

Extended aeration activated sludge process oxygen requirements

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EXTENDED AERATION ACTIVATED SLUDGE

PROCESS OXYGEN REQUIREMENTS

BY

ROBERT W.T. CHONG

**Submitted for the degree of
Master of Engineering Science
(18 credit project report)**

1986

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I would also like to thank all others who have helped with this project but not mentioned above.

ABSTRACT

A detailed and critical review on the theoretical determination of the process oxygen requirement of the extended aeration activated sludge process was carried out. The theoretical carbonaceous oxygen requirement (COR) can be calculated either from the influent carbonaceous substrate ultimate BOD or by the traditional synthesis and endogenous respiration equation. The former method was considered to be more theoretically sound than the latter method. The latter method was considered to be empirical in nature as the traditional model failed to recognise a critical feature of substrate limited process such as the "conventional" and extended aeration modes of the activated sludge process. It was argued by the writer that in a substrate limited process, the active biomass is limited as well as being directly proportional to the influent substrate mass. Based on this argument, it was hypothesised that the actual process "F/M" ratio based on the actual mass of substrate consumed and the active biomass is constant regardless of the value of the traditional empirical "F/M" (kg BOD₅/kg MLSS.d) ratio or sludge age. The new hypothesis was supported by reported literature data on the active biomass fraction of MLSS.

An oxygen mass balance was conducted on a full-scale intermittent extended aeration (IEA) plant to verify the process oxygen requirement calculated by the two methods mentioned above. It was found that the theoretical process oxygen requirement calculated by the former method agreed with the measured value obtained by the oxygen mass balance. The latter traditional method grossly overestimated the process oxygen requirement when typical coefficient values of the "conventional" process were used.

In order to conduct the oxygen mass balance for the IEA plant, aeration tests in clean water and dirty water (MLSS) were conducted. A critical review was also conducted to determine the most reliable method to interpret the aeration tests data. It was shown that the non-linear least squares parameters estimation technique was the most reliable as it provides the least squares estimates of all the oxygen

transfer parameters - including the dissolved oxygen saturation value which is not provided by other methods. Also, a new general aeration test model applicable to both surface and submerged aeration tests was developed. This new model allows the non linear least squares parameter estimation technique to be used for interpreting submerged aeration test data.

LIST OF ABBREVIATIONS

ADWF	Average dry weather flow
amm.N	Ammonia Nitrogen
ATP	Adenosine Triphosphate
BOD	Biochemical Oxygen Demand
BWL	Bottom water level
COD	Chemical Oxygen Demand
CODmn.	Permanganate COD (P.V.)
conc.	Concentration
COR	Carbonaceous oxygen requirement
DNA	Deoxyribonucleic acid
DO	Dissolved oxygen
Eff.	Effluent
ep.	Equivalent person
Eq.	Equation
F/M	Food to Microorganism Ratio
IEA	Intermittent Extended Aeration
Inf.	Influent
MCRT	Mean Cell Residence Time
MLSS	Mixed Liquor Suspended Solids
MLVSS	Mixed Liquor Volatile Suspended Solids
NOD	Nitrification oxygen demand
NOR	Net Nitrogenous Oxygen Demand
N.S.W.	State of New South Wales
nite.N	Nitrate Nitrogen
org.N	Organic nitrogen
OTE	Oxygen Transfer Efficiency (%)
OTR	Oxygen Transfer Rate
OUR	Oxygen Uptake Rate
Prim.sed.	Primary sedimentation
P.W.D.	Public Works Department
R.H.	Relative Humidity
RLU	Relative Light Unit
SOTE	Standardised OTE

LIST OF ABBREVIATIONS (Cont'd)

SOTR	Standardised OTR
SS	Suspended Solids
SVI	Sludge Volume Index
temp.	Temperature
Theo.	Theoretical
tot.N	Total nitrogen
TWL	Top Water Level
VSS	Volatile SS

NOTATIONS

ACTIVATED SLUDGE

a	= carbonaceous substrate respiration coefficient, kg O ₂ /kg BOD ₅ removed
b	= endogenous respiration coefficient, kg O ₂ /kg activated sludge
BOD _a	= available substrate BOD, mg/l
BOD ₅	= 5 day BOD, mg/l
BOD _t	= t day BOD, mg/l
BOD _u	= ultimate BOD, mg/l
BVSS	= influent biodegradable volatile suspended solids conc., mg/l
COR	= carbonaceous oxygen requirement, kg/d or kg/h.
f _b	= biodegradable cell fraction
f _{bvss}	= biodegradable MLVSS fraction
ISS	= influent inorganic SS conc., kg/kl
k	= BOD reaction time constant, d ⁻¹
k _d	= cell endogenous decay rate, d ⁻¹
k _p	= particulate substrate degradation rate, d ⁻¹
NBVSS	= influent nonbiodegradable VSS, kg/kd
NOR	= nitrogenous oxygen demand, kg/d
P _{DN}	= fraction nitrate denitrified
P _N	= fraction total nitrogen nitrified
Q	= influent flow rate, kl/d
Q _w	= waste sludge (MLSS) flow rate, kl/d
R	= oxygen consumption rate, kg/d
S	= carbonaceous substrate, kg/d
S _a	= actual available substrate, kg/d
t	= time, d
TOR	= total oxygen requirement, kg/d or kg/h
tot.N	= influent total nitrogen concentration, kg/d

U	=	modified F/M, kg BOD ₅ /kg MLVSS.d
V	=	intermittent extended aeration plant volume at BWL, kl
X	=	activated sludge mass, kg
X _a	=	activate bacterial cell mass, kg
X _e	=	effluent SS conc., kg/kl
x	=	fraction of MLSS consisting of accumulated influent nonbiodegradable VSS
Y	=	bacterial cell growth yield coefficient, kg cell produced/kg substrate utilised
θ _c	=	sludge age or MCRT, d
ΔBVSS	=	rate of accumulation of influent biodegradable VSS, kg/d
ΔS	=	carbonaceous substrate removal rate, kg/d
ΔX	=	rate of change of activated sludge, kg/d
ΔX _a	=	rate of change of active cell mass, kg/d

NOTATIONSAERATION TEST

- α = $K_L a' / K_L a$, ratio of MLSS to clean water $K_L a$
 β = C'_s / C^* , ratio of MLSS to clean water DO saturation concentration
 θ = temperature correction coefficient
 θ^* = 'true' temperature correction coefficient
 γ = weight density of water, KPa/M
 θ_d = oxygenation coefficient
 τ = $1/k_L a$, time constant, min^{-1} or h^{-1}
 τ_1 = $1/k_1$, time constant for C in clean water test
 τ_2 = $1/k_2$, time constant for C^* in clean water test
 τ'_1 = $1/k'_1$, time constant for C in MLSS test
 τ'_2 = $1/k'_2$, time constant for $C^{*'}_s$ in MLSS test
C = DO concentration in clean water. mg/l
 C_{ave} = average DO concentration in a small time interval. mg/l
 C_o = DO concentration at $t = 0$ which is the start of aeration testing. mg/l
 C_m = measured (observed) DO concentration, MG/l
 C_p = predicted (calculated) DO concentration, mg/l
 C^* = DO saturation concentration in clean water, mg/l
 C_o^* = C^* at $t = 0$ (start of test), mg/l
 C_{∞}^* = C^* at $t = \infty$ (end of test), mg/l
 C_{co}^* = C^* at $c = 0$, mg/l
 C_{ST}^* = book value surface C^* at 1 atmospheric pressure and $T^\circ\text{C}$
 C_{S20}^* = book value surface C^* at 1 atmospheric pressure and 20°C
 C_{LM}^* = logarithmic C^* , mg/l
 C_{btm}^* = C^* at bottom of tank, mg/l
 C_{top}^* = C^* at top of tank, mg/l
 C'_s = DO saturation in MLSS, mg/l

NOTATIONS (Cont'd)

- C_s^* = apparent C_s^* achieved at the end of the 'dirty' water or MLSS aeration test, mg/l
 C_{so}^* = C_s^* at $t = 0$, mg/l
 C_{soo}^* = C_s^* at $t = \infty$, mg/l
 d = water depth, m
 d_e = effective saturation depth (correction of book value surface C_s^* to this depth will yield a value equal to the test C_{soo}^*), m
 F_o = dimensionless exist gas depletion factor evaluated at zero DO concentration
 G_F = air mass flow rate, kg/min.
 H = Henry's Law constant, mg/l kPa
 $K_L a$ = overall volumetric oxygen transfer coefficient in clean water, min^{-1} or h^{-1}
 k = $k_L a$ in short
 k' = apparent k
 k_1 = $1/\tau_1$, (τ_1 = C time const.)
 k_2 = $1/\tau_2$ (τ_2 = C^* time const.)
 $K_L' a$ = $K_L a$ in MLSS (or dirty water)
 M_a = molecular weight of air, g
 M_o = molecular weight of oxygen, g
 n = number of residual readings
 OTR = oxygen transfer rate in clean water, kg/h
 OTR' = oxygen transfer rate in MLSS under field conditions, kg/h
 P_b = test atmospheric pressure, kPa
 P_s = standard atmospheric pressure, kPa
 P_{VT} = test saturated water vapour pressure, kPa
 P_{V20} = saturated water vapour pressure at 20°C , kPa

NOTATIONS (Cont'd)

R	= residual, difference between predicted and measured C, mg/l
$\sum R^2$	= sum of squares of R
r	= MLSS DO uptake rate, mg/l min. or mg/l.h.
SOTE	= standardised oxygen transfer efficiency, percent oxygen transferred from air feed to clean water at 1 atmospheric pressure, 20°C water temp. and zero DO conc.
SOTR	= standardised oxygen transfer rate to clean water
t	= time, min.
t ₀	= time zero at start of test
t _∞	= infinite time at end of test when dynamic equilibrium is established
T	= test water temperature, °C
V	= test water volume, kl or l
Y _d	= mole fraction of oxygen in air feed

Subscript terms:-

T	= test temp. values
20	= 20°C water temp. values
0	= zero time values
∞	= infinity time values
'	= MLSS or dirty water test values
1	= C function values
2	= C [*] function values

TABLE OF CONTENTS

	<u>PAGE</u>
List of Abbreviations	(vi)
List of Notations - Activated Sludge Process	(viii)
- Aeration Test	(x)
 PART 1: THEORETICAL DISCUSSIONS	
 1. <u>INTRODUCTION</u>	 1
 2. <u>PROCESS OXYGEN REQUIREMENTS</u>	 4
2.1 <u>Carbonaceous Oxygen Requirement (COR)</u>	4
2.1.1 Theoretical COR	4
2.1.2 Synthesis and Endogenous Oxygen Requirement	11
2.1.3 General Review of Activated Sludge Theory	16
2.1.3.1 F/M Ratio	22
2.1.3.2 Sludge Age	26
2.1.4 Closure for Section 2.1	32
 2.2 <u>Net Nitrogenous Oxygen Requirement (NOR)</u>	 36
2.2.1 Nitrification	37
2.2.2 Denitrification	39
2.2.3 Net Nitrogenous Oxygen Requirement	40
2.2.4 Closure for Section 2.2	40
 2.3 <u>Total Oxygen Requirement (TOR)</u>	 41
2.3.1 Closure for Section 2.3	44
 References (Section 2)	 45

TABLE OF CONTENTS (Cont'd)

	<u>PAGE</u>
3. <u>AERATION TESTS</u>	47
3.1 <u>Clean Water Aeration Test</u>	48
3.1.1 Surface Aeration Test	49
3.1.2 Submerged Aeration Test	51
3.1.3 Development of a New Submerged Aeration Test Model	54
3.1.3.1 C [*] Function	54
3.1.3.2 C Function	56
3.1.4 Numerical Examples	59
3.1.4.1 Method 1 - Nonlinear least squares Parameters Estimation Based on Eq. 5	60
3.1.4.2 Method 2 - Nonlinear least squares Parameters Estimation Based on Eq. 23	63
3.1.4.3 Method 3 - Log-Mean Saturation Direct Method	64
3.1.5 Discussion of Results	67
3.1.5.1 Comparisons of Parameters Estimates Obtained by Methods 1 and 2	67
3.1.5.2 Comparisons of SOTR obtained by Methods 1, 2 and 3	75
3.1.6 Further Developments of Submerged Aeration Test Model	78
3.1.7 Closure for Section 3.1	83
3.2 <u>Dirty Water Aeration Test</u>	84
3.2.1 Closure for Section 3.2	87
3.3 <u>Specifying Aeration Equipment Oxygenation Capacity</u>	88
3.3.1 Closure for Section 3.3	90
References (Section 3)	92

TABLE OF CONTENTS (Cont'd)

	<u>PAGE</u>
PART 2: EXPERIMENTAL WORK	93
4. <u>PLANT DESCRIPTION</u>	94
5. <u>BIODEGRADABLE VSS FRACTION</u>	97
5.1 Biodegradable MLVSS Fraction	97
5.2 Biodegradable Sewage VSS Fraction	99
6. <u>MLSS ACTIVE BIOMASS FRACTION</u>	101
6.1 Test Procedures	102
6.2 Test Results	105
7. <u>PLANT LOADINGS</u>	108
7.1 Hydraulic Loading	108
7.2 Carbonaceous Substrate Loading	109
7.2.1 BOD ₅ Loading	111
7.2.2 COD Loading	111
7.2.3 BOD _u Loading	112
7.3 Nitrogen Loading	114
8. <u>PLANT PERFORMANCE</u>	116
8.1 General Effluent Quality	116
8.2 Percentage Substrate Biodegradation	118

TABLE OF CONTENTS (Cont'd)

	<u>PAGE</u>
9. <u>PLANT PROCESS OXYGEN REQUIREMENT</u>	126
9.1 Theoretical COR Results	126
9.2 Theoretical TOR Results	129
10. <u>PLANT AERATION TESTS RESULTS</u>	135
10.1 Clean Water Aeration Test	135
10.1.1 Test Procedures	135
10.1.2 Test Results	136
10.1.3 SOTR Calculations	139
10.2 Dirty Water (MLSS) Aeration Test	143
10.2.1 Test Procedures	143
10.2.2 Test Results	144
10.2.3 OTR' Calculations	148
11. <u>PLANT OXYGEN BALANCE</u>	153
12. <u>CONCLUSIONS</u>	162
References (Sections 4 to 11)	171
Appendix A - ATP Test Data	
" B - 24h chart recording of water level in No. 1 IEA Tank	
" C1 to C4 - Clean Water Aeration Test Results for DO Probe No. 1 to No. 4 respectively	
" D1 to D4 - Clean Water Aeration Test Plots of C Vs t for DO Probes No. 1 to No. 4 respectively.	
" E1 to E3 - Plots of OUR test results for Samples No. 1 to No. 3 respectively.	

TABLE OF CONTENTS (Cont'd)

Appendix

- " F1 to F4 - Dirty Water (MLSS) Aeration Test Results for DO Probes No. 1 to No. 4 respectively.
- " G1 to G4 - Dirty Water (MLSS) Aeration Test Plots of C' Vs t for DO Probes No. 1 to No. 4 respectively
- " H - 22h chart recording of MLSS DO concentration in IEA Tank No. 1.
- " I - Computer Programme for nonlinear least squares parameter estimation for surface and submerged aeration test.
- " J - Transient Response of First Order System

EXTENDED AERATION ACTIVATED SLUDGE

PROCESS OXYGEN REQUIREMENTS

PART 1: THEORETICAL DISCUSSIONS

1. INTRODUCTION

The selection of the aeration equipment is a vital component in the design of an activated sludge plant. The aeration equipment must have sufficient capacity to provide the process oxygen requirement. However, oversizing the oxygenation capacity of the aeration equipment can lead to operational problems as it would make it difficult to maintain the anoxic conditions required for denitrification to proceed. In the extended aeration mode of the activated sludge process, nitrification is an inherent part of the process because of the high sludge age - in excess of 10 days. Hence, in an extended aeration process, denitrification is most desirable as it:

- (i) Recovers part of the oxygen taken up during nitrification and thus reduces aeration costs.
- (ii) Recovers part of the alkalinity taken up during nitrification and thus prevents lowering the Mixed Liquor Suspended Solids (MLSS) pH to undesirably low levels.
- (iii) Denitrification reduces the nitrogen level in the plant's effluent.

Also, excessive aeration due to oversized oxygenation capacity would add unnecessary costs to the plant's operation. The aeration energy cost is a major component of the operation costs of an extended aeration plant. This is especially so for the intermittent extended aeration plant where labour cost is reduced to a minimum.

In order to properly prepare the specification of the aeration equipment the process design engineer has to firstly determine the process oxygen requirement. The process oxygen requirement is generally expressed as an oxygen transfer rate in kg of oxygen per hour necessary to satisfy the peak diurnal process oxygen requirement. The process oxygen transfer rate then has to be converted to the standard oxygen transfer rate in clean (tap) water at

the standard conditions of 20°C water temperature, barometric pressure of one atmosphere, and zero dissolved oxygen concentration in the water. There are several reasons for specifying the oxygen transfer rate of the aerator in clean water. The main reasons are:

- (i) It would be preferable to test the aerators before final commissioning of the plant so that any modifications or replacement of the aerators can be carried out before wastewater is introduced into the plant.
- (ii) Aerator manufacturers are more confident in giving aerator performance guarantees in clean water than in MLSS because of:
 - (a) Uncertainties in the α and β factors which can vary with process conditions, wastewater characteristics and the types of aerators.
 - (b) Aerators are generally developed and tested with clean water due to the unavailability of treatment plants to manufacturers for testing purposes.
- (iii) The clean water aeration test is much easier to carry out than MLSS or "dirty" water test. Process conditions are difficult to control or to specify contractually for the dirty water aeration tests. Also, there could be a delay of several months to years between the installation and testing of the aerators under the design and specified process conditions as most plants will be underloaded in the early years.
- (iv) The clean water test is better established than the dirty water test.

The aim of this project is to determine the oxygen requirement in the extended aeration process which is not as clearly established as that in the "conventional" activated sludge process. The great majority of research and published literatures are on the conventional

process with F/M ratios in the range of 0.2 to 0.4 kg BOD₅/kg MLVSS.d. The adoption or extrapolation of these conventional process kinetic constants and mathematical equations to the extended aeration process with F/M ratios of less than 0.1 can be fraught with danger. This is because many of the mathematical relationships are empirically established and the kinetic constants are empirical rather than true constants. For example, the fundamental Michaelis-Menton enzymatic kinetic equation which was applied by Monod to the substrate utilization by microorganisms was derived empirically. Also, most research is carried out in laboratory or pilot plant units using soluble substrate whereas the substrate in domestic sewage is mainly particulate.

Last but not least is the use of Volatile Suspended Solids (VSS) as a measure of the amount of active biomass in the system. It is easily recognised that the VSS in a municipal wastewater aeration tank is an accumulation of active or viable biomass, non-viable biomass, undegraded particulate substrate, and non-biodegradable organics (eg. cellulose, lignin, plastic, etc.) and that the viable biomass is only a small fraction of the VSS. Hence, the mathematical relationships developed empirically for the activated sludge process may be linear over only a narrow range of F/M ratios and not over the whole range from the "conventional" F/M of 0.2 to 0.4 to the extended aeration F/M of 0.04 to 0.1. It can be shown that using the traditional synthesis and respiration equation for process oxygen requirements established for the "conventional" process can result in over estimating the oxygen requirement in the extended aeration process.

This report is divided into two parts - Part 1 dealing with the theory and Part 2 comparing the theoretical results with actual data from full scale municipal wastewater plants. In the theoretical section, the determination of the extended aeration oxygen requirement through to the testing of aerator performance is examined. It will be pointless to determine accurately the oxygen requirement if the aerator cannot be tested accurately to see that it can meet the oxygen requirement.

2. PROCESS OXYGEN REQUIREMENT

In the extended aeration process oxygen must be supplied to meet both the carbonaceous oxygen requirement in the oxidation of the carbonaceous substrate and the nitrogenous oxygen requirement in the nitrification process. From this can be subtracted the oxygen recovered in the denitrification process. The total process oxygen requirement can then be used to determine the required oxygen transfer rate of the aerator in kg of oxygen per hour. The hourly oxygen transfer rate may be required to be increased to meet the plant's diurnal peak loading.

2.1 Carbonaceous Oxygen Requirement (COR)

The COR can be determined from the theoretical oxygen requirement of the substrate or from the traditional synthesis and endogeneous respiration relationship which is derived empirically. Both methods are discussed in Sections 2.1.1 and 2.1.2 below.

2.1.1 Theoretical COR

The reaction involved in the biological treatment of carbonaceous wastewater is a combination of three stages as follows:

1. The first stage of the reaction is the transfer of the carbonaceous substrate from the wastewater to the bacterial floc by interfacial contact and associated adsorption and absorptions. This operation is fast and is generally effected in a matter of minutes (1 to 20 minutes).
2. In the second stage the substrate is partially oxidised for energy (respiration) and partially converted into new bacterial cell (synthesis). This operation proceeds in a more leisurely pace in the solid phase of the biomass and generally takes from several hours to several days to complete.

3. In the third stage, with continued aeration, the biological cells that were synthesised undergo endogenous respiration to oxidise the biodegradable portion of their cells for maintenance energy required for survival. Thus the complete reaction generally takes more than 20 days to complete.

The theoretical maximum carbonaceous oxygen requirement would be the oxygen required for the above complete reaction. This is obtained as the ultimate biochemical oxygen demand (BOD_u) in the bottle BOD_5 test where the reaction in the bottle is continued beyond the normal 5 days until the reaction is completed (normally 20 days or more).

Thus the maximum COR for any activated sludge process would be equivalent to the BOD_u of the influent wastewater to be treated. Since the organic loading of domestic sewage is generally given or measured as the five day BOD, the BOD_5 of the sewage must be converted to the BOD_u to determine the process maximum COR. The BOD reaction can generally be represented by the following first order reaction.

$$BOD_t = BOD_u (1 - 10^{-kt}) \quad - (1)$$

Where t = BOD reaction time, T,
and k = BOD reaction time constant, T^{-1} .

Hence, the ratio between BOD_5 and BOD_u is given by

$$BOD_5/BOD_u = 1 - 10^{-k5} \quad - (2)$$

The value of k is dependent on the type of substrate with reported minimum and maximum values of 0.07 and 0.3 d^{-1} respectively. More readily biodegradable substrate would have a higher k value. For sewage the reported values are generally between 0.1 and 0.17 d^{-1} . The lower k value would most probably be for sewage with a higher percentage of particulate substrate (eg. unsettled compared to settled sewage) as the particulate substrate is more slowly degraded than the soluble fraction. If the k value of the sewage to be treated is unknown, the lower value of 0.1 d^{-1} should be adopted so as to err on

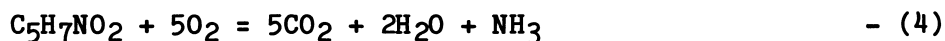
the high or safe side. Hence adopting the more conservative k value of 0.1 d^{-1} , the ratio of BOD_5 to BOD_u is

$$\text{BOD}_5/\text{BOD}_u = 0.68$$

The ratio of 0.68 based on k of 0.1 d^{-1} agrees with the generally reported literature value.

$$\therefore \text{Maximum COR} = \text{BOD}_u = 1.46 \text{ BOD}_5 \quad - (3)$$

The actual process COR would be somewhat less than the maximum COR as a portion of the substrate is converted to biological cells that are subsequently wasted from the system to maintain the design sludge age or MLSS concentration in the plant. The chemical composition of all biological cells is almost identical and can be represented by the chemical formulae $\text{C}_5\text{H}_7\text{NO}_2$. Therefore the BOD_u equivalent of a unit biological cell can be estimated from the complete oxidation of a mole of cell as follows:



$$113 + 5(32)$$

$$\therefore \text{BOD}_u/\text{unit cell mass} = 160/113 = 1.42$$

Therefore, the process COR is the ultimate BOD_u of the influent wastewater less the oxygen equivalent of the biological cell mass wasted from the system (and hence not oxidised) which can be represented as:

$$\text{COR} = 1.46 (\text{BOD}_5) - 1.42 f_b (\text{cell wasted}) \quad - (5)$$

The factor f_b in Eq. 5 represents the fraction of the cell that is biodegradable as it has been well established that a portion of the cell is not biodegradable. The nonbiodegradable fraction of the cell reportedly varies from a low of 8% by Dold et al (ref. 1) to a high of 23% by Kountz and Formey (ref. 2). A value of 10% has been reported by Eckenfelder (ref. 3).

In practice, the biological cell mass is measured by the volatile suspended solids (VSS) and Eq. 5 may be rewritten as:

$$\text{COR} = 1.46 (\text{BOD}_5) - 1.42 f_{b\text{VSS}} (\text{VSS wasted}) \quad - (6)$$

The factor $f_{b\text{VSS}}$ in Eq. 6 represents the biodegradable fraction of the VSS and is generally less than f_b due to the accumulation of nonbiodegradable VSS in the system. The accumulation of nonbiodegradable VSS results from:

- (i) The accumulation of nonbiodegradable biological cell mass due to endogenous respiration and death-lysis of the biological cells.
- (ii) The accumulation of nonbiodegradable VSS present in the influent (eg. cellulose).

McCarty and Broderon (ref. 4) have shown that the biodegradable portion of mixed liquor VSS (MLVSS) can vary from about 75% for a MCRT of about 3 days to about 40% for a MCRT of about 30 days. The results were obtained from laboratory studies treating soluble biodegradable substrates and the VSS would, hence, represents biological cell mass only. The biodegradable fraction will be further decreased by the accumulation of influent nonbiodegradable VSS. The nonbiodegradable portion of the VSS in sewage is very seldom reported. Eckenfelder (ref. 3) stated that the nondegradable COD in domestic sewage is low (usually less than 60 mg/L). Dold et al (ref. 1) found that a factor of 0.025 VSS/mg COD for the nondegradable particulate fraction of the influent COD fitted their general model for the activated sludge process. Their model is based on a structured sludge mass consisting

of active biomass, stored substrate biomass and an endogeneous biomass as well as distinguishing between soluble and particulate substrate and between biodegradable and unbiodegradable substrate COD (chemical oxygen demand). Based on these limited data, it can be estimated that approximately 10% of domestic sewage VSS is nondegradable. The fraction of the MLVSS that is composed of accumulated nondegradable influent VSS can be estimated by a mass balance at steady state. This can be illustrated by the following example of a typical 4000 equivalent population (ep) intermittent extended aeration (I.E.A.) plant, based on N.S.W. Public Works Department design criteria.

COR of 4000 ep I.E.A. Plant

Design Criteria:

- (i) BOD_5 of Sewage = 70 g/ep.d (about 290 mg/L)
SS of Sewage = 70 g/ep.d
VSS of Sewage = 75% of SS
 = 52.5 g/ep.d
 $F/M = 0.04$ kg BOD_5 /kg MLSS.d
Sludge Age = 30 days
Sewage Volume = 240 l/ep.d at ADWF
 (Average Dry Weather Flow)
Effluent BOD_5 = 15 mg/L

Effluent SS = 20 mg/L

BOD_5 loading = $4000 \times 0.07 = 280$ kg/d
MLSS = $280/0.04 = 7000$ kg
MLVSS = 70% MLSS = 4900 kg

VSS loading = $4000 \times 0.0525 = 210$ kg/d
Assume 10% influent VSS is nondegradable

.
. . nondegradable VSS loading = 21 kg/d

Mass balance of nondegradable influent VSS at steady state is given by:

$$x \text{ (MLVSS wasted/day)} = (\text{Nondegradable influent VSS/day})$$

Where x = fraction of MLVSS consisting of influent nondegradable VSS

$$\begin{aligned} & \cdot \\ \cdot \cdot \quad x &= \frac{\text{Nondegradable VSS loading/d}}{\text{MLVSS wasted/d}} \end{aligned}$$

For sludge age of 30 days, MLVSS wasted per day = $4900/30$
= 163 kg/d

$$\begin{aligned} & \cdot \\ \cdot \cdot \quad x &= 21/163 = 0.13 \end{aligned}$$

If it is assumed that all influent degradable VSS have been degraded (reasonable for sludge age of 30 days), then the remaining 87% of the MLVSS would consist of biological cell mass (both active and nonactive cells).

$$\begin{aligned} & \cdot \\ \cdot \cdot \quad \text{Biological cell mass in MLVSS} &= 0.87 \text{ MLVSS} \end{aligned}$$

Adopting McCarty and Broderickson (ref. 4) figure of 40% degradable cell mass for sludge age of 30 days, biodegradable fraction of the MLVSS is:

$$\begin{aligned} \text{Biodegradable fraction, } f_{bVSS} &= 0.4 \times 0.87 \text{ MLVSS} \\ &= 0.348 \text{ MLVSS} \end{aligned}$$

Thus, for a sludge age of 30 days, only about 35% of the MLVSS is biodegradable. This is quite an acceptable figure as it has been shown by various authors that the active mass is generally less than 20% of the MLSS for extended aeration process. Roe Jr. and Bhagat (ref. 5) using adenosine triphosphate (ATP) as a measure of active biomass found that as the MCRT increases, the active biomass fraction of the MLSS treating a soluble synthetic waste decreases and levels off at about 15% of the MLSS (MLVSS was found to be 89% of MLSS) for MCRT greater than 25 days. Nelson and Lawrence (ref. 6) quoted figures by other authors - Weddle & Jenkins (1971, ref. 15) reported 10 to 20% of MLVSS treating domestic sewage as being active mass; Patterson, Brezonik and Putman (1970, ref. 24) reported 40% viable MLVSS in a laboratory unit treating soluble synthetic substrate and 15 to 20% viability for MLVSS in a contact stabilization plant operating on domestic sewage; and Hutton (1974) reported 20% live biomass in a domestic sewage activated sludge. Nelson and Lawrence (ref. 6) using ATP as a measure of active MLVSS fraction found that at a MCRT of 12 days, the MLVSS of a laboratory activated sludge unit treating a soluble synthetic substrate consisted of about 75% degradable VSS (45% viable biomass and 30% nonviable biomass). This compares well with an estimate of 65% degradable MLVSS by McCarty and Brodersen (ref. 4). The consensus of the above authors is that both the active biomass fraction and degradable VSS fraction decrease with increasing MCRT. In extended aeration plants, the active VSS fraction would generally be less than 20% and the biodegradable VSS less than 40%.

With an estimated f_{bVSS} of about 0.35, the COR for this example can be estimated from Eq. 6 as follows:

$$\begin{aligned} \text{COR} &= 1.46 (280) - 1.42 \times 0.35 (163) \text{ kg O}_2/\text{d} \\ &= 328 \text{ kg O}_2/\text{day} \end{aligned}$$

Hence, the oxygen required per kg of BOD₅ load works out to be about 1.2 kg O₂/kg BOD₅ applied.

In this example, it is assumed that all the influent BOD₅ is removed and that the effluent BOD₅ is due mainly to effluent VSS and soluble effluent BOD₅ is negligible. Also the sludge age of 30 days is based on total MLSS wasted - ie. waste sludge stream SS plus effluent SS.

2.1.2 Synthesis and Endogenous Oxygen Requirement

Traditionally, the COR is estimated in two parts being that required for carbonaceous substrate respiration and for endogenous respiration. This is most commonly represented by the following equation:

$$R = aS + bX \quad - (7)$$

Where R = rate of oxygen consumption, kg/d

S = BOD₅ removed (assumed to be all metabolised), kg/d

X = mass of activated sludge in the system
(generally measured by MLSS or MLVSS), kg

a = empirical coefficient for carbonaceous
substrate respiration, kg O₂/kg BOD₅ removed

b = empirical coefficient for endogenous
respiration, kg O₂/kg activated sludge in system

The term X in Eq. 7 should strictly refer to the active biological cell mass only. Because of the difficulty in determining the active biomass in activated sludge, MLVSS and sometimes MLSS have been used for the term X in Eq. 7. Dividing by X , Eq. 7 becomes

$$R/X = a (S/X) + b \quad - (8)$$

Eq. 8 thus presents an empirical correlation of the specific oxygen uptake rate (R/X = oxygen consumption rate per unit mass of activated sludge) with the specific substrate utilisation rate (S/X = carbonaceous substrate removal rate per unit activated sludge mass). The specific substrate utilisation rate S/X , has also been defined as the modified F/M ratio, U , defined in terms of BOD removed rather than the BOD applied. The empirical coefficients 'a' and 'b' are usually determined from a plot of R/X versus S/X with the slope of the straight line being 'a' and the intercept being 'b'. The carbonaceous substrate S is usually measured as BOD_5 and the activated sludge X as $MLVSS$. Values of a and b would be different if other measurements such as COD and $MLSS$ are used. Some reported values of a and b are given in Table 1.

Table 1 - Kinetic Coefficients 'a' and 'b' Values

Substrate	a	b	(F/M)	Ref
Settled combined domestic and industrial	0.76/ BOD_5	0.13/VSS	(0.1-0.4)	7
Unsettled domestic	0.75/ BOD_5^*	0.03/VSS*	(0.12)*	8
Domestic	0.5/ BOD_5	0.1-0.15/VSS	Conventional	9
Domestic	0.5-0.6/ BOD_5	0.1/MLSS	-	10

* converted from COD_{MN}/BOD_5 ratio 0.5; $MLVSS/MLSS$ ratio 0.7. Also, a and b values of references 9 and 10 are illustrative values based on commonly quoted literature values.

As shown in Table 1, the reported values for 'a' are quite consistent with a range of about 50%. However, the reported values for 'b' are not consistent with a range of over 400%. Reported range for 'a' and 'b' quoted by Vasick (ref. 7) were 0.55 to 0.8 for 'a' and 0.03 to 0.17 for 'b'. Hence the COR calculated from Eq. 7 can be quite different depending on the values of 'a' and 'b' adopted. This can be illustrated by the example given in Section 1.1 for a 4000 ep I.E.A. plant as follows:

$$\begin{aligned} \text{Influent BOD}_5 &= 280 \text{ kg/d (assumed all metabolised)} \\ \text{F/M} &= 0.04 \text{ kg BOD}_5/\text{kg MLSS d} \\ \text{MLSS} &= 7000 \text{ kg} \\ \text{MLVSS} &= 0.7 \text{ MLSS} = 4900 \text{ kg} \end{aligned}$$

$$\begin{aligned} \text{COR} = R &= a (\text{BOD}_5 \text{ removed}) + b (\text{MLVSS}) \text{ kg/d} \\ &= a (280) + b (4900) \text{ kg/d} \end{aligned}$$

The calculated COR values using the 'a' and 'b' values in Table 1 are shown in Table 2.

Table 2 - Calculated COR Values

Ref.	a	b	COR (kg O ₂ /d)	COR/kg BOD ₅ (kg O ₂ /kg BOD ₅)
7	0.76/BOD ₅	0.13/VSS	850	3.035
8	0.75/BOD ₅	0.03/VSS	357	1.275
9	0.5/BOD ₅	0.1/VSS	630	2.25
	0.5/BOD ₅	0.15/VSS	875	3.125
10	0.5/BOD ₅	0.1/MLSS	840	3.0
	0.6/BOD ₅	0.1/MLSS	868	3.1

As shown in Table 2, when the COR values are expressed in terms of kg oxygen required per kg BOD₅ load, all the calculated values except for reference 8 are in excess of 2 kg O₂/kg BOD₅ load. These high COR values are fundamentally incorrect as the oxygen requirement cannot exceed the theoretical maximum oxygen requirement of 1.46 kg O₂/kg BOD₅ (as BOD_u = 1.46 BOD₅, Eq. 3, Section 2.1.1). It is certainly difficult to accept that the process COR will exceed the influent wastewater ultimate BOD. It can be seen that endogenous respiration oxygen requirement makes up for 75% or more of these exceedingly high calculated COR, and would be the main contributing factor in over estimating COR.

The COR value of 1.275 kg O₂/kg BOD₅ based on the values of 'a' and 'b' from ref. 8 compare very favourably with the theoretical COR of 1.2 kg O₂/kg BOD₅ obtained in Section 2.1.1. The value of 0.75 for 'a' from ref. 8 is within the range of 0.5 to 0.76 reported in the other references. However, the value 0.03 for 'b' is well below the values reported in the other references. The values of 'a' and 'b' are usually determined from a plot of Eq. 8 in isolation from the determination of other kinetic coefficients such as the cell yield coefficient, cell endogenous death/decay rate and substrate utilisation rate. Also, the plot is based on laboratory or pilot scale activated sludge units operating at different U loading rates. However, in ref. 8, the values of 'a' and 'b' were obtained in conjunction with the other kinetic coefficients based on data collected over 2 years from a full-scale plant operating with sludge age ranging from 10 to 20 days. The kinetic coefficients were not obtained from individual straight line plots but were obtained in toto for the complete activated sludge model using computer simulation and least squares analysis to match calculated and measured (observed) data. This is certainly a more rigorous and hence, more reliable method for determining kinetic coefficients than from individual straight line plots.

It has generally been accepted that at the low F/M loading rates of the extended aeration process, the COR is high and the majority is required to maintain the MLSS endogenous respiration rather than to treat substrate BOD. As shown in the above example, this concept could be erroneous and leads to overestimating the COR. The wide range in the values of kinetic coefficients reported (eg. four fold for the value of 'b') is not only confusing to the design engineer but also points to fundamental error in the traditional kinetic models. The traditional kinetic models were mainly developed from small scale activated sludge units treating a simple soluble biodegradable substrate and the kinetic coefficients were mainly determined for the 'conventional' activated sludge loading rates over the limited range of 0.2 to 0.4 kg BOD₅/kg MLSS.d. Adopting these kinetic coefficient values either directly or by extrapolation for the extended aeration process will in most instances, lead to serious errors.

Several authors have recognised the limitations of the traditional activated sludge model. For example, Clifft and Andrews (ref. 11) and Gujer (ref. 12) included the effect of particulate substrate in their kinetic models. Dold, Ekama and Marais (ref. 1) considered the effect of particulate substrate as well as the structured nature of MLSS. More importantly, these authors replaced the endogenous concept with a death-regeneration concept. Hence, recent developments of the activated sludge model recognised the complex nature of domestic sewage in which up to 75% of the substrate is in particulate form. Domestic sewage also contains inorganic SS and also VSS which are unbiodegradable in the time frame of activated sludge process. Also storage of substrate in biological cells as well as death-regeneration from cell lysis have been introduced. This led to a structured concept of the sludge mass consisting of an active cell mass fraction, inactive or dead cell mass fraction, stored substrate fraction (including unbiodegraded particulate substrate), and accumulated undegradable VSS and inorganic SS fractions. However, none of the new models has considered a fundamental aspect of the activated sludge process which in the opinion of the present author is the major weakness of the present models of the activated sludge

process. It is considered by the author that the fact that the activated sludge process is a substrate limited process has not been correctly addressed in the development of the activated sludge model. Because incorrect modelling of the activated process is a major reason for overestimating the COR, a basic review of the activated sludge process must be carried out. Although a rigorous review is beyond the scope of this project, a general review is given in the next section. It is hoped that this will lead to a better understanding of the activated sludge process and thus avoid fundamental errors in estimating the COR of extended aeration process.

2.1.3 General Review of Activated Sludge Process

The activated sludge process has evolved basically from the batch process. The process basic concepts were developed from the study of the growth curve of a batch culture of micro-organisms where the organic carbon substrate is the limiting substrate (i.e. all other nutrients and growth requirements are present in excess). A typical batch growth curve is illustrated in Fig. 1.

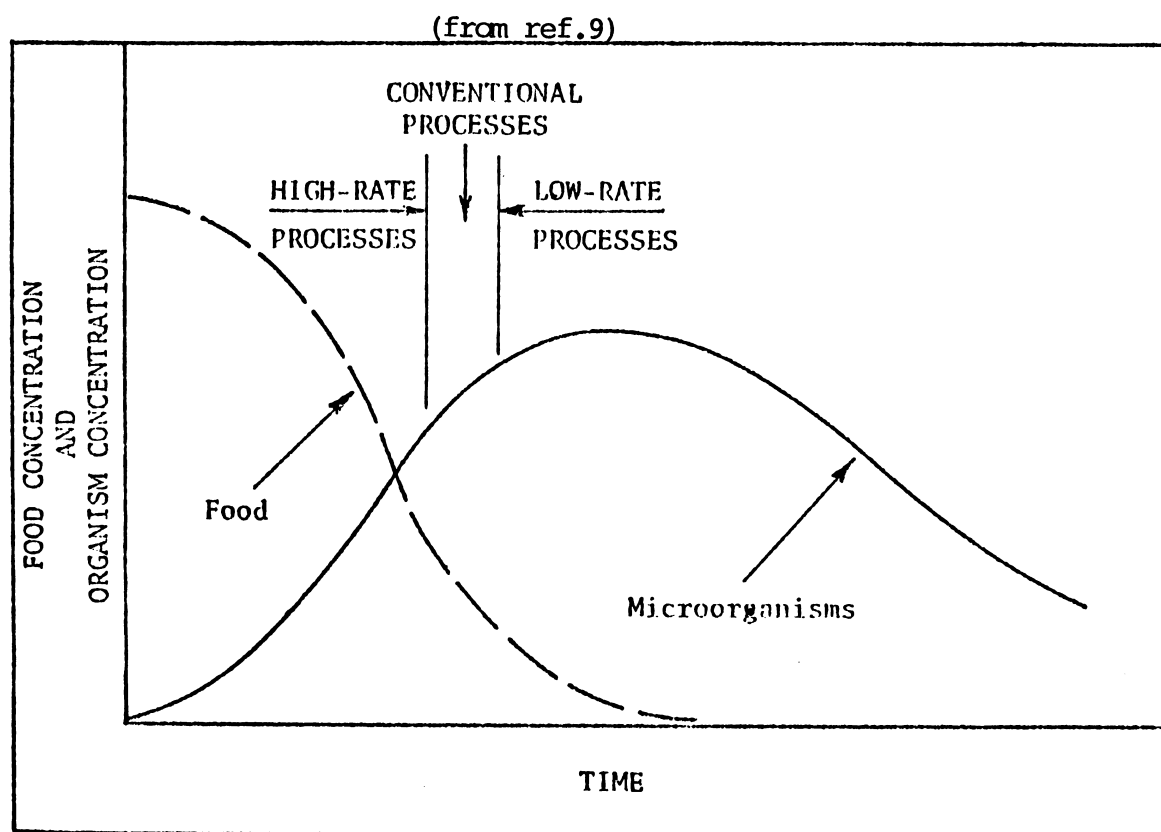
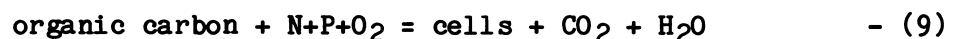


FIG.1 The Batch Biological Growth Curve showing typical ranges of operating points for continuous biological treatment process.

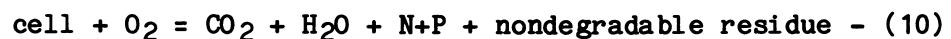
As shown in Fig. 1, initially when the substrate is in excess the bacterial cell's increase at a logarithmic rate controlled only by the cells inherent growth rate. The logarithmic growth phase will continue until the amount of substrate remaining is insufficient to maintain the logarithmic growth rate and the cell growth rate will start to decline. Hence in the declining growth phase, cell growth rate is limited by the availability of substrate. The declining growth phase will continue until the point when all the substrate has been exhausted. From this point onwards, the biological cell mass will start to decline as cell protoplasm is degraded by endogenous respiration for cellular maintenance. One implication arising from the batch growth curve is that each point in time in the curve is characterised by a particular value of both substrate and cell concentrations. This ratio of the substrate to cell concentration known as the "food-to-micro-organism" ratio has been adopted as a convenient measure of the organic loading rate of continuous activated sludge processes. It was postulated that a continuous activated sludge process may be represented as a single point on the batch growth curve and the particular F/M ratio at that point will represent the loading rate of the continuous process and hence determines its operation characteristics. The F/M ratio has been accepted as the main parameter in the design and operation of all activated sludge processes.

The two biological reactions occurring in the activated sludge process are as follows:

Growth reaction:



Endogenous decay reaction:



Based on the synthesis-oxidation reaction of Eq. 9, the cell yield of bacteria can be determined. The cell yield will depend on the type of organic carbon substrate. Reported cell yield values for several types of substrate range from about 0.3 to 0.75 kg cell formed per kg substrate BOD₅ utilised. For domestic sewage, the reported cell yield is typically between 0.5 and 0.7 kg cell/kg BOD₅ utilised. Therefore, for a domestic sewage activated sludge plant, a F/M of less than about 1.5 to 2.0 kg BOD₅/kg cell is a substrate limited system. Hence the 'conventional' and extended aeration activated sludge processes with F/M ratios of 0.2 to 0.4 and less than 0.1 kg BOD₅/kg MLSS.d respectively can be accepted to be operating as substrate limiting systems. Another implication of the batch growth curve in Fig. 1 is that in the substrate limiting declining growth phase, the rate of cell growth is a function of the limiting substrate concentration. Eqs. 9 and 10 for the biological reactions can thus be represented by the following kinetic equations.

Cell growth:

$$\Delta X = Y \Delta S \quad - (11)$$

Cell endogenous decay (or death):

$$\Delta X = -k_d X \quad - (12)$$

Where ΔX = daily rate of change of cell mass,
kg VSS/d

S = substrate utilised/day, kg BOD₅/d

X = cell mass in process system, kg VSS

Y = growth yield coefficient, cell mass
produced/mass substrate utilised,
kg VSS/kg BOD₅

k_d = endogenous decay rate, cell mass
oxidised/unit cell mass remaining/day, d⁻¹

The sludge production relationship was thus developed empirically by combining Eqs. 11 and 12.

Net sludge production:

$$\Delta X = Y\Delta S - k_d X \quad - (13)$$

Eq. 13 can be divided by X to give:

$$\frac{\Delta X}{X} = Y \frac{\Delta S}{X} - k_d \quad - (14)$$

It can be seen that $\Delta X/X$ is the reciprocal of the sludge age or MCRT θ_c and that $\Delta S/X$ is equivalent to the modified F/M ratio $U = E(F/M)$ where E is the percentage substrate removal efficiency. Eq. 14 can thus be written as:

$$\frac{1}{\theta_c} = \frac{\Delta X}{X} = YU - k_d \quad - (15)$$

The kinetic coefficients are usually determined from the straight line plot of Eq. 14 or 15 where Y is obtained from the slope and k_d from the intercept. Once values for Y and k_d are obtained, the amount of excess sludge produced per day can be obtained from Eq. 13 or the operating sludge age obtained from Eq. 15.

It is from Eq. 13 that the concept of decreasing net excess sludge production with decreasing F/M was derived. Eq. 13 shows that for a fixed ΔS , a system operating at a lower F/M and hence higher X value will produce less sludge than a system operating at a higher F/M with a lower X value. This results from the term $(-k_d X)$ in Eq. 13. If ΔS is fixed and X is increased for lower F/M, more sludge is degraded due to the endogeneous term $k_d X$. Hence, it is generally accepted that low F/M process such as the extended aeration process will produce less excess sludge per day than the higher F/M 'conventional' process. This seems to be supported by the fact that

the extended aeration process is operated at higher sludge age than the conventional process.

However, it is the present author's opinion that the above concept is faulty based on the following reasoning. Since the process is a substrate limiting system, the amount of active cells that can be sustained in the system is limited by the amount of substrate fed to it. That is, the amount of active cell mass in the system is fixed by the amount of daily substrate feed. Hence, for a fixed amount of substrate feed, it is unreasonable to expect that the F/M can be reduced by increasing the amount of active cells in the system under the substrate limiting situation. This point can be illustrated by considering the operation of the previous 4000 ep plant example. The plant could be operated at a F/M of 0.4 kg BOD₅/kg VSS.d (accepting at present that VSS is a measure of active cell mass). The BOD₅ load is 280 kg/d and hence the cell mass measured as VSS to be maintained in the system is $280/0.4 = 700$ kg VSS. If the plant is now to be operated at a F/M of 0.04 in the extended aeration mode, the active cell mass to be maintained in the system has to be increased to 7000 kg VSS. Under the substrate limiting condition, it can be seen that it is quite impossible to increase the active cell mass by a factor of 10 without increasing the substrate feed.

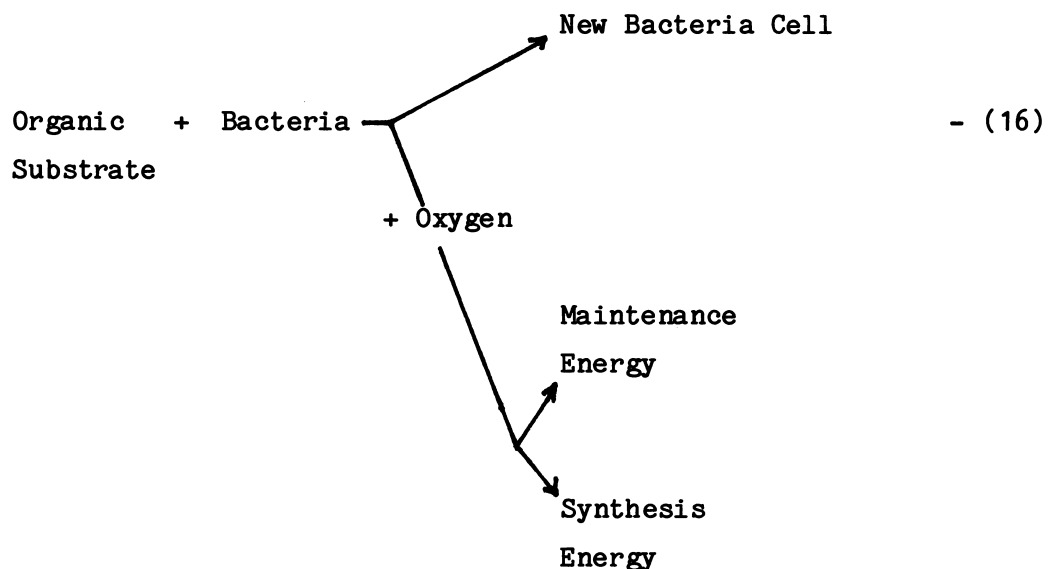
Similarly, the empirical Eq. 7 in Section 2.1.2 for determining the process COR from the synthesis-endogenous respiration relationship can be developed by combining the oxygen requirement in Eqs. 9 and 10 representing growth and endogenous decay respectively. Eq. 7 is repeated below:

$$R = aS + bX$$

Again, this equation shows that for a fixed value of S, the oxygen required increases with decreasing F/M because of increasing endogenous respiration as implied by the term bX in the equation. As was demonstrated in Section 2.1.2, this concept can lead to grossly overestimated COR values for extended aeration process. This concept

is again faulty as it fails to recognise the constraint placed on the value of X in a substrate limiting system.

It is considered by the present author that the concept of separating activated sludge production and respiration into two parts (one part for synthesis and the other for endogenous decay/ respiration) as represented by Eqs. 7, 8, 13, 14 and 15 is wrong. The misconception arises because of the misunderstanding of the endogenous metabolism which is still not fully understood by microbiologists. According to Brock (ref. 13), there is probably a minimum of energy required to maintain cell structure and integrity (called maintenance energy) and the substrate used for this maintenance energy is not available for biosynthesis and cell growth. Thus, the fraction of substrate and oxygen utilised for maintenance energy are included in the growth reaction as represented by the following equation:



To again include a separate term for endogenous decay or respiration to the kinetic equations as in Eq. 7 and 13 would be double accounting the maintenance energy requirements. Also endogenous metabolism whereby cell protoplasm is degraded for maintenance energy will only occur in the absence of substrate.

Hence, in a continuous system at steady state conditions, endogenous metabolism is not applicable and should not be considered. Endogenous metabolism would only be applicable under dynamic conditions such as are caused by diurnal load variations in municipal activated sludge plants. The daily average active cell concentration will reflect the daily average substrate loading. When peak diurnal loading occurs, the active cell concentration could increase due to additional growth. However, as substrate loading rate drops, endogenous decay could occur as the reduction in available substrate cannot sustain the peak loading cell concentration. The degree of endogenous metabolism would be greatest during the minimum diurnal loading period. Hence, it is expected that the active cell concentration would vary over a 24 hour period in line with the diurnal variation in substrate loading rate. However, the diurnal variation in active cell concentrations could be greatly dampened by substrate storage and the slow degradation of particulate substrate (refs. 1, 11 and 12).

2.1.3.1 F/M Ratio

In practice, activated sludge plants treating sewage have been successfully operated in a F/M range of from 0.04 to 0.4 kg BOD₅/kg MLSS.d. Based on the present author's postulations given above this would be quite impossible for any substrate limiting system. This is in fact not a real contradiction to the author's view if the parameters used in measuring the substrate and active cell mass are examined. In practice, the VSS is used as a surrogate measure of the active cell mass and BOD₅ as a surrogate measure of the substrate mass. Hence F/M is in practice measured as kg BOD₅/kg MLVSS or MLSS per day. MLSS has also been used because it was found that the percentage of VSS in the MLSS is generally quite constant, usually MLVSS is about 70% of MLSS.

It is well known that the MLSS from a domestic sewage activated sludge plant consists of various types of SS and not just active bacterial cells. The MLSS also consists of dead bacterial cells, non-biodegradable VSS, inorganic SS and partially degraded organic SS. The active bacterial cells may make up only a small fraction of the MLSS. Hence, MLSS or MLVSS would not be a suitable parameter for active bacterial cell concentration unless the fraction of active bacterial cells is constant in all activated sludge plants' MLSS. Since it has been found that the active cell fraction of MLSS varies with sludge age, definition of F/M ratios based on MLSS or MLVSS is incorrect. F/M ratios should be based on the actual active bacterial cell mass as treatment of the wastewater is due entirely to the activity of the active bacterial cells.

As discussed in Section 2.1.1, the viable or active cell fraction of activated sludge has been investigated by several authors. All the authors found that the active cell fraction decreases with increasing sludge age (i.e. decreasing F/M). Based on laboratory and pilot plants studies on settled sewage, Weddle and Jenkins (ref. 15) found that the percentage viability of MLVSS decreases nonlinearly with increasing sludge age and levels off at about 15% for sludge age in excess of 30 days (F/M about 0.17 kg COD/kg MLVSS.d). ATP was used as a measure of active cell fraction in the MLVSS. Similarly, Roe and Bhagat (ref. 5) found that viability fraction of the MLSS of a pilot plant treating a synthetic substrate (glucose and yeast extract) decreases with increasing sludge age, levelling off at about 10 to 15% at a sludge age of about 30 days. ATP was also used as a measure of active cell fraction in the MLSS - operating sludge ages were 0.41, 2.78, 6.86, 9.52, 17.34 and 26.57 days. Similar observations were made by Benefield, Lawrence and Randall (ref. 16) and by Nelson and Lawrence (ref. 6) except that the percent viability of the MLVSS levelled off at about 40 to 45% at a sludge age of about 12 to 14 days. Both teams of authors worked with a synthetic soluble substrate. Benefield et al (ref. 16) used the specific oxygen uptake rate and Nelson et al (ref. 6) used ATP as a measure of sludge viability. The data from these authors provide an opportunity to examine the relationship between F/M based on MLSS or

MLVSS against F/M based on active MLSS. An example based on the data from Roe and Bhagat (ref. 5) is given in Table 3. The MLSS viability fraction versus sludge age was obtained from Fig. 3 of their paper.

F/M values were not provided but calculated from the kinetic coefficient data obtained by these authors in their experiments (Eq. 2 and Table 2 in their paper): $1/\theta_c = 0.357U - 0.00767$.

Table 3 - Effect of Viability Percentage on F/M Ratios

Percent Viability	Total Sludge Age	U (kg COD/kg MLSS.d)	U (kg COD/kg viable MLSS.d)
65%	3 d	0.96	1.48
55%	5 d	0.58	1.05
35%	10 d	0.30	0.86
25%	15 d	0.21	0.84
15%	30 d	0.11	0.73
F/M Range		9:1	2:1

As shown in Table 3, the F/M ratio based on MLSS has a range of about 9 to 1 but reduces to a range of only about 2 to 1 when the F/M ratios are based on the viable MLSS fraction. This thus lends support to the view postulated by the present author that in a substrate limiting system, the actual F/M ratio is not variable but should remain relatively constant.

Although the F/M ratio range based on actual viable biomass of 2:1 is not great, it still does not fully support the author's view of a relatively constant F/M in all substrate limiting systems. This slight contradiction can be removed if the nature of the substrate is also examined. Domestic sewage is a complex organic substrate consisting of both soluble and particulate substrate with varying degree of biodegradability. Each component degrades at a different rate - simple soluble carbohydrates degrading at a rapid rate (hours)

and the larger or more complex particulate substrates degrading at slower rates (days). Gujer (ref. 12) predicted that the degradation rate for particulate substrates in settled sewage is about 0.06 d^{-1} . Clifft and Andrews (ref. 11) quoted figure from Balmat (ref. 18) that degradation rate for particulates is about $0.08 - 0.09 \text{ d}^{-1}$ for particulates greater than $1 \mu\text{m}$, 0.22 d^{-1} for particulates between 0.08 to $1 \mu\text{m}$, and 0.39 d^{-1} for soluble substrate fraction. The observed BOD curve for domestic sewage must be the summation of all the individual component's oxygen demands. Since domestic sewage organic substrate is mainly in the particulate form (about 75%), it can be seen that the actual amount of substrate available to the viable bacteria in the activated sludge process is dependent on the sludge age of the system. Since the BOD of domestic sewage can be approximated by a first order reaction, the amount of substrate actually available at different sludge ages can be estimated by Eq. 1 in Section 2.1.1. Adopting a reaction time constant of 0.1 d^{-1} , the fraction of the total substrate actually available at different sludge ages can be computed and the true F/M ratio based on actual substrate available/unit mass viable bacteria cell/d can also thus be computed. Using the data from the example given in Table 2, the true F/M can be computed by assuming that the substrate COD is equal to the ultimate BOD. The results are shown in Table 4.

Table 4 - Effects of Percentage Available Substrate
and Viable MLSS on F/M Ratios

Sludge Age	% Substrate Available	F/M (BOD _T /MLSS)	F/M (BOD _T /Viable Mass)	F/M (BOD _a /Viable Mass)
3	50%	0.96	1.48	0.74
5	68%	0.58	1.05	0.71
10	90%	0.30	0.86	0.77
15	97%	0.21	0.84	0.81
30	99.9%	0.11	0.73	0.73
F/M Range		9:1	2:1	1.1:1

BOD_T = total BOD (or BOD_u) = COD (reasonable for soluble synthetic substrate used)

BOD_a = actual BOD available

It can be seen that even with operating sludge age ranging from 3 to 30 days, the actual F/M based on kg available substrate/kg viable cell mass/day did not vary by more than 10% and fully supports the author's view of a relatively constant F/M value in any substrate limiting system.

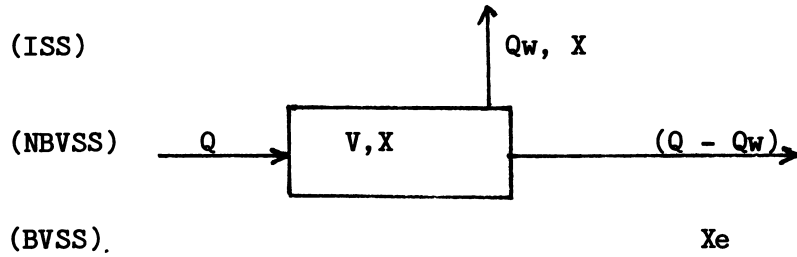
2.1.3.2 Sludge Age

Based on the new concepts of the activated sludge process postulated previously, it can be seen that the traditional method of determining excess sludge production must be revised. Essentially, the new concepts state that at steady state:-

- (i) The substrate actually available is dependent on the sludge age. If the sludge age is 5 days, then the available substrate is equivalent to the sewage BOD_5 , if $\theta_c = 10$ days, available substrate is equivalent to BOD_{10} and so forth until for sludge age of about 30 days or more the available substrate is the sewage BOD_u .
- (ii) In a substrate limiting situation, the daily production or growth of active bacterial cells is a function of the daily available substrate feed rate and the sludge yield associated with the characteristics of the substrate and bacterial population. At steady state, the growth of bacterial cells cannot exceed the available substrate input and hence endogenous decay may be excluded in the mass balance on a daily steady state basis.

Hence, it is proposed that the sludge age may be determined from the following steady state mass balance equations where excess sludge is wasted as MLSS from the intermittent extended aeration tank as shown in Figure 2.

Figure 2 - Intermittent Extended Aeration Tank



Where ISS = Inorganic SS concentration in influent, kg/kl
 NBVSS = Nonbiodegradable VSS concentration in influent, kg/kl
 BVSS = Biodegradable VSS concentration in influent, kg/kl
 Q = Influent flow rate, kl/d
 V = Reactor volume (at BWL), kl
 X = MLSS concentration, kg/kl
 Qw = Waste MLSS flow rate, kl/d
 Xe = Effluent SS concentration, kg/kl

At steady state, the mass balance is given by:-

$$Q_w X + (Q - Q_w) X_e = Q(ISS) + Q(NBVSS) + \Delta BVSS + \Delta X_a \quad - (17)$$

$\Delta BVSS$ = rate of accumulation of influent BVSS (kg/d) given by the equation

$$\Delta BVSS = Q (BVSS) (1 - 10^{-k_p t}) \quad - (18)$$

Where k_p = BVSS or particulate substrate degradation rate d^{-1}
 $t = \theta_c, d$

ΔX_a = growth of active cell mass (kg/d) given by:-

$$\Delta X_a = YQ (S_a) \quad - (19)$$

Where Y = cell yield, kg cell/kg available substrate utilised

S_a = available substrate concentration, kg/kl

The available substrate in Eq. 19 is related to the sludge age and can be approximated by the first order BOD reaction as follows:-

$$S_a = BOD_t = BOD_u (1 - 10^{-kt}). \quad - (20)$$

Where $t = \theta_c, d$

k = BOD reaction time constant, d^{-1}

Eq. 18 represents the portion of the influent BVSS that is not degraded per day. K_p in Eq. 18 is not equal to k in Eq. 20 as k_p reflects the degradation of the particulate fraction of the substrate only whereas k reflects the degradation of the whole substrate (soluble plus particulate). Also Eq. (18) is on a VSS mass basis but Eq. 20 is on an oxygen basis. Reported values of k_p vary considerably. For example Clifft and Andrews (ref. 11) gave a value of $0.33 d^{-1}$ whereas Gujer (ref. 12) gave a value of $0.06 d^{-1}$. In any case, it can be seen that the BVSS will be almost completely degraded with a sludge age of 15 to 20 days from both Eqs. 18 and 20. Hence for extended aeration plants with sludge age in excess of 20 days.

Δ BVSS may be assumed to be zero and Eq. 17 reduces to:-

$$Q_w X + (Q - Q_w) X_e = Q(I_{SS}) + Q(NBVSS) + \Delta X_a \quad - (21)$$

Eqs. 19 and 20 could also be combined to give

$$\Delta X_a = YQ BOD_u (1 - 10^{-kt}) \quad - (22)$$

Eqs. 21 and 22 can be used to estimate the daily sludge production and hence the sludge age of an extended process. It can be seen that the two kinetic coefficients required to be determined, Y and K are much more established and the reported range of their values is not as great as that reported for the endogenous respiration/decay

coefficient. According to Vasicek (ref. 7), reported range for Y is 0.5 to 0.8 on a BOD_5 basis and 0.32 to 0.47 on a COD basis - a range of 60% on BOD_5 basis and 50% on COD basis. However, for the endogenous decay coefficient, the reported range is 0.02 to 0.12 (650%) on a BOD_5 or COD basis. It is considered that the true yield coefficients for active cells reported by Ramanathan and Gaudy (ref. 17) are the most reliable. According to these authors, a useful range of biological sludge yield values for carbohydrate wastes is 0.4 to 0.6 kg/kg COD. The conclusion is based on results obtained using mixed bacterial culture from sewage in batch and continuous experiments conducted over a period of 10 years. Since soluble carbohydrates were used in all the experiments and cell yield recorded at the peak solids concentration (before the start of endogenous respiration), Y can be taken as the true yield for active mass. Also, for soluble carbohydrate substrates, COD may be assumed to be equivalent to BOD_u . Hence 0.5 kg/kg BOD_u may be a reasonable Y value to adopt for the extended aeration process (possible error of $\pm 20\%$). The higher Y values generally reported for activated sludge plants are observed total sludge yield rather than true active cell yield.

In the 4,000 ep I.E.A. plant example given in Section 2.1.1, a sludge age of 30 days was adopted to obtain a theoretical process COR of 1.2 kg O_2 /kg BOD_5 . Eqs. 21 and 22 can now be used to check if the adopted sludge age of 30 days is not too low such that the COR value is underestimated.

Sludge Age Calculation for 4000 ep I.E.A. Plant Example

BOD_5 influent load = 280 kg/d.

MLSS = 7000 kg ($F/M = 0.04$)

Since $\theta_c > 20$ days, all influent substrate would be degraded and hence available to the active biomass. Hence, from Eq. 22:-

$$\begin{aligned}\Delta X_a &= 0.5 \times 1.46 \times 280 \\ &= 204.4 \text{ kg/d}\end{aligned}$$

$$\text{Influent SS} = 280 \text{ kg/d}$$

$$\begin{aligned}\text{Influent ISS} &= 25\% \text{ SS} = 0.25 \times 280 \\ &= 70 \text{ kg/d}\end{aligned}$$

$$\text{Influent VSS} = 0.75 \text{ SS} = 210 \text{ kg/d}$$

$$\text{Influent NBVSS} = 0.1 \text{ VSS} = 21 \text{ kg/d}$$

$$\begin{aligned}. . \quad \text{Excess sludge production} &= 204.4 + 70 + 21 \\ &= 295.4 \text{ kg/d}\end{aligned}$$

$$\begin{aligned}. . \quad \theta_c &= \frac{X}{\Delta X} = \frac{7000}{295.4} = 24 \text{ d}\end{aligned}$$

It can be seen that the calculated and assumed θ_c are in close agreement (-20% difference of $\pm 20\%$ error for Y) and the assumed 30 days sludge age would not lead to an underestimated COR value. The θ_c values are in close agreement with reported values for PWD's I.E.A. plants published by Chong and Giles (ref. 18), and by Awad and King (ref. 19) for operating F/M of 0.06 to 0.08.

The calculated and assumed sludge production rates when expressed in terms of kg sludge/kg BOD_5 are 1.06 and 0.83 respectively. Very few actual figures from operating extended aeration plants have been reported in the literature. But the calculated value of 1.06 kg MLSS/kg BOD_5 is in close agreement with that reported by Johnstone (ref. 20) for two plants in U.K. The actual MLSS production observed from careful monitoring at two oxidation ditches was 1.0 kg sludge/kg BOD_5 at a F/M of 0.036 kg BOD_5 /kg MLSS.d and 1.3 kg sludge/kg BOD_5 at a F/M of 0.055.

Hence, it can be seen that Eqs. 21 and 22 proposed by the present author provide an easy method for calculating the θ_c for extended aeration plants which is in close agreement with reported values measured from operating plants. However, it should be noted that the equations are based on a simplifying assumption that all influent substrates are used for cell growth hence excluding the endogenous decay term in the continuity steady state mass balance equation. The equations are considered to be valid for estimating daily sludge production on a 24 hour quasi steady state basis. The equations are, however, not applicable for describing the dynamic behaviour of the activated sludge process such as under diurnal variation of the influent. It can be expected that active cell growth will follow the diurnal variation of the influent substrate loading rate but will be dampened by the slow degradation of particulate substrate. Active cell production is expected to be at a maximum at peak diurnal load and the cells produced may later enter into endogenous respiration during minimum diurnal load. Some cells may die and lyse and be available as a substrate for other cells. Also, aeration tanks in full scale plants are not 100% completely mixed systems. Hence cells near the inlet may be in contact with sufficient substrate for growth but cells further away may have just sufficient substrate for their maintenance energy. Cells near the outlet may not even have sufficient substrate for maintenance energy and hence undergo endogenous respiration. Because cell degradation is much slower than cell growth, accumulation of dead cell mass will occur in the system which may explain the observations made by various authors. The MLSS active cell fraction may be determined by ATP or oxygen uptake rate measurements. But measurements of growing cells, stationary cells and endogenous respiring cells are at present almost impossible. Hence, the development of a "unifying" activated sludge model is a most taunting task. This discussion is not intended to be critical of recent developments of dynamic models which have taken into account the "structured" nature of MLSS and the distinction of soluble and particulate substrates. These models have in fact provided us with a much better understanding of the dynamic behaviour of the activated sludge process and hence make more accurate

predictions. But caution should still be taken in applying the mathematical equations to systems with different conditions to those in which the equations were derived. There are still considerable variations in the values of the kinetic coefficients reported for these new models (refs. 1, 11 and 12).

2.1.4 Closure for Section 2.1

1. The oxygen required for the complete biological oxidation of a carbonaceous substrate such as sewage to end products is measured by the ultimate BOD of the substrate. Hence the theoretical maximum COR for any activated sludge process can be obtained by converting the substrate BOD₅ to BOD_u. For untreated sewage the ratio of BOD_u to BOD₅ is about 1.46 to 1. Hence the theoretical maximum COR can be estimated by the following equation.

$$\text{Maximum COR} = 1.46 \text{ BOD}_5 \quad - (1-1)$$

2. The theoretical COR would be less than the theoretical maximum COR as a portion of the substrate is converted to cell mass which is subsequently wasted from the activated sludge system. Hence the theoretical COR would be equal to the BOD_u of the substrate minus the BOD_u of the cell mass wasted from the system. Since the chemical composition of all living cells is almost identical and the four elements C, H, N and O make up 98 to 99% of the cell, the bacteria cell can be represented by the chemical formula C₅H₇NO₂. Based on this cell chemical formula, the oxygen equivalent or theoretical BOD_u of the cell can be estimated to be 1.42 kg O₂/kg cell. Hence the theoretical COR of an extended aeration process can be estimated by the equation.

$$\text{COR} = 1.46 (\text{BOD}_5) - 1.42 f_{\text{BVSS}} (\text{MLVSS}/\theta_c) \quad - (1-2)$$

Where f_{BVSS} = biodegradable fraction of the MLVSS,
(which varies with θ_c)

θ_c = sludge age, d

$MLVSS/\theta_c$ = mass of MLVSS wasted, kg/d

BOD_5 = influent BOD_5 , kg/d

COR = theoretical process COR , kg O_2 /d

3. For a domestic sewage extended aeration plant with a sludge age of 30 days, the theoretical COR computed by Eq. 1-2 is about 1.2 kg O_2 /kg BOD_5 ($f_{BVSS} = 0.35$).
4. The traditional method of obtaining the COR for synthesis and endogenous respiration has been found to grossly over-estimate the process COR for extended aeration processes. In general, the traditional method will give COR values in excess of the theoretical COR of 1.46 kg O_2 /kg BOD_5 which is unacceptable.
5. In view of the failure of the traditional method to provide reasonable estimates of the COR in the extended aerated process, a review of the concepts of the traditional activated sludge model as represented by the following two equations was undertaken.

$$\Delta x = Y S - K_d X \quad - (1-3)$$

$$R = aS + bX \quad - (1-4)$$

6. One of the unsatisfactory aspects of the traditional model is the endogenous decay and respiration terms in Eqs. 1-3 and 1-4 respectively. The concept as illustrated by the two equations incorrectly implies that all the cells undergo endogenous decay although cell synthesis is also occurring. But more importantly, the limitation placed on the amount of active biomass that can be sustained by the limiting substrate is not recognised.

7. The other unsatisfactory aspect of the traditional model lies in the use of the parameters BOD_5 and $MLVSS$ as measures of carbonaceous substrate and active biomass respectively. BOD_5 is only a measure of a portion of the substrate and $MLVSS$ is a measure of all VSS including inactive biomass, $NBVSS$ and unbiodegraded particulate substrate. Both BOD and the active fraction of $MLVSS$ vary in a non-linear fashion with sludge age and hence the linear Eqs., 1-3 and 1-4 are not valid over a wide F/M range such as from the "conventional" F/M of 0.4 to the extended aeration F/M of 0.04 kg BOD_5/kg $MLVSS.d$.
8. The consequences of the misconceptions of the endogenous factor and linear relationships incorporated in Eqs. 1-3 and 1-4 are:-
 - (i) Underestimation of sludge production in low F/M extended aeration processes as evidenced by the negative intercept generally produced by the straight line plot of Eq. 1-3. Negative sludge production is impossible.
 - (ii) Overestimation of the COR for the low F/M extended aeration process. COR in excess of the BOD_u of the influent carbonaceous substrate is impossible.
9. The following new concepts of the activated sludge process are proposed by the author:-
 - (i) For domestic sewage in which about 75% of the carbonaceous substrate is in particulate form, the amount of substrate actually available for cell metabolism is related to the sludge age of the process. Hence carbonaceous substrate loading should be measured by the available substrate rather than an arbitrary value such as BOD_5 or the total substrate concentration such as COD or BOD_u . The proposed available substrate loading parameter can be approximated by the first order BOD equation as follows:-

$$S_a = BOD_t = BOD_u (1 - 10^{-kt}) \quad - (1-5)$$

Where S_a = available carbonaceous substrate, M/L^3

k = BOD reaction rate coefficient, T^{-1}

t = θ_c , sludge age of process, T .

- (ii) In a substrate limited process such as the conventional and extended aeration activated sludge processes the quantity of active cells sustainable in the process is directly related to the quantity of substrate available to the cells. It is unreasonable to postulate that the active cell biomass can be significantly increased in the process without also increasing the amount of substrate. Hence it is unreasonable to assume that an extended process with F/M of 0.04 will have ten times the amount of active biomass than a conventional process with F/M of 0.4 when receiving the same amount of substrate. In fact, it will be more reasonable to assume that the true F/M for any substrate limited activated sludge process to remain substantially constant. Hence sludge loading rates should be measured by the true F/M rather than by the surrogate F/M represented traditionally as $kg\ BOD_5/kg\ MLVSS.d$. It is proposed that the true F/M can be represented as

$$\text{True } F/M = BOD_t / X_a \quad - (1-6)$$

Where BOD_t = available substrate (S_a), M/T

t = sludge age (θ_c), T

X_a = active biomass in system, M

10. It can be concluded from the new concepts proposed above that the sludge age is a more accurate parameter than F/M in distinguishing between substrate limited processes such as the conventional and extended aeration modes of the activated process. This will provide a better explanation of the

differences between the two modes of the activated sludge process in the following two major areas:-

- (i) The more stable or less active nature of the sludge from the extended aeration process compared to that of the conventional process is due primarily to a more complete oxidation of the particulate substrate because of the longer sludge age. Also, the amount of active biomass in the two modes would be about the same if treating the same amount of substrate - but the MLSS active biomass fraction would be smaller in the extended aeration mode because of the greater accumulation of inorganic SS and nonbiodegradable VSS in the sludge. Consequently, the sludge from the conventional process will have a greater fraction of bio-degradable VSS and hence needs further biological treatment before disposal. This is a more plausible explanation than the traditional "starvation" explanation of a greater degree of endogenous respiration in the extended aeration process being responsible for the greater stability of the extended aeration process sludge.
- (ii) The greater process oxygen requirement in the extended aeration mode compared to the conventional mode is due to the more complete oxidation of the particulate substrate and also to the nitrification oxygen requirement. This is more plausible than the traditional explanation of a much higher degree of endogenous respiration demand in the extended aeration mode.

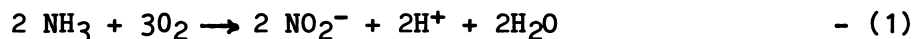
2.2 Net Nitrogenous Oxygen Requirement (NOR)

Under favourable conditions, the maximum growth rate for the nitrifying bacteria has been reported to be about $0.4 \text{ to } 0.5 \text{ d}^{-1}$ (ref. 9, Bliss). In practice, it is generally accepted that a sludge age of between 8 to 10 days is required if over 90% nitrification is to

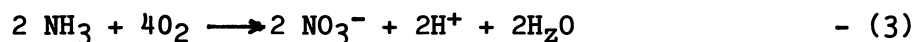
be achieved. In the extended aeration process with sludge age in excess of 10 days, over 90% nitrification is a general occurrence. In nitrifying plants it is desirable to also incorporate denitrification to reduce oxygen requirement. In soft water areas, denitrification is necessary to maintain the MLSS pH at levels required for efficient biological reactions - pH as low as 4.5 to 5.5 has been noted in nitrification only plants.

2.2.1 Nitrification

Biological nitrification involves the sequential oxidation of ammonia nitrogen (amm.N) to nitrite then to nitrate by the chemo-autotrophic bacteria Nitrosomonas and Nitrobacter respectively. The two sequential reactions can be represented as:-



Eqs. 1 and 2 can be combined to give the overall reaction for oxidation of NH_3 to NO_3^- :-



Compared to carbonaceous substrate oxidation, the nitrification process is a rather straight forward reaction involving the oxidation of a single substrate by two forms of bacteria. Hence, the oxygen required for nitrification can be simply obtained from the stoichiometric relationship in Eq. 3. It can be seen from Eq. 3 that about 4.6 kg. of oxygen is required to oxidise each kg of amm.N.

Some authors only consider the influent amm.N while others consider the influent total nitrogen (tot.N) in determining the nitrification oxygen requirement. The amount of organic nitrogen that is available for cell metabolism will depend on the sludge age as the hydrolysis of organic nitrogen (generally protein) is a slow process.

In the extended aeration process it may thus be prudent to base the nitrification oxygen requirement on the tot.N. Some of the nitrogen will be used for cell synthesis and removed from the system in the excess sludge. Estimates vary from about 5 to 35% of the influent tot.N lost this way. A part of this nitrogen will be returned as supernatant from sludge lagoons etc. This need not be considered if it is included in the influent tot.N concentration measurement.

According to Eckenfelder (ref. 3) active biomass contains about 12.3 percent nitrogen (based on the cell formula $C_5H_7NO_2$) but after endogenous oxidation contains about 7% nitrogen. This will provide an estimate of the nitrogen lost from the system as excess sludge. Based on the previous 4000 ep I.E.A. plant example, the percentage influent tot.N lost in excess sludge can be obtained as follows:-

Influent tot.N = 10 g/ep.d

.
. . total tot.N = 40 kg/d

MLSS = 7000 kg.

MLVSS = 0.7 MLSS = 4900 kg

Biodegradable cell fraction = 0.35×4900 (as per Section 2.1.1)
= 1715 kg.

Nonbiodegradable cell fraction = 0.52×4900
= 2548 kg

Sludge age = 30 days.

.
. . Biodegradable cell wasted/day = 57 kg.

.
. . Nonbiodegradable cell wasted/day = 85 kg.

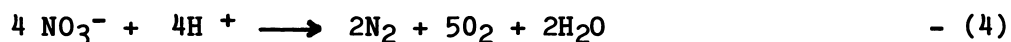
$$\begin{aligned} \text{Nitrogen lost in excess sludge} &= 0.12 \times 57 + 0.07 \times 85 \\ &= 12.79 \text{ kg/d.} \end{aligned}$$

Based on the above example, it would be reasonable to assume that about 30% of the influent tot.N will be lost in the excess sludge for extended aeration processes. But as previously noted, part of this may be fed back as lagoon supernatant.

It has generally been reported that to sustain 90% or more nitrification, a dissolved oxygen (DO) concentration between 1 to 2 mg/L must be maintained in the aeration tank. However, Pitman (ref. 21) stated that at long sludge ages of 20 days or longer, DO levels in the range of 0.5 to 1 mg/L will sustain nitrification. This is in agreement with the present author's observation that low DO values of about 0.5 mg/L during part of the day do not affect the I.E.A. plants in achieving nitrification.

2.2.2 Denitrification

Under anoxic conditions (DO levels less than 0.5 mg/L) some heterotrophic bacteria can use nitrate instead of oxygen as the terminal electron acceptor. This denitrification process can be represented by the following stoichiometric equation.



As shown in the above equation, 2.9 kg of oxygen is recovered for each kg of nitrate nitrogen (nte.N) reduced to nitrogen.

In the extended aeration process, 70 to 95% nitrogen removal can be achieved by the nitrification - denitrification process. In the FWD I.E.A. plant 80 to 95% total nitrogen removal can be readily achieved (refs. 18 & 19). The lower percentage removal values are generally due to increased organic nitrogen in the effluent. In terms of denitrification, generally over 90% of the nitrates are reduced.

2.2.3 Net Nitrogenous Oxygen Requirement (NOR)

Based on the nitrification - denitrification reaction the NOR for an extended aeration process can be obtained by the following equation.

$$\text{NOR} = 4.6P_N (\text{tot.N}) - 2.9P_{DN} P_N (\text{tot.N}) \quad - (5)$$

Where tot.N = influent total nitrogen, kg/d.

P_N = fraction tot.N nitrified

P_{DN} = fraction nitrate denitrified

The NOR for the 4000 ep I.E.A. plant example can thus be obtained as follows:-

Influent tot.N = 40 kg/d

% tot.N nitrified, $P_N = 0.7$

% nte.N denitrified, $P_{DN} = 0.9$

$$\begin{aligned} \text{. . NOR} &= (4.6 \times 0.7 \times 40) - (2.9 \times 0.9 \times 0.7 \times 40) \\ &= 56 \text{ kg/d.} \end{aligned}$$

2.2.4 Closure for Section 2.2

1. The determination of the extended aeration process NOR is rather straight forward compared to the determination of the COR involved in the oxidation of the complex carbonaceous substrate.
2. The theoretical NOR can be obtained from the following equation:-

$$\text{NOR} = 4.6P_N (\text{tot.N}) - 2.9P_{\text{DN}} P_N (\text{tot.N}) \quad - (2-1)$$

2.3 Total Oxygen Requirement (TOR)

The TOR can be obtained as a combination of the theoretical COR and NOR as represented by Eqs. 1-2 and 2-1 respectively.

For the 4000 ep I.E.A. plant example, the COR was found to be 328 kg O₂/d and the NOR was 56 kg/d. TOR is thus 384 kg/d or 1.37 kg O₂/kg BOD₅. The average hourly TOR is thus 16 kg/h.

In order to meet the diurnal peak process TOR the capacity of the aerator must be sized to satisfy the diurnal peak TOR. For a 4000 ep plant the diurnal peak flow is about 200% of the average and the diurnal peak substrate (BOD₅ and tot.N) concentrations is about 150% of the average. Hence peak diurnal substrate loadings could be 300% of the average loadings. However, it is generally accepted that there is considerable damping of the diurnal peak oxygen requirement and also that the peak oxygen requirement lags the peak substrate loading by a few hours (refs. 11 and 22). The damping effect is due to the combination of the following factors:-

- (i) Particulate substrate oxidation is a very slow process with reaction time constants of several days. According to Clifft and Andrews (ref. 11), the metabolism of particulate substrate occurs at a nearly constant rate because of the hydrolysis rates' being independent of particulate substrate concentration. Since sewage contains about 75 and 40% of the carbonaceous and nitrogenous substrates respectively in particulate form, the peak diurnal oxygen demand will be considerably damped. Clifft and Andrews (ref. 11) found that the oxygen uptake rate was independent of particulate substrate concentrations.

- (ii) Intracellular Substrate Storage Substrate storage material can be present as either particulate substrate absorbed or emmeshed in the activated sludge floc or as intracellular stored substrate. The damping effect of stored particulate substrate has already been discussed in (i) above. The intracellular storage of influent soluble substrate can also have a considerable damping effect on the peak oxygen demand. When substrate absorption into the bacterial cell exceeds the rate of metabolism (as limited by DO concentration) the absorbed substrate is converted into intracellular storage products such as glycogen and poly- β -hydroxybutyric acid (PHB). The stored substrate can then be oxidised when influent substrate loading drops off - thus damping the peak oxygen demand or utilisation rate. From laboratory experiments conducted with a soluble yeast extract, Selna and Schroeder (ref. 22) found that the oxygen uptake response through the transient loading period (square wave transients) is much less than would be predicted by steady-state parameters. They concluded that for municipal treatment plants with quasi-sinusoidal transient wave loading of amplitude 1.5 to 2 times average, the assumption of mean daily parameter values based on steady state conditions is a satisfactory explanation. Based on model simulation of substrate storage activated sludge models, Clifft and Andrews (ref. 11) found that for soluble substrate influent the total oxygen uptake rate lags substrate removal by 2 to 3 hours and oxygen utilisation rate was significantly damped by a factor of about 1.6.
- (iii) Dilution Effect Chen, Roth and Eckenfelder (ref. 23) reported that in a laboratory activated sludge unit treating soluble skim milk, the response of the oxygen uptake rate to transient loading indicates a slow mechanism reaction rate due to the dilution effect of longer hydraulic residence time in the aeration basin.

- (iv) Sludge Age It has generally been accepted that the response of oxygen uptake rate to diurnal substrate load variation is significantly damped in extended aeration process with high sludge age compared to the conventional process. The traditional explanation of high constant endogenous respiration rate is less acceptable in view of the previous discussions in Sections 2.1.3 and 2.1.4. Possible reasons could be the higher hydraulic residence time (hence higher dilution effect) and sludge quantity in the longer sludge age process. This could result in reduced contact between active cell and substrate. Gujer (ref. 12) found that the maximum to average respiration rate of activated sludge subjected to a ratio of maximum to average COD removal rate of 1.5 reduces with increasing sludge age as well as with the increasing ratio of particulate to soluble COD substrate.

In the case of the 4000 ep I.E.A. plant example, the diurnal peak to average oxygen requirement for a diurnal peak to average substrate loading ratio of 3 to 1 can be estimated from the information discussed above.

COR	=	328 kg/d
NOR	=	56 kg/d
TOR	=	384 kg/d
Average TOR	=	16 kg/h
Soluble Substrate COR	=	$0.25 \times 328 = 82$ kg/d (25% soluble)
Soluble Substrate NOR	=	$0.6 \times 56 = 34$ kg/d (60% soluble)
Soluble Substrate TOR	=	116 kg/d
Average Soluble		
Substrate TOR	=	4.8 kg/d

Damping factor for soluble substrate stored intracellularly assumed to be 1.6 (ref. 11 - Clifft and Andrews)

. . Peak soluble substrate TOR = $\frac{3 \times 4.8}{1.6} = 9$ kg/d

Assume peak to average particulate substrate TOR =
1:1 (ref. 11)

.
. . Peak particulate substrate TOR = average particulate TOR
= 16 - 4.8
= 11.2 kg/d

.
. . Peak TOR = 9.0 + 11.2 = 20.2 kg/d

.
. . Peak to average TOR = $\frac{20.2}{16.0} = 1.26$

In view of the long sludge age (30 days) and hydraulic retention time (38 h at ADWF) of the 4000 ep I.E.A. plant, it is possible that provision for diurnal peak oxygen requirement is not necessary. Also operation of the plant on the intermittent basis with effluent decanting after a 3 or 4 hour cycle would provide further damping of the peak hourly oxygen requirement. However, as a prudent measure to take account of changing sludge age etc. a factor of 1.25 may be applied.

2.3.1 Closure of Section 2.3

1. The extended aeration process TOR can be obtained as the sum of the theoretical COR and NOR.
2. A factor of 1.25 may be conservatively applied for the diurnal peak hourly TOR for sizing the aeration equipment hourly oxygenation capacity.
3. For the PWD 4000 ep I.E.A. plant example, TOR can be estimated by a simple rule of thumb figure of 1.7 kg O₂/kg BOD₅ (includes a factor of 1.25 for diurnal peak TOR).

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3. AERATION TESTS

In the previous sections, the theoretical derivation of the oxygen requirements in the activated sludge process and in particular the extended aeration variant of the process was discussed in detail. It was shown that the traditional synthesis and endogenous oxygen requirements method can grossly over-estimate the process oxygen requirements for the extended aeration process. For example, the traditional method can yield COR values exceeding the theoretical maximum COR which is equal to the carbonaceous BOD_u of the substrate. However, these excessive oxygen requirement figures are still commonly quoted in the literature and used in the design of extended aeration processes. One of the reasons for the continuing use of the excessive values in process design is that aeration tests to determine the actual oxygen usage in full scale plants are seldom conducted or reported. In addition to the difficulties in conducting aeration tests in operating full scale plants, misinterpretation of aeration test data can lead to over-estimation of the oxygen transfer capability of the aeration equipment. The over-estimated oxygen transfer capability of the aeration equipment could be misinterpreted as a high process oxygen requirement - hence giving false support to the excessive process oxygen requirement design figures.

It can be seen from the above discussions that accurate assessment of the oxygen transfer rate and efficiency of aeration equipment is not only important for verifying contractor's guarantees but also for checking the validity of the adopted design process oxygen requirements.

Reported oxygenation capacities of aeration equipment are generally for oxygen transfer rates in clean water. The clean water oxygen transfer rates are then generally converted to transfer rates in mixed liquor by adopting literature values for α and β . Aeration tests in activated sludge mixed liquor are seldom carried out because of difficulties in controlling the test conditions such as the MLSS oxygen up-take rates. The present published mixed liquor or

"dirty water" aeration test models rely on the assumption of a constant MLSS oxygen uptake rate for interpretation of the oxygen transfer rate. Also, the present models are generally based on surface aerators where oxygen transfer takes place above the surface of the water. At present, aeration tests models for submerged aerators where oxygen transfer takes place under the water are not as well developed. The main drawback of existing submerged aeration test models is in the method of determining the DO saturation concentration which lies between the surface and mid-depth saturation values. Because of the factors that have been just discussed, accurate data on actual oxygen transfer and usage in operating plants are seldom reported in the literature.

In the following sections, aeration tests in clean and 'dirty' (mixed liquor) water will be discussed for both surface and submerged aerators. Also, the development of a new submerged aeration test model will be presented which hopefully will provide a simpler and better method for calculating oxygen transfer rates for submerged aerators. In addition a new model for interpreting dirty water aeration tests under varying MLSS oxygen uptake rate condition will be presented.

3.1 Clean Water Aeration Test

The clean water aeration test for both surface and submerged aerators will be discussed in Sections 3.1.1 and 3.1.2. Since the surface aeration test model has been well developed and established, Section 3.1.1 will concentrate mainly on the methods of interpreting the test data which have gained a lot of attentions in recent literature. In Section 3.1.3, the theoretical derivation of a new submerged aeration test model will be presented by the author.

3.1.1 Surface Aeration Test

The rate of oxygen transfer in the unsteady-state clean water aeration test is presented by the familiar two-film model linear first order differential equation

$$\frac{dc}{dt} = K_L a (C^* - C) \quad -(1)$$

Where $\frac{dc}{dt}$ = oxygen transfer rate, M/L^3t ,
dt

$K_L a$ = overall volumetric oxygen transfer
coefficient, t^{-1} , hereafter noted as K for simplicity

C^* = dissolved oxygen (DO) saturation concentration
under the test conditions of temperature and
pressure, M/L^3 ,

and C = DO concentration at time t , M/L^3 .

In the surface aeration test C^* is constant and equal to C_{∞}^* , the saturation concentration at infinite time. Therefore, for an initial DO value of C_0 at time zero (t_0), the integrated form of Eq.1 is

$$\ln \frac{(C^* - C_0)}{(C^* - C)} = Kt \quad -(2)$$

$$\text{or } C = C^* - (C^* - C_0)e^{-Kt} \quad -(3)$$

All three Eqs. 1, 2 and 3 can be used to estimate the oxygen transfer parameters K and C^* . The advantages and disadvantages of the three different parameter estimation techniques have been reviewed by Boyle, Berthouex and Rooney (ref. 1), by Stenstrom, Brown and Hwang (ref. 2), and by Brown and Baillo (ref. 3). Briefly, the main disadvantage of the differential or direct method based on Eq. 1 is that the noise in the data is magnified. The process of approximating

the transfer rate dc/dt by taking differences between successive DO concentration values results in a variable with larger error than the error in C itself. The main disadvantage of the log deficit method based on Eq. 2 is that a value of C^* has to be either assumed, measured or determined from book values. An incorrect selection of C^* will bias the estimate of K . Selection of a low C^* will overestimate K . It is considered that the best method for selecting C^* is to estimate a value which gives the "best" straight line (or correlation coefficient) or the minimum residual sum of squares. This subject has been well discussed by Kayser (ref. 4). The only disadvantage of the "exponential" nonlinear least squares method based on Eq. 3 is that the method requires the use of a computer or advanced scientific calculator. This is the method recommended by Brown et al. Also, Stenstrom et al stated that the consensus of all investigators quoted by them is that this is the most desirable of the 3 methods discussed above. The main advantages of this technique summarised by Stenstrom et al and Brown et al are:

1. The nonlinear least squares technique fits the exponential equation directly to the test DO concentration-time data to estimate the oxygen transfer parameters. The other methods require transformation of the test data (i.e. to dc/dt or log deficit values).
2. Truncation of the test data is not required.
3. C^* is estimated directly from the test data without introducing error by incorrectly estimating C^* from book values.
4. The residuals (differences between calculated and measured C values) are more uniform than other methods.
5. Least squares estimates are provided for all 3 parameters K , C^* and C_0 .

6. The parameter estimates appear to be more precise than those obtained by the other methods.

3.1.2 Submerged Aeration Test

One of the basic difference between surface aeration and submerged aeration is in the value of C^* . As defined by Henry's Law, C^* varies with the oxygen partial pressure of the air in contact with the water. For surface aerators, oxygen transfer occurs at the free water surface. Hence C^* is taken as the saturation concentration corresponding to the atmospheric oxygen partial pressure. This C^* is commonly referred to as the "surface" C^* value.

For submerged aerators, oxygen transfer occurs at the air/water interface of the submerged air bubbles. Hence C^* is taken as the saturation concentration corresponding to the average oxygen partial pressure of the air bubbles in the water.

The main difficulty in interpreting submerged aeration test data is that C^* cannot be obtained from standard tables as in the case of surface aeration tests because the oxygen partial pressure of the air bubbles varies with water depth as well as with the oxygen transfer rate. Hence in submerged aeration C^* varies with tank depth as well as with the oxygen transfer capability of the aeration equipment being tested and must be estimated for each individual test. The methods that can be used to estimate C^* are listed below in increasing order of advantage.

- (i) Ignoring oxygen partial pressure reduction due to the transfer of oxygen from the air bubbles to the water, C^* can be taken as the saturation concentration corresponding to the pressure at mid depth (ie. the atmospheric pressure plus the water hydrostatic pressure at mid-depth). This C^* is commonly referred to as the "mid-depth" C^* value.

- (ii) Correcting the "mid-depth" C^* value for oxygen partial pressure reduction by assuming an oxygen transfer efficiency (ie. percentage of oxygen in the air bubbles being transferred into the water). A trial and error solution must be used to find an assumed oxygen transfer efficiency that will agree with the eventually calculated oxygen transfer efficiency. This C^* is commonly referred to as the "corrected mid-depth" C^* value.
- (iii) Adopting the logarithmic mean saturation value of the tank bottom and surface saturation values. This C^* is commonly referred to as the "log mean" C^* value.
- (iv) The C^* value can be measured by running the aeration test until a constant DO concentration is obtained (ie. no change in DO for say 10 to 20 minutes). However, only the final saturation value C_{∞}^* is measured.
- (v) Estimating C^* by the nonlinear least squares estimation technique from the actual DO concentration-time data of the aeration test.

The other basic difference between the surface and submerged aeration test is that C^* is constant throughout the surface aeration test but C^* changes throughout the submerged aeration test. In the surface aeration test the atmospheric pressure remains constant and hence C^* will also remain constant. However, in the submerged aeration test the oxygen partial pressure of the air bubbles changes throughout the test due to the transfer of oxygen from the air bubbles to the water. The oxygen partial pressure reduction is the highest at the start of the test when the oxygen transfer driving force is great because of the low DO concentration in the water. The driving force decreases as the DO concentration rises and hence the oxygen partial pressure reduction falls off throughout the test. Hence C^* increases throughout the test until the final C_{∞}^* value is reached.

It can be concluded from the previous discussions that the exponential nonlinear least squares technique is the most desirable parameter estimation technique. However, in the case of the submerged aeration test C^* varies with time during the unsteady test. Hence an analysis based on Eq. 3 which considers C^* to be constant and equal to C_{∞}^* will yield an apparent volumetric oxygen transfer coefficient K' , instead of a true K . According to Brown et al, the surface aeration Eqs.1 and 3 could be modified as follows for submerged aeration unsteady tests.

$$\frac{dC}{dt} = k' (C_{\infty}^* - C) \quad -(4)$$

$$\text{and } C = C_{\infty}^* - (C_{\infty}^* - C_0)e^{-K't} \quad -(5)$$

The relationship between the true (K) and apparent (K') transfer coefficient is given by the following equation:

$$K = \frac{K'}{1 - K'/2\theta_d} \quad -(6a)$$

$$\text{Where } \theta_d = \frac{(M_o/M_a)G_F Y_d}{C_{\infty}^* V} \quad -(6b)$$

and M_o and M_a = molecular weights of oxygen and air respectively, M/mol;

G_F = mass flow rate of feed gas, M/T;

Y_d = mole fraction of oxygen in dry feed gas;

V = volume of test water, L^3 .

Generally, for systems where the oxygen transfer (stripping or absorption) efficiency is in the order of 0.2 (ie. 20% of oxygen in the air feed is transferred to the water), the true K would be about 10% greater than the apparent K' . However, for high efficiency submerged aeration systems, the oxygen transfer efficiency (OTE) may approach 0.5, and K could differ from K' by more than 30%.

Therefore, it is considered by the present author that in order to apply the exponential nonlinear least squares parameter estimation technique for all submerged aeration tests, an exponential equation which is not based on an assumed constant C^* should be developed. The development of such an equation by the present author is presented below.

3.1.3 Development of a New Submerged Aeration Test Model

As discussed in Section 3.1.1, the non-linear least squares technique provides an elegant and accurate method for estimating the oxygen transfer parameters in the exponential Eq. 3 for the surface aeration test. The technique could also be applied for submerged aeration tests by adopting the modified exponential Eq. 5 which assumes a constant C^* and provides an estimate of the apparent rather than true K . Although the use of Eq. 5 can provide reasonable estimates of the oxygen transfer rate, it is not an accurate model of the submerged aeration test in which C^* is not constant but increases during the test. It is considered by the present author that a varying C^* cannot be ignored in the submerged aeration test model. Submerged aeration test models such as the log mean saturation or log mean deficit models which account for a varying C^* during the test have been presented by Lakin and Salzan (ref. 5) and by Frazer (ref. 6). However, the non-linear least squares technique cannot be applied to these models and the less accurate differential-direct method must be used to estimate the oxygen transfer parameters. Thus, it is the present author's aim to develop in the following sections a new submerged aeration test model which not only accounts for the varying C^* value but also allows the oxygen transfer parameters to be obtained by the better and more straightforward non-linear least squares parameter estimation technique.

3.1.3.1 C^* Function

In the submerged aeration test, oxygen transfer takes place from the rising air bubbles released below the free surface of the water being aerated. As the air bubbles rise from the plane of release to the free surface, their oxygen partial pressure will be

reduced due to the reduction in total pressure (change in hydrostatic pressure with depth) as well as the depletion of their oxygen content. The average total (or hydrostatic) pressure of the air bubbles in the aeration tank is constant throughout the aeration test as the water depth does not change. Therefore, the rate of reduction in oxygen partial pressure during the aeration test will be directly proportional to the rate of oxygen depletion from the rising air bubbles. Thus, a time function for C^* may be developed along the following lines:-

- (i) Since oxygen obeys Henry's Law, C^* is directly proportional to the average oxygen partial pressure of the air bubbles in the aeration tank.
- (ii) The average oxygen partial pressure of the air bubbles increases throughout the test as the rate of oxygen depletion decreases during the test. Therefore C^* must also increase during the test in proportion to the increase in oxygen partial pressure.
- (iii) As the rate of increase in C during the test is equal to the rate of oxygen depletion, the rate of increase in C^* must be in proportion to the rate of increase in C . Hence, the time function of C^* must be exponential and similar to the time function of C as described by Eq. 3. Also, since C^* and C arrive at the maximum value C_{∞} at the same time, both C^* and C must have the same time constant τ (where $\tau = K^{-1}$).

Therefore it is considered that C^* could be approximated by the following first order exponential equation:-

$$C^* = C_{\infty}^* - (C_{\infty}^* - C_0^*)e^{-Kt} \quad \text{---(7)}$$

At $t=0$, $C^* = C_0^* = DO$ saturation concentration at the start of the test, M/L^3 .

At $t=\infty$, $C^* = C_{\infty}^* = \text{DO saturation concentration at the end of the aeration test when dynamic equilibrium has been established, } M/L^3.$

Eq. 7 can now be examined to see if it can model the physical phenomenon of the submerged aeration test. Eq. 7 shows that C^* increases from an initial minimum value, C_0 , to a final maximum value, C_{∞}^* , in an exponential manner. This fits the physical model of the submerged aeration test where at the start of the test, C is at a minimum and the driving force $(C^* - C)$ and, hence, air bubble oxygen depletion rate are at the maximum. Therefore, the air bubbles' oxygen partial pressure is at its lowest value at the start of the test and C^* must also be at a minimum. As C increases during the test, the driving force and oxygen depletion rate also decrease - resulting in increasing average oxygen partial pressure and, hence, also increasing C^* . Both C and C^* will increase toward the maximum or equilibrium saturation value C_{∞}^* during the aeration test. The dynamic equilibrium state will be reached after an extended period of aeration. At the dynamic equilibrium state, both C and C^* will be equal to C_{∞}^* and the driving force and net oxygen transfer rate will be nil. Thus, it can be seen that Eq. 7 is a much more accurate model for C^* in the submerged aeration test than the constant C_{∞}^* adopted in Eqs. 4 and 5 of the modified surface aeration model.

3.1.3.2 C Function

Substituting Eq. 7 for C^* in Eq. 1 gives

$$\frac{dc}{dt} = K (C_{\infty}^* - (C_{\infty}^* - C_0^*) e^{-kt} - C) \quad -(8)$$

Rearranging Eq. 8 gives

$$\frac{dc}{dt} + KC = KA - KBe^{-kt} \quad -(9)$$

where: $A = C_{\infty}^*$
 $B = C_{\infty}^* - C_0^*$

Eq. 9 can now be integrated by Lagrange's "Method of variation of parameters" as follows:-

(a) The homogeneous solution of Eq. 9 is

$$\frac{dc}{dt} = -KC \quad -(10)$$

Integration gives:-

$$C = e^{-kt} \quad -(11)$$

(b) The general solution of Eq. 9 can now be obtained by putting

$$C = uv \quad -(12)$$

where $u = f(t)$
 $v = \text{homogeneous solution} = e^{-kt}$

$$\therefore \frac{dc}{dt} = C' = u'v + uv'$$

or $C' = u'e^{-kt} - Kue^{-kt} \quad -(14)$

Substituting Eq. 14 for C' and Eq. 12 for C into Eq. 9 gives

$$u'e^{-kt} - Kue^{-kt} + Kue^{-kt} = KA - KBe^{-kt} \quad -(15)$$

$$\therefore u'e^{-kt} = KA - KBe^{-kt} \quad -(16)$$

$$\therefore u' = KAe^{kt} - KB \quad -(17)$$

Intergrating Eq. 17 gives

$$u = Ae^{kt} - KBt + D \quad -(18)$$

Substituting Eq. 18 for u in Eq. 12, the general solution becomes:-

$$C = A - KBte^{-kt} + De^{-kt} \quad -(19)$$

Applying the boundary conditions of $C = C_0$ at $t=0$, Eq. 19 gives:-

$$C_0 = A + D \quad -(20)$$

or $D = - (A - C_0) \quad -(21)$

Substituting Eq. 21 for D into Eq. 19 gives:

$$C = A - KBte^{-kt} - (A - C_0)e^{-kt} \quad -(22)$$

or $C = C_{\infty}^* - (C_{\infty}^* - C_0)e^{-kt} - K(C_{\infty}^* - C_0^*)te^{-kt} \quad -(23)$

Eq. 23 is, thus, the new model proposed for the submerged aeration test. It can be seen that:

- (1) Eq. 23 shows that the rate of increase in C is less than that predicted by the modified surface aeration model of Eq. 5 by a factor equal to $K(C_{\infty}^* - C_0^*)te^{-kt}$. This factor accounts for the lowering of the D0 saturation value due to the gas-side oxygen depletion in the submerged aeration test. In the submerged aeration test, the initial D0 saturation value C_0^* is less than the final saturation value C_{∞}^* ; whereas in the surface aeration test, the D0 saturation value is constant and equal to C_{∞}^* throughout the test.

Also, the factor $K(C_{\infty}^* - C_0)e^{-kt}$ decreases with time and is consistent with the fact that the gas-side oxygen depletion rate also diminishes with time.

- (2) The boundary conditions $C=C_0$ at $t=0$ and $C = C_{\infty}^*$ at $t=\infty$ are satisfied by Eq. 23. Also, if C_0 is equal to C_{∞}^* , Eq. 23 reduces to Eq. 3 which is the exponential equation for the surface aeration test.

Therefore, it can be seen that the submerged aeration test is more correctly modelled by Eq. 23 than by Eq. 5. Also, Eq. 23 is a much more general model as it can also be used to describe the surface aeration test because it reduces to Eq. 3 when $C_0^* = C_{\infty}^*$.

Eqs. 3 and 23 can also be derived from the principles of the transient response of a simple first order system. The unsteady-state surface aeration test could be considered as the transient response of a first order system to an input step function which is equal to C_{∞}^* . The unsteady - state submerged aeration test could be considered as the transient response of a first order system to the input function $C_{\infty}^* - (C_{\infty}^* - C_0)e^{-kt}$ which is a combination of a step function C_{∞}^* and an exponential function $(C_{\infty}^* - C_0)e^{-kt}$. The derivations of Eqs. 3 and 23 as a transient first order system are given in Appendix 1.

3.1.4 Numerical Examples

Whether Eq. 23 is in fact a better model of the submerged aeration test than Eq. 5 can be examined by the following numerical examples. If Eq. 23 is a better model than Eq. 5, it should provide lower residuals - i.e. provide a better fit to the test data.

In the following two numerical examples the parameter estimates obtained by the 'exponential' nonlinear least squares estimation technique using Eq. 5 and 23 are compared. In addition, the calculated Standardised Oxygen Transfer Rates (SOTR) are also compared to that obtained by the log mean saturation method presented by Fraser (ref. 6). The different methods are briefly described below.

3.1.4.1 Method 1 - Nonlinear Least Squares Parameter Estimation
Based on Eq. 5

In the nonlinear least squares method referred to as the 'complex' method by Stenstrom et al, the parameters C_{∞}^* , C_0 and k' are estimated by fitting Eq. 5 directly to the observed DO concentration - time data. The 'best' parameter estimates are considered to be the set that provides the least residual sum of squares, $\sum R^2$. The residual, R , being the error or difference between the observed and calculated DO values. The exponential search for the 'best' set begins by evaluating the residual sum of squares of four (guessed) initial sets of parameter estimates.

One of these four sets will provide the greatest $\sum R^2$ and represents the poorest set of parameter estimates. This becomes the first set to be rejected and be replaced by a new and better set with a lower $\sum R^2$. A new set can be generated by calculating the centroid of the remaining three sets (by averaging the remaining three estimates for each parameter) and then projecting from the rejected parameter estimate through the centroid a specific distance. The initial projection distance (called gamma) beyond the centroid is selected at 1.3 times the distance from the rejected parameter estimate to the centroid. If the new set obtained in this manner has greater $\sum R^2$ than the rejected set, gamma is then halved to reduce the projection distance. This process is repeated until a replacement set with lower $\sum R^2$ than the rejected set is obtained. The next worst set is then selected for replacement by another new and better set and the whole process is continued until a termination criterion is obtained. Generally, the termination criterion is an error improvement criterion - the search is terminated when significant improvement in the error, $\sum R^2$, cannot be achieved by further iterations. This condition will occur when the four sets of parameter estimates converge to the optimal values.

When the best estimates for C_{∞}^* and K' with the least $\sum R^2$ have been obtained, the SOTR can be calculated as follows:

K' is corrected to the Standard temperature of 20°C as follows:

$$K'_{20} = K'_T \theta^{(20-T)} \quad -(24)$$

Where K'_{20} = apparent K at 20°C , h^{-1} ,
 K'_T = apparent K at test temp. $T^{\circ}\text{C}$, h^{-1}
 and θ = temp. correction coefficient = 1.024

The effective saturation depth is then obtained as follows:-

$$de = \frac{1}{\gamma} \left\{ \frac{C_{\infty T}^*}{H_T Y_d} - P_b + P_{VT} \right\} \quad -(25)$$

where de = effective saturation depth, m,
 γ = weight density of water, kN/m^3 ,
 $C_{\infty T}^*$ = average saturation concentration at test conditions, mg/L ,
 H_T = Henry's Law constant, mg/L kPa ,
 Y_d = Mole fraction of oxygen in air feed,
 P_b = test atmospheric pressure, kPa ,
 and P_{VT} = test saturated water vapour pressure, kPa .

The saturation concentration $C_{\infty 20}^*$ at standard temperature and pressure (20°C and 1 atmosphere respectively) can then be obtained by:

$$C_{\infty 20}^* = C_{\infty T}^* \left\{ \frac{C_{s20}^*}{C_{sT}^*} \right\} \left\{ \frac{P_s + \gamma de - P_{v20}}{P_b + \gamma de - P_{v20}} \right\} \quad -(26)$$

where C_{s20}^* = book value surface saturation at 20°C , mg/L ,
 C_{sT}^* = book value surface saturation at $T^{\circ}\text{C}$, mg/L ,
 P_s = standard atmospheric pressure, kPa ,
 P_{v20} = saturated water vapour pressure at 20°C , kPa ,
 and the other parameters as defined previously.

The standardised oxygen transfer rate can then be calculated as follows:

$$SOTR = K'_{20} \cdot C_{\infty 20}^* \cdot V \cdot 10^{-3} \cdot 60 \quad -(27)$$

Where V = test water volume, m^3
 $SOTR$ = kg oxygen transferred / h at standard conditions of $20^\circ C$, one atmosphere and at zero DO Concentration.

The standardised oxygen transfer efficiency (stripping or absorption efficiency) can then be obtained,

$$SOTE = \frac{SOTR}{\text{oxygen supply rate}} \quad -(28)$$

where $SOTE$ = % oxygen in air feed absorbed or transferred under the standard conditions of $20^\circ C$, 1 atm. and nil DO ($C=0$).
 and oxygen supply rate = air flow rate (m^3/h) x air density ($1.205 \text{ kg}/m^3$) x weight fraction of oxygen (0.232).

$SOTR$ can also be calculated from the true K as given by Eq. 6a and 6b as follows:-

$$\phi_{dT} = \frac{(M_o/M_a) G_F Y_d}{C_{\infty V}^*} \quad -(29)$$

where ϕ_{dT} = oxygenation coefficient as defined by Eq. 6b
 $K_T = \frac{K'_T}{1 - K'_T/2 \phi_{dT}} \quad -(30)$

where K_T = true transfer coefficient at $T^\circ C$ and defined by Eq. 6a.

The true temperature correction coefficient θ^* can be obtained as follows:

$$\left(\frac{\theta^*}{\theta}\right)^{(20-T)} = \left(\frac{C_{ST}^*}{C_{s20}^*}\right) \left(\frac{2\theta dT - k'T}{2\theta d20 - \theta^{(20-T)} K'_T}\right) \quad -(31)$$

where $\theta d20 = \theta dT \left\{ \frac{C_{sT}^*}{C_{s20}^*} \right\}$ -(32)

Finally, SOTR is obtained as follows:

$$SOTR = K_{20} \text{ Fo } C_{\infty 20}^* \text{ V. } 10^{-3} 60 \quad -(33)$$

Where $\text{Fo} = \text{dimensionless exit depletion factor evaluated at zero DO}$
 $= 1 - (K'_{20}/2\theta d20)$ -(34)

3.1.4.2 Method 2 - Nonlinear Least Squares Parameter Estimation Based on Eq. 23

The nonlinear least square technique for estimating the parameters in the exponential Eq. 23 is the same as those for estimating the parameters in Eq. 5. The only difference is that four parameters k , C_o , C_{∞}^* and C_o are to be estimated instead of three. Also the true K and not the apparent K' is estimated. When the best set of parameter estimates which provides the least $\sum R^2$ has been obtained, SOTR can then be calculated as follows:

The true transfer coefficient at 20°C can be obtained as follows:

$$K_{20} = K_T \theta^{(20-T)} \quad -(35)$$

It should be noted that depending on the test data and whether data at the beginning of the test have been truncated, the DO concentration C_o at time $t = 0$ is not necessarily equal to zero. Hence C_o^* at time $t=0$ also does not necessarily corresponds to the saturation value at $C=0$. The saturation value at $C=0$, C_{co}^* , can be obtained from Eq. 7 and 23. Eq. 23 can be used to determine the time t_{co} for $C=0$. t_{co} can then be substituted into Eq. 7 to find C_{co}^* . The saturation value at standard conditions of 20°C , 1 atmosphere and nil DO can then be obtained as follows:

$$C_{co20}^* = C_{coT}^* \left\{ \frac{C_{s20}^*}{C_{ST}^*} \right\} \left\{ \frac{P_s + 0.5d\gamma - P_{v20}}{P_b + 0.5d\gamma - P_{v20}} \right\} \quad -(36)$$

Where C_{co20}^* = DO saturation at standard conditions of 20°C , 1 atmosphere and zero DO, mg/L,

C_{coT}^* = DO saturation at test temperature, pressure and at $C=0$, mg/L,

$0.5d\gamma$ = average or mid-depth hydrostatic pressure, kPa,

d = water depth, m, and the other parameters as defined previously.

SOTR is then calculated as follows:

$$SOTR = k_{20} C_{co20}^* V \cdot 10^{-3} \quad -(37)$$

The standard oxygen transfer efficiency SOTE can be calculated by Eq. 28.

3.1.4.3 Method 3 - Log Mean Saturation Direct Method

The log mean saturation method presented by Fraser is based on a stepwise integration method to determine the change in the saturation value C^* during the submerged aeration test.

Basically, the method involves the determination of the log mean saturation values at various time intervals of the submerged aeration test, where,

$$C_{LM}^* = \frac{C_{btm}^* - C_{top}^*}{\ln (C_{btm}^*/C_{top}^*)} \quad -(38)$$

Where C_{LM}^* = log mean saturation, mg/L,
 C_{btm}^* = saturation value at bottom of tank, mg/L,
 C_{top}^* = saturation value at top of tank, mg/L,

C_{btm}^* is constant throughout the aeration test and is estimated from the surface saturation value (obtained from standard tables) and pressure of the air bubbles at the tank bottom.

$$C_{btm}^* = C_{ST}^* (P_b + d/10.34). \quad -(39)$$

C_{top}^* varies during the aeration test due to reduction in the mole fraction of oxygen (or oxygen partial pressure) as oxygen is transferred out of the air bubbles during their rise to the top. C_{top}^* for each sampling time interval can be obtained by firstly estimating the oxygen transfer rate OTR, from the change in C during the time interval. The oxygen transfer efficiency, OTE, and Y_{top} for the time interval can then be obtained as follows:

$$O.T.E. = OTR/(\text{oxygen supply rate}) \quad -(40)$$

$$Y_{top} = (1 - OTE)/(4.76 - OTE) \quad -(41)$$

$$\text{and finally } C_{top}^* = C_{ST}^* P_b Y_{top}/0.21 \quad -(42)$$

The driving force ($C_{LM}^* - C_{ave}$) for each time interval can then be obtained from the log mean saturation value and average DO value for each time interval. It can be seen from Eq. 1 that a plot of the oxygen transfer rate, dc/dt , versus the driving force, $C_{LM}^* - C_{ave}$, for the respective sampling time intervals will yield a straight line with a slope equal to the mass transfer coefficient, K.

It should be noted that in this case, K is the mass and not the volumetric transfer coefficient and has the units of kg/h. (mg/L) .

The log mean saturation method accounts for the change in C^* due to the reduction in the oxygen partial pressure in the rising air bubbles during the submerged aeration test. Hence, in this respect it can be considered to be a better model of the submerged aeration test than Eq. 5. However, as discussed earlier the direct method of analysis can introduce significant error in the parameter estimates. The main disadvantages of this method compared to the nonlinear least squares method are:-

- (i) It does not provide least squares estimates for C_{Lm}^* .
- (ii) C_{Lm}^* has to be estimated from book values of C_s^* .
- (iii) The test concentration - time data have to be transformed to dc/dt and C_{Lm}^* - Cave variables which magnify the noise in the data. The transformed variables can have substantially larger error than the error in C itself.
- (iv) The method of curve fitting used to smooth out the noise in the test concentration - time data could produce bias in the parameter estimates.

Because of the above disadvantages, the direct method is not recommended by the authors quoted in references 1, 2 and 3.

In the log mean saturation method, SOTR has to be obtained by trial and error. The log mean saturation value at the standard temperature of 20°C , C_{LM20}^* , has to be calculated from an assumed SOTR value. K_{20} and C_{LM20}^* are then multiplied to obtain the final SOTR value which must be the same as the initially assumed SOTR value. If it is not, then the assumed SOTR value is incorrect and a different value must be selected.

In the following two numerical examples, the three different methods discussed above are compared. In the computer programme written for the nonlinear least squares methods, the exponential search for the 'best' parameter estimates is terminated when the error cannot be further improved after fifteen consecutive halvings of the gamma factor. Also, the computer programme is applicable to both methods. The first method is selected by entering $C_{\infty}^* = C_o^*$ whereby the general Eq. 23 is reduced to the modified surface model Eq. 5. The computer programme is given in Appendix I.

3.1.5 Discussion of Results

The first example is based on data presented by Brown and Bailod (ref. 3). The data were obtained from a clean water field test conducted on a coarse bubble submerged aeration system installed in a 13.1 m diameter tank. The test conditions are as follows:

Ave. water temp. = 14.5°C ; Barometric pressure = 97.8 kPa
Tank depth = 6.16 m ; Tank vol. = 830 m^3
Diffuser submergence = 5.55 m
Air flow rate = $0.426 \text{ m}^3/\text{s}$ at 1 atm., 20°C and 0% R.H.

The second example is based on the clean water submerged aeration test data presented by Fraser (ref. 6). The test conditions are as follows:-

Water temp. = 19.1°C ; Barometric pressure = 0.986 atm.
Water depth = 9.15 m ; Water vol. = 1323 m^3
Air flow rate = $15.57 \text{ m}^3/\text{min}$ at 20°C and 1 atm.

3.1.5.1 Comparisons between Method 1 and Method 2 parameter estimates

Three separate computer runs were conducted using the modified surface aeration model Eq. 5 (method 1) to estimate the parameters C_{∞}^* , C_o and K' by the nonlinear least squares technique.

Different sets of initial parameter estimates were used in each of the computer runs. In Example 1, Table 1A shows that all the three computer runs converged to the same final or 'best' set of estimates. This indicated that the unique or global optimal values for the parameters were obtained. The final set of parameter estimates gave a residual sum of squares, $\sum R^2$, of 0.01617. The estimated error for C is 0.030 mg/L ($=\sqrt{\sum R^2/n}$, where n equals the number of residuals). Similarly, for example 2, all three computer runs converged to the same final or 'best' set of parameter estimates as shown in Table 2A. The final set gave a $\sum R^2$ of 0.03768 and an estimated error of 0.058 mg/L for C.

In method 2, the parameters C_{∞}^* , C_0^* , C_0 and K in Eq. 23 were estimated by the nonlinear least squares technique using the same initial sets of estimates as used in method 1. Tables 1B and 2B showed that for both examples, the computer runs did not converge to the same final or best sets of parameter estimates. However, all the computer runs provided better estimates with lower $\sum R^2$ than obtained by method 1. In Example 1, Run No. 3 gave the best set of parameter estimates with the lowest $\sum R^2$ of 0.01251 and an estimated error of 0.026 mg/L for C. This is a substantial improvement of 29% for $\sum R^2$ and 15% for the estimated error in C over those obtained by method 1. Similarly for example 2, Run No. 3 provided the best set of parameter estimates with the lowest $\sum R^2$ of 0.0273 and an estimated error of 0.050 mg/L for C. This also represents a substantial error improvement over method 1 - by about 38% for $\sum R^2$ and about 16% for the estimated error in C. Tables 1C and 2C show the comparisons for the predicted DO values, C_p , and residuals, $R = C_m - C_p$, obtained by methods 1 and 2. It can be seen that Method 2 provided a marginal improvement in the distribution of the residuals over Method 1.

TABLE 1A EXAMPLE 1 - METHOD 1 - (MODIFIED SURFACE MODEL EQ. 5)

	C_{∞}^* (mg/L)	C_o (mg/L)	K' (min. ⁻¹)
Run No. 1	10.12	0.784	0.1130
Initial sets of estimates	11.40	1.120	0.0869
	12.30	1.360	0.0848
	13.40	1.520	0.0711
Run No. 2	11.35	1.19	0.1150
Initial sets of estimates	11.37	1.25	0.1070
	11.39	1.28	0.1010
	11.41	1.32	0.0952
Run No. 3	11.36087	1.310969	0.112803
Initial sets of estimates	11.37454	1.242567	0.1086574
	11.41373	1.118648	0.09525541
	11.41689	1.131725	0.09437152
Final sets of estimates	11.42645	1.121553	0.08691792
	"	"	"
	"	"	"
	"	"	"
Residual sum of squares of final estimates = 0.01617168			

TABLE 1B EXAMPLE 1 - METHOD 2 (NEW SUBMERGED MODEL EQ. 23)

RUN 1				
	$\frac{C_{\infty}}{C_0}$ (mg/L)	C_0 (mg/L)	$\frac{C_{\infty}}{C_0}$ (mg/L)	K (min. ⁻¹)
Initial Sets of Estimates	10.12	0.784	8.51	0.1130
	11.40	1.120	10.90	0.0869
	12.30	1.360	11.90	0.0848
	13.40	1.520	12.20	0.0711
Final Sets of Estimates	11.41373	1.118648	10.49029	0.9525543
	"	"	"	"
	"	"	"	"
	"	"	"	"
Residual sum of squares = 0.01501388				
RUN NO. 2				
Initial Sets of Estimates	11.35	1.19	8.85	0.1150
	11.37	1.25	9.05	0.1070
	11.39	1.28	9.71	0.1010
	11.41	1.32	10.70	0.0952
Final Sets of Estimates	11.38701	1.196382	9.432811	0.104677
	"	"	9.432809*	"
	"	"	9.432814	"
	"	"	9.43281	"
Residual sum of squares = 0.01251846*				
RUN NO. 3				
Initial Sets of Estimates	11.36087	1.310969	8.523885	0.112803
	11.37454	1.242567	8.99288	0.1086574
	11.41373	1.118648	10.49029	0.09525541
	11.41689	1.131725	10.5715	0.09437152
Final Sets of Estimates	11.38711	1.200221	9.407058	0.1048809
	"	"	"	"
	"	"	"	"
	"	"	"	"
Residual sum of squares = 0.0125089				

TABLE 1C EXAMPLE 1 - MEASURED V's PREDICTED D.O. VALUES

		METHOD 1 - MODIFIED SURFACE MODEL		METHOD 2 - NEW SUBMERGED MODEL	
Time t (min.)	Measured Cm (mg/L)	Predicted Cp (mg/L)	Residual Cm-Cp (mg/L)	Predicted Cp (mg/L)	Residual Cm-Cp (mg/L)
2	2.77	2.766	+ 0.004	2.791	-0.021
4	4.15	4.148	+ 0.002	4.145	+0.005
6	5.35	5.309	+ 0.041	5.294	+0.056
8	6.25	6.285	- 0.035	6.267	-0.017
10	7.08	7.106	- 0.026	7.091	-0.011
12	7.80	7.795	+ 0.005	7.786	+0.014
14	8.34	8.375	- 0.035	8.371	- 0.031
16	8.85	8.862	- 0.012	8.864	- 0.014
18	9.28	9.271	+ 0.009	9.279	+ 0.001
20	9.62	9.615	+ 0.005	9.627	- 0.007
22	9.93	9.904	+ 0.026	9.919	+ 0.011
25	10.24	10.253	- 0.013	10.270	- 0.030
30	10.70	10.667	+ 0.033	10.681	+ 0.019
35	11.00	10.935	+ 0.065	10.943	+ 0.057
40	11.14	11.108	+ 0.032	11.108	+ 0.032
45	11.20	11.220	- 0.020	11.213	- 0.013
50	11.25	11.293	- 0.043	11.279	- 0.029
55	11.30	11.340	- 0.040	11.320	- 0.020
Residual Sum of Squares $\sum R^2$		0.016171/68		0.0125089	
Estimated error, $(\sum R^2/n)^{0.5}$		0.030 mg/L		0.026 mg/L	

N.B. The Cp and Cm - Cp results have been rounded to three decimal point significant figures.

TABLE 2A EXAMPLE 2 - METHOD 1 (MODIFIED SURFACE MODEL EQ.5).

	C_{∞} (mg/L)	C_0 (mg/L)	K' (min. ⁻¹)
Run No. 1	9.8	1.7	0.34
Initial sets of	13.0	2.3	0.27
estimates	15.0	2.6	0.24
	17.0	3.0	0.20
Run No. 2	9.0	1.7	0.40
Initial sets of	11.0	2.0	0.34
Estimates	13.0	2.3	0.21
	16.0	3.0	0.18
Run No. 3	9.0	1.7	0.25
Initial sets of	11.0	2.0	0.22
estimates	14.0	2.6	0.20
	16.0	3.0	0.18
final sets of	12.73754	2.218277	0.1990204
estimates	"	"	"
	"	"	"
	"	"	"
Residual sum of squares of final estimates = 0.03768371			

TABLE 2B EXAMPLE 2 - METHOD 2 (NEW SUBMERGED MODEL EQ. 23)

RUN No. 1				
	$\frac{C_{\infty}}{C_0}$ (mg/L)	C_0 (mg/L)	$\frac{C_{\infty}}{C_0}$ (mg/L)	K (min. ⁻¹)
Initial sets of estimates	9.8	1.7	6.0	0.34
	13.0	2.3	9.0	0.27
	15.0	2.6	10.4	0.24
	17.0	3.0	12.0	0.20
Final sets of estimates	12.69987	2.221248	11.39023	0.2265716
	"	"	"	"
	"	"	"	"
	"	"	"	"
Residual sum of squares = 0.03614324				
RUN NO. 2				
Initial sets of estimates	9.0	1.7	5.0	0.40
	11.0	2.0	7.0	0.34
	13.0	2.3	9.0	0.21
	16.0	3.0	12.3	0.18
Final sets of estimates	12.30866	2.290798	8.422061	0.3085176
	"	"	"	"
	"	"	"	"
	"	"	"	"
Residual sum of squares = 0.02889214				
RUN NO. 3				
Initial sets of estimates	12.69987	2.221248	11.39023	0.2265717
	12.30866	2.290798	8.422061	0.3085176
	12.26994	2.307475	8.148417	0.3166945
	12.26911	2.325237	8.191678	0.3152863
final sets of estimates	12.37773	2.26703	8.928765	0.2931828
	"	"	8.928768 *	0.2931827*
	"	"	8.928768	0.2931827
	"	"	8.928771	0.2931826
* Residual sum of squares = 0.02738562				

TABLE 2C EXAMPLE 2 - MEASURED Vs PREDICTED D.O. VALUES

		METHOD 1 - MODIFIED SURFACE MODEL		METHOD 2 - NEW SUBMERGED MODEL	
Time t (min.)	Measured Cm (mg/L)	Predicted Cp (mg/L)	Residual Cm-Cp (mg/L)	Predicted Cp (mg/L)	Residual Cm-Cp (mg/L)
0	2.29	2.218	+ 0.072	2.267	+ 0.023
0.05	3.21	3.215	- 0.005	3.209	+ 0.001
1	4.06	4.117	- 0.057	4.082	- 0.022
2	5.58	5.672	- 0.092	5.628	- 0.048
3	6.91	6.947	- 0.037	6.923	- 0.013
4	8.07	7.992	+ 0.078	7.996	+ 0.074
5	8.93	8.849	+ 0.081	8.876	+ 0.054
6	9.59	9.551	+ 0.039	9.592	- 0.002
8	10.54	10.597	- 0.057	10.634	- 0.094
10	11.26	11.300	- 0.040	11.300	- 0.040
12	11.79	11.772	+ 0.018	11.718	+ 0.072
Residual Sum of squares $\sum R^2$		0.03768371		0.02738562	
Estimated error $(\sum R^2/n)^{0.5}$		0.058 mg/L		0.050 mg/L	

3.1.5.2 Comparison of SOTR obtained by Methods 1, 2 & 3

In calculating the SOTR and SOTE, the following values were used or calculated:

<u>Method 1</u>	<u>Example 1</u>	<u>Example 2</u>
T, °C	14.5	19.1
K'_T , min ⁻¹	0.0869	0.1990
K'_{20} , min ⁻¹ ,	0.0990	0.2033
$C_{\infty T}^*$, mg/L	11.43	12.74
H, mg/L kPa	0.489	0.445
Yd	0.21	0.21
P_b , kPa	97.8	99.88
P_{vT} , kPa	1.65	2.21
γ , kPa/m (or kN/m ³)	9.8	9.8
de, m	1.55	3.49
C_{s20}^* , mg/L	9.07	9.07
C_{sT}^* , mg/l	10.18	9.24
Ps, kPa	101.3	101.3
P_{v20} , kPa	2.34	2.34
$C_{\infty 20}^*$, mg/L	10.51	12.64
Mo, g/mole	32	32
Ma, g/mole	28.97	28.97
G_F , kg/min	30.84	18.76
V, lx10 ⁻⁶	0.83	1.323
$\emptyset_{dT}$, min ⁻¹	0.754	0.258
K_T , min ⁻¹	0.0922	0.3239
$\emptyset$	1.024	1.024
$\emptyset^*$	1.0242	1.026
K_{20} , min ⁻¹	0.105	0.3315
$\emptyset_{d20}$ min ⁻¹	0.846	0.263
F_o	0.941	0.6135

Method 2

K_T, min^{-1}	0.1049	0.2932
K_{20}, min^{-1}	0.1195	0.2995
Time t when C=0, min	-1.32	-1.08
$C_{\text{coT}}^*, \text{mg/L}$	9.11	7.64
$C_{\text{co20}}^*, \text{mg/L}$	8.35	7.57
d, m	5.55	9.15

The values for C_{S20}^* , C_{ST}^* , P_s , P_b , γ and P_{v20} are as given in Method 1.

Method 3

The values for C_{S20}^* , C_{ST}^* , P_s , P_b and d are as given in Method 1.

TABLE 3 METHOD 1 - COMPARISON BETWEEN K AND K'

	Example 1	Example 2
Apparent K'_{20}, min^{-1}	0.0990	0.2033
True K_{20}, min^{-1}	0.1050	0.3315
SOTR based on K' , kg/h	52	204
SOTR based on true K, kg/h	52	204
SOTE based on K' , %	12	78
SOTE based on true K, %	12	78

It can be seen from Table 3 above that in Example 1 where the SOTE is a moderate 12% the true K is greater than the apparent K' by about 6%. In Example 2 where the SOTE is a high 78%, the true k is greater than the apparent K' by about 63%. However, the SOTR determined from the true K is identical to the SOTR determined from the apparent K' for both examples.

TABLE 4. COMPARISON OF SOTR BETWEEN METHODS 1, 2 & 3

	Example 1	Example 2
Method 1 SOTR, kg/h	52	204
Method 2 SOTR, kg/h	50	180
Method 3A SOTR, kg/h	56	185
Method 3B SOTR, kg/h	56	197
Method 3C SOTR, kg/h	55	190

Table 4 shows the SOTR values obtained by Methods 1, 2 and 3. The three versions of Method 3 (ie. 3A, 3B and 3C) are differentiated by the different concentration - time values used. The measured C_m values (column 2, Tables 1C and 2C) were used in 3A, but the predicted C_p values obtained in Methods 1 and 2 were used in Methods 3B and 3C respectively. (However, it should be noted that the C_m values in example 2 were obtained from a smooth curve of C_m Vs t as the actual test data were quite scattered - especially near the beginning of the aeration test. The curve fitting method used was not stated by Fraser.)

It can be seen from Table 4 that the SOTR values obtained by Method 2 are less than those obtained by Methods 1 and 3. In the case of Example 1 where the SOTE is moderate (12%) the difference between the SOTR values of Method 2 and 1 is quite small, but the difference between Methods 2 and 3 is quite large - about 4% and about 10 to 12% respectively. However, the opposite is observed in Example 2 where the SOTE is high (69%) - the difference in the SOTR values obtained by Methods 2 and 1 is greater (about 13%) than the difference between Methods 2 and 3 (about 3%, 9% and 6% for 3A, 3B and 3C respectively). Therefore, the results show that for a submerged aeration system with moderate or low SOTE, Method 3 could over estimate the SOTR to a greater degree than Method 1 when compared to the SOTR obtained by Method 2. However, for a submerged aeration system with high SOTE, the opposite could occur.

Table 4 also shows that Method 3 can be greatly affected by the noise or scatter in the test data and the SOTR value obtained could be biased by the method employed to smooth out the scatter in the data. In Example 1 where the data is less noisy (as shown by lower $\sum R^2$ obtained in Methods 1 and 2) the SOTR values obtained by Methods 3A and 3B are identical and that obtained by 3C, is only about 2% less. However, in Example 2 where the data is more noisy (as shown by greater $\sum R^2$ obtained in Methods 1 and 2) the SOTR values obtained by the three versions are much more different - a difference of 6% between 3A and 3B, 4% between 3B and 3C and 2% between 3A and 3C.

3.1.6 Further Developments of Submerged Aeration Test Model

In the previous development of the new submerged aeration test exponential Eq. 23

$$C = C_{\infty}^* - (C_{\infty}^* - C_0)e^{-Kt} - (C_{\infty}^* - C_0) Kt e^{-Kt}$$

it was assumed that the time constants for C and C^* are the same. If they are not the same, then Eqs. 1, 7 and 23 can be rewritten as follows:

$$\frac{dC}{dt} = K_1 (C^* - C) \quad -(43)$$

$$C^* = C_{\infty}^* - (C_{\infty}^* - C_0)e^{-K_2 t} \quad -(44)$$

$$C = C_{\infty}^* - (C_{\infty}^* - C_0)e^{-K_1 t} + K_1 (K_1 - K_2)^{-1} (C_{\infty}^* - C_0)(e^{-K_2 t} - e^{-K_1 t}) \quad -(45)$$

Where $k_1 = k_L$ a= inverse of C time constant,

k_2 = inverse of C^* time constant

Using the data given in Example 2, Table 5 below shows the estimates for the parameters in Eq. 45 obtained by the 'complex' nonlinear least squares technique discussed previously. In computer runs No. 1 and 4, K_1 values are less than K_2 . In run No. 3, K_1 is, however, greater than K_2 . In run No. 2, K_1 is almost identical to K_2 . The $\sum R^2$ value for all four sets of parameter estimates is almost the same - the difference between the lowest and greatest $\sum R^2$ is only about 0.6%. It can also be seen that the lowest $\sum R^2$ which is obtained in run No. 4 represents an insignificant improvement of about 0.2% over the $\sum R^2$ obtained by Method 2 (Table 2B). Table 6 below, however, shows a slight improvement in the residuals distribution. Also, the SOTR value obtained by the parameter estimates in run No. 4 differs from the SOTR value obtained by Method 2 by about 5% - being 190 kg/h and 180 kg/h respectively.

TABLE 5 SUBMERGED MODEL WITH K_1 & K_2

	RUN 1	RUN 2	RUN 3	RUN 4
C_{∞}^* , mg/L	12.4146978	12.3823029	12.3971256	12.4646650
C_0 , mg/L	2.2633641	2.2697787	2.2675765	2.2527372
C_0^* , mg/L	9.3278822	8.9225127	8.6969998	10.3641342
K_1 , min ⁻¹	0.2796933	0.2930514	0.3039586	0.2420697
K_2 , min ⁻¹	0.3171066	0.2932025	0.2797926	0.3783246
ΣR^2	0.0274859	0.0273620	0.0273405	0.0273188

TABLE 6 Measured Vs Predicted D.O. Values for Run 4

Time t (min)	Measured Cm (mg/L)	Predicted Cp (mg/L)	Residuals Cm - Cp
0.0	2.29	2.253	+ 0.037
0.5	3.21	3.199	+ 0.011
1.0	4.06	4.075	- 0.015
2	5.58	5.623	- 0.043
3	6.91	6.919	- 0.009
4	8.07	7.991	+ 0.07
5	8.93	8.871	+ 0.059
6	9.59	9.587	+ 0.003
8	10.54	10.635	- 0.095
10	11.26	11.311	- 0.051
12	11.79	11.741	+ 0.049

Therefore, although the improvement in the ΣR^2 obtained by the second submerged aeration model ($K_1 \neq K_2$) over the first model ($K_1 = K_2$) is only 0.2%, the SOTR values obtained by the two models differs by a much greater margin of 5%. The second submerged aeration model may be better than the first model because the interaction between C^* and C is better described by the case where $K_1 \neq K_2$.

The interaction between C^* and C is not shown explicitly in the first submerged aeration mathematical model - but only implicitly by showing that C^* also changes in an exponential manner similar to C . In an interacting system the interaction would change the effective time constants of the interacting system and one time constant would become larger and the other smaller. The interaction would be stronger in a more efficient submerged aeration system where the change in C^* is greater than in a less efficient aeration system. However, the input step function represented by C_{co} at $C=0$ is the major forcing or input function and the exponential part of the overall forcing function representing the change in C^* at $C=0$ to C_{co} is small in magnitude compared to C_{co} . In Example 1, $C_{co} = 0.80 C_{co}$ and in Example 2 where the OTE is very high, $C_{co} = 0.62 C_{co}$. Therefore, in general for most submerged aeration systems installed in activated sludge aeration tanks the effect of the interaction between C^* and C may not be great. This could be the reason for the good fit to the test concentration - time data shown by the two submerged aeration Eqs. 23 and 45 and even by the modified surface aeration Eq. 5. The effect of incomplete mixing can also be represented by a second time constant for both the surface and submerged aeration tests.

It is considered that the submerged aeration model could be further refined, such as by considering it as two first order systems in series, or by including the effect of incomplete mixing. Such further developments are however beyond the scope of this project. It is considered that the new submerged aeration test model represented by Eq. 23 is more satisfactory for interpreting submerged aeration test data and is superior to the modified surface aeration model represented by Eq. 5.

3.1.7 Closure for Section 3.1

- (1) The nonlinear least squares parameter estimation technique is the most advantageous technique for interpreting aeration test data.
- (2) For surface aeration tests, the method can be used to obtain the oxygen transfer parameters in the familiar surface aeration model exponential equation:

$$C = C^* - (C^* - C_o)e^{-kt} \quad -(4-1)$$

- (3) For submerged aeration tests, the method can be used to obtain the oxygen transfer parameters in the new submerged aeration model exponential equation developed by the present author:

$$C = C_{\infty}^* - (C_{\infty}^* - C_o)e^{-kt} - (C_{\infty}^* - C_o)kt e^{-kt} \quad -(4-2)$$

Where the variation in C^* during the aeration test is represented by the following exponential equation:

$$C^* = C_{\infty}^* - (C_{\infty}^* - C_o)e^{-kt} \quad -(4-3)$$

- (4) It is considered that the method in 3 above is more advantageous than the log mean saturation/deficit method and the modified surface aeration model method because of the following points:

- (i) It fits the test data better than the modified surface aeration model.
- (ii) It provides a simpler and more straight forward method for calculating the oxygen transfer rate. Air flow rate and equivalent depth D₀ saturation concentration are not required to be estimated to calculate the standardised oxygen transfer rate (SOTR).

- (iii) The modified surface aeration model only provides an estimate of the apparent oxygen transfer coefficient and not the true coefficient.
- (iv) The log mean saturation/deficit method is less precise as it relies on the differential-direct method for estimating the oxygen mass transfer coefficient. Also, SOTR has to be obtained by a trial and error analysis.

3.2 Dirty Water Aeration Test

The model for oxygen transfer in mixed liquor is very similar to that for oxygen transfer in clean water except that an additional factor, the DO uptake rate of the MLSS has to be introduced. If all flows to the aeration tank are stopped, the rate of change in DO concentration during the aeration test can be represented by the following equation:-

$$\frac{dc}{dt} = K_L a' (C_s' - C') - r \quad - (1)$$

Where $\frac{dc}{dt}$ = oxygen transfer rate, mg/L. h,

$$K_L a' = K_L a \text{ in MLSS, } h^{-1},$$

$$C_s' = \text{DO saturation concentration in MLSS, mg/L,}$$

$$C' = \text{DO concentration in MLSS, mg/L, and}$$

$$r = \text{DO uptake rate of MLSS, mg/L.h, assumed to be constant during the aeration test.}$$

Rearranging equation (1) gives:-

$$\frac{dc}{dt} = K_L a' C_s' - r - K_L a' C' \quad - (2)$$

Hence, a plot of $\frac{dc}{dt}$ versus C' will yield a straight line with a slope equal to $K_L a'$.

Alternatively, as proposed by Kayser (ref. 4), Eq. 2 can be transformed to an equation equivalent to that for a clean water test. For an unsteady-state aeration test in mixed liquor, the maximum DO concentration will be achieved when $dc/dt=0$ and this maximum DO concentration can be described as the apparent saturation value, C_s^{*} . Substituting C_s^{*} into Eq. 2 gives:-

$$\frac{dc}{dt} = (K_L a' C_s' - r) - K_L a' C_s^{*} = 0 \quad - (3)$$

$$\therefore C_s^{*} = C_s' - \frac{r}{K_L a'} \quad - (4)$$

$$\text{or } r = K_L a' (C_s' - C_s^{*}) \quad - (5)$$

Substituting Eq. 5 for r into Eq. 2 gives:-

$$\frac{dc}{dt} = (K_L a' C_s' - K_L a' C_s' + K_L a' C_s^{*}) - K_L a' c' \quad - (6)$$

$$\text{or } \frac{dc}{dt} = K_L a' (C_s^{*} - C') \quad - (7)$$

Eq. 7 is analogous to the Eq. 1 in Section 3.1.1 for the clean water aeration test. Hence the nonlinear least squares parameter estimation technique can also be used to determine $K_L a'$ in the same way as for clean water aeration test. The integration of Eq. 7 will give analogous exponential equations to that described in Sections 3.1.1 to 3.1.6 for the clean water tests as follows:

1. For Surface Aeration Test

$$C' = C_s^{*'} - (C_s^{*'} - C'_o)e^{-K't} \quad -(8)$$

2. For Submerged Aeration Test

$$C' = C_{s\infty}^{*'} - (C_{s\infty}^{*'} - C_o')e^{-K't} - (C_{s\infty}^{*'} - C_{so}^{*'})K'te^{-K't} \quad -(9)$$

In the development of the above equations, r is assumed to be constant. If r is not constant it will most probably decrease exponentially to a constant value. In this case $C_s^{*'}$ will increase exponentially and hence the surface aeration model will be analogous to the submerged aeration model. However the time constant for $C_s^{*'}$ may be different to the time constant for C . This may be represented by the following general equations for both surface and submerged mixed liquor aeration tests with an exponentially decreasing r and increasing $C_s^{*'}$:-

$$C' = C_{s\infty}^{*'} - (C_{s\infty}^{*'} - C_o')e^{-K_1t} + \frac{K_1(C_{s\infty}^{*'} - C_{so}^{*'})}{(K_1 - K_2)} (e^{-K_2t} - e^{-K_1t}) \quad -(10)$$

Where $C_s^{*'}$ $C_s^{*'}$ $C_{s\infty}^{*'} - (C_{s\infty}^{*'} - C_{so}^{*'})e^{-K_2t}$ -(11)

Once $K_L a'$ and $C_s^{*'}$ have been determined from the aeration test data, the oxygen transfer rate (OTR') at process conditions can be calculated from Eq. 1 as follows.

$$OTR' = K_L a' (C_s' - C').V.10^{-3} \quad -(12)$$

Where OTR' = field or process condition oxygen transfer rate, Kg O₂/h,

$K_L a'$ = oxygen transfer coefficient in mixed liquor, h⁻¹,

C'_s = mixed liquor DO saturation concentration at
field test conditions, mg/L,

C' = DO concentration of mixed liquor, mg/L

V = mixed liquor volume in aeration tank, m^3

3.2.1 Closure for Section 3.2

- (1) The dirty water (mixed liquor) aeration test model can be represented by the following equation:-

$$\frac{dc}{dt} = K_L a' (C'_s - C') - r \quad \text{---(5-1)}$$

- (2) If r is constant, Eq. 5-1 can be transformed to an equation analogous to the clean water aeration test equation as follows:-

$$\frac{dc}{dt} = K_L a' (C_s^{*'} - C') \quad \text{---(5-2)}$$

Where C_s^{*} = apparent DO saturation concentration obtained at the end of the aeration test when dynamic equilibrium is established.

- (3) Hence, the dirty water aeration test parameters, $K_L a'$ and C_s^{*} , can be obtained in a similar manner to the clean water test parameters by the nonlinear least squares estimation technique discussed in Section 3.1.

- (4) The mixed liquor DO saturation concentration (C'_s) can be obtained from the test's apparent saturation concentration (C_s^{*}) by the following equation:-

$$C'_s = C_s^{*'} + (r/K_L a') \quad \text{---(5-3)}$$

- (5) Alternatively, if r is not constant, C'_s can be obtained by aerating a settled MLSS sample to saturation.
- (6) The oxygen transfer rate (OTR') under field process conditions can be calculated as follows:

$$OTR' = K_L a' (C'_s - C'). V. 10^3 \quad -(5-4)$$

3.3 Specifying Aeration Equipment Oxygenation Capacity

The oxygenation capacity for the aeration equipment must be selected to meet the process peak diurnal TOR as determined in Section 2.3. However, the aeration equipment oxygenation capacity is generally specified at standard conditions since the equipment's field OTR' cannot be predicted with much confidence due to the present lack of data and knowledge in this area. As discussed in a review paper by Stenstrom and Gilbert (ref. 8), the field OTR' is affected by the wastewater characteristics, degree of treatment, type of aeration device, turbulence, process conditions and tank geometry - hence making the field performance of an aeration device unique in each application. Therefore the process TOR must be converted to the SOTR value in accordance with the following relationship between the aeration equipment's field OTR' and SOTR:-

$$SOTR = \frac{C_{20}^* \theta^{20-T}}{\alpha (\beta C_T^* - C')} OTR' \quad -(1)$$

Where SOTR = oxygen transfer rate in clean water at standard conditions, Kg O₂/h,

OTR' = oxygen transfer rate in mixed liquor and field (process) conditions, Kg O₂/h,

C_{20}^* = DO saturation concentration in clean water at 20°C, mg/L

C_T^* = DO saturation concentration in clean water at field temperature, mg/L,

α = K_{La}'/K_{La} , ratio of mixed liquor to clean water K_{La} ,

β = C_s'/C^* , ratio of mixed liquor to clean water DO saturation concentration,

θ = temperature correction coefficient

C' = process mixed liquor DO concentration, mg/L

In the extended aeration process, it is considered that a mixed liquor DO concentration of 1 mg/L during diurnal peak load conditions will be adequate for the process.

Assuming $\alpha = 0.85$, $\beta = 0.9$ and $\theta = 1.024$ as adopted by the P.W.D. for design of the intermittent extended aeration process, Eq. 1 gives:-

$$SOTR = \frac{9.07 \times 1.024^{20-T}}{0.85 (0.9C_T^* - 1)} OTR' \quad -(2)$$

Assuming $T = 20^\circ\text{C}$, Eq. 2 gives

$$SOTR = 1.5 OTR' \quad -(3)$$

For the 4000 ep IEA plant example, it was estimated in Section 2.3 that the process peak diurnal TOR was about 1.7 Kg $\text{O}_2/\text{Kg BOD}_5$ load. Hence the SOTR of the aeration equipment can be specified as 2.55 Kg $\text{O}_2/\text{Kg BOD}_5$.

This value is about 6% higher than the 2.4 Kg $\text{O}_2/\text{Kg BOD}_5$ adopted by the PWD which does not make any allowance for the diurnal peak loading factor (1.25 adopted in the above example). However, the PWD's aeration equipment specification specified that the equipment must be capable of providing a maximum SOTR (at the treatment works

site) that is 12.5% greater than the normal SOTR of 2.4 Kg O₂/Kg BOD₅ applied - in effect providing a safety factor of 1.125. Hence the aeration equipment is required to provide a normal SOTR of 2.4 Kg O₂/h and a maximum SOTR of 2.7 Kg O₂/h.

3.3.1 Closure for Section 3.3

- (1) Each application of aeration equipment in the field is unique as the OTR is affected by several factors such as the wastewater characteristics, type of aeration device, tank configuration, process operation conditions, etc.
- (2) Since the factors are difficult to predict, the aeration equipment oxygenation capacity is generally specified as the SOTR which can be more readily estimated by the Contractor and performance guarantees can thus be provided with more confidence.
- (3) The process design engineer will, thus, have to convert the process oxygen requirement or field OTR' to the SOTR as follows:-

$$SOTR = \frac{C_{20}^* \theta^{20-T}}{\alpha (\beta C_T^* - C')} OTR' \quad (6-1)$$

- (4) The α , β and θ coefficients are affected by the factors as stated in (1) above and it is considered that data available at present are inadequate for accurate predictions under different field conditions.
- (5) Adopting typical values of 0.85 for α , 0.9 for β and 1.024 for θ , the relationship between SOTR and OTR' at 20°C and 1 mg/L DO concentration is as follows:-

$$SOTR = 1.5 OTR'$$

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- (6) Hence, for a peak diurnal TOR of $1.7 \text{ Kg O}_2/\text{Kg BOD}_5$ applied as determined in Section 2.3 for the 4000 ep. I.E.A. plant example, the aeration device must be capable of providing a SOTR of $2.55 \text{ Kg O}_2/\text{Kg BOD}_5$.
- (7) The estimated SOTR in (6) above is in close agreement with the PWD's design criteria of a normal SOTR of $2.4 \text{ Kg O}_2/\text{Kg BOD}_5$ and a maximum SOTR of $2.7 \text{ Kg O}_2/\text{Kg BOD}_5$.

EXTENDED AERATION ACTIVATED SLUDGE PROCESS

OXYGEN REQUIREMENTS

PART 2: EXPERIMENTAL WORK

EXPERIMENTAL WORK

The experimental work involved collecting data from a full scale IEA plant to determine its actual oxygen requirements for comparison with the theoretical oxygen requirements as discussed in Part 1 of this report.

The field work involved:

1. The monitoring of influent flow over a 24-hour period and the collection of influent, effluent and MLSS samples.
2. Determination of the oxygen transfer rate of the aeration equipment in both clean water and MLSS.
3. ATP measurements to determine the active biomass fraction of MLSS.

Samples collected in the field were kept in a freezer and transported back to the laboratory for analysis. Influent and effluent samples were analysed for BOD₅, BOD₃₀ (BOD_u), COD, amm.N, org.N, nte.N, pH, SS and VSS. MLSS samples were analysed for SS, VSS, COD, BOD₅ and BOD₃₀. Some influent and MLSS samples were also analysed for the biodegradable VSS fraction by aerating the samples in the laboratory for 30 days or more.

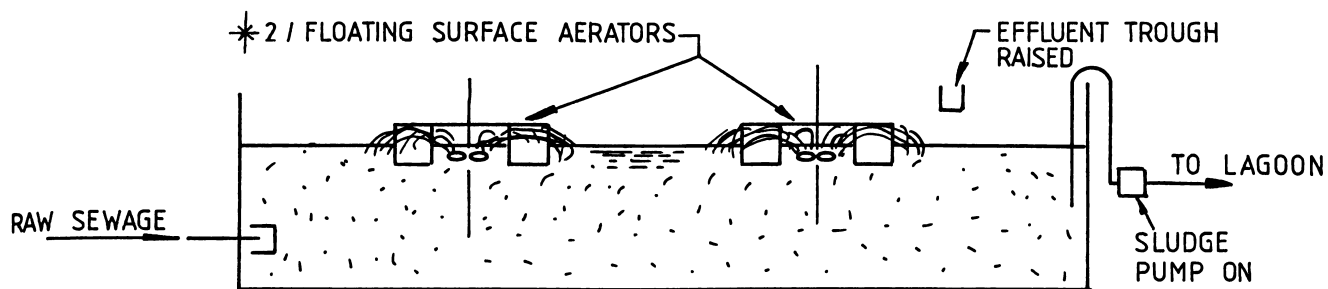
4. PLANT DESCRIPTION

Most of the experimental data were collected from a 16,000 ep IEA plant at Forster. Forster is a very popular coastal resort area and the influent to the plant is predominantly domestic in origin. Because of the seasonal variation in plant loading, four IEA tanks were constructed in parallel. All four tanks are in operation during the peak summer holiday loading period and only two or three of the tanks are required during the off-holiday seasons.

Sewage is delivered via two rising mains to an inlet works consisting of a mechanically raked bar screen, Parshall flume and a grit chamber. The sewage is then split four ways to feed each of the four IEA tanks. Effluent from the IEA tanks discharges to an effluent flow balancing tank and is then pumped to an ocean outfall. Excess sludge is pumped from the IEA tanks to two sludge lagoons and the supernatant liquor displaced from the lagoon gravitates back to the inlet works.

Each of the 4000 ep IEA tanks measures 37m x 12.5m x 5m and is operated in a cyclic manner. The adopted cycle length is 4 hours beginning with a 2.5 hour aeration phase, followed by a 1 hour settling phase and ending with a 0.5h decanting phase. The operating phases are shown diagrammatically in Fig. 1. Aeration is provided through two submerged clusters of nozzles (jets). MLSS is pumped through the nozzles to which air is supplied under pressure from an air blower. Thus, a mixture of air and water is jetted out in the horizontal plane from the cluster, providing both mixing and oxygen to the MLSS. A submersible pump is attached on top of each jet cluster and air is supplied by one blower connected to both clusters.

When aeration ceases, the MLSS settles in the tank leaving a clear layer of treated effluent at the top to be decanted off. Decanting is achieved by lowering a trough from TWL to BWL at a rate of 16.7mm/min. TWL and BWL are at 4.27 and 3.27 metre depths respectively. The trough extends across the whole width of the tank

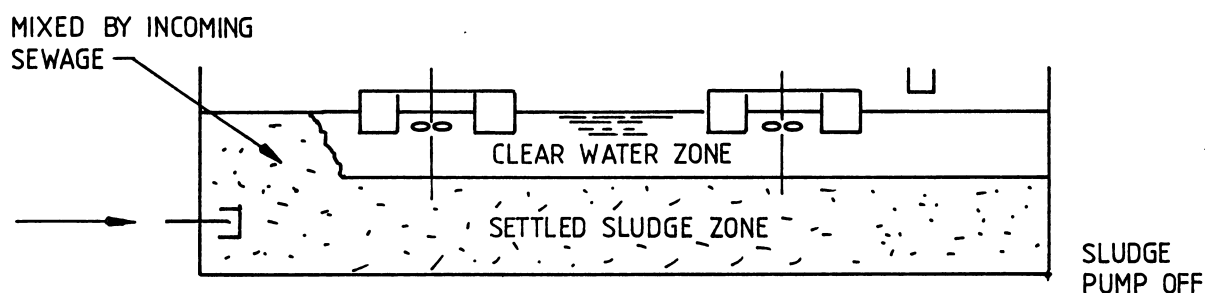


OXIDATION OR REACTION PHASE.

(AERATION / MIXING)

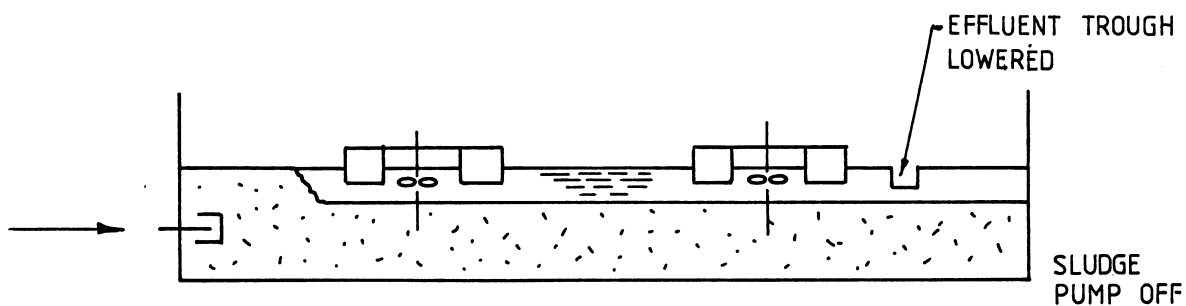
NOTE

SLUDGE PUMP OPERATES
DURING AERATION / MIXING
PHASE.



SEPARATION PHASE.

(I) SETTLING PERIOD



SEPARATION PHASE.

(II) DECANT PERIOD

* NOTE

SUBMERGED AERATORS
USED AT FORSTER.

FIGURE 1.

OPERATING PHASES.

(12.5m) and is connected to two outlet pipes by a butyl rubber seal. Floating scum skimmers are attached to the trough to prevent scum from being decanted with the effluent.

The critical operational requirement in obtaining the desired effluent quality from the IEA tank is to prevent the entrainment of sludge in the effluent being decanted. Sludge will not be scoured from the settled sludge blanket if a depth of 1m is maintained between the top of the sludge blanket and the water surface during decanting. At the design MLSS of about 4600 mg/l, the sludge blanket will settle to a level 1m below the BWL if the SVI (after 60 mins. settling) is 150 ml/g or less. If the sludge SVI exceeds 150 ml/g, the MLSS concentration will have to be lowered to maintain the required 1m depth between the top of the sludge blanket and water surface. Theoretically, if the SVI increases to 300 ml/g (very poor bulking sludge) the MLSS concentration will have to be lowered to half the design value. Except for a greater quantity of sludge to be wasted, this will not have any significant effect on the effluent quality - as effluent quality will not be affected by operating at an F/M of 0.08 instead of 0.04 kg BOD₅/kg MLSS per day.

Excess sludge is pumped from the IEA tank to the sludge lagoons by a positive displacement pump (2.5 l/s) during the aeration phase. The operating sludge age is thus determined by dividing the tank MLSS volume by the volume of sludge wasted per day.

More detailed information on the IEA plant can be found in References 9 and 18 for Sections 2.1 to 2.3.

5. BIODEGRADABLE VSS FRACTIONS

Biodegradable VSS fractions of MLVSS and influent sewage VSS were determined in order that the theoretical COR and sludge age could be calculated. 250ml samples of MLSS and sewage were aerated in the laboratory for periods of 35 to 40 days. Air diffuser stones connected to aquarium tank air pumps were used to continuously aerate the samples held in 600ml flasks. Approximately every 7 days, 10ml of the samples were removed for VSS concentration determinations.

5.1 Biodegradable MLVSS Fraction

MLSS samples from the Forster IEA and the Castle Hill conventional activated sludge plants were tested for comparison. Because of the foaming caused by the aeration, a considerable amount of solids adhered to the flask wall above the liquid and formed into a dried crust which was difficult to dislodge.

The results obtained are shown in the following Table 1.

Table 1 - Decrease in MLVSS Concentration with Aeration

Time d	Forster Tank 1			Castle Hill		
	MLSS mg/l	MLVSS	MLVSS /MLSS	MLSS	MLVSS	MLVSS /MLSS
0	3490	2810	0.81	2790	2260	0.81
8	3680	2850	0.77	1950	1490	0.76
14	2900	2290	0.79	1500	1150	0.77
22	2330	1740	0.75	1570	1070	0.68
29	2320	1680	0.72	1020	770	0.75
35	1930	1450	0.75	-	-	
40				1160	860	0.74

Ignoring the solids adhering to the flask wall, the degradable MLVSS fraction can be estimated from the difference in MLVSS concentration between day 0 and day 35 or 40. For example, degradable MLVSS fraction for the MLSS sample from Forster Tank No. 1 is as follows:

$$\text{Degradable MLVSS fraction} = \frac{2810 - 1450}{2810} = 0.48$$

The actual degradable fraction would be somewhat less as some of the MLVSS lost would be adhered to the flask wall and not degraded. This factor can be overcome by estimating the amount of VSS degraded from the change in the MLVSS to MLSS ratio rather than from the total loss in MLVSS concentration. The amount of MLVSS degraded can be estimated from the following equation:

$$\frac{X_1 - x}{Y_1 - x} = \frac{X_2}{Y_2} \quad - (1)$$

where X_1 = Initial MLVSS
 Y_1 = Initial MLSS
 x = MLVSS degraded
 X_2/Y_2 = Lowest MLVSS/MLSS ratio obtained

Hence, degradable fraction = x/X_1 . The calculated results are shown in Table 2 below.

Table 2 - Biodegradable MLVSS Fractions

	Forster Tank No. 1	Castle Hill
Biodegradable Fraction	0.38	0.50
(Ignoring SS on flask wall)	(0.48)	(0.66)

The lowest MLVSS/MLSS ratios (0.72 and 0.68) and the lowest MLVSS concentrations (1450 and 770 mg/l) if ignoring SS on flash wall, were used to calculate the degradable MLVSS fractions in Table 2.

The biodegradable MLVSS fraction of 0.38 obtained for the Forster IEA plant agrees closely with the adopted figure of 0.35 for the extended aeration process. The higher figure of 0.50 for the Castle Hill conventional activated sludge plant also agrees with the theory that the biodegradable fraction would be higher in a conventional plant operating at a lower sludge age.

5.2 Biodegradable Sewage VSS Fraction

Two 24-hour composite raw sewage samples from the Forster IEA plant were tested. In addition, a grab influent sample from the Bathurst IEA plant was also tested. The sewage from Forster is predominantly domestic in nature whereas the sewage from Bathurst has a significant portion of trade wastes. The results obtained are shown in Table 3 below.

Table 3 - Biodegradable Sewage VSS Fraction

Time d	Forster 1		Forster 2		Time d	Bathurst	
	SS mg/l	VSS mg/l	SS mg/l	VSS mg/l		SS mg/l	VSS mg/l
0	160	160	130	130	0	593	553
7	100	100	220	180	10	110	110
14	115	115	165	155	17	130	100
20	90	90	315	285	23	160	125
32	90	90	115	110	35	110	105
35	80	80	110	105	38	120	85
Degradable Fraction	0.50		0.19			0.85	

The results shown in Table 3 are disappointing as they indicate that the biodegradable VSS fraction varies from about 0.19 to 0.85. For the Forster No. 1 sample, the VSS to SS fractions were unaccountably all equal to 1.0. Sample 2 from Forster indicated considerable increases in both VSS and SS, probably due to bacterial growth. Clumping of solids into a few large sludge flocs (up to 1cm in diameter) occurred in all three samples such that taking of representative samples for solids determinations was most difficult. It is considered that in order to obtain reliable results, the sludge flocs must be properly dispersed prior to obtaining samples for solids determination. Also, the contribution of non-biodegradable VSS from bacterial growth in the samples should be taken into account. Although the results are not very reliable, it can be reasonably accepted that the non-biodegradable VSS fraction of the influent sewage would not be less than the figure of 0.1 adopted in Section 2.1, as the highest biodegradable VSS fraction obtained is 0.85.

6. MLSS ACTIVE BIOMASS FRACTION

Microbial ATP measurements were conducted to provide an estimation of the active biomass fraction in MLSS. ATP is a high-energy phosphate nucleotide component found in all living cells and serves as a prime energy carrier. It is generated during certain oxidation-reduction reactions and used during biosynthesis reactions. Because ATP takes part in all metabolic pathways, it is of paramount importance in the energy budget of the living cells and its level in the cells is thus strictly regulated. Hence, any particular type of cell has a relatively constant level of ATP under normal conditions. However, changes in the metabolic rate due to stress, toxins etc., will reflect the cellular ATP level. ATP is thus an excellent parameter for measuring microbial activity or viability. It also has the advantage over other compounds such as DNA (deoxyribonucleic acid) that it is specific to living cells. ATP has a fast turnover time of less than one second and is rapidly broken down in dead cells by autolysis in a few minutes (Ref. 1, Henk Vanstaen). Whereas DNA, cell protein, organic nitrogen etc., are partially retained by non-viable cells.

In order to determine the active or viable biomass mass fraction of activated sludge, the ATP content of a 100% viable activated sludge must be known. The viable fraction of any MLSS can then be obtained by dividing the respective ATP/MLSS ratio by the maximum ATP/MLSS ratio at 100% viability. According to Nelson and Lawrence (Ref. 2), the MLSS culture is essentially 100% viable when grown at the highest growth rate before washout occurs - i.e. at MCRT very close to zero (0.5 days or less).

The ATP assay is based on the bioluminescence (light-producing) reaction between ATP and the luciferase enzyme derived from fireflies. This reaction is linear and the light output (photons) is directly proportional to the amount of ATP present if all other components are present in excess. The light output is measured in Relative Light

Units (RLUs) which are converted to ATP concentrations based on calibrations with ATP standards. Although the methodology has been available for over 20 years at least, the test has not been widely applied due to the difficulty in ATP extraction techniques and in obtaining highly purified ATP and the luciferase enzyme. However, with the availability of specialised commercial reagents and equipment in the last few years, the ATP assay is now relatively simple to perform. The reagents are, however, rather expensive - especially when only a small number of tests is involved.

6.1 Test Procedures

The first step in the assay of microbial ATP is to extract the ATP from the bacterial cells. Traditionally, boiling water, buffer (e.g. TRIS-EDTA), acids (e.g. perchloric), solvents and other compounds have been used. Unfortunately, some of these extraction procedures may result in a falsely low ATP value. According to Stanley (Ref. 3), a number of the above extractants are potent quenching (colour absorbing) agents whilst others require some post-extraction processing prior to analysis. According to Roe and Bhagat (Ref. 4) and Patterson, Brezonik and Putnam (Ref. 5), the extraction process is also very dependent on temperature and maximum extraction may not occur until 100°C is reached. Hence, boiling the extractant in a water bath for 10 minutes as adopted by some authors may not be adequate to achieve complete extraction.

To overcome the above problems, proprietary extractant or "releasing reagent" was used for the present experiment. Also, to overcome the quenching effect due to the presence of colouring matters and turbidity in the MLSS sample, the method of "internal standardisation" as discussed by Stanley (Ref. 4) was adopted. This is carried out by performing a second light measurement on the sample after a known amount of ATP standard (usually in sufficient quantity to produce 2 to 5 times the amount of light measured by the first reading) has been added to the sample. The increase in the second light reading is due to the amount of ATP added.

The commercial reagents and light measuring equipment used are manufactured by the firm of LUMAC. These consisted of a portable luminometer (Model 1070), 25mM Hepes buffer (LUMIT BUFFER), purified luciferin-luciferase enzyme (LUMIT-HS), nucleotide releasing agent for microbial cells (TRANS-NRB) and ATP standards. The test procedures adopted are as follows:

1. Dilute MLSS sample with distilled water (10:1 dilution ratio).
2. Pipette 0.5ml of diluted MLSS into a cuvette and mix vigorously with 0.5ml of TRANS-NRB nucleotide releasing reagent. It is claimed that the extraction of ATP through the cell wall and membrane by the extractant is complete within 10 sec. (2:1 dilution in this step).
3. Let the suspension settle and pipette 100 μ l to a cuvette (triplicates). If required, the sample may be further diluted with LUMIT buffer (50 μ l to 450 μ l; 10:1 dilution).
4. Insert the cuvette in the counting chamber of the luminometer for light measurement.
5. Press the instrument "START" button and inject LUMIT-HS as soon as the "INJECT" light comes on.
6. Measure the light emission on "SIGNAL" measuring mode and record the highest RLU (Relative Light Unit) reading (X_1) obtained by the ATP and LUMIT-HS reaction.
7. Remove the sample from the counting chamber and add 10 μ l ATP standard, press "START" button and replace sample in the counting chamber after the "INJECT" light has come on. Omit the first two RLU readings (erroneous due to moving of the cuvette) and record the third RLU reading (X_2).

8. Obtain a RLU reading on a blank made up from 50 μ l of TRANS-NRB and 450 μ l LUMIT buffer. (LUMIT, TRANS-NRB and LUMIT-HS are registered trade names for the reagents).

The ATP in the sample was calculated as follows:

$$\text{ATP/RLU} = \frac{\text{ATP (added)}}{X_2 - X_1} = K \quad - (1)$$

where K = ratio of ATP to RLU in $\mu\text{gATP/RLU}$

$$\therefore \text{ATP in sample} = K (X_1 - \text{blank}) \mu\text{g}/100\mu\text{l} \quad - (2)$$

$$\therefore \text{ATP in MLSS sample} = K(X_1 - \text{blank}) \times D \mu\text{g}/100\mu\text{l} \quad - (3)$$

$$= K(X_1 - \text{blank}) \times D \times 10^4 \mu\text{g}/\text{l} \quad - (4)$$

(where D = dilution ratio)

$$\therefore \text{ATP/MLSS ratio} = \frac{K(X_1 - \text{blank}) D \times 10^4}{\text{MLSS concentration}} \mu\text{g}/\text{mg} \quad - (5)$$

The maximum ATP/MLSS ratio of 5.5 μg ATP/mg MLSS for 100% viable MLSS reported by Roe Jr. and Bhagat (Ref. 4) was adopted to calculate the active biomass (or viability) fraction of the MLSS samples. The maximum figure of 5.5 μg ATP/mg MLSS was obtained by the authors by extrapolating a plot of ATP/MLSS against MCRT (ranging from 0.41 to 26.57 days) to a MCRT of zero. ATP/MLSS ratio at maximum viability was found to be much lower by other researchers. Upadhyaya and Eckenfelder (Ref. 6) by extrapolation obtained a maximum ratio of only 1.11 $\mu\text{gATP/mgMLVSS}$. However, the plot did not include any data with MCRT less than 10 days. Nelson and Lawrence (Ref. 2) by extrapolation obtained a maximum ratio of 2.5 $\mu\text{gATP/mgMLVSS}$ (data based on MCRT from 0.5 to 12 days). However, Roy, LeDuy and Roy (Ref.7) also obtained quite high ATP/MLVSS ratio in a 1-year survey of

a full scale plant - occasionally reaching as high as 4 to 5 $\mu\text{gATP}/\text{mgMLVSS}$. Roe and Bhagat (Ref. 4) postulated that the lower values could be due to less efficient extraction (water bath instead of hot sand bath being used to boil the extractant) or to the ATP measurement taken when the culture was not completely viable. The lower values obtained could also be due to quenching or inhibiting factors as discussed by Stanley (Ref. 3). Since none of these factors is likely to be present in the procedures adopted by the present author, it is considered that the higher ratio of 5.5 $\mu\text{gATP}/\text{mgMLSS}$ for 100% viable MLSS is more appropriate for estimating the active biomass fraction for this experiment. The active biomass fraction, f_a , is thus calculated as follows:

$$f_a = \frac{K(X_1 - \text{blank}) D \times 10^4}{(\text{MLSS conc.}) 5.5} \quad - (6)$$

6.2 Test Results

Several MLSS samples collected from the Forster and Bathurst IEA plants and the Castle Hill conventional activated sludge plant were tested. The results obtained are shown in the following Tables (RLU data given in Appendix A).

Table 4 - Forster IEA Plant: Active MLSS Fraction

	$\mu\text{gATP}/\text{mg MLSS}$	Active MLSS Fraction
Tank 1: Afternoon sample	0.40	0.07
: Morning sample	0.16	0.03
Tank 2: Afternoon sample	1.10	0.20
: Morning sample	0.19	0.03
Tank 3: Afternoon sample	0.59	0.11
: Morning sample	0.59	0.11
	Ave. = 0.51	Ave. = 0.09

Table 5 - Bathurst IEA Plant: Active MLSS Fraction

	$\mu\text{gATP/mg MLSS}$	Active MLSS Fraction
Tank 1: am start aeration	0.51	0.09
: pm start aeration	0.38	0.07
: am end aeration	0.61	0.11
: pm end aeration	0.61	0.11
Tank 2: am start aeration	0.48	0.09
: pm start aeration	0.72	0.13
: am end aeration	0.43	0.08
: pm end aeration	0.68	0.12
	Ave. = 0.55	Ave. = 0.10

Table 6 - Castle Hill Conventional Plant: Active MLSS Fraction

	$\mu\text{gATP/mg MLSS}$	Active MLSS Fraction
Sample 1	3.39	0.62
Sample 2	2.29	0.42
Sample 3	2.84	0.52
	Ave. = 2.84	Ave. = 0.52

Table 4 and 5 showed that only about 10% of the MLSS are active biomass for the Forster and Bathurst plants when operating at F/M of 0.05 and 0.03 kgBOD₅/ kg MLSS/day respectively. Whereas the Castle Hill plant operating at F/M of 0.2 to 0.25 has an active biomass fraction of 52%. The results obtained from both the Forster and Bathurst IEA plants tend to agree with the results published by other authors that the active biomass or viability fraction level off at

about 15 to 20% at long sludge age (Ref. 4 and 8). The higher ATP level and active biomass fraction for the Castle Hill conventional activated sludge plant also agree with the results obtained by Roe and Bhagat (Ref. 4) and by Roy et al (Ref. 7). However, caution must be taken in comparing the present results with previously reported results because of the different extraction and ATP standardisation technique used. Most of the published ATP results were based on the use of boiling Tris buffer for ATP extraction and RLU Vs ATP curves based on ATP standards prepared in distilled water.

Based on the active biomass fraction obtained, the F/M ratios for the Forster and Bathurst IEA plants were 0.5 to 0.3kg BOD₅/kg active biomass respectively and the F/M for the Castle Hill conventional plant was about 0.38 to 0.48kg BOD₅/kg active biomass. Therefore, the F/M ratios for the three plants were almost the same when based on the actual active biomass instead of the MLSS values. This supports the author's postulation that the F/M should remain substantially the same for all substrate limited activated sludge processes as discussed in Section 2.1.3.1. This also supported the author's view that process oxygen requirements based on linear plots of oxygen uptake rate versus F/M traditionally adopted for conventional activated sludge plants are not appropriate when extrapolated to extended aeration processes.

7. PLANT LOADINGS

In order to verify the theoretical TOR for extended aeration plants as discussed in Part 1 of this report, a survey was conducted to establish the actual TOR of the Forster IEA plant. This consisted of measuring the plant's loading and performance, and oxygen transfer rate of the aerators.

7.1 Hydraulic Loading

For the month of October, 1984, the flow to the plant as recorded by the inlet flow meter ranged from 2.24 to 3.94 ML/d, averaging 2.77 ML/d. The higher flows in excess of 3 ML/d would be associated with the rainy days during the period. Only 3 of the 4000 ep IEA tanks were in use during this period as the fourth tank is only required during the Christmas holiday period. Based on the P.W.D. design loading parameter of 240 l/ep.d, average hydraulic load for the period was 11,540 ep or 3850 ep/tank (96% of design hydraulic load).

Each IEA tank was operated on a 4-hour cycle providing 6 cycles per day. The variation in flow per cycle over the 24-hour period from 11.00 a.m. 29/10/84 to 11.00 a.m. 30/10/84 was measured by recording the change in water level in the No. 1 IEA tank with a "Manning" dipper flow and level recorder (Model F3000). The inflow per cycle can be estimated by the change in water level in the tank just before and after the decant phase. The chart recording of the water level in the tank over the 24-hour period is given in Appendix B. To account for the inflow during decant, the plot of the rising water level was extrapolated to the end of the decant phase. The waste sludge pumped from the tank during each cycle was also included in the estimate of the inflow to the tank. The variation in the flow for the six cycles during the 24-hour period is shown in Table 7 below.

Table 7 - Variation in Cycle Flow for Tank No. 1

Date	Time	Cycle No.	Flow (kl)	% of Average Flow
29/10/84	11 am to 3 pm	1	140.8	96
	3 pm to 7 pm	2	151.0	103
	7 pm to 11 pm	3	184.3	125
30/10/84	11 pm to 3 am	4	80.7	55
	3 am to 7 am	5	136.2	93
	7 am to 11 am	6	188.0	128
Total = 881 kl/day				
Average = 146.8 kl/cycle				

Table 7 shows that the peak cycle loading occurred during cycle No. 6 between 7 am to 11 am and the minimum occurred during cycle No. 4 between 11 pm to 3 am. However, the peak to average cycle loading ratio is only about 1.3:1 and the peak to minimum being only about 2.3:1. Cyclic operation of the tank thus provided a significant dampening of the diurnal variation in the hydraulic loading for the process. The total flow for the day to Tank 1 was 881 kl/d (3700 ep hydraulic load). The flow recorded by the plant's flow recorder was 3.17 ML or 1056.7 kl/d per tank which is about 20% greater than that recorded in Tank No. 1. Uneven flow splitting between the 3 tanks was probably the cause of the discrepancy.

7.2 Carbonaceous Substrate Loading

The carbonaceous substrate loading on the Forster IEA plant for the 24-hour period from 29th to 30th October, 1984, was obtained by collecting composite samples of the influent and analysing for BOD₅, COD and BOD₃₀ concentrations. Flow to IEA Tank No. 1 was also monitored during this period (discussed in Section 7.1), thus enabling the substrate mass loading on the tank to be calculated.

Composite effluent samples from Tank No. 1 were also collected and analysed for substrate removal performance. The composite influent and effluent samples were also analysed for nitrogen and solids. MLSS samples were also collected and analysed for BOD₅, COD, BOD₃₀, SS and VSS for mass balances to be calculated. These results will be discussed in latter sections.

Influent samples were collected just downstream of the inlet works grit chamber and included the supernatant returned from the sludge lagoon. Flow weighted samples were collected by an automatic sampler (Manning Model No. F4040) which was automatically paced by a portable flow recorder (Manning Model No. F3000) to collect a sample after every 13 kl of flow has passed through the inlet flume. The samples were composited every 4 hours corresponding to the 4-hour cycle of Tank No. 1. The substrate loading per cycle was calculated by multiplying the 4-hourly composite sample substrate concentrations by the corresponding flow to the tank. The results obtained are as follows:

Table 8 - Carbonaceous Substrate Loading - Tank 1

Cycle No.	Flow kl	BOD ₅		COD	
		mg/l	kg	mg/l	kg
1	140.8	137	19.3	381	53.6
2	151.0	274	41.4	762	115.1
3	184.3	370	68.2	791	145.8
4	80.7	290	23.4	615	49.6
5	136.2	242	33.0	498	67.8
6	188.0	87	16.4	264	49.6
24hr Total	881		201.7		481.5
Average	146.8	229	33.6	547	80.3

7.2.1 BOD₅ Loading

The BOD₅ loading on Tank 1 over the 24-hour period was measured to be 201.7 kg. Based on the unit loading design parameter of 0.07 kg/ep.d, the load is equivalent to about 2900 ep or 73% of the IEA tank's BOD₅ design loading. This is less than the hydraulic loading of about 3700 ep (based on unit flow of 240 l/ep.d) due to the sewage being weaker than the assumed BOD₅ concentration of 292 mg/l BOD₅. The measured average BOD₅ concentration was about 229 mg/l. The average BOD₅ loading per cycle was about 33.6 kg and the peak to average and peak to minimum cycle loading ratios were 2:1 and 4:1 respectively. The peak to average cycle BOD₅ loading is, however, still less than the theoretical ratio of 3:1 assumed for the diurnal hourly variation in Section 2.3. Hence, the cyclic operation and dilution effect possibly provided a damping factor of 50% on the diurnal peak to average BOD₅ loading. Combining this with the damping effects of the slow particulate substrate degradation rate and storage of soluble substrate during peak loading (theoretically found to reduce a 3:1 ratio to 1.25:1 in Section 3), it is likely that allowance for diurnal peak oxygen demand is not required for the IEA tank.

7.2.2 COD Loading

The COD test could give a close approximation of the process TOR if the amount of chemically oxidisable inorganics (e.g. sulphides) and non-biodegradable organics present were low. For a completely biodegradable carbonaceous substrate, the COD would be equivalent to about 1.1 times the BOD_u (assuming 10% of substrate ending up as non-biodegradable cell mass). If the BOD_u is about 1.46 times BOD₅, then COD of the substrate would be about 1.6 times BOD₅. Reported COD/BOD₅ ratio for domestic sewage range from about 1.25 to 2.5. The COD/BOD₅ ratios of the sewage flow to IEA Tank 1 are given in Table 9 below.

Table 9 - COD/BOD₅ Ratio - IEA Tank No. 1

Cycle	1	2	3	4	5	6
COD/BOD ₅	2.78	2.78	2.14	2.12	2.05	3.03

The average COD/BOD₅ ratio was 2.4 and is within the reported range for domestic sewage. The COD loading over the 24-hour period was 481.5 kg. The maximum to average and maximum to minimum cycle COD loading ratios were 1.8 and 2.9 respectively - being slightly less than the BOD₅ variation of 2:1 and 4:1 for peak to average and peak to minimum cycle BOD₅ respectively.

7.2.3 BOD_u Loading

Determination of the BOD_u of the carbonaceous substrate was attempted by the addition of nitrification inhibiting chemicals as recommended in Standard Methods (Ref. 9). However, the results obtained were most disappointing as some of the nitrification inhibited BOD_u results turned out to be greater than the uninhibited BOD_u results. Inhibited BOD_u three to eight times greater than the uninhibited BOD_u were obtained. It is postulated that the inhibiting chemicals added were biodegradable when incubated over 30 days, although they were not degraded over the 5 days of incubation for the BOD₅ test.

The carbonaceous substrate BOD₅ was thus estimated indirectly from the TOR which was determined by the use of a well nitrified effluent seed in the BOD₃₀ test. The BOD₃₀ thus represents the carbonaceous substrate oxygen demand plus the nitrification oxygen demand of the sample. The carbonaceous substrate BOD_u can thus be estimated by subtracting the nitrification oxygen demand from the BOD₃₀ value. Because of the high dilution of the sample for the BOD₃₀ test, a mass balance of the amm.N and nte.N of the sample before and after 30 days incubation was not feasible. Thus the actual

degree of nitrification in the BOD₃₀ sample cannot be measured. Instead, it was assumed that nitrification of the amm.N was completed within the 30 days incubation period and the nitrification oxygen demand (NOD) was calculated from the amm.N concentration in the raw sewage sample. The test results and calculated carbonaceous substrate BOD_u are given in the following Table 10.

Table 10 - Calculated Carbonaceous BOD_u - Forster Tank 1

Cycle No.	Flow kl	BOD ₃₀ with Nitrification mg/l	amm.N mg/l	Calculated NOD mg/l	Calculated Carbonaceous BOD _u mg/l	kg
1	140.8	305	26.6	122	183	25.8
2	151.0	500	38.1	175	325	49.1
3	184.3	857	34.4	158	699	128.8
4	80.7	976	27.2	125	851	68.7
5	136.2	400	26.6	122	278	37.9
6	188.0	233	13.4	62	171	32.2
Total	881					342.5
Ave.	146.8	514.8			389	57.1

N.B. (1) NOD = amm.N x 4.6

(2) BOD_u = BOD₃₀ - NOD

The calculated carbonaceous BOD_u loading was about 342.5 kg over the 24-hour period. The peak to average and peak to minimum cycle loading were 2.3:1 and 4:1 respectively and agreed very closely with the ratios of 2:1 and 4:1 respectively obtained for the BOD₅ loading.

The BOD_u/BOD₅ ratios are shown in the following table:

Table 11 - Carbonaceous BOD_u/BOD_5 ratios

Cycle No.	1	2	3	4	5	6	
BOD_u/BOD_5	1.34	1.19	1.89	2.93	1.15	1.97	1.75

The BOD_u/BOD_5 ratios for the six cycles vary from a minimum of 1.15 to a maximum of 2.93. The maximum ratio appears to be unreasonably high compared to the rest of the results and also to literature values. The mean ratio was 1.75 with a standard deviation (σ_{n-1}) of 0.68. All the results were within the range of 1.75 ± 0.68 except for the maximum ratio of 2.93. The mean ratio of 1.75 is about 20% greater than the theoretical value of 1.46 adopted in Section 2.1.1. However, if the maximum value of 2.93 is omitted, the mean ratio becomes 1.51 which is only about 3% higher than the theoretical ratio. Omitting the maximum ratio, also reduces the overall error of the data with σ_{n-1} being 0.39. Hence, the possible error is significantly reduced from $\pm 39\%$ to $\pm 26\%$ (estimated error = mean/σ_{n-1}). Therefore, the BOD_u/BOD_5 results obtained agreed reasonably well with literature values if the unusually high ratio of 2.93 is omitted.

7.3 Nitrogen Loading

In order to assess the process NOR, the nitrogen loading on the IEA Tank No. 1 was measured over the 24-hour period. The results are shown in the following Table 12.

Table 12 - Nitrogen Loading - IEA Tank No. 1

Cycle No.	Flow kl	amm.N		org.N		nte.N	amm.N + org.N	
		mg/l	kg	mg/l	kg	mg/l	mg/l	kg
1	140.8	26.6	3.7	11.5	1.6	0.6	38.1	5.4
2	151.0	38.1	5.8	22.4	3.4	0.6	60.5	9.2
3	184.3	34.4	6.3	17.1	3.2	0.6	51.5	9.5
4	80.7	27.2	2.2	18.2	1.5	0.6	45.4	3.7
5	136.2	26.2	3.6	18.2	2.5	0.4	44.8	6.1
6	188.0	13.4	2.5	10.1	1.9	0.6	23.5	4.4
Total	881		24.1		14.1			38.3
Ave.	146.8	27.4	4.0	16.0	2.4	0.6	43.5	6.4

Table 12 shows that the total nitrogen loading on IEA Tank 1 over the 24-hour period was about 38.8 kg which was made up of about 63% amm.N, 36% org.N and 1% nte.N. The peak to average and peak to minimum cycle loading were about 1.6 and 2.9 respectively for amm.N and about 1.5 and 2.6 respectively for combined amm.N and org.N. Hence, the cycle variation in nitrogen loading was less than that for BOD_u (2.3:1 and 4:1), BOD₃₀ (2.1:1 and 3.6:1), COD (1.8:1 and 2.9:1) and BOD₅ (2:1 and 4:1). The measured average tot.N concentration of about 44 mg/l agrees very closely with the P.W.D. design figure of about 42 mg/l (10g/ep at 240 l/ep) adopted in Section 2.2 whereas the measured average BOD₅ concentration of about 229 mg/l is about 30% less than the P.W.D. design figure of 292 mg/l (70g BOD₅/ep and 240 l/ep).

8. PLANT PERFORMANCE

8.1 General Effluent Quality

In addition to the effluent samples collected in conjunction with the influent samples during the 24-hour period of 29-30/10/84, effluent samples were also collected periodically during 1984 from all four IEA tanks. A summary of the 50 and 90 percentiles of all the effluent samples' test data for 1984 is given in Table 13 below.

Table 13 - Forster IEA Plant Effluent Quality for 1984

	No. of Samples	Range		50 Percentile	90 Percentile
BOD ₅ , mg/l	60	1 to	17	4	11
SS, mg/l	60	1 to	30	6	17
amm.N, mg/l	61	0 to	3.6	0.5	2.6
org.N, mg/l	61	0.6 to	3.1	1.4	2.7
nte.N, mg/l	61	0.2 to	26.7	8.1	35.4
tot.N, mg/l	61	4.0 to	28.4	12.8	27.8
MLSS, mg/l	49	1190 to	7210	3257	4659
MLVSS, mg/l	49	1010 to	5870	2621	3705
SVI, ml/g	42	77 to	353	244	394

The test data were found to fit the log-normal distribution. The statistical 50 and 90 percentiles were calculated as follows:

$$\bar{X} = \sum \ln x / n \quad - (1)$$

$$\sigma = [\sum (\ln x - \ln \bar{X}) / n]^{1/2} \quad - (2)$$

$$X'_{50\%} = \ln \bar{X} + \ln \sigma \quad - (3)$$

$$X'_{90\%} = \ln \bar{X} + 1.28 \ln \sigma \quad - (4)$$

It can be seen from Table 13, that the plant performed very well throughout 1984 with the effluent BOD₅ and SS well within the design standard of 15 mg/l BOD₅ and 20 mg/l SS (50% obtained were 4 and 6 mg/l respectively) and the required/licence standard of 20 mg/l BOD₅ and 30 mg/l SS (90% obtained were 11 and 17 mg/l respectively). Also, the effluent amm.N level was low - 50% and 90% being 0.5 and 2.6 mg/l respectively. Based on the average influent amm.N concentration of 27.4 mg/l obtained over the 24-hour period of 29-30/10.84, this represents over 98% nitrification on the average and over 90% nitrification for 90% of the time. The effluent nte.N level was higher than the design figure of about 3 mg/l (i.e. 90% denitrification) - 50% and 90% figures obtained were about 8 and 35 mg/l nte.N respectively. This indicated that the plant was being operated with an excess of aeration. It can be concluded from the above results that the plant's aerators have sufficient oxygenation capacity to meet the plant's process oxygen requirements.

The SVI ranged from 77 to 353 ml/g with a 50% value of 244 ml/g and 90% value of 394 ml/g. The sludge was thus generally of a bulking nature. However, good quality effluent was still being obtained by increasing sludge wasting to maintain a minimum depth of 1m between the water level and the top of the sludge blanket during effluent decanting. Sludge wasting was generally set at 20 minutes per cycle during the aeration phase. However, if the sludge blanket level starts to rise, the waste sludge pump could be set to run continuously over several hours for a few days to reduce the sludge blanket height. This resulted in the wide operating range of 1190 to 7210 mg/l in the MLSS concentration as shown in Table 13.

8.2 Percentage Substrate Biodegradation

The percentage substrate degraded can be obtained by a mass balance based on the influent and effluent data collected over the 24-hour sampling period. The mass balance should also include the waste sludge stream. Hence, MLSS samples were also collected and analysed for BOD₅, COD and BOD₃₀. During the sampling period, the waste sludge pump was set to pump for 20 minutes during the aeration phase of each cycle. The waste sludge pump capacity is 2.5 l/s, hence sludge wasted per cycle was 3 kl. The nitrogen content of the MLSS samples collected during the 24 h sampling period was not determined but MLSS samples were collected at a later date and analysed for org.N contents. MLSS samples were collected from all 4 IEA tanks and the results are shown in Table 14 below.

Table 14 - Org.N Content in MLVSS

Tank No.	1	2	3	4	Average
Org.N mg/l	143	202	272	299	229
MLVSS mg/l	1880	2330	2950	2630	2447.5
org.N/MLVSS	7.6%	8.7%	9.2%	11.4%	9.4%

As shown in Table 14, the average MLVSS nitrogen content of 9.4% agrees reasonably with the values of 12.3% for bacterial cells and 7% for aerobically digested sludge reported by Eckenfelder Jr. (Ref. 10). Adopting the biodegradable MLVSS fraction of 38% (given in Table 2. Section 8.1 for Tank No. 1), the theoretical MLVSS nitrogen content can be calculated as follows:

$$\begin{aligned}\text{MLVSS org.N} &= (0.38 \times 0.123 + 0.62 \times 0.07) \\ &= 9.0\%\end{aligned}$$

Therefore, a MLVSS nitrogen content of 9% is a reasonable figure to adopt for mass balances around Tank No. 1 for the 24-hour sampling period. The lower value of 7.6% in Table 14 for Tank No. 1 was on a sample collected on 8/3/85 which had a much lower MLVSS concentration of 1880 mg/l compared to the sample collected on the 30/10/84 which had a MLVSS concentration of 2810 mg/l. It is considered that the value of 9% would be more representative of the MLSS sample collected during the 24-hour period.

The effluent quality and test results of the MLSS sample collected over the 24-hour sampling period are given below.

Table 15 - Effluent BOD and COD Results

Cycle No.	Flow kl	Prim. Sed. Seed				Eff. Seed		COD	
		BOD ₅		BOD ₃₀		BOD ₃₀			
		mg/l	kg	mg/l	kg	mg/l	kg	mg/l	kg
1	137.8	10	1.4	3	0.4	11	1.5	103	14.2
2	148.0	4	0.6	1	0.1	1	0.1	51	7.5
3	181.3	11	2.0	23	4.2	24	4.4	99	17.9
4	77.7	11	0.9	23	1.8	24	1.9	99	7.7
5	133.2	8	1.1	37	4.9	7	0.9	66	8.8
6	185.0	3	0.6	9	1.7	1	0.2	51	9.4
Total	865		6.6		13.1		9.0		65.5

Table 16 - Effluent Nitrogen Results

Cycle No.	Flow kl	amm.N		org.N		nte.N		tot.N	
		mg/l	kg	mg/l	kg	mg.l	kg	mg/l	kg
1	137.8	0.8	0.11	2.5	0.34	2.2	0.30	5.5	0.76
2	148.0	0	0	2.0	0.30	3.6	0.53	5.6	0.83
3	181.3	2.0	0.36	2.8	0.51	4.3	0.78	9.1	1.65
4	77.7	2.0	0.16	2.8	0.22	4.3	0.33	9.1	0.71
5	133.2	1.4	0.19	1.4	0.19	3.9	0.52	6.7	0.89
6	185.0	0	0	2.0	0.37	2.0	0.37	4.0	0.74
Total	865		0.82		1.93		2.83		5.58

Table 17 - Calculated Effluent Carbonaceous BOD_u

Cycle No.	BOD ₅ kg	Calculated BOD _u (Prim. Seed BOD ₃₀ -NOR)		Calculated BOD _u (Eff. Seed BOD ₃₀ -NOR)		Highest BOD Value for Cycle	
		kg		kg		kg	
1	1.4	-0.11		0.99		1.4	
2	0.6	0.1		0.1		0.6	
3	2.0	2.54		2.74		2.74	
4	0.9	1.1		1.2		1.2	
5	1.1	3.7		0.03		3.7	
6	0.6	1.7		0.2		1.7	
Total	6.6	9.03		5.26		11.34	

As shown in Table 15, there is no consistency in the difference between the effluent BOD₅ and BOD₃₀ values obtained with either primary sedimentation or IEA tank effluent seed. The very low BOD₃₀ values obtained (values less than the BOD₅ values) are probably due to the high dilution of the samples and test error associated with the low BOD concentration of the samples. This resulted in the negative value and values less than the BOD₅ for calculated effluent carbonaceous BOD_u values shown in Table 17. Therefore, the highest BOD value for each cycle is adopted as an approximation of the effluent carbonaceous BOD_u to avoid negative members. It is considered that this will not introduce significant error in the eventual mass balance as the concentrations involved are small. The BOD_u values were obtained by subtracting the oxygen required for nitrifying the effluent amm.N (i.e. 4.6 x amm.N concentration) from the BOD₃₀ value.

Table 18 - MLSS BOD and COD Results

Cycle No.	Eff. Seed								Calculated Carbonaceous	
	BOD ₅		BOD ₃₀		COD		amm.N		BOD _u	
	mg/l	kg	mg/l	kg	mg/l	kg	mg/l	kg	mg/l	kg
1	1607	4.8	2886	8.66	3993	12	0.8	0.002	2884	8.65
2	1607	4.8	2886	8.66	3993	12	0	0	2888	8.66
3	1607	4.8	2886	8.66	3993	12	2.0	0.006	2877	8.63
4	1607	4.8	2886	8.66	3993	12	2.0	0.006	2877	8.63
5	1607	4.8	2886	8.66	3993	12	1.4	0.004	2881	8.64
6	1607	4.8	2886	8.66	3993	12	0	0	2886	8.66
Total		28.8		51.96		72		0.018		51.87

- NOTE: (1) Waste MLSS flow = 3 kl/cycle for all 6 cycles.
- (2) amm.N concentration is taken to be the same as that in the effluent sample.
- (3) Carbonaceous $BOD_u = BOD_{30} - 4.6 \times \text{amm.N}$
- (4) Only one MLSS sample was collected during the 24-hour sampling period. It was assumed that the sample collected during cycle No. 6 is representative for cycles 1 to 5 also.

Table 19 - MLSS Nitrogen Results

Cycle No.	amm.N		org.N		nte.N		tot.N	
	mg/l	kg	mg/l	kg	mg/l	kg	mg/l	kg
1	0.8	0.002	253	0.759	2.2	0.007	256	0.768
2	0	0	253	0.759	3.6	0.011	256.6	0.770
3	2.0	0.006	253	0.759	4.3	0.013	259.3	0.778
4	2.0	0.006	253	0.759	4.3	0.013	259.3	0.778
5	1.4	0.004	253	0.759	3.9	0.012	258.3	0.775
6	0	0	253	0.759	2.0	0.006	255	0.765
Total		0.018		4.554		0.062		4.634

- NOTE: (1) amm.N and nte.M concentrations in MLSS waste stream were assumed to be the same as that in the effluent.
- (2) org.N of waste MLSS stream based on a value of 9% N in the MLVSS. MLVSS concentration was 2810 mg/l, hence
 $\text{org.N concentration} = 0.09 \times 2810 = 253 \text{ mg/l.}$

Assuming steady-state on a daily basis, mass balances for the IEA Tank No. 1 based on the influent, effluent and waste MLSS data given above are presented in the following tables.

Table 20 - BOD₅ and Carbonaceous BOD_u Mass Balances

Cycle No.	Influent		Effluent		Waste MLSS		Biodegraded			
	BOD ₅	BOD _u	BOD ₅	BOD _u	BOD ₅	BOD _u	BOD ₅	BOD _u		
	kg	kg	kg	kg	kg	kg	kg	%	kg	%
1	19.3	25.8	1.4	1.4	4.8	8.65	13.1	68	15.75	61
2	41.4	49.1	0.6	0.6	4.8	8.66	36.0	87	39.84	81
3	68.2	128.8	2.0	2.74	4.8	8.63	61.4	90	117.43	91
4	23.4	68.7	0.9	1.2	4.8	8.63	17.7	76	58.87	86
5	33.0	37.9	1.1	3.7	4.8	8.64	27.1	82	25.56	67
6	16.4	32.2	0.6	1.7	4.8	8.66	11.0	67	21.84	68
Total	201.7	342.5	6.6	11.34	28.8	51.87	166.3	82	279.29	82

Table 21 - BOD₃₀ and COD Mass Balances

Cycle No.	Influent		Effluent		Waste MLSS		Biodegraded			
	BOD ₃₀	COD	BOD ₃₀	COD	BOD ₃₀	COD	BOD ₃₀	COD		
	kg	kg	kg	kg	kg	kg	kg	%	kg	%
1	42.9	53.6	1.5	14.2	8.66	12	32.74	76	27.4	51
2	75.5	115.1	0.1	7.5	8.66	12	66.74	88	95.6	83
3	157.9	145.8	4.4	17.9	8.66	12	144.84	92	115.9	79
4	78.8	49.6	1.9	7.7	8.66	12	68.24	87	29.9	60
5	54.5	67.8	0.9	8.8	8.66	12	44.94	83	47.0	69
6	43.8	49.6	0.2	9.4	8.66	12	34.94	80	28.2	57
Total	453.4	481.5	9.0	65.5	51.96	72	392.44	87	344	71

Table 22 - Nitrogen Mass Balance

Cycle No.	Influent		Effluent		Waste MLSS		Nitrogen Denitrified		Nitrogen Nitrified	
	tot.N	nre.N	tot.N	nre.N	tot.N	nre.N	kg	%	kg	%
	kg	kg	kg	kg	kg	kg	kg	%	kg	%
1	5.4	0.08	0.8	0.30	0.77	0.01	3.83	71	4.06	75
2	9.2	0.09	0.8	0.53	0.77	0.01	7.63	83	8.08	88
3	9.6	0.11	1.6	0.78	0.78	0.01	7.22	75	7.90	82
4	3.7	0.05	0.7	0.33	0.78	0.01	2.22	60	2.51	68
5	6.2	0.05	0.9	0.52	0.78	0.01	4.52	73	5.00	81
6	4.5	0.11	0.7	0.37	0.77	0.01	3.03	67	3.30	73
Total	38.6	0.49	5.5	2.83	4.65	0.06	28.45	74	30.85	80

The mass balances presented in Tables 20 to 22 were based on the following equations:

$$\text{Biodegraded} = \text{Influent} - \text{Effluent} - \text{Waste MLSS} \quad - \quad (1)$$

$$\text{N Denitrified} = \text{Inf. tot.N} - \text{Eff. tot.N} - \text{Waste MLSS tot.N} \quad - \quad (2)$$

$$\begin{aligned} \text{N Nitrified} &= \text{N Denitrified} + \text{Eff. nre.N} + \text{Waste MLSS} \\ &\quad \text{nre.N} - \text{Inf. nre.N} \end{aligned} \quad - \quad (3)$$

It should be noted that the normal assessment of plant performance is generally based on a mass balance between the influent and effluent only and the percentage removal would be greater than the percentage degraded. However, for an investigation of the process kinetics, the percentage degraded is the parameter to be considered. A comparison between the percentage degraded and percentage removed (i.e. influent - effluent) is given in the following Table 23.

Table 23 - Percentage Degraded Vs Removed

	BOD ₅	BOD _u	BOD ₃₀	COD	Nitrogen
% Degraded	82	82	87	71	74
% Removed	97	97	98	86	86

9. PLANT PROCESS OXYGEN REQUIREMENT

Based on the mass balances presented in the previous Section, the process oxygen requirements for the IEA Tank No. 1 can be obtained. The theoretical TOR can be calculated from the BOD_5 degraded and the nitrogen nitrified and denitrified. This TOR value can be compared to the values obtained from the BOD_{30} and COD mass balances.

9.1 Theoretical COR Results

The theoretical COR can be calculated from Eq. 1-2 (Section 2.1.4) which is:

$$COR = 1.46(BOD_5) - 1.42 f_{BVSS}(MLVSS/\theta_c) \quad - (1)$$

where BOD_5 = Influent BOD_5
 f_{BVSS} = MLVSS biodegradable fraction
 θ_c = sludge age

From Table 20 (Section 8.2), Influent $BOD_5 = 201.7 \text{ kg/d}$ and from Table 2 (Section 5.1), $f_{BVSS} = 0.38$. θ_c can be calculated as follows:

Waste sludge volume = 18 kl/d
MLSS concentration = 3490 mg/l (MLVSS = 2810 mg/l)
MLSS wasted in sludge stream = $18 \times 3.49 = 62.82 \text{ kg/d}$
Effluent flow = 865 kl/d
Effluent SS concentration = 20.5 mg/l
MLSS wasted in effluent = $0.865 \times 20.5 = 17.75 \text{ kg/d}$
Total MLSS wasted = $62.82 + 17.75 = 80.75 \text{ kg/d}$
Tank MLSS volume = $37 \times 12.5 \times 3.27 = 1512 \text{ kl}$
Tank MLSS = $1.512 \times 3490 = 5276.88 \text{ kg}$
 $\theta_c = 5276.88/80.75 = 65 \text{ days}$

The sludge age of 65 days obtained above is much greater than the figure of 30 days assumed in the example in Section 2.1.1 and the calculated value of 24 days in Section 2.1.3.2. There are two possible explanations for the higher value obtained. The first is that the raw sewage influent at this plant has a much lower non-volatile SS fraction than the typically reported value of 25%. This is supported by the test results presented in Table 24 below, which indicated that the SS of the influent sewage to the Forster plant are almost 100% volatile.

Table 24 - Forster Influent VSS/SS Ratio

Date	Cycle No.	VSS	SS	VSS/SS
		mg/l	mg/l	Ratio
9/10/84	1	340	360	0.94
"	2	204	204	1.00
"	3	256	256	1.00
10/10/84	4	176	176	1.00
"	5	180	180	1.00
"	6	260	260	1.00
31/7/84	-	185	190	0.97
14/11/84	24h composite	160	160	1.00
15/11/84	" "	130	130	1.00

The VSS/SS ratios of almost 1.0 shown in Table 24 could also be due to the effective removal of inorganic SS by the inlet works bar screen and grit removal tank and the failure of the automatic sampler to pick up any inorganic SS that passed through the inlet works.

The second possible explanation for the high sludge age is that the plant MLSS control is not operated on a sludge age or MLSS concentration basis but rather on a sludge blanket depth basis. If the settled sludge blanket starts to rise to 1m or less from the water

level during decant, the plant operator would set the sludge pump to pump continuously for several hours to bring the sludge blanket back to 1m or more below the water level. Therefore, the overall long term sludge age could be less than the 65 days based on a constant setting of 20 minutes sludge pumping per cycle. A better indication of the long term sludge age could be obtained from a 16-day period between 16/1/84 and 31/1/84 when no sludge was withdrawn from the No. 4 IEA Tank. During this period, the MLSS concentration increased from 2729 to 3652 mg/l; influent flow rate averaged 847 kl/d and effluent SS was about 11 mg/l. Thus, the sludge production can be estimated as follows:

$$\begin{aligned}\text{Average MLSS production} &= 1.512 (3652 - 2729)/16 + 0.847 \times 11 \\ &= 96.56 \text{ kg/d}\end{aligned}$$

Based on the 50 percentile/MLSS concentration of 3257 mg/l for 1984 (Table 12, Section 8.1), the overall sludge age is about 51 days. Average influent SS concentration during the 24-hour sampling period was 293 mg/l. If 25% of the influent SS was non-volatile, then the sludge age would be reduced to about 33 days which is in close agreement with the assumed value of 30 days.

Therefore, based on a f_{BVSS} value of 0.38, θ_c of 65 days, MLVSS of 2810 mg/l (4249 kg mass in aeration tank) and the influent BOD_5 given in Table 20 (Section 8.2) the theoretical COR for the 24-hour sampling period is given in Table 25 below. The calculated carbonaceous BOD_u degraded (Table 20) is also included in Table 25 for comparison with the theoretical COR.

Table 25 - Theoretical COR Vs BOD_u

Cycle No.	Influent BOD ₅ , kg	Theoretical COR, kg	Biodegraded BOD _u , kg	COR/BOD _u Ratio
1	19.3	22.3	15.8	1.41
2	41.4	54.5	39.8	1.37
3	68.2	93.7	117.4	0.80
4	23.4	28.3	58.9	0.48
5	33.0	42.3	25.6	1.65
6	16.4	18.0	21.8	0.83
Total	201.7	259.1	279.3	0.93

It can be seen from Table 25 that over the 24-hour period, the theoretical COR compares very favourably with the calculated carbonaceous BOD_u degraded - COR/BOD_u ratio being 0.93. However, the comparison for the individual cycles is unfavourable with COR/BOD_u ratio varying from 0.48 for cycle No. 4 to 1.65 for cycle No. 6.

The poor match between the COR and BOD_u for each cycle could be due to the oxygen requirement (consumption) lagging behind the substrate loading by several hours. It can, therefore, be concluded that the theoretical COR matches the BOD_u degraded when compared on a 24-hour steady-state basis but not on a cycle dynamic-state basis.

9.2 Theoretical TOR Results

The theoretical TOR can be obtained from the sum of the COR and NOR. The theoretical NOR can be calculated from Eq. 2-1 (Section 2.2.4), which is:

$$\text{NOR} = 4.6P_N(\text{tot.N}) - 2.9P_{\text{DN}} \cdot P_N(\text{tot.N}) \quad - (2)$$

where P_N = fraction of tot.N nitrified
 P_{DN} = fraction of nte.N denitrified
tot.N = influent tot.N

Based on the nitrogen mass balance given in Table 22 (Section 8.2), the theoretical NOR and TOR results for the 24-hour sampling period are given in Table 26 below. The BOD_{30} and COD degraded results (Table 21) are also included in Table 26 for comparisons with the theoretical TOR results. Since the BOD_{30} results were obtained from samples seeded with the plant's nitrified effluent, the results would represent the ultimate carbonaceous and nitrification oxygen demand of the samples. However, the BOD_{30} results would not account for the reduction in TOR due to denitrification. The COD results could also be representative of the TOR if all the substrate was biodegradable. Similarly, the COD results also do not account for denitrification.

Table 26 - Theoretical TOR Vs BOD_{30} and COD

Cycle No.	Theoretical TOR, kg	Degraded BOD_{30} , kg	TOR/ BOD_{30} Ratio	Degraded COD, kg	TOR/COD Ratio
1	29.9	32.7	0.91	27.4	1.09
2	69.7	66.7	1.04	95.6	0.73
3	109.1	144.8	0.75	115.9	0.94
4	33.5	68.2	0.49	29.9	1.12
5	52.3	44.9	1.16	47.0	1.11
6	24.3	34.9	0.70	28.2	0.86
Total	318.8	392.4	0.81	344	0.93

It can be seen from Table 26 that the correlation between TOR and COD was better than the correlation between TOR and BOD₃₀. The good correlation between TOR and COD is suprising and could possibly be due to the influent substrate being substantially biodegradable. Unfortunately, determination of the biodegradable fraction of the influent VSS could not be reliably obtained as was discussed in Section 5.2.

The correlation of TOR with BOD₃₀ and COD may be improved if the latter two values were adjusted for the oxygen recovered by the denitrification process. Comparisins of the TOR results with the adjusted BOD₃₀ and COD results are given in Table 27 below.

$$\text{Adjusted BOD}_{30} = \text{BOD}_{30} - 2.9 \times \text{nitrogen denitrified}$$

$$\text{Adjusted COD} = \text{COD} - 2.9 \times \text{nitrogen denitrified}$$

Table 27 - Theoretical TOR Vs BOD₃₀ and COD Adjusted
for Denitrification

Cycle No.	Theoretical TOR, kg	Adjusted BOD ₃₀ , kg	TOR/BOD ₃₀ Ratio	Adjusted COD, kg	TOR/COD Ratio
1	29.9	21.6	1.38	16.3	1.83
2	69.7	44.6	1.56	73.5	0.95
3	109.1	123.9	0.88	95.0	1.15
4	33.5	61.8	0.54	23.5	1.43
5	52.3	31.8	1.64	33.9	1.54
6	24.3	26.1	0.93	19.4	1.25
Total	318.8	309.9	1.03	261.5	1.22

It can be seen from Table 27, that the theoretical TOR result agrees very closely with the total BOD₃₀ result that was adjusted for oxygen recovery due to denitrification. The TOR/adjusted BOD₃₀ ratio was 1.03 compared to the TOR/unadjusted BOD₃₀ ratio of 0.81 given in Table 25. However, the correlation between the TOR and adjusted BOD₃₀ results for each cycle was again unsatisfactory. This could again be due to the time lag between actual degradation of the substrate and the substrate loading to the plant. The average TOR per cycle was about 53.1 kg and the peak cycle to average cycle TOR was 2.1 to 1 and the peak to minimum was about 4.5:1. The average denitrification adjusted BOD₃₀ per cycle was about 51.7 kg and the peak to average and peak to minimum cycle ratios were 2.4 to 1 and 5.7 respectively. Although there is an apparent time lag between substrate degradation and substrate loading, no diurnal TOR damping as discussed in Section 2.3 was apparent from the results.

Table 27 showed that the correlation of TOR with the denitrification adjusted COD results was not as good as that with the unadjusted COD. The reason for this is not apparent.

In general, Tables 26 and 27 showed that the best correlation was obtained between the theoretical TOR and denitrification adjusted BOD₃₀ results based on a 24-hour steady-state assumption. Based on the influent BOD₅ loading of 201.7 kg/d, the theoretical TOR was about 1.58 kg O₂/kg BOD₅ and the adjusted BOD₃₀ was about 1.54 kg O₂/kg BOD₅. These values are about 14 to 17% greater than the theoretical design value of 1.37 kg O₂/kg BOD₅ obtained in Section 2.3. The reasons for the higher values would be due to the following:

- i) The experimental influent carbonaceous BOD_u/BOD₅ ratio was higher than the theoretical ratio - 1.5:1 (Section 7.2.3) and 1.46:1 respectively.
- ii) The experimental tot.N to BOD₅ ratio of 0.19:1 was greater than the design assumption of 0.14:1 (10g to 70g per ep respectively).

- iii) The experimental θ_c was 65 days compared to the design assumption of 30 days.
- iv) The absence of non-volatile SS in the experimental influent compared to a design assumption of 25% non-volatile SS in the influent.

The TOR value based on the traditional synthesis and endogenous respiration model would depend on the values of the synthesis respiration coefficient (a) and endogenous respiration coefficient (b) adopted for Eq. 7 (Section 2.1.2). If the typical conventional activated sludge values of $a = 0.5 \text{ kg O}_2/\text{kg BOD}_5$ and $b = 0.1 \text{ kg O}_2/\text{kg MLVSS}$ as presented in Table 1 (Section 2.1.2) were adopted, Eq. 7 (Section 2.1.2) becomes:

$$\text{COR} = 0.5 (\text{kg BOD}_5 \text{ removed/degraded}) + 0.1(4249) \quad - \quad (3)$$

However, if the values presented by Fujimoto et al (Ref. 8, Table 1, Section 2.1.2) of $a = 0.75 \text{ kg O}_2/\text{kg BOD}_5$ and $b = 0.03 \text{ kg O}_2/\text{kg MLVSS}$ were adopted, Eq. 7 becomes:

$$\text{COR} = 0.75 (\text{kg BOD}_5 \text{ removed/degraded}) + 0.03(4249) \quad - \quad (4)$$

Typically, both Eqs. 3 and 4 are based on kg BOD₅ removed, but it is considered that kg BOD₅ degraded is the more correct value to use. TOR values based on both kg BOD₅ removed and degraded calculated from Eqs. 3 and 4 are presented below:

Table 28 - Traditional Synthesis/Endogenous Respiration TOR

	TOR from Eq. 3	TOR from Eq. 4
Based on BOD ₅ removed	582.2 kg O ₂	333.5 kg O ₂
Based on BOD ₅ degraded	567.8 kg O ₂	311.9 kg O ₂

As can be seen in Table 28, the traditionally calculated TOR values based on Eq. 4 compares favourably with the theoretical TOR and denitrified adjusted BOD₃₀ values but that based on Eq. 3 is much too high.

10. PLANT AERATION TEST RESULTS

The IEA tanks at Forster were installed with submerged jet aeration equipment as described in Section 4. Unsteady-state aeration tests were conducted in clean water in Tank No. 1 and in dirty water (MLSS) in Tank No. 3.

10.1 Clean Water Aeration Test

A clean water aeration test was conducted in IEA Tank No. 1 on 13/12/1983 with the air blower feeding air to the two jet clusters set at high speed (same operating speed as during the 24-hour sampling period discussed in Section 9). Based on factory tests on the air blower, the air flow rate was 392 l/s at an ambient air temperature of 20°C and 1 atmospheric pressure. The tank was filled with clean tap water to a depth of 3.35m. Four YSI DO probes and meters were used to measure the DO concentration during the tests. Three of the DO meters (Model 56) provide digital readout of the DO concentration to two decimal points and the other (Model 58) provides reading on a linear scale to one decimal point. All probes were installed with self-stirrers to ensure adequate liquid velocity across the probe membrane.

10.1.1 Test Procedures

Two DO probes were installed at mid-depth around each of the jet clusters. The four probes were calibrated by Winkler titration and installed in the tank a day before the tests. About 2 hours before the test, about 1.5 kg of cobalt chloride hexahydrate predissolved in a bucket of tap water was evenly distributed into the aeration tank while the aerators were operating. About 200 kg of sodium sulphite powder was then added to the aeration tank with the submersible pumps operating but not the air blower. When steady minimum DO readings close to zero were recorded on all four DO probes, the air blower was started to begin reaeration of the test water. DO readings were then recorded every 2 minutes for the first 40 minutes of reaeration and

then every 5 minutes until readings remained constant for at least 10 minutes. When the final constant DO readings were recorded, a sample of the test water was collected and its DO concentration obtained by Winkler titrations. The Winkler DO test result was then used to provide a calibration factor for the DO probe readings. The test water temperature, ambient air temperature and barometric pressure were also recorded at the start and end of the aeration test.

10.1.2 Test Results

Both the modified surface aeration model and the new submerged aeration model as developed in Section 3.1.3 were used to analyse the aeration test data. As discussed in Section 3.1.2, the modified surface aeration model adopted by Brown et al (Ref. 12) for the submerged aeration test is represented by the following equation (Section 3.1.2, Eq. 5).

$$C = C_{\infty}^* - (C_{\infty}^* - C_0) e^{-k't} \quad - (1)$$

where C = DO concentration
 C_{∞}^* = DO saturation concentration (assumed to be constant throughout the test)
 C_0 = DO at time zero (at start of test)
 k' = apparent overall volumetric oxygen transfer coefficient
 t = time

The new submerged aeration model as developed by the present author is represented by the following equations (Section 3.1.3.2, Eq. 23):

$$C = C_{\infty}^* - (C_{\infty}^* - C_0)e^{-kt} - k(C_{\infty}^* - C_0)t e^{-kt} \quad - (2)$$

where C_{∞}^* = Final DO saturation concentration at the end of the test
 C_0^* = Initial DO saturation concentration at the start of the test

As indicated by the above equation, the DO saturation concentration in the new submerged aeration model is not constant but varies during the test according to the following equation (Section 3.1.3.1, Eq. 7):

$$C^* = C_{\infty}^* - (C_{\infty}^* - C_0^*) e^{-kt} \quad - (3)$$

The non-linear least squares estimation technique as discussed in Section 3.1.4.1 was used to determine the oxygen transfer parameters in Eqs. 1 and 2. The four DO probe readings together with the DO values predicted by the modified surface model (Eq. 1) and the new submerged aeration model (Eq. 2) are given in the tables in Appendix C1 to C4. Also plots of the best fit curves obtained by the non-linear least squares estimation technique for both Eqs. 1 and 2 are given in Appendix D1 to D4 for probes 1 to 4 respectively.

It can be seen from the tables in Appendix C1 to C4 that both Eqs. 1 and 2 fitted the test data very well. Also, except for Probe No. 1, the plots of the best fit lines for Eqs. 1 and 2 are almost identical. However, as shown in Table 1 below, Eq. 2 for the new submerged model fitted the test data of all four probes slightly better than Eq. 1.

Table 1 - Clean Water Test Results - Error Estimates

	Eq. 1: Modified Surface Model		Eq. 2: New Submerged Model	
	ΣR^2	$(\Sigma R^2/n)^{1/2}$	ΣR^2	$(\Sigma R^2/n)^{1/2}$
Probe No. 1	0.2324	0.08 mg/l	0.0885	0.05 mg/l
Probe No. 2	0.5147	0.12 mg/l	0.4831	0.12 mg/l
Probe No. 3	0.0973	0.05 mg/l	0.0936	0.05 mg/l
Probe No. 4	0.0645	0.05 mg/l	0.0640	0.05 mg/l
Average	0.2272	0.075 mg/l	0.1823	0.068 mg/l

The parameter estimates obtained for Eqs. 1, 2 and 3 are given in Table 2 below:

Table 2 - Clean Water Test Results - Parameter Estimates

	Modified Surface Model			New Submerged Model			
	C_O mg/l	C_{∞}^* mg/l	k' h^{-1}	C_O mg/l	C_O^* mg/l	C_{∞}^* mg/l	k h^{-1}
Probe 1	0.24	9.34	2.602	0.08	7.28	9.28	3.325
Probe 2	-0.25	9.25	2.735	-0.23	8.08	9.24	3.085
Probe 3	0.03	8.95	3.124	0.06	7.42	8.93	3.695
Probe 4	0.15	9.15	3.165	0.16	8.11	9.14	3.551

Table 2 showed that both k' and k for Probes 1 and 2 are much lower than that for Probes 3 and 4. This is most probably due to an uneven distribution of air flow to the two jet clusters, with more air flowing to the cluster around which Probes 3 and 4 were installed. Also the DO concentration values in Table 2 need to be corrected by the Probe calibration factor. A DO of 9.36 mg/l was obtained by the Winkler test at the end of the test at $t = 135$ mins. The corrected DO values are shown in Tables 3 and 4 below.

Table 3 - Corrected DO Values for Modified Surface Model

	Probe DO at 135 mins. mg/l	Calibration Factor of Probe	Corrected C_O mg/l	Corrected C_{∞}^* mg/l
Probe 1	9.25	1.0119	0.24	9.45
Probe 2	9.16	1.0218	-0.26	9.45
Probe 3	8.93	1.0482	0.03	9.38
Probe 4	9.10	1.0286	0.15	9.41

Table 4 - Corrected DO Values for New Submerged Model

	Probe Calibration Factor	Corrected C_o mg/l	Corrected C_{co}^* mg/l	Corrected C_o^* mg/l	Corrected C_{∞}^* mg/l
Probe 1	1.0119	0.08	7.35	7.37	9.39
Probe 2	1.0218	-0.24	8.29	8.26	9.44
Probe 3	1.0482	0.06	7.77	7.78	9.36
Probe 4	1.0286	0.16	8.32	8.34	7.40

NOTE: C_{co}^* = DO saturation at $C=0$

where as C_o^* = DO saturation at $t=0$

10.1.3 SOTR Calculations

Modified Surface Model SOTR Calculations

Since an equal number of probes was located in each half of the aeration tank (2 in each half) it is possible to use the average k' and C_{∞}^* values of the four probes to calculate SOTR as follows:

$$\begin{aligned}
 k_T' &= (2.602 + 2.735 + 3.124 + 3.165)/4 \\
 &= 2.9065 \text{ h}^{-1}
 \end{aligned}$$

The test water temperature was 25.3°C. Thus from Eq. 24 (Section 3.1.4.1):

$$\begin{aligned}
 k_{20}' &= 2.9065 (1.024)^{-5.3} \\
 &= 2.5632 \text{ h}^{-1}
 \end{aligned}$$

From Eq. 25 (Section 3.1.4.1), the effective saturation depth is given as:

$$de = \frac{1}{\gamma} \left\{ \frac{C_{\infty T}^*}{H_T Y_d} - P_b + P_{VT} \right\}$$

where γ = water density = 9.8 kPa/m
 $C_{\infty T}^*$ = average C_{∞}^* of the 4 probes = 9.42 mg/l
 H_T = Henry's Law Const. = 0.396 mg/l. kPa
 Y_d = air feed oxygen mole fraction = 0.21
 P_b = test atmospheric pressure = 101.6 kPa
and P_{VT} = test saturated water vapour pressure = 3.16 kPa

$$\therefore de = \frac{1}{9.8} \left\{ \frac{9.42}{0.396 \times 0.21} - 101.6 + 3.16 \right\}$$

$$= 1.514m$$

The depth of water in the tank was 3.35m and the water mid-depth was thus 1.675m. Hence, the effective saturation depth was about 10% less than the water mid-depth.

The saturation concentration at standard conditions (20°C and 1 atmospheric pressure) is given by Eq. 26 (Section 3.1.4.1) as follows:

$$C_{\infty 20}^* = C_{\infty T}^* \left\{ \frac{C_{S20}^*}{C_{ST}^*} \right\} \left\{ \frac{P_s + \gamma de - P_{V20}}{P_b + \gamma de - P_{V20}} \right\}$$

$$= \frac{9.42 \times 9.07}{8.2} \left\{ \frac{101.3 + 9.8 \times 1.514 - 2.34}{101.6 + 9.8 \times 1.514 - 2.34} \right\}$$

$$= 10.39 \text{ mg/l}$$

$$\begin{aligned}\text{Test water volume} &= 37 \times 12.5 \times 3.35 \\ &= 1549 \text{ m}^3\end{aligned}$$

$$\begin{aligned}\therefore \text{SOTR} &= k_{20}' \times C_{\infty 20}^* \times 1.549 \\ &= 2.5632 \times 10.39 \times 1.549 \\ &= \underline{41.25 \text{ kg O}_2/\text{h}}\end{aligned}$$

New Submerged Model SOTR Calculations

From Table 2, Section 10.1.2, the average k_T for the four probes is 3.414 h^{-1} .

$$\begin{aligned}\therefore k_{20} &= 3.414 (1.024)^{-5.3} \\ &= 3.0107 \text{ h}^{-1}\end{aligned}$$

From Table 4, Section 13.1.2, the average test saturation concentration at zero DO, C_{COT}^* is 7.93 mg/l . From Eq. 36 (Section 4.4.2), the saturation concentration at zero DO and standard conditions is thus:

$$\begin{aligned}C_{\text{C020}}^* &= C_{\text{COT}}^* \frac{(C_{\text{S20}}^*)}{(C_{\text{ST}}^*)} \left\{ \frac{P_s + 0.5 d\gamma}{P_b + 0.5 d\gamma} - \frac{P_{\text{V20}}}{P_{\text{V20}}} \right\} \\ &= \frac{7.93 \times 9.07}{8.2} \left\{ \frac{101.3 + 0.5 \times 3.35 \times 9.8 - 2.34}{101.6 + 0.5 \times 3.35 \times 9.8 - 2.34} \right\} \\ &= 8.75 \text{ mg/l}\end{aligned}$$

$$\begin{aligned}\therefore \text{SOTR} &= 3.0107 \times 8.75 \times 1.549 \\ &= \underline{40.81 \text{ kg O}_2/\text{h}}\end{aligned}$$

It can be seen that the SOTR values obtained by the two models differ by only about 1%. As discussed in Section 3.1.5, differences in the SOTR values would be small for submerged aeration systems with low oxygen stripping or transfer efficiency as the change in DO saturation concentration during the test is also small. Hence, departure of the modified surface model, which assumes a constant saturation value, from the new submerged model is also smaller. The standardised oxygen transfer efficiency of this aeration system can be calculated according to Eq. 28, Section 3.1.4.1, as follows:

$$\begin{aligned} \text{SOTE} &= \frac{40.81 \times 100}{0.392 \times 3600 \times 1.205 \times 0.232} \\ &= 10.3\% \end{aligned}$$

Therefore, it could be concluded that for submerged aeration systems with low to moderate SOTE, the modified surface model would be satisfactory for estimating SOTR. This is reflected by the good fit of both models to the test data. Table 1, Section 10.1.2, showed that although the new submerged model fitted the data slightly better than the modified surface model, the estimated errors for both models are small - being 0.068 mg/l for the former and 0.075 mg/l for the latter. However, for aeration systems with higher SOTE, the departure of the modified surface model increases resulting also in greater difference in the SOTR and estimated error of the two models. This can be illustrated by the results of this aeration test and the two examples discussed in Section 3.1.4 as shown in the following Table:

Table 5 - Differences in SOTR and Estimated Error

	SOTE = 10%	SOTE = 12%	SOTE = 69%
Modified Surface SOTR	41.25	52	204
New Submerged SOTR	40.81	50	180
% Difference in SOTR	1%	4%	13%

10.2 Dirty Water (MLSS) Aeration Test

An aeration test in MLSS was conducted in IEA Tank No. 3 on 9/10/84 with the air blower operating at the same high speed as during the clean water aeration test and the 24-hour sampling period. Tank No. 3 is identical to Tank No. 1 and it is assumed that the aeration characteristics in both tanks are the same. Tank No. 3 had been taken out of operation for 6 weeks. Although it was not fed with any sewage during the 6 weeks or during the test, the MLSS still had a substantial oxygen uptake rate as measured during the test. Similarly to the clean water test, 4 YSI Model 56 DO probes were installed in the tank - two probes in each half or aeration cell. All the probes were fitted with self-stirrers and the DO meters provided digital DO reading to two decimal points.

10.2.1 Test Procedures

The four DO probes were calibrated in clean water against the Winkler test before being installed in the aeration tank. The submersible pumps on top of the two jet clusters were then switched on to mix the tank contents and also to allow any residual DO in the tank to be biologically consumed. When stable minimum DO readings close to zero were obtained, the air blower was switched on to start aerating the MLSS. DO readings were taken every half minute for the first 5 minutes, then every minute for the next 15 minutes, followed by every two minutes until 40 minutes, every 5 minutes until 80 minutes and finally every 10 minutes until the test was stopped at 180 minutes when the rise in DO was less than 0.1 mg/l over a 20 minute interval. The MLSS temperature was also measured during the test.

After 12 minutes of aeration, a MLSS sample was collected for oxygen uptake rate measurement. Two further samples were collected, one at 65 minutes and one at about 145 minutes of the test. The oxygen uptake rate (OUR) tests were conducted in standard BOD bottles into which were inserted a YSI DO probe designed to fit snugly into the neck of the BOD bottle. The probe was fitted with a self-stirrer

to mix the MLSS and also to ensure sufficient liquid velocity to rapidly replace the oxygen deficient liquid film on the probe's oxygen permeable membrane. After the OUR test, the MLSS concentration of the sample was measured. The MLSS sample temperature was also measured at the start and end of the OUR test.

A portion of the first MLSS sample collected for OUR test was also used to determine its DO saturation concentration. The MLSS sample was allowed to settle in a 5 litre plastic bucket and the clear supernatant obtained was decanted into a beaker. The supernatant was then aerated using a fish aquarium air pump and diffuser stone. The rise in DO concentration was noted until a constant reading over 20 minutes was obtained. The supernatant sample is then taken to be saturated and the DO saturation concentration measured by the Winkler test. The temperature of the supernatant sample was also measured just prior to the Winkler test.

10.2.2 Test Results

OUR Test Results

The 3 MLSS samples collected during the aeration test were aerated in a beaker using a fish aquarium type air pump and diffuser stones. When the sample's DO rose to about 7 to 9 mg/l, the samples were transferred to standard BOD glass bottles where the drop in DO concentrations with time were monitored by a BOD bottle DO probe. The results of the 3 tests are shown in Appendix E1 to E3 where the fall in DO concentrations with time are plotted. The first sample (Test No. 1) was collected near the beginning of the aeration test, the second sample near the middle and the third sample near the end. The plots shown in Appendix D1 to D3 resemble typical OUR plots where (except for a few initial data points) the plot is a straight line which then slopes away with a decreasing gradient due to either or a combination of limiting DO and substrate concentrations. The OUR is obtained from the slope of the straight line section of the plot. The specific OUR is then obtained by dividing the OUR by the sample MLSS concentration. The OUR results are given in Table 6 below.

$$\beta = C'_S / C^* = 7.8/8.5 = 0.92$$

The β value of 0.92 obtained compares favourably with the value of 0.9 adopted in Section 3.3 and with commonly quoted literature values.

$K'_L a$ and α Values

Table 5 in Section 10.1.3 showed that the SOTE of the aeration system is relatively moderate (about 10%) and thus the modified surface aeration model would be satisfactory for estimating SOTR - the difference between the modified and new submerged model SOTR was only about 1%. OTE for the MLSS aeration test would be less than that for the clean water test since β was less than 1 and α would also be expected to be less than 1. Hence, it is expected that there will be no significance difference between the OTR' calculated by both models. Calculation of the OTR' under process conditions would be simpler using the modified surface model as C'_S does not vary with the MLSS DO concentration. Therefore, only the modified surface aeration model was adopted to analyse the MLSS aeration test data.

The OUR test results (Table 6 above) show that the OUR of the MLSS was relatively constant throughout the MLSS aeration test. Thus, the modified surface model for the MLSS aeration test, represented by Eq. 8 in Section 3.2, can be used to analyse the aeration test data. The equation for the model is:

$$C' = C_S^{*'} - (C_S^{*'} - C'_0) e^{-k't} \quad - (4)$$

where C' = MLSS DO conc.
 $C_S^{*'}$ = Apparent MLSS DO saturation conc.
 C'_0 = MLSS DO conc. at $t=0$
 k' = Apparent $K'_L a'$ in MLSS

The non-linear least squares estimation technique as discussed in Section 3.1.4.1 was used to obtain the parameters in Eq. 4 above. The test DO probe results and predicted DO values and residuals obtained from Eq. 4 are given in Appendices F1 to F4. Plots of the predicted curves (best fit curves) are also given in Appendices G1 to G4.

Appendices F1 to F4 and G1 to G4 show that the MLSS aeration test data were more scattered than the clean water aeration test data. This would be due to the effect of the MLSS oxygen uptake which magnifies the effect of departures from ideal complete mixing. As a volume of MLSS leaves the direct aeration zone its DO will drop due to oxygen uptake of the MLSS. Each probe had some poor data points with estimated error of 0.3 mg/l or more. The estimated errors and parameters for Eq. 4 of the modified surface aeration model are given in Table 7 below:

Table 7 - MLSS Aeration Test - Estimated Error and Parameters

	ΣR^2	$(\Sigma R^2/n)^{1/2}$ mg/l	C'_O mg/l	C'_S mg/l	$K_L a'$ mg/l
Probe No. 1	1.255	0.15	0.65	4.27	1.75
Probe No. 2	0.887	0.12	0.24	4.15	1.76
Probe No. 3	1.186	0.15	0.82	4.62	2.10
Probe No. 4	1.534	0.17	0.73	4.87	2.04
Average	1.2155	0.15	0.61	4.48	1.91

Table 7 above shows that the average estimated error for the 4 probes was 0.15 mg/l which is twice the error of 0.075 for the clean water test (Table 1). Table 7 also shows that the $K_L a'$ for probes 3 and 4 are about 18% greater than that for probes 1 and 2 - similarly to the results obtained in the clean water test (Table 2). This again reflects the uneven airflow distribution between the two jet clusters.

The average K_{La}' for the 4 probes was 1.91 h^{-1} . The MLSS temperature during the test was 19.1°C . K_{La}' at 20°C is obtained as follows:

$$\begin{aligned}(K_{La}')_{20} &= (K_{La}')_T \times 1.024^{20-T} \\ &= 1.91 \times 1.024^{0.9} \\ &= 1.95 \text{ h}^{-1}\end{aligned}$$

From Section 10.1.3, K_{La} in clean water at 20°C as obtained by the modified surface aeration model was 2.56 h^{-1} . Thus, the alpha factor, α , is as follows:

$$\alpha = 1.95/2.56 = \underline{0.76}$$

The α factor of 0.76 is less than the value of 0.85 adopted in Section 6 for converting the process oxygen requirement to SOTR requirement. However, it compares favourably with α values quoted in the review paper by Stenstrom and Gilbert (Ref. 10). Gilbert obtained α from 0.7 to 0.8 in a 3m deep tank installed with a coarse bubble sparger diffuser with airflow rate varying from about 439 to 1043 l/s. Airflow rate for the present system is about 392 l/s. The α value of 0.85 adopted by the PWD was based on surface aerators.

10.2.3 OTR' Calculations

The field or process oxygen transfer rate, OTR' , can be calculated from Eq. 12 given in Section 3.2 which is:

$$OTR' = K_{La}' (C_s' - C').V.10^{-3} \quad - (5)$$

where OTR' = field or process condition OTR, kg/h
 K_{La}' = K_{La} in MLSS, h^{-1}
 C_s' = MLSS DO saturation at field condition, mg/l

Table 6 - Aeration Test OUR Results

	Sample Temperature		MLSS conc. mg/l	OUR mg/l.h	Specific OUR mg/g.h
	Start, °C	End, °C			
Sample No. 1	20.5	21.1	1908	16.93	8.87
Sample No. 2	19.9	20.9	2197	17.15	7.81
Sample No. 3	19.6	20.5	2051	16.56	8.07
Average	20.0	20.8		16.88	8.25

Table 6 above shows that the OUR and specific OUR are relatively constant throughout the aeration test. The specific OUR of the third sample collected near the end of the aeration test is only about 10% less than that of the first sample collected near the beginning of the aeration test. It is considered that significant error would not be introduced by assuming a constant OUR in the interpretation of the MLSS aeration test results.

The average specific OUR of 8.25 mg/g MLSS.h appears to be high since the aeration tank was taken out of operation for about 6 weeks. It is considered that this is due to the death and lysis of microbial cells which releases substrate back to the system.

Wastewater C_s^i and β Values

A portion of the MLSS sample collected for the first OUR test was settled in a plastic bucket and the supernatant was transferred to a beaker where it was aerated until saturation was reached. The C_s^i value of the supernatant was then determined by the Winkler (azide modification) test. The C_s^i value obtained was 7.80 mg/l at a temperature (of sample) of 23.4°C. From Table 421:1 of "Standard Methods" (Ref. 9), the clean water saturation value at 23.4°C is 8.50 mg/l. The value for β was thus:

$$\begin{aligned} C' &= \text{process MLSS DO conc., mg/L} \\ V &= \text{Aeration tank volume, m}^3 \end{aligned}$$

C'_S can either be measured or determined from the aeration test apparent DO saturation C_S^{*} .

Measured C'_S

In Section 10.2.2, a sample of settled MLSS was aerated in a 5L plastic bucket and the measured saturation concentration was 7.8 mg/L at a temperature of 23.4°C. This value represents the "surface" wastewater saturation concentration as the sample was aerated in a shallow bucket. The actual C'_S value for the submerged aeration system could be estimated either using the "mid-depth" or the "effective saturation depth" factor to correct the surface C'_S value. Adopting the latter, the effective saturation depth was determined in the clean water aeration test in Section 10.1.3 to be 1.154m. The actual C'_S is thus estimated as follows:

$$\begin{aligned} C'_S &= \frac{7.8 (101.3 + 9.8 \times 1.514 - 2.88)}{(101.3 - 2.88)} \\ &= 8.98 \text{ mg/L} \end{aligned}$$

$$(C'_S)_{20} = (C'_S)_T \frac{(C_{S20}^*)}{(C_{ST}^*)}$$

$$\text{where } (C'_S)_{20} = C'_S \text{ at } 20^\circ\text{C}$$

$$(C'_S)_T = C'_S \text{ at } 23.4^\circ\text{C} = 8.94 \text{ mg/L}$$

$$\begin{aligned} (C_{S20}^*) &= \text{standard clean water surface saturation at } 20^\circ\text{C} \\ &= 9.07 \text{ mg/L} \end{aligned}$$

$$\begin{aligned} (C_{ST}^*) &= \text{standard clean water surface saturation at } 23.4^\circ\text{C} \\ &= 8.5 \text{ mg/L} \end{aligned}$$

$$\begin{aligned}\therefore (C'_S)_{20} &= \frac{8.98 \times 9.07}{8.5} \\ &= 9.58 \text{ mg/l}\end{aligned}$$

Alternatively, $(C'_S)_{20}$ could be determined from the β factor (0.92) and the clean water C_{∞}^* value (10.39 mg/l) determined in the clean water aeration test.

$$\begin{aligned}(C'_S)_{20} &= 0.92 \times 10.39 \\ &= 9.56 \text{ mg/l}\end{aligned}$$

Estimation of C'_S from C_S^*

C'_S can be estimated from C_S^* according to Eq. 4 in Section 3.2 as follows:

$$C'_S = C_S^* + (r/K_L a') \quad - (6)$$

C_S^* was estimated in the MLSS aeration test to be about 4.48 mg/l at a temperature of 19.1°C. The OUR, r , was estimated to be about 16.88 mg/l.h at an average temperature of 20.4°C. ' r ' thus has to be corrected back to a temperature of 19.1°C before it can be applied in Eq. 6. The temperature correction factor, θ , for the OUR test was not determined. Thus, the θ factor for the BOD5 test was adopted as a substitute. Metcalf and Eddy (Ref. 14) reported that θ varies from 1.056 in the temperature range between 20 and 30°C, to 1.135 in the temperature range between 4 and 20°C. Adopting the latter value to correct ' r ' from 20.4°C to 19.1°C gives -

$$\begin{aligned}(r)_{19.1} &= 16.88 \times 1.135^{-1.3} \\ &= 14.34 \text{ mg/l.h}\end{aligned}$$

$$\therefore (C'_S)_{19.1} = 4.48 + (14.34/1.91)$$

$$= 11.99 \text{ mg/l}$$

$$\therefore (C'_S)_{20} = 11.99 \times \frac{9.07}{9.24}$$

$$= 11.77 \text{ mg/l}$$

It can be seen that the above value of 11.77 mg/l is about 23% higher than the value of 9.58 mg/l estimated from the measured surface C'_S value. It is considered that this value is too high as it is greater than the clean water aeration test C^*_{20} value of 10.39 mg/l thus yielding a β value greater than 1. It is considered that the 'r' value determined in the BOD bottle could be much higher than the actual 'r' value in the aeration tank thus yielding an erroneous high C'_S value from Eq. 6. The more vigorous mixing provided in the BOD bottle would provide a greater contact and also greater rate of oxygen transfer into the activated sludge floc, thus resulting in a greater OUR in the BOD bottle than in the less well mixed aeration tank MLSS. Adopting the formerly determined $(C'_S)_{20}$ value of 9.58 mg/l, the actual OUR of the MLSS in the aeration tank can be estimated as follows:

$$\begin{aligned} r &= K_L a' (C'_S - C^{*'}_S) & - (7) \\ &= 1.91 (9.58 \times 9.24/9.07 - 4.48) \\ &= 10.08 \text{ mg/l} \end{aligned}$$

Therefore, the actual OUR in the aeration tank could be 42% less than the OUR determined in the BOD bottle test. Sampling error could also affect the values between the BOD bottle test OUR and aeration tank OUR.

Adopting the $(C'_S)_{20}$ value of 9.58, OTR' at 20°C with C' at 1 mg/l is as follows:

$$\begin{aligned}(\text{OTR}')_{20} &= 1.95 (9.58 - 1.0) 1.549 \\ &= \underline{25.9 \text{ kg O}_2/\text{h}}\end{aligned}$$

11. PLANT OXYGEN BALANCE

In Section 9, the process oxygen requirement of the No. 1 IEA tank was determined over a 24-hour period. These results can be compared with the oxygen transferred to the system to satisfy or meet the process oxygen requirements. The influent and effluent DO concentrations were not measured. However, it is considered that this will not introduce significant error to the oxygen balance around the IEA tank. Effluent is decanted after about 60 minutes of settlement during which the tank MLSS DO was generally reduced to less than 0.2 mg/l. DO in the influent would also be generally low as the sewage is septic on arrival at the plant. The influent flow during the 24-hour survey period was about 881 kl. Hence, 1 mg/l DO concentration in the influent or effluent would only represent 0.881 kg of oxygen which is negligible compared to the OTR' of 25.9 kg/h as determined in Section 13.

In order to calculate the oxygen transfer according to Eq. 7 in Section 10.2.3, the DO concentration in the MLSS must be known. The DO concentration in No. 1 IEA tanks was monitored over a period of about 22 hours between the 9th and 10th of October, 1984. This was about 20 days before the 24-hour sampling period when the process oxygen requirement of the tank was determined. It is considered that the tank DO concentration would be substantially the same for both periods as there were no substantial differences in the plant's loading, operations or performance during both periods. Thus, it is considered DO readings from the earlier period can be adopted to determine the oxygen transfer for the latter period without introducing significant error. The estimated average MLSS DO concentration as well as the maximum DO reached during each aeration phase for the 24-hour sampling period of 29-30/10/84 are given in Table 1 below.

Table 1 - Aeration Phase DO Concentration Data

Cycle No.	Time	Maximum DO mg/l	Average DO mg/l
1	11 am to 3 pm	0.7	0.2
2	3 pm to 7 pm	2.6	0.8
3	7 pm to 11 pm	3.3	0.7
4	11 pm to 3 am	5.7	3.3
5	3 am to 7 am	6.0	4.3
6	7 am to 11 am	1.4	0.2

The chart recording of the MLSS DO concentration in IEA Tank No.1 is given in Appendix H. It can be seen from the chart that the rise in DO during the aeration phase is non-linear. The average MLSS DO concentration during each of the cycle aeration phases given in Table 1 above was determined by obtaining the average DO over half hourly intervals and averaging these values over the 2 1/2 hour aeration phase.

Table 1 above together with the DO chart recording show that the MLSS DO concentration steadily increased from a low average of about 0.2 mg/l between 7 am to 3 pm to a high average MLSS DO of about 4.3 mg/l between 3 am to 7 am when most of the MLSS oxygen demand has been met.

This fact together with the excellent effluent quality and high percentage substrate removal indicated that the plant has adequate aeration capacity to meet the process oxygen requirements for the 24 h period.

A typical 4-hour operating cycle would provide 2 1/2 hour aeration, 1 hour settlement and 1/2 hour decant. However, the actual cycle adopted could be slightly different as the cycle would be fine-tuned to suit each plant. The actual cycle operating during the 24-hour sampling and DO monitoring periods for the No. 1 IEA plant is shown in Table 2 below.

Table 2 - 4-Hour Operating Cycle

Step	Functions	Remarks
1	1 min. only air blower on	Coarse bubble aeration
2	80 mins. blower + pump on	Full jet aeration
3	5 mins. only blower on	Coarse bubble aeration
4	40 mins. blower + pump on	Jet aeration
5	17 mins. only blower on	Coarse bubble aeration
6	40 mins. blower + pump off	Settlement phase
7	53 mins. trough lowering	Decant starts when trough reaches water level
8	2 mins. trough sit at BWL	
9	2 mins. trough raise	

Table 2 above shows that in steps 1, 3 and 5 only the air blower is in operation and the submersible pumps are not operating to provide the motive force to create the jets of diffused air bubbles through the nozzles. Hence, during these steps of the cycle, the aerators behave as a coarse air bubble aerator. These steps are provided to clear the jet nozzles of rags or other materials that would block them up. Thus, the 4-hour cycle provides 2 hours of jet aeration plus 23 minutes of coarse air bubble aeration. Reported typical oxygen transfer efficiency for coarse bubble aerators ranges from 4 to 8% depending on depth (Ref. 13). Adopting a value of about 5% appears reasonable for estimating the OTR' - i.e. coarse bubble aeration is assumed to be half as efficient as jet aeration (10.3% oxygen transfer efficiency determined in Section 10.1.3).

The average MLSS DO concentrations given in Table 1 were adopted to calculate the oxygen transferred to the MLSS during the 6 cycles of the 24-hour sampling period. The OTR' were calculated at a MLSS temperature of 20°C for convenience (measured MLSS temperature = 19.1°C). It is considered that temperature would not significantly affect the OTR' values due to its opposite and hence compensating

effect on C_s' and $K_L'a$ (C_s' decreases with rising temperature whereas $K_L'a$ increases with rising temperature). Calculation for Cycle No. 1 is given below as an example.

OTR' Due to Jet Aeration

$$\begin{aligned} (OTR')_{20} &= 1.95 (9.58 - 0.2) 1.549 \\ &= 28.33 \text{ kg/h} \end{aligned}$$

Aeration provided over 2 hours, thus oxygen transferred
= 2×28.3
= 56.7 kg

OTR' Due to Coarse Bubble Aeration

$$\begin{aligned} (OTR')_{20} &= 1/2 \times \text{jet aeration } (OTR')_{20} \\ &= 14.17 \text{ kg/h} \end{aligned}$$

Aeration provided over 23 minutes, thus oxygen transferred
= $14.17 \times 23/60$
= 5.4 kg

. . . Oxygen transferred during Cycle No. 1

$$\begin{aligned} &= 56.7 + 5.4 \\ &= 62.1 \text{ kg} \end{aligned}$$

The oxygen transferred for each of the 6 cycles is given in Table 3 below.

Table 3 - Oxygen Transferred Under Process Conditions

Cycle No.	1	2	3	4	5	6
kg O ₂ transferred	62.1	58.1	58.8	41.6	35.0	62.1

Table 3 above showed that oxygen transferred over the 24-hour period was 317.7 kg. Since the MLSS DO concentration drops to about 0.2 mg/l or less during the settlement and decant phase of each cycle, it can be taken that all the oxygen transferred into the MLSS was utilised for oxidation of the sewage substrate. Table 3 shows that the average oxygen transferred and hence consumed per cycle was 53 kg. This gives a peak to average cycle oxygen utilisation ratio of 1.17 which compares reasonably with the theoretical ratio of 1.25 diurnal peak to average hourly TOR ratio adopted in Section 2.3. These data thus showed considerable damping of the diurnal peak to average TOR as discussed in Section 2.3. Also the OTR' or oxygen utilised value of 317.7 kg compares very favourably with the theoretical TOR and measured TOR (BOD₃₀ adjusted for nitrification and denitrification) values given in Table 26 of Section 9.2. Comparisons of the TOR values determined by the various methods that were discussed in this report are summarised in Table 5 below.

Table 5 - Summary of Process TOR Values

Method	TOR kg O ₂ /day	TOR/BOD ₅ Ratio	SOTR/BOD ₅ Ratio
1. Measured OTR'	317.7	1.58 (1.85)	2.70 (3.15)
2. Theoretical	318.8	1.58 (3.32)	2.70 (5.65)
3. Measured BOD ₃₀	309.9	1.54 (3.7)	2.63 (6.32)
4. Measured COD	344.0	1.71 (3.46)	2.92 (5.90)
5. Traditional Eq. A1	333.5	1.65 (3.3)	2.82 (5.64)
6. Traditional Eq. A2	311.9	1.55 (3.1)	2.65 (5.30)
7. Traditional Eq. B1	582.2	2.89 (5.78)	4.94 (9.88)
8. Traditional Eq. B2	567.8	2.82 (5.64)	4.82 (9.64)
9. Rule of Thumb	276.3	1.37 (1.7)	2.06 (2.55)
10. FWD Design	322.7	1.6 (1.8)	2.4 (2.7)

NOTE:

1. Method 1 is based on the following mass balance:
$$\text{OTR}' + \text{Influent } O_2 = O_2 \text{ utilised} + \text{Effluent } O_2 + \text{MLSS } O_2.$$

As discussed in Section 11, Influent and Effluent DO are negligible compared to OTR'. MLSS DO is also considered to be negligible due to the low waste MLSS flow rate. Hence, the oxygen mass balance equation can be reduced to $\text{OTR}' = O_2 \text{ utilised} = \text{TOR}.$
2. Method 2 is based on the theoretical TOR given in Tables 25 and 26 of Section 9.2. TOR is derived from COR calculated from Eq. 1-2 (Section 2.1.4) with measured f_{BVSS} of 0.38 and θ_c of 64 days plus NOR calculated from Eq. 2-1 (Section 2.2.4).
3. Method 3 is based on measured total BOD_{30} degraded but theoretically adjusted for oxygen recovered through the denitrification process. Results are summarised in Table 26, Section 9.2
4. Method 4 is based on measured COD with no adjustment for denitrification. Results are summarised in Table 25, Section 9.2.
5. Method 5 is based on the traditional synthesis and endogenous respiration oxygen requirements as represented by Eq. 7, Section 2.1.2, with the coefficients $a = 0.75$ and $b = 0.03$ as given in Table 1, Section 2.1.2. The Eq. is based on BOD_5 removed (influent BOD_5 minus effluent BOD_5).
6. Method 6 is as in Method 5 but the equation is based on BOD_5 degraded rather than removed (i.e. BOD_5 of waste MLSS stream included).

7. Method 7 is as in Method 5 but using typical "conventional" activated sludge process coefficients of $a = 0.5$ and $b = 0.1$ as given in Table 1, Section 2.1.2.
8. Method 8 is as in Method 7 but the equation is based on BOD_5 degraded rather than removed.
9. Method 9 is based on the rule of thumb design figures as calculated theoretically from the influent ultimate carbonaceous BOD and NOR for the typical design example of a 4000 IEA plant are discussed in Sections 2.3 and 2.3.1.
10. Method 10 is based on typical PWD design criteria for an IEA plant. $SOTR/BOD_5$ ratios of 2.4 and 2.7 are adopted to specify the aerator's normal and maximum SOTR respectively. Allowance for diurnal peak cycle oxygen requirement (TOR) is not specified but lumped into a general safety factor of 1.125 used in specifying the maximum SOTR.
11. TOR/BOD_5 and $SOTR/BOD_5$ ratios given in brackets are based on the diurnal peak cycle TOR and SOTR (except for Method 10 as stated above).

It is considered that Method 1 is a very reliable method in measuring the process oxygen requirement (i.e. TOR) as it is based on an oxygen mass balance around the IEA tank. It can be seen in Table 5 that the theoretically derived TOR (Method 2) is almost identical to the TOR value obtained by Method 1. Also the theoretical TOR agrees very closely with the TOR based on the measured ultimate BOD (BOD_{30}) mass balance obtained by Method 3. Therefore, it can be concluded that the theoretical method is reliable in calculating the process oxygen requirement for the design of an extended aeration process.

Table 5 also shows that the TOR calculated by Method 6 agrees quite well with the TOR obtained by Method 1. Thus, the use of the traditional synthesis and endogenous respiration equation for design purposes is also satisfactory provided that the correct values for the empirical coefficient 'a' and 'b' are used and also that BOD₅ degraded rather than removed (i.e. influent minus effluent BOD₅) be used. Both Methods 7 and 8 which are based on the "conventional" activated sludge typical values for 'a' and 'b' grossly over-estimated the process oxygen requirement.

The TOR based on the FWD design criteria also agrees very closely with the TOR obtained by Method 1. However, the SOTR/BOD₅ ratio of 2.4 is about 11% less than the ratio of 2.7 obtained by Method 1. The difference is due largely to the difference in the FWD adopted design α value of 0.85 compared to the actual α value of 0.76 obtained for the submerged jet aerators. If the α value was 0.85 (typical for surface aerators) then the SOTR/BOD₅ ratio obtained by Method 1 would be 2.4 - the same as the FWD design criterion. The difference in the MLSS DO concentrations used in Method 1 compared to an assumed value of 1 mg/l used by the FWD for converting OTR' to SOTR would also result in a difference in the SOTR/BOD₅ ratios. The average MLSS DO concentration of 1.6 mg/l obtained over the 24-hour period would result in a higher SOTR/BOD₅ ratio than the assumed MLSS DO of 1 mg/l. If an α of 0.85 and an MLSS DO concentration of 1 mg/l were adopted, Method 1 would yield an average SOTR/BOD₅ ratio of 2.23 and a peak cycle SOTR/BOD₅ ratio 2.61. Thus, it can be seen that the FWD design criteria of a normal and maximum SOTR/BOD₅ ratios of 2.4 and 2.7 respectively are satisfactory for design purposes, if the α value is not significantly less than 0.85. Also, the safety factor of 1.125 adopted by the FWD in deriving the maximum SOTR/BOD₅ ratio is adequate in satisfying the peak cycle TOR. Method 1 gave a peak to average cycle TOR of 1.17. Table 5 showed that Methods 2 to 8 over-estimated the peak cycle TOR. Table 6 summarises the percentage differences in the values obtained by Method 1 compared with the values obtained by the other methods.

Table 6 - Percent Differences from Method 1 Values

Method	% Difference	% Difference TOR/BOD ₅		%Difference SOTR/BOD ₅	
	TOR	Average	Peak Cycle	Average	Peak Cycle
2	+0.3	0	+79.5	0	+79.4
3	-2.5	-2.5	+100.0	-2.6	+100.0
4	+8.3	+8.2	+87.0	+8.1	+87.3
5	+5.0	+4.4	+78.4	+4.4	+79.0
6	-1.8	-1.9	+67.6	-1.9	+68.3
7	+83.3	+82.9	+212.0	+83.0	+213.7
8	+78.7	+78.5	+204.9	+78.5	+206.0
9	-13.0	-13.3	-8.1	-23.7	-19.0
10	+1.6	+1.3	-2.7	-11.1	-14.3

Finally, it should be noted that a nitrogen to BOD₅ ratio of 0.14 in the influent sewage was adopted in calculating the FWD SOTR/BOD₅ ratio of 2.4. The actual nitrogen to BOD₅ ratio measured for the influent sewage for the 24-hour sampling period was 0.19. The higher ratio of 0.19 would also result in a higher measured SOTR/BOD₅ ratio compared to the FWD value. This point together with those for the different α and MLS DO concentration can also explain the lower rule of thumb values derived for the typical 4000 ep IEA plant example (Method 9).

12. CONCLUSIONS

1. In the design of an extended aeration activated sludge plant, the daily process oxygen requirement can be calculated from the theoretical carbonaceous oxygen requirement (COR) and nitrogenous oxygen requirement (NOR) of the influent.

The COR can be calculated according to the following equation:

$$\text{COR} = 1.46(\text{BOD}_5) - 1.42f_{\text{BVSS}}(\text{MLVSS}/\theta_c)$$

where BOD_5 = Influent BOD_5 degraded
 f_{BVSS} = MLVSS biodegradable fraction
 θ_c = sludge age

The NOR can be calculated according to the following equation:

$$\text{NOR} = 4.6P_N(\text{tot.N}) - 2.9P_{\text{DN}} P_N (\text{tot.N})$$

where tot.N = influent total nitrogen
 P_N = fraction tot.N nitrified
 P_{DN} = fraction nitrate denitrified

The theoretical process total oxygen requirement (TOR) is thus:

$$\text{TOR} = \text{COR} + \text{NOR}$$

2. The theoretical process TOR calculated by the equations given in Conclusion 1 above was found to agree with the TOR of a 4000 ep Intermittent Extended Aeration (IEA) plant measured over a 24-hour period. The calculated theoretical TOR was about 319 kg O_2/d . The measured TOR based on an oxygen balance on the plant over the 24-hour period was about 318 kg O_2/d .

3. The equations given in Conclusion 1, however, cannot predict the diurnal variation in the IEA plant's process TOR as the equations do not take into account damping factors such as the cyclic operations of the plant; dilution effect; substrate storage by bacteria; the slower and steadier rate of biodegradation of particulate substrates; release of substrate by death and lysis of bacteria.

The diurnal peak to average cycle theoretical TOR ratio calculated by the equations in Conclusion 1 was about 2.1 to 1.0. The measured peak to average cycle TOR ratio based on the 24-hour oxygen balance was only about 1.17 to 1. This showed a damping factor of about 0.56 (56%) for the peak cycle to average cycle TOR for the IEA plant operating on a 4-hour cycle.

It is considered that a general safety factor of 1.25 would be more than adequate for sizing the aerator capacity for an IEA plant.

4. The general design figure of 2.4 kg O₂/kg influent BOD₅ adopted by the PWD for specifying the normal standard oxygen transfer rate (SOTR) of aerators for IEA plants was found to be valid if:
 - i) the α factor of the aerator is not less than 0.80;
 - ii) the ratio of influent tot.N to BOD₅ is not greater than 0.20;
 - iii) the sludge age is not greater than 65 days.
5. Great caution must be adopted in applying the following traditional synthesis and endogenous respiration empirical relationship in the determination of the extended aeration process COR.

$$\text{COR} = aS + bX$$

where a = empirical coefficient for kg O_2 utilised per
kg BOD_5 oxidised
 S = kg BOD_5 oxidised
 b = empirical coefficient for kg O_2 utilised for
endogenous respiration per kg MLVSS under aeration
 X = MLVSS under aeration

6. Adopting the typical literature values of 0.5 for 'a' and 0.1 for 'b', the traditional equation given in Conclusion 5 over-estimated the process TOR by more than 70%. Also, the calculated COR exceeded the ultimate BOD of the influent carbonaceous substrate. This is theoretically incorrect and points to the fact that the coefficients are not true constants. It was found that the values of $a = 0.75$ and $b = 0.03$ as determined by Fujimoto et al (Ref. 11) are more appropriate to the extended aeration mode of the activated sludge process. Using these coefficient values, TOR was calculated to be about 312 kg compared to the measured value of 318 kg.
7. Therefore, one must not lose sight of the fact that the following two general equations of the traditional activated sludge model are basically empirical in nature:

$$\text{COR}/X = aU + b$$

$$1/\theta_c = YU - K_d$$

where θ_c = sludge age or MCRT
 Y = sludge or growth yield coefficient,
kg MLVSS produced per kg BOD_5 removed
 U = modified F/M , kg BOD_5 removed per day per kg
MLVSS

= $E(F/M)$ where E is the BOD₅ removal efficiency
(% BOD₅ removed)

K_d = endogenous decay rate, kg MLVSS destroyed or
degraded per day per kg MLVSS remaining

The other parameters are as defined in Conclusion 5 above.

8. A major limitation of the above equations is the fact that the traditional "F/M" ratio or "U" is an empirical parameter because:

- i) MLVSS is not an accurate measure of the active biomass which is responsible for the biological reactions occurring in the activated sludge process. The active biomass fraction of the MLSS from the 4000 ep IEA plant was found to be only about 0.09 based on measurement of bacterial ATP in the MLSS. This agrees with the findings of several authors who reported that the MLSS active biomass fraction decreases with increasing sludge age levelling off at about 0.1 to 0.2 at long sludge ages of around 30 days.
- ii) BOD₅ is also not an accurate measure of the actual substrate available or degraded by the active biomass. The percentage degradation of a complex carbonaceous substrate such as sewage is time dependent. Hence, the amount of carbonaceous substrate degraded, or in other words actually available, is dependent upon the sludge age. Typically, a plant operating at a sludge age of 5 days may only degrade about 70% of the sewage carbonaceous substrate but a plant operating at 30 days sludge age may degrade close to 100% of the substrate.

9. The empirical nature of the two traditional equations given in Conclusion 7 would account for the great range in the reported values of the coefficients - up to a 5 fold range for 'b' and k_d . Based on the non-linear variations of the MLSS active biomass fraction and the substrate BOD degradation with time, it is most doubtful that the traditional empirical equations are in fact linear.
10. However, it is considered that the most critical flaw in the traditional activated sludge model is the failure to recognise that in a substrate limited process such as the "conventional" or extended aeration process, the active biomass in the process is directly proportional to and limited by the available substrate mass. It is theoretically not feasible to increase the process biomass without also increasing the substrate mass available to the process.

Hence, the view that an extended aeration process designed on an empirical "F/M" of 0.04 would have ten times more active biomass than a "conventional" process designed on an empirical "F/M" of 0.4 when receiving the same quantity of substrate is a misconception. Also, the traditional view that the extended aeration process has higher process oxygen requirements and less excess biomass production than the "conventional" process because of increased endogenous respiration and decay due to a greater amount of biomass in the former process are also misconceptions.

11. Based on the arguments presented in Conclusion 10, it is the author's hypothesis that in a substrate limited activated sludge process, the true or actual F/M ratio based on the actual available substrate degraded (or consumed) and active biomass should remain relatively constant and does not vary significantly with variations in the empirical "F/M" ($\text{kg BOD}_5/\text{kg MLSS.d}$) or sludge age (θ_c). This hypothesis is supported by literature data on the variation of the MLSS active biomass fraction with θ_c or the empirical "F/M" ratio as presented in Section 2.1.3.1.

12. The implications of the new hypothesis are:

- i) The active biomass is directly proportional to and limited by the amount of substrate consumed.
- ii) The "actual" substrate available for consumption is related to the sludge age and for domestic sewage may be represented by:

$$S_a = BOD_u (1 - 10^{-K\theta_c})$$

where BOD_u = substrate ultimate BOD conc.

K = BOD reaction time constant

Hence, for domestic sewage, the actual amount of substrate degraded will increase with sludge age resulting in a greater process oxygen requirement.

- iii) From (i) and (ii) above, it can be seen that θ_c is a more correct or appropriate parameter than the empirical "F/M" ratio for defining the "conventional" and extended aeration modes or variations of the activated sludge process.

13. The actual process oxygen requirement of a full-scale activated sludge plant can be reliably estimated by carrying out an oxygen mass balance.

14. In order to carry out the oxygen mass balance, the oxygen transfer rate (OTR) of the aerators must be determined. The non-linear least squares parameter estimation technique such as that described by Brown et al (Ref. 12) is most reliable for interpreting aeration test data as it also provides the least squares estimate of the dissolved oxygen (DO) saturation value.

15. Interpretation of submerged aeration test data is more complicated than that for surface aeration test data due to the variation of the DO saturation concentration with time in the former test.
16. It was found that for submerged aeration with low to moderate oxygen transfer efficiency of about 12%, the surface aeration model is a good approximation of the submerged aeration test and error in the OTR value obtained is insignificant (only about 4%).
17. However, for submerged aeration with high oxygen transfer efficiency of about 69%, the error in the OTR value obtained by the surface aeration model may be as high as 13%. It is considered that the higher error may be due to the fact that the surface aeration model assumes a constant DO saturation value but the value actually varies very significantly for submerged aeration with high oxygen transfer efficiency.
18. A new clean water submerged aeration test model which takes into account the variation of the DO saturation concentration during the aeration test was developed by the author. The new model describes the variations in the DO concentration and the saturation value during the submerged aeration test by the following equations:

$$C = C_{\infty}^* - (C_{\infty}^* - C_0)e^{-kt} - (C_{\infty}^* - C_0)kte^{-kt}$$

$$C^* = C_{\infty}^* - (C_{\infty}^* - C_0^*)e^{-kt}$$

where C = DO concentration
 C_0 = C at $t=0$
 t = time
 C^* = DO saturation concentration
 C_{∞}^* = C^* at $t=\infty$
 C_0^* = C^* at $t=0$
 k = $K_L a$, the overall volumetric oxygen transfer coefficient.

19. The significant advantages of the new model presented in conclusion 18 are:

- i) The non-linear least squares parameter estimation technique can be used with the model to provide least squares estimates of all the parameters, including the saturation values C_0^* and C_{∞}^* which are not provided in other submerged aeration test models such as the log-mean models.
- ii) It is a general model applicable to both submerged and surface aeration tests. In the surface aeration test, C^* is constant and C_0^* is thus equal to C_{∞}^* .

The equation of the new model thus reduces to the surface aeration test model as follows:

$$C = C_{\infty}^* - (C_{\infty}^* - C_0)e^{-kt}$$

$$C^* = C_{\infty}^*$$

20. As presented by Kayser (Ref. 13), the dirty water (MLSS) aeration test model is analogous to the clean water aeration test model if the oxygen uptake rate (r) of the MLSS remains constant during the aeration test. The dirty water aeration test model for surface aeration is represented by the following equation:

$$C' = C_S^{*'} - (C_S^{*'} - C_O')e^{-k't}$$

where C' = DO concentration in MLSS

$C_S^{*'}$ = apparent DO saturation conc. in MLSS

C_O' = MLSS DO conc. at $t=0$

k' = $K_L a$ in MLSS

The apparent MLSS DO saturation value is related to the actual MLSS saturation value, C_S' , by the following equation:

$$C_S^{*'} = C_S' - r/k'$$

where r = MLSS oxygen uptake rate

C_S' = real MLSS DO saturation conc.

21. The new clean water submerged aeration test model, given in Conclusion 18, can be similarly modified for MLSS submerged aerations tests as follows:

$$C' = C_{S\infty}^{*'} - (C_{S\infty}^{*'} - C_O')e^{-k't} - (C_{S\infty}^{*'} - C_{SO}^{*'})k'te^{-k't}$$

$$C_S^{*'} = C_{S\infty}^{*'} - (C_{S\infty}^{*'} - C_{SO}^{*'})e^{-k't}$$

where $C_{S\infty}^{*'}$ = apparent MLSS DO saturation conc. at $t=\infty$

$C_{SO}^{*'}$ = apparent MLSS DO saturation conc. at $t=0$

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APPENDIX A

ATP TEST RESULTS

CASTLE HILL

SAMPLE 1 (D = 200:1, ATP standard added = 5×10^{-3} μ g,
MLSS = 2250 mg/l, Blank = 0.13 RLU)

X ₁	X ₂	μ g ATP/mg MLSS
21.0	50.9	3.10
22.4	47.0	4.02
23.9	58.4	<u>3.06</u>
	Average	<u>3.39</u>

SAMPLE 2 (D = 200:1, ATP standard added = 5×10^{-3} μ g,
MLSS = 2330 mg/l, Blank = 0.13 RLU)

X ₁	X ₂	μ g ATP/mg MLSS
15.97	45.5	2.30
17.83	51.3	<u>2.27</u>
	Average	<u>2.29</u>

SAMPLE 3 (D = 20:1, ATP standard added = 10^{-1} μ g,
MLSS = 3700 mg/l, Blank = 0.007 RLU)

X ₁	X ₂	μ g ATP/mg MLSS
59.5	107.2	(6.74)
45.7	136.5	2.72
54.1	146.2	<u>3.18</u>
	Average	<u>2.95</u>

(D = 200:1, ATP standard added = 2×10^{-2} μ g)

X ₁	X ₂	μ g ATP/mg MLSS
6.20	28.2	3.05
6.25	28.7	3.01
5.97	28.4	<u>2.88</u>
	Average	<u>2.98</u>

(D = 1000:1, ATP standard added = 2×10^{-3} μ g)

X ₁	X ₂	μ g ATP/mg MLSS
1.34	3.45	3.43
1.23	3.41	<u>3.05</u>
	Average	<u>3.24</u>

(D = 2000:1, ATP standard added = 1×10^{-3} μg)

X_1	X_2	$\mu\text{g ATP/mg MLSS}$
0.83	2.56	2.59
0.7	2.83	<u>1.7</u>
	Average	<u>2.18</u>

Average for Sample = 2.84

FORSTER (D = 20:1, Blank = 0.04 RL.U)

Tank 1 - Afternoon Sample (MLSS = 4490 mg/l)

X_1	X_2	ATP Added	$\mu\text{g ATP/mg MLSS}$
2.07	27.0	10^{-1} μg	0.36
2.24	6.82	2×10^{-2} μg	0.43
3.49	6.31	2×10^{-2} μg	<u>(1.09)</u>
	Average		<u>0.40</u>

Tank 1 - Morning Sample (MLSS = 4080 mg/l)

X_1	X_2	ATP Added	$\mu\text{g ATP/mg MLSS}$
4.00	15.39	1×10^{-2} μg	0.17
3.33	14.06	1×10^{-2} μg	0.15
3.51	13.80	1×10^{-2} μg	<u>0.17</u>
	Average		<u>0.16</u>

Tank 2 - Afternoon Sample (MLSS = 4320, ATP added = 2×10^{-3} μg)

X ₁	X ₂	$\mu\text{g ATP/mg MLSS}$
3.23	6.19	1.00
3.26	6.27	0.99
3.71	6.28	<u>1.32</u>
	Average	<u>1.10</u>

Tank 2 - Morning Sample (MLSS = 3990, ATP added = 1×10^{-2} μg)

X ₁	X ₂	$\mu\text{g ATP/mg MLSS}$
8.57	30.90	0.19
8.59	30.80	0.19
8.53	29.60	<u>0.20</u>
	Average	<u>0.19</u>

Tank 4 - Afternoon Sample (MLSS = 3980, ATP added = 2×10^{-2} μg)

X ₁	X ₂	$\mu\text{g ATP/mg MLSS}$
4.17	10.24	0.68
2.77	7.90	0.53
3.79	10.58	<u>0.56</u>
	Average	<u>0.59</u>

Tank 4 - Morning Sample (MLSS = 4270, ATP added = 1×10^{-2} μg)

X ₁	X ₂	$\mu\text{g ATP/mg MLSS}$
6.93	12.57	0.57
6.69	12.74	0.52
6.91	11.73	<u>0.67</u>
	Average	<u>0.59</u>

BATHURST (D = 20:1, Blank = 0.01)

Tank 1 - am Start (MLSS = 3620 mg/l, ATP added = 2×10^{-3} μg)

X ₁	X ₂	$\mu\text{g ATP/mg MLSS}$
7.36	25.9	0.44
8.11	24.1	0.56
8.13	25.5	<u>0.52</u>
	Average	<u>0.51</u>

Tank 1 - pm Start (MLSS = 3690 mg/l, ATP added = 2×10^{-3} μg)

X ₁	X ₂	$\mu\text{g ATP/mg MLSS}$
6.40	23.8	0.40
5.62	21.9	0.37
7.08	28.2	<u>0.36</u>
	Average	<u>0.38</u>

Tank 1 - am End (MLSS = 3470 mg/l, ATP added = $2 \times 10^{-3} \mu\text{g}$)

X ₁	X ₂	$\mu\text{g ATP/mg MLSS}$
11.72	34.1	0.60
12.33	34.2	0.65
10.68	34.6	<u>0.51</u>
	Average	<u>0.59</u>

Tank 1 - pm End (MLSS = 4160 mg/l, ATP added = $2 \times 10^{-3} \mu\text{g}$)

X ₁	X ₂	$\mu\text{g ATP/mg MLSS}$
12.49	32.3	0.61
14.27	40.4	0.52
14.59	34.6	<u>0.70</u>
	Average	<u>0.61</u>

Tank 2 - am Start (MLSS = 3480 mg/l, ATP added = $2 \times 10^{-3} \mu\text{g}$)

X ₁	X ₂	$\mu\text{g ATP/mg MLSS}$
11.80	35.8	0.51
12.18	38.3	0.49
10.33	35.0	<u>0.43</u>
	Average	<u>0.48</u>

Tank 2 - pm Start (MLSS = 3720 mg/l, ATP added = $2 \times 10^{-3} \mu\text{g}$)

X_1	X_2	$\mu\text{g ATP/mg MLSS}$
12.70	37.8	0.54
12.55	28.8	0.83
13.76	32.3	<u>0.80</u>
	Average	<u>0.72</u>

Tank 2 - am End (MLSS = 3780)

X_1	X_2	ATP Added	$\mu\text{g ATP/mg MLSS}$
5.68	13.8	$1 \times 10^{-3} \mu\text{g}$	0.37
8.71	15.0	$1 \times 10^{-3} \mu\text{g}$	(0.73)
8.95	28.1	$2 \times 10^{-3} \mu\text{g}$	<u>0.49</u>
	Average		<u>0.43</u>

Tank 2 - pm End (MLSS = 3820 mg/l, ATP added = $2 \times 10^{-3} \mu\text{g}$)

X_1	X_2	$\mu\text{g ATP/mg MLSS}$
10.63	26.9	0.68
11.12	30.2	0.61
9.41	22.7	<u>0.74</u>
	Average	<u>0.68</u>

APPENDIX C1

CLEAN WATER AERATION TEST - PROBE NO. 1 RESULTS

TIME	PROBE DO Reading	MODIFIED SURFACE MODEL		NEW SUBMERGED MODEL	
		Predicted DO	Residual	Predicted DO	Residual
mins.	mg/l	mg/l	mg/l	mg/l	mg/l
0	0.08	0.24	-0.16	0.08	0.00
2	0.82	1.00	-0.18	0.85	-0.03
4	1.70	1.69	0.01	1.55	0.15
8	2.70	2.91	-0.21	2.80	-0.10
10	3.25	3.44	-0.19	3.36	-0.11
12	3.95	3.93	0.02	3.86	0.09
14	4.30	4.38	-0.83	4.33	-0.03
16	4.70	4.79	-0.09	4.76	-0.06
18	5.15	5.17	-0.02	5.15	0.00
20	5.58	5.52	0.06	5.51	0.07
22	5.75	5.84	-0.09	5.84	-0.09
24	6.15	6.13	0.02	6.14	0.01
26	6.45	6.39	0.06	6.42	0.03
28	6.68	6.64	0.04	6.67	0.01
30	6.88	6.86	0.02	6.90	-0.02
32	7.18	7.07	0.11	7.11	0.07
37	7.60	1.51	0.09	7.57	0.03
42	7.98	7.87	0.11	7.93	0.05
47	8.15	8.16	-0.01	8.21	-0.06
52	8.45	8.39	0.06	8.44	0.01
57	8.60	8.57	0.03	8.62	-0.02
62	8.75	8.72	0.03	8.76	-0.01
67	8.88	8.84	0.04	8.87	0.01
72	8.98	8.94	0.04	8.96	0.02
77	9.05	9.02	0.03	9.03	0.02
82	9.10	9.08	0.02	9.08	0.02

TIME	PROBE DO	MODIFIED SURFACE MODEL		NEW SUBMERGED MODEL	
		Predicted DO	Residual	Predicted DO	Residual
mins.	Reading mg/l	mg/l	mg/l	mg/l	mg/l
87	9.10	9.13	-0.03	9.13	-0.03
92	9.18	9.17	0.01	9.16	0.02
97	9.18	9.20	-0.02	9.19	-0.01
102	9.18	9.23	-0.05	9.21	-0.03
107	9.18	9.25	-0.07	9.22	-0.04
112	9.25	9.27	-0.02	9.23	0.02
117	9.25	9.28	-0.03	9.24	0.01
122	9.25	9.29	-0.04	9.25	0.00
127	9.25	9.30	-0.05	9.26	-0.01

APPENDIX C2

CLEAN WATER AERATION TEST - PROBE NO. 2 RESULTS

TIME	PROBE DO	MODIFIED SURFACE MODEL		NEW SUBMERGED MODEL	
		Predicted DO	Residual	Predicted DO	Residual
mins.	Reading mg/l	mg/l	mg/l	mg/l	mg/l
0	0.01	-0.25	0.26	-0.22	0.24
2	0.88	0.58	0.30	0.59	0.29
4	1.20	1.33	-0.13	1.33	-0.14
6	1.77	2.02	-0.25	2.02	-0.25
8	2.38	2.65	-0.27	2.65	-0.27
10	2.94	3.23	-0.29	3.22	-0.28
12	3.75	3.75	0.00	3.75	0.00
14	4.20	4.23	-0.03	4.23	-0.03
16	4.54	4.67	-0.13	4.66	-0.12
18	5.19	5.07	0.12	5.06	0.13
20	5.49	5.43	0.06	5.43	0.06
22	5.84	5.77	0.07	5.76	0.08
24	6.04	6.07	-0.03	6.07	-0.03
26	6.45	6.35	0.10	6.35	0.10
28	6.66	6.60	0.06	6.60	0.06
30	6.89	6.83	0.06	6.83	0.06
35	7.48	7.33	0.15	7.33	0.15
40	7.74	7.72	0.02	7.73	0.01
45	8.04	8.03	0.01	8.04	0.00
50	8.34	8.28	0.06	8.29	0.05
55	8.55	8.48	0.07	8.49	0.06
60	8.69	8.64	0.05	8.65	0.04
65	8.79	8.76	0.03	8.77	0.02
70	8.89	8.86	0.03	8.87	0.02
75	8.96	8.94	0.02	8.95	0.01
80	9.02	9.01	0.01	9.01	0.01
85	9.03	9.06	-0.03	9.06	-0.03

TIME	PROBE DO	MODIFIED SURFACE MODEL		NEW SUBMERGED MODEL	
		Predicted DO	Residual	Predicted DO	Residual
mins.	mg/l	mg/l	mg/l	mg/l	mg/l
90	9.09	9.10	-0.01	9.10	-0.01
95	9.08	9.13	-0.05	9.13	-0.05
100	9.12	9.15	-0.03	9.15	-0.03
105	9.12	9.17	-0.05	9.17	-0.05
110	9.17	9.19	-0.02	9.19	-0.02
115	9.15	9.20	-0.05	9.20	-0.05
120	9.15	9.21	-0.06	9.21	-0.06
125	9.16	9.22	-0.06	9.22	-0.06

APPENDIX C3

CLEAN WATER AERATION TEST - PROBE NO. 3 RESULTS

TIME	PROBE DO Reading	MODIFIED SURFACE MODEL		NEW SUBMERGED MODEL	
		Predicted DO	Residual	Predicted DO	Residual
mins.	mg/l	mg/l	mg/l	mg/l	mg/l
0	0.19	0.03	0.16	0.06	0.13
2	0.90	0.91	-0.01	0.92	-0.02
4	1.59	1.71	-0.12	1.70	-0.11
6	2.31	2.42	-0.11	2.41	-0.10
8	2.93	3.07	-0.14	3.05	-0.12
10	3.71	3.65	0.06	3.63	0.08
12	4.18	4.17	0.01	4.16	0.02
14	4.64	4.65	-0.01	4.63	0.01
16	5.14	5.07	0.07	5.06	0.08
18	5.55	5.46	0.09	5.45	0.10
20	5.80	5.80	0.00	5.80	0.00
22	6.10	6.11	-0.01	6.11	-0.01
24	6.45	6.39	0.06	6.40	0.05
26	6.67	6.65	0.02	6.65	0.02
31	7.18	7.17	0.01	7.19	-0.01
36	7.58	7.58	0.00	7.60	-0.02
41	7.83	7.89	-0.06	7.91	-0.08
46	8.13	8.14	-0.01	8.16	-0.03
51	8.31	8.32	-0.01	8.34	-0.03
56	8.46	8.47	-0.01	8.48	-0.02
61	8.57	8.58	-0.01	8.59	-0.02
66	8.65	8.65	0.00	8.66	-0.01
71	8.73	8.73	0.00	8.73	0.00
76	8.78	8.78	0.00	8.78	0.00
81	8.82	8.82	0.00	8.82	0.00
86	8.86	8.85	0.01	8.84	0.02
91	8.87	8.87	0.00	8.86	0.01

TIME	MODIFIED SURFACE MODEL			NEW SUBMERGED MODEL	
	PROBE DO	Predicted DO	Residual	Predicted DO	Residual
	Reading				
mins.	mg/l	mg/l	mg/l	mg/l	mg/l
96	8.89	8.89	0.00	8.88	0.01
101	8.90	8.90	0.00	8.89	0.01
106	8.91	8.91	0.00	8.90	0.01
111	8.92	8.92	0.00	8.91	0.01
116	8.93	8.93	0.00	8.91	0.02
121	8.93	8.93	0.00	8.92	0.01

APPENDIX C4

CLEAN WATER AERATION TEST - PROBE NO. 4 RESULTS

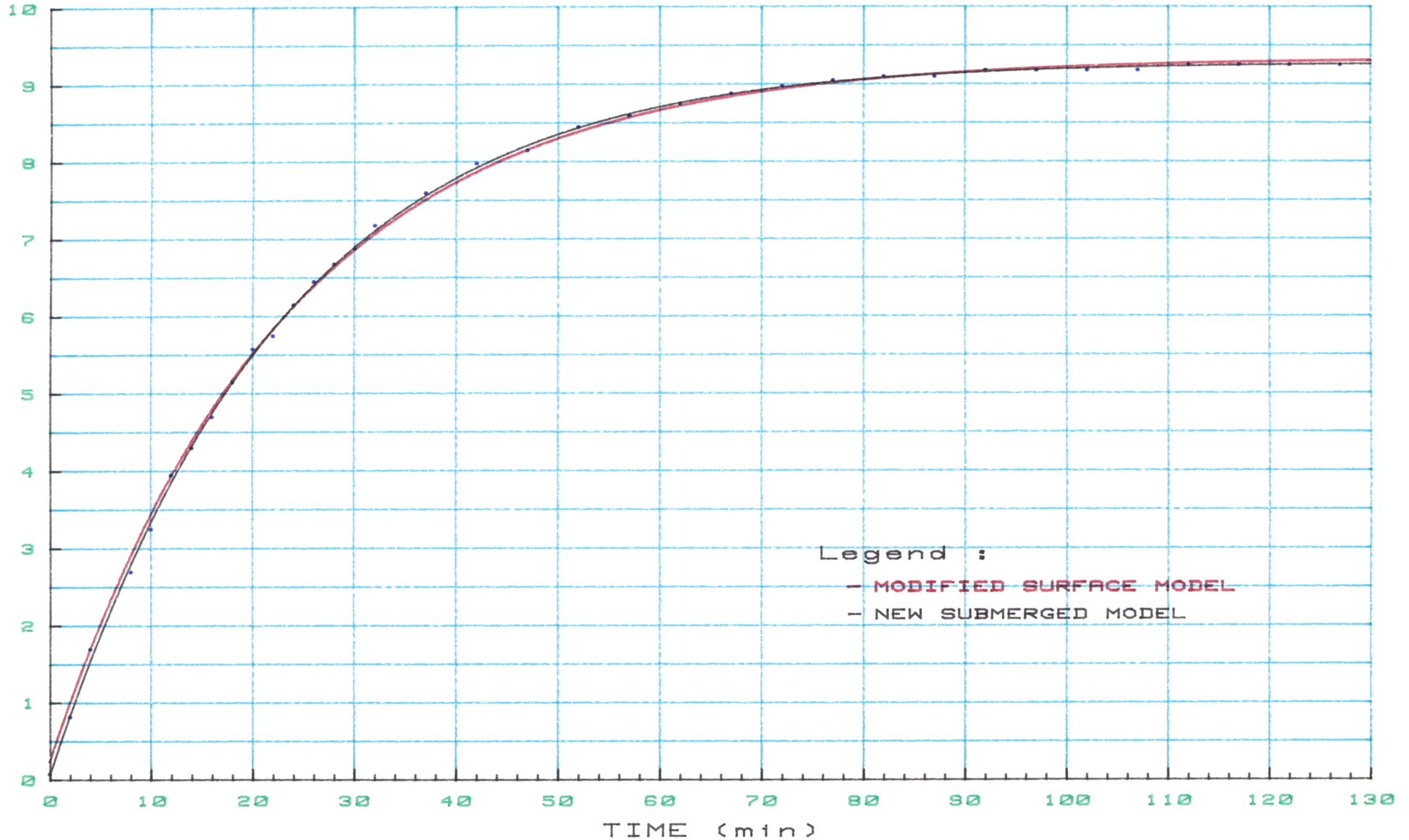
TIME	PROBE DO Reading	MODIFIED SURFACE MODEL		NEW SUBMERGED MODEL	
		Predicted DO	Residual	Predicted DO	Residual
mins.	mg/l	mg/l	mg/l	mg/l	mg/l
0	0.25	0.15	0.10	0.16	0.09
2	1.00	1.05	-0.05	1.05	-0.05
4	1.80	1.86	-0.06	1.86	-0.06
6	2.60	2.59	0.01	2.59	0.01
8	3.20	3.25	-0.05	3.24	-0.04
10	3.80	3.84	-0.04	3.83	-0.03
12	4.40	4.37	0.03	4.37	0.03
14	4.80	4.85	-0.05	4.85	-0.05
16	5.30	5.28	0.02	5.28	0.02
18	5.70	5.67	0.03	5.67	0.03
20	6.10	6.02	0.08	6.02	0.08
22	6.30	6.33	-0.03	6.33	-0.03
24	6.70	6.62	0.08	6.62	0.08
26	6.90	6.87	0.03	6.87	0.03
31	7.40	7.40	0.00	7.41	-0.01
36	7.80	7.81	-0.01	7.81	-0.01
41	8.00	8.12	-0.12	8.13	-0.13
46	8.40	8.36	0.04	8.37	0.03
51	8.50	8.54	-0.04	8.55	-0.05
56	8.70	8.69	0.15	8.69	0.01
61	8.80	8.79	0.01	8.80	0.00
66	8.90	8.88	0.02	8.88	0.02
71	8.90	8.94	-0.04	8.94	-0.04
76	9.00	8.99	0.01	8.99	0.01
81	9.00	9.03	-0.03	9.03	-0.03
86	9.10	9.06	0.04	9.05	0.05
91	9.10	9.08	0.02	9.07	0.03
96	9.10	9.10	0.00	9.09	0.01
101	9.10	9.11	-0.01	9.10	0.00

CLEAN WATER AERATION TEST

TEST No. : 1

PROBE No. : 1

C (mg/L)



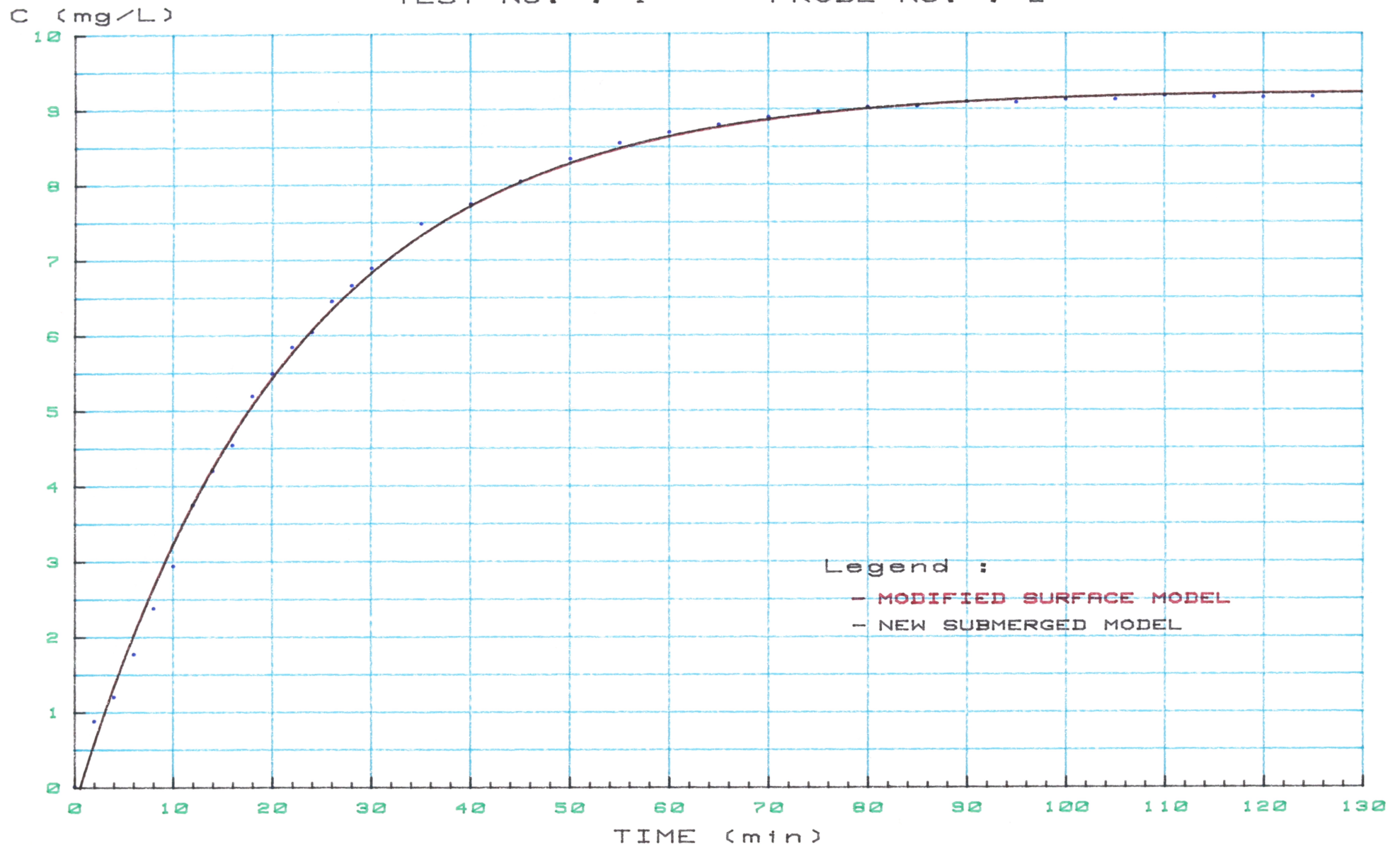
Program : SGE3R

Sgs Br, PWD, NSW.

CLEAN WATER AERATION TEST

TEST No. : 1

PROBE No. : 2



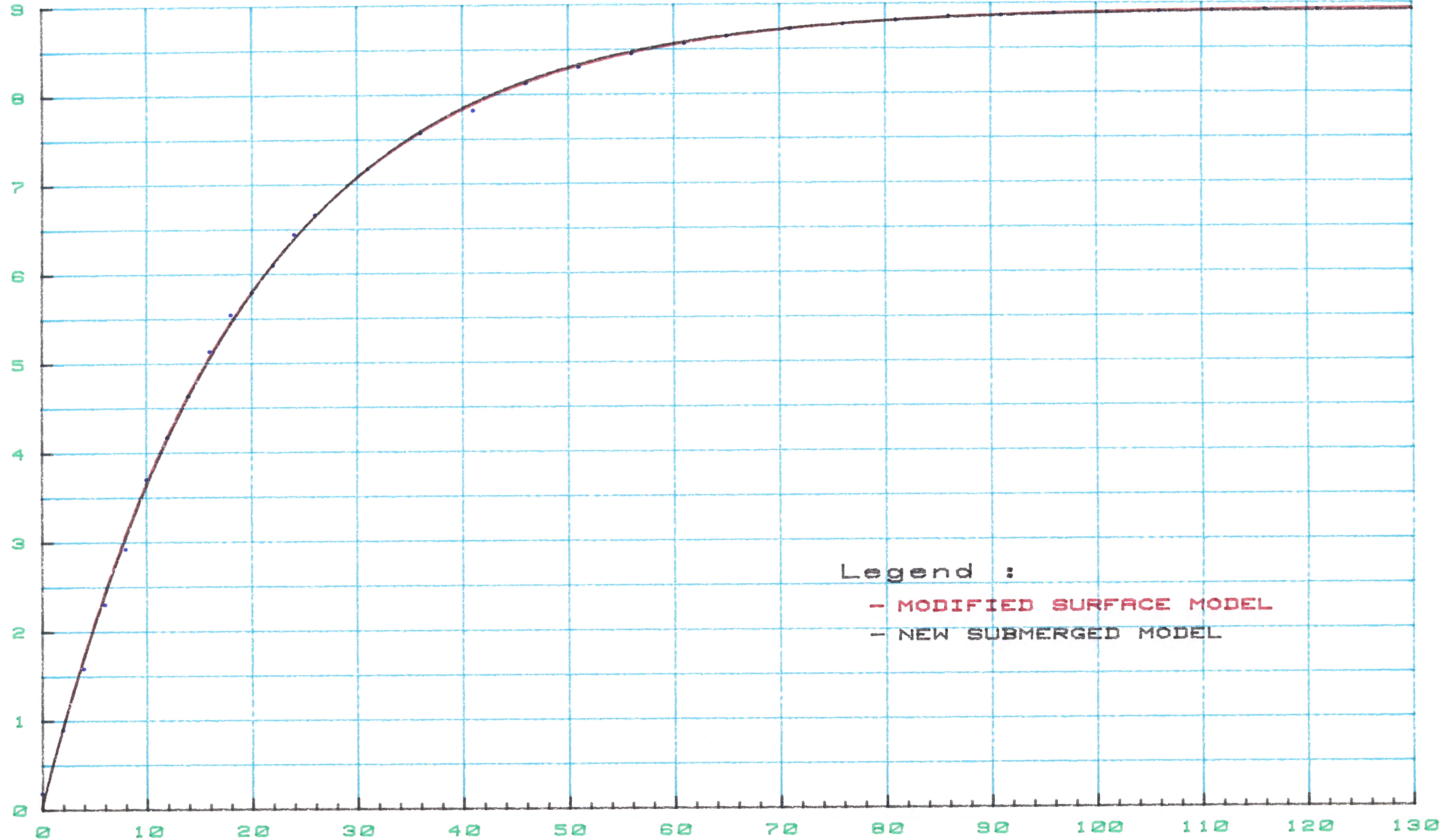
Program : SGEBR
Sgs Br, PWD, NSW.

CLEAN WATER AERATION TEST

TEST No. : 1

PROBE No. : 3

C (mg/L)



Legend :

- MODIFIED SURFACE MODEL

- NEW SUBMERGED MODEL

TIME (min)

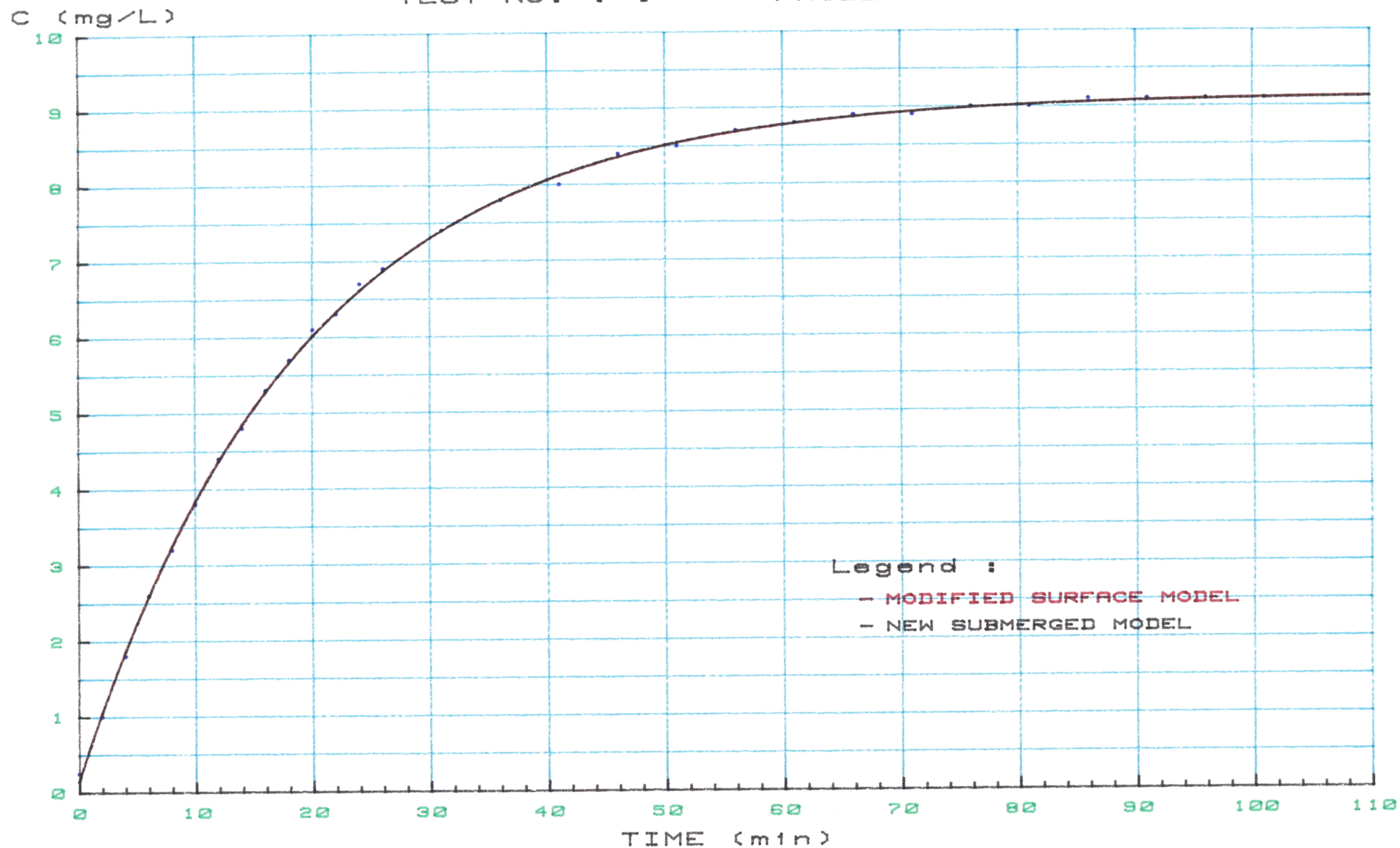
Program : SGEBR

Sgs Br, PWD, NSW.

CLEAN WATER AERATION TEST

TEST No. : 1

PROBE No. : 4



Program : SGEBR

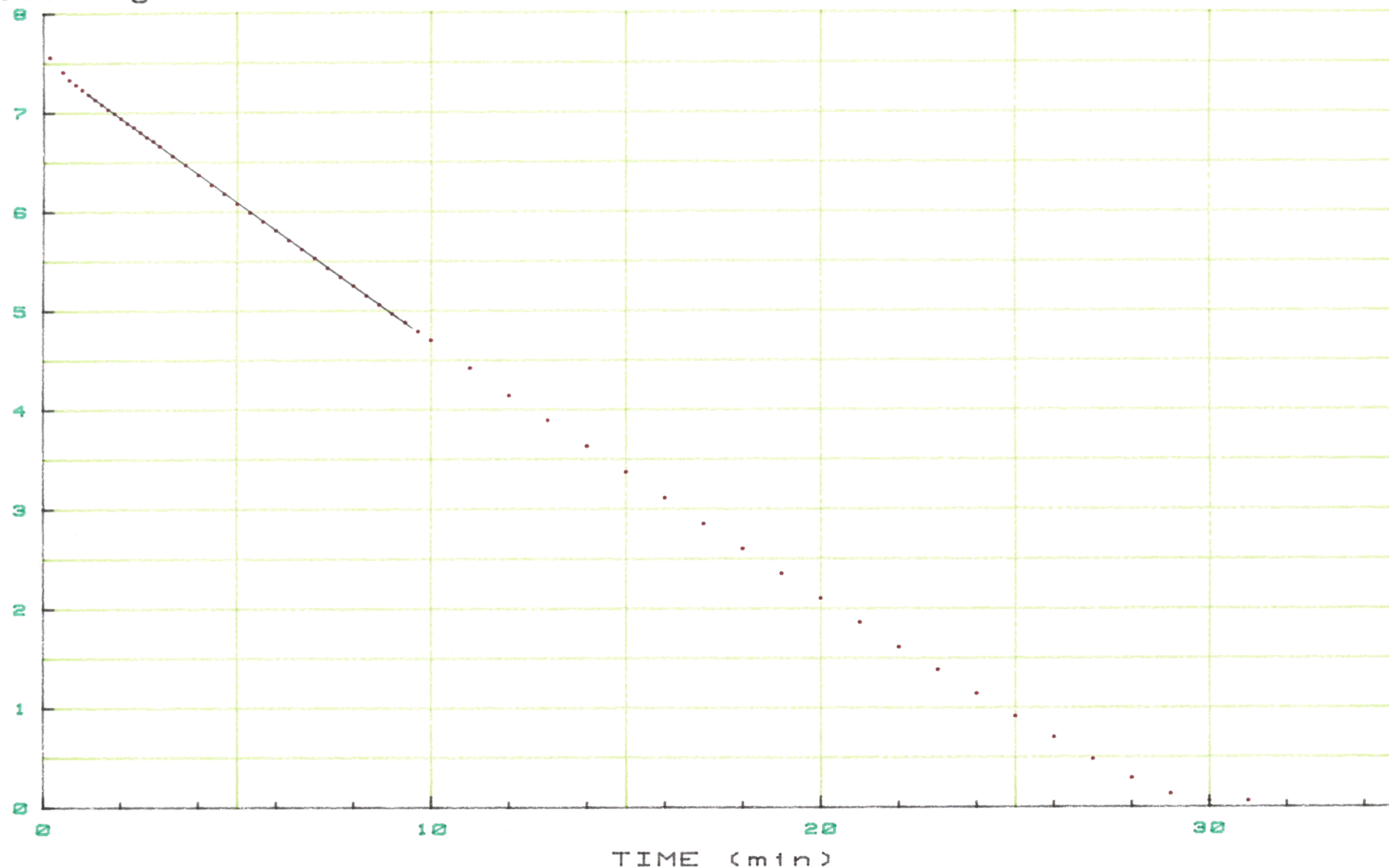
See Br. PWD, NSW.

TESTING DATE : 9/10/84

OXYGEN UPTAKE RATE TEST TANK No. 3

SAMPLE No. : 1

D.O. (mg/L)

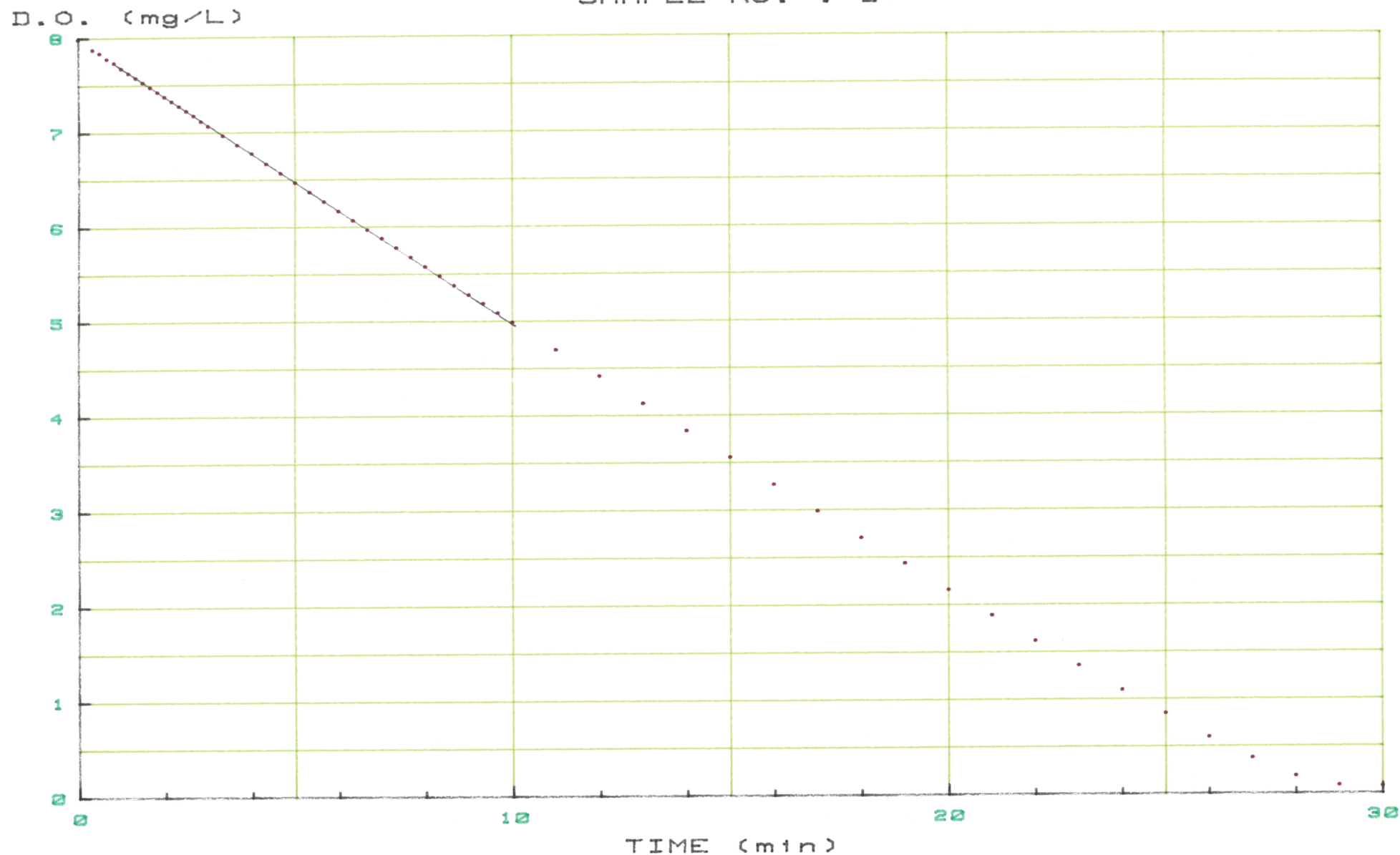


Program : SGEER

Sgs Br, PWD, NSW.

OXYGEN UPTAKE RATE TEST TANK No.3

SAMPLE No. : 2



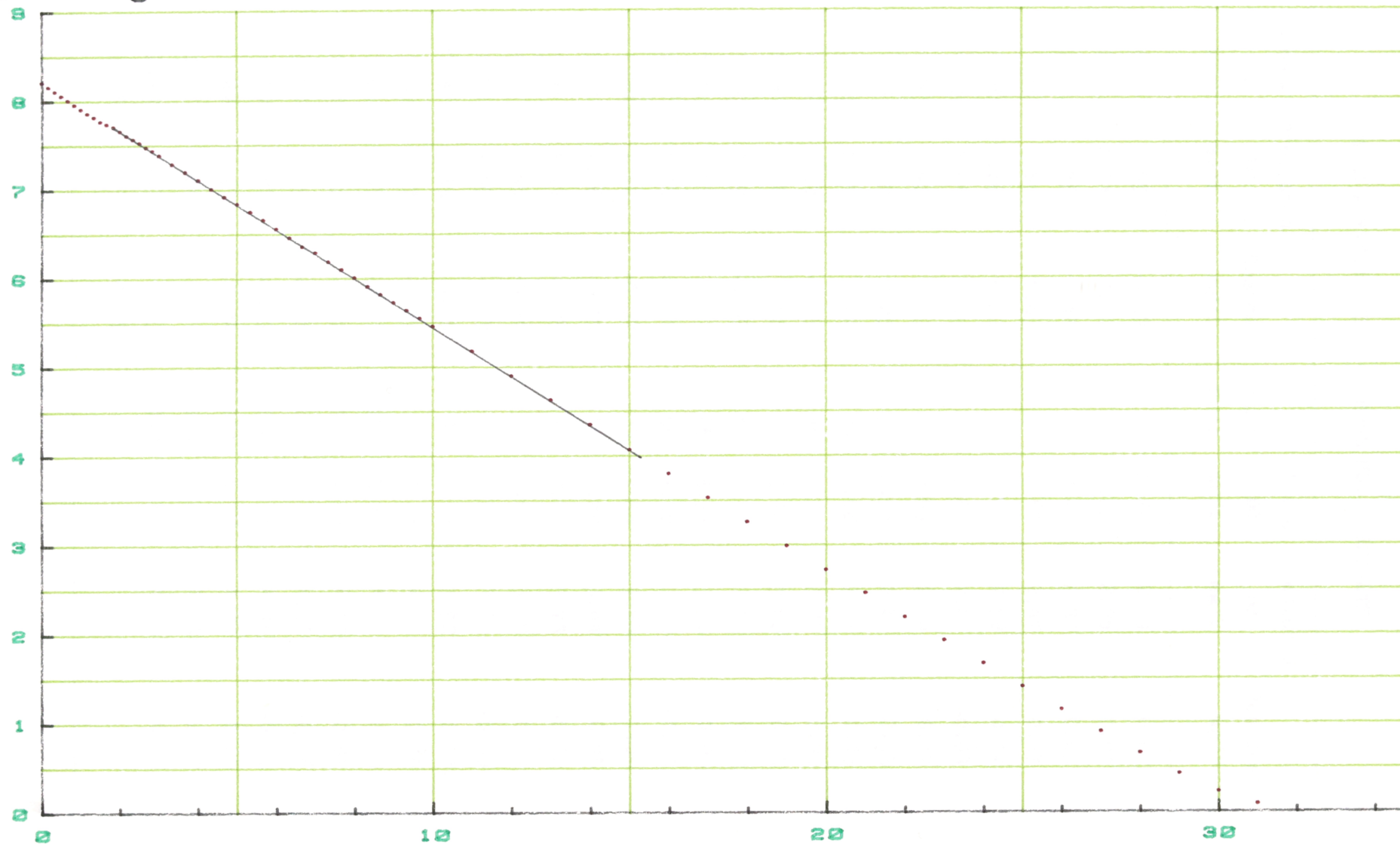
Program : SGEBR

Sgs Br, PWD, NSW.

OXYGEN UPTAKE RATE TEST No. 3

SAMPLE No. : 3

D.O. (mg/L)



Program : SGE3R

Sgs Br, PWD, NSW.

APPENDIX F1

MLSS AERATION TEST - PROBE NO. 1 RESULTS

TIME	Probe	Predicted		Time	Probe	Predicted	
mins.	DO	DO	Residual	mins.	DO	DO	Residual
	mg/l	mg/l	mg/l		mg/l	mg/l	mg/l
0	0.26	0.65	-0.39	19	2.24	2.19	0.05
0.5	0.81	0.71	0.10	20	2.25	2.25	0.00
1.0	0.90	0.76	0.14	22	2.42	2.37	0.05
1.5	1.26	0.81	0.45	24	2.64	2.47	0.17
2.0	1.08	0.86	0.22	26	2.72	2.58	0.14
2.5	1.10	0.91	0.19	28	2.63	2.67	-0.04
3.0	1.08	0.96	0.12	30	2.93	2.76	0.17
3.5	1.05	1.00	0.05	32	2.98	2.85	0.13
4.0	1.12	1.05	0.07	34	3.08	2.93	0.15
4.5	1.05	1.10	-0.05	36	3.30	3.00	0.30
5.0	1.20	1.14	0.06	38	3.16	3.08	0.08
5.5	1.13	1.19	-0.06	40	3.11	3.14	-0.03
6.0	1.24	1.23	0.01	45	3.33	3.30	0.03
6.5	1.27	1.28	-0.01	50	3.30	3.43	-0.13
7.0	1.29	1.32	-0.03	55	3.58	3.54	0.04
7.5	1.21	1.36	-0.15	60	3.69	3.64	0.05
8.0	1.16	1.41	-0.25	65	3.63	3.73	-0.10
8.5	1.07	1.45	-0.38	70	3.69	3.80	-0.11
9.0	1.26	1.49	-0.23	75	3.81	3.86	-0.05
9.5	1.37	1.53	-0.16	80	3.78	3.92	-0.14
10	1.50	1.57	-0.07	90	4.11	4.01	0.10
11	1.64	1.65	-0.01	100	3.93	4.07	-0.14
12	1.70	1.72	-0.02	110	4.03	4.12	-0.09
13	1.63	1.79	-0.16	120	4.10	4.16	-0.06
14	1.70	1.87	-0.17	130	4.17	4.19	-0.02
15	1.89	1.93	-0.04	140	4.16	4.21	-0.05
16	2.01	2.00	0.01	160	4.36	4.24	0.12
17	2.10	2.07	0.03	180	4.41	4.25	0.16
18	2.04	2.13	-0.09				

APPENDIX F2

MLSS AERATION TEST - PROBE NO. 2 RESULTS

TIME	Probe	Predicted		Time	Probe	Predicted	
mins.	DO	DO	Residual	mins.	DO	DO	Residual
	mg/l	mg/l	mg/l		mg/l	mg/l	mg/l
0	0.16	0.24	-0.08	19	1.91	1.91	0.00
0.5	0.31	0.30	0.01	20	2.04	1.97	0.07
1	0.46	0.35	0.11	22	2.24	2.10	0.14
1.5	0.53	0.41	0.12	24	2.36	2.21	0.15
2	0.85	0.46	0.39	26	2.46	2.32	0.14
2.5	0.68	0.52	0.16	28	2.52	2.43	0.09
3	0.63	0.57	0.06	30	2.53	2.52	0.01
3.5	0.79	0.62	0.17	32	2.73	2.62	0.11
4	0.72	0.67	0.05	34	2.79	2.70	0.09
4.5	0.87	0.72	0.15	36	2.91	2.79	0.12
5	0.89	0.77	0.12	38	2.84	2.86	-0.02
5.5	0.76	0.82	-0.06	40	2.96	2.94	0.02
6	0.75	0.87	-0.12	45	3.12	3.10	0.02
6.5	0.88	0.92	-0.04	50	3.32	3.24	0.08
7	0.87	0.96	-0.09	55	3.37	3.37	0.00
7.5	0.88	1.01	-0.13	60	3.48	3.47	0.01
8	0.81	1.06	-0.25	65	3.57	3.57	0.00
8.5	0.93	1.10	-0.17	70	3.53	3.65	-0.12
9	0.99	1.15	-0.16	75	3.63	3.71	-0.08
9.5	0.83	1.19	-0.36	80	3.85	3.77	0.08
10	1.00	1.23	-0.23	90	3.96	3.87	0.09
11	1.26	1.32	-0.06	100	3.87	3.94	-0.07
12	1.21	1.40	-0.19	110	3.99	3.99	0.00
13	1.36	1.48	-0.12	120	3.97	4.03	-0.06
14	1.40	1.55	-0.15	130	4.02	4.06	-0.04
15	1.74	1.63	0.11	140	4.01	4.08	-0.07
16	1.63	1.70	-0.07	160	4.13	4.11	0.02
17	1.77	1.77	0.00	180	4.17	4.13	0.04
18	1.88	1.84	0.04				

APPENDIX F3

MLSS AERATION TEST - PROBE NO. 3 RESULTS

TIME	Probe	Predicted	Residual	Time	Probe	Predicted	Residual
mins.	D0	D0	D0	mins.	D0	D0	D0
mg/l	mg/l	mg/l	mg/l	mg/l	mg/l	mg/l	mg/l
0.5	0.75	0.88	-0.13	19	2.60	2.67	-0.07
1	0.94	0.95	-0.01	20	2.70	2.73	-0.03
1.5	1.12	1.01	0.11	22	3.20	2.86	0.34
2	1.29	1.07	0.22	24	3.01	2.98	0.03
2.5	1.35	1.14	0.21	26	3.10	3.09	0.01
3	1.23	1.20	0.03	28	3.24	3.20	0.04
3.5	1.43	1.26	0.17	30	3.48	3.29	0.19
4	1.41	1.31	0.10	32	3.48	3.38	0.10
4.5	1.48	1.37	0.11	34	3.46	3.47	-0.01
5	1.48	1.43	0.05	36	3.67	3.55	0.12
5.5	1.49	1.48	0.01	38	3.73	3.62	0.11
6	1.60	1.54	0.06	40	3.85	3.69	0.16
6.5	1.57	1.59	-0.02	45	3.71	3.84	-0.13
7	1.51	1.64	-0.13	50	4.01	3.96	0.05
7.5	1.43	1.70	-0.27	55	3.95	4.07	-0.12
8	1.69	1.75	-0.06	60	4.18	4.16	0.02
8.5	1.60	1.80	-0.20	65	4.19	4.23	0.04
9	1.40	1.85	-0.45	70	4.35	4.30	0.05
9.5	1.78	1.89	-0.11	75	4.30	4.35	-0.05
10	1.72	1.94	-0.22	80	4.13	4.39	-0.26
11	2.00	2.03	-0.03	90	4.45	4.46	-0.01
12	1.93	2.12	-0.19	100	4.13	4.51	-0.38
13	2.25	2.21	0.04	110	4.50	4.54	-0.04
14	2.39	2.29	0.10	120	4.56	4.57	-0.01
15	2.41	2.37	0.04	130	4.81	4.58	0.23
16	2.43	2.45	-0.02	140	4.74	4.60	0.14
17	2.57	2.52	0.05	162	4.71	4.61	0.10
18	2.60	2.60	0.00	180	4.62	4.62	0.00

APPENDIX F4

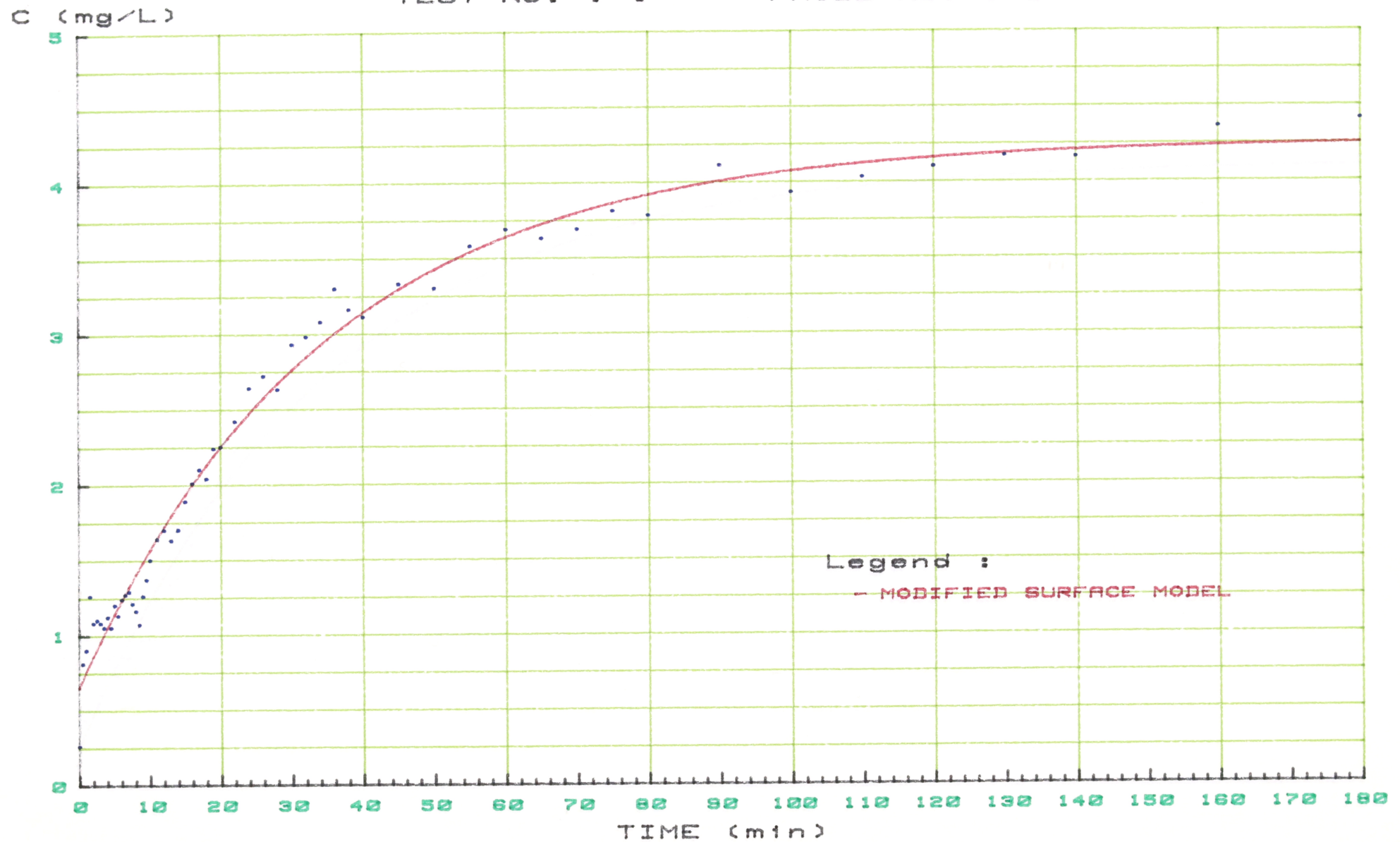
MLSS AERATION TEST - PROBE NO. 4 RESULTS

TIME	Probe	Predicted	Residual	Time	Probe	Predicted	Residual
mins.	DO	DO	mg/l	mins.	DO	DO	mg/l
mg/l	mg/l	mg/l	mg/l	mg/l	mg/l	mg/l	mg/l
0.5	0.70	0.80	-0.10	19	3.00	2.70	0.30
1	1.08	0.87	0.21	20	2.70	2.77	-0.07
1.5	1.16	0.94	0.22	22	3.28	2.91	0.37
2	1.07	1.00	0.07	24	3.11	3.04	0.07
2.5	1.22	1.07	0.15	26	3.13	3.16	-0.03
3	1.28	1.13	0.15	28	3.17	3.27	-0.10
3.5	1.22	1.20	0.02	30	3.48	3.38	0.10
4	1.30	1.26	0.04	32	3.42	3.47	-0.05
4.5	1.36	1.32	0.04	34	3.71	3.57	0.14
5	1.40	1.38	0.02	36	3.78	3.65	0.13
5.5	1.45	1.44	0.01	38	3.84	3.73	0.11
6	1.47	1.49	-0.02	40	3.83	3.81	0.02
6.5	1.39	1.55	-0.16	45	3.90	3.97	-0.07
7	1.53	1.61	-0.08	50	4.20	4.11	0.09
7.5	1.30	1.66	-0.36	55	4.26	4.23	0.03
8	1.60	1.72	-0.12	60	4.35	4.33	0.02
8.5	1.59	1.77	-0.18	65	4.42	4.41	0.01
9	1.60	1.82	-0.22	70	4.53	4.49	0.04
9.5	1.69	1.87	-0.18	75	4.27	4.55	-0.28
10	1.74	1.92	-0.18	80	4.25	4.60	-0.35
11	1.86	2.02	-0.16	90	4.71	4.67	0.04
12	1.84	2.12	-0.28	100	4.36	4.73	-0.37
13	2.32	2.21	0.11	110	4.67	4.77	-0.10
14	2.55	2.30	0.25	120	4.81	4.80	0.01
15	2.60	2.38	0.22	130	5.02	4.82	0.20
16	2.30	2.47	-0.17	140	4.93	4.83	0.10
17	2.51	2.55	-0.04	162	5.11	4.85	0.26
18	2.70	2.62	0.08	180	4.89	4.86	0.03

DIRTY WATER (MLSS) AERATION TEST

TEST No. : 1

PROBE No. : 1



Program : SGEBR
Sge Br, PWD, NSW.

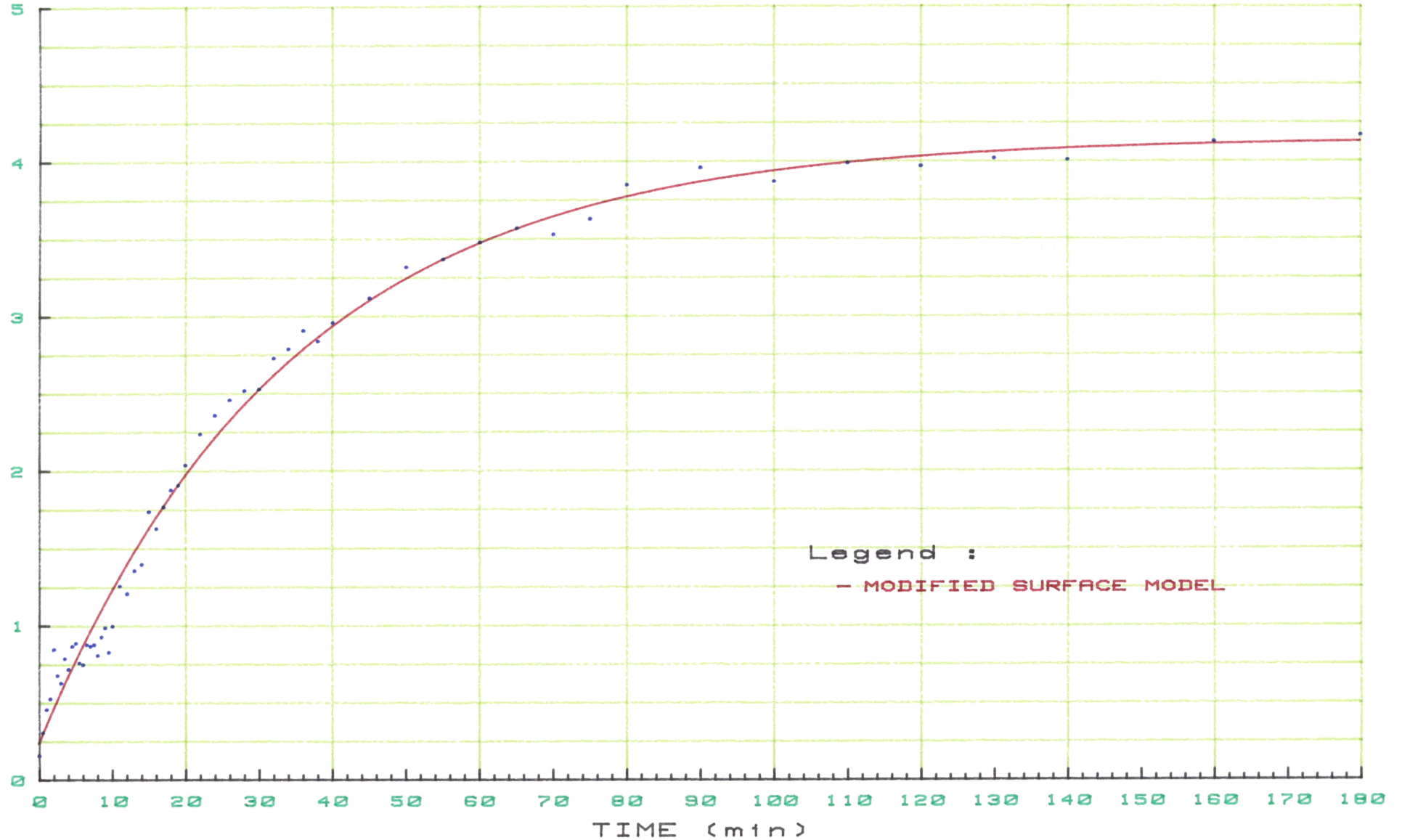
TESTING DATE : 9/10/84

DIRTY WATER (MLSS) AERATION TEST

TEST No. : 1

PROBE No. : 2

C (mg/L)



Legend :

- MODIFIED SURFACE MODEL

Program : SGEBR

Sge Br, PWD, NSW.

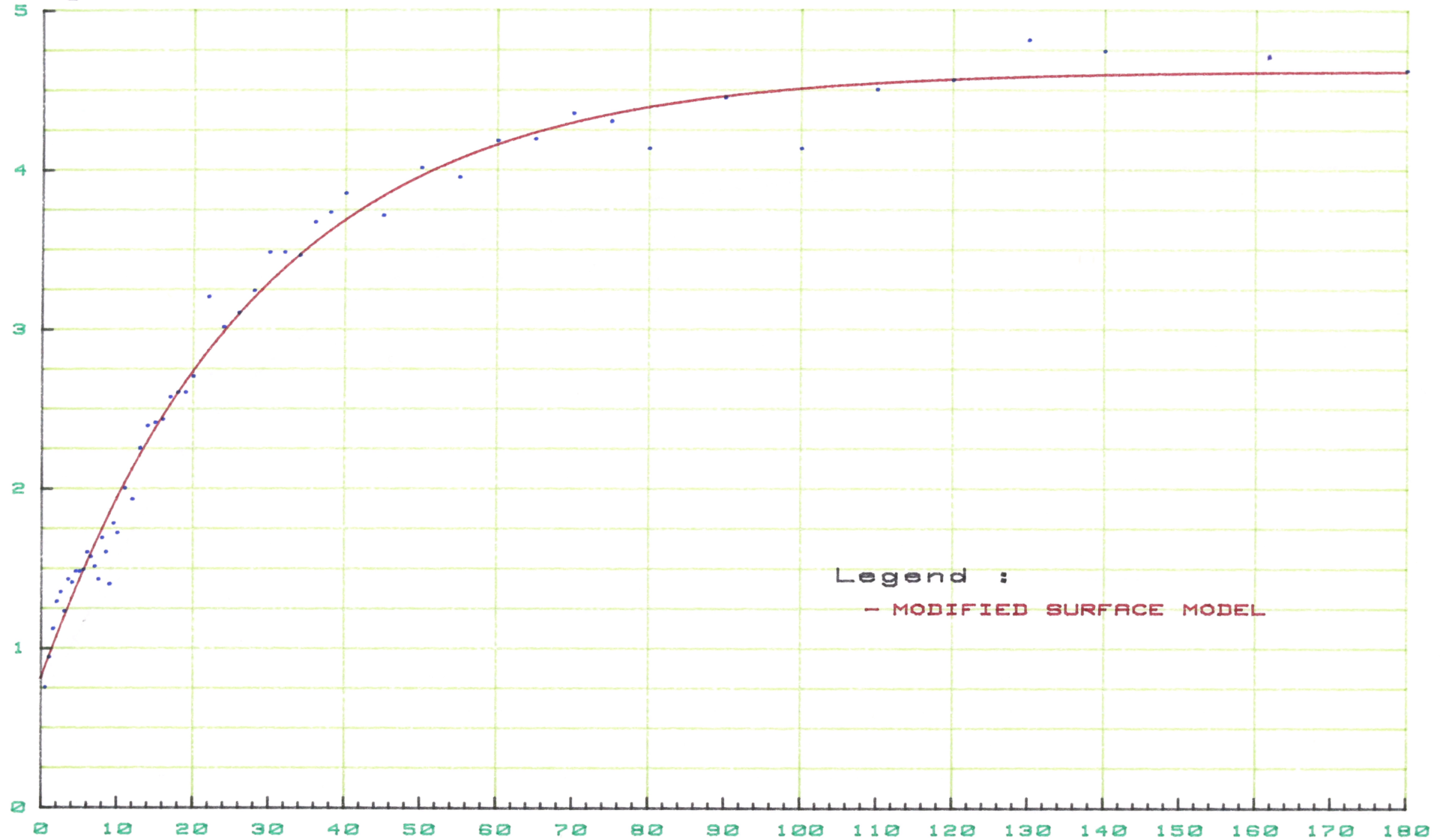
TESTING DATE : 9/10/84

DIRTY WATER (MLSS) AERATION TEST

TEST No. : 1

PROBE No. : 3

C (mg/L)



Program : SGE3R

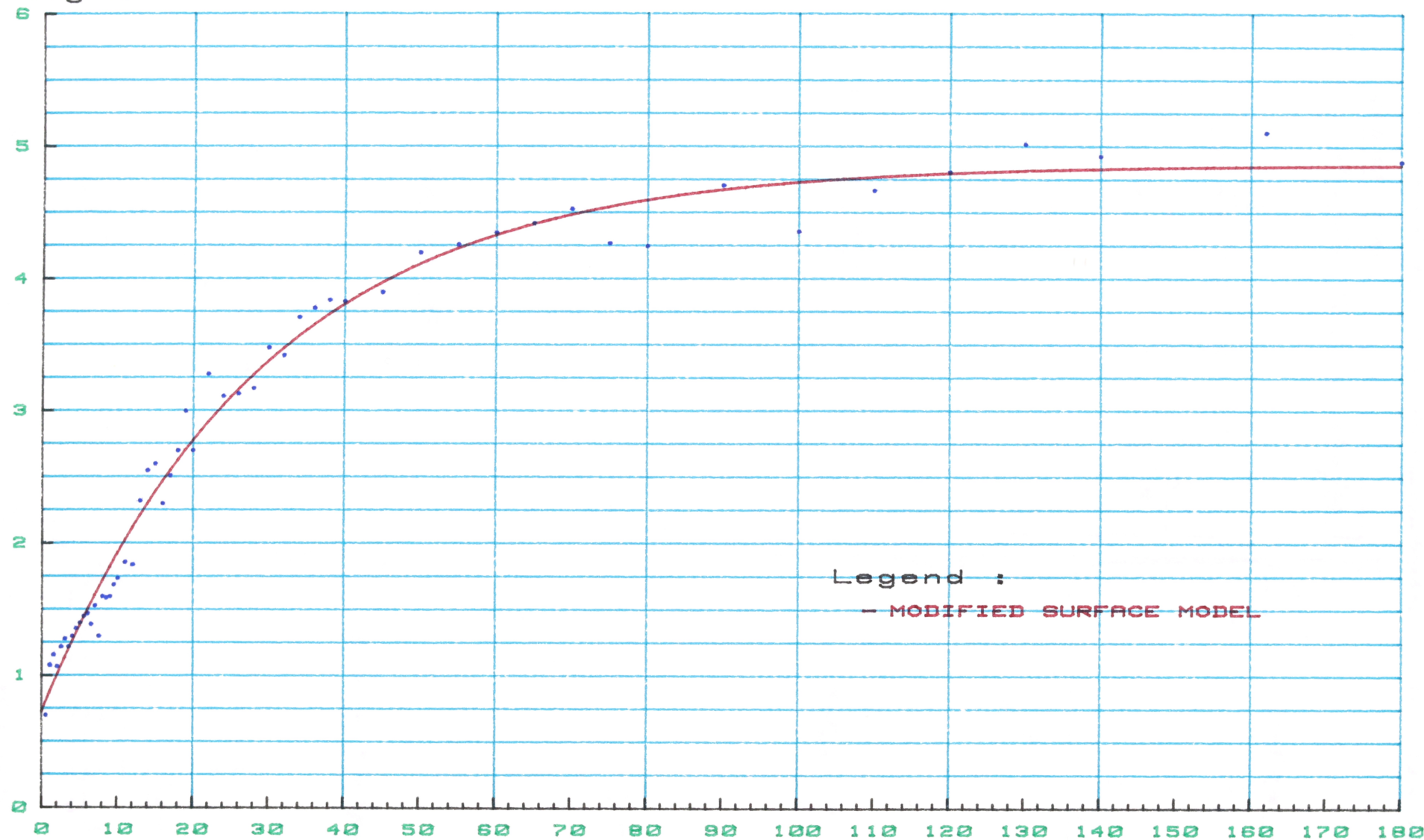
Sgs Br, PWD, NSW.

DIRTY WATER (MLSS) AERATION TEST

TEST No. : 1

PROBE No. : 4

C (mg/L)

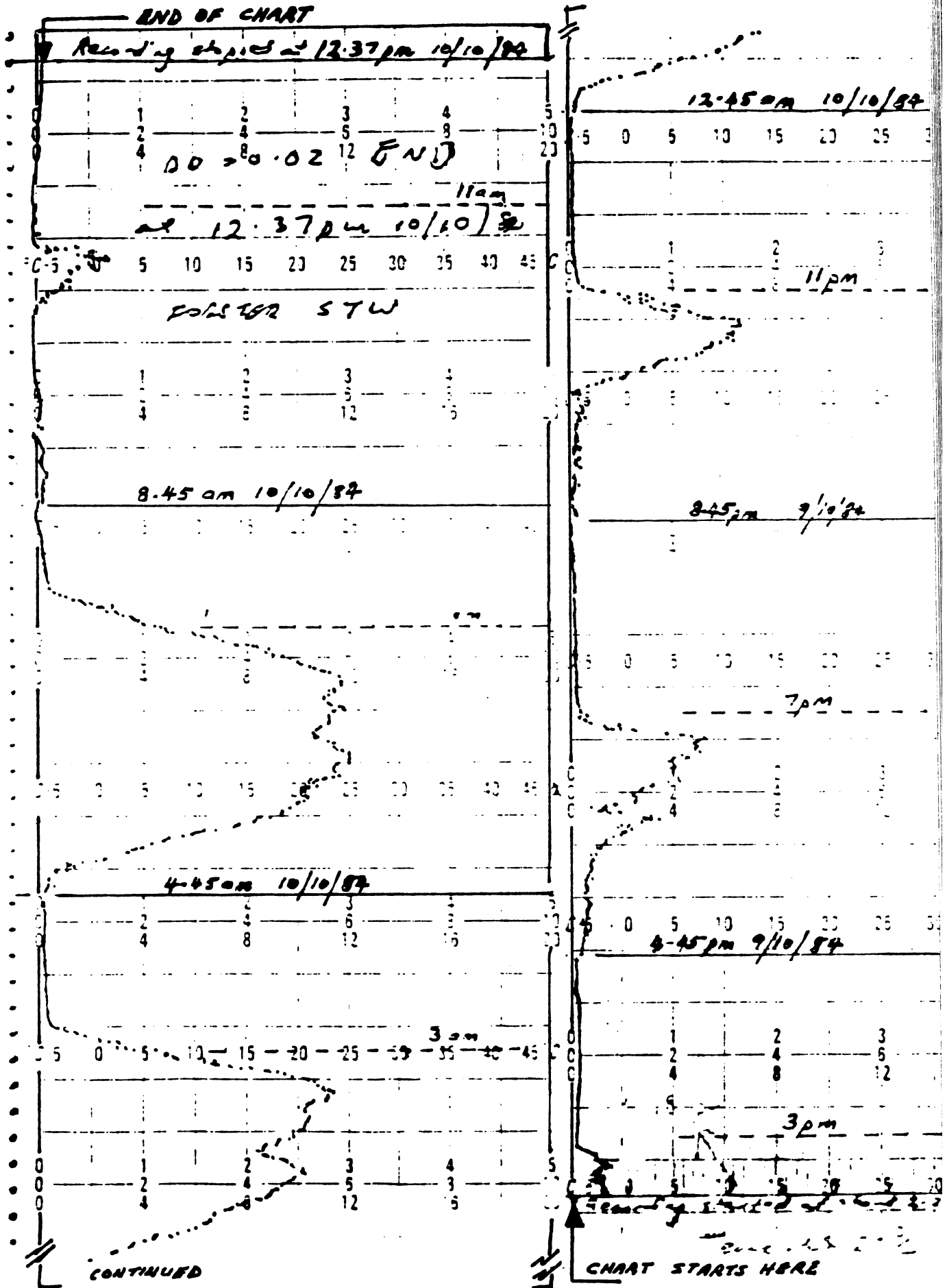


Program : SGE3R

Sgs Br, PWD, NSW.

- 203 -
APPENDIX H

24 h Chart Recording of MLSS DO Concentration in No. 1 IFA Tank.



APPENDIX I

1	SSSSSSSS	UUU	UUU	SSSSSSSS	SSSSSSSS	AAA	TTTTTTTTTTTT
2	SSSSSSSS	UUU	UUU	SSSSSSSS	SSSSSSSS	AAA	TTTTTTTTTTTT
3	SSSSSSSS	UUU	UUU	SSSSSSSS	SSSSSSSS	AAA	TTTTTTTTTTTT
4	SSS	SSS	UUU	SSS	SSS	AAA	TTT
5	SSS	SSS	UUU	SSS	SSS	AAA	TTT
6	SSS	SSS	UUU	SSS	SSS	AAA	TTT
7	SSS	UUU	UUU	SSS	SSS	AAA	TTT
8	SSS	UUU	UUU	SSS	SSS	AAA	TTT
9	SSS	UUU	UUU	SSS	SSS	AAA	TTT
10	SSSSSSSS	UUU	UUU	SSSSSSSS	SSSSSSSS	AAA	TTT
11	SSSSSSSS	UUU	UUU	SSSSSSSS	SSSSSSSS	AAA	TTT
12	SSSSSSSS	UUU	UUU	SSSSSSSS	SSSSSSSS	AAA	TTT
13	SSS	UUU	UUU	SSS	SSS	AAAAA	TTT
14	SSS	UUU	UUU	SSS	SSS	AAAAA	TTT
15	SSS	UUU	UUU	SSS	SSS	AAAAA	TTT
16	SSS	UUU	UUU	SSS	SSS	AAA	TTT
17	SSS	UUU	UUU	SSS	SSS	AAA	TTT
18	SSS	UUU	UUU	SSS	SSS	AAA	TTT
19	SSSSSSSS	UUUUUUUU	SSSSSSSS	SSSSSSSS	SSSSSSSS	AAA	TTT
20	SSSSSSSS	UUUUUUUU	SSSSSSSS	SSSSSSSS	SSSSSSSS	AAA	TTT
21	SSSSSSSS	UUUUUUUU	SSSSSSSS	SSSSSSSS	SSSSSSSS	AAA	TTT
22							
23							
24							
25	BBBBBBBB	AAA	SSSSSSSS				
26	BBBBBBBB	AAA	SSSSSSSS				
27	BBBBBBBB	AAA	SSSSSSSS				
28	BBB	BBB	AAA	SSS	SSS		
29	BBB	BBB	AAA	SSS	SSS		
30	BBB	BBB	AAA	SSS	SSS		
31	BBB	BBB	AAA	SSS	SSS		
32	BBB	BBB	AAA	SSS	SSS		
33	BBB	BBB	AAA	SSS	SSS		
34	BBBBBBBB	AAA	SSSSSSSS				
35	BBBBBBBB	AAA	SSSSSSSS				
36	BBBBBBBB	AAA	SSSSSSSS				
37	BBB	BBB	AAAAA	SSS	SSS		
38	BBB	BBB	AAAAA	SSS	SSS		
39	BBB	BBB	AAAAA	SSS	SSS		
40	BBB	BBB	AAA	SSS	SSS		
41	BBB	BBB	AAA	SSS	SSS		
42	BBB	BBB	AAA	SSS	SSS		
43	BBBBBBBB	AAA	SSSSSSSS				
44	BBBBBBBB	AAA	SSSSSSSS				
45	BBBBBBBB	AAA	SSSSSSSS				

LPTSPL Version 3.0(104) running on PRX under AMOS/L Version 1.1A(68)

File DSK1: SUSSAT.BAS[150,1] printed on Wednesday, January 9, 1985 02:46:52 PM
Copy 1 of 2 Forms: NORMAL

File DBK1:SUSSAT.BAS[150,1] printed on 09-Jan-85 14:46:52

Page 1

```

1 REM " ***** THIS PROGRAM WAS WRITTEN FOR THE OPERATION SECTION *****
2 REM " ***** SEWERAGE BRANCH, PUBLIC WORKS DEPARTMENT, NEW SOUTH WALES. *****
3 REM " *****
4 REM " ***** PROGRAMMED BY : H.G. LOI *****
5 REM " ***** PROGRAM NAME : SUSSAT.BAS *****
6 REM " *****
7 REM " ***** THIS PROGRAM IS BASED ON THE NON LINEAR EXPONENTIAL CURVE TO ANALYSE"
8 REM " ***** THE OXYGEN TRANSFER IN TWO CASES : SURFACE & SUBMERGED AERATIONS *****
9 REM " *****
10
11
12 2: MAP1 INAME, S, 10
13 MAP1 AA, S, 64
14 MAP1 FMT2, S, 80, " ** ***** ** ***** ** ***** ** ***** ** *****"
15 MAP1 FMT4, S, 80, " ** ***** ** ***** ** ***** ** ***** ** *****"
16 MAP1 FMT5, S, 80, " *** ** ** ** ** ***** "
17 MAP1 FMT6, S, 80, " ** ***** ** ***** ** ***** ** ***** "
18
19 3: STR$IZ 80
20 10: DIM T(100), C(100), E(5), C1(100, 5), DEL(100, 5), TR(5, 4), TR1(5, 4), T1(100)
21 12: SIGNIFICANCE 7
22 31: AA="*****"
23 55: SSM$=TIME
24 L=LEN(SSM$)
25 OUT$="US0000.LST"
26 IF L>4 OUT$(3, 4)=SSM$(L-3, 4) ELSE OUT$(7-L, L)=SSM$
27 REM " ***** OPEN THE OUTPUT FILE *****"
28 OPEN #100, OUT$, OUTPUT
29 PRINT "ENTER THE DATA FILE NAME : " : INPUT INAME
30 REM " ***** OPEN THE DATAFILE ***** "
31 OPEN #200, INAME, INPUT
32 INPUT LINE #200, NAME$
33 INPUT LINE #200, DET$
34 INPUT LINE #200, T1$
35 INPUT LINE #200, DAT$
36 INPUT #200, TS, PB
37 INPUT #200, TO
38 INPUT #200, N
39 FOR I=1 TO N
40 INPUT #200, T(I), C(I)
41 NEXT I
42 REM " ***** THE SURFACE OR SUBMERGED AERATION SELECTION ***** "
43 REM " ***** IF A1$="Y" THEN SURFACE AERATION, OTHERWISE SUBMERGED AERATION ***** "
44 INPUT LINE #200, A1$
45 IF A1$="Y" THEN NUM=4 : GOTO 90
46 INPUT #200, NUM
47 FOR I=1 TO NUM
48 REM " ***** THE TRIAL COEFFICIENTS ***** "
49 REM " ***** C=TR(I, 1) : Co=TR(I, 2) : Cs=TR(I, 3) : K1a=TR(I, 4) ***** "
50 INPUT #200, TR(I, 1), TR(I, 2), TR(I, 3), TR(I, 4)
51 NEXT I
52 GOTO 91
53 90: FOR I=1 TO 4
54 INPUT #200, TR(I, 1), TR(I, 2), TR(I, 4)
55 NEXT I
56 REM " ***** THE TIME ADJUSTMENT TO ***** "

```

File D8K1:SUBSAT.BAS[150,1] printed on 09-Jan-85 14:46:58

Page 2

```

1  91 FOR J=1 TO N
2    T1(J)=T(J)-T0
3    IF T1(J)<0 THEN 92
4    GOTO 94
5    92 T1(J)=0
6    94 NEXT J
7  REM " ***** COPY THE ORIGINAL TRIAL SET COEFFICIENTS ***** "
8  680:FOR I=1 TO NUM
9    FOR J=1 TO 4 : TR1(I,J)=TR(I,J) : NEXT J
10   NEXT I
11  REM " ***** CALCULATE THE C vs T FOR EACH SET ***** "
12  690:FOR I=1 TO NUM
13    L=I : E=0 : GOSUB 5000 : E(I)=E
14    NEXT I
15  REM " ***** SORTING & REJECT THE SET WITH THE LARGEST LEAST SQUARE ***** "
16  X=0 : Z=0
17  800:PRINT "INPUT THE FINISH CRITERIA F : " : INPUT F
18  PRINT "INPUT THE NO. OF ITERATION M : " : INPUT M
19  Z=Z+M
20  850:X=X+1
21  IF X<Z THEN 880
22  GOTO 1340
23  880 GOSUB 7000
24  PRINT "THE REJECTED SET NO IS: ",K,TAB(50),"(",X,")"
25  REM " ***** CREATE A NEW SET BASED ON GAMA G=1.3 ***** "
26  GOSUB 6000
27  1120:G=1.3 : W=0
28  1125:W=W+1
29  TR(K,1)=G*(A-TR(K,1))+A
30  TR(K,2)=G*(B-TR(K,2))+B
31  TR(K,4)=G*(S-TR(K,4))+S
32  IF TR(K,3)=0 THEN 1180
33  TR(K,3)=G*(D-TR(K,3))+D
34  1180:E=0 : L=K : GOSUB 5000
35  REM " ***** CHECK E & E(K) ***** "
36  IF E<E(K) THEN 1315
37  G=Q/2 : IF W>15 THEN 1340
38  PRINT " *GAMA HALVING NO : ",W : GOTO 1125
39  1315 E(K)=E : IF E(K)<F THEN 1340
40  GOTO 850
41  1340:PRINT "AFTER THE ITERATION NO : ",X
42  REM " ***** LIST OF THE TRIAL SET COEFFICIENTS AFTER X ITERATIONS ***** "
43  PRINT "THE FINAL SET DATA AS FOLLOWS : "
44  PRINT "      Cs      Co      Cso      K1a      E      "
45  FOR I=1 TO NUM
46    PRINT USING FMT2,TR(I,1),TR(I,2),TR(I,3),TR(I,4),E(I)
47  NEXT I
48  PRINT
49  PRINT "IS THE RESULT SATISFIED ? Y OR N "
50  INPUT A2$ : IF A2$="Y" THEN 1684
51  GOTO 800
52  1684 K=1
53  FOR I=2 TO NUM
54
55
56
57
58
59
60

```



```

1      IF E(K)<E(I) THEN 1685
2      K=I
3      1685 NEXT I
4      E=0 : L=K : GOSUB 5000
5      R=0 : R1=0
6      FOR I=1 TO N
7      DEL(I,K)=C(I)-C1(I,K)
8      R1=R1+DEL(I,K) : R=R+ABS(DEL(I,K))
9      NEXT I
10     REM " ***** WHEN C=0 THEN FIND T & Cso ***** "
11     IF A1$="Y" GOTO 1719
12     PRINT "THE FOLLOWING IS THE CALCULATION FOR T @ C=0 BY USING TRIAL METHOD "
13     1717: INPUT "INPUT TL = ", TL
14     H=(TR(K,1)-TR(K,2))*EXP(-TR(K,4)*TL)
15     V=TL*TR(K,4)*(TR(K,1)-TR(K,3))*EXP(-TR(K,4)*TL) : TRI=TR(K,1)-H-V
16     PRINT "TRIAL C = ", TRI
17     PRINT "IS GOOD ENOUGH ? Y OR N "
18     INPUT A2$ : IF A2$="Y" : C80=TR(K,1)-(TR(K,1)-TR(K,3))*EXP(-TR(K,4)*TL) : GOTO 1719
19     1718: GOTO 1717
20     1719: PRINT "DO YOU WANT TO HAVE A PRINT ? Y OR N "
21     INPUT A2$ : IF A2$="Y" THEN 1736
22     GOTO 2061
23     1736: IF A1$="Y" THEN 1742
24     Z1$="SUBMERGED" : GOTO 1744
25     1742: Z1$="SURFACE"
26     REM " ***** LIST OF THE OUTPUT ***** "
27     1744: PRINT #100,
28     PRINT #100,
29     PRINT #100, AA; AA,
30     PRINT #100,
31     PRINT #100, TAB(15); "DONE BY : ", NAME$, TAB(48); "ON : ", DET$, TAB(71); "PROGRAM NAME : SUSSAT.BAS, DATAFILE NAME : ", INAME
32     PRINT #100, TAB(15); "SEWERAGE BRANCH ( T11 ), PUBLIC WORKS DEPARTMENT, NEW SOUTH WALES. "
33     PRINT #100,
34     PRINT #100, AA; AA
35     PRINT #100,
36     PRINT #100,
37     PRINT #100,
38     PRINT #100,
39     PRINT #100,
40     PRINT #100, TAB(15); "THIS IS THE ", Z1$, " AERATION TEST OF ", T1$
41     PRINT #100,
42     PRINT #100, TAB(15); "TESTING DATE : ", DAT$
43     PRINT #100,
44     PRINT #100, TAB(15); "TEST NO : ", TS, TAB(35); "PROBE NO : ", PB
45     PRINT #100,
46     PRINT #100, TAB(15); "THE INITIAL TRIAL SET COEFFICIENTS AS FOLLOWS : "
47     PRINT #100,
48     PRINT #100, "          Cs          Co          Cso          KLa          "
49     PRINT #100, "          (mg/l)      (mg/l)      (mg/l)      (1/min)      "
50     PRINT #100,
51     PRINT #100,
52     FOR J=1 TO NUM
53     PRINT #100, USING FMT6, TR1(J,1), TR1(J,2), TR1(J,3), TR1(J,4)
54     NEXT J
55     PRINT #100,

```

```
1      PRINT
2      PRINT "IF YOU WANT TO HAVE ANOTHER RUN, PLEASE CREATE ANOTHER DATA FILE"
3      PRINT "AND THEN ENTER THE NEW FILE NAME AFTER TYPE < RUN USSAT (RETURN) >"
4      PRINT
5      PRINT AA
6      PRINT
7      PRINT
8      PRINT " HAVE A GOOD TIME MATE, BYE NOW..... "
9      PRINT
10     PRINT
11     END
12
13     .5000 FOR J=1 TO N
14         IF TR(L,3)=0 THEN 5010
15         H=(TR(L,1)-TR(L,2))*EXP(-T1(J)*TR(L,4))
16         V=T1(J)*TR(L,4)*(TR(L,1)-TR(L,3))*EXP(-T1(J)*TR(L,4))
17         C1(J,L)=TR(L,1)-H-V : GOTO 5020
18     5010 C1(J,L)=TR(L,1)-(TR(L,1)-TR(L,2))*EXP(-TR(L,4)*T1(J))
19     5020 E=E+(C(J)-C1(J,L))^2
20     NEXT J
21     RETURN
22
23     6000 A=0 : B=0 : S=0 : D=0
24     FOR I=1 TO NUM
25         IF I=K THEN 6010
26         A=A+TR(I,1) : B=B+TR(I,2) : S=S+TR(I,4)
27         IF TR(I,3)=0 THEN 6010
28         D=D+TR(I,3)
29     6010 NEXT I
30     A=A/(NUM-1) : B=B/(NUM-1) : S=S/(NUM-1) : D=D/(NUM-1)
31     RETURN
32
33     7000 K=1
34     FOR J=2 TO NUM
35         IF E(K)<E(J) THEN 7010
36         GOTO 7020
37     7010 K=J
38     7020 NEXT J
39     RETURN
```

APPENDIX J

Transient Response of a First Order System

The response or output variable y of a system to a forcing or input variable x is described generally by the equation:-

$$\frac{Y(s)}{X(s)} = G(s)$$

Where $Y(s)$ = Laplace transform of the output deviation variable Y ,

$X(s)$ = Laplace transform of the input deviation variable X ,

$G(s)$ = Laplace transform of the transfer function which relates the variables Y and X

s = Laplace transform variable. The Laplace transform merely changes Y and X from being a function of t (time) to a function of s by:-

$$f(s) = \int_0^{\infty} f(t) e^{-st} dt$$

The deviation variables Y and X are defined by:-

$$Y = y - y_s \quad (2)$$

$$X = x - x_s \quad (3)$$

Where the subscript s indicates the steady state values at $t = 0$ (ie. before the transient response occurs). The introduction of the deviation variables results in a transfer function that is free of initial conditions as the initial values of X and Y are zero.

By definition (ref. 7) the transfer function of a first order system is given by the following equation:-

APPENDIX J (Cont'd)

$$G(s) = \frac{1}{\tau s + 1} \quad (4)$$

where τ = system's time constant.

It will be shown by the following derivation that the transfer function of the unsteady - state aeration test is the same as that of a first order system defined by Eq. 4.

Unsteady - state Aeration Test

The change in the DO concentration, C , during reaeration can be regarded as the output variable or response to the input variable C^* with the system time constant $\tau = K^{-1}$.

The equation describing the aeration test is given by:-

$$\frac{dc}{dt} = K(C^* - C) \quad (5)$$

Substituting τ^{-1} for K , x for C^* and y for C into Eq. 5 gives

$$\tau \frac{dy}{dt} = (x - y) \quad (6)$$

At steady state, $\frac{dy}{dt} = 0$, and Eq. 6 yields

$$x_s = y_s = C_o \quad (7)$$

Introducing the deviation variables given by Eq. 2 and 3 into Eq. 6 gives:

```

1      PRINT #100,TAB(15),"THE FINAL SET COEFFICIENTS AS FOLLOWS : "
2
3      PRINT #100,
4      PRINT #100,"          Cs          Co          Cso          K1a          E "
5      PRINT #100,"          (mg/l)      (mg/l)      (mg/l)      (1/min)      (mg/l)      "
6      PRINT #100,
7      FOR I=1 TO NUM
8      PRINT #100,USING FMT4,TR(I,1),TR(I,2),TR(I,3),TR(I,4),E(I)
9      NEXT I
10     PRINT #100,
11     PRINT #100,TAB(15),"THE BEST FIT SET IS SET NUMBER : ",K
12     PRINT #100,
13     SEE=SQR(E(K)/N)
14     PRINT #100,TAB(15),"THE STANDARD ERROR OF ESTIMATION IS : ",SEE
15     PRINT #100,
16     PRINT #100,TAB(15),"THE TEST AND PREDICTED RESULTS : "
17     PRINT #100,
18     PRINT #100,"          T          C(TEST)      C(PRE'D)      RES'L      "
19     PRINT #100,"          (min)      (mg/l)        (mg/l)        (mg/l)      "
20     PRINT #100,
21     FOR I=1 TO N
22     PRINT #100,USING FMT5,I,T(I),C(I),C1(I,K),DEL(I,K)
23     NEXT I
24     PRINT #100,
25     IF A1$="Y" GOTO 1900
26     PRINT #100,TAB(15)," WHEN   T (min) = ",TL
27     PRINT #100,
28     PRINT #100,TAB(15)," THEN   C = 0,  &   Cso (mg/l) = ",C50
29     PRINT #100,
30     1900:PRINT #100,
31     PRINT #100,TAB(15),"THE LEAST SQUARE E = ",E
32     PRINT #100,
33     PRINT #100,TAB(15),"THE RESIDUAL ANALYSIS : "
34     PRINT #100,
35     PRINT #100,"          RESIDUAL      RES'L(ABS)      "
36     PRINT #100,
37     PRINT #100,"          SUM          ",R1,TAB(40),R
38     PRINT #100,
39     R1=R1/N : R=R/N
40     PRINT #100,"          MEAN          ",R1,TAB(40),R
41     PRINT #100,
42     PRINT #100,
43     PRINT #100,
44     PRINT #100,
45     PRINT #100,
46     PRINT #100,"-----END-----"
47     PRINT #100,
48     PRINT #100,
49     PRINT #100,
50     2061:CLOSE #100,
51     XCALL SPOOL,OUT$,"PRX",101
52     CLOSE #200,
53     PRINT
54     PRINT
55     PRINT AA

```

APPENDIX J (Cont'd)

$$\tau \frac{dY}{dt} = (X - Y) \quad (8)$$

$$\left(\frac{d(y - y_s)}{dt} \right) = \frac{dY}{dt} = \frac{dy}{dt} \text{ as } y_s \text{ is constant}$$

$$\tau \frac{dY}{dt} + Y = X \quad (9)$$

Taking the Laplace transform of Eq. 9 gives

$$\tau s Y(s) + Y(s) = X(s) \quad (10)$$

$$\frac{Y(s)}{X(s)} = \frac{1}{\tau s + 1} \quad (11)$$

Thus, the transfer function given by Eq. 11 for the aeration test is the same as Eq. 4 for a first order system.

Surface Aeration Test

In the surface aeration test, C^* is constant. Hence the input is a step function of magnitude C^* .

$$\begin{aligned} x &= C^* \\ xs &= C_o \text{ (from Eq. 7)} \\ X &= C^* - C_o \end{aligned} \quad (12)$$

The Laplace transform of Eq. (12) is

$$\begin{aligned} X(s) &= \frac{(C^* - C_o)}{s} \\ Y(s) &= X(s) \cdot G(s) = \frac{(C^* - C_o)}{s} \cdot \frac{1}{(\tau s + 1)} \end{aligned} \quad (13)$$

APPENDIX J (Cont'd)

The partial fraction of Eq. 13 is:

$$Y(s) = \frac{(C^* - C_0)}{s(\tau s + 1)} = \frac{A}{s} + \frac{B}{(\tau s + 1)} \quad (14)$$

$$\therefore \frac{(C^* - C_0)}{s(\tau s + 1)} = \frac{A\tau s + A + Bs}{s(\tau s + 1)} \quad (15)$$

$$\therefore A = (C^* - C_0) \quad (16)$$

$$B = -\tau A = -\tau(C^* - C_0) \quad (17)$$

Substituting Eq. 16 and 17 for A and B into Eq. 14 gives

$$Y(s) = \frac{(C^* - C_0)}{s} - \frac{\tau(C^* - C_0)}{(\tau s + 1)} \quad (18)$$

$$\therefore Y(s) = \frac{(C^* - C_0)}{s} - \frac{(C^* - C_0)}{(s + 1/\tau)} \quad (19)$$

From Table 2.1 in ref. 6,

$$\frac{(C^* - C_0)}{s} = \text{Laplace transform of } (C^* - C_0),$$

and $\frac{(C^* - C_0)}{(s + 1/\tau)} = \text{Laplace transform of } (C^* - C_0)e^{-t/\tau}$

Thus the inverse of Eq. 19 is

$$Y = y - y_s = (C^* - C_0) - (C^* - C_0)e^{-t/\tau} \quad (20)$$

As $y_s = C_0$ (from Eq. 7) and $\tau^{-1} = k$, Eq. (20) gives

$$C = y = C^* - (C^* - C_0)e^{-kt} \quad (21)$$

APPENDIX J (Cont'd)

The surface aeration test can thus be represented graphically by Fig. 1.

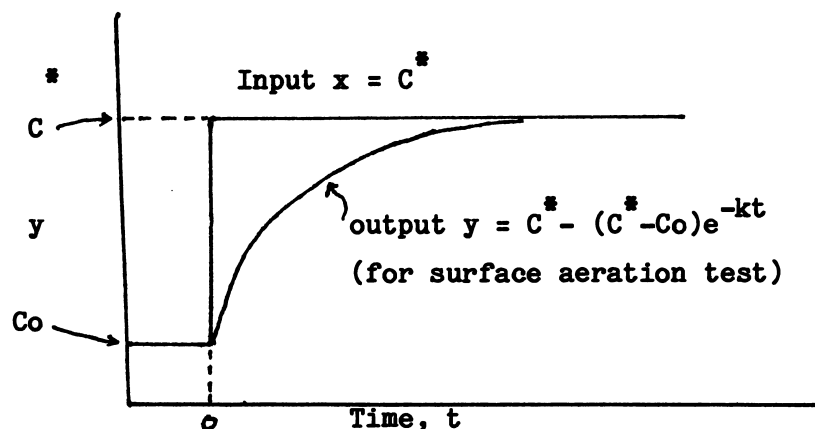


Fig. 1 Transient response of a First Order system to a step input function C^*

Submerged Aeration Test

In submerged aeration, C^* is not constant but varies exponentially as given by the following equation:-

$$C^* = C_{\infty}^* - (C_{\infty}^* - C_o^*)e^{-kt} \quad (22)$$

$$\therefore x = C^* = C_{\infty}^* - (C_{\infty}^* - C_o^*)e^{-t/\tau} \quad (23)$$

$$x_s = C_o \quad (\text{from Eq. 7})$$

$$\text{and} \quad X = (C_{\infty}^* - C_o) - (C_{\infty}^* - C_o^*)e^{-t/\tau} \quad (24)$$

$$\therefore X(s) = \frac{(C_{\infty}^* - C_o)}{s} - \frac{(C_{\infty}^* - C_o^*)}{s + 1/\tau} \quad (25)$$

$$\therefore X(s) = \frac{(C_{\infty}^* - C_o)}{s} - \frac{\tau(C_{\infty}^* - C_o^*)}{\tau s + 1} \quad (26)$$

APPENDIX J (Cont'd)

$$\therefore Y(s) = X(s) \cdot G(s) = \left\{ \frac{(C_{oo}^* - C_o)}{s} - \frac{\tau(C_{oo}^* - C_o)}{(\tau s + 1)} \right\} \frac{1}{(\tau s + 1)} \quad (27)$$

$$\therefore Y(s) = \frac{(C_{oo}^* - C_o)}{s(\tau s + 1)} - \frac{\tau(C_{oo}^* - C_o)}{(\tau s + 1)^2} \quad (28)$$

$$\therefore Y(s) = \frac{(C_{oo}^* - C_o)(\tau s + 1) - \tau(C_{oo}^* - C_o)s}{s(\tau s + 1)^2} \quad (29)$$

The partial fraction of Eq. 29 is

$$\frac{C_{oo}^* \tau s - C_o \tau s + C_{oo}^* - C_o - \tau C_{oo}^* s + \tau C_o s}{s(\tau s + 1)^2} = \frac{D}{s} + \frac{E}{(\tau s + 1)} + \frac{F}{(\tau s + 1)^2} \quad (30)$$

$$\therefore \frac{(C_{oo}^* - C_o) + (\tau C_o - C_o)s}{s(\tau s + 1)^2} = \frac{D(\tau s + 1)^2 + Es(\tau s + 1) + Fs}{s(\tau s + 1)^2} \quad (31)$$

$$\begin{aligned} \therefore (C_{oo}^* - C_o) + (\tau C_o - C_o)s &= D(\tau s)^2 + 2D\tau s + D + E\tau s^2 + Es + Fs \\ &= D\tau^2 s^2 + E\tau s^2 + 2D\tau s + Es + Fs + D \\ &= (D\tau^2 + E\tau)s^2 + (2D\tau + E + F)s + D \end{aligned} \quad (32)$$

$$\therefore D = (C_{oo}^* - C_o) \quad (33)$$

$$\therefore D\tau^2 + E\tau = 0 \quad (34)$$

$$\therefore E = -D\tau = -\tau(C_{oo}^* - C_o) \quad (35)$$

$$\therefore 2D\tau + E + F = \tau(C_o - C_o) \quad (36)$$

$$\therefore F = \tau(C_o - C_o) - 2D\tau + E$$

$$\therefore F = \tau(C_o - C_o) - D\tau$$

APPENDIX J (Cont'd)

$$\dots F = \tau(C_o^* - C_o) - \tau(C_{oo}^* - C_o)$$

$$\dots F = -\tau(C_{oo}^* - C_o) \quad (37)$$

Substituting Eq. 33, 35 and 37 for D, E and F into Eq. 30 gives:

$$Y(s) = \frac{(C_{oo}^* - C_o)}{s} - \frac{\tau(C_{oo}^* - C_o)}{(\tau s + 1)} - \frac{\tau(C_{oo}^* - C_o)}{(\tau s + 1)^2} \quad (38)$$

$$Y(s) = \frac{(C_{oo}^* - C_o)}{s} - \frac{(C_{oo}^* - C_o)}{(s + 1/\tau)} - \frac{1/\tau(C_{oo}^* - C_o)}{(s + 1/\tau)^2} \quad (39)$$

From Table 2.1 ref. 3, it can be seen that:

$$\frac{(C_{oo}^* - C_o)}{s} = \text{Laplace transform of } (C_{oo}^* - C_o)$$

$$\frac{(C_{oo}^* - C_o)}{(s + 1/\tau)} = \text{Laplace transform } (C_{oo}^* - C_o)e^{-t/\tau}, \text{ and}$$

$$\frac{1/\tau(C_{oo}^* - C_o)}{(s + 1/\tau)^2} = \text{Laplace transform of } 1/\tau(C_{oo}^* - C_o)t.e^{-t/\tau}$$

Thus, the inverse of the Laplace transform of Eq. (39) is:

$$Y = y - y_s = (C_{oo}^* - C_o) - (C_{oo}^* - C_o)e^{-t/\tau} - 1/\tau(C_{oo}^* - C_o)t.e^{-t/\tau}$$

$$\dots y = C_{oo}^* - (C_{oo}^* - C_o)e^{-kt} - K(C_{oo}^* - C_o)t.e^{-kt} \quad (40)$$

APPENDIX J (Cont'd)

The submerged aeration test can thus be represented graphically by Fig. 2.

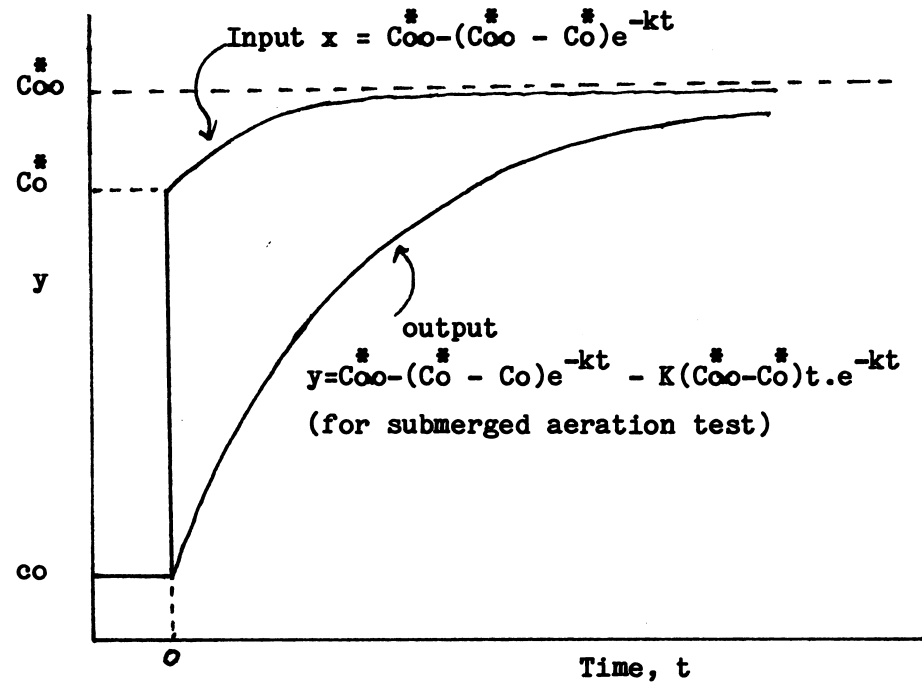


Fig. 2 Transient response of a First Order System to a combined step and exponential function.