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The Marginal Costs of Alternative Levels of Water Quality in The Upper Mississippi River

By

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FOREWORD

This Bulletin is published in furtherance of the purposes of the Water Resources Research Act of 1964. The purpose of the Act is to stimulate, sponsor, provide for, and supplement present programs for the conduct of research, investigations, experiments, and the training of scientists in the field of water and resources which affect water. The Act is promoting a more adequate national program of water resources research by furnishing financial assistance to non-federal research.

The Act provides for establishment of Water Resources Research Institutes or Centers at Universities throughout the Nation. On September 1, 1964, a Water Resources Research Center was established in the Graduate School as an interdisciplinary component of the University of Minnesota. The Center has the responsibility for unifying and stimulating University water resources research through the administration of funds covered in the Act and made available by other sources; coordinating University research with water resources programs of local, State and Federal agencies and private organizations throughout the State; and assisting in training additional scientists for work in the field of water resources through research.

This report is the twenty-fifth in a series of publications designed to present information bearing on water resources research in Minnesota and the results of some of the research sponsored by the Center. In this investigation the research was directed towards providing a better understanding of river pollution problems through the detailed analysis of a particular problem. The metnod of analysis presented has general applicability to other situations where several sources of organic waste are being discharged into a section of a river upon which DO Standards have been set.

The model presented is most useful in cases where several sewage treatment plants are located along a river in such a manner that the DO concentration in the river decreases between successive plants, and where the treatment plants are administered under one management whose primary objective is to maintain a given river standard at a minimum cost. This model can be modified to be applied to various river basin configurations with any number of sources of treated or untreated organic wastes plus any number of tributaries.

The author is indebted to Professor John Waelti who not only guided the development of this project but also gave frequent advise and council throughout the graduate program. The author is grateful to the members of the dissertation committee, Professors Waelti, Fishel, and Allred, for their comments and suggestions. Special thanks is due Professor Walter Fishel for his detailed review of this manuscript.

CHAPTER I

INTRODUCTION

A. Statement of the Problem

The gradual pollution of the Mississippi River in the vicinity of the Twin Cities has become an issue of public concern in recent years. Population and industrial growth in the Twin Cities has not only increased the amount of wastes produced, but has also concentrated these wastes in a small geographic area. The water borne wastes from the metropolitan area are transported by interceptor sewer lines to treatment plants which are located along the Mississippi River. The wastes are treated at these plants and the effluent is discharged into the river. Since the increase in the capacity of these waste treatment facilities has not kept pace with the increase in the amount of liquid wastes, the strength as well as the amount of effluent discharged from these treatment plants has increased. The obvious result has been a steady deterioration of the quality of the Mississippi River water below the Twin Cities.

The use of natural waterways for waste disposal and the subsequent adverse effect on the quality of the water is competitive with downstream water uses. This fact is illustrated by the uses of the Mississippi River above and below the Twin Cities metropolitan area. Upstream from the Twin Cities the water is of sufficient quality for recreational and aesthetic uses as well as for potable water supplies. Downstream from the Twin Cities the river actually becomes an open sewer at critical low flows with the result that recreational and aesthetic activities along with the use of the river for potable water supplies are not only curtailed but are dangerous to human health.

1. Pollution and Contamination

Water pollution refers to the reduction of water quality to the extent that the water is less useful for some particular purpose. Water pollution is not an absolute term but is relative to intended water use. An upstream waste discharger may pollute the river water for one particular downstream use such as swimming but not necessarily for other downstream uses such as navigation. It is therefore essential to define water pollution in terms of present and potential water use.

In contrast to pollution, contamination refers to the impairment of the water to the extent that a health hazard is created. The abatement of contamination is the first step toward water quality management. Therefore in most cases waste discharges with the potential of creating a health hazard are chlorinated to reduce the threat of contamination.

2. Economics of Water Quality Control

It is apparent that society as a whole desires multiple use of water resources. It is also painfully apparent that, in the absence of external constraints, streams and rivers provide a financially inexpensive method of waste disposal. The proximity of streams into which wastes can be discharged conveniently and cheaply is attractive to industries and municipalities. The waste assimilation capacity of running water is an economic resource used by these waste dischargers. However, the economic loss to society resulting from polluted waterways is also significant and represents a cost which is shifted onto society by waste dischargers. The reconciliation of water use for waste assimilation and its use for water supplies, recreational and aesthetic activities, and other uses that require high levels of water quality is a question of allocating limited water supplies among alternative uses.

Most would agree that the price and market system provides an effective means for the efficient allocation of resources. However, in order to achieve an optimal allocation of resources through the market system, the costs and benefits of any decision by an economic entity must be incident on that decision making unit. The essence of the pollution problem is that in the usual process of waste disposal in natural waterways, this condition is not satisfied. The social costs resulting from the water uses foregone due to the deterioration of the water quality are not taken into account by the individual waste dischargers. The discharge of waste into a river creates an external diseconomy which in the absence of corrective policy prevents the achievement of an optimum allocation of water resources.

The basic principles of economics are applicable to the water borne waste disposal situation. Castle analyzed the underlying economics as follows:

> There are essentially three variables relevant here (a) the value of water to the polluter; (b) the damage caused by the discharge of the pollutant; and (c) the cost of treatment prior to causing external damages.
>
> If (a) is greater than (b) or (c) then the use of the water by the polluter is justified economically. If (b) is greater than (c) then inplant treatment is justified. If (c) is greater than (b) then the external costs should be suffered. If (a) is less than (b) or (c) then the use of water is not justified economically.[1]

Given the values of (a), (b) and (c) arising from the marginal units of wastes entering a river, the optimal amount of treatment can be defined by applying Castle's analysis. In effect, this procedure equates the marginal social costs and marginal social benefits of water quality control.

The application of these economic principles to a specific case requires the measurement or estimation of the total and marginal values of the three relevant variables. Significant difficulties arise in the estimation of the value of the damage caused by waste discharges. These include not only the value of any physical damages but also the value of the water uses foregone as a result of a decrease in water quality. Even though the value of water quality is non-marketable and therefore difficult to estimate, it does possess a price in terms of the general well-being of society and therefore must be treated as a decision variable in formulating public policy concerning water quality control.

3. Alternative Institutional Arrangements

Several institutional arrangements have been proposed and implemented to force waste dischargers to account for the external costs of waste disposal. By assigning property rights in such a way that all waste producing units of production in a river basin are socialized or monopolized under one management, the external costs of waste disposal are internalized. This method of organizing units of production is generally rejected in this country as being contradictory to other goals and objectives of our society. However, these methods of organizing production have been applied in other countries and they do have desirable properties with regard to waste disposal problems.

Another system for internalizing the external costs of waste disposal is the imposition of an effluent tax on waste dischargers. For optimal results the amount of tax on each discharge should just equal the social costs imposed by that discharge. As discussed above, the difficulties involved in measuring or estimating these social costs are significant. The political difficulties involved in its establishment and the lack of the cost data necessary to implement it have been effective barriers to the use of the effluent tax system.

The most widespread method employed in this country for forcing waste dischargers to account for the social costs of water quality deterioration is the determination and enforcement of safe minimum standards of pollution in the natural waterways. The safe minimum standard concept is defined by Wantrup as the avoidance of immoderate possible losses thus minimizing the probability of maximum losses.[1] The safe minimum standard in the Mississippi

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[1] Emery Castle, "Economics of Water Pollution Control," Journal of Water Pollution Control Federation, Vol. 38, No. 5 (May, 1966), p. 790.

[1] Ciriacy-Wantrup, Resource Conservation Economics and Policies (California: University of California Press, 1963), p. 88.

River as set by the Minnesota Pollution Control Agency in conjunction with the Federal Water Pollution Control Administration is defined in terms of minimum dissolved oxygen concentrations that must be maintained to assure adequate water quality for intended uses. By choosing the intended uses of various sections of the river and thus the necessary dissolved oxygen concentrations the government agencies are in effect making value judgments for society regarding water quality. In several cases, notably in the Ruhr River Basin in Germany,[1] waste producing units have joined together to treat wastes collectively thus gaining economies of scale in waste treatment, while satisfying the river standard.

The State of Minnesota has recently created a Sewer Service Board by passage of the Metropolitan Sewer Act[2] which will take over the operation of all sewage disposal facilities in the metropolitan area by January 1, 1971. The consequences of this consolidation of independent sewage districts is the subject of this study.

4. The Purpose of This Study

The purpose of this study is to identify the physical, institutional and economic interrelationships that affect and are affected by the quality of the water in the Mississippi River in the Twin Cities area. It is proposed that an indepth examination of this particular case of river pollution will lead not only to conclusions and policy implications pertaining to a specific segment of the Mississippi River, but also to the general understanding of river pollution problems and methodology for analyzing them.

B. The Hypothesis of this Study

The hypothesis tested in this thesis is that a cost minimizing management plan can be devised which will maintain the current dissolved oxygen (DO) river standard in the study area in a manner that is both physically and economically feasible under existing conditions. In order for this management plant to be physically feasible the treatment plants located in the study area must be capable of removing a sufficient amount of the waste load to maintain the DO standard in the river. For the purposes of this study, a management plan is assumed to be economically feasible if the increase in the total annual cost of treatment in the study area does not exceed 10% of the present annual expenditure.

If the hypothesis is accepted, it follows that no additional

[1] Allen Kneese, The Economics of Regional Water Quality Management (Baltimore: Johns Hopkins Press, 1964), Chapter 7.

[2] State of Minnesota, 1969 Session Laws, Chapter 449.

treatment facilities and only moderate increases in annual costs are needed to achieve the DO river standard under existing conditions. The implementation of a management plan for maintaining the DO river standard in the study area would require a reorganization of the institutional structure in order to coordinate the management of the five independent municipal sewage treatment plants in the study area. The Sewer Service Board would accomplish the necessary reorganization by taking over the management of these five treatment plants. If the hypothesis is accepted, then the Sewer Service Board could maintain the DO river standard at a moderate increase in annual treatment expenditure by providing a more efficient allocation of treatment among the five treatment plants.

An additional result of the acceptance of the hypothesis is that minimum cost management plants for maintaining alternative water quality standards can be estimated. In this way the marginal costs of changing the river standard in the study area can be estimated.

The hypothesis can be rejected for one of two reasons. If the present treatment facilities in the study area are not adequate to maintain the DO river standard then the hypothesis will be rejected. This result would support the conclusion that additional treatment capacity is needed to achieve the DO standard. The hypothesis will also be rejected if the increase in the total annual cost of sewage treatment needed to maintain the DO river standard exceeds 10% of the current annual cost. In this case the cost of maintaining the DO river standard would be significantly greater than the current annual cost and the costs of alternative systems for dealing with water quality deterioration would need to be compared to the benefits derived from water quality control.

C. The Objective of this Study

The first objective of this study is to identify the physical, institutional and economic conditions related to water quality which currently exist in the study area. Chapter II presents the physical relationships that are used to predict the waste assimilation capacity of the river and the institutional arrangement under which the waste load is presently regulated. Chapter III reports the results of a treatment cost survey of the five municipal treatment plants in the study area.

The second objective of this study is to propose an analytical framework for combining the physical properties of the river with the costs of treatment to obtain minimum cost strategies for attaining specified river standards. This is done under the assumption that all five treatment plants are subject to a central authority whose objective is to maintain the river standard at a minimum cost. A model to estimate these minimum cost management schemes is presented in Chapter IV.

A third objective of this study is to determine the highest possible DO river standard that is physically feasible in the study area with existing treatment facilities during summer low flows. This solution was obtained from the predicted capacity of this segment of the Mississippi River for waste assimilation and is presented in Chapter IV.

The final objective of the study is to test the hypothesis that a minimum cost management scheme can be found which will maintain the current DO river standard in the study area in a manner that is both physically and economically feasible under existing conditions. This objective is accomplished by presenting a model which combines the predicted waste assimilation capacity of the river with the results of the cost survey, and then solving this model for the least cost management scheme for achieving the current river standard in the study area. If a physically and economically feasible solution is obtained then the hypothesis will be accepted, otherwise it will be rejected. The result of this test is given in Chapter V.

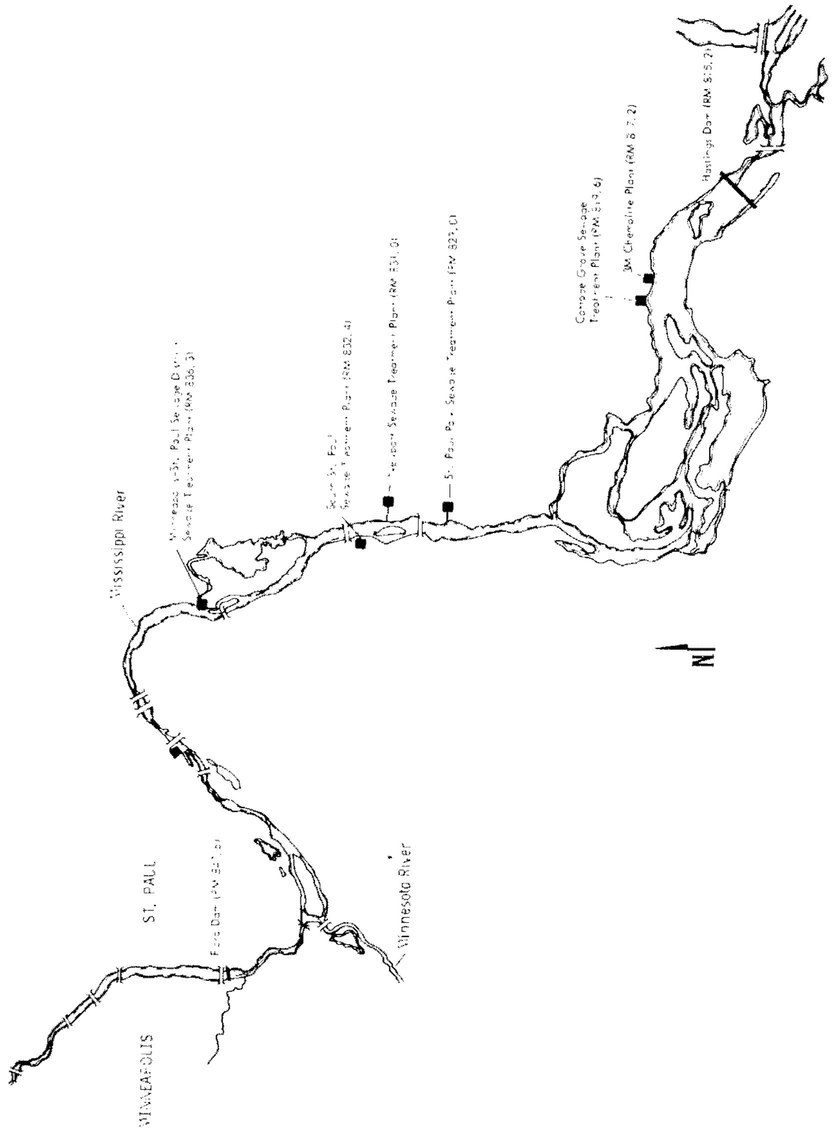

FIGURE 1. A MAP OF THE STUDY AREA

CHAPTER II

THE UNDERLYING PHYSICAL AND INSTITUTIONAL SETTING

A. The Study Area

The study area was defined to include that portion of the Mississippi River in and below the Twin Cities Metropolitan area with the most severe pollution as measured by the dissolved oxygen concentration. This area is located between Lock and Dam Number 1 (Ford Dam) at river mile 847.6[1] and Lock and Dam Number 2 (Hastings Dam) at river mile 815.2 (Figure 1). The lowest dissolved oxygen concentration occurs most frequently between the Minneapolis-St. Paul Sanitary District (MSSD) Sewage Treatment Plant at river mile 836.3 (Pig's Eye Island) and Lock and Dam Number 2. The beginning of the study area is far enough upstream to include the major sources of organic wastes. The end of the study area was designated at the Hastings Dam since the reaeration[2] of the water occurring at this dam is sufficient to raise the dissolved oxygen concentration up to acceptable levels, even when the dissolved oxygen concentration is zero immediately upstream from the dam.

Of the seven principal organic waste sources (Table 1), five are municipal sewage treatment plants. After primary and secondary treatment[3] the effluents from these plants are discharged into the river. The Mississippi River contains some organic wastes as it flows into the study area as does the Minnesota River. The Minnesota Mining and Manufacturing Company Chemolite Plant is the only other significant source of organic waste in the study area.

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[1] River mile zero is located at the confluence of the Ohio and Mississippi Rivers.

[2] The significance of reaeration is discussed more fully in Section D.

[3] Primary treatment removes 35% to 40% of the organic waste load. The combination of primary and secondary treatment removes between 40% and 90% of the organic waste load. See Louis Klein, River Pollution III. Control (Washington: Butterworths, 1966), Chapter 3.

TABLE 1. LOCATION OF WASTE SOURCES IN THE STUDY AREA

Waste Source	River Mile
Incoming Mississippi River (Ford Dam)	847.6
Minnesota River	844.2
Minneapolis-St. Paul Sewage Treatment Plant	836.3
South St. Paul Sewage Treatment Plant	832.4
Newport Sewage Treatment Plant	831.0
St. Paul Park Sewage Treatment Plant	829.0
Cottage Grove Sewage Treatment Plant	819.6
Minnesota Mining and Manufacturing Chemolite Plant	817.2
Hastings Dam	815.2

Source: "Memorandum on the Waste Assimilation Capacity of the Mississippi River in the Twin Cities Metropolitan Area" (Minneapolis: Minnesota Pollution Control Agency, March 27, 1969), p. 15.

B. The Institutional Setting

1. The Federal Government

The Federal Water Pollution Control Act of 1956 is the basic Federal legislation concerning water quality control.[1] This act was amended in 1961, again in 1965 and in 1966. The Water Quality Act of 1965 created the Federal Water Pollution Control Administration within the Department of Health, Education and Welfare. In 1966 the Federal Water Pollution Control Administration was transferred to the Department of the Interior.

The Water Quality Act of 1965 contained a provision that required the states to develop water quality standards for all interstate waters and to submit a plan for implementation and enforcement of these standards. If these requirements were not met by June 1967, the Federal Water Pollution Control Administration would set and enforce the necessary standards.

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[1] John J. Waelti, "Understanding the Water Quality Controversy in Minnesota," Extension Bulletin 359 (University of Minnesota: Agricultural Extension Service, July 1969).

2. The State Government

Water pollution control legislation in Minnesota dates back to 1885.[1] This early legislation was primarily concerned with the contamination of potable water supplies. As the pollution problem became more acute, state legislators gradually increased the power and scope of the pollution control authorities. The Minnesota Pollution Control Agency, which evolved from a succession of prior agencies, was created in 1967, and presently consists of nine part-time policy makers appointed by the Governor and a full-time professional staff. This agency not only has jurisdiction over water quality, but also over solid waste disposal and air quality. In addition to setting and enforcing water quality standards, the agency's staff keeps abreast of the latest technology in waste treatment, advises waste dischargers of the most economical methods of pollution control, and processes local applications for federal grants for waste treatment facilities.

The Water Pollution Control Commission, which preceeded the Minnesota Pollution Control Agency, began working on a general framework of water classification and quality standards in 1963. The project was completed in 1966 and was subjected to five public hearings. The classification scheme and quality standards for all interstate waters were adopted in March of 1967, meeting the deadline set by the Federal Water Pollution Control Administration. These interstate standards were approved by the Secretary of the Interior on June 18, 1968. Subsequently, the Minnesota Pollution Control Agency has established water quality criteria and standards for all intrastate waters.

3. The Metropolitan Government

At the local level of government organization, the Metropolitan Council grew out of the Metropolitan Planning Commission which was created by the state legislature in 1957. The primary job of the Metropolitan Planning Commission was to complete a Metropolitan Development Guide, which was accomplished by 1966. As a result of the Guide, the Metropolitan Planning Commission proposed the establishment of a metro agency with broad planning and development powers to guide development. The Metropolitan Council was established for that purpose by the state legislature in 1967[2].

In 1969 the state legislature passed the Metropolitan Sewer Act[3]

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[1] Bob Ray (ed.), "Your Minnesota Pollution Control Agency" (Minneapolis: Minnesota Pollution Control Agency), p. 3.

[2] State of Minnesota, 1967 Session Laws, Chapter 896.

[3] State of Minnesota, 1969 Session Laws, Chapter 449.

creating the Sewer Service Board under the Metropolitan Council. This organization was charged with planning, operating, and maintaining all the municipal sewage treatment facilities in the Metropolitan area, to be effected by January, 1971. This board will, in effect, consolidate all the sewage treatment plants in the study area and manage them as one sewer system.

C. Classification and Sources of Pollutants[1]

1. Nondegradable Pollutants

Pollutants may be classified according to the effect they have on receiving water and, consequently, on the downstream water use. Nondegradable pollutants include stable chemical compounds and physical pollutants which are not altered by the natural biological processes in running water. These substances are diluted in the water but are not changed in form or reduced in quantity.

Nondegradable pollutants enter the river primarily from industrial effluents and agricultural runoff. These sources commonly contain nondegradable inorganic chemicals such as chlorides, nitrates, phosphates, sulfates, various metallic salts and radioactive materials. Common organic nondegradables include DDT, 2,4-D, hard detergents, resins, coal, tar and dyes. Many physical pollutants such as silt and suspended matter are also nondegradable.

2. Degradable Pollutants

Degradable or biodegradable pollutants are waste materials that are changed in form and/or reduced in quantity by the biological process that occurs in natural waterways. The principal sources of these substances are industrial and municipal effluents. Recently, however, a number of biodegradable agricultural chemicals have been developed.

Nearly all biological pollutants are degradable. Many chemical pollutants, especially organics including oil, grease, soft degergents, organic nitrogen nitrites, ammonia, phenols, and cyanides, are also biodegradable.

D. Degradable Wastes and Dissolved Oxygen

1. Self-Purification

When degradable pollutants are introduced into the ecological

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[1] Sections C and D draw from the following sources: Louis Klein, River Pollution I. Chemical Analysis (London: Butterworths, 1959), River Pollution II. Causes and Effects (London: Butterworths, 1962), and River Pollution III. Control (Washington: Butterworths, 1966); and Allen Kneese, The Economics of Regional Water Quality Management (Baltimore: Johns Hopkins Press, 1964).

system of a river, the micro-organisms, mainly bacteria, feed on the substance and through their digestive system chemically oxidize the pollutant into stable inorganic bicarbonates, nitrates, sulfates, phosphates, and other end products. The bacteria actually break down the unstable complex waste materials into stable end products. This natural phenomenon is termed self-purification and will occur spontaneously in rivers provided the amount of waste does not exceed the waste assimilation capacity of the stream. The waste assimilation capacity of a river is a function of physical and biological phenomena such as the amount of waste products in the water, the amount and velocity of flow, the depth, the turbulence, and the concentration of dissolved oxygen in the water.

2. Dissolved Oxygen

Several biological characteristics of rivers are significant in measuring the extent of river pollution. These characteristics include concentration of suspended solids, concentration of volatile solids, total coliform density, fecal coliform density, total phytoplankton density, organism data on bottom sediments, turbidity, concentration of biochemical oxygen demand (BOD) and dissolved oxygen (DO) concentration. Of these measures of pollution the most useful and generally accepted criterion for quantifying pollution from organic or oxygen demanding wastes is in terms of BOD and the usual measurement of the extent of organic pollution in a river is the DO concentration.

If the receiving water contains sufficient dissolved oxygen the bacteria will utilize this free oxygen and the self-purification process will proceed aerobically. Aerobic self-purification is beneficial since only harmless end products are produced. However, if the water contains no DO -- the waste load has exceeded the capacity of the stream for waste assimilation to the extent that all the DO has been utilized -- bacteria that are capable of anaerobic digestion will take over and as a result noxious odors such as hydrogen sulfide will be produced as by-products. Under these conditions the waterway is septic.

The rate at which oxygen is demanded by the bacteria in the stream (for the breakdown of organic wastes) is known as the biochemical oxygen demand (BOD). The BOD occurs in several stages, rapidly for the first five days then slower for the next 15-20 days and then faster again for the next five days. The standard BOD calculations are based on the amount of oxygen demanded in a five-day period. With time understood to be five days, the BOD is stated as an amount rather than as a rate. This quantity can be calculated for each oxygen demanding waste load and is dependent on the physical properties of the waste.

Under normal conditions a free flowing river obtains DO from reaeration and from photosynthesis occurring in aquatic plant life. Reaeration occurs at the interface of the water surface and the atmosphere. The water actually absorbs free oxygen molecules from the atmosphere. The rate of reaeration depends on the velocity of flow, the amount of surface area, the temperature, the depth and the other physical characteristics of the stream. The presence of turbulent water or water falls increases the rate of reaeration significantly. The amount of oxygen absorbed from reaeration in a given period of time far exceeds the amount gained from photosynthesis. For the practical purposes of this study, the photosynthesis effect can be ignored.

It should be recognized that the waste assimilation capacity of a river is a scarce resource which must be allocated among competitive uses. The BOD and DO concentrations are measures of the amount of this resource that is being consumed by water users along a river.

3. The Dissolved Oxygen Sag Curve

Both the removal of DO by aerobic decomposition and the reaeration of a river are functions of time. These two rates, the reaeration rate and the deoxygenation rate, occur simultaneously downstream from an oxygen demanding waste source. In the normal situation, the deoxygenation rate will exceed the reaeration rate initially and the DO concentration in the stream will decrease. As the waste products are oxidized by the micro-organisms the deoxygenation rate will decrease. Eventually at some downstream location the deoxygenation rate will become less than the reaeration rate and the DO concentration in the stream will increase. If the physical conditions of the river are uniform, the reaeration rate will remain constant throughout the self-purification process.

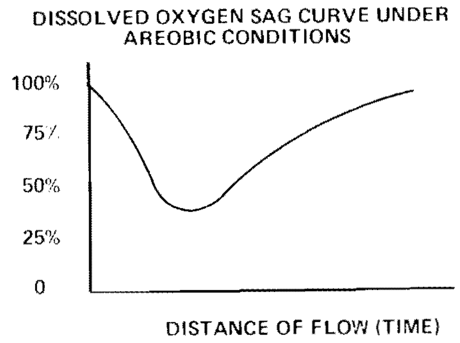
This combination of the deoxygenation rate and the reaeration rate produces a DO "sag curve," which illustrates the changes in the DO concentration downstream from an oxygen demanding waste source (Figure 2).

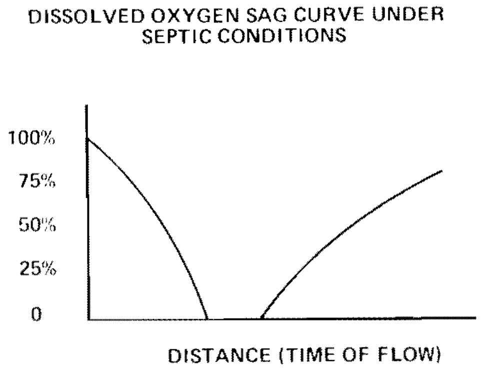
4. Dissolved Oxygen Requirements

Generally the more desirable forms of aquatic life in rivers require higher levels of DO than the less desirable forms of aquatic life. It is possible to identify levels of DO that are necessary for the survival of different species of aquatic life, and thus the desirability of the river for various commercial, recreational and aesthetic uses.

A minimum of five milligrams of DO per liter of water must be maintained for the survival of pollution sensitive aquatic life including Group I fish as defined by the Federal Water Pollution Control Administration.

> Group I Fish - Are those generally sought after by sport fishermen and include but are not limited to the following species: Walleyed Pike, Northern Pike,





SOURCE: JOHN J. WAELTI, "UNDERSTANDING THE WATER QUALITY CONTROVERSY IN MINNESOTA," EXTENSION BULLETIN 359 (UNIVERSITY OF MINNESOTA: AGRICULTURAL EXTENSION SERVICE, JULY 1969).

FIGURE 2. DISSOLVED OXYGEN SAG CURVES

Sauger, Black Crappie, White Crappie, Largemouth Bass, Smallmouth Bass, Rock Bass, White Bass, Bluegill, Channel Catfish, Sturgeon, Flathead Catfish, Green Sunfish, Pumpkinseed Sunfish, and Brown Trout.[1]

A minimum of three milligrams of DO per liter of water must be maintained for the survival of pollution tolerant species of aquatic life including Group II fish as defined by the Federal Water Pollution Control Administration.

Group II Fish - Are those generally sought after by commercial fishermen in this area and include but are not limited to the following species: Carp, Quillback, Sheepshead, Brown Bullhead, Bigmouth Buffalo, Northern Carpsucker, Northern Redhorse, Longnose Gar, Shortnose Gar, Bowfin, Mooneye, Gizzard Shad, Common Sucker, Spotted Sucker, Yellow Bullhead, Black Bullhead, Golden Shiner, Perch, and River Sucker.[2]

A zero level of DO produces septic conditions in the stream and the only species that can survive are bacteria which are capable of anaerobic digestion. Septic conditions reduce the desirability of the stream for recreational and aesthetic uses because of the noxious odors and the elimination of the fish population.

E. Approximation of the Dissolved Oxygen Sag Curve

Various mathematical approximations of the DO sag curve have been proposed. The most widely accepted approximation was first conceived by Streeter and Phelps[3] in 1925. Based on this earlier work Camp[4] and Dobbins[5] developed two differential equations which

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[1] A Report on Pollution of the Upper Mississippi River and Major Tributaries (Washington: United States Department of the Interior, Federal Water Pollution Control Administration, July, 1966), p. VI-12.

[2] Ibid.

[3] H. W. Streeter and E. B. Phelps, "Study of the Pollution and Natural Purification of the Ohio River," Public Health Bulletin 146 (Washington: U. S. Public Health Service, February, 1925).

[4] T. R. Camp, Water and Its Impurities (New York: Reinhold Publishing Company, 1963).

[5] W. E. Dobbins, "BOD and Oxygen Relationships in Streams," Journal of the Sanitary Engineering Division, Vol. 90, No. 3 (June, 1964).

describe the process and predict the DO concentration at any point in a flowing river under certain assumptions.

The first differential equation states that the rate of change of BOD with respect to time, is proportional to the initial amount of BOD.[1]

(1) $$\frac{dL}{dt} = -kL_o$$

where $\frac{dL}{dt}$ = the change in BOD concentration per unit change in time in mg/l per day

L_o = initial concentration of BOD in the stream (just below waste source) in mg/l

k = deoxygenation rate constant, given by $(\text{days})^{-1}$

t = time in days.

The integration of equation (1) over time yeilds:[2]

(2) $$L_t = L_o 10^{-kt}$$

where L_t = BOD concentration in the stream at time t in mg/l.

The second differential equation used to predict the DO sag curve states that the rate of change of the DO deficit concentration with respect to time is a function of deoxygenation and reaeration, which are occurring simultaneously. The DO deficit concentration at any time "t" is the difference between the DO concentration at time "t" and the DO concentration at 100% saturation.[3]

(3) $$\frac{dD}{dt} = kL_o - rD_o$$

where $\frac{dD}{dt}$ = the change in the dissolved oxygen deficit concentration per unit change in time in mg/l per day

[1] Louis Klein, River Pollution II. Causes and Effects (London: Butterworths, 1962), p. 226.

[2] Ibid., p. 226.

[3] A Report on Pollution of the Upper Mississippi River and Major Tributaries (Washington: United States Department of the Interior, Federal Water Pollution Control Administration, July, 1966), p. V-2A.

D_o = initial dissolved oxygen deficit concentration in the stream (just above waste discharge) in mg/l

r = reaeration rate constant, given by $(\text{days})^{-1}$.

The integration of equation (3) over time yields:[1]

(4) $$D_t = \frac{k}{r-k}\left(10^{-kt} - 10^{-rt}\right) L_o + D_o 10^{-rt}$$

where D_t = dissolved oxygen deficit concentration in the stream at time "t" in mg/l.

Equation (4) can be used to predict the DO deficit concentration at any point "t" downstream from a waste discharge. Additional factors such as photosynthesis, second stage BOD, and BOD from accumulated bottom deposits can be incorporated into equation (3) with the resulting integrated form yielding an equation similar to equation (4) but with additional terms to account for these additional effects.

As previously indicated, BOD is a measure of the rate of oxygen utilization with the time period understood to be five days. "Ultimate" or "total" BOD is an indicator of the total oxygen demand. It is used for the DO sag curve predictions and is calculated by the following equation derived from equation (2)[2].

(5) $$L_u = \frac{L_5}{1 - 10^{-5k}}$$

where L_u = ultimate BOD concentration in mg/l

L_5 = five day BOD concentration in mg/l

The temperature of the waste receiving water affects the DO sag curve in two ways. First, the values of the reaeration and deoxygenation rate constants vary proportionately with temperature. Second, the saturation concentration of DO varies inversely with the temperature.

The temperature effect on the deoxygenation and reaeration rate constants is expressed in equations (6) and (7).[3]

(6) $$k_{T_2} = k_{T_1}\,\theta^{(T_2 - T_1)}$$

[1] Ibid., p. V-2A.

[2] Louis Klein, River Pollution II. Causes and Effects (London: Butterworths, 1962), p. 226.

[3] B. E. Babbit and R. E. Baumann, Sewerage and Sewage Treatment (New York: John Wiley and Sons, 1958), p. 348.

$$(7) \qquad r_{T_2} = r_{T_1} \Phi^{(T_2 - T_1)}$$

where T = temperature in degrees centigrade

k_{T_1} = deoxygenation rate constant at temperature "T_1"

k_{T_2} = deoxygenation rate constant at temperature "T_2"

r_{T_1} = reaeration rate constant at temperature "T_1"

r_{T_2} = reaeration rate constant at temperature "T_2"

Θ = deoxygenation temperature adjustment coefficient, equal to 1.047 during the summer and 1.110 during the winter[1]

Φ = reaeration temperature adjustment coefficient, equal to 1.016[2]

The relationships between the temperature of the waste receiving water and the DO saturation concentration is given in Table 2.

The following assumptions concerning the physical properties of the wastes and the receiving river are necessary in order to apply equation (4) to the current situation in the study area:

1) BOD from runoff, scour, second stage BOD and bottom deposits is assumed to be negligible

2) The temperature within each reach of the river is assumed constant

3) The effluents are assumed to have zero DO concentrations

4) Complete mixing is assumed at the points where effluents enter the river.

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[1] These rates are reported by the Federal Water Pollution Control Administration in <u>A Report on Pollution of the Upper Mississippi River and Major Tributaries</u> (Washington: Department of the Interior, Federal Water Pollution Control Administration, July, 1966), p. V-4A.

[2] <u>Ibid</u>., p. V-4A.

TABLE 2. SOLUBILITY OF OXYGEN IN FRESH WATER UNDER AN ATMOSPHERIC PRESSURE OF 760 M.M. OF MERCURY THE DRY ATMOSPHERE CONTAINING 20.9% OXYGEN

Temperature in Degrees Centigrade	Saturation Concentration in mg/l
0	14.62
5	12.37
10	10.92
15	9.76
20	8.84
25	8.11
30	7.53

Source: B. E. Babbit and R. E. Bauman, <u>Sewerage and Sewage Treatment</u> (New York: John Wiley and Sons, 1958), p. 345.

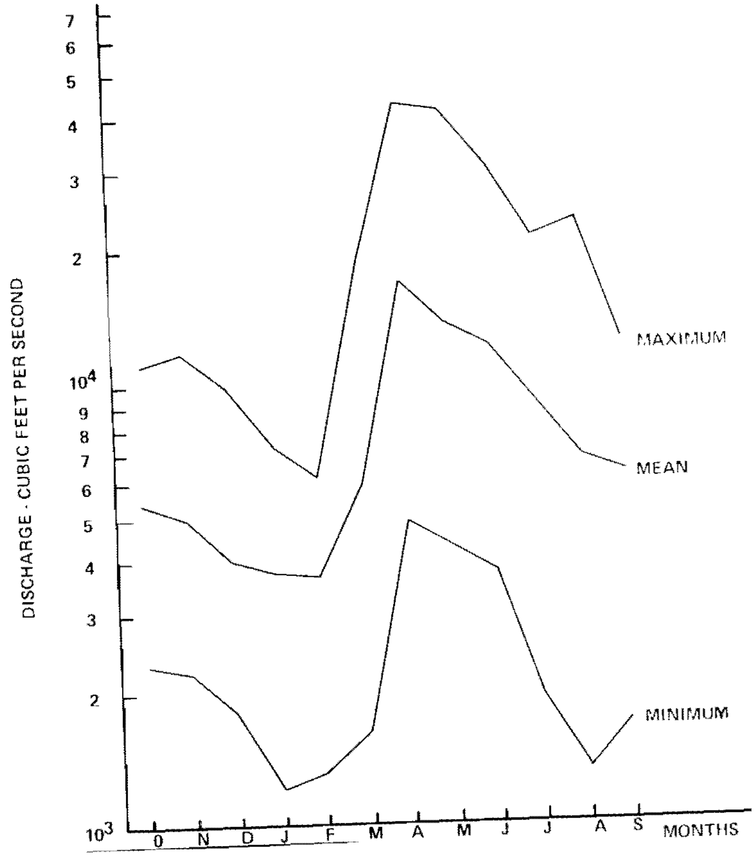
F. The Physical Properties of the Mississippi River

As illustrated in Figure 3, the flow of the Mississippi River in the study area follows a seasonal pattern with one period of low flow occurring in July, August, and September and another low flow period in December, January, and February.

As shown in Table 2 warmer water has a lower saturation concentration of DO. In addition the bacterial action proceeds faster in warmer water thus utilizing DO at a faster rate. If the same amount of oxygen demanding wastes are entering the river in summer and winter, the above physical properties cause the minimum DO concentration to be significantly lower in the summer, which results in the lowest DO concentrations occurring during the summer low flow period.

In order to evaluate the capacity of a river for waste assimilation, it is necessary to determine a design flow on which to base the calculations. It is desirable to use a design flow with a low probability of occurrence to insure that critical levels of DO will not prevail frequently or for long periods of time. The design flow used most often to accomplish this result is the <u>seven consecutive day summer low flow that occurs once in ten years</u> (SCD). The SCD flow is used to define the current river standard in the study area.

In 1965 the Federal Water Pollution Control Administration



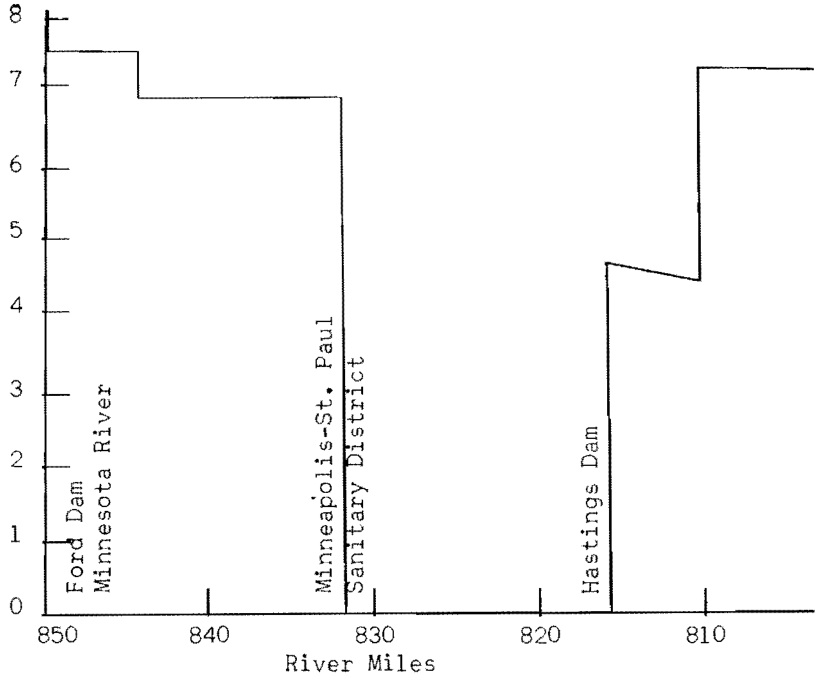
SOURCE: A REPORT ON POLLUTION OF THE UPPER MISSISSIPPI RIVER AND MAJOR TRIBUTARIES, (WASHINGTON, D.C.: UNITED STATES DEPARTMENT OF THE INTERIOR, FEDERAL WATER POLLUTION CONTROL ADMINISTRATION, JULY 1966) FIGURE V-1.

FIGURE 3. MAXIMUM, MEAN AND MINIMUM MONTHLY DISCHARGES OF THE MISSISSIPPI RIVER AT LOCK AND DAM NUMBER 1

reported the results of a detailed study of the physical conditions of the Upper Mississippi and Minnesota Rivers.[1] Using existing data on the discharges of the Mississippi River at Lock and Dam Number 1 and the Minnesota River near Carver, which covered a period from October, 1940 through September, 1964, the agency computed the SCD flow for each of these rivers. These flows are given in the Incoming River Conditions section of Appendix B.

In another Federal Water Pollution Control Administration study conducted in 1966 the DO sag curve for the Mississippi River from river mile 875 down to river mile 765 was predicted using the DO deficit and BOD equations (1) through (7), the resulting curve is shown in Figure 4.

FIGURE 4. THE PREDICTED DISSOLVED OXYGEN SAG CURVE AT THE SCD FLOW IN THE MISSISSIPPI RIVER



Source: A Report on Pollution of the Upper Mississippi River and Major Tributaries (Washington, D.C.: United States Department of the Interior, Federal Water Pollution Control Administration, July, 1966), Figure V-36.

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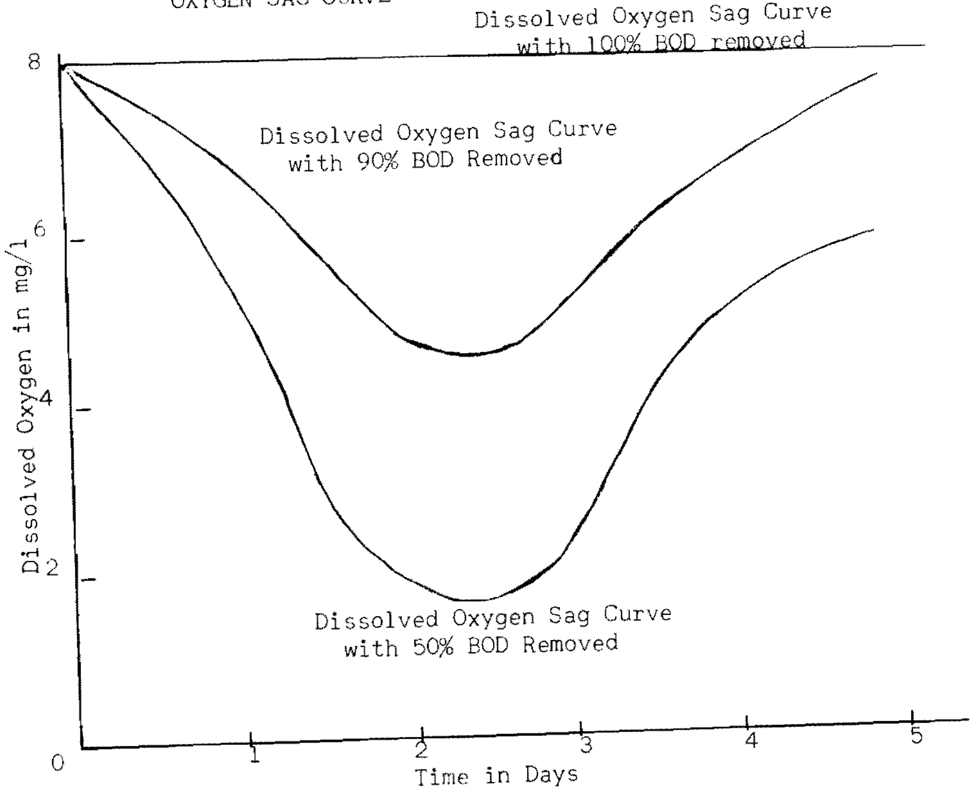
[1] Report on Hydrographic Studies of the Mississippi, Minnesota and St. Croix Rivers (Washington: United States Department of Health, Education and Welfare, Federal Water Pollution Control Administration, December, 1965).

Figure 4 represents the DO situation in the Mississippi River which would occur at the SCD design flow with 1964-65 waste loadings. The only significant change in the waste loading from 1964-65 to 1970 has been an increase in the percent BOD removed by the MSSD plant from 31% to 48.8%. This decrease in waste loading into the river does not raise the DO sag curve in Figure 4 appreciably, and septic conditions still prevail for a considerable segment of the river at the SCD flow. Thus, under present conditions the treatment facilities discharging into the Mississippi River from Lock and Dam Number 1 down to the Hastings Dam are discharging more BOD into the river than the river can assimilate at the SCD design flow without the DO concentration dropping below the current standard of 3 mg/l.

G. The Underlying Economic Relationships

The dissolved oxygen sag curve predictive equations can be viewed as the underlying physical relationship analogous to the production function in microeconomic theory. Given a situation with one treatment plant discharging its effluent into a stream, it is possible to relate the minimum level of dissolved oxygen occurring in the stream with the amount of BOD discharged from the plant. The amount of BOD discharged varies inversely with the amount of BOD removed by the plant, as illustrated in Figure 5.

FIGURE 5. THE EFFECT OF THE PERCENT BOD REMOVED ON THE DISSOLVED OXYGEN SAG CURVE



The functional relationship between the percent of BOD removed by a sewage treatment plant and the minimum DO concentration occurring downstream is similar to the traditional static production function with output defined as the minimum DO concentration and the inputs combined into one composite input which is the percent BOD removed by the plant. This quasi-production function (Figure 6) does not conform exactly to the usual case since the marginal cost of the aggregate input is not constant but varies through the domain of the function. The total annual cost of the composite input for a typical sewage treatment plant, developed more fully in Chapter III, takes the form shown in Figure 7.

The dissolved oxygen sag predictive equation (4) can be used to specify the quasi-production function of the typical sewage treatment plant. It is first noted that the low point of the DO sag curve occurs at some distance downstream from the plant. In the formulation of the predictive DO deficit equation, distance downstream is measured in time of flow.[1] Therefore, equation (4) can be rewritten as:

(8) $$D_{t_o} = \frac{k}{r-k}\left(10^{-kt_o} - 10^{-rt_o}\right) L_o + D_o 10^{-rt_o}$$

where D_{t_o} = the dissolved oxygen deficit concentration in the river at the low point of the dissolved oxygen sag curve in mg/l

t_o = the time of river flow from the BOD source to the low point of the dissolved oxygen sag curve in days.

The concentration of BOD below the outfall of the sewage treatment plant (L_o) is determined by the concentration of BOD and the quantity of flow of the river just above the outfall and the concentration of BOD and the quantity of flow being discharged into the river. This relationship is expressed as:

(9) $$L_o = \frac{Q_o F + Q_1 M}{Q_o + Q_1}$$

where Q_o = the flow of the river just above the outfall of the plant in liters/second

Q_1 = the flow of the effluent from the plant in liters/second

[1] At the SCD flow the river would flow approximately 9.5 miles/day in the vicinity of the Ford Dam. The velocity of flow would decrease as the water moved downstream and it would flow at approximately 1.8 miles/day near the Hastings Dam.

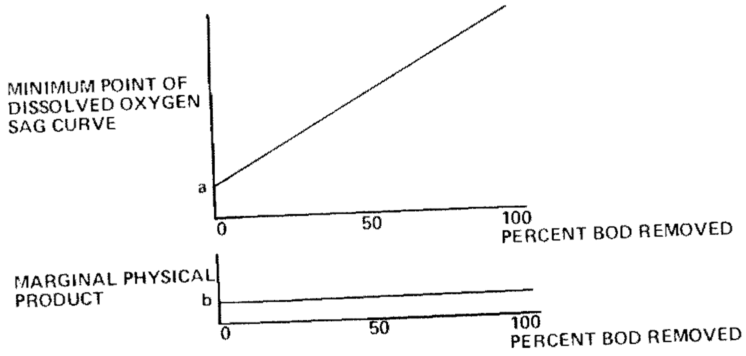
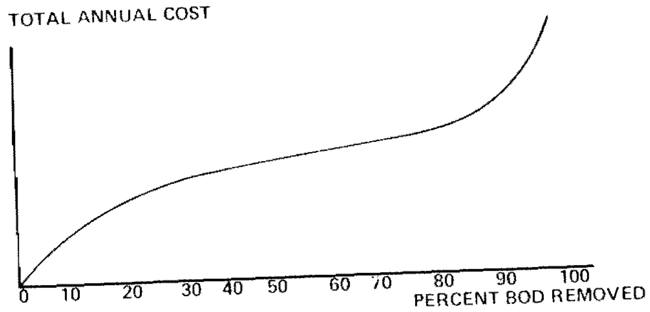


FIGURE 6. THE QUASI-PRODUCTION FUNCTION AND MARGINAL PHYSICAL PRODUCT OF A TYPICAL SEWAGE TREATMENT PLANT



SOURCE: C. S. REVELLE, D. P. LOUCKS AND W. R. LYNN, "A MANAGEMENT MODEL FOR WATER QUALITY CONTROL," JOURNAL WATER POLLUTION CONTROL FEDERATION, VOL.39, NO. 7, (JULY 1967), P. 1169.

FIGURE 7. THE TOTAL ANNUAL COST CURVE OF A TYPICAL SEWAGE TREATMENT PLANT

F = the concentration of BOD in the river just upstream from the outfall in mg/l

M = the concentration of BOD in the effluent from the plant in mg/l.

All of the quantities in equation (9) are exogenous to the individual sewage treatment plant except the concentration of BOD in the effluent of the plant (M). The concentration of BOD in the effluent of the plant is related directly to the percentage of BOD removed by the plant. The functional relationship is:

$$E = \frac{P-M}{P} \times 100 \qquad (10)$$

where E = the percent of the BOD in the influent that is removed by the plant.

P = the concentration of the BOD in the influent of the plant in mg/l.

By solving equation (10) for (M) and substituting the result into equation (9) the concentration of BOD in the river just below the outfall of the plant becomes a linear function of the percent of the BOD removed by the plant. Similarly by substituting this expression of (L_o) into equation (8), the low point of dissolved oxygen sag curve (D_{t_o}) becomes a linear function of the percent of the BOD removed by the plant (E). This relationship is:

$$D_{t_o} = a - bE \qquad (11)$$

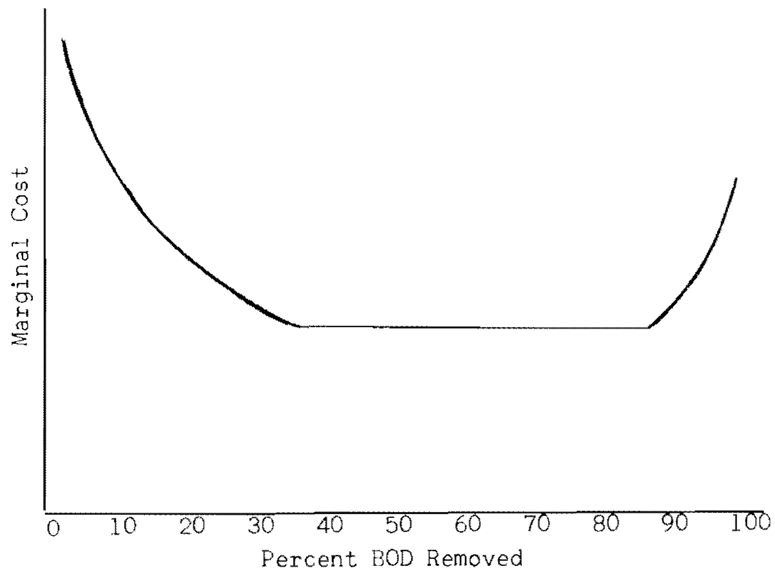
where
$$a = \frac{k}{r-k}\left(10^{-kt_o} - 10^{-rt_o}\right)\left[\frac{Q_oF + Q_1P}{Q_o + Q_1}\right] + D_o 10^{-rt_o}$$

$$b = \frac{k}{r-k}\left(10^{-kt_o} - 10^{-rt_o}\right)\left[\frac{PQ_1}{(Q_o + Q_1)\ 100}\right]$$

All the quantities in the expressions of "a" and "b" are exogenous to the individual treatment plant and, therefore, are constants. The quasi-production function in equation (11) is thus a linear relationship. As a result of the linearity of the quasi-production function, the marginal physical product (b) of the composite input (E) is constant over the entire domain of the function (Figure 6).

The marginal cost of a one unit rise in the minimum DO concentration is computed by multiplying the reciprocal of the marginal physical product by cost or price of an additional unit of input.[1] Since the marginal physical product for the plant is constant throughout the domain and the cost or price of the composite input is constant from 35% to 90% BOD removed, the marginal cost of an additional unit of output is constant between 35% and 90% BOD removed. As shown in Figure 7 between 0% and 35% BOD removed the marginal cost of a typical sewage treatment plant is decreasing as the percent BOD removed increases, and between 90% and 100% BOD removed the marginal cost is increasing as the percent BOD removed increases. The marginal cost curve for a typical sewage treatment plant with primary and secondary treatment facilities is shown in Figure 8.

FIGURE 8. THE MARGINAL COST FUNCTION OF A TYPICAL SEWAGE TREATMENT PLANT WITH PRIMARY AND SECONDARY TREATMENT



In the case of more than one sewage treatment plant located along a river in such a way that the dissolved oxygen concentration in the river decreases between any two plants and the low point of the dissolved oxygen sag curve occurs downstream from the plants, all of the plants are producing the same output which is the minimum level of dissolved oxygen occurring downstream. From

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1

$$MC = \frac{1}{b} P$$

where MC = the marginal cost of a one unit rise in the minimum DO concentration in the river

b = the marginal physical product of the treatment plant

P = the price of an additional unit of input.

equation (11) it is clear that the marginal physical product (b) of any particular plant is constant. However, the intercept value (a) is dependent of the amount of BOD removed by the upstream plants.[1] The cost of additional units of the composite input for any particular plant is also constant in the range between 35% and 90% BOD removed. Therefore, the marginal cost of a one unit rise in the minimum dissolved oxygen level is also constant, for any individual sewage treatment facility.

In order to determine the least cost strategy for raising the minimum dissolved oxygen concentration occurring downstream in the above situation, the marginal cost of a one unit rise in the dissolved oxygen sag curve would be computed and ranked for each plant. To increase the minimum dissolved oxygen concentration with the least increase in cost, the percent BOD removed by the plant with the lowest marginal cost would be increased. When this plant is treating to its physical maximum of 90% BOD removed and the minimum dissolved oxygen concentration is to be increased further, the percent BOD removed by the plant with the second lowest marginal cost would be increased. This procedure would be continued until the highest possible minimum dissolved oxygen concentration would be attained. This would occur when each of the plants in the series was removing 90% of the BOD in its influent.

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[1] If an upstream plant increases its percent BOD removed the intercept values for all downstream plants will increase and vice versa.

CHAPTER III

THE COSTS OF WASTE TREATMENT

A. Introduction

Given the increase in the water borne wastes that are generated in the Twin Cities metropolitan area, it is obvious that some increase in waste reduction must be employed if the Mississippi River is not destined to become an open sewer. Numerous processes and methods for reducing the organic waste load in water borne wastes are available for use by municipal sewage treatment plants.[1] This analysis deals with the current situation in the study area and does not postulate any changes in the number, location or treatment methods of the five plants in the study area. The independent variable of primary interest here is the percent BOD removed by each of the five municipal treatment plants.

At the present time there are five independently operated municipal treatment plants located in and below the Twin Cities which are in the reach of the Mississippi River that is affected by the water borne wastes from the metropolitan area. These plants receive domestic and industrial waste water from interceptor sewer lines, treat the waste water to reduce the concentration of pollutants, and then discharge the treated water into the Mississippi River. The treatment process used by each of these plants is listed in Table 3.

TABLE 3. THE TREATMENT PROCESSES USED BY THE MUNICIPAL TREATMENT PLANTS IN THE STUDY AREA

Plant	Treatment Process	Type of Treatment
MSSDA[a]	Activated sludge including sludge incineration	Primary and secondary
South St. Paul	Trickling filters, anaerobic stabilization pond and sludge incineration	Primary and secondary
Newport	Contact stabilization	Primary and secondary
St. Paul Park	High-rate trickling filter	Primary and secondary
Cottage Grove	Activated sludge	Primary and secondary

[a] Minneapolis-St. Paul Sanitary District.

Source: Sewage and Water Planning Report (Metclaf and Eddy, November, 1968), pp. 41-42.

[1] See Kneese, Chapter 2.

In addition to the municipal sewage treatment plants there are several sources of industrial wastes which enter the Mississippi River in the study area. Of these sources only one, the Minnesota Mining and Manufacturing Company Chemolite Plant, is a significant contributor of BOD. The Northwestern Refining Company, St. Paul Ammonia Products Company, Great Northern Oil Company and the Cenex Corporation[1] also discharge small quantities of BOD into this section of the river, but these wastes loads were not considered in this study because of their relatively small effect on the DO concentration in the river. The Chemolite plant discharges 5.76 MGD[2] of waste water which contains 120 mg/1 of five day BOD.[3]

The maintenance of water quality via waste water treatment requires large capital investments in treatment facilities in addition to annual operating, administrative, and maintenance expenditures. In order to make rational decisions concerning the desired level of water quality, the decision maker must know the costs of achieving, and the environmental effects of alternative water quality standards.

This chapter is primarily concerned with the estimation of total annual costs (including the amortized capital outlays) of the five municipal treatment plants in the study area as a function of their percent BOD removed. These estimated cost functions are combined with the physical data on the assimilation capacity of the Mississippi River in the study area to determine the costs of alternative levels of water quality given in terms of minimum dissolved oxygen standards.

B. Financing Waste Treatment Expenditures

The capital costs of municipal sewage treatment plants are financed with revenue bonds which are normally issued for 20 to 25 year repayment period. The life of conventional sewage treatment facilities is usually considered to be 40 years. Including obsolescence the life is normally considered to be about 25 years but in some rapidly growing areas the obsolescence factor has been assumed to be large enough to shorten the life of treatment plants to 10 to 15 years.[4] Thus the life of the plant and the repayment

[1] The Cenex Corporation Plant is being phased out of production.

[2] Million gallons per day.

[3] A Report on Pollution of the Upper Mississippi River and Major Tributaries (Washington: United States Department of the Interior, Federal Water Pollution Control Administration, July 1966), Table IV-18.

[4] Richard Frankel, Water Quality Management: An Engineering Economic Model for Domestic Waste Disposal (Unpublished Ph. D. Dissertation, Berkeley: University of California, 1965).

period roughly coincide so that the amortized yearly capital expenditure can be considered as the annual capital cost.

The annual costs of the municipal sewage treatment plants in the study area are financed by user charges and taxes. In most cases the charges for sewage treatment are collected along with the charges for water use. Thus, to some degree, the charges are pro-rated on the amount of water used. This system is also used for the industrial polluters who discharge waste water into municipal sewage systems.

C. The Cost Curves of Conventional Sewage Treatment Plants

Between 35% and 90% BOD removed, the total annual cost functions of conventional sewage treatment plants increase approximately linearly as the percent BOD removed increases. From 0% to 35% BOD removed the total annual costs increase at a decreasing rate and from 90% to 100% BOD removed the total annual costs increase at an increasing rate. On the evidence obtained in empirical investigations, Frankel[1] estimated the shape and magnitude of the relationship between total annual costs and percent BOD removed for several different treatment plants of different design capacities (Figure 9). Frankel computed the total annual cost per million gallons a day treated in order to compare the cost functions of treatment plants of different sizes. This transformation of the vertical axis does not alter the description of the cost curve given above.

All five of the municipal treatment plants in the study area have the necessary plant and equipment for primary and secondary treatment and none of them have facilities for tertiary treatment. For the purposes of this study it was assumed that each treatment plant provides at least primary treatment (35% BOD removed) and that none of them can accomplish more than secondary treatment (90% BOD removed). Therefore, for each of the five plants there is a cost function similar to Figure 9, but for this analysis only the linear portion between 35% BOD removed and 90% BOD removed is important.

The linear portion of the total annual cost curve for each municipal sewage treatment plant was estimated on the basis of information provided by the plant operators. No attempt was made to estimate the portions of the cost curve from 0% to 35% BOD removed or from 90% to 100% BOD removed. These upper and lower portions of the cost curve remained undefined.

D. The Cost Data

The first step in the cost survey was to obtain the total annual costs and percent BOD removed by each plant for 1968. The

1 Ibid.

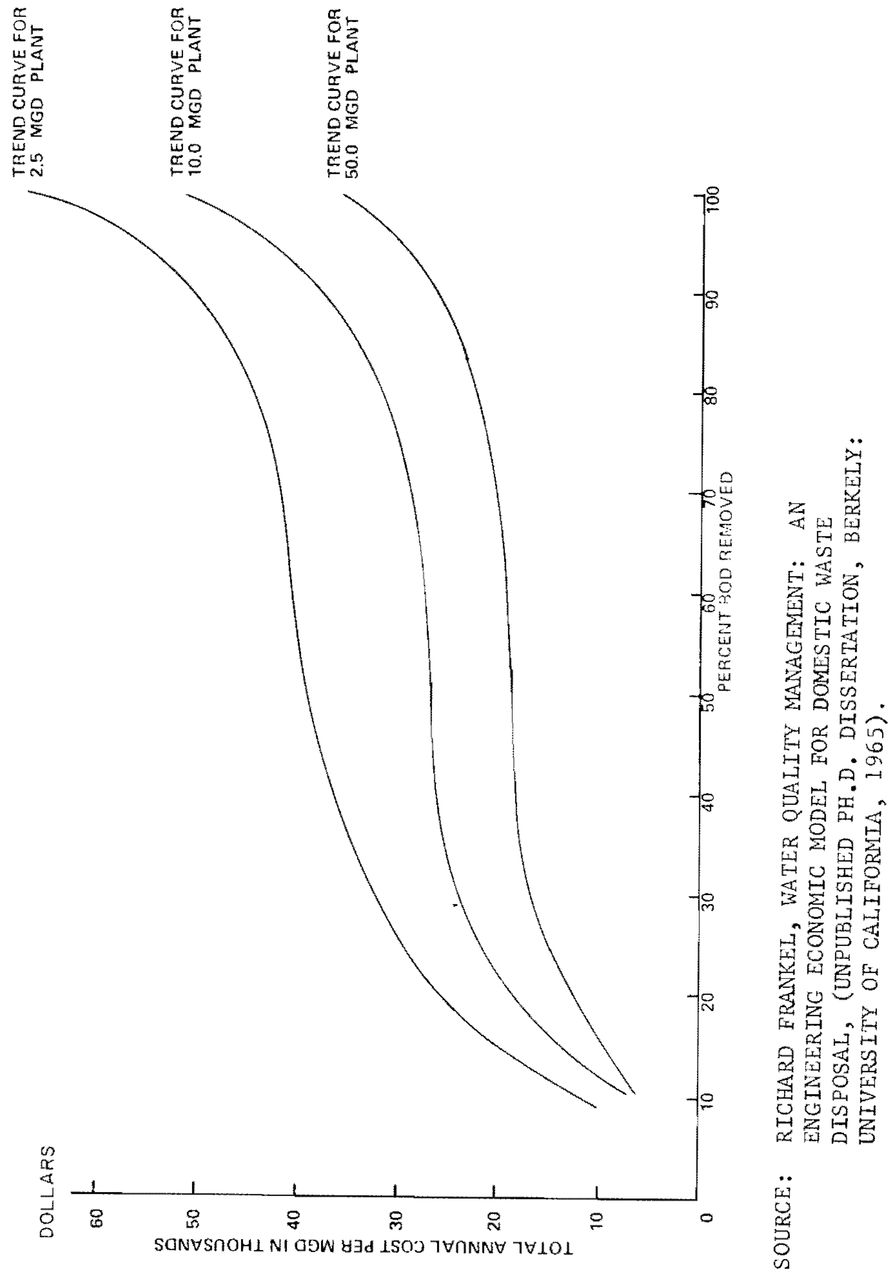

FIGURE 9. THE TOTAL ANNUAL COST OF REMOVING BIOCHEMICAL OXYGEN DEMAND

cost figures from the accounting records of the plants and the average percent BOD removed are given in Table 4.

TABLE 4. THE TOTAL ANNUAL COSTS AND AVERAGE PERCENT BOD REMOVED FOR 1968

Plant	Total Annual Costs for 1968	Average Percent BOD Removed for 1968
MSSD	$3,239,811	48.8
South St. Paul[a]	205,374	90.0
Newport	9,112	90.0
St. Paul Park	23,102	72.0
Cottage Grove	50,000	90.0

[a] Data obtained covered the period from April 1, 1968 to March 31, 1969

Source: Survey of treatment plant operators in September and October, 1969.

The data in Table 4 gives one point on each cost curve. At least one other point on the linear portion of the cost curve was needed to obtain a value for the slope of this function. All of the plant operators were able to estimate the cost of performing only primary treatment (35% BOD removed) by not operating their secondary treatment facilities. The operator of the MSSD sewage treatment plant was also able to estimate, on the basis of engineering studies, the total annual cost of removing 90% of the BOD entering the plant. From these responses and the actual cost data for 1968 the cost curves between 35% and 90% BOD removed were calculated.

MSSD : $C_1 = 5017\ E_1 + 2{,}994{,}981$

South St. Paul : $C_2 = 231\ E_2 + 184{,}584$

Newport : $C_3 = 89\ E_3 + 1{,}102$

St. Paul Park : $C_4 = 9\ E_4 + 22{,}454$

Cottage Grove : $C_5 = 364\ E_5 + 17{,}240$

where C_i = total annual cost of treatment plant i

E_i = percent BOD removed by treatment plant i, for $35 \leq E_i \leq 90$.

The coefficient of the E_i term is the cost of removing an additional percent of BOD from the effluent of the i^{th} treatment plant between 35% and 90% BOD removed. This coefficient varies over a wide range, depending on the amount and strength of the influent, the design capacity and the treatment process of the individual treatment plants,[1] the percent of joint pollutants[2] removed, and the over-all efficiency of operation of each plant. The constant term in each cost function is the vertical intercept value of the linear portion of the cost curve if the cost curve was linear from 0% to 35% BOD removed. However the cost curve is undefined in this lower portion and the constant term has no significance other than to vertically position the linear portion of the cost function. Some of the relevant variables that affect the cost functions of the five plants are shown in Table 5.

TABLE 5. VARIABLES AFFECTING COST FUNCTIONS

Plant	Average Flow in MGD	Strength of Influent in mg/l of Five Day BOD	Percent of Total Suspended Solids Removed
MSSD	188.60	251	66
South St. Paul	14.20	1,298	89
Newport	.06	157	71
St. Paul Park	.35	217	67
Cottage Grove	.42	240	72

Source: A Report on Pollution of the Upper Mississippi River and Major Tributaries (Washington, D.C.: United States Department of the Interior, Federal Water Pollution Control Administration, July, 1966), Table IV-6.

To compare the cost functions of the five treatment plants with each other and with Frankel's empirical results, it was necessary to transform the dependent variable of the cost functions to the total annual cost per million gallon per day treated. This transformation yielded the following equations for the linear portion of the cost curves which are graphed in Figure 10:

MSSD : $CM_1 = 27\ E_1 + 15{,}880$

South St. Paul : $CM_2 = 16\ E_2 + 12{,}999$

Newport : $CM_3 = 1{,}534\ E_3 + 19{,}000$

[1] See Table 3.

[2] Joint pollutants include suspended solids, volatile solids, coliform density, phytoplankton density and turbidity.

St. Paul Park : $CM_4 = 26 E_4 + 64,154$

Cottage Grove : $CM_5 = 856 E_5 + 40,565$

where CM_i = the total annual cost per million gallon per day treated at the i^{th} treatment plant

E_i = the percent BOD removed by the i^{th} treatment plant, for $35 \leq E_i \leq 90$.

The cost curves in Figure 10 are quite similar to those obtained by Frankel, shown in Figure 9. The differences in the cost functions follow from the differences in the sizes, treatment processes, and influents of the treatment plants. As Frankel observed in general, the larger the treatment facility, the lower the cost per million gallons per day of waste treated and vice versa. This suggests that economies of scale can be achieved in waste treatment, and partially explains why the cost curves of the two larger treatment plants are somewhat lower than the cost curves of the three smaller plants. However, this is not the entire explanation.

Another portion of the explanation lies in the fact that there are a number of different secondary treatment processes.[1] The anaerobic stabilization pond used by the South St. Paul Plant is relatively inexpensive. This process is especially suited to the South St. Paul conditions because of the high concentration of BOD in the influent. However, with this high concentration of BOD in the influent, even with 90% of the BOD removed the effluent is still relatively strong compared to the effluents of the other plants.

The differences in the cost curves of the three smaller plants with higher costs per million gallons per day treated are more difficult to explain. The cost of primary treatment at these plants is nearly the same, as would be expected with similar sized plants (Figure 10). However, the difference in the cost of removing 90% of the BOD by these plants can only be explained by the differences in treatment processes, particular physical conditions and the over-all efficiencies of the plant operations. It should also be noted that because of the deficiencies in the operational data kapt at these smaller plants, the estimates of the cost functions by the plant operators are probably less reliable than the data obtained at the two larger plants where more complete information on the costs of operation were available.

[1] See Table 3. These processes have varying economic advantages depending on the physical characteristics of the particular situation.

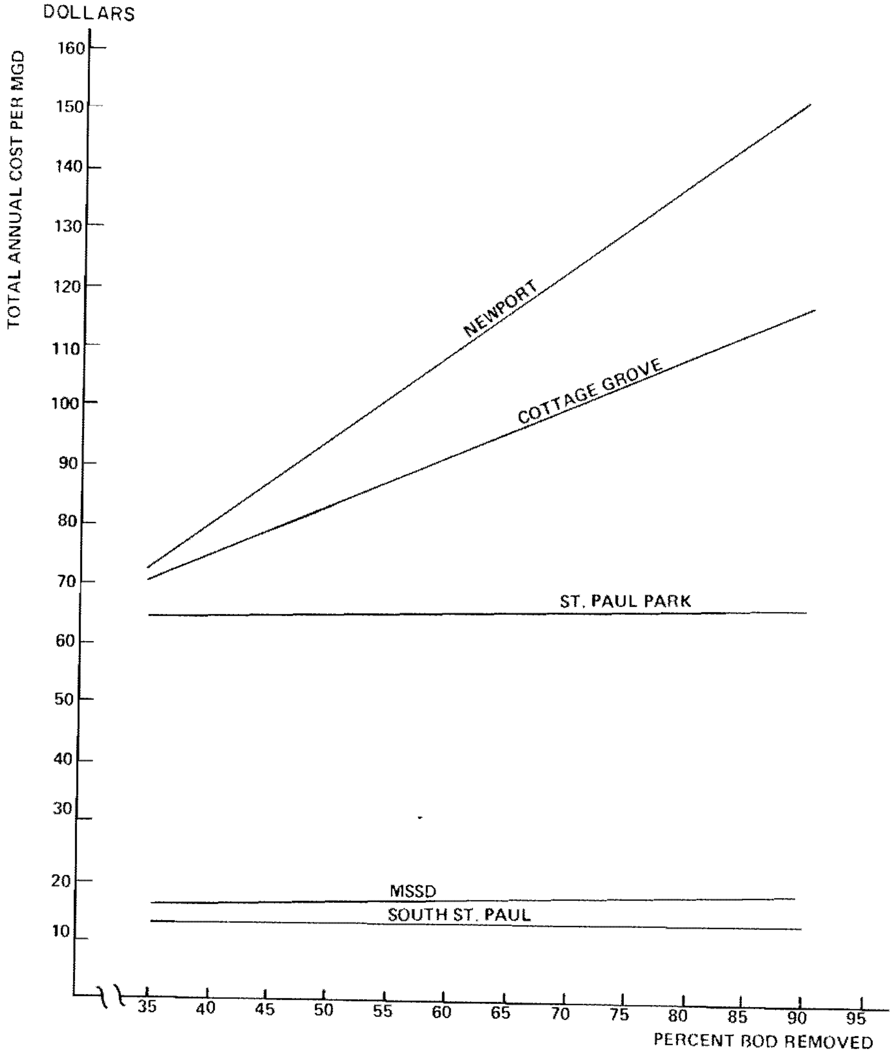

FIGURE 10. THE TOTAL ANNUAL COST PER MGD FOR THE FIVE TREATMENT PLANTS

CHAPTER IV

THE ANALYTICAL MODEL

A. The Model

The model utilized in this study was basically the formulation developed by Revelle, Loucks and Lynn.[1] They developed this mode' to find least cost methods of sewage treatment needed to maintain dissolved oxygen water quality standards in situations similar to that in this study area.

The mathematical formulation of the model was set up in such a way that the least cost management plan for maintaining the DO river standard could be obtained by solving the system with the linear programming algorithm. This formulation of the problem has four parts: first, the objective function; second, a set of constraints and a set of equalities reflecting the physical capabilities and operation of the sewage treatment plants; third, a set of "inventory" equations which record changes in BOD and DO deficit concentrations from point to point in the river where mixing of different concentrations occurs; and fourth, a set of specified "institutional" constraints on the DO deficit concentration at specified intervals in the river.

1. The Objective Function

The objective function is the sum of the total annual treatment costs of the five municipal sewage treatment plants in the study area. The minimization of this sum yields the least cost means of treating the municipal wastes for the entire study area. The formulation of the objective function is based on the individual plant cost functions estimated in Chapter III as

(12) $$TC_i = C_iE_i + K_i$$

where TC_i = total annual cost of the i^{th} treatment plant

E_i = percent of BOD removed by the i^{th} treatment plant, $35 \leq E_i \leq 90$

C_i = the change in annual cost per unit change in the percent BOD removed by the i^{th} treatment plant

[1] C. S. Revelle, D. P. Loucks and W. P. Lynn, "A Management Model for Water Quality Control," Journal of Water Pollution Control Federation, Vol. 39, No. 7 (July, 1967), pp. 1164-83; and D. P. Loucks, C. S. Revelle and W. P. Lynn, "Linear Programming Models for Water Pollution Control," Management Science, Vol. 14, No. 4 (December, 1967), pp. 166-181.

K_i = constant term in cost equation of i^{th} treatment plant.

The total cost function, entered as the objective function in the model, is

(13) $$COST = \sum_{i=1}^{5} TC_i = \sum_{i=1}^{5} C_iE_i + \sum_{i=1}^{5} K_i$$

where COST = total annual cost of sewage treatment for study area.

The objective function used in this model is given in Appendix A.

2. The Treatment Plant Constraints and Efficiency Equations

As discussed in Chapter II, each of the sewage treatment plants in this study was assumed to carry out at least primary treatment (35% BOD removed) and was assumed not to be capable of tertiary treatment (above 90% BOD removed). This constraint on the capabilities of each plant was entered into the model as

(14) $$E_i \geq 35\% \text{ and } E_i \leq 90\% \quad i = 1,\ldots,5$$

The percent BOD removed by each treatment plant is determined by the concentration of BOD in the water entering the plant and the concentration of BOD in the water leaving the plant and entering the river. By definition the percent BOD removed by each plant is

(15) $$E_i = \frac{P_i - M_i}{P_i}$$

where P_i = concentration of BOD in the influent of the i^{th} treatment plant in mg/l

M_i = concentration of BOD in the effluent of the i^{th} treatment plant in mg/l.

Rewriting the above equation yields

(16) $$E_i + \frac{1}{P_i} M_i = 1$$

which is the formulation of efficiency conditions used in this model. The treatment plant constraints and efficiency equations for the five plants in the study are given in Appendix A.

3. The Inventory Equations

At each point where a discharge of waste water enters the river, a mixing of DO deficit and BOD concentration occurs. For this formulation it was assumed that instantaneous complete mixing takes place at these points. The set of equations which together specify the concentrations of BOD and DO deficits in the river, after complete mixing, are referred to as "mass balance" relationships. The mass balance relationships used in the model are:[1]

DO deficit mixing equation

(17)
$$D_j \sum_{i=0}^{i=j} Q_i = XE_{j-1} \sum_{i=0}^{i=j-1} Q_i + T_i Q_i$$

where D_j = DO deficit concentration in the river at the beginning of the j^{th} reach, after complete mixing, in mg/l

XE_{j-1} = DO deficit concentration in the river just upstream from the j^{th} reach in mg/l

T_i = DO deficit concentration in the effluent of the i^{th} treatment plant in mg/l

Q_i = rate of flow entering the j^{th} reach from i^{th} treatment plant in liters/second,[2]

equation (17) is rewritten as

(18)
$$D_j \sum_{i=0}^{i=j} Q_i - XE_{j-1} \sum_{i=0}^{i=j-1} Q_i = T_i Q_i$$

which is the form used in the model.

BOD mixing equation

(19)
$$L_j \sum_{i=0}^{i=j} Q_i = F_{j-1} \sum_{i=0}^{i=j-1} Q_i + M_i Q_i$$

[1] The MSSD plant is labeled plant 1 (i=1), the South St. Paul plant is labeled plant 2 (i=2), and so forth in succession down to the Cottage Grove plant which is labeled plant 5 (i=5). Similarly, the segment of the river between the MSSD outfall down to a point just upstream from the South St. Paul outfall will be labeled reach 1 (j=1), reach 2 (j=2) extends from the South St. Paul outfall down to a point just upstream from the Newport outfall and so forth down through reach 5 (j=5) which is from the Cottage Grove outfall down to a point just upstream from the Chemolite Plant outfall. Reach 6 (j=6) extends from the Chemolite Plant outfall down to the Hastings Dam.

[2] Q_i with i=0 is the rate of flow entering reach 1 from upstream in mg/l.

where L_j = BOD concentration in the river at the beginning of the j^{th} reach, after complete mixing, in mg/l

F_{j-1} = BOD concentration in the river just upstream from the j^{th} reach in mg/l

M_i = BOD concentration in the effluent of the i^{th} treatment plant in mg/l,

equation (19) is rewritten as

(20)
$$L_j \sum_{i=0}^{i=j} Q_i - F_{j-1} \sum_{i=0}^{i=j-1} Q_i - M_i Q_i = 0$$

which is the form used in the model.

In the DO deficit mixing equation for reach 1, all the quantities except D_1 are known input data for this model. In the BOD mixing equation for reach 1 all the quantities except L_1 and M_1 are known input data. Hence the BOD concentration in the river after mixing (L_1) and therefore, the DO sag curve for reach 1 are dependent (among other things) on the concentration of BOD in the effluent of treatment plant 1 (M_1).

For reaches 2-5 (j=2, ...,5) all the quantities in equation (18) are known except D_j and XE_j. The DO deficit at the end of any reach (XE_j) is given by equation (4) which, if rewritten using the consistent notation, becomes

(21)
$$XE_j = \frac{k_j}{r_j - k_j} \left(10^{-k_j t_j} - 10^{-r_j t_j}\right) L_j + \left(10^{-r_j t_j}\right) D_j$$

where XE_j = DO deficit concentration in the river at the end of the j^{th} reach in mg/l

k_j = deoxygenation rate constant for j^{th} reach

r_j = reaeration rate constant for j^{th} reach

t_j = time of flow from beginning to end of j^{th} reach in days.

Equation (21) is rewritten as

(22)
$$XE_j - \frac{k_j}{r_j - k_j} \left(10^{-k_j t_j} - 10^{-r_j t_j}\right) L_j - \left(10^{-r_j t_j}\right) D_j = 0$$

which is the form used in the model.

The BOD concentration at the end of any reach is predicted by equation (2), which rewritten with consistent notation, is

$$F_j = (10^{-k_j t_j})\, L_j \tag{23}$$

where F_j = BOD concentration in the river at the end of the j^{th} reach in mg/l.

Equation (23) is rewritten as

$$F_j - (10^{-k_j t_j})\, L_j = 0 \tag{24}$$

which is the equation used in the model.

The four equations (18), (20), (22), and (24) serve as a set of equations which can be solved for L_j and D_j for any reach ($j=2, \ldots, 5$) given the necessary input data and the values of D_{j-1}, L_{j-1}, and M_j. The inventory equations for the five reaches in this study are listed in Appendix A.

4. The Institutional Water Quality Constraints

Equation (4) can be written to predict the DO deficit concentration at any point within any of the five reaches in the study area, given the values of D_j and L_j obtained from the inventory equations, as

$$DOD_{j,t} = \frac{k_j}{r_j - k_j}\,(10^{-k_j t} - 10^{-r_j t})\, L_j + (10^{-r_j t})\, D_j \tag{25}$$

where $DOD_{j,t}$ = DO deficit concentration at time t in the j^{th} reach in mg/l.

By varying "j" over the number of reaches in the study area ($j=1, \ldots, 5$) and "t" by an incremental amount, within each reach the DO deficit concentration was predicted throughout the study area. For this study, the DO deficit concentration was predicted at intervals of .25 days of flow within each reach. The results give a series of points on the predicted DO sag curve.

The institutional constraints on the DO concentration in the river are expressed as constraints on the DO deficit concentrations at each of the points on the estimated DO sag curve. These constraints are expressed as

$$DOD_{j,t} \leq DOD_s \tag{26}$$

where DOD_s = DO deficit concentration specified by the river standard in mg/l.

The DO deficit concentration is thus constrained at intervals of .25 days of flow throughout the five reaches. It is possible that the DO in the river could drop slightly below the DO standard within any of the intervals but the violation of the standard would be for less than .25 days of flow and the consequences would not be significant. The institutional constraints for the study area are given in Appendix A.

B. The Input Data

A complete listing of the physical and economic input data for the model is given in Appendix B. Although most of this data was discussed in Chapters II and III, some of the values used warrant further explanation.

The values of the deoxygenation and reaeration rate constants for the study area were particularly troublesome. The literature indicates considerable difference of opinion as to the values of these constants, as pointed out in a discussion by Dale Bryson of the FWPCA.[1] As shown by Bryson, the values used for these constants will significantly influence the solution of this model. The values used (listed in Appendix B) are, in the author's opinion, the best estimates available. It is anticipated that as further research resolves the present controversy, more accurate estimates of these constants will be forthcoming.

C. The Linear Programming Solutions

Table 6 presents several linear programming solutions of the model under alternative DO standards. Each solution specifies the cost minimizing strategy (in terms of the percent BOD removed by each plant) and the minimum total annual cost of maintaining the given river standard in the study area.

The solutions obtained are a direct result of the differences in the marginal cost of raising the minimum DO concentration in the river by increasing the percent BOD removed by any one plant individually, while holding the percent BOD removed at the other plants constant. This marginal cost figure is $27,245.00 for the Minneapolis-St. Paul Sanitary District Plant and $4,343.00 for the South St. Paul Plant. The discharge of the Newport Plant is so small relative to the river that the difference between 35% and 90% removed affects the minimum DO concentration by less than .005 mg/l, which was not considered to be significant. The St. Paul Park and Cottage Grove Plants are located downstream from the point where the minimum DO concentration occurs and therefore the marginal cost figure is not relevant.

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[1] "Memorandum on the Waste Assimilation Capacity of the Mississippi River in the Twin Cities Metropolitan Area" (Minneapolis: Minnesota Pollution Control Agency, March 27, 1969), Appendix B.

TABLE 6. LEAST COST MANAGEMENT SCHEMES FOR MAINTAINING ALTERNATIVE DISSOLVED OXYGEN RIVER STANDARDS

	Solution With DO Standard of							
	0 mg/l		.5 mg/l		1.0 mg/l		1.5 mg/l	
	Percent BOD Removed	Total Annual Cost	Percent BOD Removed	Total Annual Cost	Percent BOD Removed	Total Annual Cost	Percent BOD Removed	Total Annual Cost
Minneapolis-St. Paul	71.357	3,352,979	74.073	3,366,605	76.788	3,380,226	79.503	3,393,848
South St. Paul	90.000	205,374	90.000	205,374	90.000	205,374	90.000	205,374
Newport	35.000	4,217	35.000	4,217	35.000	4,217	35.000	4,217
St. Paul Park	35.000	22,769	35.000	22,769	35.000	22,769	35.000	22,769
Cottage Grove	35.000	29,980	35.000	29,980	35.000	29,980	35.000	29,980
Total Annual Cost For Study Area		3,615,319		3,628,945		3,642,566		3,656,188

TABLE 6. CONTINUED

	Solution With DO Standard of							
	2.0 mg/l		2.5 mg/l		3.0 mg/l		3.43 mg/l	
	Percent BOD Removed	Total Annual Cost	Percent BOD Removed	Total Annual Cost	Percent BOD Removed	Total Annual Cost	Percent BOD Removed	Total Annual Cost
Minneapolis-St. Paul	82.218	3,407,469	84.934	3,421,095	87.649	3,434,716	90.000	3,446,511
South St. Paul	90.000	205,374	90.000	205,374	90.000	205,374	90.000	205,374
Newport	35.000	4,217	35.000	4,217	35.000	4,217	35.000	4,217
St. Paul Park	35.000	22,769	35.000	22,769	35.000	22,769	35.000	22,769
Cottage Grove	35.000	29,980	35.000	29,980	35.000	29,980	35.000	29,980
Total Annual Cost For Study Area		3,669,809		3,683,435		3,697,056		3,908,851

Given the marginal cost data the least cost method of increasing the minimum DO concentration is to increase the percent BOD removed by the South St. Paul Plant, since it has the lowest marginal cost. In order to get the minimum DO concentration up to the relevant range, it is necessary to increase the percent BOD removed by the South St. Paul Plant up to its maximum of 90% and to increase the percent BOD removed by the Minneapolis-St. Paul Sanitary District Plant (second lowest marginal cost) up to 71.357%. For higher DO standards, the percent BOD removed by the Minneapolis-St. Paul Sanitary District Plant would need to be increased above 71.357%. The highest possible DO standard that can be maintained in the study area at the SCD flow under existing circumstances is 3.43 mg/l, which is possible when the South St. Paul and Minneapolis-St. Paul Sanitary District Plants both treat at their physical maximum of 90% BOD removed.

Of particular interest is the solution obtained under the DO standard of 3 mg/l, the current river standard in the study area. The predicted DO sag curve under this solution is shown in Figure 11. This solution indicates that the percent BOD removed by the Minneapolis-St. Paul Sanitary District Plant needs to be increased from the present rate of 48.8% to 87.649%, in order to achieve the current river standard with the least increase in total annual treatment expenditures. Under this solution the South St. Paul Plant would continue to remove BOD at its present rate of 90%, while the Newport, St. Paul Park and Cottage Grove treatment plants need only perform primary treatment, resulting in lower costs at these plants. The net result of the increase in annual expenditures for treatment at the Minneapolis-St. Paul Sanitary District Plant and the decrease in annual expenditures at the Newport, St. Paul Park and Cottage Grove Plants would be a net increase in total annual cost from the 1968 figure of $3,527,399 to $3,697,056. Thus the current river standard could be maintained in the study area for an additional annual expenditure of 4.8%.

In Figure 12 the total annual cost of treatment for the study area is graphed as a function of the various DO standards maintained. Each point on the graph represents a least cost solution for meeting the corresponding DO standard on the horizontal axis. The relevant range of DO standards was between 0 mg/l and 3.43 mg/l of DO and all of the increase in the minimum DO concentration was obtained by increasing the percent BOD removed by the Minneapolis-St. Paul Sanitary District Plant. Since the marginal cost of increasing the minimum DO concentration by increasing the percent BOD removed by any one plant (while holding the percent BOD removed by the others constant) is linear, the total annual cost function for the study area (Figure 12) is linear.

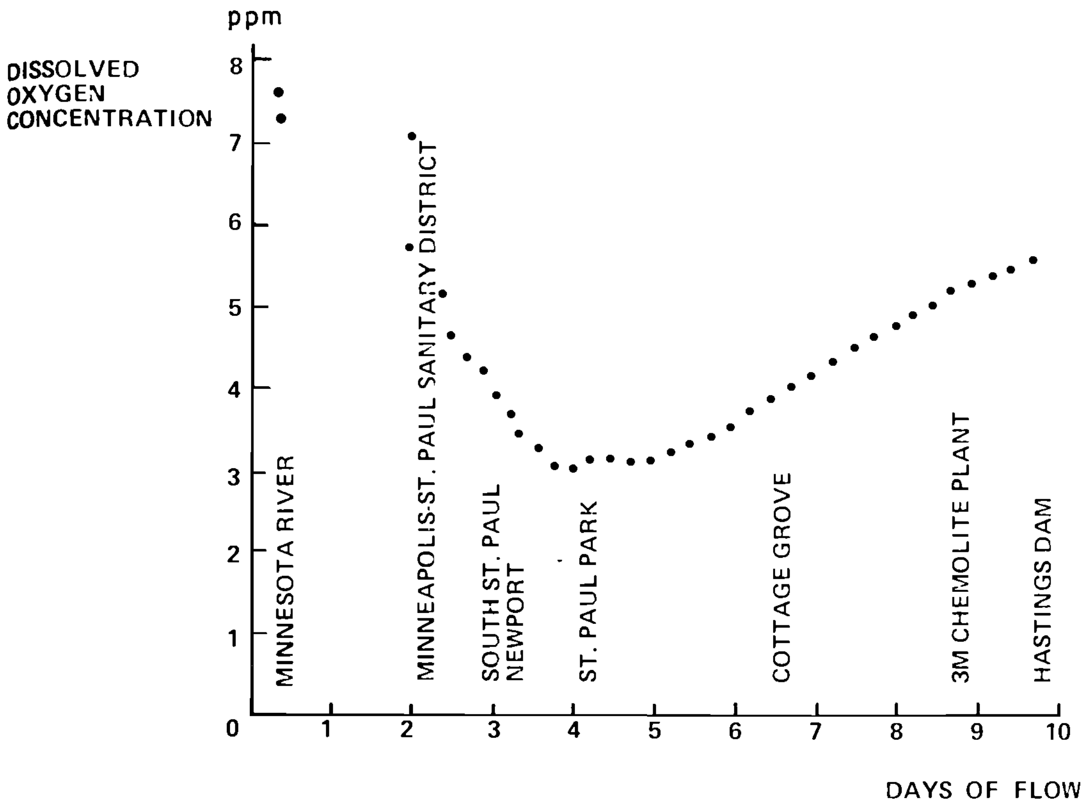

FIGURE 11. THE PREDICTED DISSOLVED OXYGEN SAG CURVE FOR MAINTAINING A RIVER STANDARD OF 3 MG/L OF DISSOLVED OXYGEN

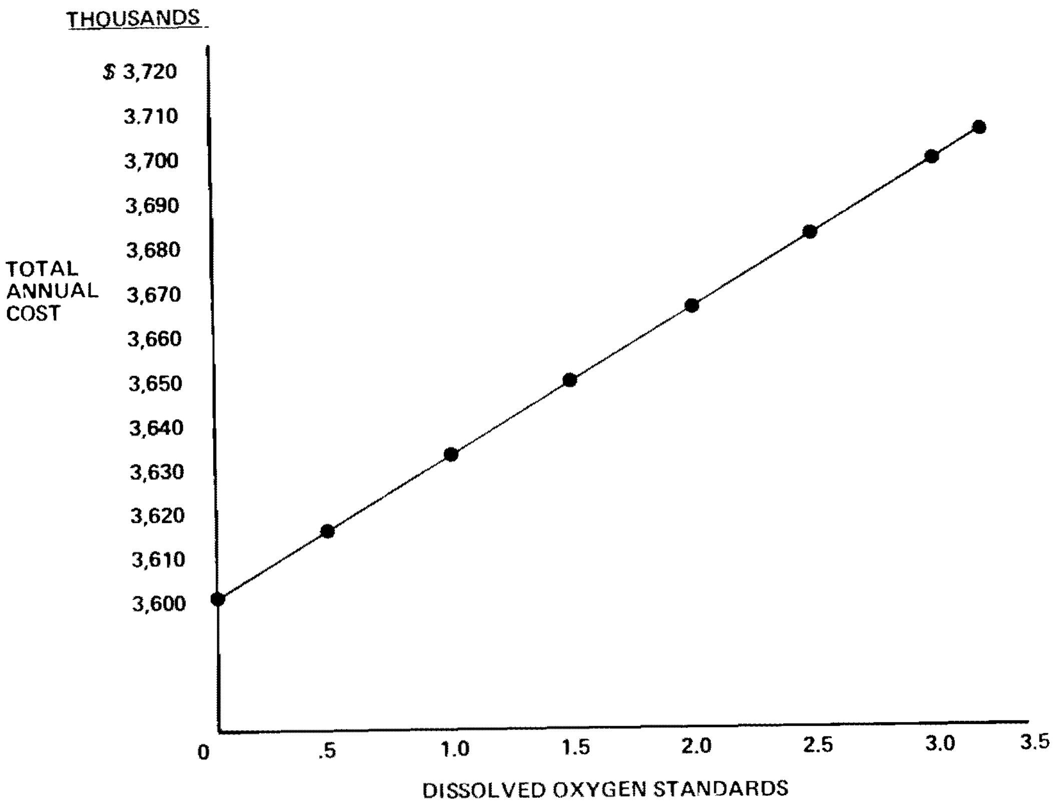

FIGURE 12. THE TOTAL ANNUAL COST OF MAINTAINING ALTERNATIVE DISSOLVED OXYGEN STANDARDS IN THE STUDY AREA

CHAPTER V

SUMMARY AND CONCLUSIONS

A. Summary of Results

The results of this study will be summarized by referring back to the four objectives stated in Chapter I, section C.

The physical, institutional and economic conditions existing in the study area were described in Chapters II and III. This detailed analysis of the underlying factors in a particular case of river pollution should lead to a better understanding of the pollution problem in similar situations. In addition the method of analysis presented here is generally applicable to other situations where several sources of organic waste are discharged into a section of a river on which DO standards have been set.

An analytical framework for combining the DO sag curve, resulting from organic waste disposal in a river, with the cost of sewage treatment was presented in Chapter IV. From this model the least cost methods of attaining alternative levels of DO in the river were estimated. The model is most useful in the case where several sewage treatment plants are located along a river in such a manner that the DO concentration in the river decreases between successive plants, and where the treatment plants are administered under one management whose primary objective is to maintain a given river standard at minimum cost. This model can be modified to be applied to various river basin configurations with any number of sources of treated or untreated organic wastes plus any number of tributaries.

The most serious limitation of the proposed model is the extensive data on the physical characteristics of the river that are needed to specify the DO sag curve and the waste assimilation capacity of the stream. Two previous studies had been completed on the waste assimilation capacity of the Upper Mississippi River and, as a result, the necessary physical data were available for the section of the river in and below the Twin Cities.

As formulated for this analysis, the model is only applicable to situations where the total annual cost function of each sewage treatment plant is linear in the relevant range of percent BOD removed. The available empirical evidence and the discussions with plant operators supported the contention that the cost function of each of the five sewage treatment plants in the study area is linear between 35% and 90% BOD removed. If in other situations the cost curves of the treatment plants are not linear a reformulation of the constraints would be necessary in order to obtain a least cost linear programming solution.

As with most investigations of this nature the results obtained from the model are only as good as the necessary input data. The physical data used to predict the DO sag curve in the study area were based on historical observations of the relevant parameters of the Mississippi River, and were therefore considered to be as reliable as can be obtained under current engineering techniques. The estimates of the total annual cost functions of the five treatment plants were based on interviews of the plant operators. These estimated cost functions are probably not as reliable as the physical data, but they are the best available under existing conditions.

The solutions obtained under alternative water quality standards should be useful as a planning device. The increases in the amount of waste removal at the treatment plants will most likely occur in increments as expansion of the plants takes place. Upon the completion of each expansion of treatment facilities, the proposed framework of analysis can be solved under the new situation, and the results can be used to guide the future course of action. Thus, by utilizing the model, agencies like the proposed Sewer Service Board can zero in on the least cost management scheme.

The final two objectives of finding the highest possible DO sag curve and testing the hypothesis are related to the solutions of the model under alternative water quality standards (Table 6 and Figure 12). Based on these solutions, the highest DO river standard that is physically possible in the study area (with the current treatment facilities) is 3.43 mg/l, at an estimated total annual cost of $3,908,851. The most economical method of maintaining the current river standard of 3 mg/l of DO would cost an estimated $3,697,056 annually or an increase in total yearly expenditure of 4.8%. This solution implies the acceptance of the hypothesis that a cost minimizing management plan can be devised which will maintain the current DO river standard in the study area in a manner that is both physically and economically feasible under existing conditions.

B. Conclusions and Policy Implications

With the acceptance of the hypothesis it can be concluded that the change in the institutional structure adopted by the Minnesota Legislature will lead to a more efficient allocation of treatment expenditures at the five municipal treatment plants in the study area. This increase in the efficiency will make it possible for the river standard to be maintained with an increase of total annual treatment costs for this stretch of the river of less than 5%.

It was determined from the oxygen profile in the study area that the location of the outfall of waste dischargers with respect to the DO sag curve in the river is an important, but apparently neglected, facet of the river pollution problem. In this study it was found that only primary waste treatment would be required in order to maintain the current DO river standard at the St. Paul Park and Cottage Grove treatment plants since they are located downstream from the low point of the DO sag curve. However, even though secondary treatment at these two plants is not needed to maintain the current river standard, the benefits of secondary treatment at these plants may exceed the additional cost in which case it would be desirable to continue primary and secondary treatment.

The determination of the highest possible DO standard that can be maintained in the study area with existing facilities indicates that more waste removal than is currently possible will be required if the river standard is to be maintained at any level above 3.43 mg/l of DO at the SCD flow.

C. Unresolved Issues

This study proposed a method of determining the marginal cost of raising the DO standard in a river but has not dealt with the marginal benefits accruing to society from changes in water quality standards. An important unresolved issue arising from this study is the question of the magnitude of the benefits which would accrue to society if the current river standard were maintained between the Ford and Hastings Dams. If these benefits exceed the treatment costs needed to achieve the standard, then the standard should be maintained and vice versa. Benefit-cost analysis might well be directed to the problem of the level of water quality to be maintained in the river. The optimum water quality standard is that standard where the marginal cost of the last unit of water quality just equals the marginal benefit accruing to society from this last unit of water quality.

The least cost methods of maintaining DO standards greater than 3.43 mg/l of DO were not included in this study. The analysis of the costs of DO standards greater than 3.43 mg/l would need to take into account the costs of alternative methods of attaining these higher DO standards. Alternative methods might include adding tertiary treatment, building additional secondary treatment plants, raising the quality of the water above the Ford Dam, or changing the location of the outfalls of the existing plants. By incorporating any of these changes and their associated costs into the proposed model, it is likely that higher river standards are possible at modest increases in yearly expenditures. Thus, the proposed model can be used as a planning device to estimate the costs of alternative methods of improving the water quality above the current standard.

The only specific assumption concerning the behavior of the Sewer Service Board was that this agency would minimize sewage treatment costs in the study area subject to maintaining the DO standard in the river. This assumption is not sufficient to define this new institution. The organization, operation, and financing of the Sewer Service Board were not specified. In practice the Sewer Service Board may have other objectives besides the one assumed, which could alter its behavior.

If implemented, the least cost solution for maintaining the current river standard would increase the annual treatment cost at the Minneapolis-St. Paul Sanitary District Plant and decrease the annual treatment cost at the Newport, St. Paul Park and Cottage Grove Plants. These changes in the annual costs of treatment would create another problem concerning the allocation of costs among the residents of the areas served by these plants. The issues involving the allocation of per capita sewage charges were not considered in this study.

APPENDIX A

THE LINEAR PROGRAMMING MODEL

I. The Objective Function

$$COST = TC_1 + TC_2 + TC_3 + TC_4 + TC_5$$

II. The treatment plant equations and constraints

$$\left(\frac{1}{P_1}\right) M_1 + E_1 = 1$$

$$\left(\frac{1}{P_2}\right) M_2 + E_2 = 1$$

$$\left(\frac{1}{P_3}\right) M_3 + E_3 = 1$$

$$\left(\frac{1}{P_4}\right) M_4 + E_4 = 1$$

$$\left(\frac{1}{P_5}\right) M_5 + E_5 = 1$$

$$E_1 \leq 90$$

$$E_2 \leq 90$$

$$E_3 \leq 90$$

$$E_4 \leq 90$$

$$E_5 \leq 90$$

$$E_1 \geq 35$$

$$E_2 \geq 35$$

$$E_3 \geq 35$$

$$E_4 \geq 35$$

$$E_5 \geq 35$$

III. Inventory Equations

A. Reach 1

$$-Q_oE_o + (Q_o + Q_1)\ D_1 = Q_1T_1$$

$$-Q_oF_o + (Q_o + Q_1)\ L_1 - Q_1M_1 = 0$$

B. Reach 2

$$XE_1 - \frac{k_1}{r_1-k_1}\ (10^{-k_1t_1} - 10^{-r_1t_1})\ L_1 - (10^{-r_1t_1})\ D_1 = 0$$

$$-(Q_o + Q_1)\ XE_1 + (Q_o + Q_1 + Q_2)\ D_2 = Q_2T_2$$

$$F_1 - (10^{-k_1t_1})\ L_1 = 0$$

$$-(Q_o + Q_1)\ F_1 + (Q_o + Q_1 + Q_2)\ L_2 - Q_2M_2 = 0$$

C. Reach 3

$$XE_2 - \frac{k_2}{r_2-k_2}\ (10^{-k_2t_2} - 10^{-r_2t_2})\ L_2 - (10^{-r_2t_2})\ D_2 = 0$$

$$-(Q_o + Q_1 + Q_2)\ XE_2 + (Q_o + Q_1 + Q_2 + Q_3)\ D_3 = Q_3T_3$$

$$F_2 - (10^{-k_2t_2})\ L_2 = 0$$

$$-(Q_o + Q_1 + Q_2)\ F_2 + (Q_o + Q_1 + Q_2 + Q_3)\ L_3 - Q_3M_3 = 0$$

D. Reach 4

$$XE_3 - \frac{k_3}{r_3-k_3}\ (10^{-r_3t_3} - 10^{-r_3t_3})\ L_3 - (10^{-r_3t_3})\ D_3 = 0$$

$$-(Q_o + Q_1 + Q_2 + Q_3)\ XE_3 + (Q_o + Q_1 + Q_2 + Q_3 + Q_4)\ D_4 = Q_4T_4$$

$$F_3 - (10^{-r_3t_3})\ L_3 = 0$$

$$-(Q_o + Q_1 + Q_2 + Q_3)\ F_3 + (Q_o + Q_1 + Q_2 + Q_3 + Q_4)\ L_4 - Q_4M_4 = 0$$

E. Reach 5

$$XE_4 - \frac{k_4}{r_4-k_4}\ (10^{-r_4t_4} - 10^{-r_4t_4})\ L_4 - (10^{-r_4t_4})\ D_4 = 0$$

$$-(Q_o+Q_1+Q_2+Q_3+Q_4)\ XE_4 + (Q_o+Q_1+Q_2+Q_3+Q_4+Q_5)\ D_5 = Q_5T_5$$

$$F_4 - (10^{-k_4t_4})\ L_4 = 0$$

$$-(Q_o+Q_1+Q_2+Q_3+Q_4)\ F_4 + (Q_o+Q_1+Q_2+Q_3+Q_4+Q_5)\ L_5 - Q_5M_5 = 0$$

IV. DO Constraints

A. Reach 1

$$\frac{k_1}{r_1-k_1}\ (10^{-k_1t} - 10^{-r_1t})\ L_1 + (10^{-r_1t})\ D_1 \leq DOD_s$$

where t varies from 0 to .993 by increments of .25 with both end points included.

B. Reach 2

$$\frac{k_2}{r_2-k_2}\ (10^{-k_2t} - 10^{-r_2t})\ L_2 + (10^{-r_2t})\ D_2 \leq DOD_s$$

where t varies from 0 to .356 by increments of .25 with both end points included.

C. Reach 3

$$\frac{k_3}{r_3-k_3}\ (10^{-k_3t} - 10^{-r_3t})\ L_3 + (10^{-r_3t})\ D_3 \leq DOD_s$$

where t varies from 0 to .866 by increments of .25 with both end points included.

D. Reach 4

$$\frac{k_4}{r_4-k_4}\ (10^{-k_4t} - 10^{-r_4t})\ L_4 + (10^{-r_4t})\ D_4 \leq DOD_s$$

where t varies from 0 to 3.245 by increments of .25 with both end points included.

E. Reach 5

$$\frac{k_5}{r_5-k_5}\ (10^{-k_5t} - 10^{-r_5t})\ L_5 + (10^{-r_5t})\ D_5 \leq DOD_s$$

where t varies from 0 to 1.222 by increments of .25 with both end points included.

APPENDIX B

INPUT DATA

I. Incoming River Conditions

A. Mississippi River at Lock and Dam Number 1 (River Mile 847.7)

Discharge (seven consecutive day once in 10 year summer low flow	48,137 l/sec.
Dissolved oxygen saturation concentration	7.500 mg/l
Dissolved oxygen concentration	7.500 mg/l
Ultimate BOD concentration[a]	4.185 mg/l
Water temperature	30.0°C
Deoxygenation rate constant at 30°C[b]	.079 $(\text{days})^{-1}$
Reaeration rate constant at 30°C[b]	.176 $(\text{days})^{-1}$

Source: A Report on Pollution of the Upper Mississippi River and Major Tributaries (Washington, D.C., United States Department of the Interior, Federal Water Pollution Control Administration, July, 1966), Table V-5. This source will hereafter be referred to as FWPCA Report, July, 1966.

[a] The ultimate BOD concentration was calculated using equation (5) with "k" at 30°C equal to .079 $(\text{days})^{-1}$.

[b] The deoxygenation and reaeration rate constants at 30°C were computed using equations (6) and (7) with "k" at 20°C equal to .05 $(\text{days})^{-1}$ and "r" at 20°C equal to .15 $(\text{days})^{-1}$.

B. Minnesota River at Confluence with Mississippi (River Mile 844.0)

Discharge (seven consecutive day once in 10 year summer low flow	9,967 l/sec.
Dissolved oxygen saturation concentration	7.000 mg/l
Dissolved oxygen concentration	5.500 mg/l
Ultimate BOD concentration	3.348 mg/l
Water temperature	26.7°C
Deoxygenation rate constant at 30°C	.079 $(\text{days})^{-1}$
Reaeration rate constant at 30°C	.176 $(\text{days})^{-1}$

Source: FWPCA Report, July, 1966, Table A-2.

C. Mississippi River Just Upstream From MSSD Plant (River Mile 836.3)

Discharge (seven consecutive day once in 10 year summer low flow)	58,104 l/sec.
Dissolved oxygen saturation concentration	7.500 mg/l
Dissolved oxygen concentration	6.564 mg/l
Ultimate BOD concentration	2.859 mg/l
Water temperature	30.0°C
Deoxygenation rate constant at 30°C	.079 $(\text{days})^{-1}$
Reaeration rate constant at 30°C	.176 $(\text{days})^{-1}$

Source: These conditions were computed by combining the incoming river conditions of the Mississippi and Minnesota Rivers using equations (2) and (4) to predict the BOD and DO concentrations.

II. Treatment Plant Data

Treatment Plant	Ultimate BOD Concentration of Influent in mg/l[a]	Average Discharge in l/sec.	DOD in Effluent in mg/l	Total Annual Cost Function[b]
MSSD	420.174	8,263	7.5	$TC_1 = 5017E_1 + 2{,}994{,}981$
South St. Paul	2172.852	622	7.5	$TC_2 = 231E_2 + 184{,}584$
Newport	262.818	3	7.5	$TC_3 = 89E_3 + 1{,}102$
St. Paul Park	363.258	15	7.5	$TC_4 = 9E_4 + 22{,}454$
Cottage Grove	401.760	18	7.5	$TC_5 = 364E_5 + 17{,}240$

Source: FWPCA Report, July, 1966.

[a] These values were calculated from the data presented in FWPCA Report, July, 1966, Table IV-7 converting five day BOD into ultimate BOD.

[b] From cost survey reported in Chapter III.

III. Other Input Data

A constant waste source of 200.880 mg/l of ultimate BOD with a flow of 252 l/sec. from the Minnesota Mining and Manufacturing Chemolite Plant at river mile 817.2 was included as the only significant industrial source of BOD in the study area.[1]

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[1] Source: FWPCA Report, July, 1966, Table IV-19.

Beginning of Reach	River Mile at Beginning of Reach	Time of Flow From Beginning to End of Reach in Days[a]	Deoxygenation Rate Constant for Reach in $(days)^{-1}$ at 30°C	Reaeration Rate Constant for Reach in $(days)^{-1}$ at 30°C
Ford Dam	847.7	.358	.079	.176
Minnesota River	844.0	1.591	.079	.176
MSSD	836.3	.993	.158	.176
South St. Paul	832.4	.356	.158	.176
Newport	831.0	.866	.158	.176
St. Paul Park	829.0	3.245	.158	.176
Cottage Grove	819.6	1.222	.158	.176
Chemolite	817.2	1.078	.158	.176

[a] Computed from mean flow velocities given in Report on Hydrographic Studies of the Mississippi, Minnesota and St. Croix Rivers, Washington, D.C.: United States Department of Health, Education and Welfare, Federal Water Pollution Control Administration, December, 1965, Figures IV-1 through IV-5.

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