MODELLING WATER QUALITY IN THE UPPER KLIP RIVER

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DECLARATION BY CANDIDATE

I hereby declare that the dissertation presented herein is my own work, and has not been submitted for a degree at any other university

RODERICK WILLIAM ARNOLD

CONTENTS

SYNOPSIS

In the planning of water resources systems it is becoming increasingly important to consider the quality of the water in addition to the quantity, particularly in the more densely populated regions. The Pretoria-Witwatersrand-Vereeniging complex lends itself to the use of systems analysis techniques for the optimal planning of water and wastewater facilities on a regional basis and such an approach will certainly have to be employed in order to keep pace with the growth of the region. To include quality (in terms of the biochemical oxygen demand, BOD) as a decision variable in the analysis, consideration should be given to the optimisation of the siting and sizing of wastewater treatment plants and to the simulation of the organic self-purification process in the receiving waters.

In this study both of these aspects are investigated. A computer model for the design of an activated sludge sewage treatment plant is developed, but, following further investigations, it is found that for South African design criteria the BOD of the sewage effluent is not a decision variable in the design process and can therefore be omitted from the regional planning picture.

An hourly BOD-dissolved oxygen simulation model using the modified Streeter-Phelps equation is developed for a simple stream system. The model solves the partial differential equations by means of a two-step finite difference technique which overcomes certain difficulties inherent in the standard finite difference method. Calibration of the model is undertaken with field quality data obtained in an intensive sampling survey and verification is attempted using data from a separate survey; a new method of calibration using linear programming techniques to minimise the errors between observed and simulated values is tested. In spite of a somewhat inadequate database, the results are reasonably good and encouraging for further work in this field.

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LIST OF ABBREVIATIONS

Abbreviation	Description
A.M.S.L.	Above mean sea level
BOD	Biochemical oxygen demand
BOD_5	Oxygen demand from 5-day BOD test
BOD_{20}	Oxygen demand from 20-day BOD test
COD	Chemical oxygen demand
DO	Dissolved oxygen
MLVSS	Mixed liquor volatile settleable solids
NVSS	Non-volatile settleable solids
OD	Oxygen demand
SUR	Substrate utilisation rate (F/M ratio)
SVI	Sludge volume index
TOC	Total organic carbon
VSS	Volatile settleable solids

LIST OF SYMBOLS

Symbol	Description	Units
a'	Fraction of substrate removed and used for energy production	
A	Cross-sectional area	L^2
A1, A2, A3, A2A	Finite difference coefficients for BOD	
b	Micro-organism decay coefficient	T^{-1}
$b_{i,n}$	Observed BOD for minimum error calibration	ML^{-3}
b'	Rate of oxygen utilised for endogenous respiration rate	T^{-1}
B	General symbol for pollutant concentration	ML^{-3}
B1, B2, B3	Finite difference coefficients for DO	
$\bar{c}$	Mean cross-sectional concentration	ML^{-3}
C	Dissolved oxygen concentration	ML^{-3}
C_s	Saturation concentration for DO	ML^{-3}
$d_{i,n}$	Observed DO for minimum error calibration	ML^{-3}
D	Dissolved oxygen deficit	ML^{-3}
	Displacement of axis in derivation of sinusoidal function for photosynthetic production of DO	
E_s	Specific efficiency of activated sludge system	
F/M	Food to micro-organism ratio	
H	Mean depth of flow	L
I	Integer used in the two-step finite difference method to subdivide the time step into smaller time steps for stability of the calculation in step 2	
K	BOD removal rate constant	T^{-1}
K_1, K1	Effective deoxygenation rate coefficient in the river	T^{-1}
K_2, K2	Reaeration rate coefficient	T^{-1}
k	Maximum rate of waste utilisation per unit weight of micro-organisms	$T^{-1}\ M^{-1}L^3$

LIST OF SYMBOLS (continued)

Symbol	Description	Units
k_1'	Laboratory BOD decay rate coefficient	T^{-1}
k_s'	Biological extraction of BOD rate coefficient	T^{-1}
k_3'	$= K_1 - k_1$, rate coefficient incorporating all effects making up the difference between laboratory and field estimates of deoxygenation coefficients	T^{-1}
K_s	Waste concentration at which rate of waste utilisation per unit weight of micro-organisms is one half the maximum rate	ML^{-3}
l	Characteristic length in convective length calculation (section 7.4.2)	L
L	BOD concentration	ML^{-3}
L_o	Initial BOD concentration	ML^{-3}
L_c	Convective length for dispersion studies	
L_r	Length of reach	
M	Month of year (= 1,2, ..., 12)	
$M_{i,n}$, $N_{i,n}$	Error terms for dissolved oxygen constraints in minimum error calibration method	ML^{-3}
P	Net dissolved oxygen sources and sinks $= P_1 + R_1$	$ML^{-3}T^{-1}$
P_t	Total sludge yield	MT^{-1}
P_x	Excess micro-organism production rate	MT^{-1}
P_w	Excess NVSS in wastage	MT^{-1}

LIST OF SYMBOLS (continued)

Symbol	Description	Units
P_1	Photosynthetic production of DO	$ML^{-3}\ T^{-1}$
P_i, R_i	Source and sink terms for dissolved oxygen constraints in minimum error calibration method	$ML^{-3}T^{-1}$
Q	Flow rate	L^3T^{-1}
Q_a	Flow rate through reactor	$L^3\ T^{-1}$
Q_e	Effluent flow rate	$L^3\ T^{-1}$
Q_f	Wastewater flow rate	$L^3\ T^{-1}$
Q_r	Activated sludge recycle rate	$L^3\ T^{-1}$
Q_w	Rate of sludge wastage	$L^3\ T^{-1}$
r	Recycle ratio ($= Q_r/Q_f$)	
R	Hydraulic radius	L
R_1	Constant dissolved oxygen source/ sink term	$ML^{-3}\ T^{-1}$
S	General term for substrate concentration	ML^{-3}
	Pollutant source/sink term	$ML^{-3}\ T^{-1}$
S_1	Substrate concentration of fresh feed	ML^{-3}
S_o	Substrate concentration of combined feed	ML^{-3}
S_e	Substrate concentration after biological treatment	ML^{-3}
S_n	Concentration of non-biodegradable matter	ML^{-3}
S_f	Energy slope	
S_i, T_i	Source/sink terms for BOD constraints in minimum error calibration method	$ML^{-3}\ T^{-1}$
t	Time	T
t_p	Time of passage	T
T	Temperature	^oC

LIST OF SYMBOLS (continued)

Symbol	Description	Units
U	Mean river velocity along a reach	LT^{-1}
$\bar{u}$	Mean flow velocity across a section	LT^{-1}
u^*	Shear velocity ($=\sqrt{gRS_f}$)	LT^{-1}
$U_{i,n}$, $V_{i,n}$	Error terms for BOD constraints in minimum error calibration method	ML^{-3}
V	Reactor volume	L^3
V_o	Occupied channel volume	L^3
X	Concentration of organisms in the reactor	ML^{-3}
X_f	Concentration of organisms in fresh feed	ML^{-3}
X_e	Concentration of organisms in the final effluent	ML^{-3}
X_r	Concentration of organisms in the recycled sludge	ML^{-3}
X_{no}	Concentration of NVSS in combined feed	ML^{-3}
X_{nf}	Concentration NVSS in fresh feed	ML^{-3}
X_{ne}	Concentration NVSS in final effluent	ML^{-3}
X_{ne}	Concentration of NVSS in recycled sludge	ML^{-3}
Y	Growth yield coefficient, i.e. mass of organism synthesised per unit mass of substrate utilised	ML^{-3}
α	'fudge' factor governing the conversion of light energy into DO production	
ε	Longitudinal dispersion coefficient	$L^2\ T^{-1}$
θ	Hydraulic retention time	T
θ_c	Sludge age or solids retention time	T
σ_x^2	Variance of concentration distribution with respect to distance along the stream	L^2

CHAPTER 1

INTRODUCTION

1.1 Background

The scarcity of South Africa's water resources is well appreciated, if not so much by the layman then at least by planners. With the development of a more industrialised society and a relatively high population growth the problem of fresh water supplies into the twenty-first century presents a formidable challenge to the engineers and decision makers involved.

1.1.1 The Pretoria-Witwatersrand-Vereeniging (PWV) Complex

The PWV complex, by far the largest of the industrial growth points in South Africa, has a current water consumption averaging 1 700 megalitres per day, and is increasing at a rate of six percent per year (Stephenson, 1978a). The Witwatersrand region stretches along a ridge running in an east-west direction which is the source of a number of streams flowing both to the north towards the Limpopo River and to the south towards the Vaal River. Since the Rand Water Board, which supplies the PWV complex, draws a substantial portion of the water from the Vaal Barrage, this location results in the area being probably the only major industrial centre in the world which is not situated on a major source of water; in fact it actually pollutes its own water source by discharging its effluent into the Vaal tributaries.

Such a situation clearly lends itself to engineering systems analysis for the optimal planning of the resources. Stephenson (1978a) has outlined such a system described by a series of equations and constraints which are linear except for the pollution load balance equations. He considers the water requirements of the Witwatersrand being met by (a) surface resources, (b) partially treated wastewaters returned to the river or (c) reclaimed wastewaters. The number of combinations of these resources that could be employed is immense and Stephenson shows how the problem could be formulated for optimisation using linear programming and separable programming techniques.

Clearly water quality is of prime importance in the system, since various degrees of treatment are required for each of the three sources of supply mentioned above, as well as for water discharged out of the system for downstream users.

A regional model would thus consist of an overall optimisation model with a series of sub-models for both simulation and optimisation of all the sub-systems comprising the various rivers down to the individual water and wastewater treatment plants. Such a model would need to simulate and optimise both water quantities and qualities in order to allow optimal planning of the location and sizing of water and wastewater treatment plants and sewers, as well as standards of effluent quality required.

It was with such a model in mind that the study reported on here was undertaken, with the following objectives:

(a) to formulate an optimisation model for a sewage treatment plant, and

(b) to develop a simulation model of the dissolved oxygen balance in a stream downstream of the effluent discharge point.

1.2 A note on measures of oxygen demand

Considerable criticism has been levelled at the use of the biochemical oxygen demand (BOD) test as a measure of the carbonaceous energy content (in terms of oxygen demand) of waters and wastewaters. Alternative measures are the chemical oxygen demand (COD) and total organic carbon tests (TOC). The merits and demerits of these various tests, with regard to river quality modelling, are discussed in appendix B.

It must be pointed out at this stage though, that the kinetics of the reactions applicable to the biological purification process are dependent on the biodegradable oxygen demand. Non-biodegradable and very slowly biodegradable oxygen demands do not contribute to the processes concerned. Biodegradable oxygen demand is measured by the ultimate BOD test (20-day test) and in this study the term BOD is used for all discussion purposes.

It should therefore be borne in mind throughout that, unless otherwise specified, the unsubscripted term BOD refers here to ultimate biodegradable oxygen demand which for practical purposes is the same as the biodegradable fraction of the oxygen demand measured by the COD test.

CHAPTER 2

REVIEW OF PREVIOUS RESEARCH

The following review outlines some of the more important literature relating to this study. It is by no means a complete list of all works in the field, nor a complete list of all literature studied during the course of the writer's investigations; it is merely a mention of some of the important works contributing directly or indirectly to this study.

2.1 Modelling of the activated sludge process

The modelling of an activated sludge treatment plant is an extremely complex task and most models in the literature have been developed for a specific plant, and for a specific purpose. This is particularly so in the case of optimisation models which are frequently developed for plant operation purposes. However a few attempts at general sewage treatment models have been made.

McKinney (1962) developed a rational theory for the complete mix activated sludge process. His approach implies that the effluent quality depends on the hydraulic retention time, whereas Lawrence and McCarty (1970), in their unified theory, use the sludge age as the more important parameter. The latter work has tended to be more popular in recent papers.

Eckenfelder, in 'Water Quality for Practising Engineers' (1970), gives design equations for the activated sludge process in which the food to micro-organism ratio, F/M (see section 3.4), is kept within limits to ensure satisfactory settling, thus resulting in a short sludge age. Ramalho (1977) expands on this to develop a design mathematical model in which the governing criterion may be either the F/M ratio or the substrate removal rate. In his procedure, sludge age does not feature and he uses hydraulic retention time as a basic design parameter. Metcalf and Eddy (1974) in their book 'Wastewater Engineering' outline a design procedure following closely the model of Lawrence and McCarty with the hydraulic sludge age forming a basic parameter.

In South Africa, Pitman (1975) showed that excellent settling and dewaterability could be obtained at longer sludge ages than those required to keep the F/M ratio within the limits recommended by Eckenfelder. For local conditions the most significant work has been that done at the University of Cape Town under Professor G.v.R. Marais. In 'The Activated Sludge Process, Part I, Steady State Behaviour' by Marais and Ekama (1976), the Lawrence and McCarty model is developed for long sludge ages which have tended to become more popular for activated sludge design in this country in recent years. In that paper the authors also advocate the use of chemical oxygen demand (COD) rather than biochemical oxygen demand (BOD) as a measure of the energy content, in terms of oxygen, of the wastewater. At long sludge ages the accumulation of non-biodegradable matter becomes a factor for consideration and therefore requires the separation of this inert fraction into settleable and dissolved portions. Marais and Ekama emphasised the fundamental importance of sludge age as a parameter and the hydraulic control of sludge age by direct wasting from the reactor was recommended. This latter method was first suggested by Walker (1971). A subsequent paper by Ekama and Marais (1977) develops a model for the dynamic behaviour of the activated sludge process.

A few general optimisation models have been developed. The CIRIA model (CIRIA (1970)) for cost effective sewage treatment includes primary, secondary and tertiary treatment of the liquor stream, as well as thickening, digestion and disposal of the sludge stream. Middleton and Lawrence (1976) developed a least cost model (based on the Lawrence and McCarty relationships) and found that the choice of sludge age did not significantly affect the total system cost and could therefore be selected freely for desired effluent quality (sludge ages up to twenty days were considered). Leslie Grady (1977) developed a simplified optimisation method using a dynamic programming procedure which can be performed on a programmable calculator.

2.2 Modelling the dissolved oxygen balance in streams

Phelps (1944) describes much of the early knowledge of the processes of stream pollution, as well as detailing the development of the well known Streeter-Phelps equations and their analytical solution in the form of the dissolved oxygen sag equation. Velz, in his book 'Applied Stream Sanitation' (1970), proposes a 'rational method' of determining the dissolved oxygen sag curve which he considers is preferable to the simplified dissolved oxygen sag equation of Streeter and Phelps.

Numerous authors have developed models based on the Streeter-Phelps equations, but modified to include other factors such as dispersion, nitrification, algal photosynthesis and respiration, benthal demand, sludge deposits and biological demands; many of the latter effects are frequently lumped together in a single source/sink term.

Thomann (1972) develops analytical solutions to include the above effects in a systems approach, as do Rinaldi, et al (1979). Analytical solutions are also used by Li (1962), Gunnerson and Bailey (1963), Holley (1969) and Fan, et al (1971), whereas Chevereau (1973) used numerical techniques to solve the equations and used COD to represent the ultimate BOD of the water.

The two-step explicit finite difference method used by Chevereau was initially developed for the BOD-DO equations by Dresnack and Dobbins (1968) to overcome certain difficulties inherent in the standard finite difference methods. Bella and Dobbins (1968) also studied the difference modelling of stream pollution in great detail.

Beck and Young (1975) developed a lumped parameter BOD-DO model conceptualised by a transportation delay system plus a continuously stirred tank reactor. An important aspect of their model was that it used daily BOD data, thereby recognising the difficulties involved in obtaining more frequent data over long time periods. De Boer (1976, 1979) used the method of characteristics in a moving cell model of the dissolved oxygen and other quality parameters, thereby avoiding the problem of numerical dispersion.

Dobbins (1964) investigated methods of determining the various constants in the BOD-DO equations and proposed a rational theory for estimating the surface reaeration rate. Camp (1965) on the other hand found that reaeration was small compared to photosynthetic production of dissolved oxygen, and also concluded that the removal of BOD by settling could be very large compared to the removal by biochemical oxidation. O'Connor and Di Toro (1970) studied the diurnal variation of photosynthetic production of dissolved oxygen and developed an oxygen balance model that includes this effect. Increases in DO during the night were reported by Gunnerson and Bailey (1963) and this was attributed to variations in the algal respiration rate. Edberg and Hofsten (1973) described <u>in situ</u> and laboratory tests on oxygen uptake of bottom sediments and found that the latter gave consistently lower values than the former. A number of models for estimating reaeration rates have been proposed (O'Connor and Dobbins, 1958; Churchill and Buckingham, 1962; Bansal, 1973; Foree, 1976).

CHAPTER 3

WASTEWATER TREATMENT PLANT MODELLING

In keeping with the initial idea of developing a regional optimisation model to be used for the optimal siting and sizing of sewage works, consideration was given at an early stage to the development of a simple sewage works optimisation model. It was these investigations that ultimately led to the writer's decision to abandon the idea of the development of such an optimisation model in favour of a simulation study confined to the river only. The reasons for this decision will become clear later in this chapter.

3.1 Choice of wastewater treatment process to be modelled

Two important means of biological sewage treatment used in large works today are:

(i) The trickling filter process, and

(ii) the activated sludge process.

(i) The trickling filter process is based on the fact that when sewage trickles over a suitable contact surface, a slimy zoogleal film forms on the surface due to the development of growths of bacteria and other micro-organisms. These films have the ability (1) of adsorbing matter that was previously held in suspension or solution in the sewage, (2) of abstracting from the adsorbed substances the energy or nutriment necessary for film maintenance and growth, and (3) of transferring back to the liquid the end products of decomposition (Imhoff and Fair, 1964, p.91).

The trickling filter usually consists of a circular structure packed with a suitable contact surface medium (such as crushed stone, gravel or clinker). The sewage is sprayed into the upper surface of the packing by means of fixed nozzles or a rotating sprinkler and is allowed to trickle through the packing into underdrains below. It then passes through a sedimentation tank after which it may undergo further biological treatment by means of either recycling or a second trickling filter tank.

Advantages of the trickling filter process are its ability to withstand shock loads, its simple operation and the fact that no artificial aeration is required.

(ii) In the activated sludge process, growths of biological floc develop spontaneously during the process of aeration and this floc tends to settle out to form a sludge when aeration and agitation of the mixture is stopped. A certain amount of this sludge is wasted and the rest is recycled to be mixed with the influent sewage. This latter process has been found to greatly increase the rate at which flocculation takes place and results in a high degree of purification.

The activated sludge process therefore consists of four main stages following primary treatment:

1. Aeration in a reactor tank. This is achieved by means of diffused air units or mechanical aerators.

2. Sludge separation by means of a secondary settling tank.

3. Recycling of settled sludge to be mixed with the influent sewage to the reactor.

4. Thickening and wastage of a portion of the sludge for disposal.

The activated sludge process is considered to be more economical than trickling filters for high BOD removal efficiencies (Ramalho, 1977, p.69) due to the high cost of packing material in the latter process.

For major sewage treatment plants, the tendency in South Africa in recent years has been towards the activated sludge process. A considerable amount of local research has gone into this process mainly because it has been found that design criteria applied overseas are often not applicable to South African conditions. It was therefore decided to use the activated sludge process for the optimisation model.

3.2 Basics of the design of activated sludge processes

3.2.1 Mixing regimes

The behaviour of the activated sludge process is influenced by the method of mixing applied in the reactor. The two extremes of the mixing regimes involved are:

(a) Plug flow in which the raw sewage may be considered to enter at one end of a rectangular reactor tank and, without mixing, emerge at the other end in the same sequence in which it entered.

(b) Completely mixed system in which the influent sewage is assumed to undergo immediate dispersion in the reactor and the reactor effluent is assumed to be representative of the mixed liquor throughout the reactor.

McKinney (1962) was the first to develop the mathematics of the completely mixed process, followed by the work of Lawrence and McCarty (1970). In practice the activated sludge process