computer simulation. These methods generate repeating sequences of random numbers, hence the name pseudo-random. If care is exercised in the design of the number generator, very long sequences may be generated before such a repetition occurs, so that for practical purposes the sequence can be said to produce random numbers. This characteristic of reproducibility is an asset when it is necessary to repeat operations on random numbers for checking purposes.

Pseudo-random numbers may be defined as a reproducable sequence of random numbers developed with a deterministic process that behaves like a random sequence when subjected to certain standard statistical tests. Methods for generating pseudo-random numbers include the mid-square method, the mid-product method, and the additive, multiplicative, and mixed congruential methods (Knuth, 1969). A random sample with any given distribution may be obtained from a given sequence of uniformly distributed random numbers by employing the inverse probability integral transform. Depending on the distribution type and the difficulty in computing the inverse transform, various numerical analysis techniques may be employed. Marsaglia's technique was used to generate and transform random numbers in this study (Marsaglia, 1964).

7.2 Effect of Parameter Variability

The base physical system was employed initially to assess the effects of variability of the wastewater inputs and stream conditions. This system involves a 64 mile stream length with a 2/1 dilution ratio. Successively but cumulatively, the stochastic generator for each variable was incorporated in the water quality simulation model. For each additional variable, 100 simulation runs were executed. The results of these simulations are summarized in terms of the water quality frequency response functions relating the percent of time any particular water quality level is violated. Initially the exogenous (wastewater input) variables were incorporated in the order: wastewater BOD concentration (LW), wastewater flow (QW), wastewater temperature (TW) and similarly the endogenous (stream) variables were incorporated in the order: stream BOD concentration (LRO), stream temperature (TRO), rate of deoxygenation (K_1), rate of reaeration (K_2). Subsequently, both exogenous and endogenous variables were made part of the water quality simulation model.

The water quality frequency response functions determined by these simulations are displayed in Figure 7.1 as cumulative frequency distribution functions (cdf's) for the 1 and 32 plant systems. The low-frequency responses are summarized in Table 7.1. From Table 7.1 and Figure 7.1, the general observation may be made: the lower the frequency, the greater the difference between the minimum dissolved oxygen levels of the single and multiple plant systems and the greater the number of stochastic variables, the greater this difference.

At a 1 percent frequency of occurrence, with all variables stochastic, the single plant system is anaerobic at or around the point of critical deficit while the 32 plant system produces a minimum DO of 4.6 mg/l. At the 5, 10, and 20 percent of frequencies, the differences between the minimum DO levels produced by the 1 and 32 plant systems are 2.5, 1.7, and 1.1 mg/l respectively. These minimum DO differences can be seen from the probability plots in Figure 7.2.

Frequency plots for the systems of various plant numbers are presented in Figure 7.3. It has been demonstrated by Figure 7.2 that the single plant system produced large low frequency variation while the 32 plant system produced a lower variance water quality response. The low-frequency responses for intermediate plant number systems may be ob-

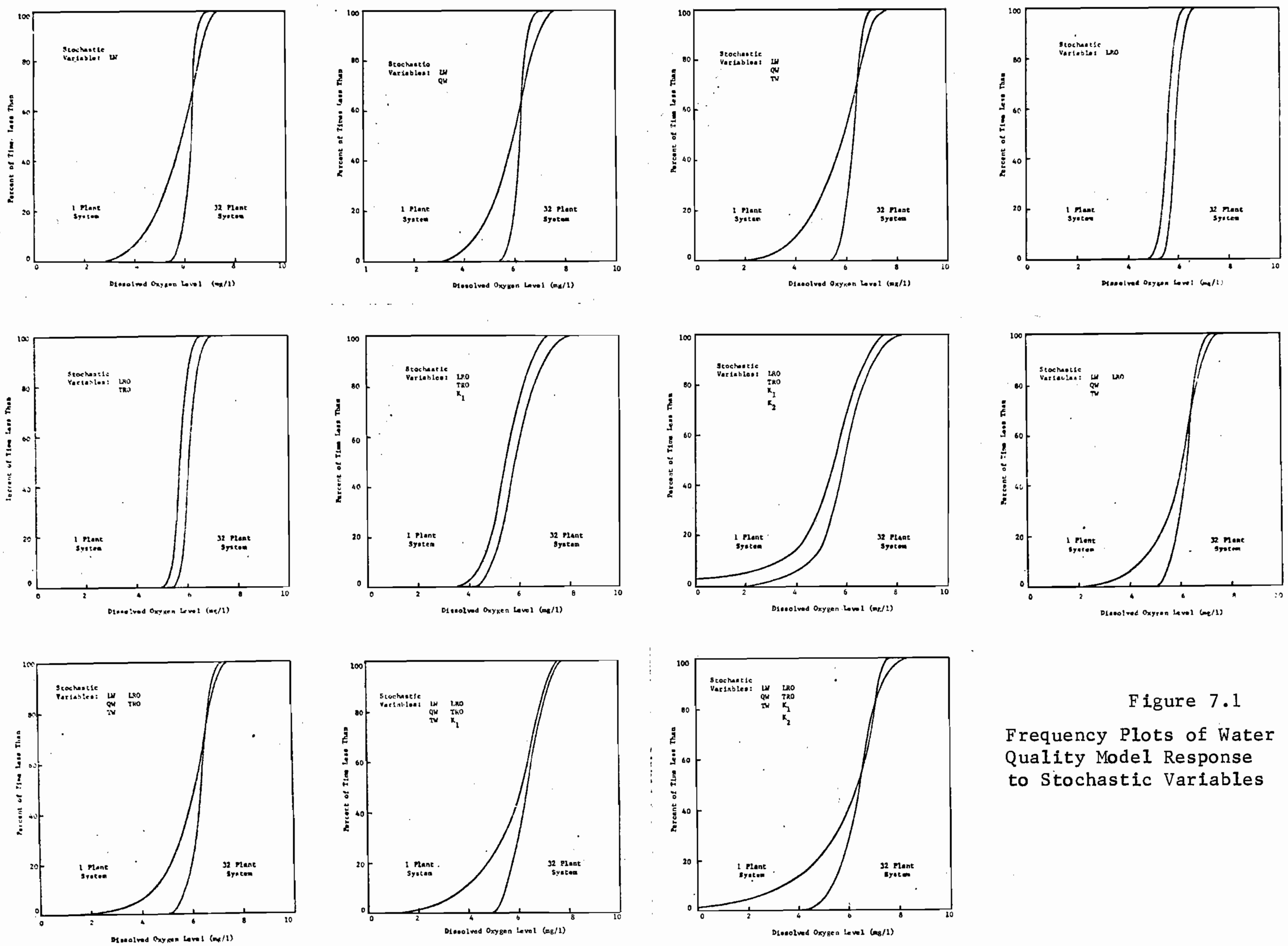


Figure 7.1

Frequency Plots of Water Quality Model Response to Stochastic Variables

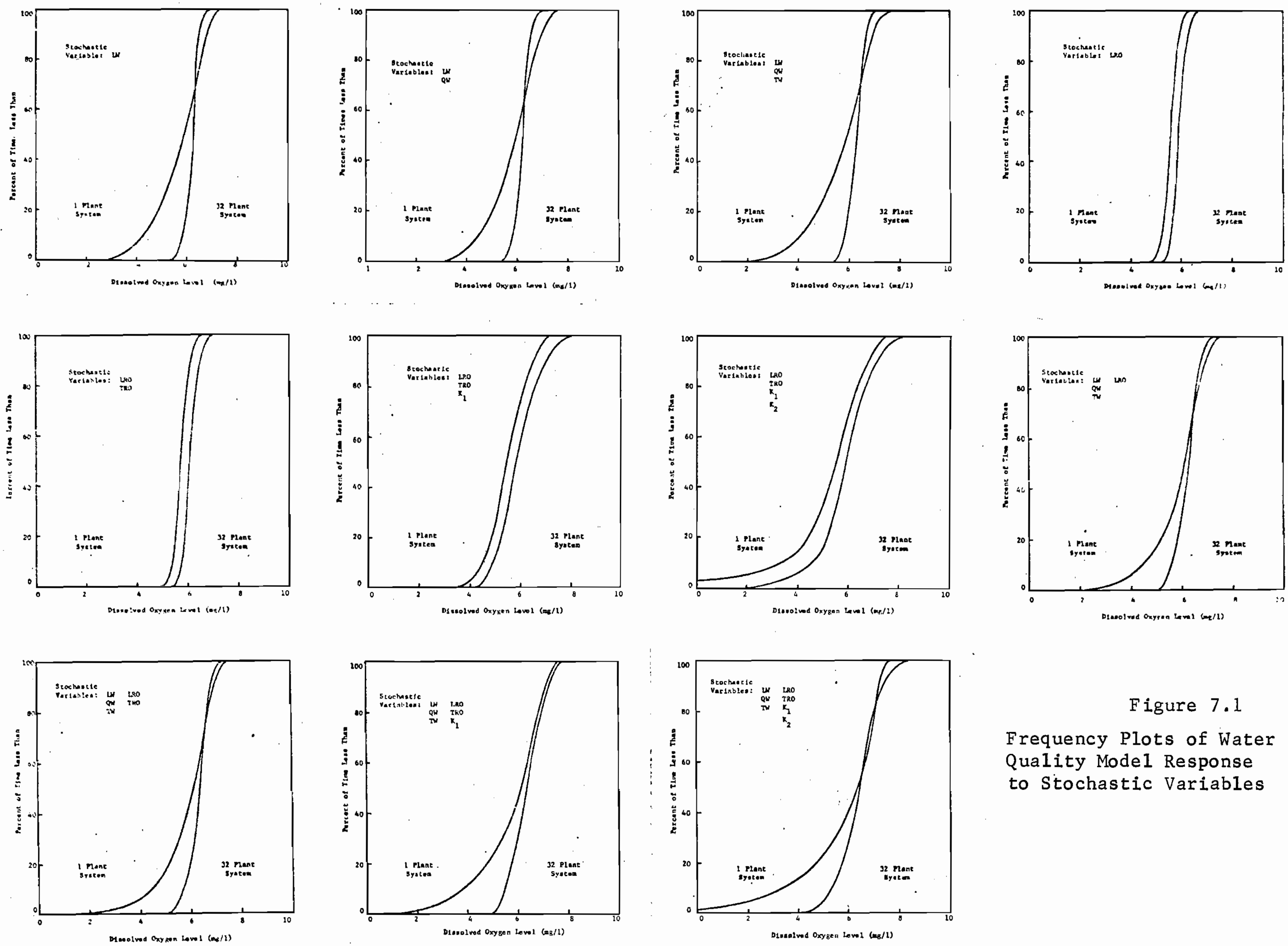


Figure 7.1

Frequency Plots of Water Quality Model Response to Stochastic Variables

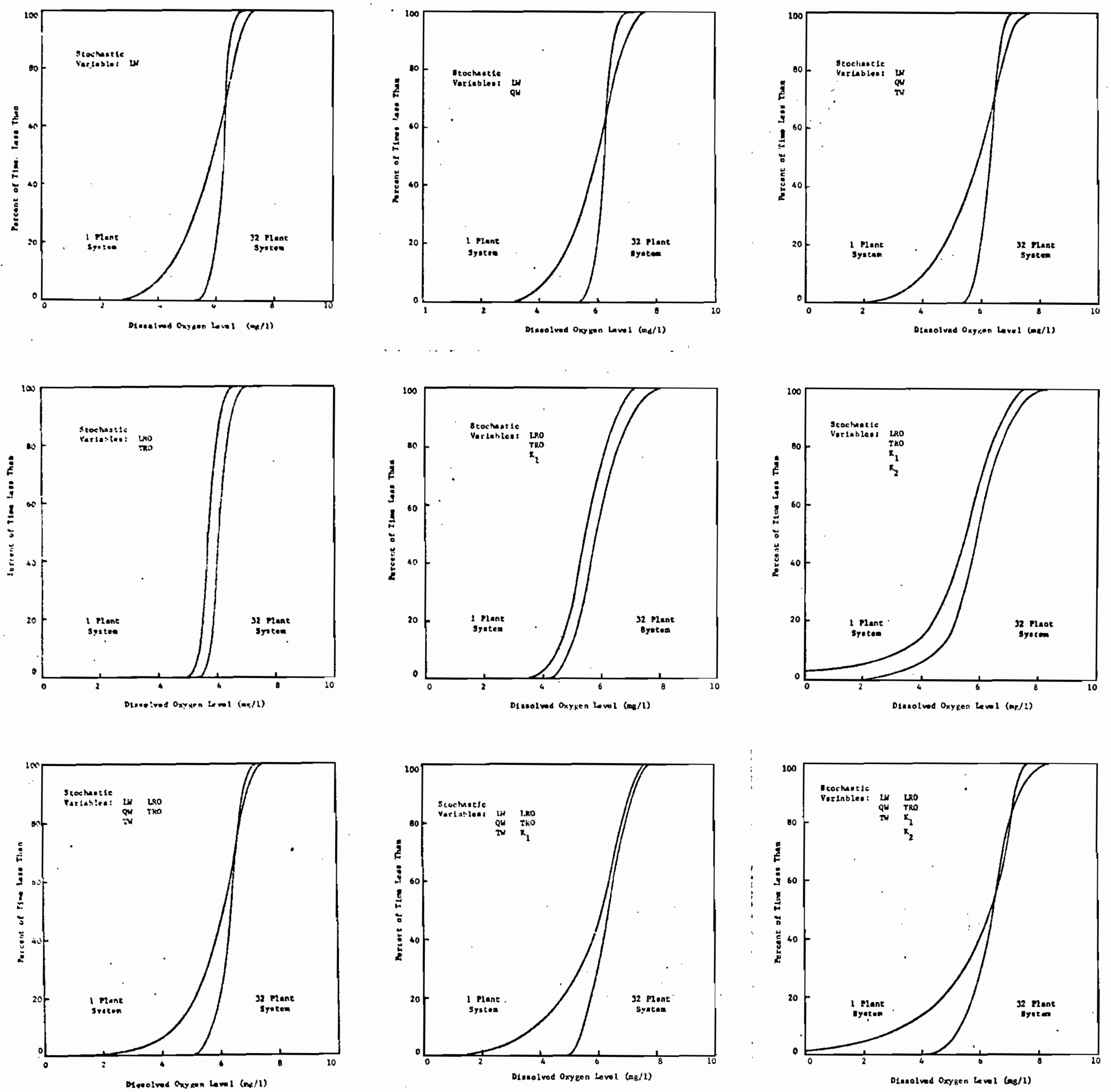


Figure 7.1

Frequency Plots of Water Quality Model Response to Stochastic Variables

Table 7.1

Low Frequency Minimum DO Response of the Water Quality Model with Stochastic Variables

Stochastic Variables	DO Level (in mg/l) Less than Given Frequency							
	1%		5%		10%		20%	
Number of Plants	1	32	1	32	1	32	1	32
LW	3.2	5.6	3.9	5.7	4.3	5.8	4.9	6.0
LW, QW	3.2	5.6	3.9	5.7	4.3	5.8	4.9	6.0
LW, QW, TW	2.4	5.4	3.4	5.7	4.0	5.8	4.7	6.0
LRO	5.0	5.4	5.1	5.5	5.0	5.6	5.4	5.7
LRO, TRO	5.0	5.4	5.2	5.6	5.4	5.8	5.5	5.9
LRO, TRO, K_1	3.8	4.6	4.2	4.7	4.6	4.9	4.9	5.3
LRO, TRO, K_1, K_2	0.0	1.4	1.6	3.8	3.3	4.6	4.4	5.2
LW, QW, TW, LRO	2.6	5.2	3.8	5.4	4.4	5.6	5.2	5.9
LW, QW, TW, LRO, TRO	2.4	5.2	3.8	5.4	4.4	5.6	5.2	5.9
LW, QW, TW, LRO, TRO, K_1	1.6	5.0	2.8	5.3	3.8	5.5	4.8	5.7
LW, QW, TW, LRO, TRO, K_1, K_2	0.0	4.6	2.0	5.0	3.5	5.4	4.7	5.8

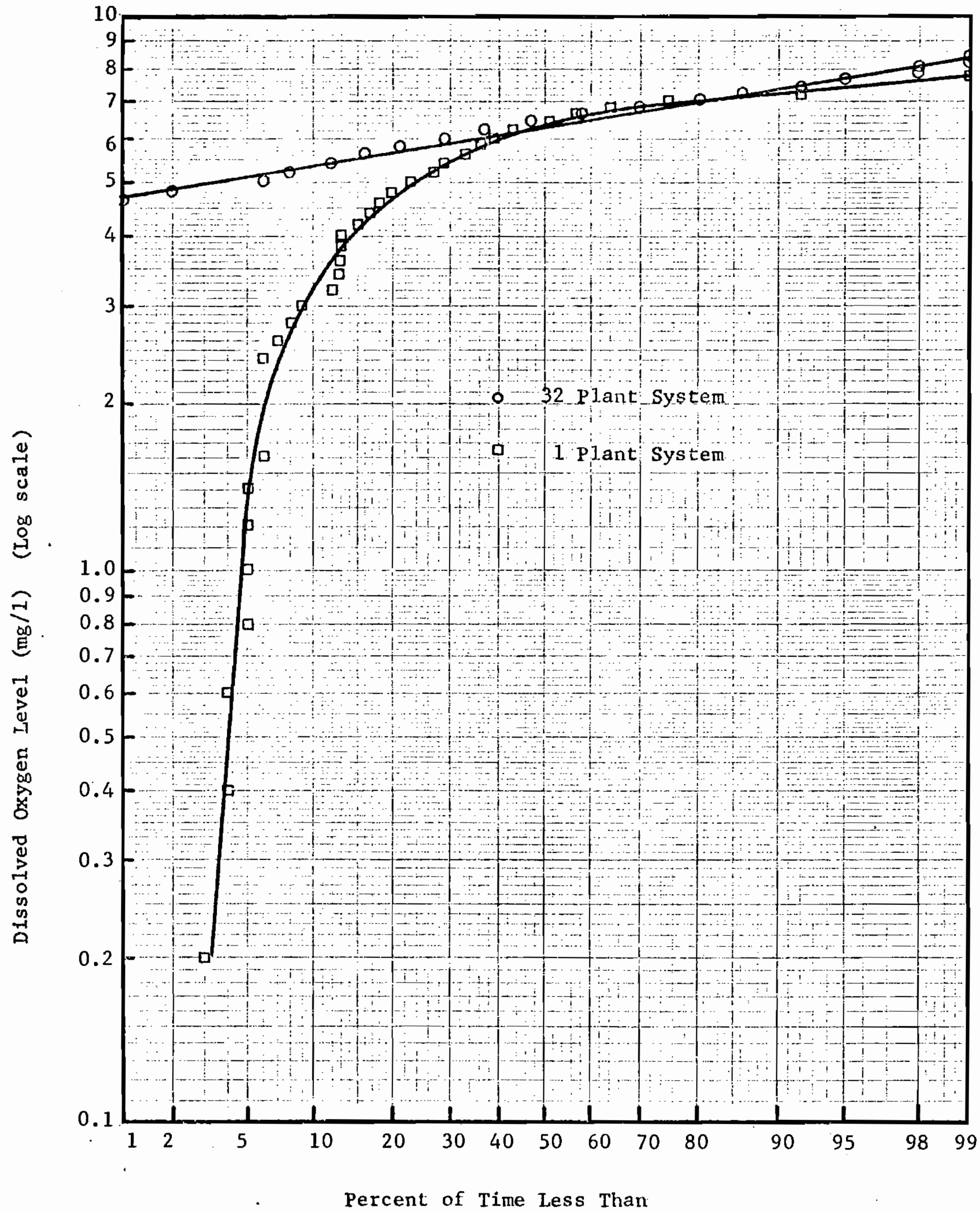


Figure 7.2

Probability Plot of Stochastic Model Response

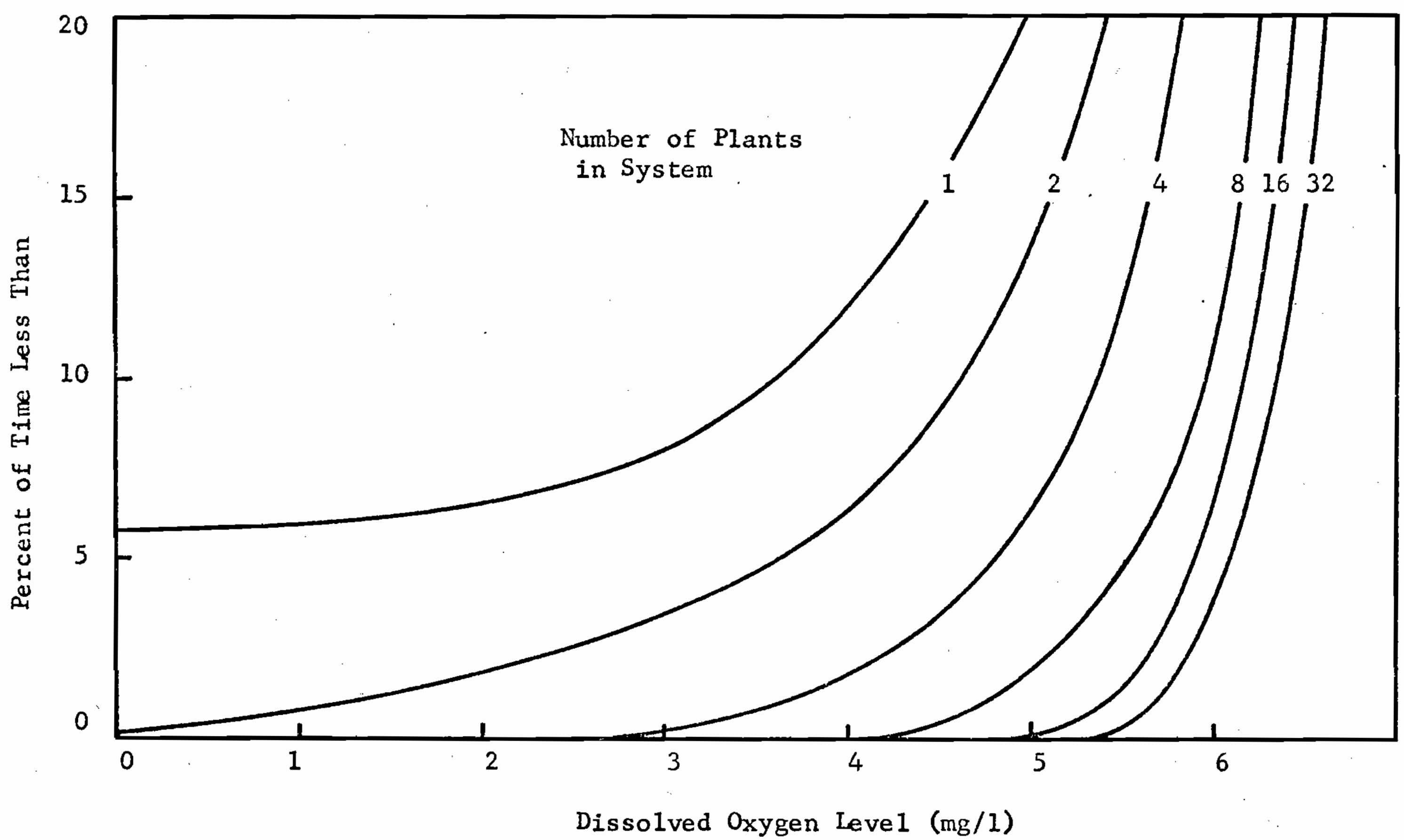


Figure 7.3

Low Frequency Plots for Various Plant Number Systems served from Figure 7.3. The 2 plant system still results in anaerobic conditions at a finite probability level while the 4 plant system is essentially operating at a DO level above 2.5 mg/l. A breakoff is seen in the neighborhood of an 8 plant system where an increase in plant number brings only marginally improved low frequency response.

The question of how increasing the distance between plants affects the water quality frequency response is now addressed. The answer involves a determination of the effect of variability on the system response in conjunction with the effect of stream length. A series of simulations similar to those described previously were conducted on stream systems of varying length. One hundred simulations were executed for each plant number system (1, 2, 4, 8, 16, and 32 plants) of each stream system length (64, 128, 192, 256, 320, and 384 miles). Some statistics from these simulations are summarized by Table 7.2 and the corresponding cumulative distribution functions are presented in Figure 7.4 for the 1 and 32 plant systems. The length of the system does not affect the frequency response of the single plant system; however, with more than one plant in the system, a given DO level is violated increasing less frequently with an increase in distance between plants. The mean response also improves in accordance with the results presented in Chapter IV.

7.3 Effect of Dilution on the Water Quality Frequency Response

The previous experiments on the 64 mile stream system with a 2/1 dilution ratio were repeated for a series of dilution ratios: 1/1, 2/1, 4/1, 10/1, 20/1, 40/1, and 80/1. The dilution ratio is defined as the ratio of the streamflow before wastewater discharge to the wastewater flow. In each case, 100 simulations were executed employing the same models for the stochastic variables previously described (LW, TW, QW, LRO, TRO, K_1 and K_2). The results of these simulations are reported in terms of the water quality frequency responses which are plotted in Figure 7.5 for the various dilution ratios. Some statistics of these simulations are summarized in Table 7.3

From an inspection of Figure 7.5, it is evident that the variance of the system response for any number of plants decreases as the dilution

Table 7.2

Statistical Characteristics of DO Response for Various System Lengths (in mg/l)

Stream Length (mi)	64			128			192			256			320			384		
Number Plants	μ	σ	C_v	μ	σ	C_v	μ	σ	C_v	μ	σ	C_v	μ	σ	C_v	μ	σ	C_v
1	5.73	1.80	.314	5.90	1.36	.231	5.76	1.80	.313	5.30	1.85	.349	5.60	1.73	.309	5.27	1.67	.317
2	5.71	1.70	.298	6.19	1.42	.229	6.28	1.39	.221	6.55	1.34	.205	6.66	1.25	.188	6.68	1.25	.187
4	5.98	1.32	.221	6.49	1.05	.162	6.59	1.05	.159	6.97	0.89	.128	6.88	1.12	.163	7.26	0.89	.123
8	5.73	1.53	.267	6.43	0.99	.154	6.87	0.75	.109	6.93	0.79	.114	7.28	0.69	.095	7.41	0.72	.097
16	5.82	1.35	.232	6.47	0.96	.148	6.85	0.60	.088	7.21	0.55	.076	7.41	0.65	.088	7.67	0.55	.072
32	6.40	0.77	.120	6.73	0.80	.119	7.08	0.65	.092	7.42	0.48	.065	7.52	0.60	.080	7.78	0.42	.054

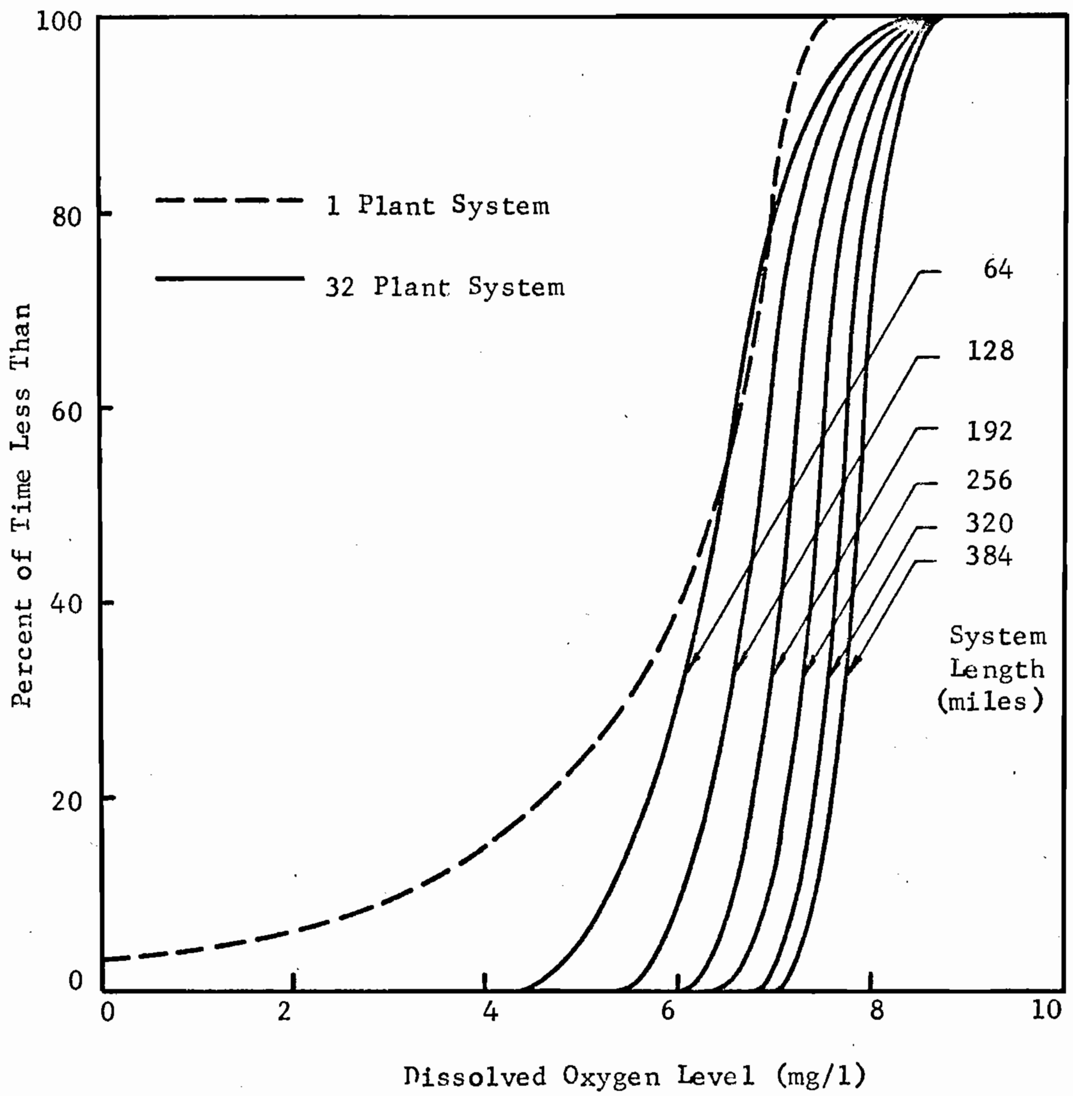


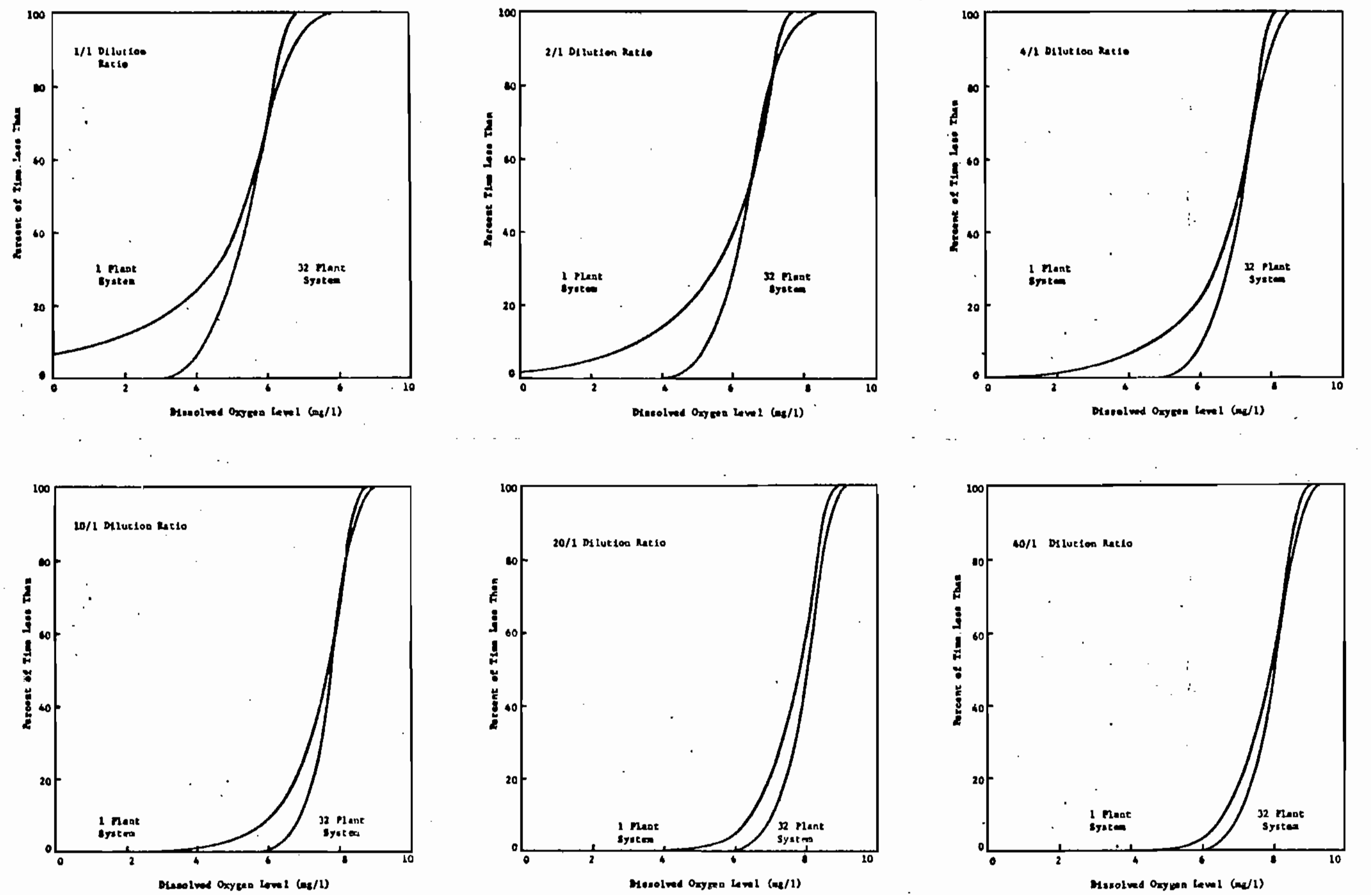
Figure 7.4

Frequency Plot for 32 Plant Systems for Various System Lengths

Table 7.3

Statistical Characteristics of DO Response for Various Dilution Ratios (in mg/l)

Dilution Ratio	1			2			4			10			20			40			80		
Number of Plants	μ	σ	C_v	μ	σ	C_v	μ	σ	C_v	μ	σ	C_v	μ	σ	C_v	μ	σ	C_v	μ	σ	C_v
1	4.77	1.94	.407	5.90	1.36	.231	6.65	1.39	.209	7.12	1.11	.156	7.58	0.99	.131	7.51	0.91	.121	7.71	0.79	.102
2	4.78	1.93	.404	5.97	1.40	.235	6.62	1.16	.175	7.41	0.87	.117	7.73	0.75	.097	7.65	0.79	.103	7.72	0.88	.114
4	5.11	1.51	.295	6.15	1.23	.200	6.73	1.08	.160	7.48	0.81	.108	7.61	0.75	.099	7.74	0.75	.097	7.60	0.83	.109
8	4.84	1.63	.337	5.97	1.20	.201	6.87	0.85	.124	7.28	0.72	.099	7.60	0.85	.112	7.67	0.76	.099	7.59	0.88	.116
16	4.94	1.47	.298	5.97	1.29	.218	6.69	0.80	.120	7.42	0.73	.098	7.72	0.73	.095	7.86	0.72	.092	7.65	0.89	.116
32	5.56	0.92	.165	5.93	1.04	.167	6.86	0.94	.137	7.57	0.74	.098	7.66	0.80	.104	7.76	0.75	.097	7.72	0.86	.111



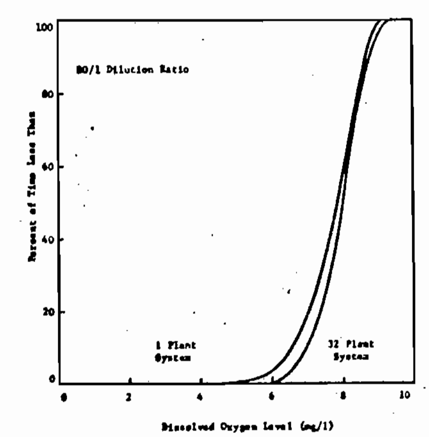


Figure 7.5

Frequency Plot of Stochastic Model Response For Various Dilution Ratios ratio increases and for any dilution ratio decreases as the number of plants increases. Correspondingly, there is an increase in the mean response with an increase in either dilution ratio or plant number or both, in accordance with the results reported in Chapter IV. Because the mean response increases and the variance of the response decreases with an increase in dilution ratio, the coefficient of variation decreases at an even greater rate.

The low frequency water quality responses to the centralization of wastewater discharges are summarized in Table 7.4 for various dilution ratios. It is seen from Table 7.4 that the difference in minimum stream DO response between the 1 and 32 plant systems at a given low frequency level decreases significantly with an increase in dilution ratio. For example, $\Delta\ DO_{min}$ is 3.9 mg/l for a 1/1 dilution ratio and 0.4 at an 80/1 dilution ratio at the 5 percent frequency. This conclusion is evident from an inspection of Figures 7.6 and 7.7 which present plots of DO_{min} and $\Delta\ DO_{min}$, respectively, for dilution ratios at the 1, 5, 10, and 20 percent frequency levels.

In order to allow a comparison of results between the deterministic and the stochastic analyses, correlograms of the minimum stream DO experienced by the deterministic system and the mean and median DO_{min} experienced by the stochastic system are presented in Figure 7.8 and 7.9, respectivly. Correlograms are presented for the 1 and 32 plant systems with 64 mile system lengths, in which the plotting points represent the various dilution ratios.

7.4 Results of Yearly Simulation

The water quality simulations discussed up to this point have been set in the framework of a particular low flow season. In this case, there was no difference between days simulated in the same season of the same year and days simulated in the same season of different years, except in terms of stochastic variability. A simulation situation was established whereby each simulation was a specific day of the year. A yearly simulation treated the sequence of consecutive days of the year. Thus, a relationship did exist between values simulated from one day and the next

TABLE 7.4

Low Frequency Dissolved Oxygen Levels for Various Dilution Ratios and Plant Numbers (in mg/1)

Frequency (percent)	Dilution Ratio 1		2		4		10		20		40		80	
	Number of Plants in System													
	1	32	1	32	1	32	1	32	1	32	1	32	1	32
1	0.0	3.4	0.0	4.6	0.0	5.2	3.6	6.1	4.8	6.2	5.0	6.0	5.2	6.0
5	0.0	3.9	2.0	5.0	3.4	5.7	5.3	6.6	6.0	6.7	6.2	6.6	6.2	6.6
10	1.4	4.2	3.5	5.4	4.6	6.1	6.0	6.9	6.5	7.0	6.6	7.0	6.5	7.0
20	3.6	4.8	4.7	5.8	5.8	6.5	6.7	7.3	7.0	7.4	7.1	7.4	7.0	7.4

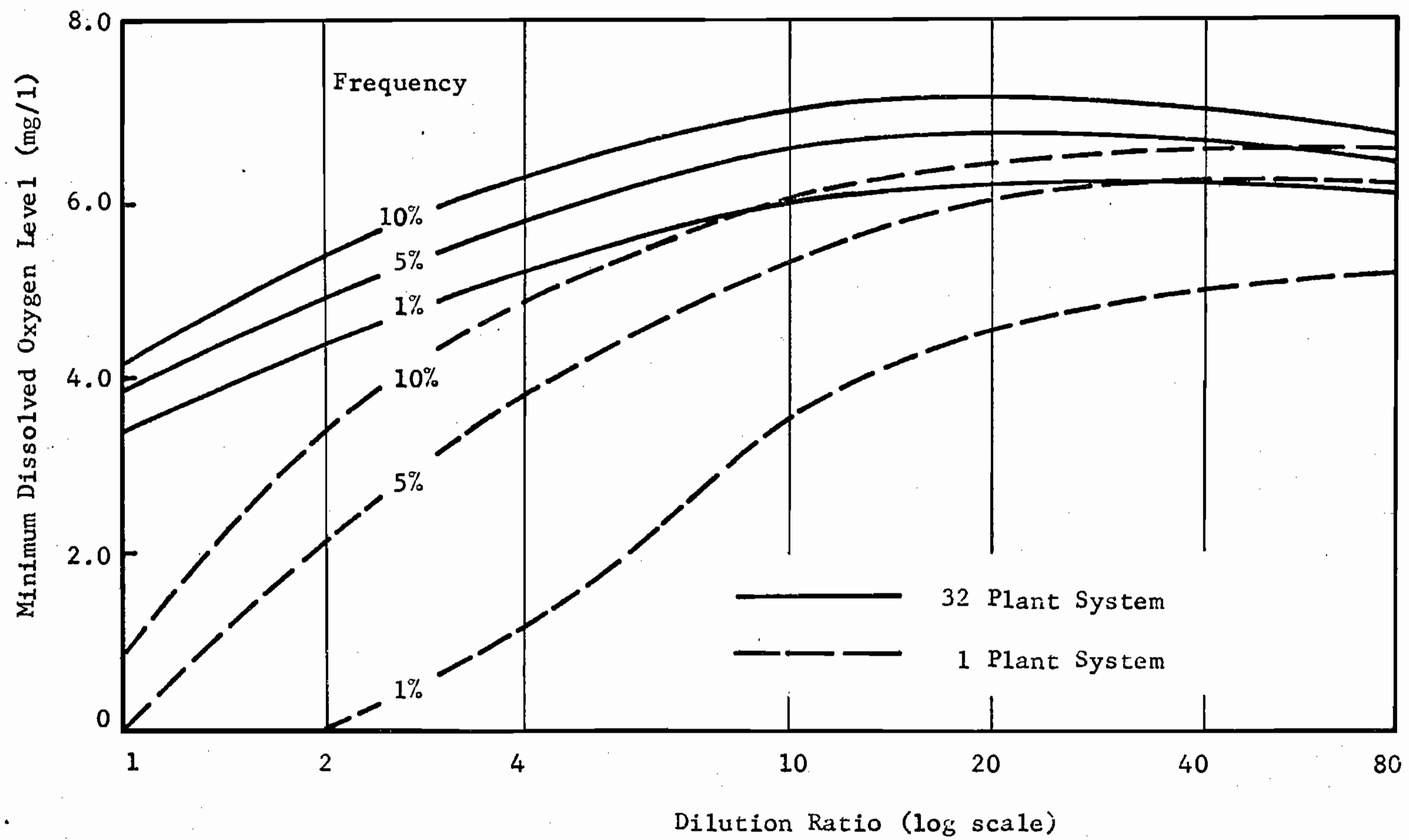


Figure 7.6

Low Frequency Plots for Various Dilution Ratios

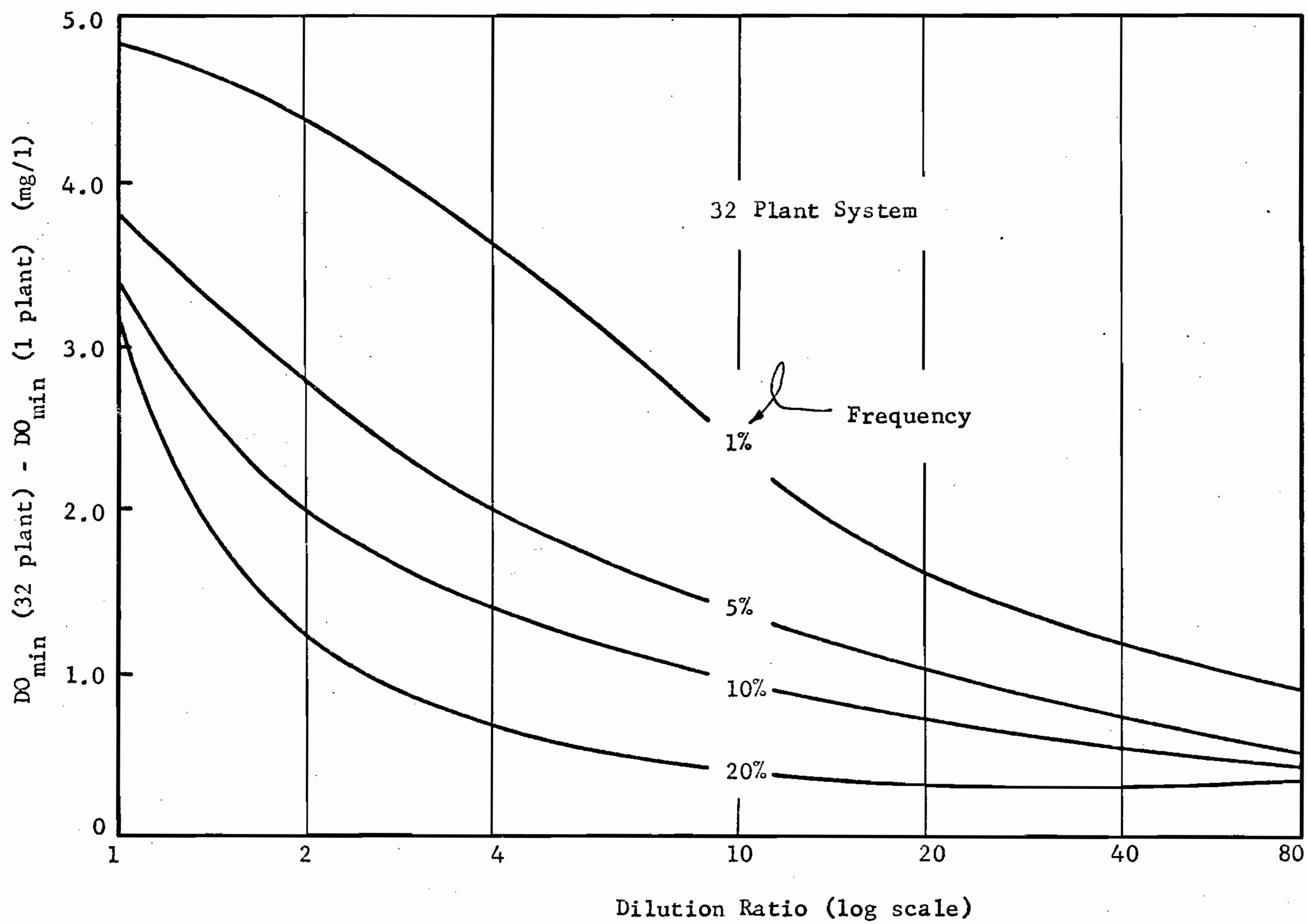


Figure 7.7

Low Frequency Plots of Water Quality Improvement of 32 Plant System

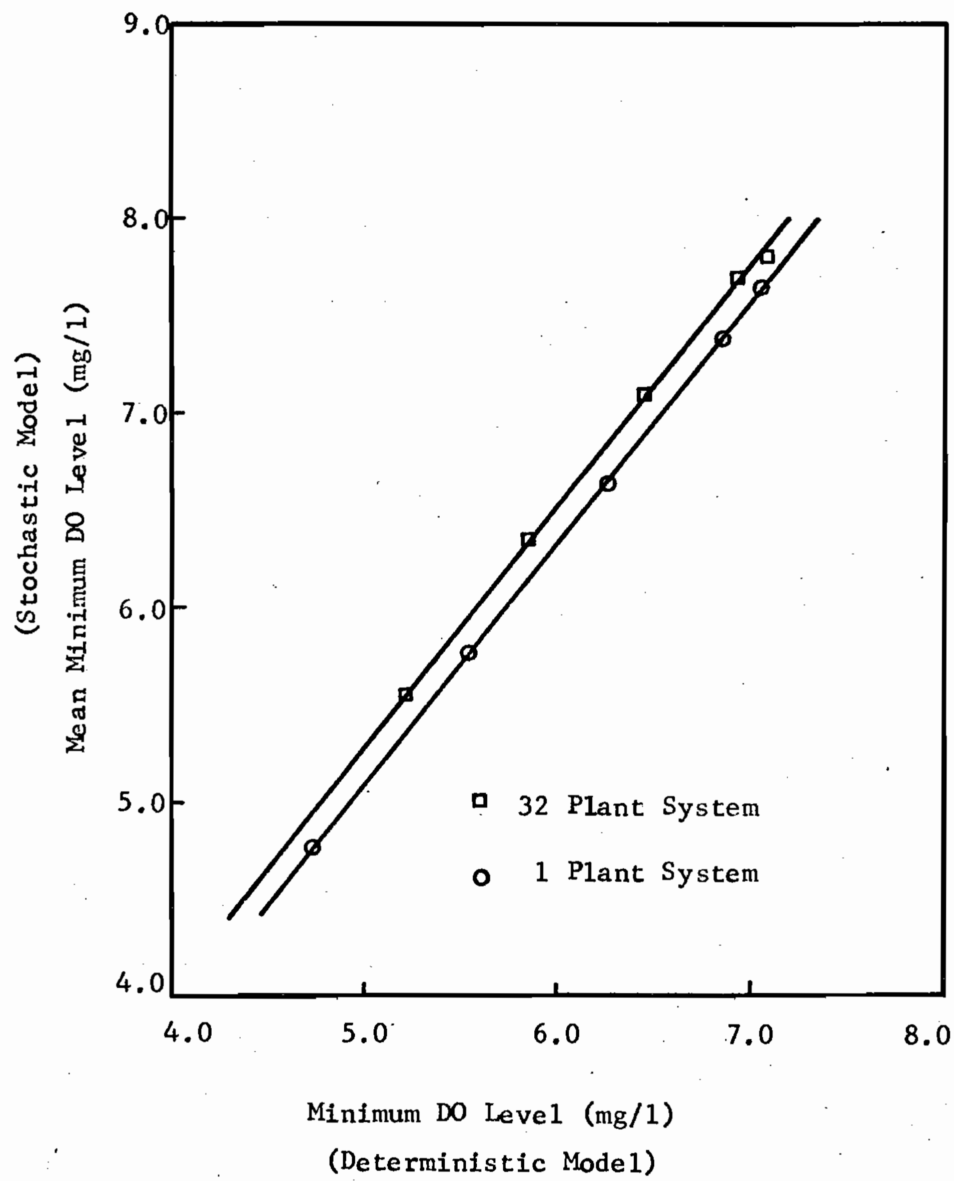


Figure 7.8

Correlation Plot for Stochastic-Deterministic Model Response - Mean Values

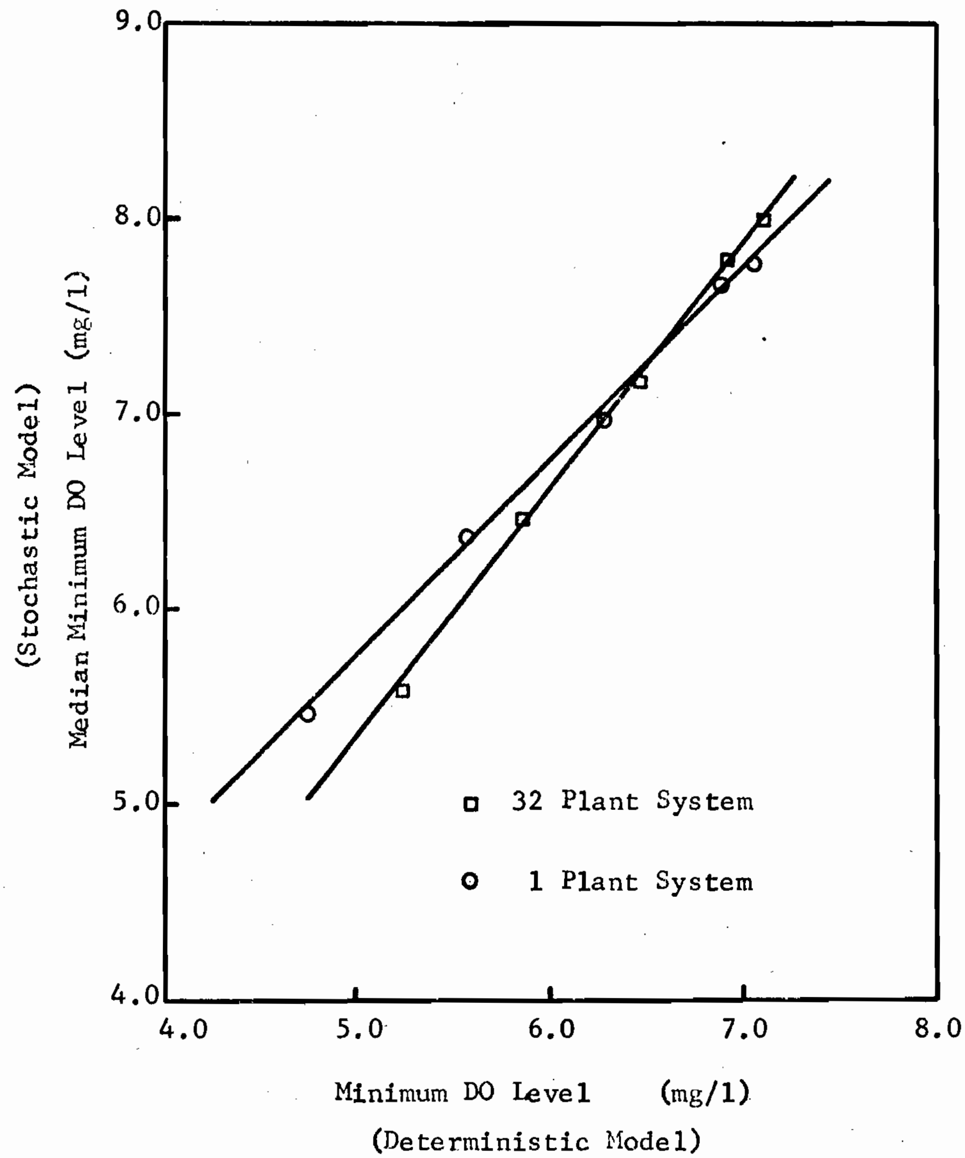


Figure 7.9

Correlation Plot for Stochastic-Deterministic Model Response - 50 Percent Values for time dependent variables. These variables are the wastewater and stream water temperature, the stream DO, and the streamflow. The simulation run numbers were then days based on a real time clock. The previous computer program was modified to incorporate the stochastic daily streamflow model and the real time clock, and then employed as the yearly water quality simulation model.

The output from these simulation runs are presented in Figure 7.10 as minimum dissolved oxygen cumulative distribution functions. These cdf's are plotted for the 1 and 32 plant systems. It is evident that responses for both plant systems contain very large variances. This is largely due to the variance of the deterministic functions incorporated in the water quality model. Because these yearly simulations are fairly expensive, only one year was simulated. Thus, the resulting cdf's represent a small sample of years and the critical seasons within the year and may not be stable functions, particularly at the low and high frequencies. It is suspected that the 32 plant system would produce a response with less low frequency variation than that indicated by the simulation of one year. This hypothesis was not further tested for several reasons. Firstly, this hypothesized phenomenon was tested and verified from the low flow season simulations based on 100's of runs. Secondly, an inspection of Table 7.5 indicates a decreasing trend in the system variance with the number of plants in the system. At the level of plant disaggregation reached by 32 plants, a sudden increase in system variance appears. Thirdly, the system being studied is a hypothetical one and it was not the exact functional response that was to be determined but the direction and magnitude of the response. Since the system is a hypothetical one, it was thought that the expense of the simulation of many years was unwarranted.

7.5 Summary of Results

The water quality impact of regional wastewater centralization due to stochastic variability was examined with stochastic simulation models. Simulations were executed in either one of two temporal frameworks: daily simulation of the critical low flow season and daily simulation of the

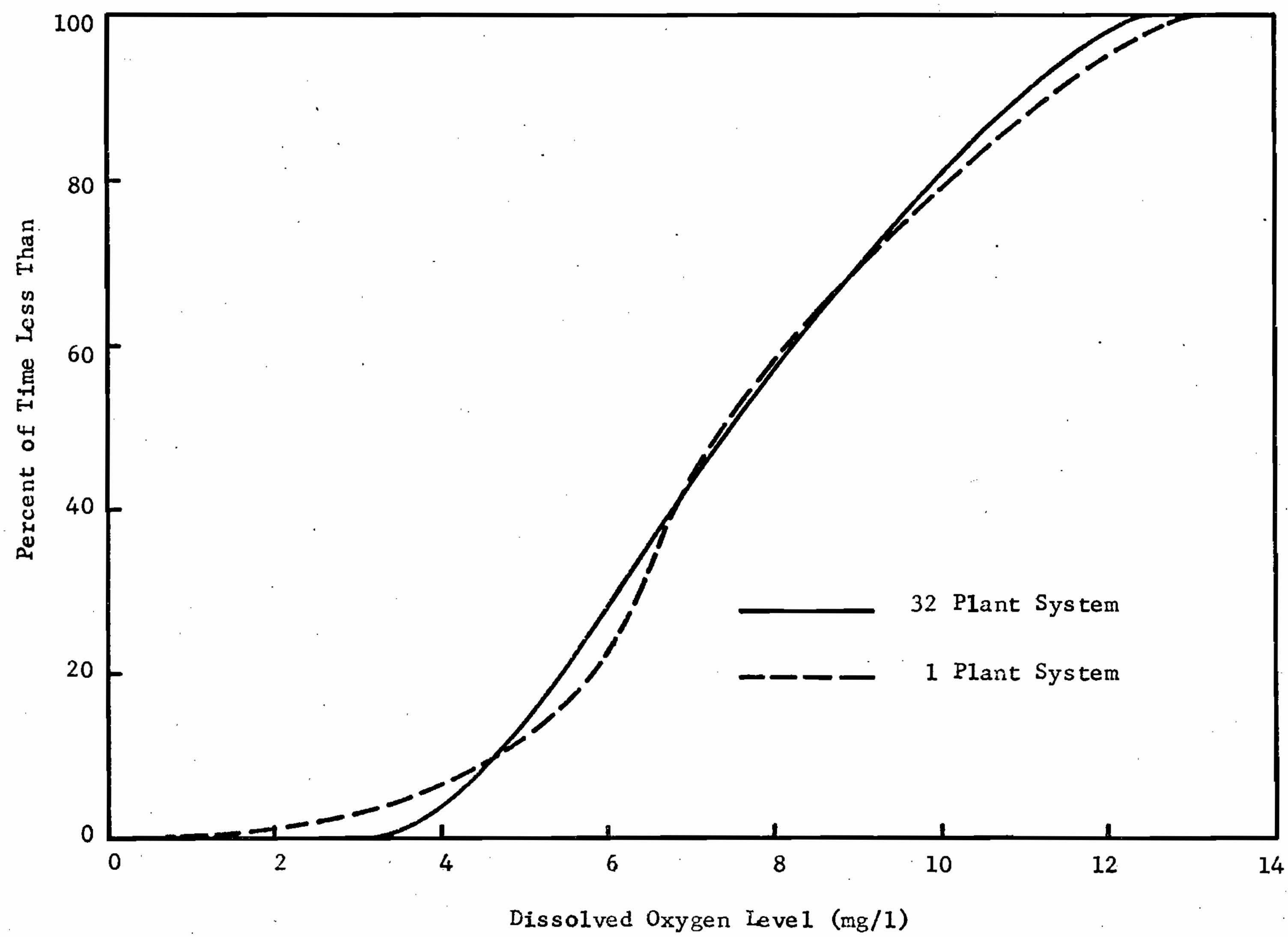


Figure 7.10
Frequency Plot of Water Quality Response for Yearly Simulation

TABLE 7.5

Minimum Dissolved Oxygen Levels from Yearly Simulation
(in mg/1)

Frequency DO Level Is Exceeded	Number of Plants					
	1	2	4	8	16	32
1%	2.3	1.7	4.1	3.4	3.5	3.2
5%	4.1	4.1	4.5	4.5	4.6	4.3
10%	4.9	4.8	4.9	5.0	5.2	4.8
20%	5.8	5.6	5.5	5.6	5.7	5.5
50%	7.3	7.1	7.5	7.1	7.4	7.7
Mean	7.73	7.23	7.65	7.51	7.46	7.75
Standard Deviation	2.45	2.07	2.14	2.19	1.92	2.38

entire calendar year. Systems with various degrees of centralization of wastewater treatment facilities were examined for different stream lengths and dilution ratios.

Increasing the state of aggregation or equivalently increasing the number of wastewater discharge points resulted in minimum DO frequency responses with smaller variances. Thus, at lower frequency levels, the multi-plant system resulted in a minimum stream DO significantly larger than that of the single plant system. The mean responses for short system lengths increased only marginally with an increase in plant number, in accordance with the results of the deterministic analysis.

Water quality simulations were undertaken for systems of various lengths. Increasing system length had the effect of not only increasing the mean minimum DO response but also decreasing the variance of the response of multi-plant systems. The response of the single plant system is not length dependent, and the cdf's of minimum DO did not change from the above. Thus, system decentralization reaps the benefits from both the deterministic effect of increased mean response with length and the stochastic effect of decreased variance of the response.

Repeating this simulation experiment for stream systems with various dilution ratios indicated that the variance of the minimum DO response decreased with increased dilution ratios. Since the mean response increased with increased dilution ratios in accordance with the results of the deterministic analysis, the coefficient of variation of the DO response decreased at a rate faster than that at which mean response increased or variance of the response decreased. It is concluded that the decentralized wastewater system significantly outperforms the centralized system when the dilution ratio is small. As the dilution ratio increases, this difference in performance decreases; conversely, this difference increases with an increase in system length.

The results of these simulations indicate that there is variation not only in the value of the critical oxygen deficit but also in the location of this deficit. Thus, associated with the probability distribution of the minimum dissolved oxygen level is a probability distribution of the location of the critical DO deficit. A study of this distribution of the location of the critical deficit reveals that there is a tendency for the variance of the distribution to decrease with an increase in the number of plants in the system. This observation was noted for all stream system lengths and dilution ratios that were explored. Furthermore, systems with a fixed number of plants exhibited distributions of critical DO deficit location whose variances decreased with increased dilution ratios.

CHAPTER VIII

ECONOMICS OF REGIONAL WASTEWATER MANAGEMENT

A management decision concerning the size, number, and location of wastewater treatment facilities to service a region must be made through a consideration of both the cost of constructing and operating alternative systems and the water quality benefits derived from the operation of those systems. The "best" decision may be mathematically determined through appropriate optimization techniques if the cost and benefit functions and the constraints of the alternative systems are known. There are several problems facing the regional wastewater manager in this regard. First, he must specify the alternative systems to be evaluated, and an initial screening of the sea of alternatives makes heavy demands on his professional judgment; a judgment which is honed from a delicate understanding of the physical problems as well as the socio-political milieu of the application. Secondly, the cost functions associated with system components to be employed in an optimal system selection may be determined only from economically optimal component designs. In addition, the benefits derivable from alternative systems may be indiscernable, especially for intangibles; thus, a quantitative benefit function may be intractable. Thirdly, the constraints on the system plan reflect a collective value of the people under whose authority the plan is undertaken. A budgetary constraint is the most common constraint in management decisions regarding investment in public services. Inherent in the nature of this constraint is a philosophy couching this collective value which is usually expressed by a political policy. The translation of this public value into a public policy is too often inaccurate. In view of these problems, the general concepts of utilitarianism and the public welfare may be mathematically manipulated with an optimization technique to produce numbers which even the manipulator may not fully appreciate.

Thus, it is evident that what might be a refined decision making technique may in fact be prohibitively crude. However, the purity of the decision may be refined through a refined knowledge on which this decision is based. In this light, the purpose of this research has been to refine knowledge concerning the performance of regional wastewater systems and their impact on the environment. It is from this knowledge that an intelligent decision regarding viable alternatives for water quality improvement systems may be made.

Rather than employing a formal optimization technique, this chapter will explore the nature of the cost functions of wastewater system components in order to allow a comparison of the investments an alternative system requires with the water quality levels the system produces. An attachment of dollar values to the water quality levels will not be undertaken. Although such a procedure would allow a direct monetary comparison, this procedure is not as meaningful in a study which is not addressed to a specific application.

8.1 Cost Indexes in Wastewater Management

The economic evaluation of engineering alternatives requires the use of not only fundamental concepts of engineering economics but also cost indexes by which the values of construction goods and services at different periods in time may be measured. Two sets of cost indexes are available to estimate, up-date, and compare the costs of wastewater management alternatives.

The Engineering-News Record (ENR) indexes are the older and a more generally accepted means for estimating total construction cost and historical updating. The Construction Cost Index was created in 1921 to diagnose price changes that occurred during and immediately following World War I and to evaluate their effect on construction costs. The index was designated as a general purpose construction cost index to chart basic costs. It is a weighted aggregate index of constant quantities of structural steel, portland cement, lumber, and common labor. This hypothetical block of construction, priced weekly from cost data of 20 selected major U.S. cities, was valued at $100 in 1913. The original use of common labor in the Construction Cost Index was based on the idea that it set the trend for all wage rates. However, in the 1930's wages plus fringe benefits climbed faster for laborers than for the skilled trades, in percentage terms. The ENR Building Cost Index was introduced in 1938 to weigh the impact of skilled labor on construction cost trends. For its average component, it uses skilled trades, an average of carpenter, bricklayer, and structural iron workers' wages. Its materials component is the same as that used in the Construction Cost Index. The building Cost Index also represents a hypothetical construction block valued at $100 in 1913.

Both the ENR Construction and Building Cost Indexes measure the effects of wage rate and materials price trends. The two cost indexes have developed a widening divergence over the years. This has resulted from the faster rate of increase in common laborers' rates than in skilled laborers' and from the use of nearly three times the manhours of labor in the Construction Cost Index as in the Building Cost Index. Thus, the Building Cost Index is generally the more applicable in measuring the degree of change because its skilled labor component is more representative of labor's share of the total cost of labor and materials in most types of construction. These indices are compiled and published monthly by Engineering-News Record.

The second set of cost indexes is that of the U.S. Environmental Protection Agency having its origins in 1961 with the Division of Water Supply and Pollution Control of the Department of Health, Education and Welfare. The Division undertook to design cost indexes for both sewage treatment plant and sewer construction, since none had existed specifically for these types of construction. The sewage treatment plant construction cost index (WSP-STP index) was developed and published by the USPHS in 1963, and subsequently, the sewer construction cost index (WSP-S index) was published in 1964. Monthly summaries continuously updating both indexes are prepared by the EPA. These indexes do not include the cost of land, engineering, legal and financial services, and errors, omissions, and changes subsequent to contract award. The indexes do include the cost of materials, contractors' equipment, process equipment (for sewage treatment works), labor, and the contractor's overhead and profit. These indexes are calculated for the same 20 trade centers employed in the ENR indexes and the national index is a simple average of these centers. The 36 month period from January 1, 1957, through December 31, 1959, was selected as the base period for the index and given a value of 100.

The later set of indexes was chosen for use in this study since these indexes were developed especially for estimating construction costs of alternative wastewater systems and are judged to be more accurate for these purposes.

8.2 Cost Functions of System Components

The choice of components for regional wastewater systems is a function of the physical and social morphology of the region. An urban region may be comprised of collection networks, interceptor pipes, and treatment plants while a total rural region may have none of these components. An urban center in an industrialized nation may employ technologically advanced treatment methods while that in an underdeveloped nation may resort to simpler and cheaper methods of treatment or no treatment at all. For the purposes of this study, three system components are examined: collection networks, interceptor pipes, and treatment facilities.

Interceptor System Costs

The cost determinants of wastewater transportation are classified into three basic categories by Dajani (1971). These categories are the sets of technological, site, and morphological variables. The technological variables affecting system cost may include such variables as the minimum and maximum allowable flow velocities, minimum allowable pipe diameter, minimum allowable pipe cover, and construction technologies. The site conditions may include the local terrain, soil conditions, climate, location to markets, per capita wastewater generation, spatial and temporal wastewater generation patterns, and the relative factor prices of right-of-ways, materials, and services. Similarly, the set of morphological variables may include the total population, service area, population density, density distribution over the service area, and the shape of the service area.

Given a wastewater flow to be transported over a given distance, the cost of the transportation system may be expressed as follows

$$C = a + bD^2 + cX^2 \tag{8.1}$$

in which C is the cost per unit length of pipe, D is the pipe diameter, X is the average depth of excavation for the trench, and a, b, and c are constants. The flow in the pipe may be expressed as a function of the two variables D and X with the equation of continuity and the equation of open channel or closed conduit flow. Substitution of this relationship into equation (8.1) yields a total cost function expressed as a function of flow, pipe slope, pipe length, average trench depth, and some constants. There is an infinite mix of pipe diameters that will accommodate the flow condition, and the economically optimal diameter and slope may be found from differentiating the total cost function (Dajani, 1971).

In cases where a less sophisticated estimate for pipe cost is required, an expression of the following form is often employed:

$$C = aQ^{b} \tag{8.2}$$

in which C is the cost in \$/mi, Q is the design flow in MGD, and a and b are constants. In a study of regional sewage systems in the Chicago area, Bauer (1962) reported the following equation as being typical

$$C = 4.0 \times 10^{4} Q^{0.50} \tag{8.3}$$

which is based on an ENR Construction Cost Index of about 1,000. In another study by Spencer (1958), cost data reported for the Buffalo area of New York indicated that the construction cost of trunk sewers based on an ENR Construction Cost Index of about 1,000 could be described by the following equation

$$C = 4.6 \times 10^{4} Q^{0.55} \tag{8.4}$$

In the event that gravity flow sewers become prohibitatively deep or that wastewater must be transported uphill, pumping stations may become necessary. Cost data for such stations are scarce but these data do indicate economies of scale as reported by Benjes (1960) for Mid West pumping stations operating at about 30 feet of head. Associated with sewage pumping stations are pressure or force mains. A study by Linaweaver and Clark (1964), based on cost data collected from across the nation, indicates that the cost of water transmission in forcemains may be described by the following

$$C = 1865\,Q^{0.6} \qquad (8.5)$$

in which C is the cost per mile (ENR Construction Cost Index = 877) and Q is the daily flow in MGD. This formulation is based on certain assumptions regarding interest rate, power costs, pumping efficiencies, pipe friction, and head. Although its applicability to a particular problem must be carefully evaluated, the economies of scale should not change so drastically as to invalidate its use for initial planning purposes (Deininger and Su, 1971).

Collection Network Costs

The cost of an urban wastewater collection network is a function of a number of variables. Some of these variables are internal to the technology of the service in question, others are a function of the specific site providing the problem setting, while a third set involves factors which relate to the morphology of the region in question. Some variables may be easily classified in the above manner, while others may fall in more than one category. Dajani's classification of cost determinants of wastewater collector networks into these three basic categories was discussed previously. Let the vectors (T), (S), and (M) denote the sets of technological, site, and morphological variables, respectively, and (f) be a functional relationship such that:

$$\text{Network Cost} = f(T, S, M) \qquad (8.6)$$

In a typical situation where both the site and technology are given, a cost estimation model may relate network cost to some descriptors of urban structure. Attempts to develop such estimating models have been frustrated by present methods of engineering design which do not rigorously incorporate local economic considerations such as local factor prices. This leads to a lack of optimality in the engineering design of sewer systems. Dajani reasons that it is theoretically and conceptually unacceptable to attempt to develop estimating models on the basis of sets of suboptimal cost data.

It has been shown that a wastewater collection network can be formulated as a non-linear programming problem (Holland, 1966). Applying this method of cost-optimization to a set of hypothetical networks covering an area of given size, developed at a variety of population densities, Dajani developed generic network cost functions for analytical purposes. These cost functions map the locus of all points relating minimum network cost and some parameter of urban structure. When this locus was mapped for cost vs. population density, it was found that a minimum-cost density exists.

The minimum cost density is obtained only in systems which include both fully-utilized main lines and under-utilized lateral lines. The first type exhibits economics of scale, while the existence of the second adds to the basic cost function of lines running full, a penalty function which decreases as the density increases. Total network costs for a given population density are shown to exhibit diseconomies of scale as the total area (or population served) is increased. This indicates a continuously increasing unit cost as the size of a constant-density service area is increased.

The basic relationship which Dajani developed from the set of fully-utilized networks may be expressed by any of the following three equivalent forms:

$$\text{Network Cost} = a\,A^{\alpha}\,D^{\delta} \tag{8.7a}$$

$$= a\,A^{\alpha-\delta}\,P^{\delta} \tag{8.7b}$$

$$= a\,P^{\alpha}\,D^{\delta-\alpha} \tag{8.7c}$$

for $D \geq d$. The penalty function imposed on low density systems with under-utilized networks may be similarly expressed in any of the following equivalent forms:

$$\text{Penalty Cost} = A^{\alpha}\,(bD^{-\gamma} - c) \tag{8.8a}$$

$$= b\,P^{-\gamma}\,A^{\alpha+\gamma} - cA^{\alpha} \tag{8.8b}$$

$$= P^{\alpha}\,D^{-\gamma}\,(bD^{-\gamma} - c) \tag{8.8c}$$

for $D \leq d$, in which a, b, c are constants, A is the gross area in acres, D is the gross population density in persons per acre, P is the total population, α is the exponent with a value greater than unity, δ and γ are exponents with values less than unity, d is the population density beyond which all links are utilized to their design capacity. The values of these coefficients and exponents, obtained from regressions of cost data, are presented in Table 8.1 for 10 acre and 20 acre city building blocks. These equations are based on optimal solutions of continuous cost functions but the actual designs must incorporate "even" sizes of pipe. The results from an integer programming problem solution (Dajani, 1973) indicate that these costs may be as much as 20 percent higher than that given by equations (8.9a) and (8.9b). Nevertheless, these same diseconomies of scale still exist, and it is the form of the cost functions rather than their exact values that is of interest.

Wastewater Treatment Costs

The cost of wastewater treatment is composed of the initial construction cost and the present value of the annual operation and maintenance costs. The unit costs of wastewater treatment depends on the type of treatment, the degree of treatment, and the size of the treatment works. Most studies on the economics of wastewater treatment facilities have concluded that the cost functions for a given degree of treatment may be expressed by the following general logarithmic form:

$$\text{Total Cost} = r\,P^{\sigma} \tag{8.9}$$

in which P is the population served, r is the coefficient, and σ is an exponent with a value less than unity, reflecting an economy of scale.

Shah and Reid (1970) have noted that economics of scale affect the unit construction costs of different types of secondary treatment facilities by a factor which changes very little from type to type, and Deininger (1969) has noted that these economics of scale in construction and operation are world wide. Table 8.2 presents cost functions which were calculated by Dajani (1971) by reducing the relationships ob-

TABLE 8.1

Total Cost Functions of Collection Networks

Gross Size of City Building Blocks	Total Network Cost = Basic Network Cost + Penalty Cost	
20 acre	$41.91\ A^{1.17}\ D^{0.30} + A^{1.17}\ (418D^{-0.3} - 92)$	(8.9a)
10 acre	$46.35\ A^{1.17}\ D^{0.30} + A^{1.17}\ (813D^{-0.4} - 111)$	(8.9b)

Source: J.S. Dajani, "Network Evaluation of Wastewater Collection Economics," Ph.D. Dissertation, Northwestern University, Evanston, Illinois, June 1971.

tained by the U.S. Public Health Service (1964), based on the WPC-STP index of 100, and by Smith (1968), based on June 1967 dollars, to the general format of equation (8.9). The validity of these studies as well as most others does not go beyond treatment plants serving populations in excess of 100-200 thousand persons. Few studies include plants serving as many as one million persons. More recently, Butts and Evans (1970), in a study of Illinois municipal wastewater treatment plants, revealed that after a certain plant size the economy of scale decreases and that this may occur at plant sizes with a design population equivalent as small as 10,000 perssons. Variations of the above format exist. Regional, temporal, local, and technical differences account for variations in the values of the parameters obtained by different researchers. These functions are presented to indicate the general form and order of magnitude of the relationships involved. It should be noted that employing the equations of Table 8.2 for larger plant sizes may result in conservative cost estimates and thus present a bias in favor of centralized wastewater treatment facilities.

Net Economy Functions

The components involved in a metropolitan wastewater system will include collection networks and treatment plants if the region is undeveloped or interceptor systems and treatment plants if the area is developed and sewered but without treatment. Similarly, a rural or country scale region would most likely involve interceptor and treatment plants as system components. In view of the component cost functions presented above, a regional wastewater management system would employ components with economies of scale in addition to those with diseconomies of scale. In the planner's search for an overall minimum cost system, the economic trade-offs between different system components must be investigated. There is a state of aggregation or centralization of wastewater systems that produces the lowest unit cost for given morphological conditions. There is also a state of aggregation that results in the highest level of the water quality, and a trade-off between system cost and water quality must likewise be investigated.

Table 8.2

Total Cost Functions of Treatment Facilities

Type	U.S. Public Health Service	Robert Smith
Primary Treatment	$480\ P^{0.69}$	$530\ P^{0.70}$
Trickling Filters	$760\ P^{0.66}$	$320\ P^{0.80}$
Activated Sludge	$450\ P^{0.72}$	$440\ P^{0.77}$

Sources: Calculated from information published in:

1. U.S. Public Health Service, Modern Sewage Treatment Plants--How Much Do They Cost? (Washington, D.C., Government Printing Office, 1964).

2. Robert Smith, "Cost of Conventional and Advanced Treatment of Wastewater," Journal of the Water Pollution Control Federation, Vol. 40, September 1968, pp. 1546-1574.

by J. S. Dajani, "Network Evaluation of Wastewater Collection Economics," Ph.D. Dissertation, Northwestern University, Evanston, Illinois, June 1971.

Such an analysis has not been possible because of the lack of information concerning the water quality effects of wastewater system centralization.

In attempting to develop a complete economic analysis of the system, however, a problem arises with respect to the ranges of validity of the system component cost functions. They have been derived within a limited population range, and the following analysis assumes that these cost functions can be extrapolated to cover the untested range of higher populations and larger areas with sufficient accuracy to reveal general system characteristics.

8.3 Some Numerical Examples

With the use of the above cost functions for system components, it is possible to estimate the cost of implementing alternative regional wastewater management plans. The alternatives considered herein are given by the size, number, and location of wastewater treatment plants incorporated in these plans. Thus, for different conditions of regional morphology, the costs of systems representing various degrees of centralization were estimated and compared to the water quality levels resulting from the operation of these systems.

The system components employed in this analysis were interceptor pipes, collection networks, and treatment plants. Since the cost functions for these components were derived on different economic bases, they must be expressed in terms of equivalent monetary units. The WPC-S and WPC-STP indexes were employed to update these functions to 1972 U.S. dollars. The following updated cost functions were employed:

$$C_I = 65{,}000\ Q^{0.5} \tag{8.10}$$

$$C_N = 56.3\ A^{1.17}\ D^{0.30} + A^{1.17}\ (561D^{-0.30} - 123) \tag{8.11}$$

$$C_T = 635\ P^{0.77} \tag{8.12}$$

where C_I, C_N, and C_T are the costs of interceptors (\$/mi), collection networks (\$), and treatment plants (\$), respectively, Q is the design flow (MGD), A is the service area (acres), D is the population density (persons/acre), and P is the total population.

Two sets of experiments were conceived in which system cost estimates were undertaken. The first set considered systems of interceptors and treatment plants while the second set considered systems of networks and treatment plants.

The first set of experiments considering interceptor systems involved a series of various sytem lengths. Each system length was divided into a series of 32 equally sized cells. Various plant number systems were considered for each system length. It was further assumed that each cell in the system generated the same mean quantity of wastewater and that this wastewater was drained to the downstream corner of the cell which was designated a potential treatment site. The 32 plant system treated wastewater at each of these sites, the 16 plant system treated wastewater at every second site downstream, and so on to the single plant system which treated wastewater only at the last downstream site. Thus, the cells were aggregated in pairs in the downstream direction until the wastewater of the entire region was treated at the downstream corner of the system. This procedure resulted in the same physical systems of equally spaced plants analyzed in Chapter IV (as depicted by Figure 4.1). This is the most economical system of progressive downstream aggregation.

Cost information for these systems and measures of their water quality impact for systems providing a 2/1 dilution ratio are presented in Table 8.3. The observations on water quality impact are of course the same as those made in Chapter VII. The unit costs of these wastewater systems have minimums at intermediate plant numbers for short system lengths and at large plant numbers for longer systems. For example, the minimum cost system is between 2 and 4 plants for the 64 mile stream, between 4 and 16 for the 128 mile system, and at 32 plants for streams 192 miles and longer. In all but the 64 mile stream system, the multi-plant system was not only more economical but also resulted in a higher water quality than the centralized single plant system.

Table 8.3

Unit Costs of Wastewater Systems - Interceptors and Treatment
(in 1972 dollars per capita)

and Minimum DO Levels at Percent of Time Less Than
(in mg/1)

Number of Plants / System Length (mi)			1	2	4	8	16	32
64	Cost		44	42	42	44	47	52
	DO_{min}	1%	0.0	1.0	3.6	4.6	5.4	5.6
		5%	0.0	3.2	4.4	5.4	5.6	6.1
		Mean	5.7	5.7	6.0	5.7	5.8	6.4
128	Cost		65	56	51	49	50	52
	DO_{min}	1%	0.2	0.6	3.8	3.9	3.9	5.3
		5%	3.2	3.3	4.4	5.0	4.8	5.5
		Mean	5.9	6.2	6.5	6.4	6.5	6.7
192	Cost		85	70	61	55	53	52
	DO_{min}	1%	0.0	1.6	3.6	4.9	5.2	5.2
		5%	0.0	3.6	4.6	5.5	5.9	6.1
		Mean	5.8	6.3	6.6	6.9	6.9	7.1
256	Cost		106	84	70	61	56	52
	DO_{min}	1%	0.0	1.8	4.4	4.6	5.7	6.0
		5%	0.7	3.2	5.3	5.5	6.2	6.5
		Mean	5.3	6.6	7.0	6.9	7.2	7.4
320	Cost		127	99	80	67	58	52
	DO_{min}	1%	0.0	1.5	2.7	5.6	5.5	5.5
		5%	0.0	4.3	4.7	6.0	6.5	6.7
		Mean	5.6	6.7	6.9	7.3	7.4	7.5
384	Cost		147	113	89	73	61	52
	DO_{min}	1%	0.0	2.3	4.2	5.3	5.5	6.9
		5%	2.1	3.9	5.6	6.2	6.8	7.1
		Mean	5.3	6.7	7.3	7.4	7.7	7.8

At shorter system lengths, a trade-off of cost for water quality may be made between intermediate and large plant number systems. At longer system lengths, both factors of cost and water quality weigh heavily in favor of decentralized multi-plant systems.

The second set of experiments considering wastewater collection networks involved a 64 mile stream system with various plant numbers and population densities. The analysis assumed that the number of networks serving the region was equal to the number of treatment plants. Some results from the cost estimations of these systems in addition to measures of water quality resulting from the operation of these systems are reported in Table 8.4. It is evident that at any level of aggregation, the unit costs decrease with an increase in density for density levels commonly experienced in U.S. cities today. At low densities, a continuous decrease in unit cost is experienced with an increase in plant number up to a 32 plant system. At population densities above 17 persons per acre the least cost solutions result from intermediate plant number systems. For example, at a population density of 20 ppa the least cost system lies between 8 and 16 plants while at a population density of 25 ppa the least cost system lies between 4 and 8 plants. Thus, low density regions may be serviced at a lower cost and a higher water quality level with large plant number systems; at high population densities a trade-off of cost for water quality may be made between intermediate and large plant number systems.

Table 8.4

Unit Costs of Wastewater Systems - Networks and Treatment
(in 1972 dollars)

and Minimum DO Levels of Percent of Time Less Than
(in mg/1)

Number of Plants		1	2	4	8	16	32
Population Density (persons per acre)							
5		571	514	465	422	386	356
10		233	213	197	185	175	168
15		145	136	128	123	120	119
20		107	102	98	97	97	98
25		87	84	82	82	84	87
DO_{min}	1%	0.0	1.0	3.6	4.6	5.4	5.6
	5%	0.0	3.2	4.4	5.4	5.6	6.1
	Mean	5.7	5.7	6.0	5.7	5.8	6.4

CHAPTER IX

SUMMARY AND CONCLUSIONS

A water quality evaluation of regional wastewater system centralization was undertaken to test the hypothesis that water quality improvement may result from the spatial and temporal variation of wasteloads attributed to decentralized regional systems. The evaluation employed water quality models developed for both deterministic and stochastic analyses. Each analysis considered a set of experiments which involved a determination of the water quality resulting from alternative degrees of regional wastewater aggregation. The experiments treated not only the degree of aggregation or equivalently the density of plants in the system, but also the stream system length as an indicator of regional morphology and the dilution ratio as an indicator of stream size. The water quality assessment was made in terms of the minimum dissolved oxygen level experienced by the system. The base or control condition of the experiment employed a set of nominal system and model parameters as presented in Table 3.5. A sensitivity analysis explored the model response to the variation in the parameter values in order to isolate the parameters to which water quality was most sensitive: streamflow and loading conditions (wastewater BOD and wastewater flow). The water quality evaluation of deterministic and stochastic systems involved variations on and comparisons with the nominal system.

With a water quality evaluation of regional wastewater system centralization completed, an economic evaluation was undertaken. Cost functions for regional wastewater system components were developed, and costs of the physical systems hypothesized in the water quality analyses were estimated with these cost functions. Two sets of wastewater systems were investigated, each with two components. One system involved interceptors and treatment plants and the other involved collection networks and treatment plants. Cost estimates were developed over system length for the former system and over population density for the latter system. Finally, a comparison was made between the water quality impact of various regional wastewater systems and the economic impact of these systems.

9.1 Water Quality Analysis

The deterministic water quality evaluation considered various states of wastewater aggregation or, equivalently, treatment plant densities expressed as the number of plants in the system (1, 2, 4, 8, 16, and 32 plant systems). The stream system length was divided into a number of equally sized sanitary drainage districts, equal to the number of treatment plants. It was assumed that each plant in a particular system discharged the same quantity and quality of wastewater. The experiment on plant density began with a 32 plant system and subsequently aggregated plants in pairs in the downstream direction. A water quality evaluation was made after each stage of plant aggregation through the water quality model. This aggregation process was repeated for streams of various lengths (64, 128, 192, 256, 320, and 384 miles) and for streams providing various dilution ratios (1/1, 2/1, 4/1, 10/1, 20/1, 40/1, and 80/1).

The results of these experiments on deterministic water quality systems are summarized below:

1. For a given stream size, or dilution ratio, and a given stream system length, an increase in the disaggregation state of wastewater treatment plants results in an improved quality.
2. This water quality improvement is negligible for short stream system lengths typical of metropolitan regions (0.3 mg/l DO increase from 1 to 32 plant systems, 64 mile stream) while it is considerable for longer stream system lengths indicative of more rural regions. The magnitude of the improvement may be greater than a 2 mg/l increase in the minimum stream dissolved oxygen level.
3. The greater part of the water quality improvement due to decentralization of plants is achieved by a disaggregated state of approximately 8 plants. Further increases in disaggregation state result in only marginal improvements over an 8 plant system. This conclusion holds for all system lengths that were explored.
4. The water quality improvement attributed to decentralized regional wastewater treatment systems is also a function of dilution ratio. For a given stream system length, the water quality improvement

due to a fixed disaggregation state increases with a decrease in the dilution ratio.

5. For a given stream system length and disaggregation state, the minimum DO experienced increases with an increase in dilution ratio to a maximum after which it decreases with further increases in dilution ratio.
6. The dilution ratio at which DO_{min} is maximized decreases with an increase in distance between plants.
7. For a given disaggregation state, ΔDO_{min} (the difference between the minimum DO of the n plant system and that of the 1 plant system) increases with an increase in distance between plants at a decreasing rate.
8. As the disaggregation state is increased, ΔDO_{min} is increased at a decreasing rate such that at a disaggregation state of 8 plants the increase in ΔDO_{min} due to further disaggregation is negligible.
9. From an assessment of the sensitivity of these experiments to the natural stream condition of flow augmentation with length, it is concluded that although DO_{min} is generally larger for short system lengths and smaller for longer system lengths, there exists the same general increase in water quality with an increase in disaggregation state.

The water quality impact of regional wastewater system centralization due to stochastic variability was evaluated with stochastic simulation models. Simulations were executed in one of two temporal frameworks: daily simulation of the critical low flow season and daily simulation of the entire calendar year. Systems with various states of aggregation of treatment plants were evaluated for the same stream system lengths and dilution ratios examined by the deterministic evaluation of spatial variability of treatment plants.

The results of these simulation experiments on stochastic water quality systems are summarized below:

1. Decreasing the state of aggregation or equivalently increasing the number of wastewater discharge points results in minimum DO frequency responses with increasingly smaller variances. Thus, at the lower frequency levels, the disaggregated multi-plant systems result in minimum DO concentrations significantly larger than that of highly aggregated systems.
2. Water quality simulations undertaken for systems of various lengths revealed that increasing system length had the effect of not only increasing the mean minimum DO response in accordance with the results of the deterministic analysis but also decreasing the variance of the response of disaggregated systems. Since the response of the single plant system is not length dependent, system decentralization has the benefit of both the deterministic effect of increased mean response with increased length and the stochastic effect of decreased variance of water quality response with increased plant number.
3. The simulation experiment for stream systems with various dilution ratios revealed that the variance of the minimum DO response decreases with an increase in dilution ratio. Since the mean response increases with increased dilution ratio in accordance with the deterministic analysis, the coefficient of variation of the DO_{min} frequency response decreases at a rate faster than that at which mean response increases or variance of the response decreases.
4. At low frequencies, the value of Δ DO_{min} increases significantly with an increase in disaggregation state. At a state of approximately 8 plants, further disaggregation results in only marginal increases in Δ DO_{min} over that of an 8 plant system. This conclusion was drawn for all system lengths explored.
5. Results of a yearly water quality simulation revealed that the variance of system response is even larger than that revealed by the critical period simulations and that a smaller relative difference occurs between the responses of single and multi-plant systems.

9.2 Economic Analysis

The components of regional wastewater management systems that were investigated are: interceptor systems, collection networks, and treatment plants. Cost functions were developed for these components based on information presented in published economic studies. The cost functions were updated to a common economic base condition with the WPC-S and WPC-STP indexes. With these cost functions, estimates of the economic impact of the physical system hypothesized in the water quality analysis were undertaken.

The first set of regional wastewater management systems analyzed was composed of interceptors and treatment plants. Cost estimates of these systems were made for stream systems of various lengths. The shorter stream systems depict developed metropolitan regions while the longer stream systems depict county-level or basin-level regions. The second set of systems analyzed was composed of collection networks and treatment plants. Cost estimates were made with the shortest (64 mile) stream system, depicting an undeveloped metropolitan region, for a series of population densities.

The results of these economic evaluations are summarized below.

1. The unit costs of wastewater systems employing interceptors and treatment plants have minimums at intermediate disaggregation states for short system lengths and at high disaggregation states for longer system lengths. At longer stream system lengths (192 miles and longer), the disaggregated plant systems are not only more economical but also result in a higher water quality than that of any system with a lesser disaggregation of plants. At shorter system lengths (128 miles and shorter), a trade-off of cost for water quality may be made between systems of intermediate and high disaggregation states.
2. The unit costs of wastewater systems employing collection networks and treatment plants decrease with an increase in population density for any level of aggregation. At low population densities a continuous decrease in unit cost is experienced with an increase in disaggregation state. At population densities above

17 persons per acre, the least cost solution results from intermediate disaggregation states. Thus, low density regions may be serviced at a lower cost and a higher water quality with decentralized regional wastewater management systems; at higher population densities a trade-off of cost for water quality may be made between intermediate and highly decentralized systems.

9.3 Consideration of Related Factors

In addition to the factors addressed in this study, a number of other factors not specifically considered may have a bearing on the results of this research. The nature of their effects are described and the response of wastewater management systems to these effects is discussed.

1. The stochastic water quality model developed for the purposes of this study was a steady state model. Although input wasteloads and stream conditions were assigned day-to-day variability, they were assumed to be constant long enough for the system to achieve a steady state condition. In reality, there is short term, hour-to-hour, variability. The effect of short term variability on the system is of importance when longitudinal dispersion is considered. Li (1972) has shown that the effect of longitudinal dispersion is negligible in a steady state system while it may be considerable for the case of short term variability of input wasteloads (Li, 1972). This was demonstrated analytically using a periodic BOD input without random variability. Although the knowledge of short term effluent BOD variability is scant at best and a simulation of this case is obstructed by this lack of knowledge, it should be noted that, on the basis of Li's study, longitudinal dispersion may significantly reduce the amplitudes of DO fluctuations along the stream (up to 50 percent of the steady state value). It should also be noted that there are larger fluctuations due to short term variability which would cause a greater variance of the system response than that determined by the steady state analysis of day-to-day variability. Thus, the consideration of short term variability would combine the opposing forces of decreasing vari-

ability of DO response due to dispersion and increased variability of DO response due to the increased variability of inputs on a short term basis.

2. Depending on the nature of the watercourse, the action of photosynthetic organisms may have an insignificant effect on the DO response or it may dominate the DO response. Dissolved oxygen is added and depleted by photosynthesis in a diurnal cycle with the oxygen added usually more than that depleted. A daily simulation would consider this net positive oxygen addition for applications in which photosynthesis is significant. An hourly simulation of dissolved oxygen would consider the diurnal fluctuation of dissolved oxygen. In the former case, the conclusions of this study would be essentially unaltered, and in the latter case, the conclusions regarding water quality improvement by decentralized systems would be amplified.

3. The effect of the second or nitrogenous stage of the biochemical oxygen demand may be significant. Typically, the nitrogenous BOD is not actively exerted until the 4th to 8th day after discharge from a treatment plant. This time lag is usually attributed to the development of an acclimatized nitrifying bacterial population. The exertion of nitrogenous BOD has been described as a first order reaction incorporating a lag by Thomas (1940) and as a second order reaction which inherently incorporates a lag by ReVelle, et al (1965). From studies on the Grand River by Courchaine (1963), the lag period of nitrogenous BOD exertion was found to be in the order of 4 days. In the 32 plant 64 mile stream system, the discharge from the first plant has mixed with the discharge from the last plant within 2 days. Thus, the effect of nitrogenous BOD on the n plant system would be experienced well downstream of the last plant in the system. This effect would be analogous to that of the carbonaceous demand but displaced in space and time.

   The assignment of values to parameters in the nitrogenous BOD formulation is made with less experience and confidence than with carbonaceous BOD formulations. Data on treatment plant effluent nitrogenous BOD variability is likewise more scant than that for carbonaceous BOD. These factors make a model for nitrogenous BOD effects less accurate than the model employed in this study. Since there is similitude in the impact of carbonaceous and nitrogenous BOD exertion on the stream and since the nitrogenous effects are less well defined, nitrogenous BOD exertion was not incorporated into the water quality model employed in this study although its potential importance is noted.

4. The deoxygenation and reaeration coefficients were assumed to be independent of the BOD concentration. It has been demonstrated that these coefficients are in fact not independent of pollution concentration. The coefficient of deoxygenation increases with increased concentration while the coefficient of reaeration decreases with concentration; however, little is known about the functional relationships involved (Tsivoglou and Wallace, 1972). Generally, it may be stated that the net effect of these changes in rate coefficients results in a more deteriorated water quality than that determined by holding these coefficients constant. This observation would magnify the water quality improvement attributed to decentralized wastewater systems by this study.

5. The characteristics of the physical system simulated were assumed to be as simple as possible to enable conclusions to be stated as generally as possible. Stream characteristics will vary from application to application and will become more and more complex. Such a complexity is that of a branched stream system. This case is cited as an example of how the general results reported above might be used to shed light on a more complex system.

A branched stream system would provide a greater opportunity for system alternatives. Two branches meeting at a confluence would allow the system designer to take advantage of the assimilative capabilities of both branches. In effect, the branches could be viewed as parallel systems up to the point of confluence. Downstream from this point, the outputs of the branches are simply added to form an input to the mainstream. The effect of a multi-plant loading on each of the branches would be proportional to the relative sizes of the branches and discharge loads while the effect of the mainstream would be dependent on these factors plus the distance from the discharge points to the point of confluence. This reasoning could be extended to higher order streams.

Finally, it should be stated that this study was not directed specifically to a general purpose model applicable to an evaluation of wastewater centralization for all regional applications although the models developed in this study may be modified to accomplish such an end. Rather, the intention was to study the behavior of water quality systems relative to the practice of regional wastewater centralization in order to contribute information for an evaluation of that practice. It is through such evaluation that an understanding of man's environment may be developed to allow a rational approach to its management.

NOTATION

A	cross-sectional area of stream channel service area amplitude of sine term in a periodic function net rate of removal of oxygen by benthal demand
a	least squares estimate of α constant
ASCE	American Society of Civil Engineers
a_t	t^{th} residual in time series analysis
B	channel breadth backwards operator amplitude of cosine term in a periodic function
b	constant least squares estimate of β
BOD	biochemical oxygen demand
C	unit cost
c	escape coefficient constant dissolved oxygen concentration
cdf	cumulative distribution function
cfs	cubic feet per second
c_s	dissolved oxygen concentration of saturated solution
C_v	coefficient of variation (= σ/μ)
^oC	degrees Centigrade
D	dissolved oxygen deficit coefficient of molecular diffusivity (= 0.8×10^{-4} sq. ft/hr) pipe diameter population density in persons per acre
d	population density beyond which all links in a network are used to their design capacity d^{th} difference in time series analysis
D_L	longitudinal mixing coefficient
D_o	initial DO of stream-wastewater mixture

DO	dissolved oxygen concentration
DR_{min}, Dr_{min}, D_{min}	minimum stream dissolved oxygen concentration
DR,Dr	stream dissolved oxygen concentration
DRO,Dr_o	initial DO of stream prior to mixing with wastewater
DSAT,D_{S_T}	saturated dissolved oxygen concentration at temperature T
DW,D_w	dissolved oxygen concentration of wastewater
ΔDO_{min}	difference between DO_{min} of the n plant and single plant systems also known as the water quality improvement of the n plant system
E	energy dissipation per unit mass of fluid expected value
e	base of natural (Naperian) logarithms
EBOD	effluent BOD
EDO	effluent DO
ENR	Engineering News Record
EPA	Environmental Protection Agency
ESS	effluent SS
F(i),f(t)	deterministic response function
fps	feet per second
fph	feet per hour
F_r	Froude number
^oF	degrees Fahrenheit
G	mean temporal velocity gradient
g	gravitational constant
gpcpd	gallons per capita per day
H	depth
H_o	null hypothesis
H_1	alternate hypothesis
hr	hour
Δh	change in water surface elevation
$(\Delta h)_{\frac{1}{2}}$	"half height" or water surface elevation change required for the downstream deficit to take the value of half the upstream deficit

i	index for reach of stream time of the i^{th} observation on a variable
IBOD	influent BOD
ISS	influent suspended solids
j	harmonic number ranked observation
k	number of variables in an analysis
K_1	deoxygenation coefficient (base 10)
k_1	deoxygenation coefficient (base e)
K_2	reaeration coefficient (base 10)
k_2	reaeration coefficient (base e)
k_3	rate constant for BOD removal by sedimentation or adsorption or both (if positive) or BOD addition by resuspension (if negative)
L	BOD concentration stream length
ℓ	lag number
ln	natural (Naperian) logarithm
L_o	initial BOD of stream-wastewater mixture
LR, L_r	stream BOD concentration
LRO, L_{r_o}	initial BOD of stream prior to mixing with wastewater
LW, L_w	BOD concentration of wastewater
M	difference between number of concordant and number of discordant pairs logarithmic mean
m	constant number of harmonics in a periodic regression model
MGD	million gallons per day
mg/l	milligrams per liter
mi	mile(s)
MN	Manning number

N	number of plants fundamental period number of observations in a sample
n	number of plants fundamental period number of observations in a sample Manning number index of stream type in terms of physical mixing conditions
N_c	number of concordant pairs
N_d	number of discordant pairs
NID	normal independently distributed
NAS	National Academy of Science
NRC	National Research Council
OD	oxygen deficit
P	total population
p	autoregressive model order
ppa	persons per acre
Q	flow rate
q	unit runoff (cfs/sq. mi) stochastic function moving average model order
$\bar{Q}$	mean treatment plant flow
QR,Q_r	stream flow
QRO,Qr_o	initial stream flow prior to mixing with wastewater
QW,Qw	wastewater flow
R	hydraulic radius velocity rate of addition of BOD along a stream reach by local runoff
r	simple linear correlation coefficient of sample constant
RR	removal ratio (= (IBOD-EBOD)/IBOD)
$R_{1.23....}$	multiple linear correlation correlation coefficient with dependent variable 1 and independent variables 2,3,....

${}_{\ell}R_n$	autocorrelation coefficient of lag ℓ from a sample of size n
S	sum of squares slope of stream channel
s	sample standard deviation length of stream system
SS sq.mi. sq.ft.	suspended solids square mile(s) square foot (feet)
$s_{1.23....}$	standard error of estimate with dependent variable 1 and independent variables 2,3,....
T	a statistic temperature fundamental period
t	time number of tied observations in a group
T_{min}, t_{min}	time flow required to reach point of critical deficit from point of introduction of wastewater
To TR,Tr	initial temperature of stream-wastewater mixture stream temperature
TRO,Tr_o	initial stream temperature prior to mixing with wastewater
TW,Tw	wastewater temperature
μ_i	sample standard unit of x_i
USPHS	United States Public Health Service
V	velocity variance
v_i	sample standard unit of y_i random component of stochastic function at time i
V_j	percent of the total variance accounted for by the j^{th} harmonic
v_t	random component of stochastic function at time t
W WPC-S WPC-STP	Kendall Coefficient of Concordance USPHS sewer construction cost index USPHS sewage treatment plant construction cost index

X	breadth-to-depth ratio of stream channel depth of trench
x	distance downstream
x_i	i^{th} observation on random variable x
y_i	i^{th} observation on random variable y
y_t	recorded observation at time t
z_t	residual of t^{th} data trace in time series analysis
$*$	subscript denoting parameter values at the end of a stream reach
α	intercept of the linear regression model level of significance area exponent in network cost function
α_o	mean value of a periodic function
α_1	amplitude of a periodic function
α_2	phase angle of a periodic function
β	slope of the linear regression model
β_i	i=0,1,....,m autoregressive coefficients
α_j, β_j	harmonic coefficients for the j^{th} harmonic in the periodic regression model
δ, γ	population and population density exponents in network cost function
ϵ_i	random departure from a regression model at time i random normal deviate
μ	population and sample mean
ϕ	constant for temperature correction of K_2 (= 1.024)
ϕ_p	constant associated with order p autoregressive function (= $1-\phi_1-\phi_2-\dots-\phi_p$)
ϕ_s	non-dimensional variable which varies with channel geometry

ϕ_v	non-dimensional variable which is a measure of surface velocity
ρ	correlation coefficient
ρ_1, ρ_2	autocorrelation coefficient of lag 1, 2
σ	population and sample standard deviation population exponent in treatment cost function
σ^2	population and sample variance
τ	Kendall rank correlation coefficient
θ	constant for temperature correlation of K_1 (= 1.047)
θ_q	constant associated with order q moving average function (= $1-\theta_1-\theta_2-\ldots.\theta_q$)
ω	constant (= $2\pi/365$ radians)
ω_i	independent random error term

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