Canada Mortgage
and Housing Corporation

Société canadienne
d'hypothèques et de logement

Environment Canada

Environnement Canada

Pilot Plant Studies of Rotating Biological Contactors Treating Municipal Wastewater

Sewage Collection and Treatment
Report SCAT - 2

NHA 5362 80/07

SCAT RESEARCH REPORTS

These RESEARCH REPORTS describe the results of research and development projects in the municipal and domestic wastewater collection, treatment and disposal fields. The projects are identified and developed through the Interdepartmental Committee on Sewage Collection and Treatment (SCAT Research Committee) with the technical assistance of Environment Canada.

Enquiries pertaining to the Wastewater Technology Research Program should be directed to:

Technical Research Division
Policy Development and Research Sector
Canada Mortgage and Housing Corporation
Montreal Road
Ottawa, Ontario K1A 0P7

PILOT PLANT STUDIES OF ROTATING BIOLOGICAL CONTACTORS TREATING MUNICIPAL WASTEWATER

by

K.L. Murphy and R.W. Wilson
International Environmental Consultants Ltd.

for

Canada Mortgage and Housing Corporation

Report SCAT-2
July 1980

This document may be obtained from

Information and Communications Centre
Canada Mortgage and Housing Corporation
Ottawa, Ontario
K1A 0P7

Publications Section (Water)
Environmental Protection Service
Environment Canada
Ottawa, Ontario
K1A 1C8

REVIEW NOTICE

This project was carried out under contract for the Policy Development and Research Sector, Canada Mortgage and Housing Corporation, through a grant provided under Part V of the National Housing Act. Technical guidance and assistance was provided by Dr. B.E. Jank, Environment Canada.

This report has been reviewed by Canada Mortgage and Housing Corporation and Environment Canada. The views, conclusions and recommendations expressed herein are those of the authors and do not necessarily reflect the views and policies of either Canada Mortgage and Housing Corporation or Environment Canada. Mention of trade names or commercial products does not constitute an endorsement or recommendation for use.

La version française de ce rapport est en préparation.

Cat. No. En 43-51/2E
ISBN 0-662-11013-7
BEAUREGARD PRESS LIMITED

ABSTRACT

This study reports the findings of a twelve-month pilot program which examined the performance of rotating biological contactors (RBC) for the treatment of municipal wastewater. During this period suitable design criteria for removal of BOD_5 and combined BOD_5 plus nitrification were examined. Phosphorus removal, the effects of diurnal and shock loadings, and considerations of process scale-up were also evaluated.

For evaluating the efficiency of the RBC systems, the best design parameters were determined to be mass loading of the total BOD_5 for BOD_5 removal, and mass loading of filtrable TKN for combined BOD_5 removal plus nitrification.

The study demonstrated that treated effluent containing filtrable BOD_5 concentrations consistently less than 10 mg/L could be produced.

RÉSUMÉ

Les auteurs présentent les résultats d'un programme d'étude à l'échelle pilote, d'une durée de douze mois, de l'efficacité des disques biologiques pour le traitement des eaux usées urbaines. Les critères de construction pour l'élimination de la DBO_5 associée ou non à la nitrification ont été examinés au cours de ce programme, et l'élimination du phosphore, les effets des charges journalières et de charges massives soudaines ainsi que les possibilités de l'application du traitement à l'échelle réelle ont également été évalués.

Il a été déterminé que les meilleurs paramètres pour l'évaluation de l'efficacité des disques biologiques sont la charge massique de la DBO_5 totale pour l'élimination de la DBO_5 et la charge massique de l'azote Kjeldahl total filtrable pour l'élimination de la DBO_5 combinée à la nitrification.

Les résultats indiquent qu'une DBO_5 filtrable constamment inférieure à 10 mg/l peut être obtenue dans les effluents traités.

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

CONCLUSIONS AND RECOMMENDATIONS

CONCLUSIONS

The rotating biological contactor (RBC) can provide effective removal of filtrable BOD_5 and TKN from municipal wastewater. The process is mechanically simple, and the active biomass attached to the discs is not adversely affected by wide variations in hydraulic flow.

Design Guidelines

Mass loading was found to be the most suitable design parameter to adopt as a basis for sizing RBC systems. For removal of BOD_5 the mass loading is expressed as the mass of BOD_5 applied per unit disc area and time (e.g. kg $BOD_5/1000\ m^2{\cdot}d$). For combined BOD_5 removal plus nitrification the loading is expressed as the mass of filtrable TKN per unit disc area and time (e.g. kg $TKN_f/1000\ m^2{\cdot}d$).

The results of this study were used to develop recommended design loading (Table 1). The selection of these loadings was based on a full scale, short hydraulic detention RBC system experiencing a peak to mean to minimum diurnal flow of 2:1:0.5. The loadings provide 25 percent greater surface area for BOD_5 removal, and 35 percent more for combined BOD_5 removal plus nitrification, than necessary for systems where full equalization of the raw wastewater flow is available. The data apply only to primary treated municipal wastewater. BOD_5 and suspended solids concentrations listed under "Design Objective" represent the average, not the maximum, anticipated effluent concentrations at the indicated loadings.

Table 1 indicates that a loading of 6.0 to 6.5 kg total $BOD_5/1000\ m^2{\cdot}d$ would be selected to meet an average treated effluent objective of 15 mg/L for both BOD_5 and suspended solids, assuming a minimum wastewater temperature of 8 to 10°C. This loading is 20 to 30 percent higher than the design guideline for RBC's established by the Ontario Ministry of the Environment, but is substantially less than those calculated from the design manuals of RBC manufacturers.

TABLE 1 DESIGN GUIDELINES FOR RBC TREATMENT OF CLARIFIED MUNICIPAL WASTEWATER

RBC System	Design Objective (average value)	Design Parameter	Design Loading (kg/1000 $m^2 \cdot d$)			
			20°C	15°C	10°C	5°C
BOD_5 Removal	$BOD_5 \leq 15$ mg/L SS $\leq$ 15 mg/L	BOD_5 load (total)	8.5*	8.5	6.5	5.0
BOD_5 Removal	$BOD_5 \leq 30$ mg/L SS $\leq$ 30 mg/L	BOD_5 load (total)	17*	17	13	10
BOD_5 Removal** Plus Nitrification	TKN < 5 mg/L	TKN load (filtrable)	1.10	0.65	0.40	0.25

* Assumes no further increase in removal rates beyond 15°C

** Based on data using wastewater with a ratio of total BOD_5 to TKN_f ~6:1

Performance under Transient Loading

The 2.0 m diameter RBC evaluated during this study had a hydraulic residence time of three hours or less depending on the loading. This is typical of many full scale systems. Under these conditions it has little capacity to equalize or absorb variable or shock loads. Significant variations in effluent quality were observed within 30 minutes of a change in influent conditions. These will occur in RBC systems subjected to normal diurnal variations in flow and BOD_5 and TKN concentration, unless additional equalization is provided or a suitable design factor is applied when calculating surface area requirements.

Suspended Solids Characteristics

This study demonstrates that treated effluent containing filtrable BOD_5 concentrations consistently less than 10 mg/L can be produced. Under these conditions the final effluent quality, measured as total suspended solids and total BOD_5, was limited by the efficiency of the final solids' separation process. Some of the settleable solids in treated RBC effluent had low settling velocities, and conservative overflow rates (e.g. $\leq$1.0 m/h) should be selected for the final clarifier in order to obtain their maximum removal.

RBC systems should include a process for primary solids removal. The concentration of non-settleable suspended solids in clarified effluent was consistently less when primary clarified wastewater, as opposed to raw degritted wastewater, was fed to the RBC.

In most cases it should be possible to achieve suspended solids' concentrations of 15 mg/L or less in the final effluent, assuming low BOD_5 loadings and proper primary clarification.

Phosphorus Removal

Virtually complete removal of filtrable P was achieved by the addition of alum or ferric chloride at the influent end of the RBC tank. Total concentrations less than 1.0 mg/L in final clarified effluent are possible assuming proper solids' separation.

The chemical addition increased sludge production in the system. This increase was greater than stoichiometric predictions, but was consistent with observed effects of chemical addition to suspended growth biological systems (Sutton et al, 1977). The presence of alum or ferric chloride did not increase the concentration of non-settleable suspended solids in the effluent, nor did it adversely effect BOD_5 or TKN removal efficiencies.

Scale-Up Considerations

A 0.5 m RBC was operated in parallel with the 2.0 m diameter unit during part of the research program in order to determine the effects of process scale-up. Under identical operating conditions the 0.5 m RBC averaged 16 percent higher mass removals of COD per unit area and time than did the 2.0 m unit. The difference in removals was statistically significant at a confidence level of 95 percent. Under combined BOD_5 removal plus nitrifying conditions, mass removals of filtrable TKN were similar for both units. This would indicate that the oxygen transfer capability of the 2.0 m unit may be limiting at high loadings.

Full sized RBC shafts are generally 3.5 m in diameter. When designing for BOD_5 removal it is recommended that a scale-up factor of 25 percent increase in surface area requirements be applied to 0.5 m RBC pilot data. A 10 percent increase in surface area is recommended for scale-up from 2.0 m RBC test data. No scale-up factor is required in systems for combined BOD_5 removal plus nitrification.

Intermittent Flow

RBC's operating under intermittent flow conditions can operate efficiently if sufficient equalization is available to maintain 10 percent of the average system loading during no flow periods. It is unclear whether the RBC can perform efficiently over the long term with no equalization of intermittent flows. As a zero flow condition enhances solids sloughing from the dics and increases the risk of system freezing in winter, some form of equalization is recommended.

RECOMMENDATIONS

1) The design loadings (Table 1) were derived from test results using only one source of wastewater. It is recommended that several field evaluations be conducted to see whether the design guidelines generally apply to most Canadian municipal wastewaters. Specifically, it is recommended that loads and effluent quality be monitored at several full-scale RBC facilities in Canada during winter, to determine whether the effluent objectives for BOD_5 and TSS can be achieved at the recommended BOD_5 loadings.
2) It was concluded that diurnal factors of 1.25 for BOD_5 removal and 1.35 for combined BOD_5 removal plus nitrification should be used when designing full scale RBC systems based on steady-state test data. This implies a need of an additional 25 and 35 percent RBC surface area, respectively, above that required for steady-state operation. An extended pilot plant, or full-scale comparison of parallell RBC units would be valuable in verifying the suitability of these diurnal loading parameters.
3) This study did not evaluate the nitrification of secondary treated wastewater with the RBC. Consequently, it was not possible to establish whether the surface area required for combined BOD_5 removal and nitrification is significantly different than that required for nitrification alone. Parallel studies of this should be undertaken.

1 INTRODUCTION

The rotating biological contactor (RBC) is a supported growth wastewater treatment process which was developed and introduced on a commercial scale in Europe in the late 1950's and early 1960's. It consists of a series of closely spaced discs anchored on a horizontal shaft that is supported above the surface of the wastewater in a semi-circular or rectangular tank. The shaft rotates exposing the biological growth on the disc surfaces alternately to the wastewater and atmosphere.

Application of RBC technology in North America began only in the late 1960's, and by 1975 there were still only a few small package plants operating in Canada.

International Environmental Consultants Limited (IEC) were retained by Canada Mortgage and Housing Corporation to plan and organize a 12-month pilot plant study (using a 2-m diameter commercial RBC unit treating municipal wastewater) to characterize the RBC's performance capabilities, and to establish general design guidelines. The study was jointly undertaken by IEC and Environmental Protection Service (EPS), Environment Canada, at the Wastewater Technology Centre of Canada Centre for Inland Waters, Burlington, Ontario. It was designed to determine:

a) the most appropriate criteria for optimum design of RBC's treating municipal wastewater,
b) the long term performance and effluent variability as measured by BOD_5 and suspended solids at various organic loadings between approximately 3 to 30 kg $BOD_5/1000\ m^2 \cdot d$,
c) the effect of diurnal and other dynamic variations in flow and concentrations on RBC performance,
d) the characteristics of the suspended solids in the RBC clarifier effluent,
e) the feasibility of using chemical addition to the RBC for phosphorus removal,
f) the factors affecting scale-up from small pilot plant RBC's to full scale design, and
g) the effects of intermittent flow on RBC performance and operation.

2 EQUIPMENT AND PROCEDURES

2.1 Study Organization

International Environmental Consultants Ltd. was responsible for the experimental design, monthly reviews and progress reports, analysis of the results and preparation of the final report. The Environmental Protection Service provided the pilot equipment, laboratory facilities and day to day engineering supervision of the program. A pilot plant operator and a laboratory technician were hired by IEC to work on-site for the duration of the study.

2.2 Pilot Plant Layout

The data in this study were obtained from the pilot plant illustrated in Figure 2-1. Coarse screened wastewater from the Burlington Skyway Water Pollution Control Plant entered a 0.76 m circular grit tank providing a hydraulic residence of 6.5 minutes. Variable speed Moyno pumps were used to feed the degritted wastewater to the units.

As many as three RBC's were operated simultaneously in some stages of the study. The main test unit was a 2.0 m diameter, four-stage RBC with 734 m^2 of media surface area. High density polyethylene discs were fixed on a horizontal shaft that rotated in a semi-circular steel tank holding 3.55 m^3 of liquid. It was located outdoors and equipped with a semi-circular double shelled steel hood, with a 5.0 cm foam insulating layer. The 2.3 m diameter primary clarifier, indicated as a dashed circle in Figure 2-1, was not used until the end of the regular test schedule. Throughout most of the program the degritted wastewater was fed directly to the 2.0 m unit, while a portion of the treated effluent went to a 1.6 m diameter final clarifier. The flow entering the clarifier was kept at a constant overflow of 0.6 m/h in order to maximize the removal of suspended solids.

Both identical 0.5 m diameter, four-stage pilot units had a total media area of 23.2 m^2, and a liquid volume capacity of 128 L. These were operated for the first two and half months, after which time only one was used. Except for a four-week period, they were kept inside the Wastewater Technology Centre. Hoods installed over each prevented excessive heat transfer. The 0.76 m primary clarifier (Figure 2-1) was used during the last three months of the pilot study, while prior to this no primary treatment was employed beyond that of coarse screening and degritting.

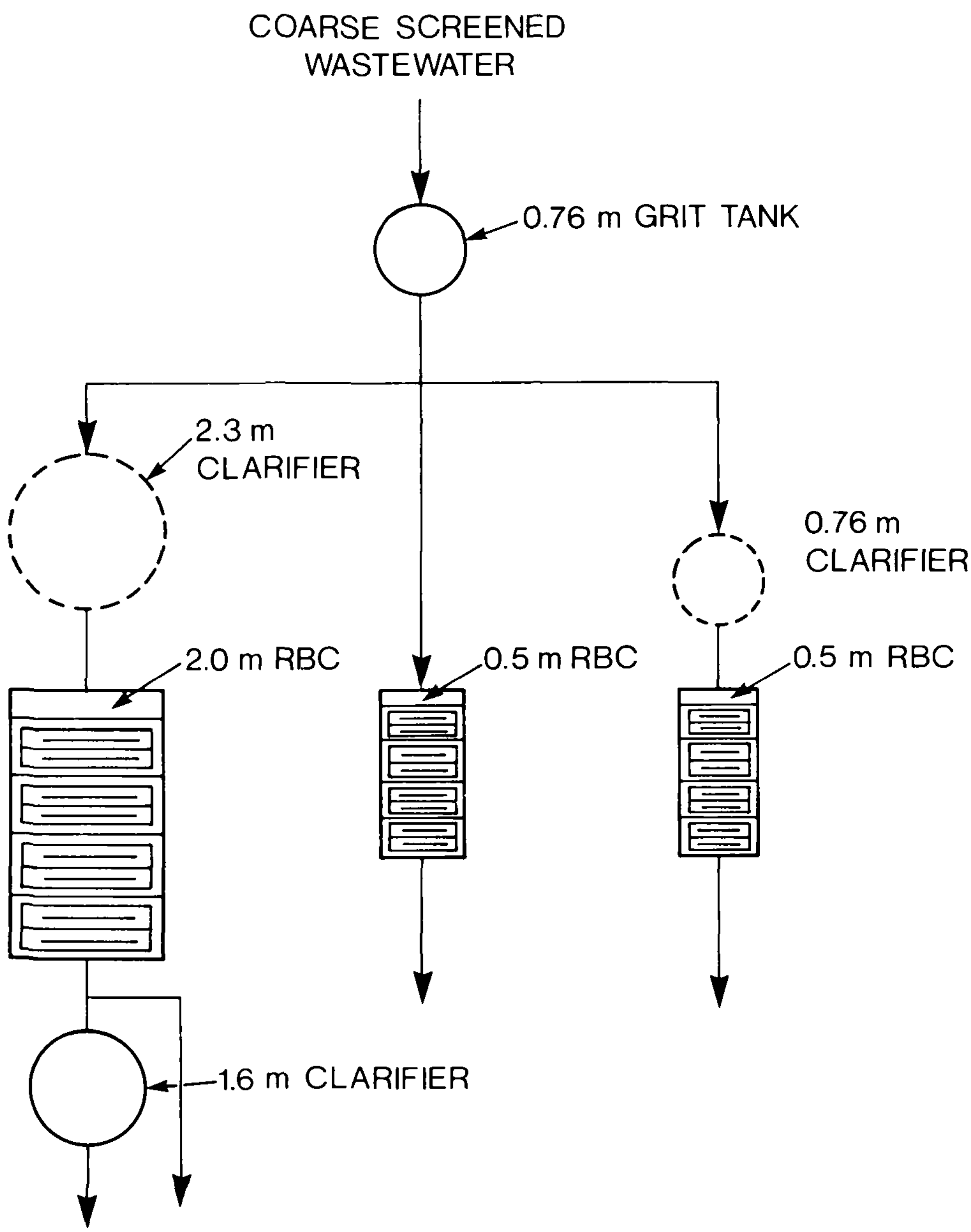

FIGURE 2-1 SCHEMATIC OF PILOT PLANT FACILITIES

By operating the 2.0 m and 0.5 m RBC's as shown in Figure 2-1, direct comparisons could be made between the two systems. Except for scale, they were in most ways similar, containing four compartments in series and having a media rotation such that their peripheral velocities were 0.33 m/s. The ratio of surface area to liquid volume was 207 m^2/m^3 for the 2.0 m RBC, and 181 m^2/m^3 for the smaller units. Dye tracer studies indicated no significant short circuiting of wastewater through any unit. The hydraulic characteristics of the 0.5 m unit approximated two Continuous Stirred Tank Reactors (CSTR) in series; those of the 2.0 m RBC were shifted more toward a plug flow system.

2.3 Summary of Experimental Program

The 0.5 m RBC's were installed and started-up in June 1976 and operated in parallel until the beginning of September. One ran continuously to the end of the test schedule in mid June, 1977. Acclimation of the pilot plants was completed for both BOD_5 removal and nitrification within 10 days of start-up, at an initial hydraulic loading of 4.2×10^{-3} $m^3/m^2{\cdot}d$.

The 2.0 m RBC was purchased by EPS and installed at the Wastewater Technology Centre during June and July 1976. It was started-up on July 16 and operated continuously for the remaining eleven months of the program.

The different series of RBC tests are described in detail, and the results presented in chapters 3 through 9. The task for each phase of the study, the time period involved and the experimental methods used are outlined in Table 2-1. Unless otherwise specified, analytical results were obtained from 24-hour refrigerated composite samples.

2.4 Wastewater Characteristics

The raw degritted wastewater, from the Burlington Skyway Water Pollution Control Plant, showed significant monthly variations in BOD_5, suspended solids and filtrable TKN. The average concentrations for these variables were 120 mg/L, 230 mg/L and 20 mg/L respectively, over the test period.

The wastewater had maximum pollutant levels between December and February. The monthly average BOD_5 and TKN concentrations increased to 180 mg/L and 29 mg/L respectively in January. Increased infiltration, during spring runoff in March, caused a rapid decrease in BOD_5 and TKN concentrations, to 100 mg/L and 17 mg/L respectively.

TABLE 2-1 RBC EXPERIMENTAL PROGRAM

Study Phase	Methodology	Time Period
PROCESS DESIGN CRITERIA		
1) Identifiy major and minor variables affecting performance.	Time series analysis of the 2.0 metre RBC performance during transient loading conditions.	September - October, 1976
2) Identify most appropriate criteria for design.	a) One test operating in a carbon removal mode.	
	b) One test operating in a carbon removal plus nitrifying mode.	
LONG TERM VARIABILITY		
1) Assess long term variability of effluent BOD_5 and suspended solids.	Monitor treated effluent BOD_5 and suspended solids at various RBC loadings during a 16 week winter operating period.	January - May, 1977
CHARACTERIZATION OF EFFLUENT SUSPENDED SOLIDS		
1) Characterize suspended solids in final clarifier effluent.	Settleability testing and microscopic examination of treated effluent suspended solids.	January - May, 1977
2) Determine the effect of primary clarification on RBC performance.	Operated a 0.5 m RBC with a primary clarifier for nine weeks in parallel with the 0.2 m RBC with and without primary clarification.	February - May, 1977
RESPONSE TO DYNAMIC LOADING		
1) Determine the effects of diurnal variations in flow and concentration.	Compare parallel operation of two 0.5 m RBC's, one operating as a control unit at pseudo steady state and the second operating as a test unit with an induced sinusoidal diurnal flow.	June - August, 1976
2) Characterize RBC response to dynamic operation.	Time Series Analyses of 2.0 m RBC operation	September - October, 1976
SCALE-UP		
1) Determine what factors, if any, should be used to scale-up results from pilot testing to full scale design.	Comparison of the 0.5 m and 2.0 m RBC's operated in parallel with identical feed and over a wide range of loadings.	September - December, 1976
PHOSPHORUS REMOVAL		
1) Assess the feasibility of chemical addition to the RBC to achieve phosphorus removal.	Monitor phosphorus removals by adding excess ferric chloride and alum to the RBC tank for consecutive periods of two weeks.	May - June, 1977
INTERMITTENT FLOW		
1) Study the effects of intermittent flow on RBC performance.	Conduct screening studies of the RBC operating at different intermittent and low flow conditions.	August 1976 December 1976

3 SELECTION OF DESIGN CRITERIA

3.1 Introduction

This chapter describes the results of two series of experimental runs in which the 2.0 m unit was operated under transient loading conditions.

A mathematical approach known as time series analysis was applied to the results of the dynamic runs to determine the importance of different input variables such as TOC, TKN and flow on RBC performance. This technique was used to identify variables offering the best statistical basis for use in RBC design.

3.2 An Overview of RBC Design and Performance

As the practical application of the RBC has evolved over the last 15 to 20 years in Europe and North America, several different methodologies for design have been proposed. Some researchers have attempted to adopt a traditional approach used with suspended-growth biological processes, by defining an organic loading or F:M ratio (Kornegay and Andrews, 1967; Pretorius, 1971; Willard et al, 1972). A theoretical mixed liquor volatile suspended solids' concentration (MLVSS) is determined by scraping the biomass from a known area of media, calculating the mass of volatile solids per unit area, and multiplying this value by the total media surface available. This total mass of volatile solids is then divided by the liquid volume of the RBC to arrive at the MLVSS. This approach has not enjoyed wide acceptance for RBC design, perhaps because of the difficulty in measuring or predicting the MLVSS concentrations necessary for the calculations.

Mathematical models have been widely used to describe or predict RBC performance. For use in design, Joost (1969) proposed the following empirical model based on substrate concentration, temperature and residence time:

$$\frac{\%\ BOD_5\ \text{removed}}{\text{Stage}} = K \times C^a \times R^b \times T^c \times S^d$$

where: K is a specific parameter for each waste,
C is the substrate concentration,
R is the reactor constant,
T is the temperature, and
S is the hydraulic residence time, and

a, b, c, d are dimensionless parameters.

This model suffers from two handicaps typical of many of those developed: first because it was derived using bench scale and small pilot scale data, and therefore its results may not apply to the larger, full scale RBC's; and second, and more importantly, because for every RBC a separate set of parameter values must be derived. Joost did not determine whether a common set of parameters, generally applicable to municipal wastewater treatment, could be developed.

A common approach to RBC design has been the use of substrate loading rate per unit area of disc. In bench scale studies, Stover and Kincannon (1976) showed that the amount of oxygen demand removed from a wastewater by the RBC depended on the oxygen demand applied per unit area and time, rather than on the organic concentrations or flow rate. This was supported by the results of a more recent study conducted by Poon et al (1977), who used a factorial design to determine which variables among flow, BOD_5 concentration, recirculation, chloride concentration and organic loading rate per unit area, had the greatest effect on BOD_5 removal efficiency. The organic load was found to be the variable that most closely correlated with effluent BOD_5 concentration. None of these variables had a large effect on the percentage of BOD_5 removal.

Mass loading has also been proposed as the major parameter for designing nitrifying RBC systems. Based on pilot plant studies, Murphy et al (1977) and Weng and Molof (1974) concluded that the mass of TKN removed per unit disc area and time was constant at a given temperature, and therefore was independent of flow or TKN concentration.

Most examples of practical design for RBC systems have come from manufacturers' design manuals, but recently results from long term and full scale studies in North America have allowed an independent review of their predictions. Table 3-1 shows the loadings which would be selected to achieve a treated effluent quality of 15 mg/L total BOD_5 and 15 mg/L SS, using design procedures from three different sources. The first two examples were calculated from data given in the design manuals of two North American RBC suppliers, Autotrol Corporation and Envirodisc Corporation. The third example was calculated using a European design procedure given by Steels (1974). The wastewater characteristics selected (see the bottom of the Table) are typical of those used during the experimental program discussed later in this chapter. No allowances have been made for safety factors or for reduced operating efficiencies at temperatures lower than 13°C. That the design loadings data are not in close agreement, suggests the need for independent confirmation of this work.

TABLE 3-1 DESIGN EXAMPLES FOR RBC TREATMENT

Information Source	Design Loadings: 90% Total BOD_5 Removal (kg total BOD_5/1000 m^2•d)	Design Loadings: Combined BOD_5 Removal Plus Nitrification 90% NH_4-N Removal (kg total BOD_5/1000 m^2•d)
1. Autotrol Corporation[(a)] (Autotrol, 1976)	26	17
2. Envirodisc Corporation[(a,b)] (Envirodisc, 1977)	12	8
3. Steels (1974)	14	--

(a) Raw Wastewater
Total BOD_5 = 150 mg/L
Filt BOD_5 = 40 mg/L
NH_4-N = 25 mg/L
Operating Temperature = 13°C

(b) BOD_5 and NH_4 diurnal concentration peaking factor = 1.4

Table 3-2 is a summary of some of the long term operating data available in the literature. The results of the full scale operations at Pewaukee and Gladstone show that the RBC can produce high quality effluents even during cold weather operation. The BOD_5 and suspended solids' concentrations in the treated effluents compare favourably to the concentrations that can be achieved with an activated sludge or similar process. Nevertheless, the relatively low loadings that were used at both the Pewaukee and Gladstone plants, indicate they were designed to allow for future increases in flow. It is impossible to determine from these data whether the same treatment efficiencies are possible at the higher loadings shown in Table 3-1

The RBC's used at Stevenage, Brampton and Yellowknife were small package plants. These systems contained internal primary and secondary clarification zones, and a sludge storage zone under the rotating media. The total hydraulic volume of the package plants provided theoretical hydraulic residence times in excess of 24 hours, several times that of the RBC tanks at Gladstone or Pewaukee. Although the performances of both types of systems (Table 3-2) were similar, the package plants may be better able to equalize organic surges or short duration overloads because of the longer hydraulic residences. The anaerobic sludge storage compartment in the package plant RBC's releases some organic material back into the rotating disc section as the settled sludge is solubilized to organic acids and other degradation products. This might be beneficial to systems that experience periods of low or intermittent flow. The slow release of filtrable organic carbon from the sludge zone may provide sufficient substrate to prevent excessive biomass sloughing during periods when the external feed source is interrupted.

TABLE 3-2 RBC PLANT OPERATING DATA

Plant Type and Location	BOD_5 Loading kg/1000 m^2•d	Temperature °C	Final Effluent (average values) BOD_5 mg/L	SS mg/L
1. Pewaukee, Wisc. Pilot Plant Data (Antonie et al, 1974)	~9		20	20
2. Pewaukee, Wisc. Full Scale Operation (Antonie et al, 1974)	6.4	7	21	16
	12	Summer	16	14
3. Gladstone, Mich. Full Scale Operation (Malhotra et al, 1975)	6.0	8	18	17
	11.7	12-18	19	13
4. Stevenage, U.K. Package Plant (Bruce et al, 1973)	~6*	6	37	37
	~6	8	23	32
	~6	11-12	19	26
	~6	17	14	23
5. Brampton, Ontario (Ahlberg and Kwong, 1974)	2.2**	5-10	6	8
	4.1	17-19	16	7
	9.4	12-15	22	16
	9.4	16-18	34	13
	13.4	8-15	41	28
6. Yellowknife Package Plant (Forgie et al, 1974)	4.9**	Summer	17	9
	10	Summer	16	13
	15	Summer	18	14
	18	Summer	22	12

* Estimated load entering disc section from the internal clarifier.
** Total load entering the package plant (i.e. to internal clarifier).

Although the cost of providing long retention times in treatment facilities serving population equivalents of 10 or even 100 people may be reasonable, significantly higher cost will result if the practice is extended to larger populations. Consequently, one of the questions examined during this study was whether an RBC with a relatively short hydraulic detention can be used for small, variable and intermittent wastewater sources.

3.3 Time Series Analysis of Dynamic Operating Data

Few comprehensive attempts have been made to study RBC performance under dynamic operating conditions. In the studies presented in Section 3.2, mechanistic or semi-empirical deterministic models have been proposed to predict the steady-state behaviour. They must attempt to incorporate mixing characteristics, retention times, active biomass film thickness, rotational speed, available surface area, system geometry, and diffusion coefficients for oxygen and substrate transfer. The methods rely on what are at best estimates, and therefore the steady-state parameter values obtained from them cannot take account of highly variable influent and effluent conditions as occur in many practical situations. In order to arrive at a reliable estimate of the system's stability, techniques must be used that can describe the responses to non-steady state conditions.

In the statistical procedure of time series analysis, input data is collected at discrete time intervals and used to evaluate dynamic process responses. Therefore, because the relative importance of interactions among input variables can be determined solely from the observed data, the uncertainty in selecting coefficients in the mechanistic models is eliminated.

Application of time series analysis requires specific experimental designs, which generally meet the following criteria:

1) Many paired data points are necessary ($\geq$100) at discrete equispaced time intervals.
2) As determined through observation or residence time studies, the sample period should be less than the process response time.
3) The effect of different influent levels of each variable is determined by observing the process when step changes are introduced.
4) A randomized factorial design is used so that subsequent analysis can separate the effects of each variable with maximum efficiency.

3.4 Experimental Program

3.4.1 Experimental Design. The first step was to select the variables to be evaluated. Flow ($m^3/m^2 \cdot d$), BOD_5 concentration (mg/L), BOD_5 loading (kg/1000 $m^2 \cdot d$), TKN concentration (mg/L), and TKN loading (kg/1000 $m^2 \cdot d$) were judged those probably exerting the greatest influence on the performance of RBC's treating municipal wastewater, and therefore were chosen for primary analysis.

Flow and TKN were measured directly. In order to estimate the oxygen demand, the influent was analyzed for filtrable organic carbon (TOC). Every tenth influent sample collected during the experimental runs was analyzed for BOD_5 and TOC, so that the two variables could be correlated.

To determine effluent quality TOC, TKN and $NO_3 + NO_2$ - N concentrations were monitored.

An examination of the effects of the RBC's mechanical design features on treatment efficiency was not the purpose of this study, consequently, variables such as shaft rpm, peripheral velocities, and media surface to volume ratio were maintained at constant values throughout the program.

To accommodate the above design requirements a two-level, three-factor, complete factorial design was selected. The three main variables were flow, TOC concentration, and TKN concentration; since the TKN and TOC loads are a function of the interactions between flow and concentration, the design was able to separate the effect of each variable. The design variables and run conditions are presented in Table 3-3.

Since domestic sewage was used, the design required artificial interference with the influent variables in order to eliminate their natural correlation, through the addition of dextrose as an organic carbon spike and ammonium chloride as a nitrogen spike. Two separate runs were conducted with the 2.0 m RBC, one at loading conditions which permitted only organic carbon removal, and the other at a lower loading which permitted carbon removal and nitrification. When necessary during the nitrification test, bicarbonate was added to the raw degritted wastewater feed in order to avoid any pH/alkalinity limitation on the nitrification rate.

3.4.2 Pilot Plant Operation. The 2.0 m RBC was operated at the mean values of the flow, TKN concentration and TOC concentration, for two days prior to the start of the experiments. Two BIF pumps were used to add the concentrated solutions of dextrose and ammonium chloride directly into the raw sewage line feeding the unit. Grab samples of

TABLE 3-3 EXPERIMENTAL DESIGN OF DYNAMIC RUNS

Influent Variable	Design Level	Operating Mode: Run 1 Carbon Removal	Run 2 Carbon Removal Plus Nitrification
Hydraulic Flow Rate	+	30.5×10^{-2}	13.7×10^{-2}
($m^3/m^2 \cdot d$)	-	14.3×10^{-2}	5.5×10^{-2}
Filtrable TOC	+	80-90	80-90
(mg/L)	-	50-60	50-60
Filtrable TKN	+	25	40-50
(mg/L)	-	10	20-30
TOC Loading	+	26	10.4
($kg/1000\ m^2 \cdot d$)	-	8	2.7
TKN Loading	+	7.6	5.5
($kg/1000\ m^2 \cdot d$)	-	1.4	1.2

the influent and effluent were taken every half hour during the two and a half day experimental runs, and these were analyzed for filtrable COD, TOC, NH_3-N, NO_2-N, NO_3-N and TKN. Ten percent of the samples were analyzed for BOD_5. Step changes in input variables were made at five-hour intervals. The complete factorial design required a run period of 40 hours; therefore, the additional time during each was used to replicate some of the input conditions.

Wastewater temperatures for the carbon removal run remained constant at 20°C while for the combined carbon removal plus nitrification run, the temperature varied between 15 and 17°C.

3.5 Results and Discussion

3.5.1 Experimental Data. For the data from the two experimental runs see Figures 3-1 and 3-2.

Directly measured flow rates were identical to the design levels. With a few exceptions, there was relatively good agreement between the designed input sequences and the measured influent concentrations for the two runs. This was important in

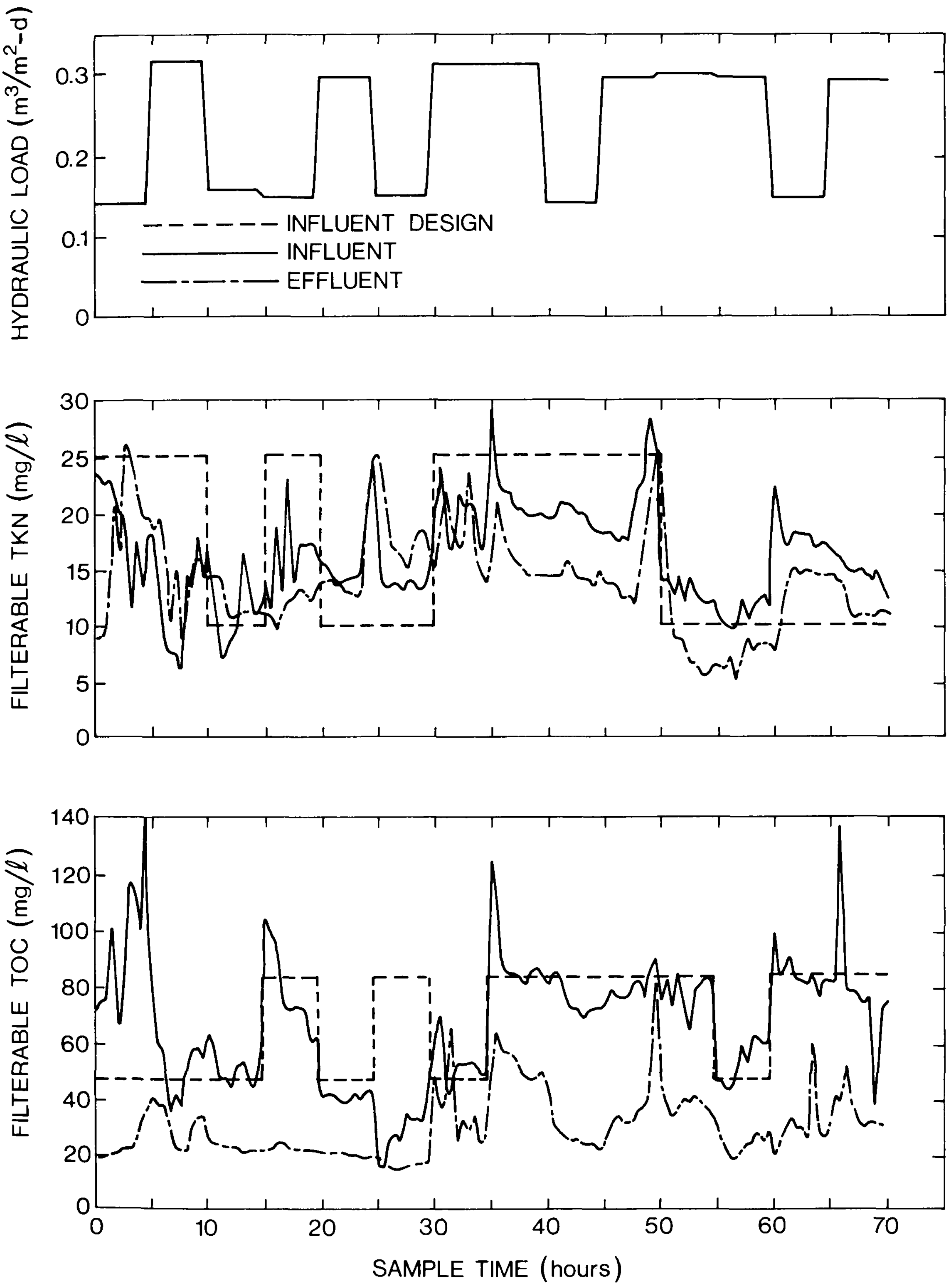

FIGURE 3-1 INPUT AND RESPONSE OF THE 2.0 m RBC
(Carbon Removal Mode)

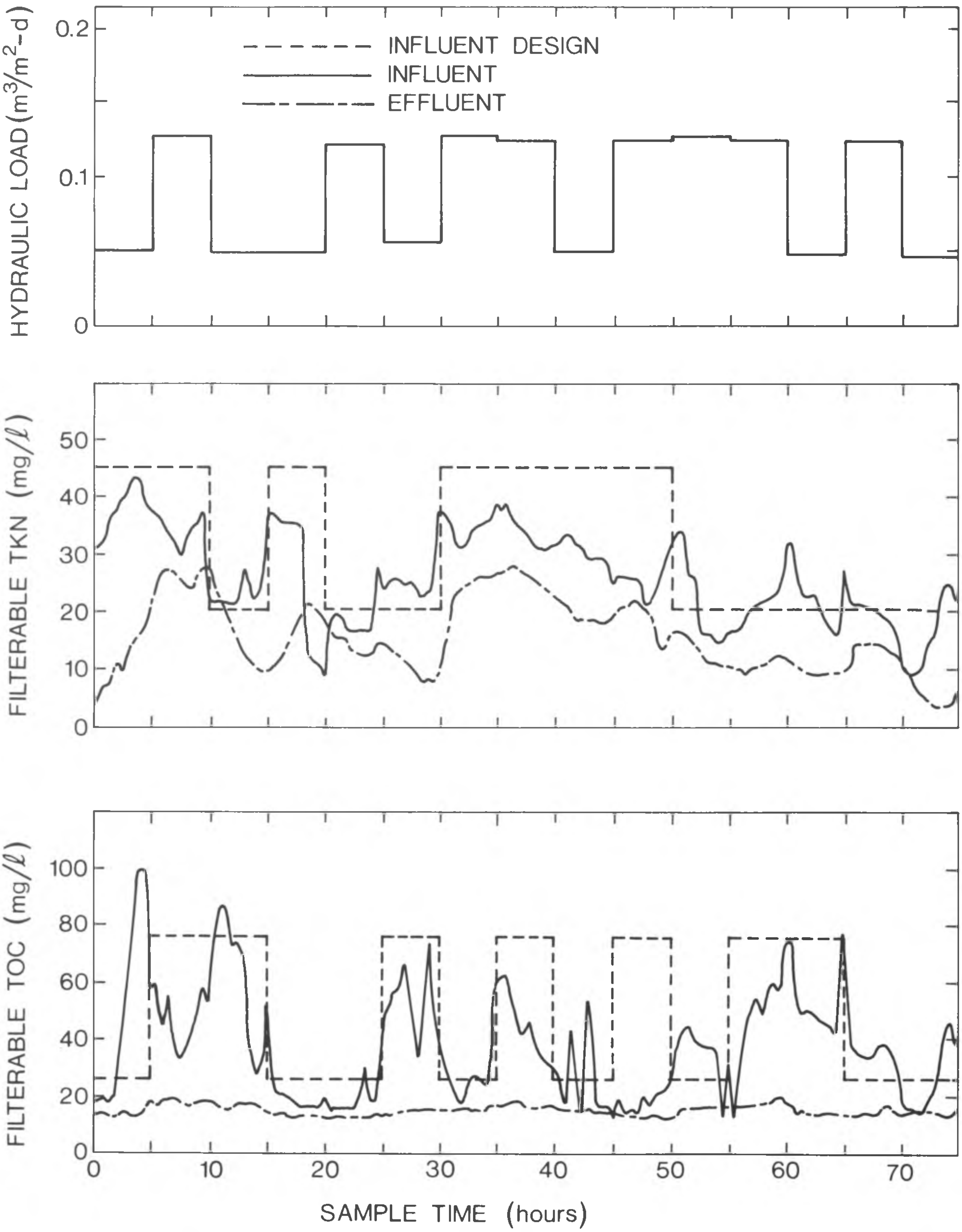

FIGURE 3-2 INPUT AND RESPONSE OF THE 2.0 m RBC (Carbon Removal Plus Nitrification Mode)

ensuring that flow, TOC concentration and TKN concentration were varied independently of one another. The first ten hours of the carbon removal run (Figure 3-1) show the effluent TKN concentration frequently exceeded influent concentrations. During this period, the raw wastewater was diluted by tap water, but because influent samples were taken before the two flows were properly mixed, the TKN concentration was underestimated. These data were therefore omitted from the analysis.

A careful review of the measured influent TOC and TKN concentrations (Figure 3-1) reveals that their input levels were not randomized to the degree designed, thus increasing the risk of the data analysis showing a significant correlation between an input variable and effluent quality where none existed.

Although during the nitrification run one high and one low period of TOC concentration were missed, this did not significantly affect the randomization of the three input variables. Therefore, the statistical integrity of the design remained intact, and the independent variables should not show any false correlations due to confounding.

Effluent TOC concentrations responded significantly to variations in loading during the test period (Figure 3-1). However, it was not possible to determine from the data which variables were responsible for the response. This was also the case for the response in effluent TKN concentration in the carbon removal plus nitrification run (Figure 3-2).

3.5.2 Cross Correlations. The first procedure used in examining the RBC response determined the cross correlation function, consisting of an array of cross correlation and auto-correlation coefficients, between influent variables and effluent responses. These coefficients can indicate whether a given output response is statistically related to changes in specific input variables. A significant correlation between an input variable X and an output response Y generally signifies a cause and effect relationship if a factorial design or other proper statistical procedure is used in conducting the experiments. A summary of the statistical procedures used in this chapter for time series analysis is provided in Appendix I. Greater detail is given by Box and Jenkins (1970).

A summary of the cross correlations investigated for both the carbon removal and the carbon removal/nitrification runs is presented in Table 3-4. The five input variables included in the analysis were filtrable TOC loading and concentration, filtrable TKN loading and concentration, and flow.

Filtrable TOC concentration was used to describe effluent quality during the carbon removal run. The data in Table 3-4 show that the correlations between effluent TOC and all input variables, except TKN concentration, were significant. This suggests

TABLE 3-4 SUMMARY OF CROSS CORRELATION RESULTS (1st Difference)

		Correlation Significance	
Input Variable	Output Variable	Run 1 Carbon Removal	Run 2 Carbon Removal Plus Nitrification
TOC Loading	(TOC)	Yes	Yes
TKN Loading	(TOC)	Yes	Yes
Flow	(TOC)	Yes	Yes
(TOC)	(TOC)	Yes	No
(TKN)	(TOC)	No	No
TOC Loading	(TKN)	N/A	Yes
TKN Loading	(TKN)	N/A	Yes
Flow	(TKN)	N/A	Yes
(TOC)	(TKN)	N/A	No
(TKN)	(TKN)	N/A	Yes
(NH_4-N)	(TKN)	N/A	Yes
TOC Loading	(NO_3 + NO_2 -N)	N/A	Yes
TKN Loading	(NO_3 + NO_2 -N)	N/A	Yes
Flow	(NO_3 + NO_2 -N)	N/A	Yes
(TOC)	(NO_3 + NO_2 -N)	N/A	No
(TKN)	(NO_3 + NO_2 -N)	N/A	Yes
(NH_4-N)	(NO_3 + NO_2 -N)	N/A	No

( )concentration
N/A not applicable

that any one, or combination of the four significant input variables, could be used in developing a predictive model for effluent quality.

There were no significant correlations between effluent TOC and influent TKN or TOC concentrations during the carbon removal plus nitrification run, but they were significant between the effluent TKN concentration and all of the input variables except input TOC concentration. During run 2, the concentration of NO_3 + NO_2-N in the treated effluent was also evaluated as a measure of effluent quality. The cross correlations for it were calculated in order to determine if the factors affecting net nitrate formation in the system were the same as those affecting TKN removal. This was done because the amount of NO_3 + NO_2-N formed was consistently less than the amount of TKN removed from the wastewater, suggesting that denitrification within anoxic zones in the slimes may have been occurring. With one exception, the cross correlations for the effluent NO_3 + NO_2-N show the same significant responses as for the TKN cross correlations. The correlation between effluent NO_3 + NO_2-N and influent NH_4-N was not found significant, whereas it was between effluent TKN and influent NH_4-N.

Impulse response weights are related to the cross correlation coefficients, and may be used to identify significant output responses caused by changes in levels of input variables. Figures 3-3, 3-4, and 3-5 show a selection of impulse response weights for effluent filtrable TOC in Run 1, and effluent filtrable TKN and NO_3 + NO_2-N in Run 2. In the figures, the zero point on the abscissa represents the instant at which a step change was made in the TOC, TKN and/or hydraulic loading entering the RBC. The series of 20 vertical lines in each graph represent the set of impulse response weights for lag 0 through lag 19. The impulse response weight at a lag of one-half hour, following the step change in the middle graph of Figure 3-3, has a value of 0.18. This value is statistically significant as it is greater than the 95 percent confidence limit indicated by the dashed lines. This is the only significant impulse response weight between zero and 10 hours following the step change, as the remaining values are less than the confidence limit. Consequently, this plot shows that a change in the TOC concentration entering the RBC caused a significant response in effluent TOC concentration approximately 30 minutes after the influent change was made. No other significant responses were observed before or after this period.

The impulse response weights for effluent TOC concentration versus influent TOC loading and hydraulic loading (Figure 3-3) indicate a rapid positive response was obtained in effluent concentration with respect to step changes in the influent. This seems to show the RBC has little capacity to equalize shock loads or rapid changes in raw wastewater conditions. Reacclimation to a new steady-state level during the carbon removal run was also rapid as no significant correlations were observed after the first hour following a step change.

The bottom graph in Figure 3-3 shows that the correlation between flow and effluent TOC concentrations is relatively weak as the impulse response weight at lag

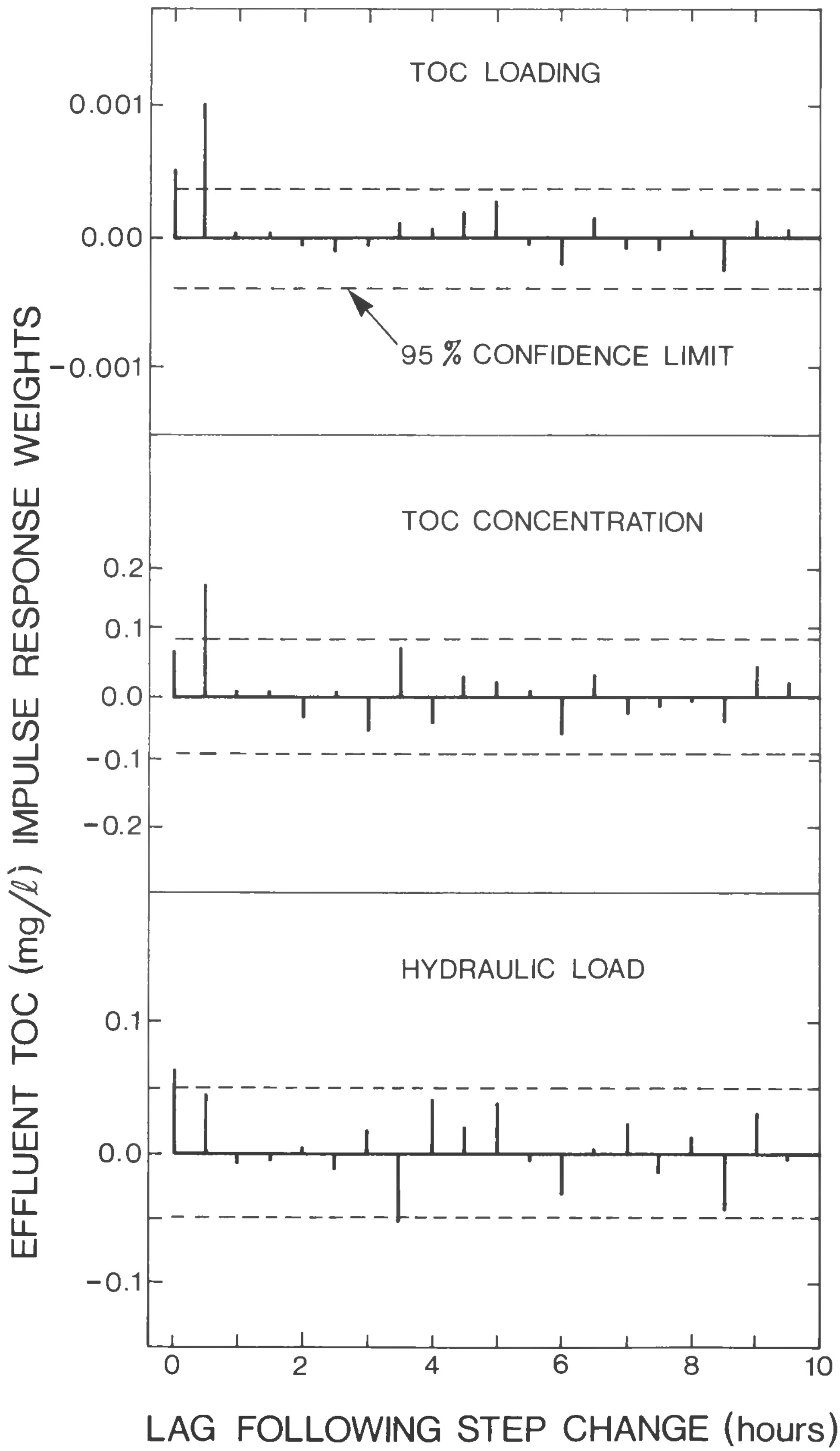

FIGURE 3-3 IMPULSE RESPONSE WEIGHTS FOR EFFLUENT TOC IN THE CARBON REMOVAL MODE (2.0 m RBC)

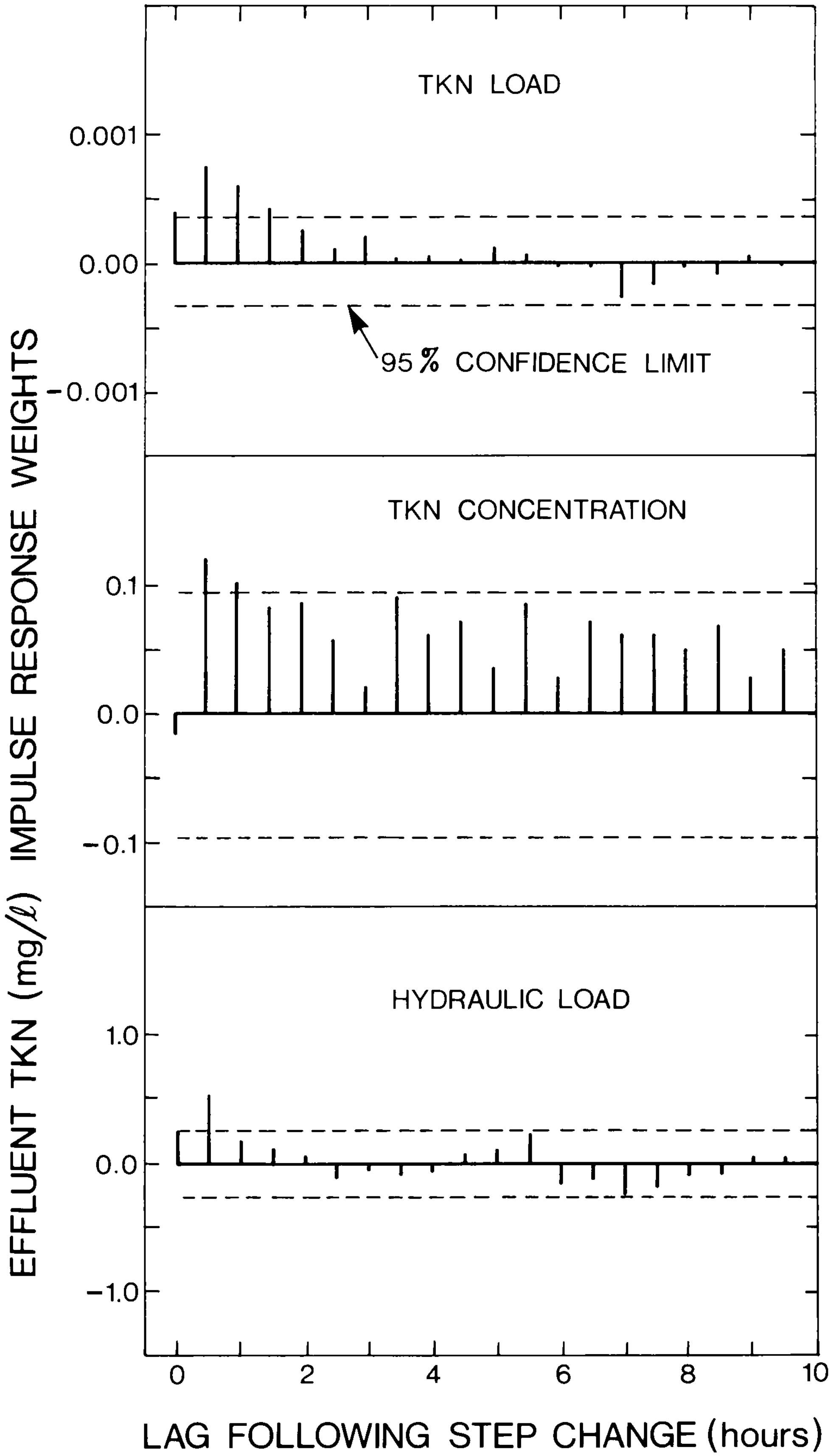

FIGURE 3-4 IMPULSE RESPONSE WEIGHTS FOR EFFLUENT TKN IN THE CARBON REMOVAL PLUS NITRIFICATION MODE (2.0 m RBC)

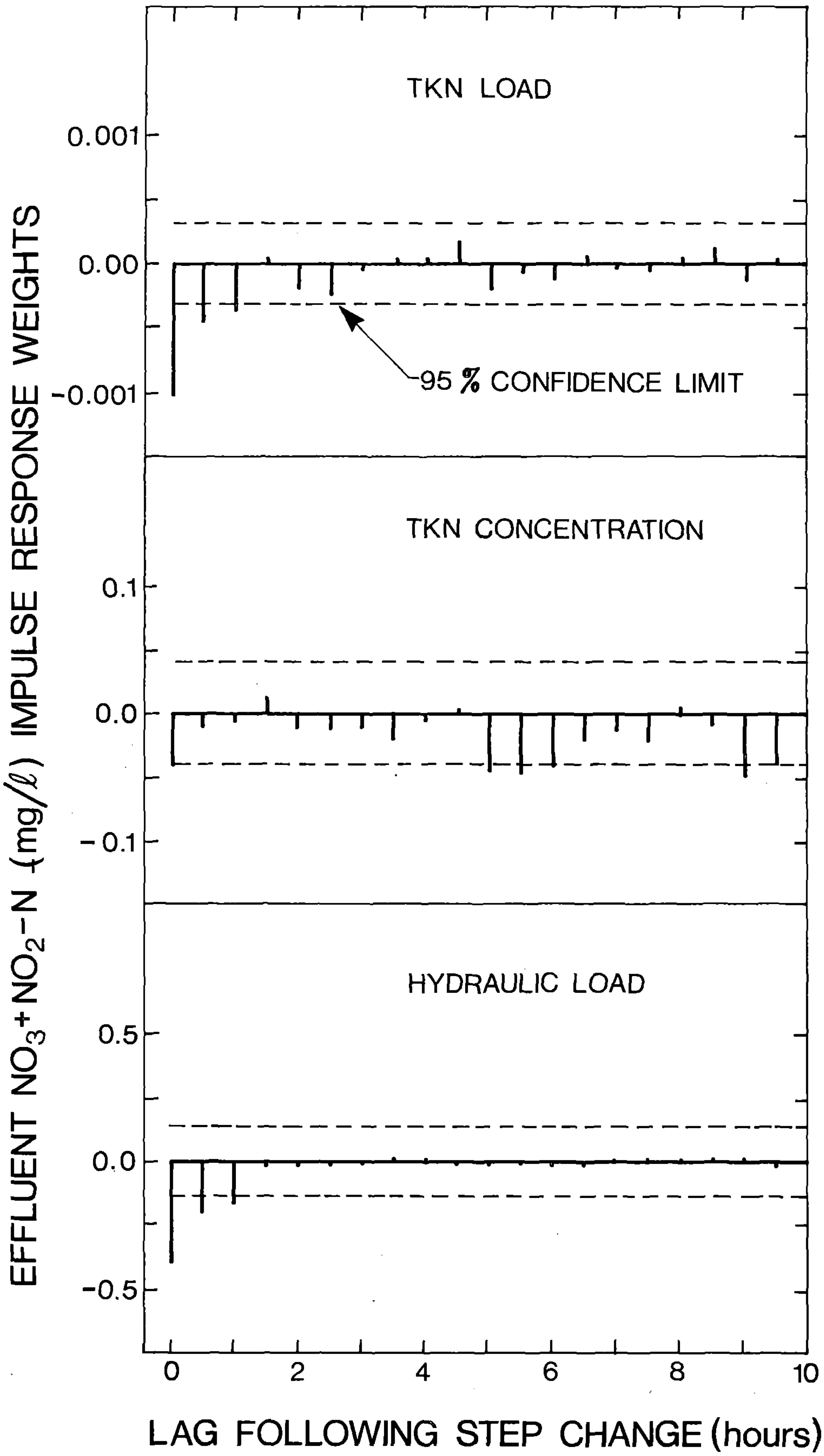

FIGURE 3-5 IMPULSE RESPONSE WEIGHTS FOR EFFLUENT $NO_3 + NO_2$- N IN THE CARBON REMOVAL PLUS NITRIFICATION MODE (2.0 m RBC)

zero is just barely significant. This suggests that influent TOC concentration or TOC loading have greater influence on effluent quality.

Significant positive correlations for filtrable TKN loading, TKN concentration and flow, versus effluent TKN concentration are presented in Figure 3-4. In every case all significant effluent responses to the step changes occurred within two hours of the change. Beyond two hours, a new steady-state operating condition was achieved.

Figure 3-5 shows that variation in TKN loading and hydraulic loads caused immediate and strong negative responses in the concentration of $NO_3 + NO_2$-N in the treated effluent. Although the impulse response weight for TKN concentration is just significant at lag 0, this is not the case at lag 1 and lag 2, when it is clear that flow is the more important influence. Table 3-4 shows no correlation between the influent NH_4-N concentration and the effluent NO_3+NO_2-N concentration. This further indicates that the TKN concentration correlation in Figure 3-5 is relatively unimportant. It would appear that the amount of $NO_3 + NO_2$-N formed during carbon removal plus nitrification is independent of TKN concentration.

3.5.3 Development of Predictive Models. Predictive time series models, known as transfer function-noise models, were developed to predict effluent quality using the procedure outlined in Appendix I. Each model used a different input variable to predict effluent TOC or TKN concentrations. A total of six were developed, three to predict effluent TOC concentration for the carbon removal mode (see Table 3-5) and three to predict effluent TKN concentration for the carbon removal mode plus nitrification mode. The input variables assessed for carbon removal were TOC loading, influent TOC concentration and hydraulic load, as the cross correlation function indicated that each of these had a significant effect on effluent TOC concentration. The input variables for the carbon removal plus nitrification models were: TKN loading, TKN influent concentration and hydraulic load.

Diagnostic checks of the models' residuals verified that each one adequately described the data, but Table 3-6 shows that the TOC and TKN loading models have the lowest residual mean square. Therefore, of the models considered, they should provide the best prediction of effluent TOC and TKN concentrations. Figures 3-6 and 3-7 provide a visual measure of the models' adequacies, by comparing the predicted responses to those observed. The predictions from both models closely follow the observed effluent concentrations. As would be expected from the lower residual mean square for the TKN

TABLE 3-5 TRANSFER FUNCTION-NOISE MODELS FOR PREDICTING EFFLUENT QUALITY FOR THE 2.0 m RBC

RBC Mode	Model No.	Variables	Model
Carbon Removal	A	Y_t = effluent TOC (mg/L) X_t = TOC load (g/d)	$Y_t = (0.00047 + 0.00107\ \beta)\ X_t + \frac{(1 - 0.589\ \beta)}{(1 - \beta)}\ a_t$
	B	Y_t = effluent TOC (mg/L) X_t = influent TOC (mg/L)	$Y_t = (0.093 + 0.072\ \beta)\ X_t + \frac{(1 - 0.482\ \beta)}{(1 - \beta)}\ a_t$
	C	Y_t = effluent TOC (mg/L) X_t = flow (L/d)	$Y_t = (0.028 + 0.055\ \beta)\ X_t + \frac{(1 - 0.350\ \beta)}{(1 - \beta)}\ a_t$
Carbon Removal Plus Nitrification	D	Y_t = effluent TKN (mg/L) X_t = TKN load (g/d)	$Y_t = \frac{(0.00054 + 0.00095\ \beta)}{(1 - 0.729\ \beta)}\ X_t + \frac{a_t}{(1 - 0.215\ \beta)(1 - \beta)}$
	E	Y_t = effluent TKN (mg/L) X_t = influent TKN (mg/L)	$Y_t = \frac{(0.084 + 0.056\ \beta)\ \beta}{(1 - 0.083\ \beta)}\ X_t + \frac{a_t}{(1 - 0.27\ \beta)(1 - \beta)}$
	F	Y_t = effluent TKN (mg/L) X_t = flow (L/d)	$Y_t = (0.000037 + 0.000018\ \beta)\ \beta X_t + \frac{a_t}{(1 - 0.42\ \beta)(1 - \beta)}$

Note: Model based on 1/2 hour sample intervals
a_t is a white noise sequence of independent random variables
β is the Backward Shift Operator

TABLE 3-6 COMPARISON OF 2.0 m RBC TRANSFER FUNCTION-NOISE MODELS FOR BEST FIT

Mode	Model	Input Variable	D.F.	Residuals Sum of Squares $(mg/L)^2$	Residuals Mean Square $(mg/L)^2$
Carbon Removal	A	TOC Load	117	3014	25.8
	B	TOC Conc.	117	3892	33.3
	C	Flow	117	4696	40.1
Carbon Removal Plus Nitrification	D	TKN Load	145	224	1.54
	E	TKN Conc.	145	269	1.86
	F	Flow	145	257	1.76

loading model compared to the TOC model, predicted values for effluent TKN are closer to the actual data than are the TOC predictions.

3.5.4 BOD_5 versus TOC Correlation. Figures 3-8 and 3-9, for which most of the data were derived from the time series runs, show the relationship between TOC and the total and filtrable BOD_5 in the raw degritted wastewater. Both figures show relatively good correlation between TOC and BOD_5, suggesting that the transfer function and noise models using TOC as the input variable could have also been successfully developed using a BOD_5 loading instead.

3.6 Summary

Based on the results presented in Section 3.5, the following conclusions can be made regarding the RBC process.

1) Time series analysis showed the TOC loading is the best of the variables evaluated in describing RBC performance when only carbon removal is of interest. As there was a close correlation between TOC and BOD_5 (Figures 3-8 and 3-9), BOD_5 loading should also be acceptable as a major design parameter.
2) TKN loading should be selected as the major design parameter for carbon removal plus nitrification.
3) Changes in raw wastewater characteristics and loading to an RBC cause rapid changes in treated effluent quality. The RBC has little capacity to equalize raw wastewater variations or absorb shock loadings.

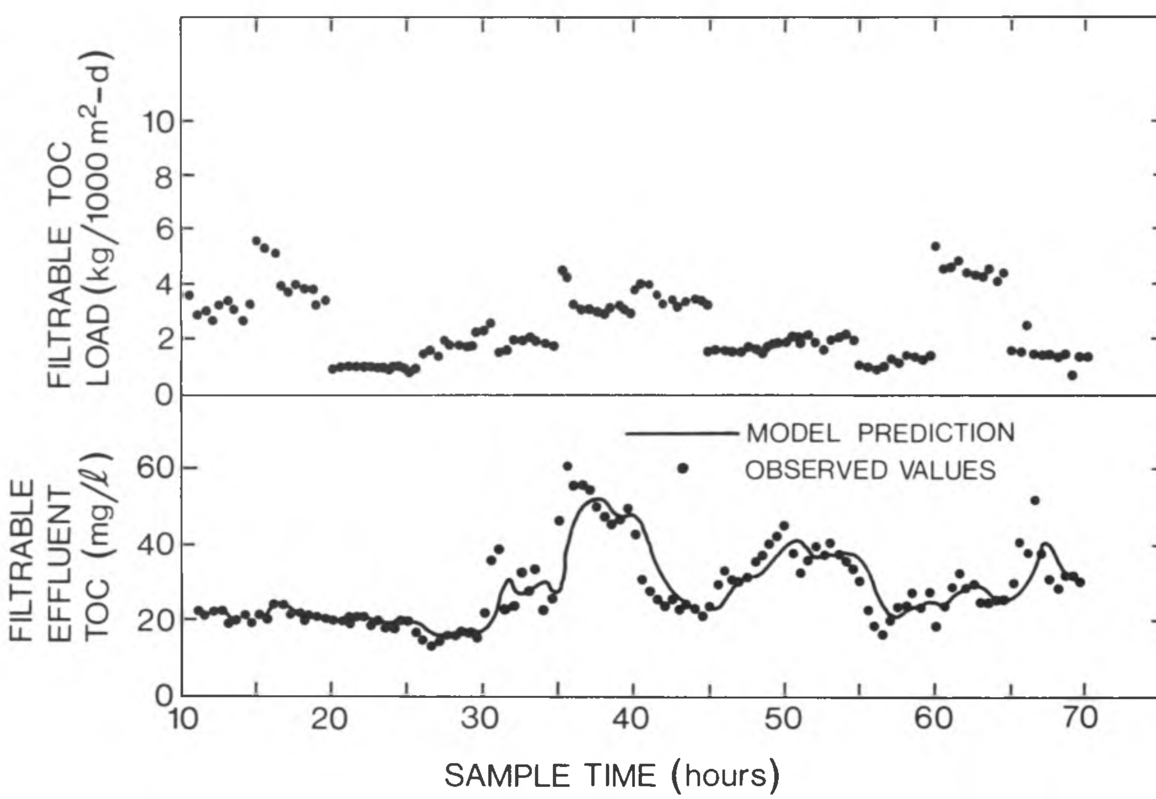

FIGURE 3-6 COMPARISON OF PREDICTED AND OBSERVED EFFLUENT FILTRABLE TOC (2.0 m RBC in the Carbon Removal Mode)

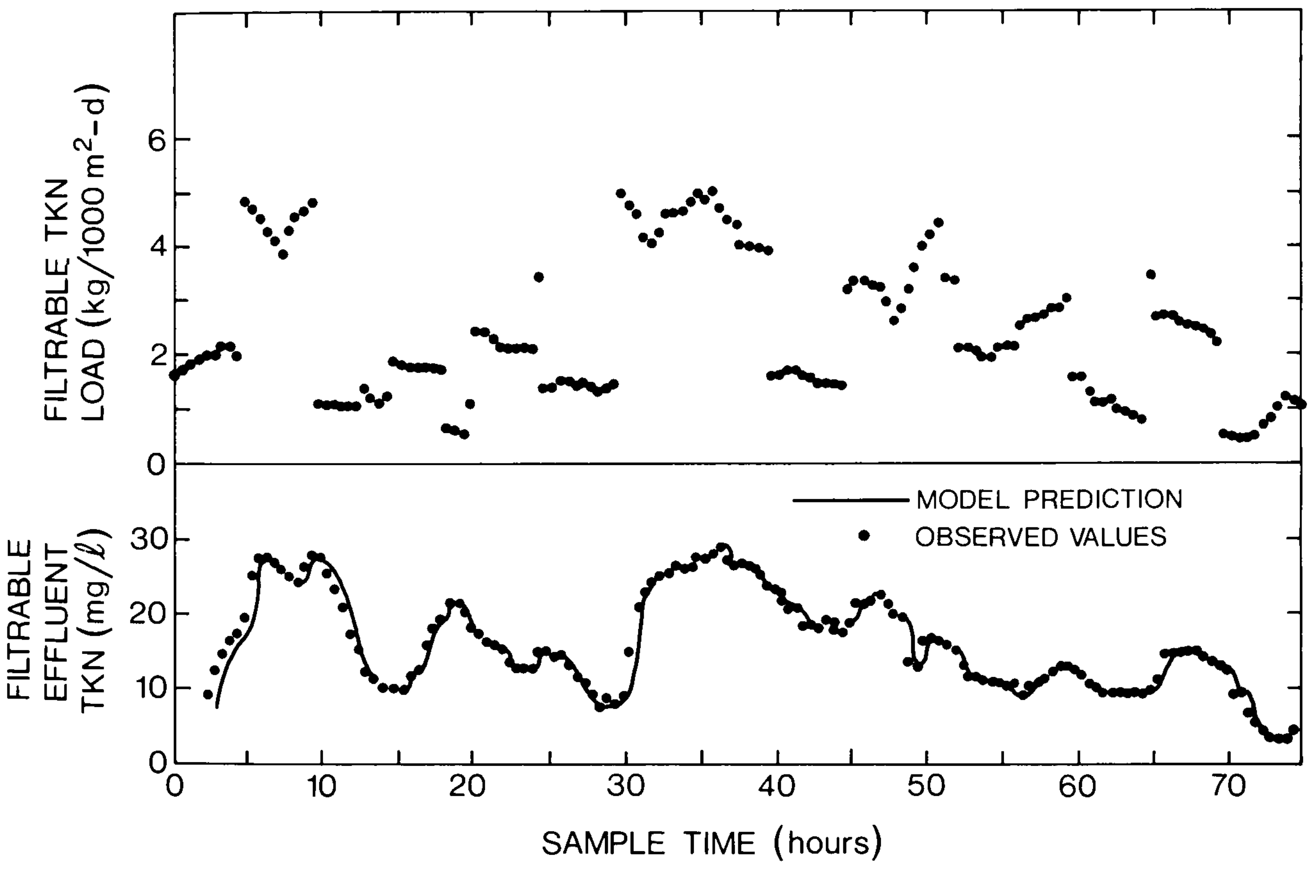

FIGURE 3-7 COMPARISON OF PREDICTED AND OBSERVED EFFLUENT FILTRABLE TKN (2.0 m RBC in the Carbon Removal/ Nitrification Mode)

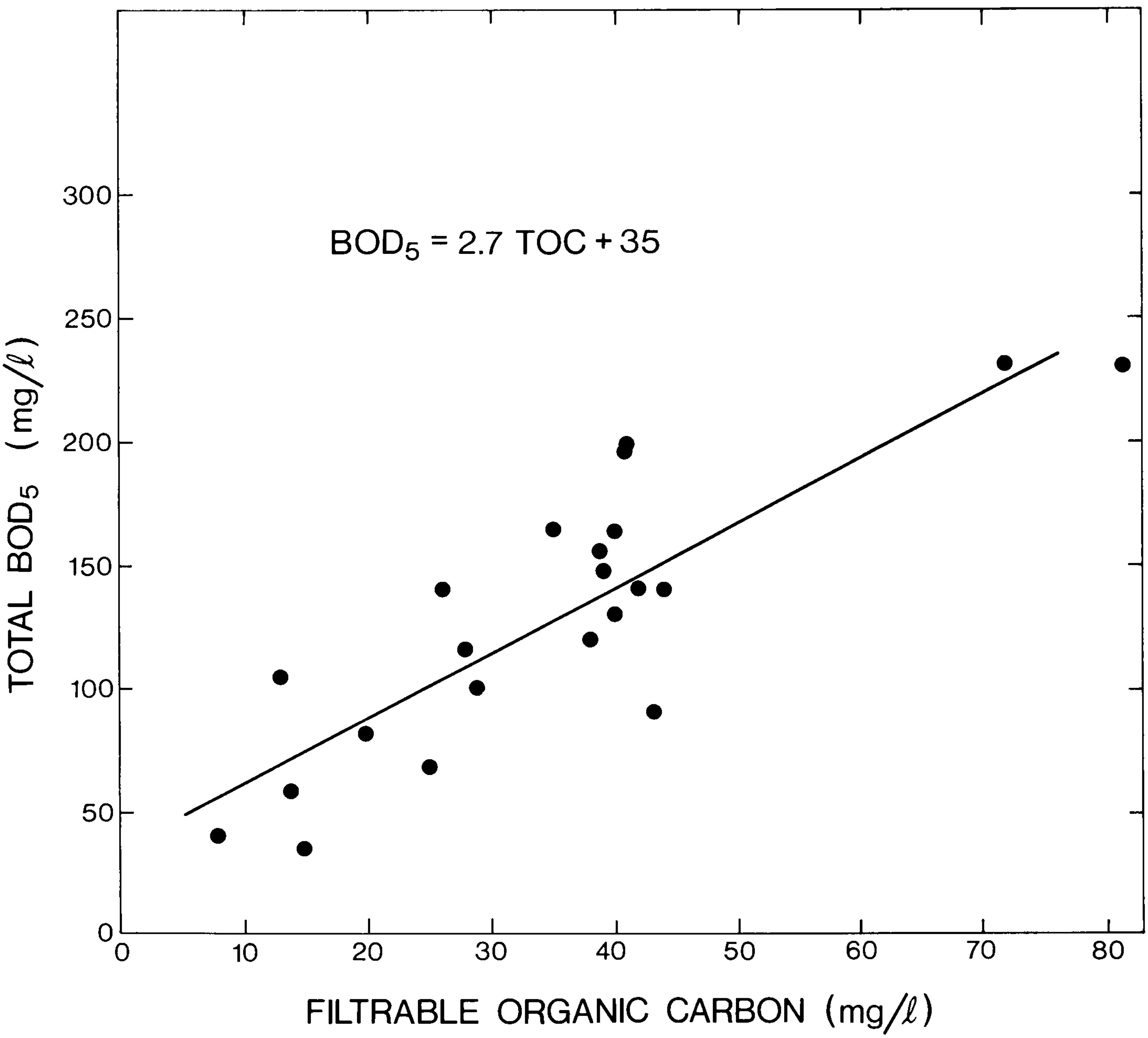

FIGURE 3-8 TOTAL BOD_5 vs TOC CORRELATION

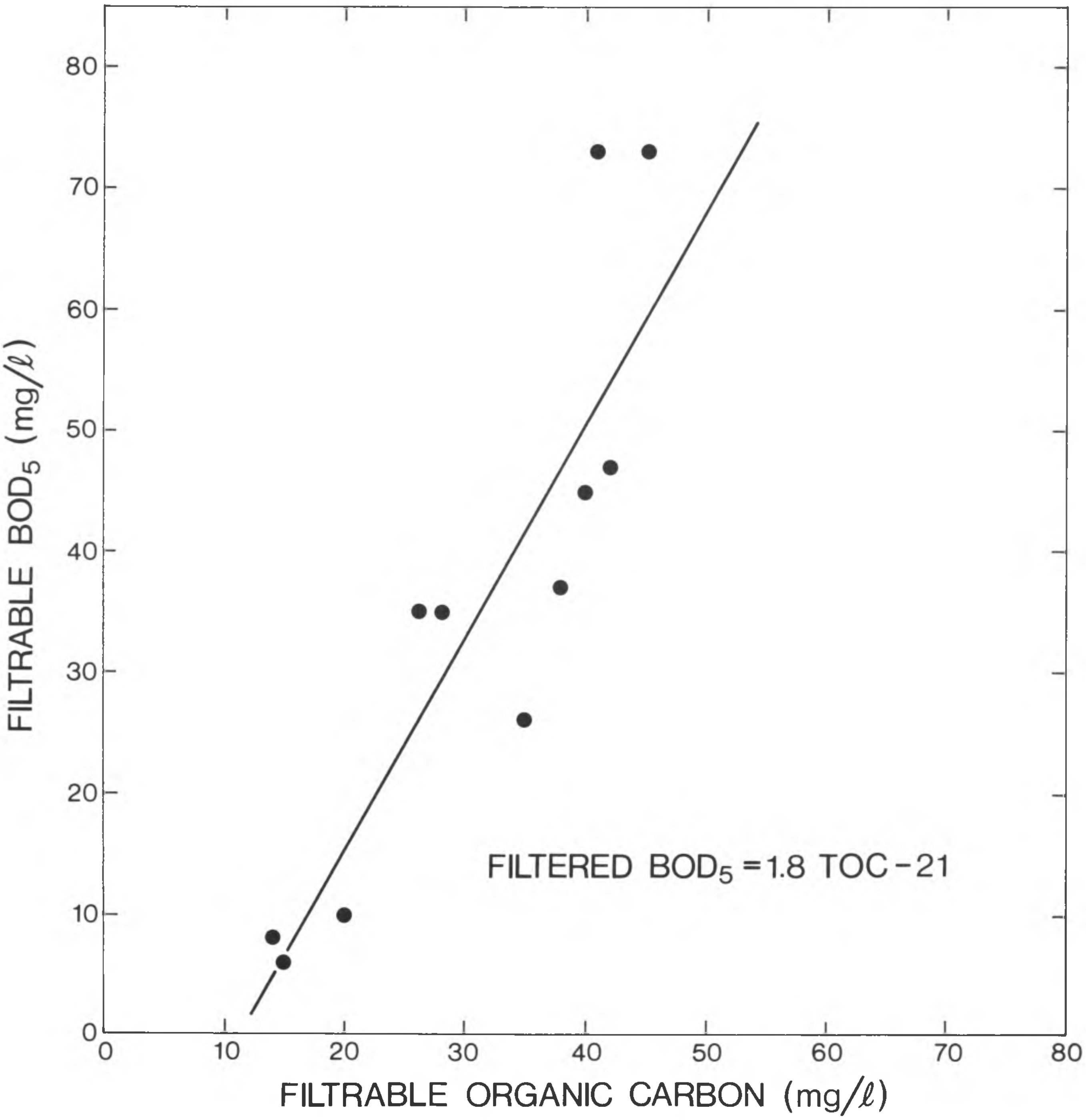

FIGURE 3-9 FILTRABLE BOD_5 vs TOC CORRELATION

4) Shock organic carbon and TKN loadings and hydraulic surges do not inhibit the active biomass or cause washout in the RBC. Steady-state performance is regained within several hours of sudden changes in feed conditions.

5) The formation of $NO_3 + NO_2$-N during nitrification is independent of the NH_4-N concentration in the influent.

Filion et al (1977) compared the gain or magnitude of response observed in a RBC to the gain observed in a pilot plant activated sludge system, while both systems were subjected to similar relative changes in loading. In the RBC the gain was 5.6 times that in the activated sludge process for carbon removal plus nitrifying systems, and 2 times that for carbon removal systems. This comparison indicates that relative to other processes, such as activated sludge which has a longer hydraulic detention time, the RBC is at a disadvantage in providing consistent effluent quality under non-steady-state conditions.

These conclusions apply to RBC systems having similar hydraulic characteristics to the four-stage, short hydraulic detention process used in the test program. Small package plant RBC's, which have much larger theoretical hydraulic residences, may exhibit longer lag periods for responses to and recovery from wastewater surges and shock loads.

4 EVALUATION OF DESIGN LOADINGS

4.1 Introduction

Pseudo steady-state runs (i.e., constant flow and naturally varying BOD_5 and TKN concentrations) were conducted, using the 2.0 m unit at various BOD_5 loadings, in order to determine a basis for selecting acceptable design loadings for BOD_5 removal and for combined BOD_5 removal plus nitrification. Performance under various loading conditions has been expressed as mass removal of BOD_5 or TKN per unit area and time. The efficiency of treatment was measured in terms of BOD_5, TKN and SS concentrations in the final effluent.

Other factors discussed in this chapter include temperature, and the effects of primary and secondary clarification on RBC performance.

4.2 Experimental Program

The 2.0 m RBC was operated from January to the beginning of May (for several weeks at a time) under various pseudo steady-state conditions so that design loadings could be developed that would provide acceptable treatment efficiencies at winter temperatures.

Raw degritted wastewater was fed to the pilot plant. A 1.2 m pilot clarifier (through which only part of the total flow passed) was used for the treated effluent at a constant overflow rate of 0.6 m/h. This low rate ensured that the observed effluent quality was a function of the RBC only and not of the clarifier operation.

The BOD_5 loading was varied between 4 and 40 kg/1000 $m^2{\cdot}d$. Loading changes were caused by intentionally varying hydraulic loads between 18×10^{-2} and 4.5×10^{-2} $m^3/m^2{\cdot}d$ and by natural variation of waste strengths at a constant hydraulic load.

A 0.5 m RBC was operated in parallel with the 2.0 m RBC during most of the period between January and the end of April 1977. Although the smaller unit was operated indoors, the operating wastewater temperature range was similar to that of the 2.0 m unit. The 0.5 m unit was used to determine the effect of primary clarification on overall performance. Before the wastewater was fed to the 0.5 m RBC, the same raw degritted wastewater used as feed for the 2.0 m RBC, was clarified in a 0.76 m pilot plant clarifier at an overflow rate of 1.0 m/h. The performance of the two pilot plants was monitored by collecting and analyzing 24-hour composite samples of influent and effluent approximately twice weekly.

All discussion of BOD_5 data in this chapter refers to carbonaceous BOD_5 only, and does not include nitrogenous oxygen demand. Allylthiourea was used to inhibit nitrification in BOD_5 tests of RBC effluent when the BOD_5 loadings to it were low.

4.3 Discussion of Results

4.3.1 Mass Loadings versus Mass Removal. The capacity for BOD_5 removal in the RBC is best shown by comparing mass loading to mass removal measured as mass of BOD_5 per unit area and unit time. Figure 4-1 presents the data for the 2.0 m RBC observed in the period between January and the beginning of May 1977, during which time there was no primary clarification of the degritted wastewater. The BOD_5 removal efficiency remained relatively constant at 75 to 80 percent until the total loading exceeded 15 kg/1000 m^2•d. Beyond this, the data show considerable scatter, although mass removals appear to be approaching a maximum or limiting value.

For the same period, filtrable BOD_5 loadings and mass removals are shown in Figure 4-2. During this time, mass loadings increased from less than 1.0 to approximately 13 kg/1000 m^2•d. Again the data show significantly more scatter at the higher loadings. Nevertheless, the filtrable BOD_5 removed increased with the loading and showed no sign of approaching a maximum within the range tested. At a load of 13 kg/1000 m^2•d the mass removal was about 7 kg/1000 m^2•d. Filtrable BOD_5 removals at loadings up to approximately 4 kg/1000 m^2•d were consistently 60 to 65 percent. The linear relationship between BOD_5 mass loading and mass removal (Figure 4-2), shows that the percentage removal of filtrable BOD_5 is relatively independent of loading. This is consistent with the data presented by Poon et al (1977), which was discussed in Section 3.2, and it suggests that the rate of removal of BOD_5 increases with the concentration in the RBC. The decrease in the efficiency of total BOD_5 removal (Figure 4-1) at the higher loadings must be attributed to an increased percentage of particulate BOD_5 passing through the system.

Both instantaneous and composited pseudo steady-state data on TKN removals for the combined BOD_5 removal plus nitrification mode are presented in Figure 4-3. The loadings applied to the 2.0 m unit from January to mid-April were too high to achieve consistent nitrification. Consequently, the data were insufficient for plotting mass loadings versus mass removals for TKN at low temperatures, as was done for BOD_5. Nevertheless, considerable data were available from the dynamic run conducted with the

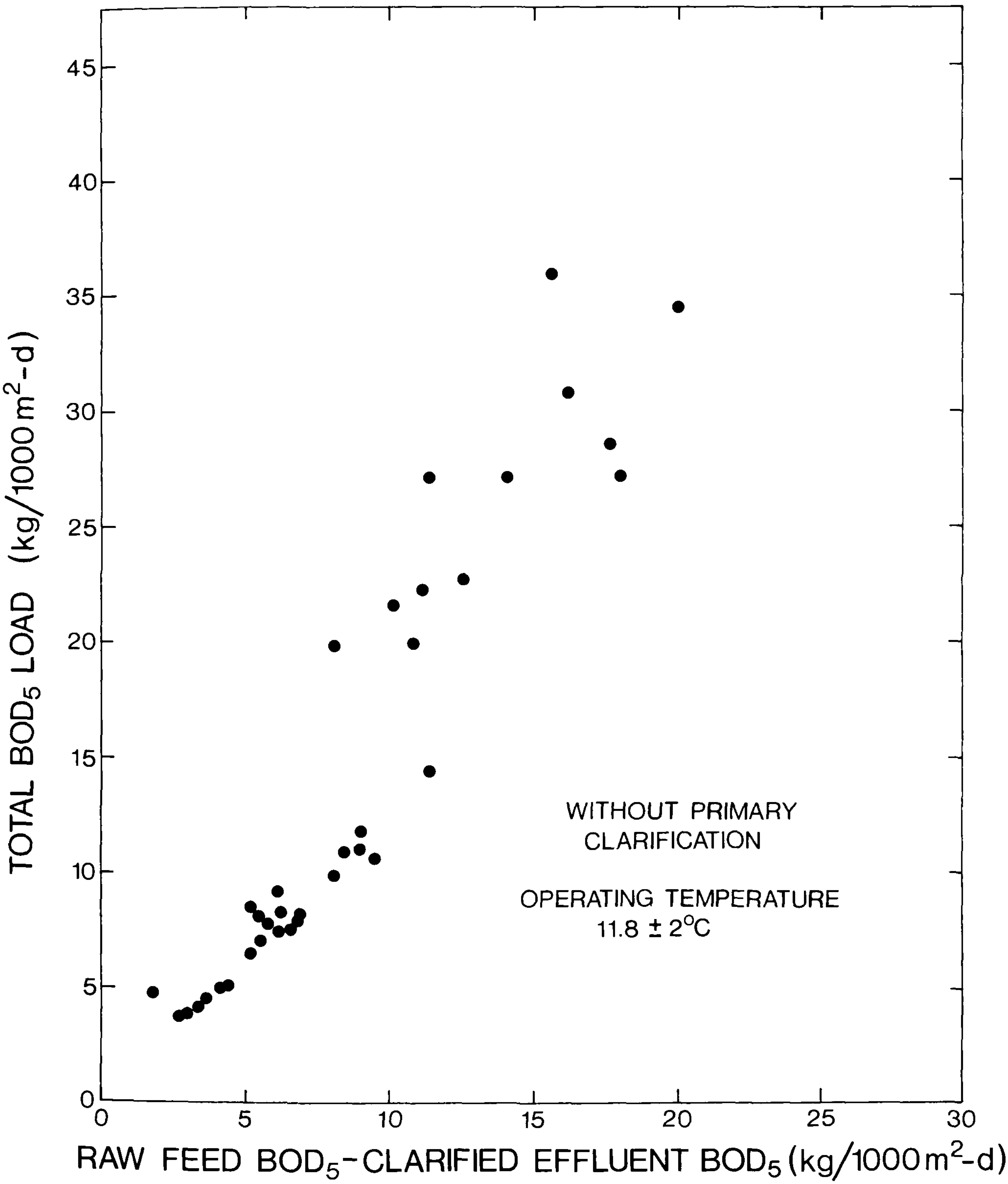

FIGURE 4-1 TOTAL BOD_5 LOADING vs TOTAL BOD_5 REMOVED (2.0 m RBC)

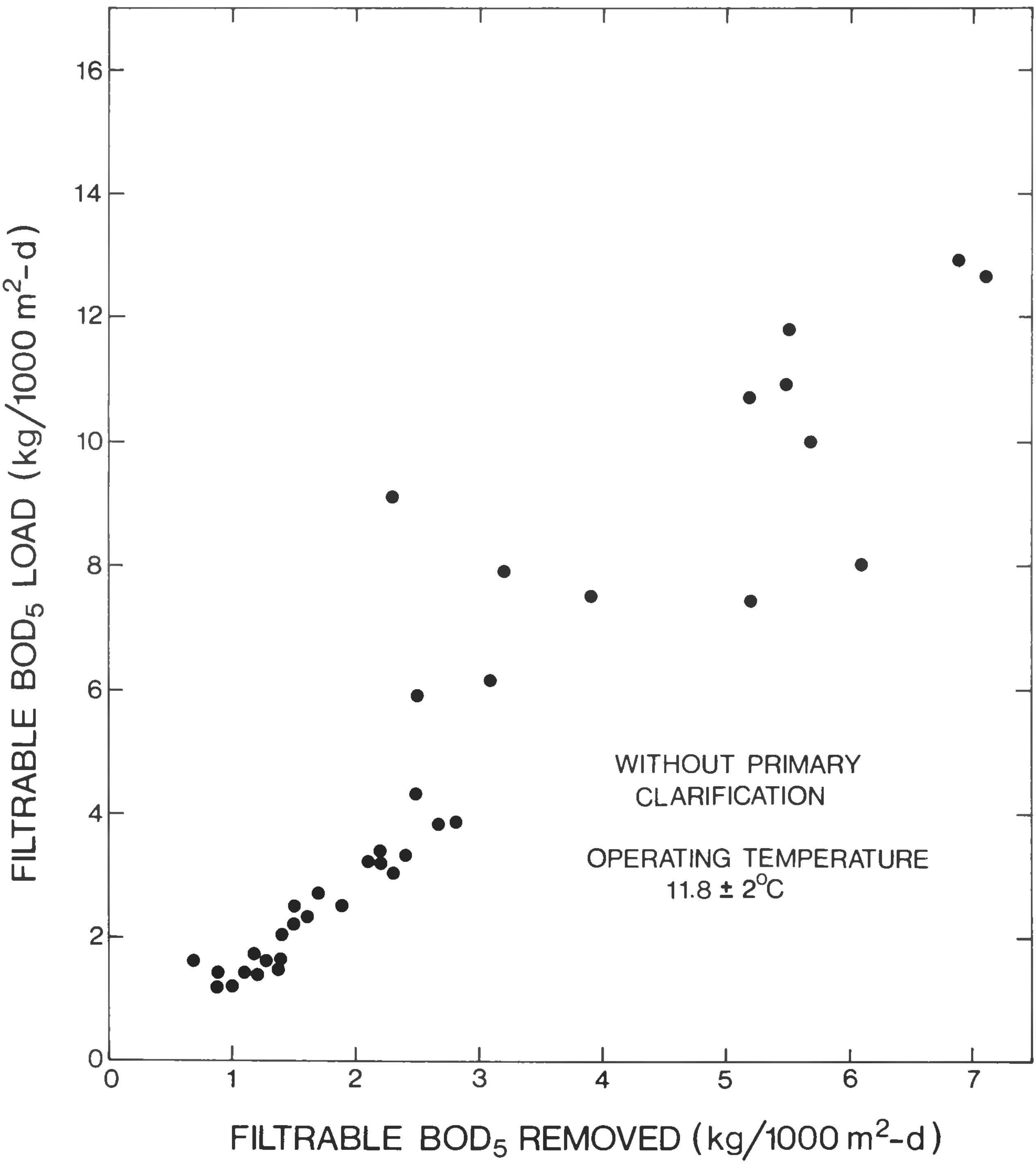

FIGURE 4-2 FILTRABLE BOD_5 LOADING vs FILTRABLE BOD_5 REMOVED (2.0 m RBC)

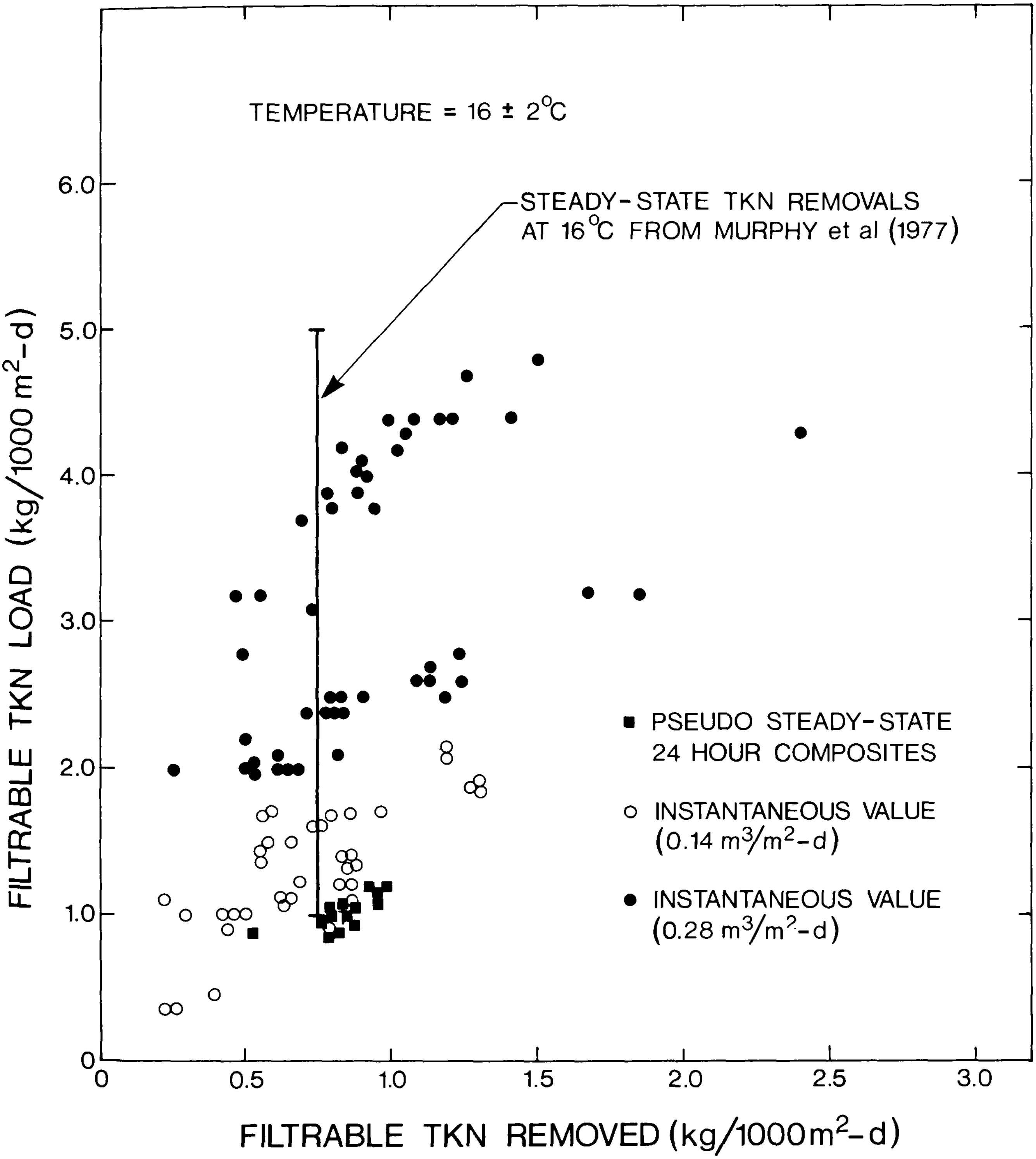

FIGURE 4-3 FILTRABLE TKN LOADING vs FILTRABLE TKN REMOVED (2.0 m RBC)

2.0 m unit for the time series analysis described previously in Section 3. The average temperature during this run was 16°C. The calculations for instantaneous mass loading and mass removals of TKN were based on a lag separation of one hour between influent and effluent. This one-hour separation represented the approximate time required for a concentration peak to pass through the RBC, as measured by dye tracer analysis. Since only steady-state data were of interest, observed data from periods within two hours following a step change in feed conditions were omitted from the analysis.

The 2.0 m RBC operated under nitrifying conditions for a five-week period in May and June, during which the average temperature was 16°C. The calculations for the mass loadings and mass removals of TKN were based on data from 24-hour composite samples.

Although the data display considerable scatter, relatively high percentage TKN removal efficiencies were achieved at low loadings. Almost all removal efficiencies found in analysis of the 24-hour composite samples were greater than 80 percent, (up to a maximum loading of 1.2 kg TKN/1000 m^2•d) and similar to the best found in analysis of the instantaneous data for the same loading range. Removal efficiencies decreased when the loading was increased beyond 1.0 to 1.5 kg/1000 m^2•d. The average TKN removals observed in this study compare favourably to the TKN removal rate reported by Murphy et al (1977) for combined BOD_5 removal plus nitrification with a RBC treating similar wastewater at the same temperature.

4.3.2 Mass Loading versus Effluent Quality. Total BOD_5 loadings are plotted against clarifier effluent BOD_5 in Figure 4-4 and against clarifier effluent suspended solids concentrations in Figure 4-5. The data show that both the concentrations of BOD_5 and suspended solids in the clarifier effluent increased with the total loading. At the lowest loadings, the suspended solids in the clarifier effluent fell to a minimum range of between 20 and 40 mg/L. At loadings less than 15 kg BOD_5/1000 m^2•d, the effluent suspended solids averaged 30 mg/L and were less than 50 mg/L approximately 80 percent of the time.

Total and filtrable BOD_5 loadings are plotted in Figure 4-6 against filtrable BOD_5 in the treated effluent. Effluent filtrable BOD_5 concentrations were consistently equal to or less than 10 mg/L at a total BOD_5 loading of 10 kg/1000 m^2•d or less. This corresponds to a filtrable BOD_5 loading of 3 kg/1000 m^2•d. The data in Figure 4-7 demonstrate that each mg of SS in the final settled effluent exerted approximately 0.3 mg of BOD_5. Consequently, an effluent containing 15 mg/L each of BOD_5 and SS could be

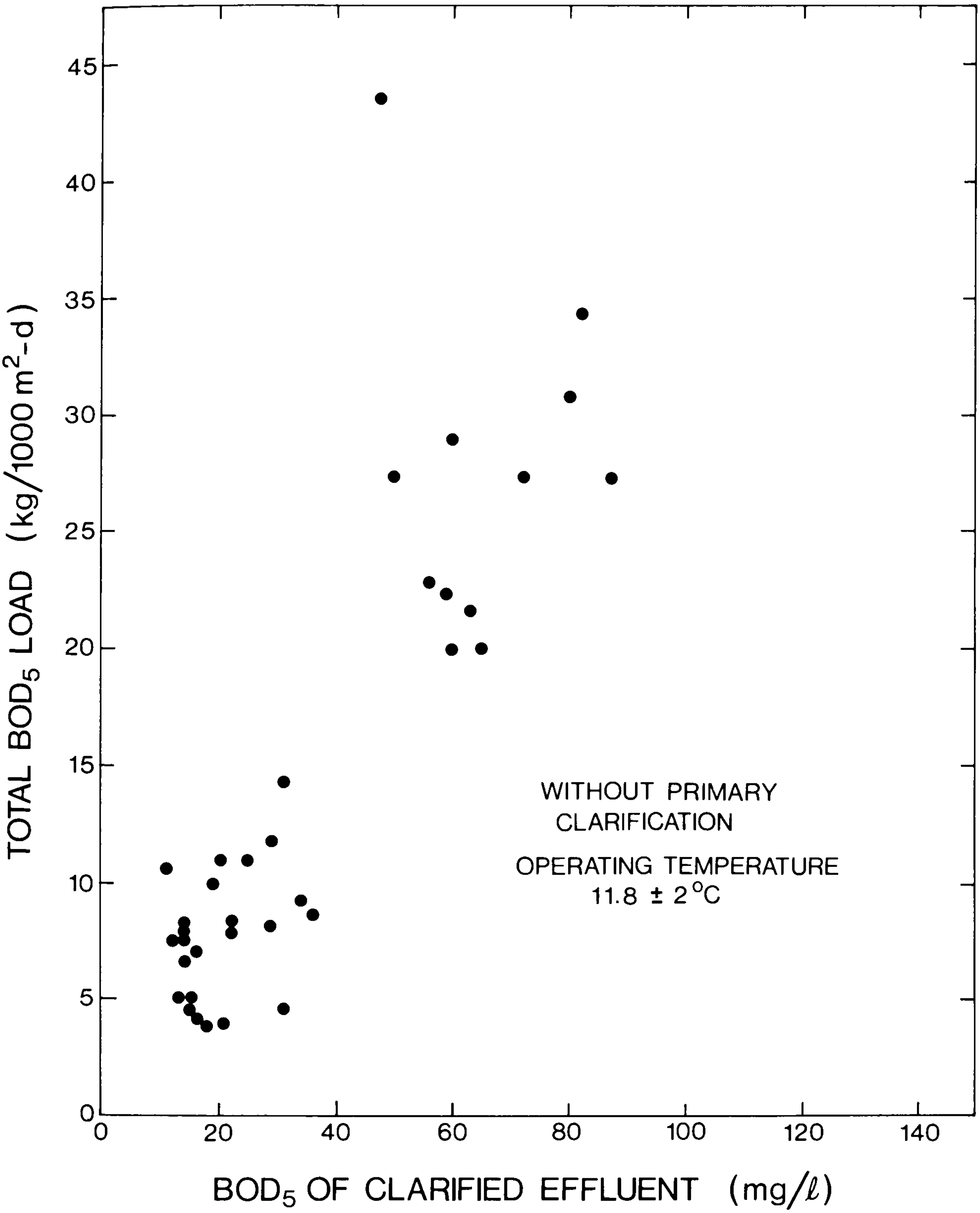

FIGURE 4-4 TOTAL BOD_5 LOADING vs CLARIFIED EFFLUENT BOD_5 CONCENTRATION (2.0 m RBC)

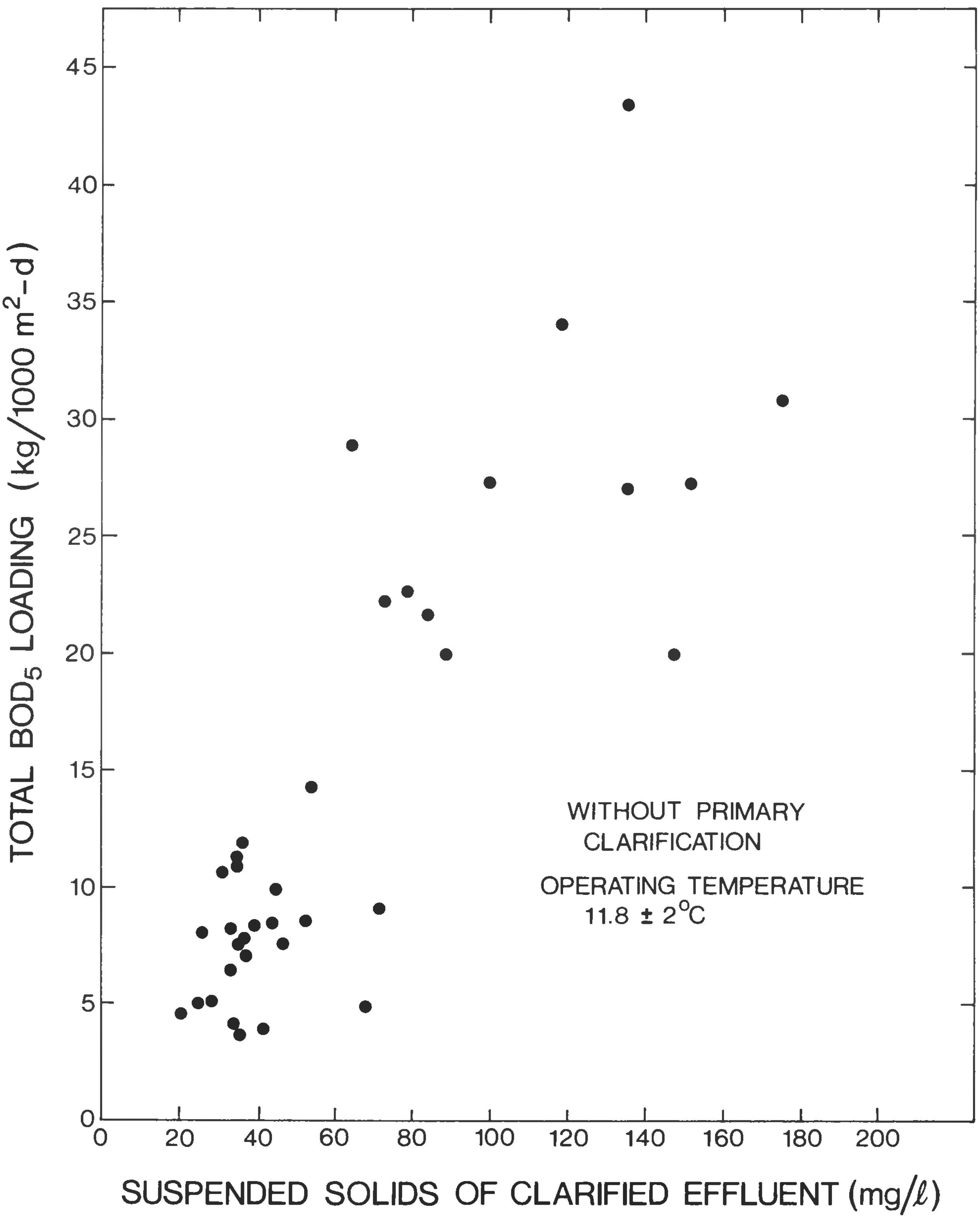

FIGURE 4-5 TOTAL BOD_5 LOADING vs CLARIFIER EFFLUENT SUSPENDED SOLIDS (2.0 m RBC)

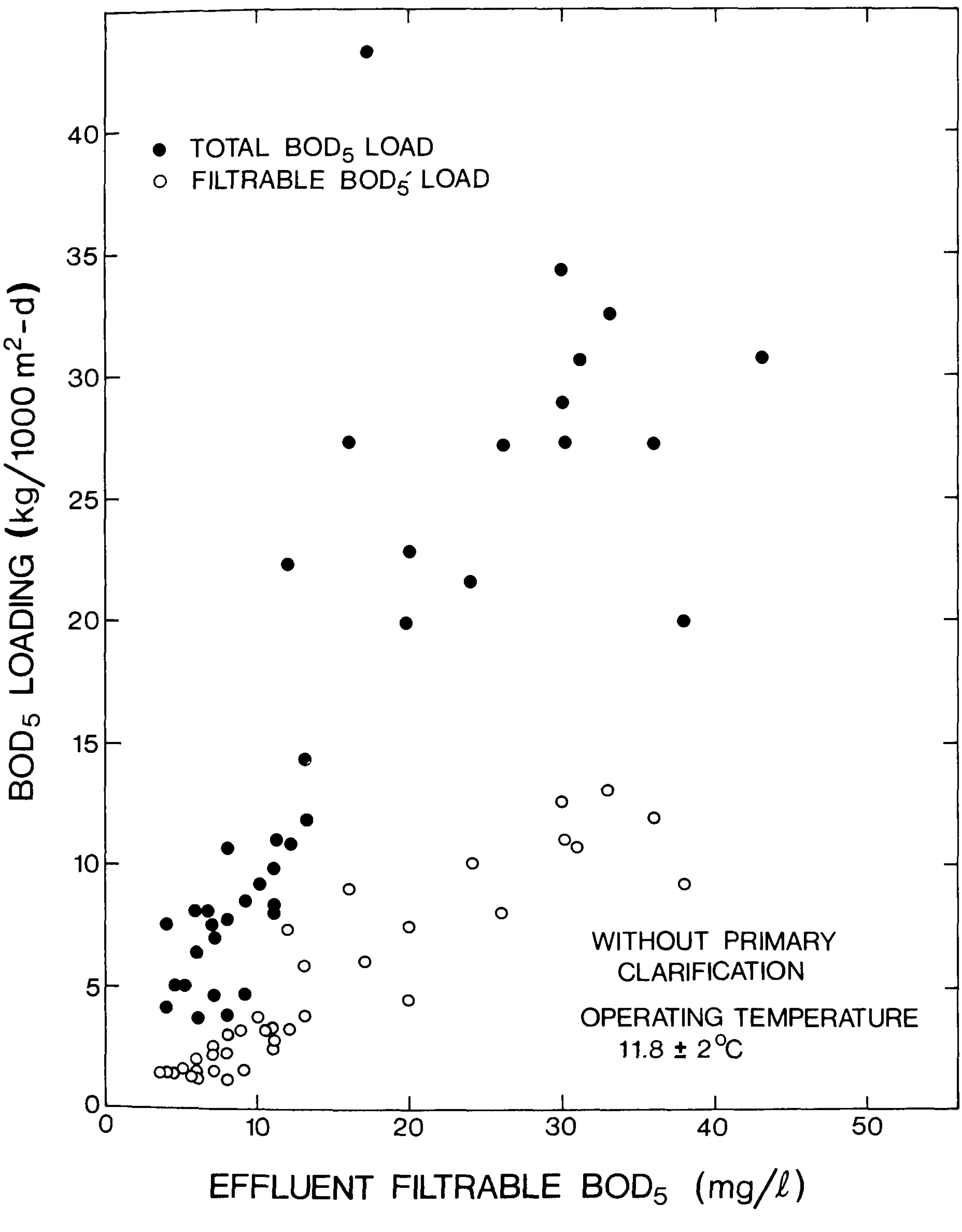

FIGURE 4-6 BOD_5 LOADING vs FILTRABLE EFFLUENT BOD_5 (2.0 m RBC)

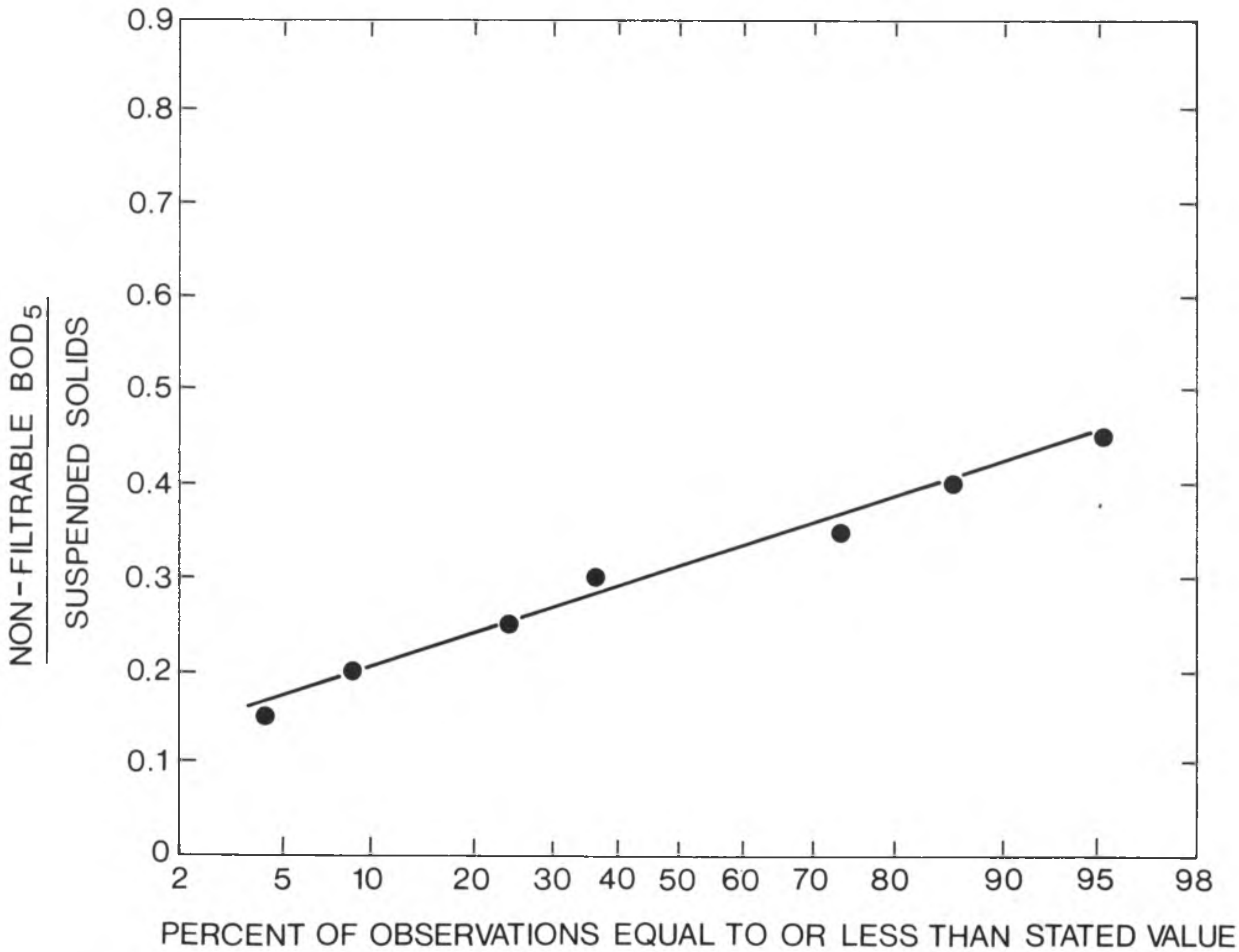

FIGURE 4-7 RATIO OF NON-FILTRABLE BOD_5 TO SUSPENDED SOLIDS IN SETTLED EFFLUENT (2.0 m RBC)

achieved at a total BOD_5 loading of 10 kg/1000 m^2•d, if some method could be found to further reduce the suspended solids concentration from those indicated in Figure 4-5. Similarly, the filtrable BOD_5 in the treated effluent at a total BOD_5 loading of 15 kg/1000 m^2•d, was approximately 16 mg/L. This allowed a final effluent of 25/25 for BOD_5 and suspended solids if efficient solids separation was possible. This indicates that the effluent quality during RBC treatment may be limited more by solids' settleability than by BOD_5 removal.

4.3.3 Clarifier Effluent versus Settled Effluent. Suspended solids' determinations were made on treated RBC effluent samples after 30 minutes of quiescent settling in an Imhoff Cone in addition to the measurement of suspended solids in the clarifier effluent. In almost all cases, better solids removal was achieved by the clarifier with an overflow of 0.6 m/h than with the 30-minute settling tests. This is demonstrated in the probability diagram in Figure 4-8 which shows that the suspended solids in the clarifier effluent were, on the average, 36 percent lower than in the settled effluent. Contrary to some views, this suggests that conservative overflow rates may be necessary for final clarification of RBC effluent if maximum removals of suspended solids are required.

4.3.4 Effect of Primary Clarification on Effluent Quality. The 0.5 m RBC was operated for 16 weeks in parallel with the 2.0 m unit during the winter. For the first two weeks of the program the 0.5 m RBC was fed raw degritted wastewater similar to the 2.0 m unit to provide comparative data before the installation of a primary clarifier.

The concentration of suspended solids in 30-minute settled samples of treated effluent versus BOD_5 loading are plotted for both RBC units in Figure 4-9. Although the data from the 0.5 m RBC were limited, there was no distinct difference found between the suspended solids concentrations in either the effluents of the 0.5 m or the 2.0 m RBC.

The primary clarifier was installed ahead of the 0.5 m RBC following the initial control period. In order to determine the effect of the primary clarifier on the effluent quality, probability plots were prepared using data obtained from experimental runs of both units, which were conducted under similar BOD_5 and hydraulic loadings, and at similar temperatures. Table 4-1 summarizes these data.

The data for suspended solids in the treated settled effluents are presented in Figure 4-10. The average solids' concentration in the settled effluent of the 0.5 m unit was approximately half that of the 2.0 m RBC. The comparison of settled BOD_5 concentrations (Figure 4-11) shows a similar advantage for the system with primary clarification. The data clearly show that the use of a primary clarifier with the smaller

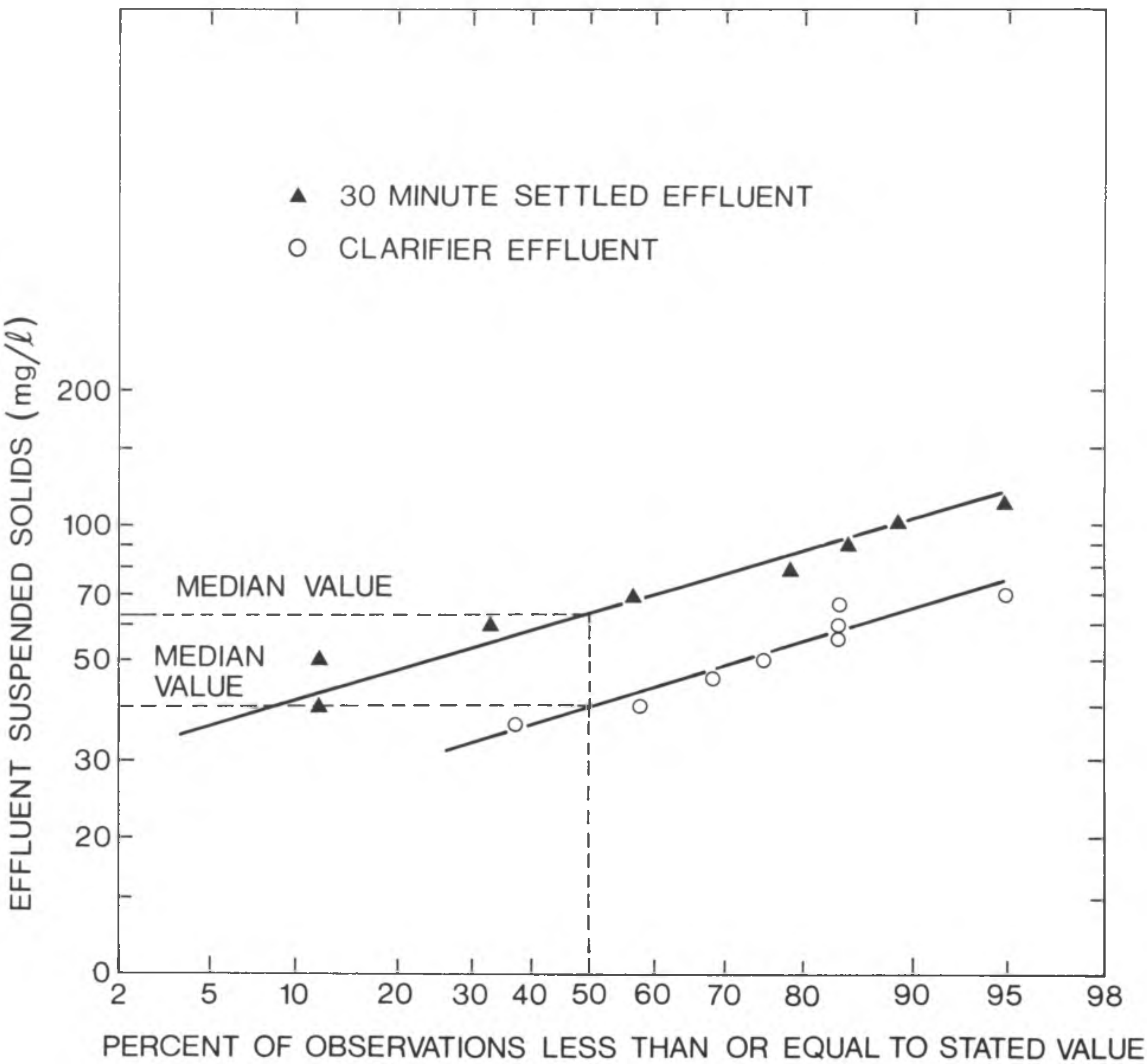

FIGURE 4-8 EFFLUENT SUSPENDED SOLIDS CONCENTRATION: CLARIFIER EFFLUENT vs 30-MINUTE SETTLED EFFLUENT (2.0 m RBC)

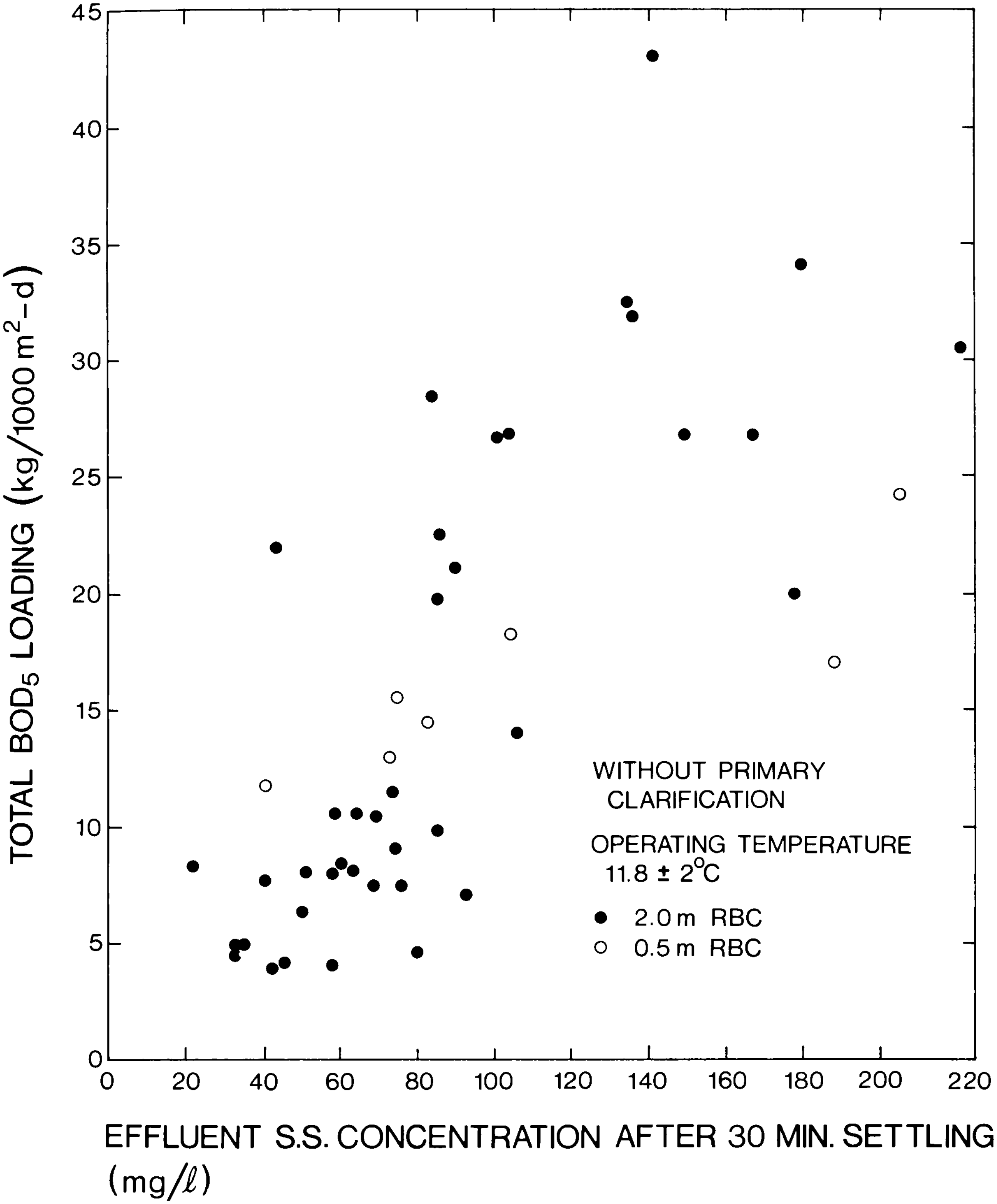

FIGURE 4-9 TOTAL BOD_5 LOADING vs EFFLUENT SS CONCENTRATION

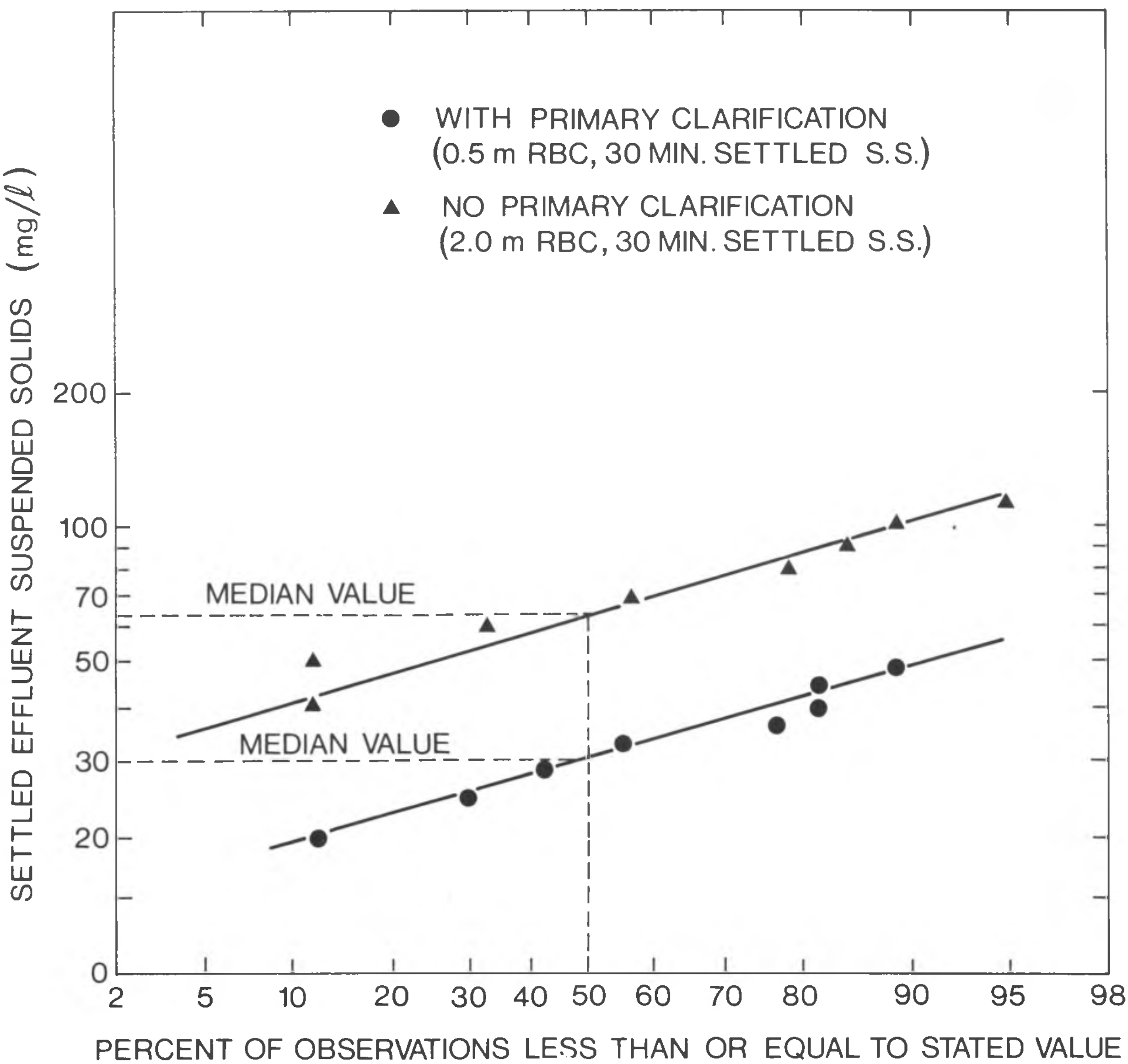

FIGURE 4-10 EFFLUENT SUSPENDED SOLIDS CONCENTRATION WITH AND WITHOUT PRIMARY CLARIFICATION

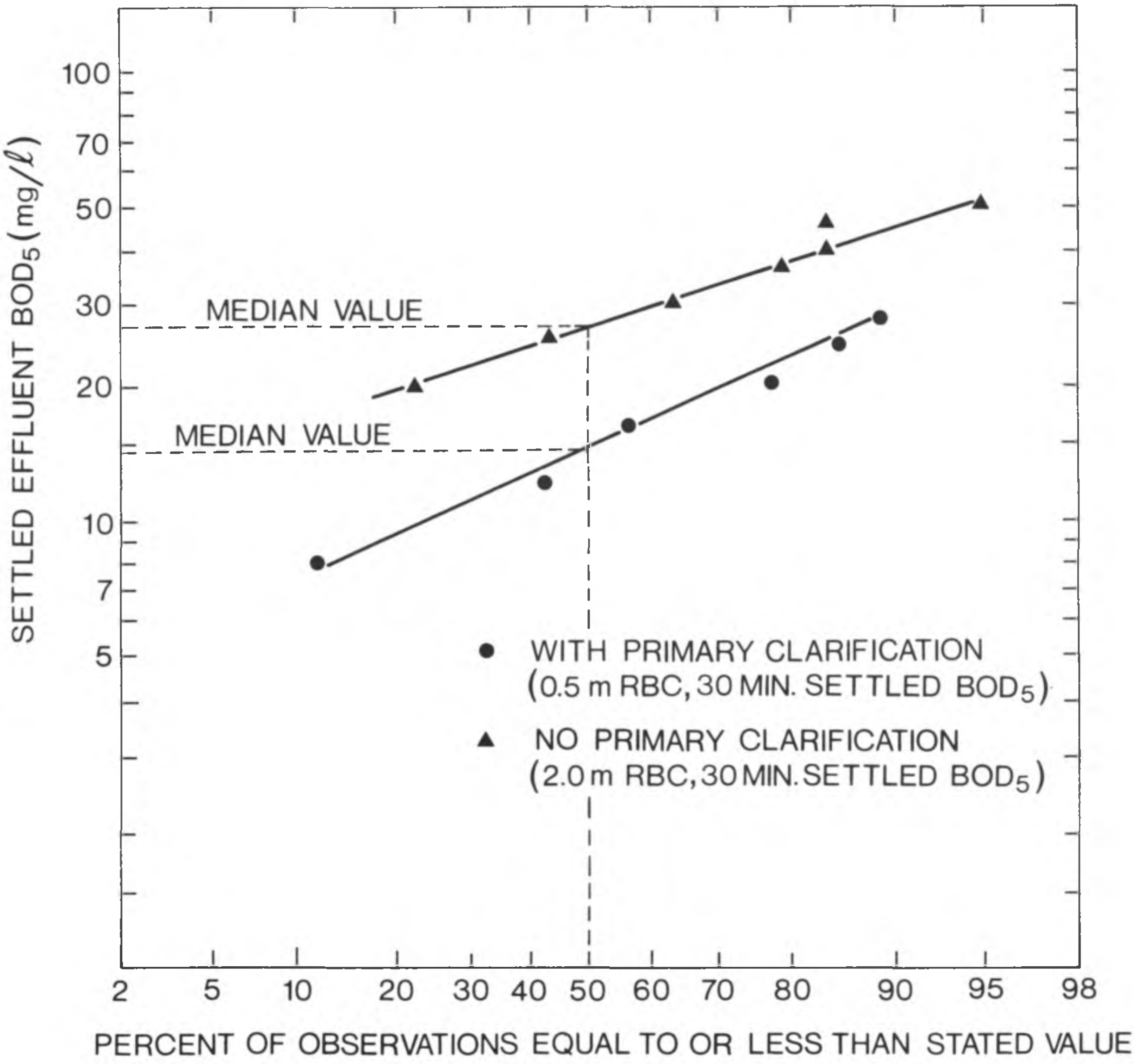

FIGURE 4-11 EFFLUENT BOD_5 CONCENTRATION WITH AND WITHOUT PRIMARY CLARIFICATION

TABLE 4-1 THE EFFECT OF PRIMARY CLARIFICATION ON EFFLUENT QUALITY (Operating Conditions During Experimental Runs)

		With Primary Clarification	Without Primary Clarification
Dates of Runs		March 1-May 5, '77	March 9-April 24, '77
Pilot Unit		0.5 m RBC	2.0 m RBC
BOD_5 Loading to RBC:	Mean	8.0	8.8
(kg/1000 m^2•d)	Range	3-14	4-13
Hydraulic Load (m^3/m^2•d)		0.06 - 0.12	0.09
Operating Temperature (°C)		12 -15	10 - 14

RBC improved treated effluent quality compared to the 2.0 m unit operating under similar conditions. Two factors could have contributed to this improvement. First, the concentration of suspended solids entering the smaller RBC was less than that for the 2.0 m unit; the primary clarifier removing approximately 30 percent of the suspended solids in the degritted wastewater. The median concentration of solids entering the 2.0 m unit was 175 mg/L, 55 mg/L higher than in the case of the 0.5 m unit, and consequently, the filtrable BOD_5 load to the smaller RBC was higher. Second, the treatment efficiency may have been affected by their difference in scale, though a comparison of suspended solids during the initial control period (Figure 4-9) would suggest this was of lesser importance.

The best effluent obtained with the 2.0 m unit at the lower loadings and under the optimum settling conditions, still contained more than 20 mg/L of suspended solids. Following the completion of the planned experimental program, a primary clarifier was installed ahead of the 2.0 m unit in order to determine if lower effluent solids concentrations were possible. Data collected for three weeks, when the hydraulic loading was set at 0.10 m^3/m^2•d, were within the range shown in Table 4-1. For this period, the mean suspended solids' concentration in the final clarifier effluent was 20 mg/L, with a low of 12 mg/L and a high 32 mg/L, significantly lower than that obtained earlier without the primary clarifier. The concentration of suspended solids entering the RBC after primary clarification was 112 mg/L, or slightly lower than the median value observed in the feed to the 0.5 m unit during the first test with primary clarification. It should be mentioned, however, that during the second study using the larger RBC, the operating temperature was significantly higher than that of the first study. It is unlikely that temperature alone was responsible for the improvement in effluent quality. It would seem that primary suspended solids removal is necessary in order to achieve best possible effluent quality with RBC treatment.

4.4 Temperature Effects

During the four-month study described in this chapter the operating temperatures in the RBC's varied between 10 and 15°C, with an average of approximately 12°C. This is typical of winter temperatures for wastewater treatment in southern Ontario. There are, however, areas in Canada where winter temperatures are much lower than 12°C and to which these results cannot be directly applied without some consideration of the resulting reduced biological activity.

Little research has been conducted to determine the temperature sensitivity of the RBC process at temperatures below 12°C. Figure C-2 in the Autotrol Design Manual (Autotrol Corporation, 1976) indicates that for BOD_5 removal no temperature correction is necessary at operating temperatures equal to or greater than 13°C. Below this value the recommended design loading is decreased (e.g. at 6°C by 45 percent). These data can be used to calculate a temperature coefficient for the following expression.

$$K_T = K_{13}\,\Theta^{T-13}$$

where:
- K_T = the design loading at T°C
- K_{13} = the design loading at 13°C
- T = temperature in °C
- Θ = temperature coefficient

Using the Autotrol data, the value for Θ was calculated to be 1.05.

Murphy et al (1977) used a 0.5 m RBC, similar to the one in this study, to determine its temperature sensitivity during combined BOD_5 removal plus nitrification. An Arrhenius constant of 14200 cal/mole was found to describe the temperature response between 7°C and 25°C, and from it a temperature coefficient for a similar temperature range to that for BOD_5 removal can be calculated. The coefficient derived was 1.1. These temperature coefficients are representative of similar ones used in other biological processes for BOD_5 and for combined BOD_5 removal plus nitrification (Eckenfelder, 1970; Wilson, 1975).

4.5 Summary

The pseudo steady-state operating results indicate that RBC systems should include a process for primary solids removal. At similar BOD_5 loadings the concentration of non-settleable suspended solids in the clarified effluent was consistently less when primary clarified wastewater, as opposed to raw degritted wastewater, was fed the RBC. The best average effluent quality observed in this study, using raw degritted wastewater as feed to the RBC, contained 20 mg/L of total BOD_5 and 30 mg/L of suspended solids.

Final treated effluent containing an average of 15 mg/L of both total BOD_5 and suspended solids was achieved with a total BOD_5 loading of 10 kg/1000 $m^2 \cdot d$, using primary clarified wastewater and operating at 12°C. At a loading of 15 kg total BOD_5/1000 $m^2 \cdot d$ the effluent would contain approximately 25 mg/L of both total BOD_5 and suspended solids.

The pseudo steady-state data (Section 4.3) indicate consistent filtrable TKN removals of 80 to 90 percent, at loadings equal to or less than 1.0 kg filtrable TKN/1000 $m^2 \cdot d$, when operating at a temperature of 16°C. This effectively results in complete nitrification.

The influence of operating temperature on biomass activity must be considered when selecting design loadings for RBC's. When it is desirable to maintain efficient year round wastewater treatment, design loadings should be selected using the temperature corrections discussed in Section 4.4.

5 EFFECT OF DIURNAL VARIATIONS

5.1 Experimental Program

So that a comprehensive understanding of process reliability may be developed, the response of RBC's to typical diurnal changes in wastewater variables, such as flow, BOD_5 and nitrogen concentration, must be determined. This is of particular importance for the RBC, as the data analysis in Section 3.0 has already shown that RBC effluent quality responds very rapidly to changes in influent variables.

In a series of experimental runs in which two 0.5 m RBC units were operated in parallel with degritted raw wastewater as a common feed source, a flow controller was used to provide a sinusoidal hydraulic loading to one of the units. It was set at a 24-hour cycle, with a peak to mean to minimum flow ratio of 2:1:0.5. The peak occurred at about 6 p.m., and the minimum flow between 5 and 7 a.m. This pattern and magnitude of flow is typical of diurnal variations at many medium-sized wastewater treatment plants. The flow variation for this study was based on representative flow data from the Burlington Skyway Water Pollution Control Plant, also the source of the degritted raw feed. A review of existing data revealed that flow peaks at the plant coincided with daily concentration peaks for COD and TKN, so that the daily variation in COD and TKN mass loading to the RBC would be even greater than indicated by the flow variation alone.

The sinusoidal flow was fed to the test RBC several days before sampling started in order to allow the system to achieve dynamic equilibrium. The control unit worked at a constant flow.

5.2 Results and Discussion

5.2.1 Hourly Variability During Diurnal Operation. Samples of influent and effleunt from the test unit were collected hourly for two periods of two days' duration. During the first period the hydraulic load to the RBC was varied between 11×10^{-2} and 42×10^{-2} $m^3/m^2{\cdot}d$, with the average loading being 21×10^{-2} $m^3/m^2{\cdot}d$. The average flow-proportioned filtrable COD loading was 20 kg/1000 $m^2{\cdot}d$, and the average total BOD_5 loading was 22 kg/1000 $m^2{\cdot}d$.

Loads were reduced during the second test period so that the effects of diurnal variations on the RBC operating in a combined BOD_5 removal plus nitrification mode could also be determined. During the second phase of the program the mean hydraulic loading was 6.7×10^{-2} $m^3/m^2 \cdot d$. The average flow-proportioned loadings for total BOD_5, filtrable COD and filtrable TKN, were approximately 9, 7 and 1.4 kg/1000 $m^2 \cdot d$, respectively.

Data showing the observed hourly changes in loadings and treatment efficiencies during the two dynamic studies are presented in Figure 5-1 and Figure 5-2. Figure 5-1 shows the hydraulic and COD loading variations during the BOD_5 removal study. At a peak to mean to minimum ratio of 3.7:1:0.3, the variation in COD loading to the unit was much higher than the variation induced by the changing flow alone, indicating that the concentration peak for COD occurred at the same time as the peak flow. The average hydraulic residence in the system was less than 40 minutes, which provided very little opportunity for the pilot plant to equalize peak loads, and therefore it is not surprising that large variations in effluent quality were observed. As was seen in the time series analysis, there was little or no lag between input and output as the peaks passed through the system.

Approximately 40 to 50 mg/L of the filtrable COD entering in the raw feed was refractory to biological treatment. It is apparent, if one subtracts 40 mg/L of COD from observed influent COD concentrations, that the biologically available COD in the raw feed could vary from almost zero to over 150 mg/L within a few hours. During the same period the available COD in the treated effluent varied from zero to approximately 100 mg/L. Notwithstanding the large variation in influent and effluent COD concentrations, flow-proportioned 24-hour composite samples collected during the test period showed that the average treatment efficiency, measured as BOD_5 removal, was better than might be anticipated from Figure 5-1. For the two-day test, the average flow-proportioned BOD_5 in the feed was 109 mg/L, while the BOD_5 in settled treated effluent was 30 mg/L. This resulted in a removal efficiency of 72 percent.

Figure 5-2 shows that diurnal variations in flow and concentration had a significant effect on effluent TKN concentration during the nitrification run. The instantaneous mass loading of TKN varied by an order of magnitude during each 24-hour period similar to the variation previously seen in the COD loading. Although the average TKN in treated effluent varied between 1.0 and 16.0 mg/L, in this case the peak concentration of TKN in the effluent lagged the peak concentration in the raw feed by one or two hours. The mean hydraulic flow to the RBC during the nitrification study was only one-third that in the BOD_5 removal study, resulting in a mean residence time of approximately two hours. Dye studies showed that concentration peaks would be observed in the effluent after about one hour.

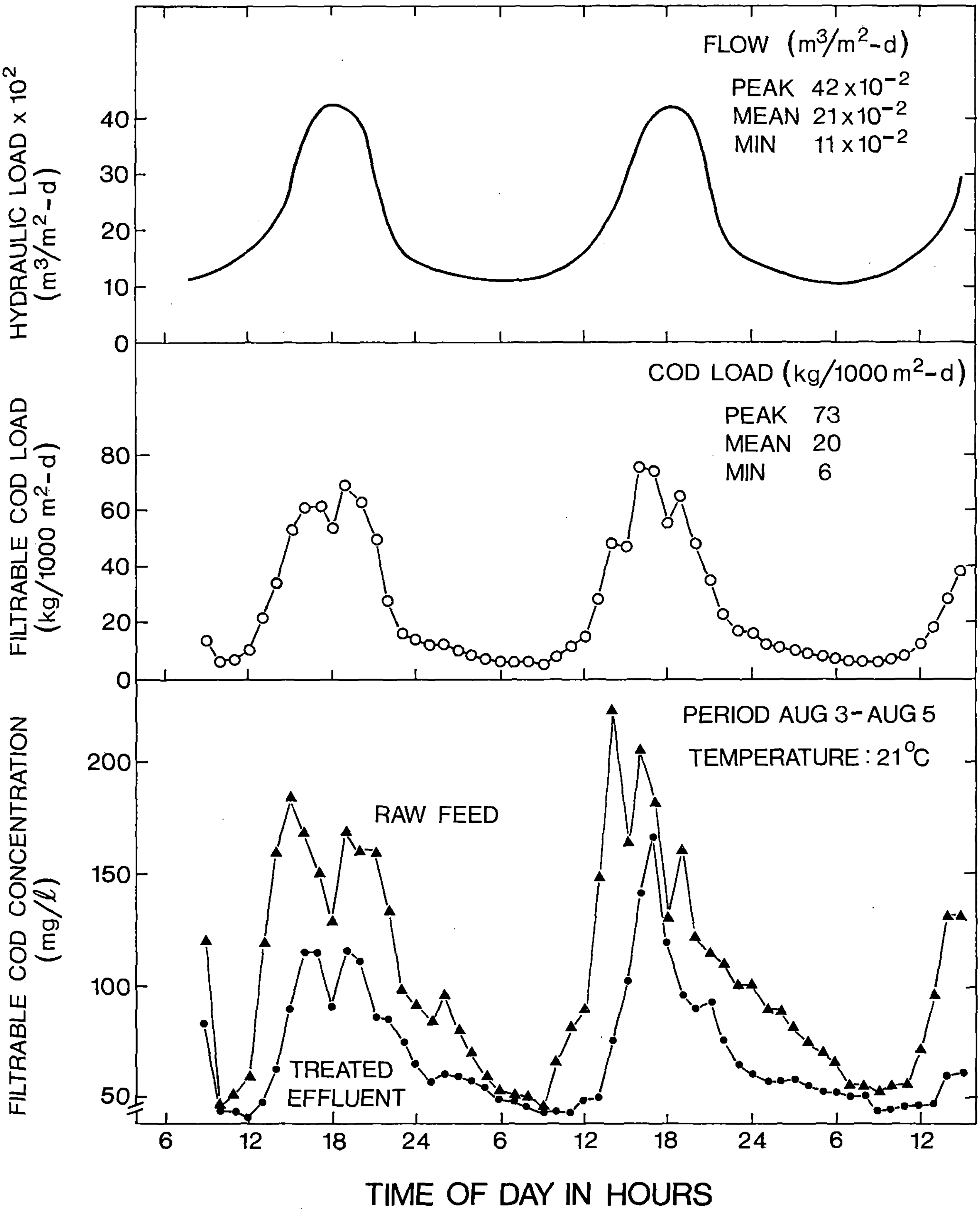

FIGURE 5-1 EFFECT OF DIURNAL FLOW AND CONCENTRATION VARIATION ON RBC PERFORMANCE (0.5 m RBC in the Carbon Removal Mode)

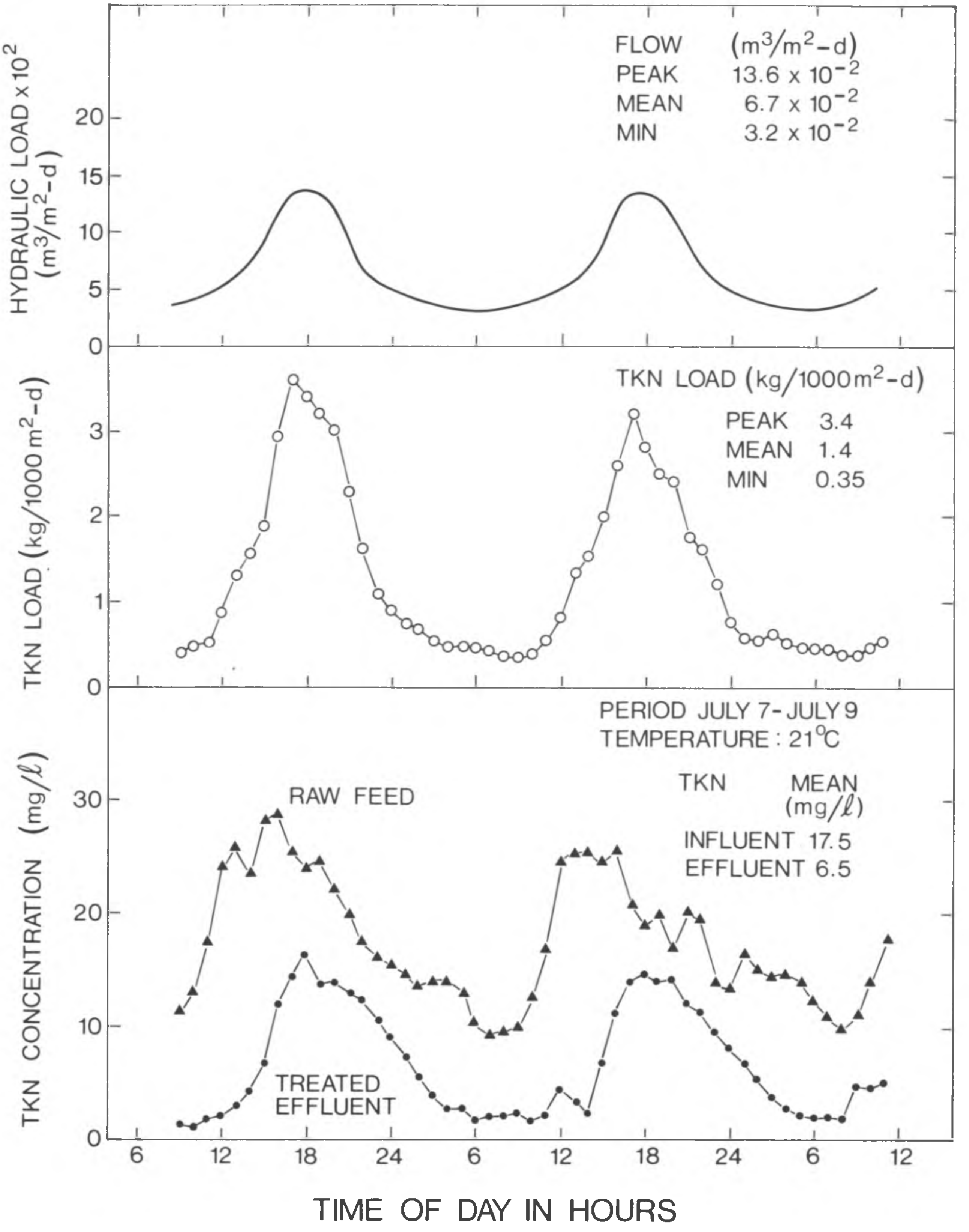

FIGURE 5-2 EFFECT OF DIURNAL FLOW AND CONCENTRATION VARIATION ON RBC PERFORMANCE (0.5 m RBC in the Carbon Removal Plus Nitrification Mode)

The filtrable COD concentration in the treated effluent during the nitrification study varied between approximately 45 and 55 mg/L, with only a few values falling outside this range. This is in sharp contrast to the data in Figure 5-1. The two-thirds decrease in hydraulic loading caused a corresponding three fold increase in hydraulic equalizing capacity. This was apparently sufficient to smooth out the variations in effluent COD concentration. The elimination from the effluent of the hourly COD peaks coincided with an increase in average BOD_5 removal of 72 to 85 percent.

5.2.2 Effect of Flow Variation Alone. Six sets of flow-proportioned 24-hour composite samples were collected from the test and control RBC's during the BOD_5 removal study. These were analyzed for BOD_5, filtrable organic carbon (TOC), filtrable COD and filtrable TKN. The diurnal sinusoidal variation in flow rate represented the only significant operating difference between the two units.

Table 5-1 is a summary of the analytical results. Asterisks in the last column indicate where differences in the operating results were statistically significant to a confidence level of 95 percent.

TABLE 5-1 THE EFFECT OF DIURNAL FLOW VARIATIONS: BOD_5 REMOVAL MODE

RBC Unit	Mean Value		Difference
	Test	Control	(Test-Control)
Raw Wastewater Characteristics			
BOD_5 (mg/L)	112	97	15*
TOC (mg/L)	31	27	4*
Filtrable TKN (mg/L)	23	20	3*
Pilot Plant Loading			
Mean Hydraulic ($m^3/m^2 \cdot d$)	21×10^{-2}	21×10^{-2}	
Mean BOD_5 ($kg/1000\ m^2 \cdot d$)	24	20	
Peak:Mean:Min. Flow	2:1:0.5	Constant	
Treated Effluent Characteristics			
BOD_5 (30 min. settled, mg/L)	32	25	7
Filtrable COD (mg/L)	64	61	3
TOC (mg/L)	21	19	2*

* Test parameter is greater than control parameter at a confidence level of 95%.

The feed to the test RBC contained significantly higher concentrations of BOD_5 and TOC than the feed to the control RBC. The TKN concentration was also significantly higher during the BOD_5 removal study. The average increase in BOD_5 and TKN concentrations were approximately 20 percent and 15 percent respectively. This is significant because flow-proportional sampling is not often done in the field, and this could lead to under-estimation of the total organic or nitrogen loads requiring treatment.

The observed hydraulic loads for the test RBC were equal to or slightly lower than those for the control pilot plant, but because of the sinusoidal variation in flow to the test unit, the BOD_5 and TKN loadings were slightly higher. These data explain the lower effluent quality from the test unit compared to that from the control. The concentration of TOC in the treated effluent was slightly less in the control than in the test RBC. It is not known whether these differences would have been observed had the average TOC loads to the two systems been identical.

Table 5-2 is a summary of similar results from five sets of 24-hour composite samples, from a combined BOD_5 removal plus nitrification operating mode. During the test, TKN loadings to the two RBC's were equal. Effluent quality deteriorated in the RBC subjected to the sinusoidal flow variation. In the test unit the average effluent TKN and TOC concentrations were higher, and the $NO_3 + NO_2$-N concentration was lower, indicating less efficient nitrification. The average TKN removal efficiency in the test unit was 58 percent compared to 71 percent in the control RBC. In both cases, only 50 percent of the filtrable TKN removed appeared in the effluent as $NO_3 + NO_2$-N. The nitrogen uptake of the biomass could account for only a part of this discrepancy. This is discussed further in the section on RBC scale-up (Section 8).

At the low loading used during the BOD_5 removal/nitrification test most of the degradable organic materials would be oxidized in the first or second stage of treatment, thus allowing nitrifiers to become a significant fraction of the biomass in the final stages. Therefore, it is not surprising that the settled BOD_5's in the treated effluent of the test and the control RBC's were the same. There was, however, a significant difference in TOC concentrations in favour of the control unit.

In summary, there was a small but significant deterioration in average effluent quality when the RBC was subjected to a diurnal flow variation in comparison to a steady flow operation. The data in Table 5-2 show that, at the same average loading, the average TKN mass removal in the test RBC was approximately 20 percent less than that in the control RBC. The reduction in treatment efficiency during the BOD_5 removal test

TABLE 5-2 EFFECT OF DIURNAL FLOW VARIATIONS: BOD_5 REMOVAL PLUS NITRIFICATION MODE

RBC	Mean Value Test	Mean Value Control	Difference (Test-Control)
Raw Wastewater Characteristics			
BOD_5 (mg/L)	114	92	22*
TOC (mg/L)	40	33	7*
Filtrable TKN (mg/L)	17	16	1
$NO_3 + NO_2$-N (mg/L)	0.4	0.3	0.1
Pilot Plant Loadings			
Mean Hydraulic ($m^3/m^2 \cdot d$)	6.5×10^{-2}	7.1×10^{-2}	
Mean TKN (kg/1000 $m^2 \cdot d$)	1.1	1.1	
Peak:Mean:Min. Flow	2:1:0.5	Constant	
Temp. (°C)	21	21	
Treated Effluent Characteristics			
BOD_5 (30 min. settled, mg/L)	17	16	1
TOC (mg/L)	19	16	3*
Filtrable TKN (mg/L)	7.2	4.7	2.5*
$NO_3 + NO_2$-N (mg/L)	4.8	5.8	-1.0*

* Test parameter is greater than control parameter to a confidence interval of 95%.

(summarized in Table 5-1) was not as well defined, although the diurnal flow pattern for this operating mode, seemed to have less of an effect on effluent quality.

5.2.3 Predicting Effluent Quality With Time Series. The hourly data collected during the two intensive sampling periods of the diurnal studies offer independent sets of results that can be used to assess the usefulness of the predictive models developed in Section 3. Dynamic models for the 0.5 m RBC were developed at the same time as those for the 2.0 m units. These transfer function-noise models (Table 5-3) are similar in form to those for the larger RBC, although the parameter values differ because of the differences in size. Because they were developed using data from a half-hour sampling interval, it was necessary to transform the models to forms that could accommodate the hourly data from the diurnal studies.

TABLE 5-3 TRANSFER FUNCTION-NOISE MODELS FOR THE 0.5 m RBC

Mode	Variables	Model
A. Models Derived from Data at 1/2 Hour Intervals		
Carbon Removal	Y_t = effluent TOC (mg/L) X_t = TOC load (g/d)	$Y_t = (0.0283 + 0.0050\ \beta)\ \beta\ X_t + \frac{(1 - 0.258\ \beta)}{(1 - \beta)}\ a_t$
Carbon Removal Plus Nitrification	Y_t = effluent TKN (mg/L) X_t = TKN load (g/d)	$Y_t = \frac{(0.0349 + 0.0140\ \beta)}{(1 - 0.75\ \beta)}\ \beta\ X_t + \frac{a_t}{(1 - \beta)}$
B. Models Transformed to Accept Hourly Interval Data		
Carbon Removal	Y_t = effluent TOC (mg/L) X_t = TOC load (g/d)	$Y_t = 0.0332\ \beta\ X_t + \frac{a_t}{(1 - \beta)}$
Carbon Removal Plus Nitrification	Y_t = effluent TKN (mg/L) X_t = TKN load (g/d)	$Y_t = \frac{(0.075 + 0.011\ \beta)}{(1 - 0.56\ \beta)}\ \beta\ X_t + \frac{a_t}{(1 - \beta)}$

In Figure 5-3 (the carbon removal mode) and in Figure 5-4 (the carbon removal/nitrification mode) the observed and predicted effluent quality are compared. The response and recovery times predicted by these models agree very closely with the experimental data, despite the order of magnitude variations in instantaneous TOC and TKN loadings. Predicted peak effluent TKN concentrations during the nitrification run were slightly lower and appear to lag the observed effluent peak by one to two hours.

Effluent TOC concentrations are extremely variable, and reflect the continuously changing influent conditions. It should be noted that sudden short duration decreases in TOC loadings occurred around hour 9 and again around hour 58. The process response was an immediate decrease in effluent TOC which was accurately predicted by the model. The close agreement between the predicted and observed responses demonstrates the usefulness of time series analysis in evaluating the RBC process.

The transformed carbon removal model (Table 5-3) was used to predict treated effluent quality, using the TOC loadings from Figure 5-3 as input data. The model predicted that the 24-hour flow-proportioned TOC concentration in the treated effluent would be 21 mg/L. The average loading over the 24-hour period was 8 kg TOC/1000 $m^2 \cdot d$.

Although it is not mathematically rigorous to use time series models to predict steady-state operations, the model was used in an attempt to predict the effluent TOC concentration that would have been produced at a similar mean TOC load, but with no diurnal variation. Under these conditions the model predicted the steady-state TOC concentration in the treated effluent to be approximately 19 mg/L. This estimate seemed reasonable, considering the results of the pseudo steady-state control runs summarized in Table 5-1. A further prediction indicated that a flow-proportioned treated effluent containing 19 mg/L TOC could be achieved by subjecting the RBC to a diurnal flow if its media surface was increased by 25 percent. This would apply to a system with a peak to mean to minimum flow ratio of 2: 1: 0.5.

A similar analysis, using the TKN load model for combined carbon removal plus nitrification, indicated that a design factor of 1.35 (i.e. 35 percent increase in surface area) would be appropriate for applying steady-state results to diurnal flow situations. These design factors for carbon removal and for carbon removal plus nitrification are unnecessary when complete flow equalization is provided.

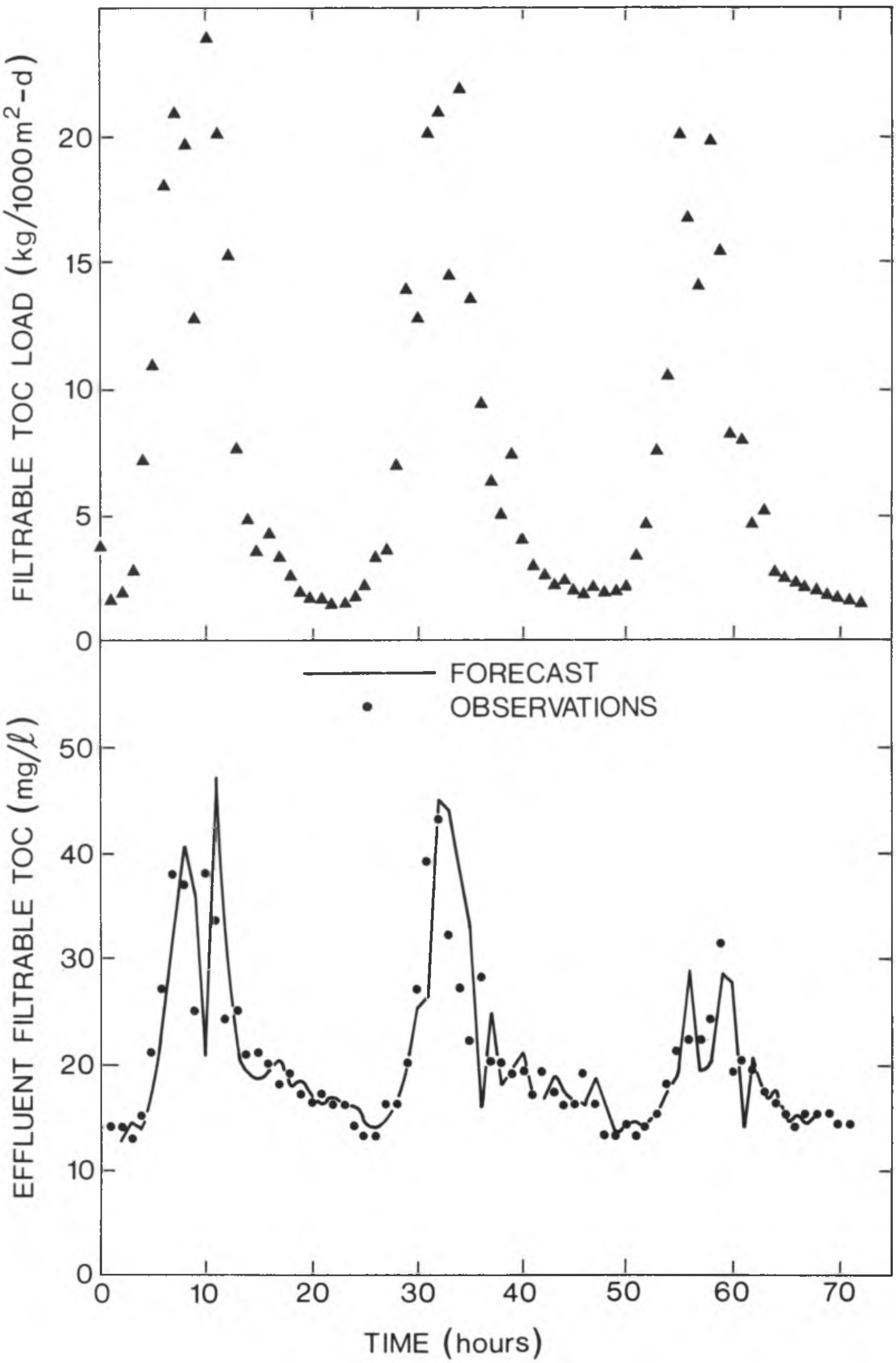

FIGURE 5-3 TRANSFER FUNCTION-NOISE MODEL PREDICTIONS OF DIURNAL VARIABILITY (0.5 m RBC in the Carbon Removal Mode)

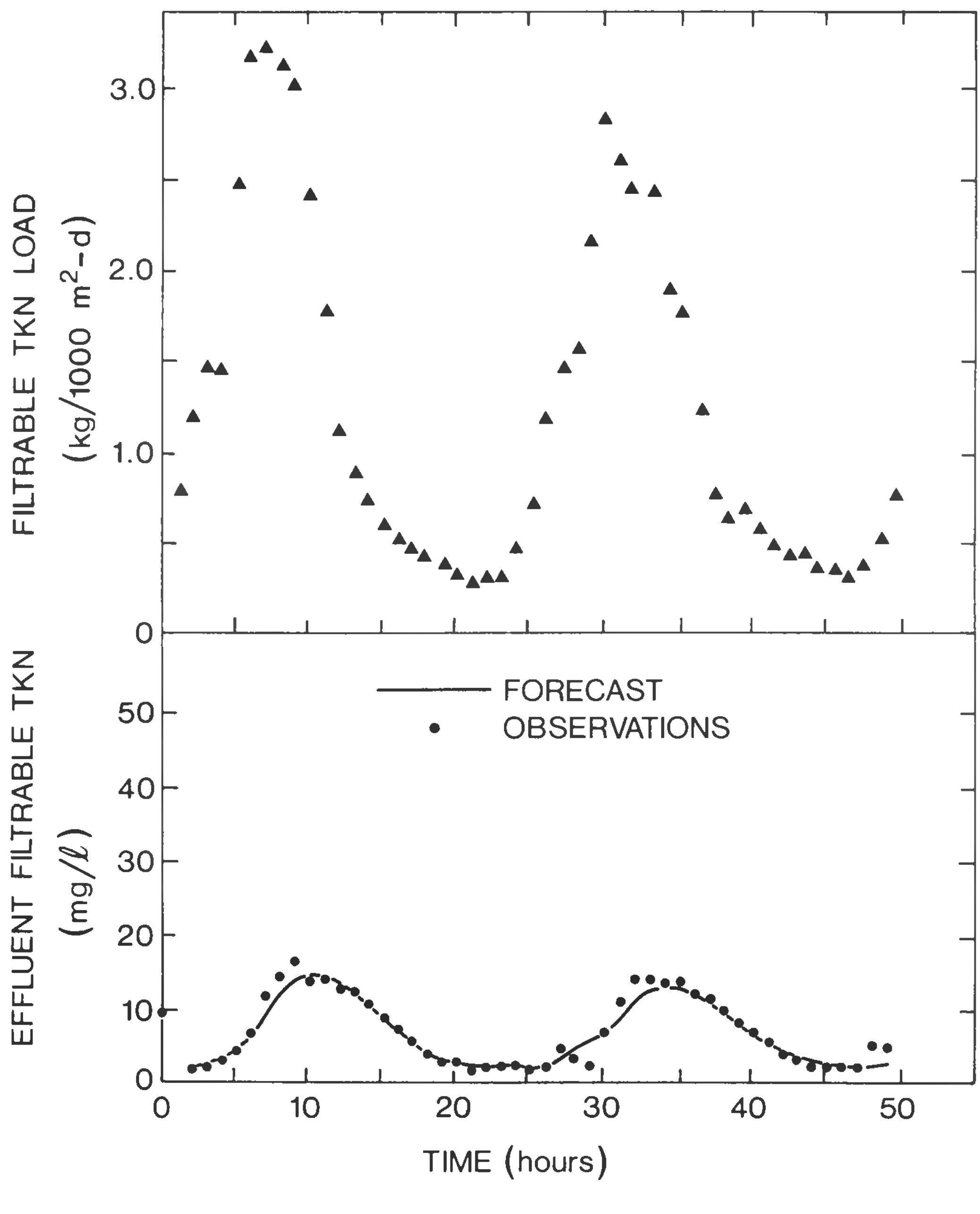

FIGURE 5-4 TRANSFER FUNCTION-NOISE MODEL PREDICTION OF DIURNAL VARIABILITY (0.5 m RBC in the Carbon Removal Plus Nitrification Mode)

5.3 Summary

The data in Figure 5-1 and Figure 5-2 indicate that treated effluent quality from an RBC operating with typical diurnal variations in flow, BOD_5 and TKN will vary dramatically over any 24-hour period. Although instantaneous BOD_5 and TKN loads can vary by an order of magnitude during a 24-hour diurnal cycle, there is no visible deterioration or change in the biomass on the discs.

Several researchers have reported that RBC's can withstand hydraulic and organic shock loads or superloads, thus avoiding washouts and other upsets typical of activated sludge systems (Antonie, 1970; Borchardt, 1971; Labella et al, 1972; Wilkey and Friedman, 1975). The results of the current studies on the effects of diurnal variations support this conclusion. However, the RBC cannot absorb these shock loads with little or no effect on treatment efficiency. The hourly data from both the BOD_5 removal and the combined BOD_5 removal plus nitrification studies demonstrated that a shock load could produce a dramatic and rapid deterioration in effluent quality, though the original effluent quality is regained almost as rapidly once it is removed. This agrees with the transient load studies reported recently by Davis (1976).

The loss in performance due to variation in hydraulic load probably results from the low hydraulic equalizing capacity of the pilot RBC, which is typical of RBC's in general. An exception is the small package plant RBC, which often has a much longer hydraulic residence due to the presence of a sludge storage zone in the tank containing the discs. Depending on how much short circuiting occurs, hydraulic residence in these package systems can be as high as 24 hours (Ahlberg and Kwong, 1974). Variations in effluent quality due to hourly load fluctuations may be less for package plants than for the RBC's evaluated in this study.

Design factors providing for an increase in RBC media area are necessary when using steady-state design loadings with wastewaters exhibiting diurnal flow variations. The test results in this section suggest that a larger safety factor is necessary for combined BOD_5 removal plus nitrification than for BOD_5 removal alone. Although, it is not possible to select optimum design factors based solely on the data analyzed in Section 5.2, the results were useful in defining a reasonable range of values. Based on the data available, it seems justified to recommend that a design factor of 1.25 be used for BOD_5 removal and one of 1.35 for BOD_5 removal plus nitrification. Thus, based on steady-state loading requirements, the RBC surface area should be increased by 25 or 35 percent at municipal wastewater plants where no flow equalization is provided.

6 CHARACTERISTICS OF SUSPENDED SOLIDS IN THE RBC

6.1 Solids Settleability

As nonsettleable solids in the RBC effluent affected its quality, it was important to determine whether they came from the discs and were biological in origin, or whether they were already present in the degritted wastewater. Table 6-1 provides a comparison between the biomass on the discs and the influent and effluent suspended solids in the wastewater. The BOD_5/SS ratios for suspended solids in the wastewater were calculated by substracting filtrable BOD_5 from total BOD_5 and dividing by the suspended solids concentrations. A similar ratio for the biomass on the discs was found by scraping samples from the discs, preparing suspensions of known suspended solids' concentrations in dechlorinated tap water, and determining the BOD_5.

TABLE 6-1 CHARACTERISTICS OF SUSPENDED SOLIDS

Source of Suspended Solids	Number of Observations	BOD_5/SS	VSS/SS
Raw Degritted Wastewater	34	0.38	0.78
RBC Effluent (30 min. settled)	33	0.34	0.76
Biomass Scrapings from Discs	16	0.16	0.72

The VSS/SS ratios for all three sources of solids were similar, while the BOD_5/SS ratio of 0.16 for the biomass scrapings was significantly lower than the ratio for either the solids entering or leaving the RBC. This suggests that the majority of the nonsettleable suspended material in the clarified effluent was already present in the degritted raw feed.

During the 16 weeks of the pseudo steady-state test for the 2.0 m RBC, several samples of effluent from the secondary clarifier overflow were collected and submitted for microscopic analysis. At the time these were taken, the BOD_5 loading to the RBC was between 6 and 10 kg/1000 m^2•d, and the suspended solids concentration in the clarified effluent varied between 30 and 40 mg/L. Three photographs taken of the suspended solids in the samples at a magnification of 500X are shown in Figure 6-1. Dr. D. Liu of the Canada Centre for Inland Waters (CCIW) determined that most of the solid

PLATE A

B

C

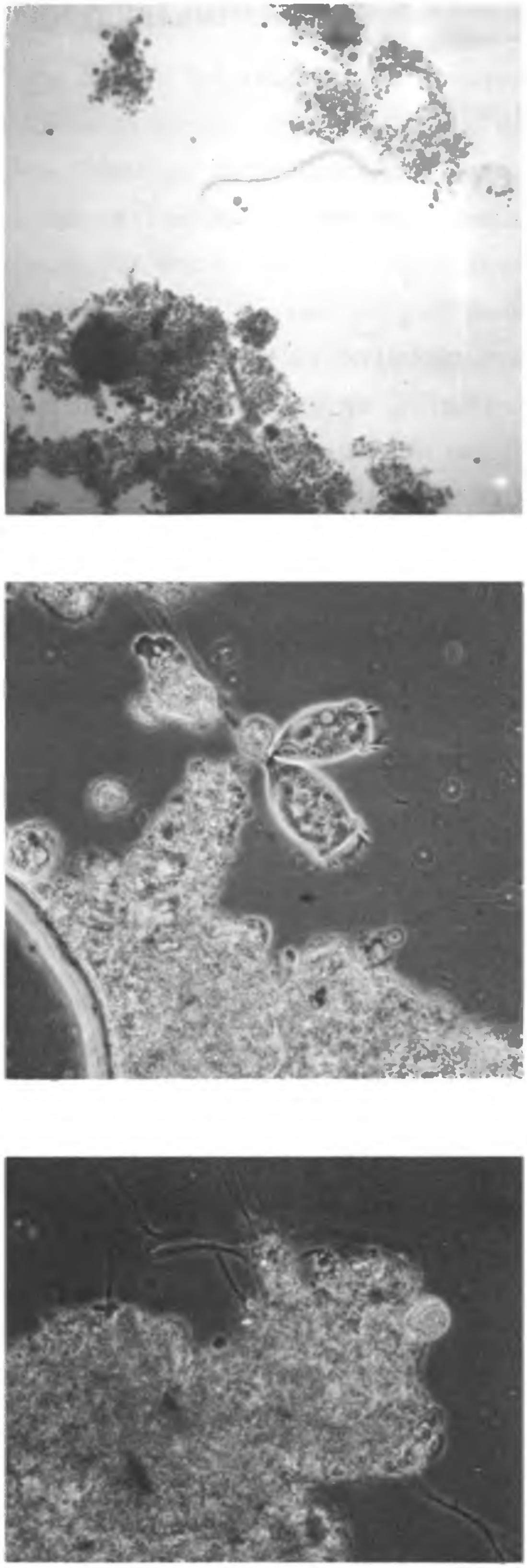

FIGURE 6-1 MAGNIFICATION OF EFFLUENT SUSPENDED SOLIDS (500X)

material in Figure 6-1C was organic but non-biological matter, indicating that much of the suspended material in the final effluent was related to the primary suspended solids in the raw degritted wastewater.

Since worms (Figure 6-1A), stocked ciliates (Figure 6-1B), and other organisms are generally not present in large numbers in overloaded biological processes, the RBC was not likely overloaded during this part of the study. This suggests that the factor limiting effluent quality was the poor solids settleability in the treated effluent.

The results of this study do not support the hypothesis, presented in other studies, that relative to other forms of high rate aerobic treatment, RBC's enhance the settling rates of suspended solids in the treated effluent. In fact, the reverse may be true. A 30-minute quiescent settling period in an Imhoff cone was insufficient to remove all the settleable suspended solids from the effluent. At a conservative overflow rate of 0.6 m/h and when BOD_5 loadings were less than 15 kg/1000 $m^2 \cdot d$, the clarifier produced an effluent consistently lower in suspended solids than was obtained in the 30-minute settling period.

The concentration of suspended solids in the effluent entering the clarifier varied between 150 and 350 mg/L. Within this range "hindered settling" occurs in the final clarifier operation. This is distinctly different from sludge blanket formation and subsequent zone settling which typically takes place in activated sludge plants. In one respect this could be advantageous as it may be unnecessary to provide continuous sludge withdrawal. Furthermore, side wall depths might be reduced to as low as two metres since there would be no need for a deep sludge compaction zone. On the other hand, without the flocculating assistance of a sludge blanket in the clarification zone the finer settleable solids may take longer to settle. Therefore, conservative overflow rates should be used in order to maximize solids' removal.

During January and February, when the BOD_5 loading to the 2.0 m RBC was $\backsim$ 20 kg/1000 $m^2 \cdot d$ and higher, traces of a greywhite growth appeared on the media. Small-particle white suspended solids were also noticed in the clarified effluent. This type of growth is typical of filamentous organisms of the genus Beggiatoa, which can develop in highly loaded biological systems and cause a deterioration in solids settleability. At the higher loadings they may have contributed to the high suspended solids concentrations in the clarifier effluent.

As most of the suspended solids entering the final clarifier of the 2.0 m pilot plant were present in the raw feed, the consistency of the settled sludge in the clarifier should have been more typically that of primary sludge than of waste activated sludge. To test this assumption, raw degritted wastewater was fed to the clarifier for two one-week periods. The results (Table 6-2) showed that the average consistency of nine samples of primary sludge compared very closely with those measured for the RBC-treated effluent.

TABLE 6-2 SETTLED SLUDGE CONSISTENCIES

Clarifier Operation	Number of Observations	TSS (%)	VSS (%)
Raw Degritted Wastewater	9	3.5	2.4
Treated RBC Effluent	43	3.4	2.3

At the same time as this study a suspended growth treatment system was operated at CCIW. Solids concentrations in the underflow, from the final clarifier in the suspended growth system, rarely exceeded 1.5 percent compared to 3.5 percent observed in the RBC clarifier.

6.2 Solids Production

Since the rate of biomass sloughing from the RBC discs was both highly variable and unpredictable, it was necessary, in order to estimate solids yields, to compensate for these fluctuations by keeping a continuous record of BOD_5 removal and solids production and then averaging the results over an extended period. Daily composite data from the 2.0 m RBC on BOD_5 removal and VSS production were averaged each week during a nine week period. Using these results the total BOD_5 removal and the total VSS produced during each period were estimated. Figure 6-2 is a cumulative plot of these data. The increase in VSS was determined by subtracting the VSS in the raw wastewater from that in the RBC effluent. The BOD_5 removal was measured as total BOD_5 entering the RBC, minus the total BOD_5 in the clarifier effluent.

Figure 6-2 shows that 0.46 kg of VSS were produced per kg of BOD_5 removed. Figure 6-3 is a similar cumulative plot of the 0.5 m RBC data, where primary clarification was employed. This plot indicates a solids yield of 0.31 kg VSS/kg BOD_5. Although the two yields differ, the amount is probably not significant. Even slight undersampling or oversampling of volatile suspended solids, by any of the raw feed or treated effluent samplers, would have introduced large errors in the yield estimates.

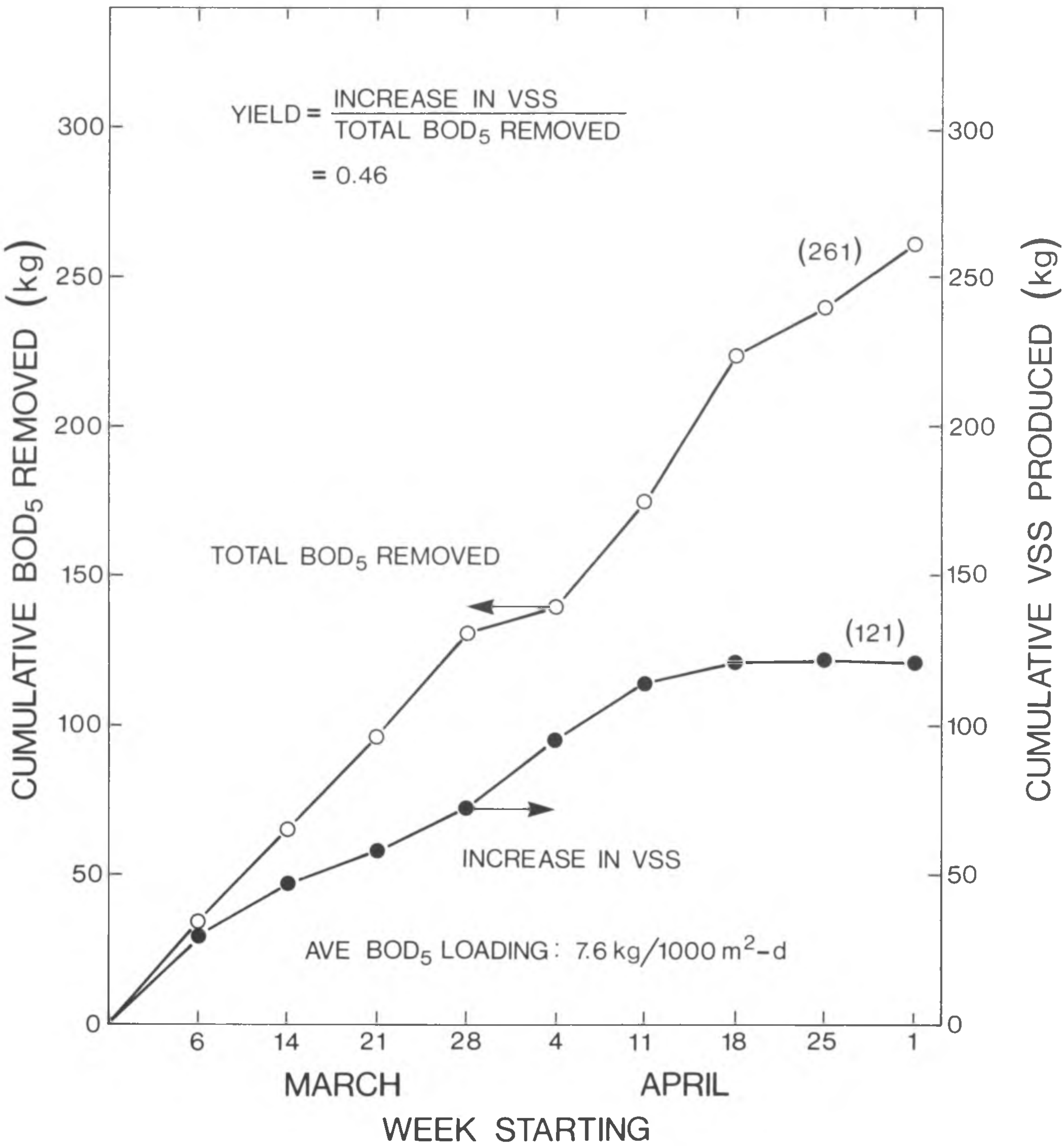

FIGURE 6-2 SOLIDS YIELD WITH THE 2.0 m RBC

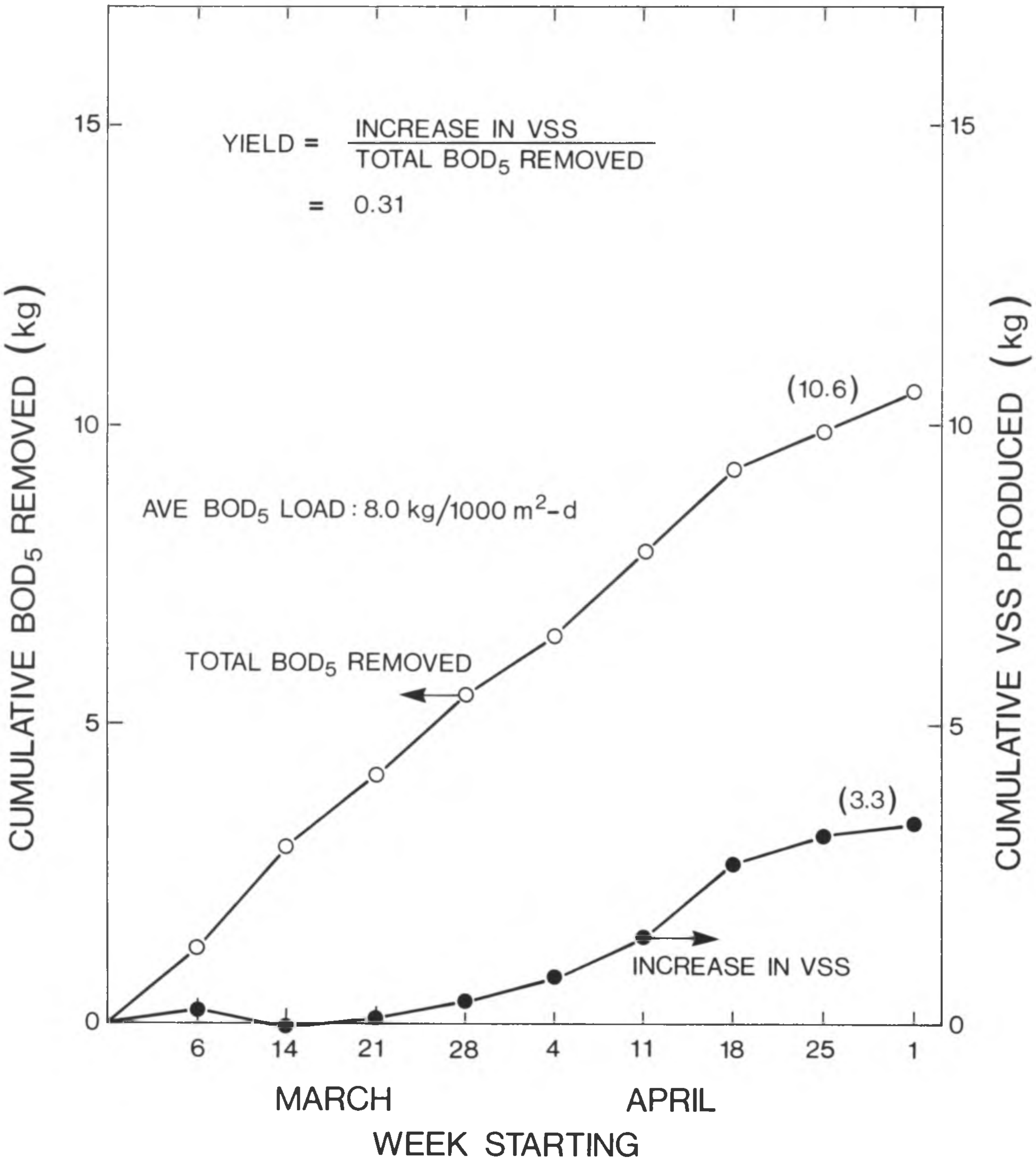

FIGURE 6-3 SOLIDS YIELD WITH THE 0.5 m RBC

7 PHOSPHORUS REMOVAL

7.1 Background

Recognition that many receiving waters are environmentally sensitive to excess nutrient addition has led to limitations on the discharge of phosphorus from municipal water pollution control plants. The most economical method to remove phosphorus from wastewater is by precipitation with iron or aluminum salts added directly to the treatment process. In some instances the applicability of RBC treatment could depend on whether it is compatible with the phosphorus removal.

Recent studies have demonstrated that efficient phosphorus removal can be achieved in activated sludge and extended aeration plants by adding the metal salts, which have been found to have no significant effect on overall treatment efficiencies, directly to the aeration basin (Sutton et al, 1977). A seven-week program was designed to investigate whether similar results could be obtained with the RBC.

7.2 Experimental Program

Raw degritted wastewater was fed to the 2.0 m RBC at a constant hydraulic loading of 4.5×10^{-2} $m^3/m^2 \cdot d$. The average BOD_5 loading during the seven week period was approximately 5 kg total $BOD_5/1000$ $m^2 \cdot d$, sufficiently low to permit nitrification to occur. During the first two weeks system performance was monitored with no chemicals being added (the control period). An alum solution was added continuously to the first stage of the four-stage RBC during the following two-week period. Ferric chloride solution was added for the last three weeks. No effort was made to optimize chemical dosages, but instead an excess of metal ion was used in each case to ensure that insufficient chemical addition could not be blamed for poor phosphorus removals. The average ratio of aluminum ion added to total PO_4-P in the raw wastewater was 1.6:1. The ratio of ferric ion to PO_4-P was 3.2:1.

7.3 Results and Discussion

7.3.1 Effect of Chemical Addition on RBC Performance. Table 7-1 is a general summary of data on loadings and effluent quality observed during the three sections of the phosphorus removal program. Neither the aluminum nor ferric ions influenced the concentration of BOD_5 or NH_4-N in the treated effluent, indicating that neither one adversely affected the active microorganisms (which include both the heterotrophic

TABLE 7-1 EFFECT OF CHEMICAL ADDITION ON RBC EFFLUENT QUALITY (2.0 m RBC)

		Chemical	
	Control	Alum	Ferric Chloride
Treatment Regime			
Period	22/04-06/05	07/05-20/05	21/05-10/06
BOD_5 Loading (Ave.) (kg/1000 m^2•d)	4.3	5.2	5.6
Temperature (°C)	13	15	17
Treated Effluent Quality			
BOD_5 (Settled, mg/L)	18	19	20
Suspended Solids (Settled, mg/L)	41	64	66
NH_4-N (Filtrable, mg/L)	1.4	1.4	1.0
pH	7.5	7.3	7.3

organisms responsible for metabolizing the BOD_5 and the autotrophic nitrifying organisms) attached to the discs.

Effluent pH values became slightly lower during chemical addition. This is because hydrolysis (which takes place when ferric chloride or alum are added to wastewater) releases hydrogen ions, neutralizing some of the alkalinity. The observed pH of the wastewater, however, was close to the range of 6 to 7, an optimum value for phosphorus precipitation when using iron or aluminium.

Chemical addition caused a significant increase in the concentration of suspended solids in the settled effluent. As was previously shown in Section 4.3.3, the standard 30-minute settling test did not always provide sufficient time for complete removal of settleable solids from the treated effluent. Consistently lower concentrations of suspended solids were observed in the effluent of the final clarifier in comparison to the 30-minute settled samples, when the clarifier was operated at a conservative loading of 0.6 m/h and with a hydraulic detention time of approximately two and one-half hours. Table 7-2 shows a similar relationship between the settled and the clarified effluent for the phosphorus removal studies.

Although the concentration of suspended solids in the settled samples increased when chemical addition started, this did not happen in the clarified effluent.

TABLE 7-2 EFFECT OF CHEMICAL ADDITION ON SOLIDS SETTLING

Treatment	Effluent Suspended Solids 30 min. Settling (mg/L)	Effluent Suspended Solids Clarifier (mg/L)
Control	41	31
During Chemical Addition	65	33

The data suggest that when alum or ferric chloride was added to the RBC settling rates of a portion of the effluent solids were reduced, but no net increase in the concentration of nonsettleable suspended material occurred.

The additional suspended solids were mainly inorganic in nature and were not accompanied by an increase in BOD_5. Furthermore, the total percentage of ferric ion increased from 1.9 percent of the suspended solids in the influent, to 9.0 percent in the settled effluent. During alum addition, the aluminum increased from 1.0 percent of the suspended solids in the influent to 3.8 percent in the settled effluent.

The data in Tables 7-1 and 7-2 demonstrate that alum and ferric chloride can be added directly to an RBC with no net effect on treatment efficiencies and effluent quality, provided that proper attention is given to the design and operation of the final clarifier.

7.3.2 Phosphorus Removal. The phosphorus removal results are summarized in Table 7-3. One-third to one-half of the phosphorus in the raw wastewater was filtrable, the remainder was associated with the suspended solids. Data from the control period show that much of the particulate phosphorus was removed during normal treatment, but there was little change in filtrable phosphorus. With chemical addition virtually complete removal of filtrable phosphorus was observed. However, the corresponding removal efficiencies for total phosphorus were only 75 to 80 percent. During alum and ferric chloride treatment the average concentrations of total phosphorus in the settled effluent samples were 1.3 and 1.4 mg/L respectively. These concentrations exceed the discharge requirement of 1.0 mg/L, which apply to secondary effluents within the Great Lakes basin. As a further 50 percent reduction in suspended solids was obtained in the pilot clarifier compared to settled samples, a similar reduction in particulate phosphorus would be expected.

TABLE 7-3 SUMMARY OF PHOSPHORUS REMOVAL

Treatment	Number of Observations	Total P			Filtrable P		
		Influent mg/L PO_4-P	Effluent (Settled)* mg/L PO_4-P	% Removed	Influent mg/L PO_4-P	Effluent (Settled) mg/L PO_4-P	% Removed
Control	6	4.7	2.4	49	2.3	1.9	17
Alum Addition	7	6.0	1.4	77	2.1	0.2	90
Ferric Chloride	8	5.5	1.3	76	2.6	0.2	92

*30-minute settling test.

There was no difference in phosphorus removal efficiencies between the alum and ferric chloride treatments and consequently, the data obtained during each treatment were combined in pooled probability plots (Figures 7-1 and 7-2). They show that effluent concentrations for both total and filtrable P were consistent, and varied within relatively small ranges.

7.3.3 Solids Production. The use of chemicals to remove phosphorus from wastewater increases the quantity of sludge for disposal; the additional sludge contains precipitated phosphate salts and aluminum or ferric hydroxides in proportion to the amount of chemical used. The chemical reactions which occur during phosphorus removal are shown in Table 7-4. Based on the stoichiometry of these equations, theoretical sludge production can be calculated for either ferric chloride or alum used in phosphorus removal (Sutton et al, 1977).

TABLE 7-4 STOICHIOMETRIC REACTIONS FOR ALUM AND FERRIC CHLORIDE IN WASTEWATER

Alum			
$Al_2(SO_4)_3 \cdot 14H_2O + 2H_2PO_4^- + 4HCO_3^-$	$\rightarrow$	$2AlPO_4\downarrow + 3SO_4^{2-} + 14H_2O + 4H_2CO_3$	(1)
$Al_2(SO_4)_3 \cdot 14H_2O + 6HCO_3^-$	$\rightarrow$	$2Al(OH)_3\downarrow + 3SO_4^{2-} + 6CO_2 + 14H_2O$	(2)
Ferric Chloride			
$FeCl_3 \cdot 6H_2O + H_2PO_4^- + 2HCO_3^-$	$\rightarrow$	$FePO_4\downarrow + 6H_2O + 3Cl^- + 2H_2CO_3$	(3)
$FeCl_3 \cdot 6H_2O + 3HCO_3^-$	$\rightarrow$	$Fe(OH)_3\downarrow + 3Cl^- + 3CO_2 + 6H_2O$	(4)

Records were kept of the total suspended solids entering and leaving the RBC during both the control and chemical addition periods. The average increases in the difference between effluent total suspended solids and influent total suspended solids for chemical addition are tabulated (Table 7-5). The increase in solids' production, which, it should be noted, was low at 0.3 kg TSS/d during the control period, was significant during the alum and ferric chloride treatment processes. This is equivalent to a yield of 0.09 kg VSS/kg total BOD_5 removed, compared to the values of 0.31 and 0.46 kg VSS/kg total BOD_5 which were observed in experimental runs conducted just prior to the chemical addition program (Figures 6-2, 6-3).

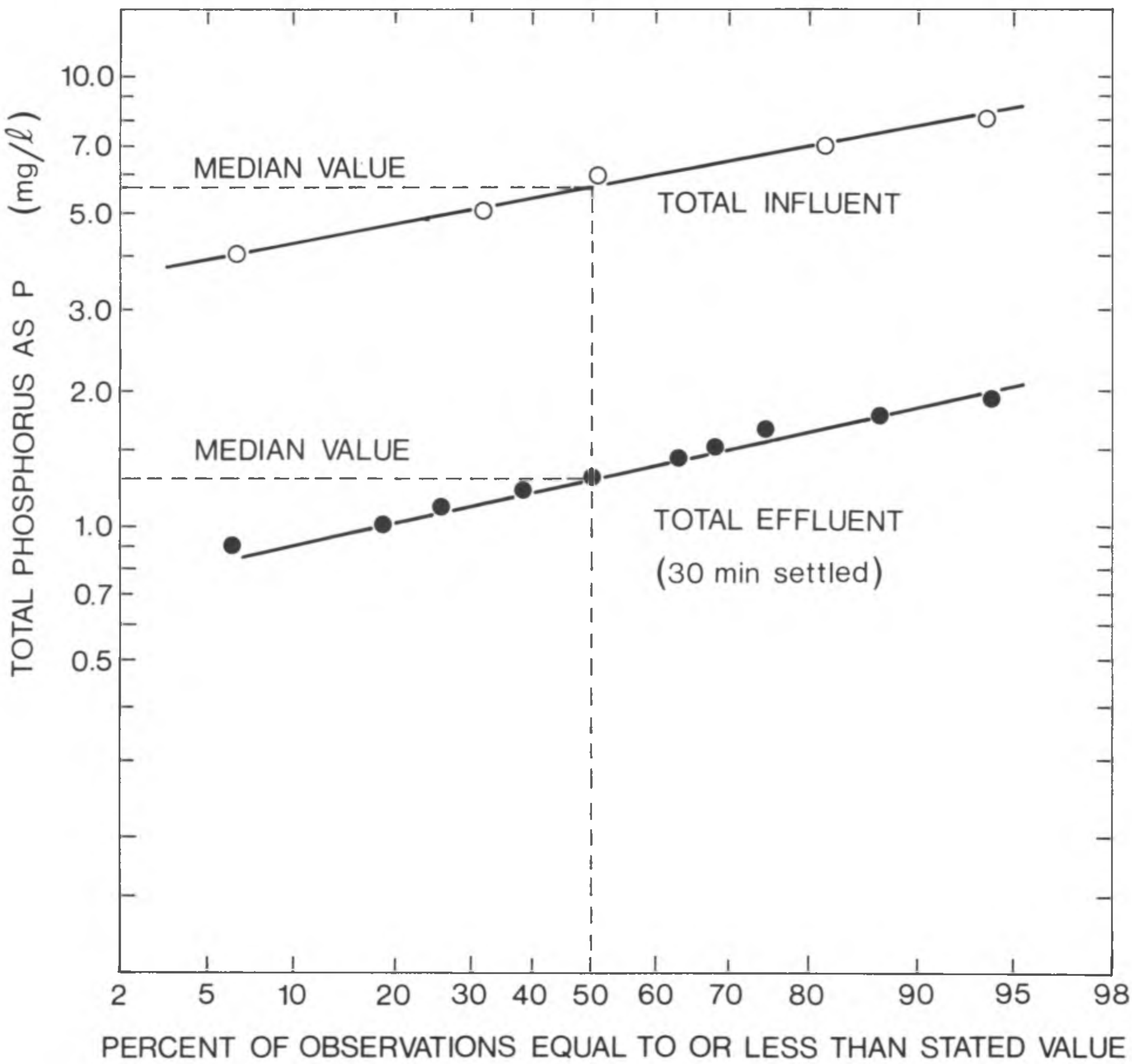

FIGURE 7-1 TOTAL PHOSPHORUS CONCENTRATIONS BEFORE AND AFTER CHEMICAL ADDITION (2.0 m RBC)

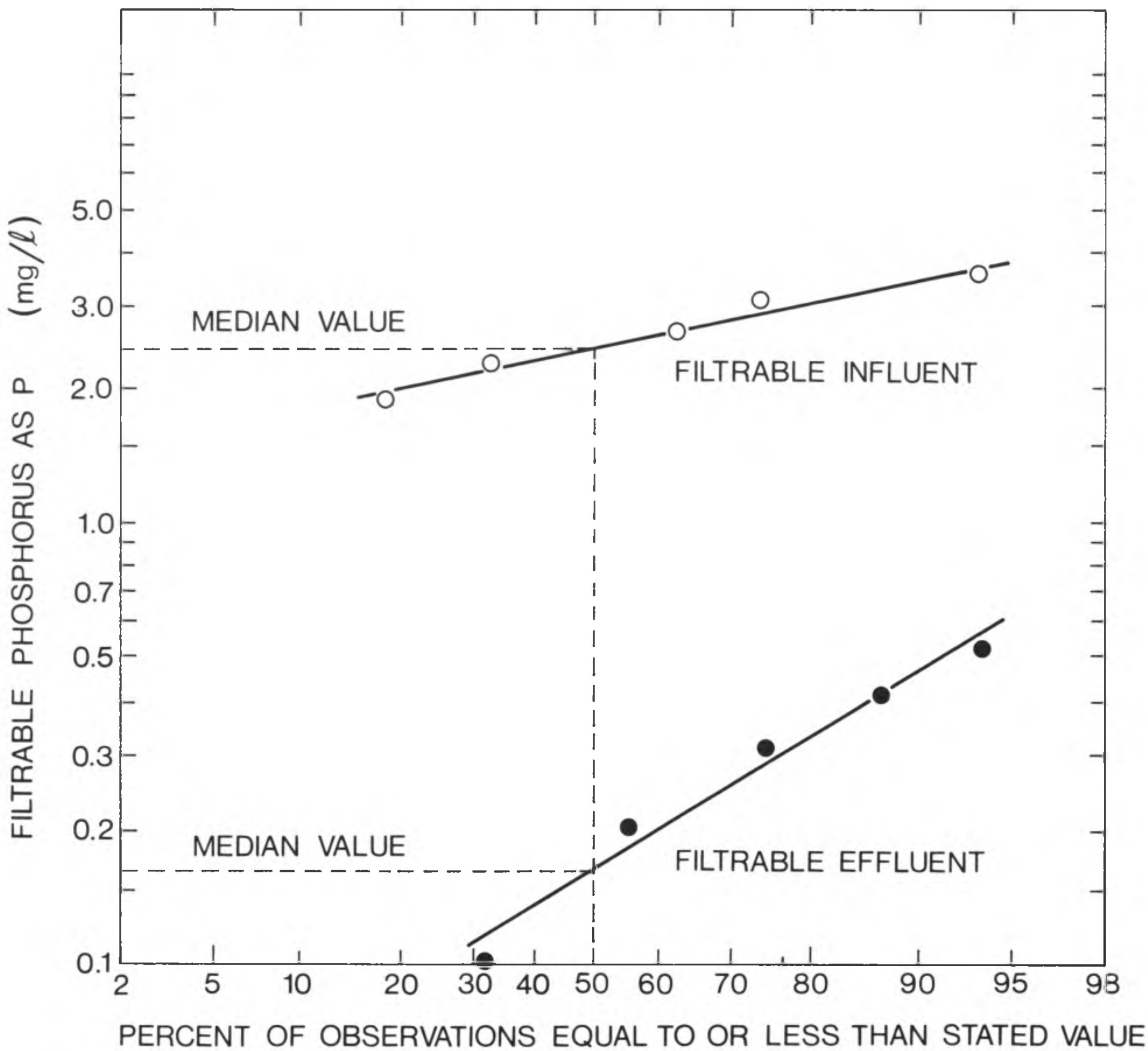

FIGURE 7-2 FILTRABLE PHOSPHORUS CONCENTRATIONS BEFORE AND AFTER CHEMICAL ADDITION (2.0 m RBC)

TABLE 7-5 EFFECT OF CHEMICAL ADDITION ON SOLIDS PRODUCTION

	Increased Total Solids Production*			
	Theoretical		Observed	
	kg TSS/d	kg TSS/kg metal	kg TSS/d	kg TSS/kg metal
Alum	1.0	3.3	2.1	7.0
Ferric Chloride	1.3	2.2	2.7	4.5

* Increase wrt sludge production during the period just prior to chemical addition.

The observed increase in total solids production during chemical addition was approximately twice that predicted by equations 1 to 4 (Table 7-4). Sutton et al (1977), using ferric chloride in parallel operated activated sludge plants, observed an increased sludge production of 1.65 over that predicted by the stoichiometric relationships. The observed ratio for the ferric chloride tests in this study was 2.1. Limited data from the U.S. Environmental Protection Agency (1976) indicated 35 percent excess sludge production over stoichiometric predictions with either alum or ferric chloride addition.

8 PROCESS SCALE-UP

8.1 Introduction

Since application of RBC technology in North America is relatively new, it is not surprising that the installation of many full scale RBC systems has been preceded by pilot plant testing. As more full scale operating data become available, there will be less need for these pilot plant trials prior to design of systems for municipal wastewater treatment. Nevertheless, where operating experience still is limited (such as in nitrification, denitrification, and treatment of industrial wastes) pilot trials will be necessary to assist in the design procedure.

Although pilot plant studies are considered essential in determining full scale design parameters for specific waste treatment applications, relatively little attention has been given in the literature to the translation of these data to full scale design. Several authors have presented mechanistic or semi-empirical mathematical models attempting to describe and predict RBC performance. One of the most comprehensive is that of Famularo et al (1976), which incorporated the mass transfer, hydraulic and kinetic processes that control BOD_5 removal. It differed from other models developed by researchers such as Grieves (1972), Friedman et al (1976) or Kornegay (1972) in that it considered simultaneous mass transfer of oxygen and substrate in both the liquid phase and the biomass, while the others neglected the possibility of oxygen transfer limitations in the process, an assumption shown to be incorrect by Torpey et al (1972).

Famularo's model predicted that direct application of pilot plant data to full scale units at constant peripheral disc speed and loading can result in underdesign. As the RBC disc diameter is increased the liquid film on the biomass is exposed to the atmosphere for longer periods resulting in greater substrate depletions and lower effective substrate concentrations in the liquid layer. Under conditions of low substrate concentrations, when substrate availability and diffusion is limiting, total removal efficiency declines as disc size increases. A sample calculation of Famularo for a four-stage system predicted a 10 percent reduction in total BOD_5 removal efficiency in going from a 2 m to a 6 m RBC.

Oxygen transfer limits treatment efficiency in RBC stages where the substrate BOD_5 concentrations are high. A reduction in total system oxygen transfer with increasing RBC size would cause a proportional decrease in BOD_5 removal. Lower DO concentrations should be observed in full scale RBC's than in pilot units under similar hydraulic and organic loadings. This has been observed in pilot scale evaluations involving industrial effluents. Chesner and Molof (1977) predicted that RBC scale-up based on peripheral velocities would result in reduced performance at any stage where DO levels fell below 2.0 mg/L.

Williamson and McCarty (1976) studied fixed film reactors and developed a simplified method to determine whether a system is oxygen or substrate limited. If the following inequality holds, the availability of the electron acceptor (in most cases oxygen) is limiting, but if it does not, the substrate or BOD_5 concentration limits the removal rate.

$$S_{oa} < \frac{V_a \, D_{cd} \, MW_a}{V_d \, D_{ca} \, MW_d} \cdot S_{od}$$

where
S_{oa} is the concentration of electron acceptor (mg/L),
S_{od} is the concentration of electron donor (mg/L),
MW_a is the molecular weight of electron acceptor,
MW_d is the molecular weight of electron donor,
D_{cd} is the diffusivity (in water) of electron donor (cm^2/d),
D_{ca} is the diffusivity (in water) of electron acceptor (cm^2/d),
V_a is the stoichiometric reaction coefficient of electron acceptor, and
V_d is the stoichiometric reaction coefficient of electron donor.

As an example, if glucose is selected as the BOD_5 source and the process temperature is 20°C, the inequality predicts that oxygen transfer will limit the process until the BOD_5 concentration is reduced to approximately 30 mg/L. Simulations presented by Famularo agree relatively well with this limit on oxygen transfer limitations.

8.2 Experimental Program

To assess the implications of designing full scale RBC systems directly from pilot plant data without the use of appropriate scale-up factors, an experimental program was designed in which the 0.5 m and 2.0 m RBC were operated in parallel using the same feed and identical hydraulic and organic loadings. The RBC's were acclimated at pseudo steady-state loadings for several days prior to the test period. During the test period, influent and effluent samples from both RBC's were collected at 30-minute intervals, for a period of two and a half days. In this time, step changes in hydraulic load, filtrable TKN and filtrable organic carbon were applied randomly at five-hour intervals.

Two separate two-and-a-half-day studies were conducted, one in which the mean hydraulic and organic carbon loadings were relatively high so that little nitrification was expected, and the second at loadings low enough to permit nitrifying populations to develop within the biomass. The range in raw feed characteristics included hydraulic loadings between 3×10^{-2} and 30×10^{-2} $m^3/m^2 \cdot d$, filtrable TKN concentrations between 10 and 45 mg/L and filtrable organic carbon concentrations from 15 mg/L to approximately 90 mg/L. The BOD_5 concentration in the degritted raw wastewater spanned the range between 35 and 250 mg/L, a range typically encountered in municipal wastewaters. Further details concerning the experimental design have been given in the discussion of time series analysis in Chapter 3.

8.3 Paired Comparison of 0.5 m and 2.0 m RBC Performance

8.3.1 Organic Carbon Removal Mode. All of the influent and effluent samples collected during the organic carbon removal study were analyzed for filtrable COD. The hydraulic loading on the RBC's during this phase of the scale-up work alternated between 15×10^{-2} and 30×10^{-2} $m^3/m^2 \cdot d$. Analysis of the flow data from the two and a half day experimental run, showed that the average hydraulic loadings for the two RBC's agreed to within 2.5 percent. As the raw degritted wastewater fed to the units was from a common source, the average organic loadings per unit disc area were also similar. The data obtained from time series analysis demonstrated that the RBC's reached steady-state operation within one or two sample periods following a step change in the feed conditions. Therefore, the steady-state performances of the two RBC's can be directly compared by reviewing all paired data generated after the second sample period. This was done by plotting the COD concentrations in the treated effluents (Figure 8-1). If both RBC's had performed identically the slope of the regression line would have been 1.0 but instead it had a slope of 0.5. As other experimental runs have shown filtrable COD concentrations of 50 mg/L or less to be resistant to biological degradation, it was not anticipated that the regression line would pass through the origin. The result indicates that the effluent from the 0.5 m RBC contained, on the average, only half the degradable COD concentration of that from the 2.0 m RBC under similar loadings and operating conditions.

Mass removal of COD per unit disc area and time was also used as a comparative variable. Instantaneous mass removals were calculated using the influent COD concentration at time "t" and the effluent concentration at time "t plus 30 minutes", as previous dye studies had indicated that at the average hydraulic load used in

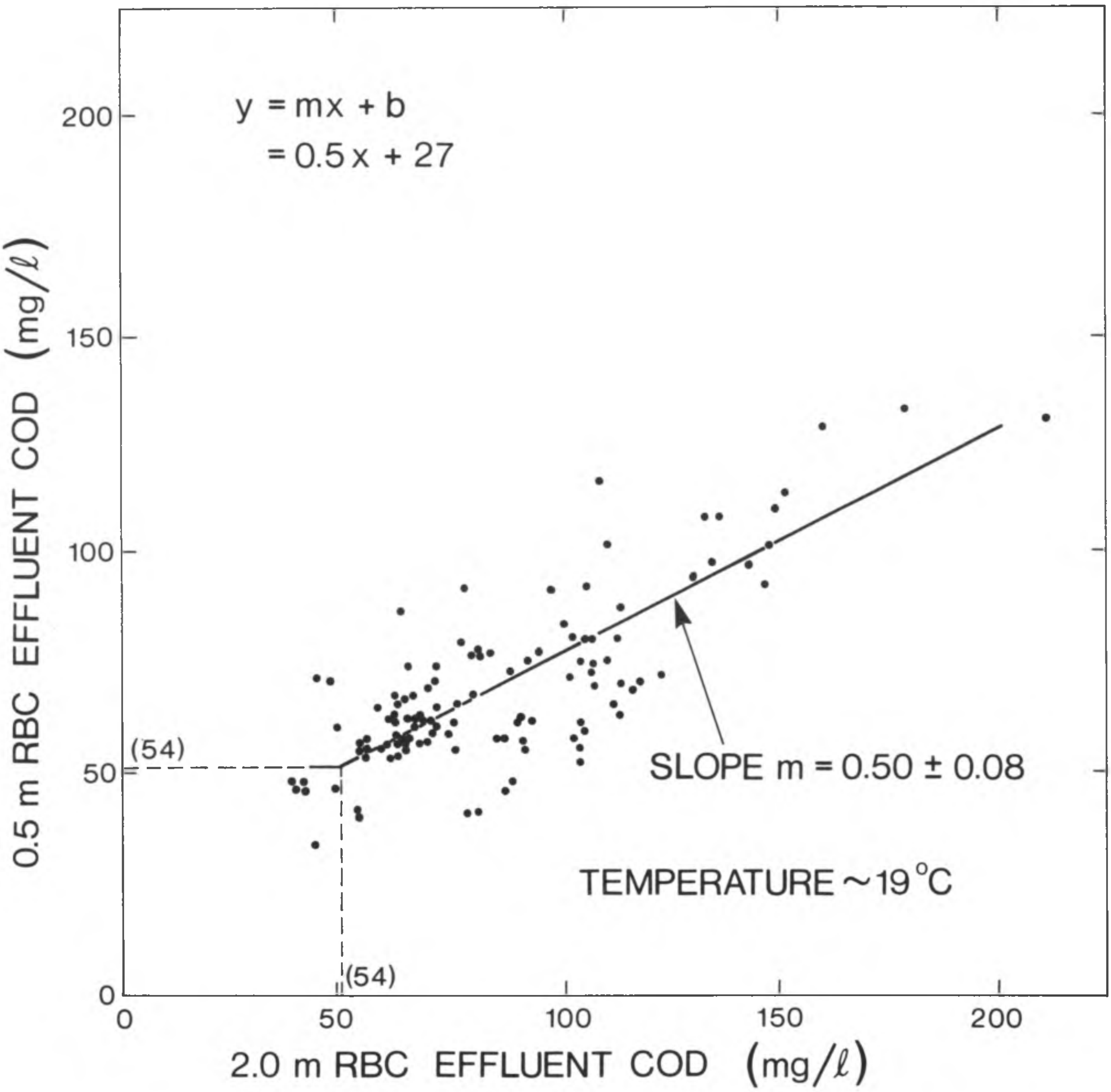

FIGURE 8-1 EFFLUENT FILTRABLE COD CONCENTRATION: 0.5 m RBC vs 2.0 m RBC (Organic Carbon Removal Mode)

this study, concentration peaks required approximately 30 minutes to pass through the system.

The calculations of mass removals are plotted in Figure 8-2. The first two lags following step changes were omitted for the reasons already mentioned. The linear least squares regression, calculated from the data in Figure 8-2, shows a slope of 1.16, illustrating again that the 0.5 m RBC provided superior performance with mass removals averaging 16 percent higher than in the 2.0 m unit. The observed differences in the performances of the two units were statistically significant to the 95 percent confidence limit.

8.3.2 Organic Carbon Removal Plus Nitrification Mode. The second scale-up study was run at an average hydraulic load of 7.5×10^{-2} $m^3/m^2 \cdot d$. This provided a low total COD loading and allowed the growth of nitrifying organisms on the discs. An analysis of the wastewater flows recorded for each RBC showed that the average hydraulic loads to the two units (measured as flow per unit area and time) differed by about 4.5 percent, with the 0.5 m RBC having the lower value.

Figure 8-3 is a plot of the filtrable effluent TKN data. The line of best fit, calculated by linear least squares regression, has an intercept at $y = 0.85 \pm 0.75$ and a slope of 0.86 ± 0.04. As anticipated, the intercept is close to the origin as most of the filtrable TKN in the wastewater is available for nitrification. The slope of the regression line, and its associated 95 percent confidence interval, show that the average TKN concentration in the treated effluent of the 0.5 m RBC was 14 percent lower than in the 2.0 m pilot plant, and therefore was statistically significant. Nevertheless, the relative difference between the performances of the two pilot units during this combined organic carbon and TKN removal study was much less than during the first study in which higher loadings were used.

The TKN mass removal data are presented in Figure 8-4. The values were calculated in a similar fashion to the COD removals shown in Figure 8-2, except that a 60-minute lag was used between influent and effluent concentrations. This was necessary because the concentration peak occurred after a longer interval than previously was the case, due to the reduced hydraulic loads employed during this phase of the study.

The 95 percent confidence limit for the slope of the regression line shows that there was no significant difference between the performance of the 2.0 m RBC and the

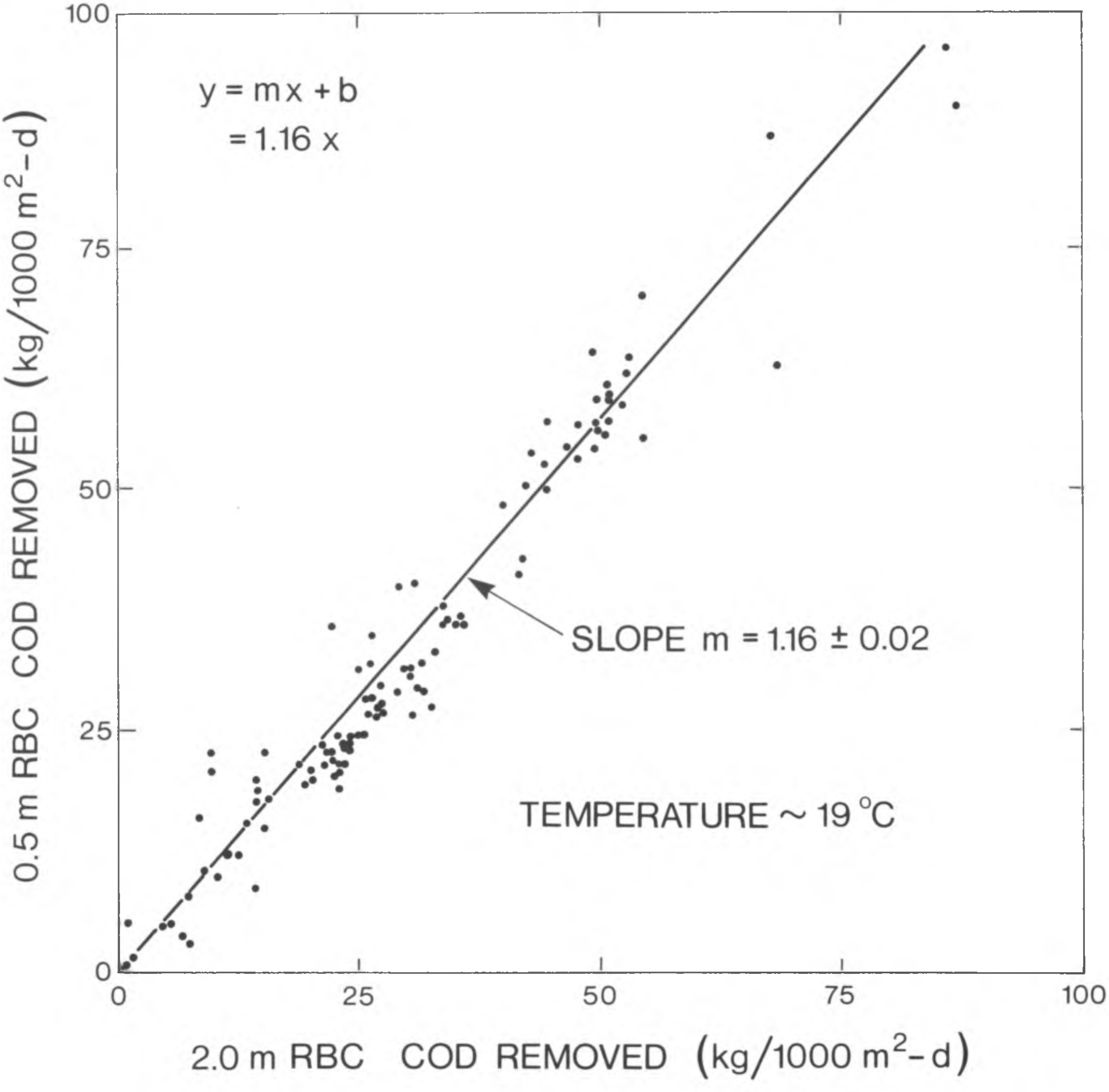

FIGURE 8-2 FILTRABLE COD MASS REMOVALS: 0.5 m RBC vs 2.0 m RBC (Organic Carbon Removal Mode)

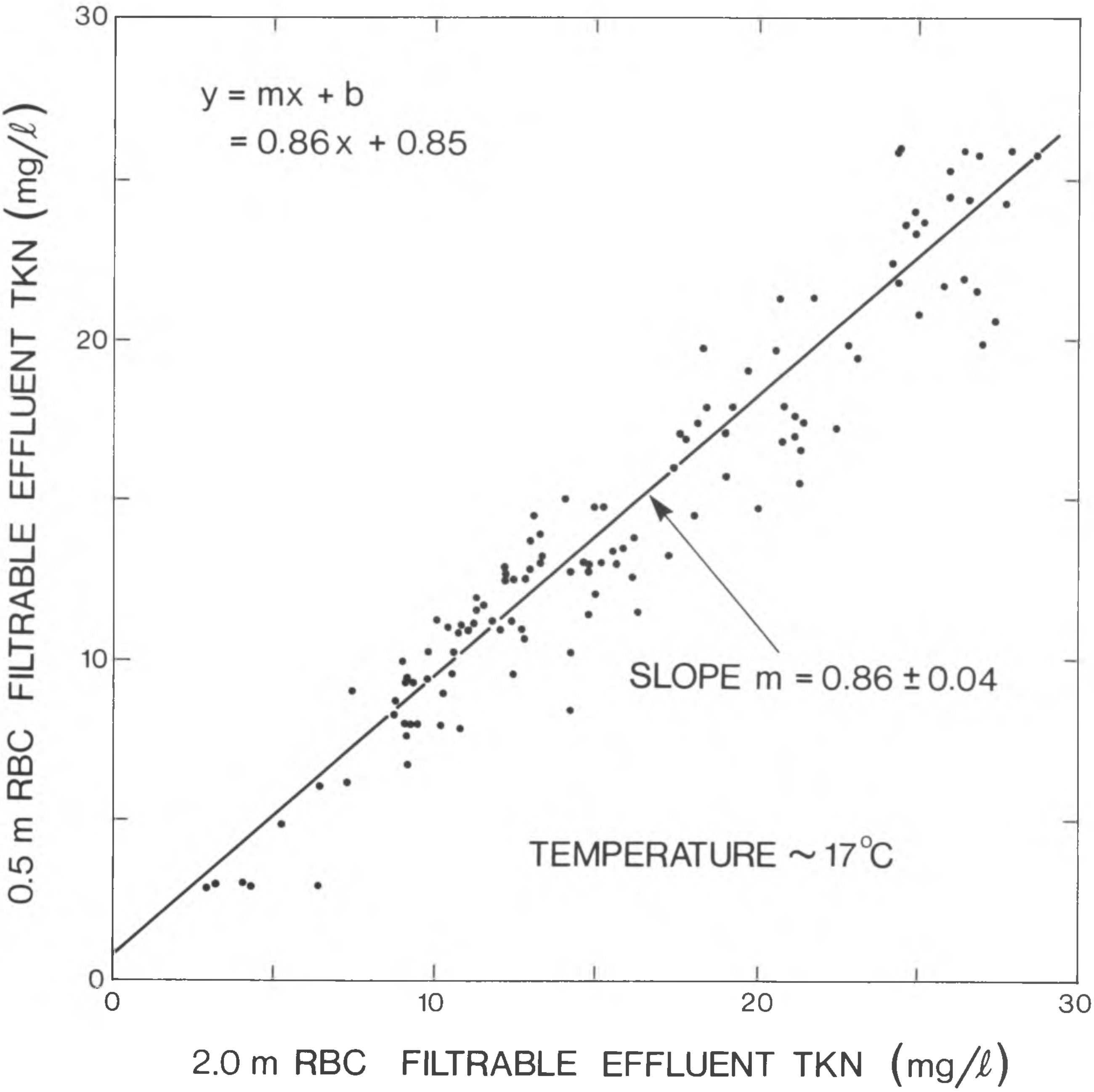

FIGURE 8-3 EFFLUENT FILTRABLE TKN CONCENTRATION: 0.5 m RBC vs 2.0 m RBC (Organic Carbon Plus Nitrification Mode)

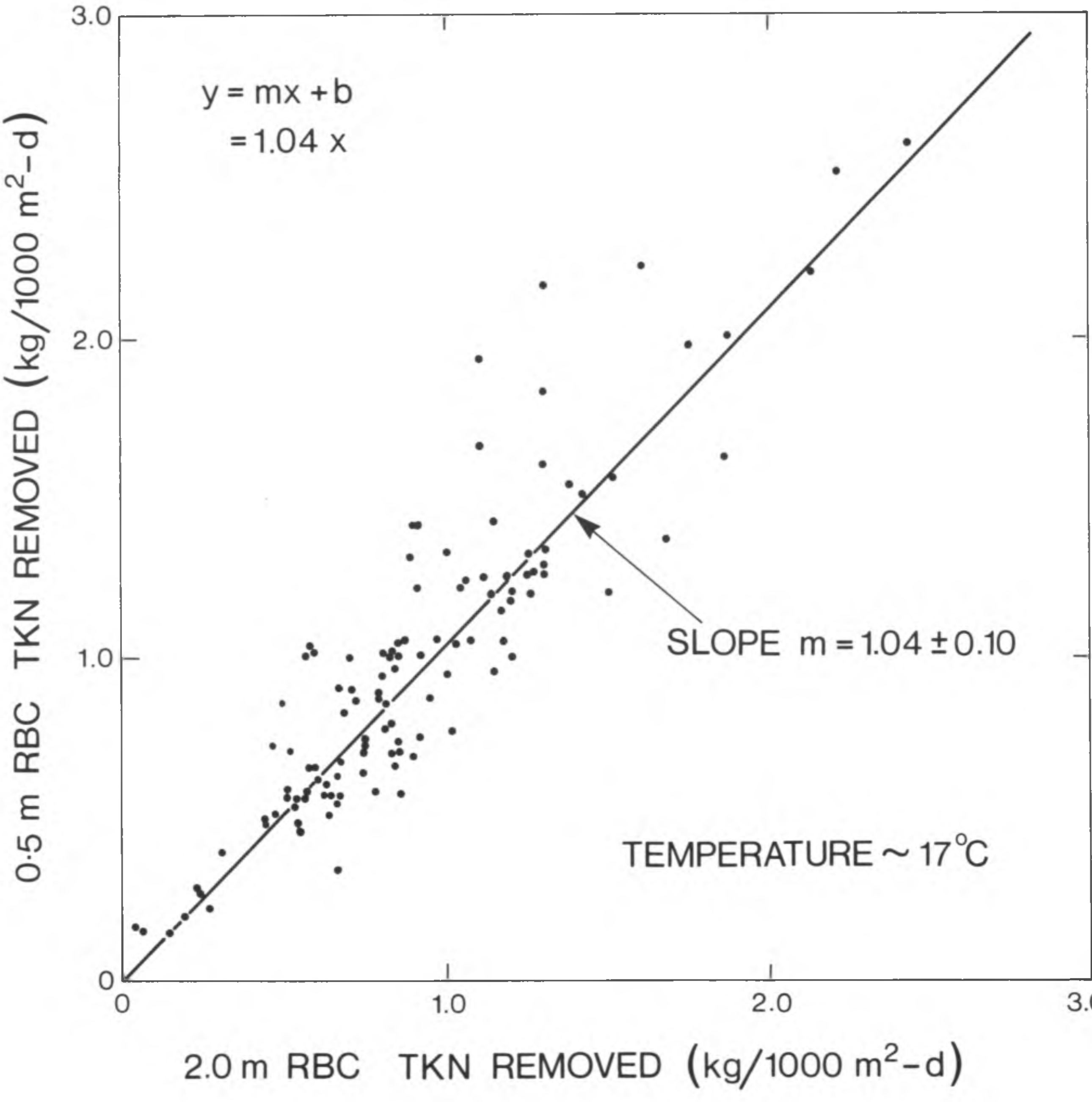

FIGURE 8-4 MASS REMOVAL OF FILTRABLE TKN: 0.5 m RBC vs 2.0 m RBC (Organic Carbon Plus Nitrification Mode)

0.5 m RBC. At equal loadings the mass of TKN removed per unit area and time was the same in both systems.

The amount of nitrate and nitrite nitrogen formed during nitrification was found to be considerably less than the amount of filtrable TKN removed, and only part of the difference could be explained by nitrogen uptake during biomass assimilation. The apparent loss of nitrogen is consistent with the results of research conducted by Wilson (1975) using similar RBC systems. One possible explanation is that some of the oxidized nitrogen formed during treatment diffused into anoxic zones deep within the biological slimes, and denitrification may have occurred as a result of endogenous respiration of the organisms. This hypothesis is consistent with results reported by Hoehn and Ray (1973), which showed that biomass density in a supported growth reactor was significantly less at depths greater than 150 microns than near the surface, due to gas production and lysing in the lower anaerobic layer.

The nitrate and nitrite nitrogen productions expressed as kg/1000 $m^2 \cdot d$ for the two RBC's are presented in Figure 8-5. These were calculated in the same way as the TKN mass removals. The data show that there is no significant difference between the two systems in the rate of formation of oxidized nitrogen.

8.3.3 Summary of Scale-up Results. These scale-up studies have demonstrated that the COD removal rate in a RBC decreases with increasing disc diameter. This supports the theory presented by Famularo (1976), which predicted lower efficiencies in larger RBC systems due to a reduction in the oxygen transfer capability per unit area and time as disc diameter increases. Famularo's model also predicted that decreases in the dissolved oxygen concentrations in the RBC stages would accompany the reduction in oxygen transfer. Dissolved oxygen concentrations were recorded in both units during the study in the nitrification mode. In the first stage, where most of the oxidizable COD would have been removed, the 2.0 m unit exhibited DO concentrations averaging almost 2 mg/L less than in the 0.5 m unit. There was little if any difference between the two RBC's in their ability to nitrify the wastewater, suggesting that some other factor besides oxygen transfer (such as TKN diffusion into the biomass) limits RBC performance during nitrifying conditions.

It is concluded that a scale-up factor is necessary when designing a full scale RBC system from pilot data for carbon removal. Because COD mass removal was approximately 15 percent lower for the 2.0 m RBC than for the 0.5 m RBC, it is recommended that an allowance for an additional 10 percent decrease be made in scaling-

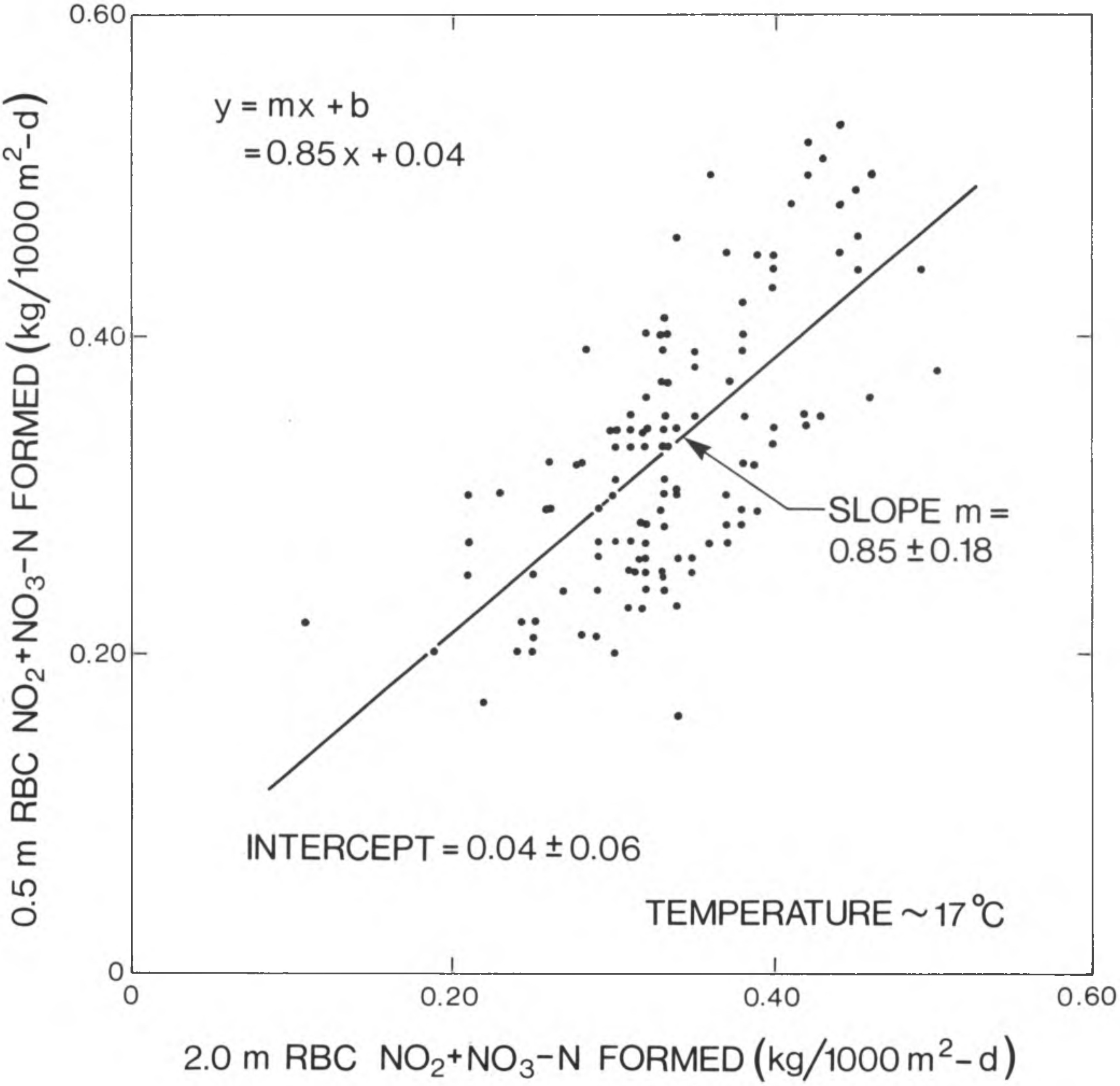

FIGURE 8-5 FORMATION OF NO_2 + NO_3 - N: 0.5 m RBC vs 2.0 m RBC (Organic Carbon Plus Nitrification Mode)

up from a 2.0 m RBC to a 3.5 m full-scale system. This suggests that surface area requirements be increased by 25 percent when designing full scale RBC facilities from 0.5 m pilot data and 10 percent from a 2.0 m unit. No scale-up factors are necessary when designing for combined carbon removal plus nitrification.

9 INTERMITTENT FLOW

9.1 Introduction

Several features of the RBC process suggest that it is well suited for small communities and for package plant applications. Low supervision requirements and process simplicity are important advantages when a treatment process for a small community is being considered. Further, there is less economy of scale for the RBC than for activated sludge or other forms of secondary biological treatment.

The extent to which the RBC may be useful for small scale installations depends largely on its ability to withstand widely varying loads and intermittent no-flow conditions. It was previously shown that the RBC can withstand large hydraulic surges without the biological slimes becoming detached from the discs and washing out of the system. Less is known, however, about its response to extended periods of very low or no-flow conditions.

Package RBC plants, complete with primary and secondary settling and sludge storage zones that are small enough for single households, are commercially available. Several researchers have attempted to determine the effectiveness of these systems under the intermittent and variable loads, which are common to all single household and small community situations. In England, Bruce and co-workers (1973) operated a 150 m^2, five-stage package system, with a steady flow for 16 hours a day and no flow for eight hours. They concluded that there was no significant difference in final effluent quality, between this mode of operation and an earlier mode. During the earlier mode the same total daily flow was applied to the RBC, but at a constant rate over 24 hours. The average loading, based on the BOD_5 load to the discs each 24 hours, was about 6 kg/1000 m^2•d. Based on instantaneous loading, the load during the intermittent flow mode would therefore have been 50 percent higher, i.e. 9 kg/1000 m^2•d. In both cases the effluent objectives of 30 mg/L suspended solids and 20 mg/L BOD_5 were achieved. A second intermittent flow test was conducted in which the same total daily hydraulic load was applied in the 16-hour flow period, and the flow varied by introducing three two-hour peaks. The small but significant decrease in effluent quality observed under these conditions was blamed primarily on poor settling in the clarifier zone due to turbulence caused by the peak flows which were at a level equal to three times the daily average. A similar decrease in quality was noticed with a constant 16-hour flow when the total daily load was increased by 30 percent.

Ahlberg and Kwong (1974) operated a five-man package RBC intermittently for two days at a constant load, followed by five days with no flow. At a hydraulic loading of 3.9×10^{-2} $m^3/m^2{\cdot}d$ during the two-day flow period, the treated effluent contained 17 mg/L of total BOD_5 and 6 mg/L of suspended solids. The total BOD_5 loading entering the primary clarifier zone was approximately 4.4 kg/1000 $m^2{\cdot}d$. Some nitrification was achieved under these conditions.

The theoretical hydraulic residence of Ahlberg and Kwong's RBC system was 2.5 days, including the internal primary and secondary clarifiers and the sludge storage zones. Although the actual residence would have been significantly less as most of the flow probably by-passed the sludge storage zone, a large portion of the two-day flow may have been retained in the system during the five-day no-flow period. The treated effluent collected during the first several hours after the flow was re-started following each five-day no-flow period would have been present in the system since the last flow period and, therefore, would be almost completely free of BOD_5 and ammonia. The same would have been true for the intermittent flow studies conducted by Bruce.

Less data are available on the response of an RBC to intermittent flows when hydraulic residence is low and flows are not equalized by clarification and sludge storage zones. As the volume of raw wastewater increases, it becomes less practical to provide theoretical hydraulic residence times as high as one or two days in a treatment system. Antonie (1970) operated a pilot RBC treating a dairy waste with a constant flow for eight hours and no flow for 16 hours a day, in order to simulate the regular wastewater flow during the eight-hour working day. The hydraulic residence in the system was approximately 60 minutes. Solids sloughed from the discs during the no-flow period each day accumulated in the wastewater in the system, thus increasing the suspended solids concentration since there was no internal clarifier or sludge storage tank. Suspended solids in the clarified effluent were high for the first couple of hours each morning after the flow was re-started. The COD removal efficiency returned very quickly to the typical steady-state levels. It was concluded that any adverse effect from having no flow for 16 hours a day was caused by the temporary increase in suspended solids concentrations in the clarifier effluent, rather than by decreased efficiency in removing filtrable substrate. That the increase in effluent solids was eliminated when a low flow was maintained, instead of eliminating the flow altogether, supports this conclusion. Solids did not accumulate in the tank overnight and COD removals remained at or above steady-state values.

Before the RBC process can be adapted to the treatment of intermittent wastewater sources, as may occur at cottage developments, rural subdivisions, schools, summer resorts or work camps, the importance of providing flow equalization must be determined. Several studies have evaluated package RBC's under field conditions. Sack and Phillips (1973) monitored a system operating for several months at a summer camp. The plant produced a satisfactory secondary effluent, but there was no opportunity to optimize the process. The nominal hydraulic residence was greater than 24 hours and an anaerobic digestion zone within the process never worked properly. A recent pilot plant demonstration of the RBC process, at a work camp on the McKenzie Highway in the Northwest Territories, has been reported by Forgie et al (1974). Unfortunately, its operation was hampered by mechanical difficulties and operator problems and few conclusions regarding the effect of the intermittent flows experienced there could be reported. Otis et al (1973) was involved in a study that compared the performance of several aerobic and physical chemical package plants for treatment of household wastes with treatment efficiencies obtained in a conventional septic system. The package system included extended and batch aeration, chemical treatment tank and an RBC septic tank combination. The RBC system performed as well as or better than the other systems, both in the controlled environment of the laboratory and the field. This, however, demonstrated a specific application of the system and provided little information on the necessity of equalization of wastewater flows when population equivalents of several hundred or more are involved.

9.2 Experimental Program

During the first of two tests, the RBC was acclimated using raw degritted wastewater at a hydraulic loading of 25×10^{-2} $m^3/m^2 \cdot d$, which provided a BOD_5 loading of approximately 25 kg/1000 $m^2 \cdot d$. The flow was then reduced by 90 percent for a five-day period, raised to the original rate for six days, and then reduced again to 10 percent for 16 hours a day in a three-day cycle. This was an attempt to simulate an intermittent operation with a small amount of flow equalization provided.

The second test was conducted during winter when the wastewater temperature was 12 to 13°C. This time, in order to simulate intermittent operations with no equalization, the RBC was operated for two days with no flow, followed by four days at a hydraulic loading of 18×10^{-2} $m^3/m^2 \cdot d$, and finishing with a three-day cycle in which there was no flow for 16 hours a day.

9.3 Results and Discussion

Figure 9-1 shows the results from the first study. The TOC, COD and TKN data represent samples composited over the period during which a steady hydraulic load was maintained. It is clear that the treated effluent quality improved during the five-day low load period, for by the time the flow was increased to the original level, complete nitrification was occurring. The degree of nitrification both before and after this five-day period was insignificant. On the last day of the low flow period, the filtrable COD in the treated effluent was less than 50 mg/L, and was non-biodegradable. The TOC and COD removals achieved during the daily cycling phase of the first run were at least equal to if not slightly higher than those at steady state, although nitrification did not recur.

At the beginning of the run a thick biomass growth was present in all four stages. Increased rates of sloughing were noticed during both the five-day and the 16-hour low flow phases of the study, but this did not appear to have any significant effect on treatment efficiencies. The thick slimes redeveloped rapidly once the higher flows were reintroduced. Some settling tests were conducted using the treated effluent, but it was impossible to determine from the limited data, whether the increase in biomass sloughing had any significant effect on the concentration of suspended solids in the clarified effluent.

The results from the second intermittent flow test are presented in Figure 9-2. This run was conducted in December, at which time the raw wastewater had increased in strength, presumably due to reduced infiltration in the Burlington Skyway Plant sewer system. A small heater had to be installed inside the insulated hood of the RBC in order to prevent freezing of the wastewater during the zero-flow periods.

The BOD_5 concentration increased from slightly over 100 mg/L in November to above 300 mg/L in December and early January. Therefore, although the hydraulic load was less than in the first run, the total BOD_5 loading was increased to between 40 and 50 kg/1000 $m^2 \cdot d$. The concentration of suspended solids in the feed followed a similar increase to values in excess of 400 mg/L. Under these conditions, it was not possible to achieve a satisfactory secondary effluent quality. Furthermore, on several occasions the concentration of suspended solids entering in the raw wastewater was artificially high due to mixed liquor spills at the Burlington Skyway plant. Nevertheless, the data in Figure 9-2 show that the elimination of flow for periods as long as two days did not seem to affect the filtrable COD removal. The data, however, are not conclusive as there were insufficient samples to allow a statistical analysis. Further work is

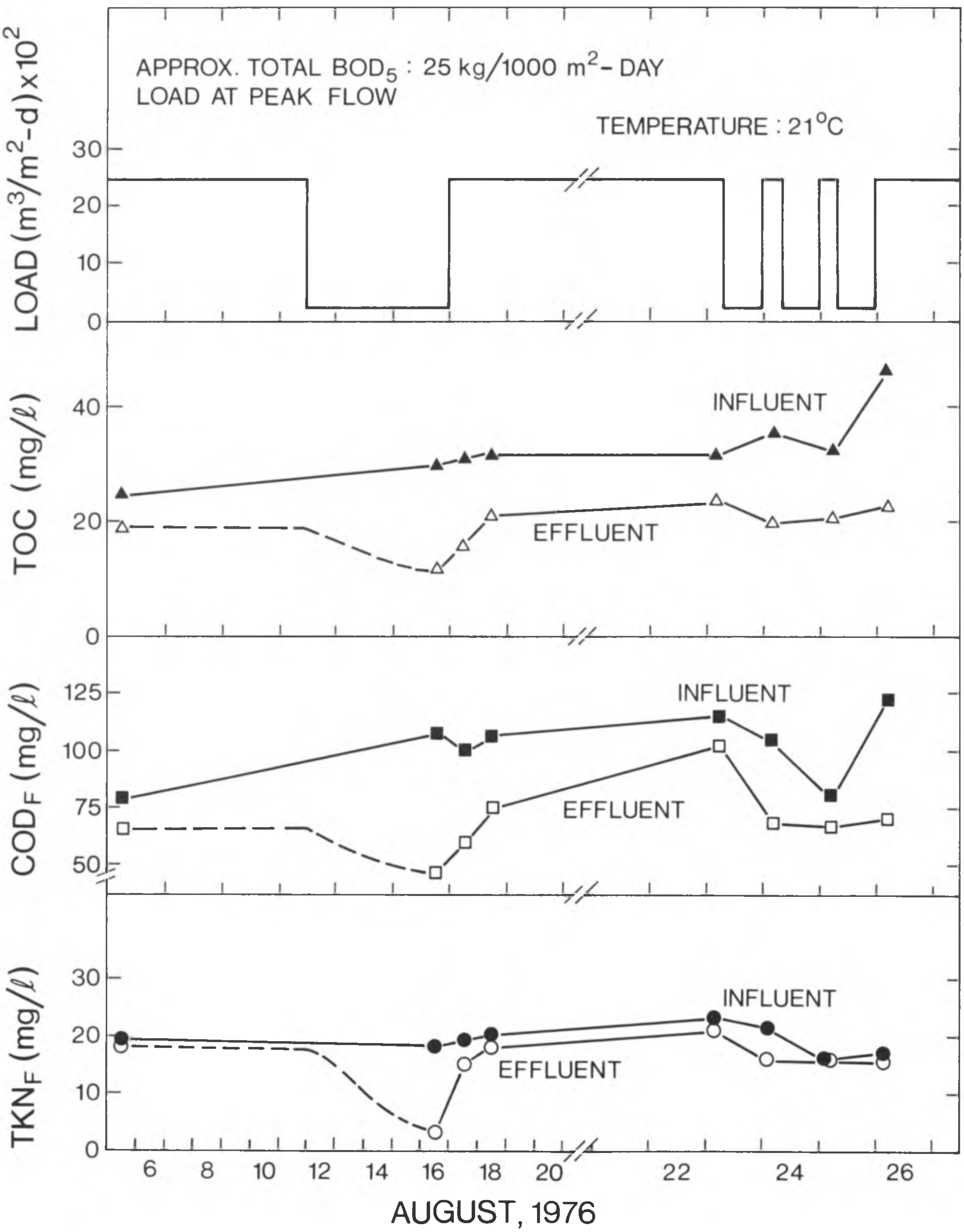

FIGURE 9-1 INTERMITTENT FLOW WITH PARTIAL EQUALIZATION (2.0 m RBC)

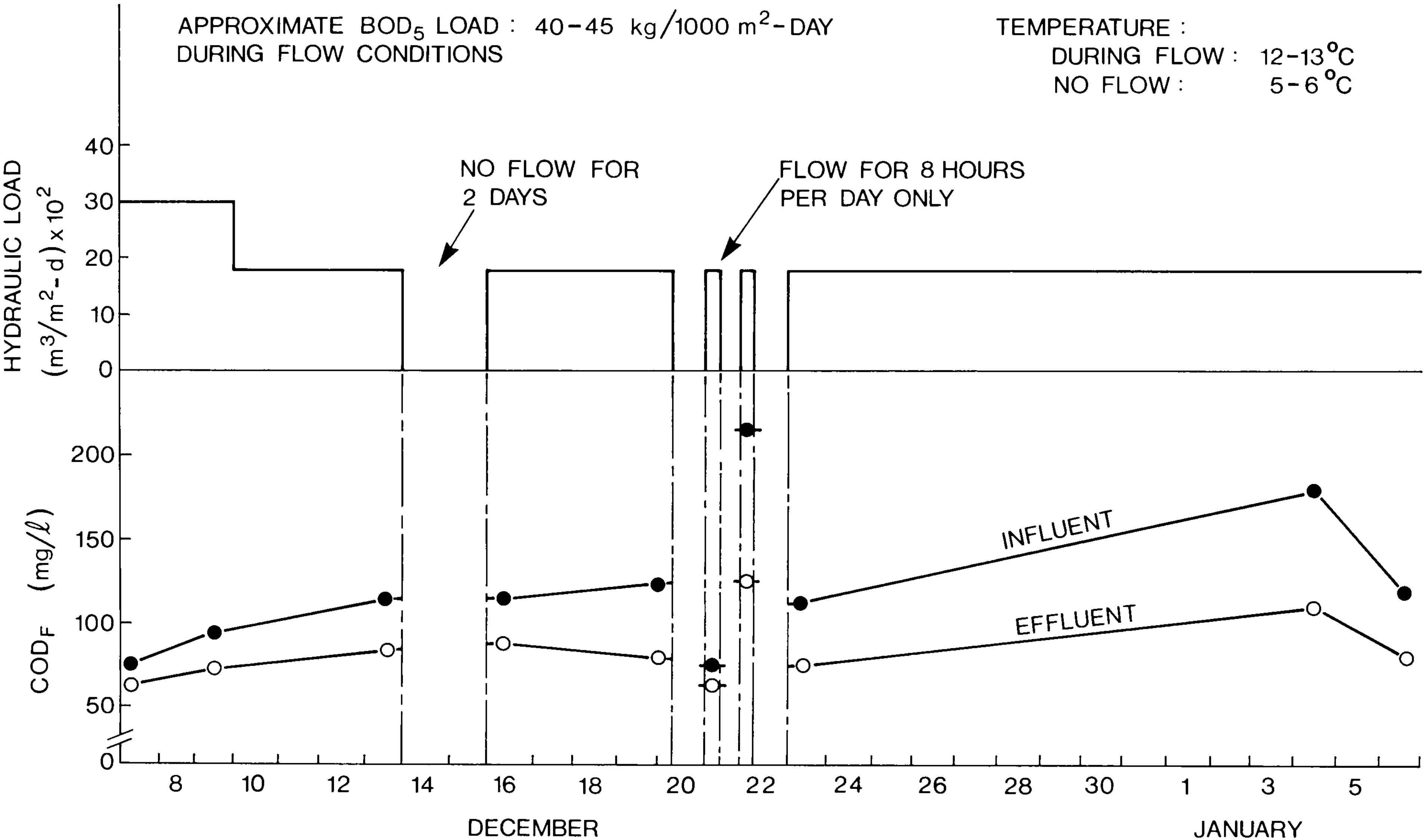

FIGURE 9-2 INTERMITTENT FLOW WITH NO EQUALIZATION (2.0 m RBC)

necessary, preferably at a low loading, with a longer cycling time during the 8-hour flow/16-hour no-flow period and with more extensive sampling. This should also include a more intensive analysis of the settleability of the suspended solids in the effluent.

10 SUMMARY OF RBC DESIGN LOADINGS

10.1 Factors Influencing Design

Normally municipal wastewater treatment plants are designed to achieve concentrations of BOD_5, TSS, TKN and/or phosphorus in the final effluent, which are less than some preset value. Since the experimental program discussed in Section 3 demonstrated that "mass loading" was the best of those parameters evaluated for predicting concentrations of BOD_5 and TKN in the treated effluent, it has been recommended as the design parameter for sizing RBC systems. For removal of BOD_5 from wastewater, the applicable mass loading is expressed as the mass of BOD_5 per unit disc area and time (e.g. kg $BOD_5/1000\ m^2{\cdot}d$). The loading for combined BOD_5 removal plus nitrification is expressed as filtrable TKN per unit area and time (e.g. kg $TKN_f/1000\ m^2{\cdot}d$).

The pseudo steady-state operating data from the 2.0 m RBC (Section 4) provides a basis for selecting RBC loadings to achieve various qualities of treated effluent. In addition, it was demonstrated that the influence of such factors as RBC size, diurnal fluctuations in wastewater flows, the efficiency of the primary solids removal process and operating temperature, must be considered when selecting a design loading for a full-scale municipal wastewater treatment facility. Greater surface area per unit mass of BOD_5 or TKN to be treated must be provided as the operating temperature decreases, as the scale of the RBC increases and as the amount of flow equalization decreases. One exception is that, when scaling up pilot data for combined BOD_5 removal and nitrification, little or no additional area is necessary.

Recommended loadings for full scale (3.5 m) RBC systems were developed for BOD_5 removal alone and for combined BOD_5 removal plus nitrification. Based on the above discussion these loadings are summarized in Table 10-1, with the detailed calculations presented in Appendix II.

In general, if the design loadings selected for primary clarified wastewater do not exceed those presented in Table 10-1, pilot plant testing should not be necessary prior to RBC design, but if higher loadings are considered, pilot testing would be required to demonstrate that the desired effluent quality can be achieved. The degree of nitrification obtained at the loadings for combined BOD_5 removal plus nitrification may not apply to systems where RBC treatment is considered as an add-on process for nitrification of a secondary treated wastewater at an existing biological treatment plant.

TABLE 10-1 DESIGN GUIDELINES FOR RBC TREATMENT OF CLARIFIED MUNICIPAL WASTEWATER

RBC System	Design Objective (average value)	Design Parameter	Design Loading ($kg/1000\ m^2 \cdot d$)			
			20°C	15°C	10°C	5°C
BOD_5 Removal	$BOD_5 \leq 15$ mg/L $SS \leq 15$ mg/L	BOD_5 loading (total)	8.5*	8.5	6.5	5.0
BOD_5 Removal	$BOD_5 \leq 30$ mg/L $SS \leq 30$ mg/L	BOD_5 loading (total)	17*	17	13	10
BOD_5 Removal Plus Nitrification	TKN < 5 mg/L	TKN loading (filtrable)	1.10	0.65	0.40	0.25

* Assumes no further increase in removal rates beyond 15°C.

10.2 Comparison of Recommended Loadings with Other Design Guidelines

In Table 10-2, examples of guidelines for RBC loadings from the literature are compared with the recommended loadings from this study.

Ontario is the only province which has established a loading guideline for RBC treatment of municipal wastewater. Assuming that the minimum wastewater operating temperature at most water pollution control plants in Ontario is 8 to 10°C, the Ministry of Environment (MOE) guideline would appear to be about 25 to 30 percent more conservative than the loading recommended from the data in this study. RBC plants for BOD_5 removal would require 25 to 30 percent more surface area using the MOE value.

A change from effluent objectives of 15/15 for BOD_5 and SS to 30/30 has a major impact on capital requirements for treatment. The loadings calculated from this study indicate that RBC plant size can be reduced by 50 percent. The economics of RBC treatment in such provinces as Alberta, where effluent objectives are commonly set at 25 to 30 mg/L of BOD_5 and SS, would be better than in Ontario.

Using the design procedures of Steels (1974), Autotrol Corporation (1976) and Envirodisc Corporation (1977), the BOD_5 loadings calculated to achieve a 15/15 effluent are considerably higher than those recommended in this study or by MOE. The loadings from Steels and Envirodisc, however, are equal to or less than the loadings recommended in this study for a 30/30 effluent.

TABLE 10-2 COMPARISON OF RBC DESIGN GUIDELINES FOR MUNICIPAL WASTEWATER

Origin	Design Objective (average value)	Design Parameter	Design Loading ($kg/1000\ m^2 \cdot d$)			
			20°C	15°C	10°C	5°C
BOD_5 REMOVAL						
This Study	$BOD_5 \leq 15$ mg/L $SS \leq 15$ mg/L	BOD_5 loading (total)	8.5	8.5	6.5	5.0
	$BOD_5 \leq 30$ mg/L $SS \leq 30$ mg/L	BOD_5 loading (total)	17	17	13	10
Ontario Ministry of the Environment (Ahlberg and Kwong, 1974)	$BOD_5 \leq 15$ mg/L $SS \leq 15$ mg/L	BOD_5 loading (total)	← 4.9 →			
Steels (1974)(1)	$BOD_5 \leq 15$ mg/L	BOD_5 loading (total)	← 14 →			
Autotrol Corp (1976)(2)	$BOD_5 \leq 15$ mg/L	BOD_5 loading (total)	26	26	23	18
Envirodisc Corp (1977)(3)	$BOD_5 \leq 15$ mg/L	BOD_5 loading (total)	13	13	9	6
BOD_5 REMOVAL PLUS NITRIFICATION						
This Study(4)	$TKN \leq 5$ mg/L	TKN loading (filtrable)	1.1	0.65	0.40	0.25
Autotrol Corp (1976)(5)	NH_4-N ≤ 2.5 mg/L	NH_4-N loading (BOD_5 loading)	5.6 (28)	4.0 (20)	2.8 (14)	1.6 (7)
Envirodisc Corp (1977)(6)	NH_4-N ≤ 2.5 mg/L	NH_4-N loading (BOD_5 loading)	1.6 (7.9)	1.6 (7.9)	1.3 (6.4)	0.8 (4.0)

(1) Design Basis: 150 mg/L total BOD_5, 40 mg/L filtrable BOD_5, 25 mg/L NH_4-N. Steels recommends that loadings be divided by safety factors ranging from 1.1 to 1.5 for situations where the population equivalent is less than 10 000.

(2) Design Basis: 150 mg/L total BOD_5, 40 mg/L filtrable BOD_5, 25 mg/L NH_4-N, Fig. C-1 and Fig. C-2 .

(3) Design Basis: 150 mg/L total BOD_5, 40 mg/L filtrable BOD_5, 25 mg/L NH_4-N, Figure C.

(4) Wastewater with total BOD_5 to filtrable TKN $\sim$6:1.

(5) Design Basis: 125 mg/L total BOD_5, 25 mg/L NH_4-N, Figure 7, Figure C-7A, Figure C-1.

(6) Design Basis: 125 mg/L total BOD_5, 25 mg/L NH_4-N, Figure J, NH_4-N conc. peaking factor = 1.4.

The NH_4-N loadings calculated using the Autotrol and Envirodisc design manuals for combined BOD_5 removal plus nitrification also appear to be high. At 10°C, the design loadings derived from them are 2.8 and 1.3 kg NH_4-N/1000 $m^2 \cdot d$, several times higher than those recommended in this study.

ABBREVIATIONS AND SYMBOLS

BOD_5	5-day biochemical oxygen demand
BOD_{5f}	filtrable 5-day biochemical oxygen demand
COD	chemical oxygen demand
CSTR	continuous stirred tank reactor
F:M	mass of BOD_5 load per mass of volatile suspended solids per day
NH_4-N	ammonia nitrogen
NO_2-N	nitrite nitrogen
NO_3-N	nitrate nitrogen
RBC	rotating biological contactor
SS	suspended solids
TKN_f	filtrable kjeldahl nitrogen
TOC	total filtrable organic carbon
PO_4-P	phosphate phosphorus
VSS	volatile suspended solids
cal/mole	calories per gram molecule

REFERENCES

Ahlberg, R.N., and T.S. Kwong (1974), "Process Evaluation of a Rotating Biological Contactor for Municipal Wastewater Treatment", Ontario Ministry of the Environment, Wastewater Treatment Section, Pollution Control Planning Branch, Research Paper No. W2041, November.

Antonie, R.L., D.L. Kluge, and J.H. Miekle (1974), "Evaluation of a Rotating Disc Wastewater Treatment Plant", J. Water. Pollut. Contr. Fed., 46(3), 498.

Antonie, R.L. (1970), "Response of the Bio-Disc Process to Fluctuating Wastewater Flows", Proceedings of the 25th Annual Purdue Industrial Waste Conference, pp. 427-435.

Autotrol Corporation (1976), Wastewater Treatment Systems, Manufacturer's Design Manual, Section C.

Borchardt, J.A. (1971), "Biological Wastewater Treatment Using Rotating Discs", Biological Waste Treatment, Wiley Interscience, New York, pp. 131-140.

Box, G.E.P. and G.M. Jenkins (1970), Time Series Analysis: Forecasting and Control, Holden Day Inc., San Francisco.

Bruce, A.M., B.L. Brown and H.T. Mann (1973), "Some Developments in the Treatment of Sewage from Small Communities", The Public Health Engineer, 72 (3), 116 (1977).

Chesner, W.H. and A.H. Molof (1977), "Biological Rotating Disc Scale-up Design: Dissolved Oxygen Effects", Progress in Water Technology, 9, pp. 811-819.

Davis, E. (1976), "Rotating Biological Contactor Evaluation Under Applied Transient Loads", Report for Technology Development Branch, Water Pollution Control Directorate, Environment Canada.

Eckenfelder, W.W. Jr., (1970), Water Quality Engineering For Practicing Engineers, Barnes & Noble Inc., New York, p. 169.

Envirodisc Corporation (1977), Rotating Disc Wastewater Treatment, Manufacturers' Design Manual.

Famularo, J., Mulligan, T., and Mueller, T.A. (1976), "Application of Mass Transfer to Rotating Biological Contactors", Presented at the 49th Annual Conference of the Water Pollution Control Federation, Minneapolis, Minnesota, October 3-8.

Filion, M.P., Murphy, K.L., and Stephenson, J.P. (1977); "Performance of a Rotating Biological Contactor Under Transient Loading Conditions", Presented at the 50th Annual Conference of the Water Pollution Control Federation, Philadelphia, October 2-7.

Forgie, D., Christensen, V., Heuchert, K., and Chong, T. (1974), "Workcamp Wastewater Treatment: Rotating Biological Contactor and Physical-Chemical Treatment", Presented at the 26th Western Canada Water and Sewage Conference, Calgary, September 27.

Friedman, A.A., Woods, R.C., and Wilkey, R.C. (1976), "Kinetic Response of Rotating Biological Contactors", Presented at the 31st Purdue Industrial Waste Conference, May 5.

Grieves, C. (1972), Dynamic and Steady State Models for the Rotating Biological Disc Reactor, Ph.D. Thesis, Clemson University, South Carolina.

Hoehn, R.C. and Ray, A. (1973) "Effects of Thickness on Bacterial Film", Jour. Wat. Poll. Contr. Fed., 45(10), pp. 2302-2320.

Joost, R.H. (1969), "Systemation in using the Rotating Biological Surface (RBC) Waste Treatment Process", Proc. 24th Purdue Industrial Waste Conference, 365-373.

Kornegay, B.H. (1972), "Modelling and Simulation of Fixed-Film Biological Reactors", Presented at the 8th Annual Workshop of the Association of Environmental Engineering Professors on Mathematical Modelling in Environmental Engineering, School of Civil Eng., Georgia Inst. of Tech., Atlanta.

Kornegay, B.A. and J.F. Andrews (1967), "Kinetics of Fixed Film Biological Reactors", Jour. Wat. Poll. Contr. Fed., 40(11), Part 2, pp. 460-476.

Labella, S.A., Thaker, I.H. and Tehan, J.E. (1972), "Treatment of Winery Wastes by Aerated Lagoon, Activated Sludge, and Rotating Biological Contactor, "Proceedings of the 27th Purdue Industrial Waste Conference, pp. 803-816.

Malhotra, S.K., T.C. Williams, and W.L. Morley (1975), "Performance of a Bio-Disc Plant in a Northern Michigan Community", Presented at the 48th Annual Conference of the Water Pollution Control Federation, Miami, Florida, October.

Murphy, K.L., P.M. Sutton, R.W. Wilson and B.E. Jank (1977), "Nitrogen Control: Design Considerations for Supported Growth Systems", Jour. Wat. Poll. Contr. Fed., 49(4): 549-557.

Otis, R.J., N.J. Hutzler, and W.C. Boyle (1973), "On-site Household Wastewater Treatment Alternatives, Laboratory and Field Studies", Presented at the Rural Environmental Engineering Conference, Warren, Vermont, September, 26-28.

Poon, C.P.C., Ya-Len Chao, and W.J. Mikucki (1977), "Evaluation of Factors Controlling the Performance of a Rotating Biological Contactor", Presented at the A.S.C.E. National Environmental Engineering Conference, July.

Pretorius, W.A. (1971), "Some Operational Characteristics of a Bacterial Disc Unit", Water Research, 5, P 1141 1146.

Sack, W.A., and S.A. Phillips (1973), "Evaluation of the Bio-Disc Treatment Process for Summer Camp Application", U.S. Environmental Protection Agency, EPA 670/2-73-022, August.

Steels, H.I. (1974), "Design Basis for the Rotating Disc Process", Effluent and Water Treatment Jour. 14(9), pp. 434-445, August.

Stover, E.L. and D.F. Kincannon (1976), "Evaluating Rotating Biological Contactor Performance", Water and Sewage Works, pp. 88-91, March.

Sutton, P.M., K.L. Murphy, B.E. Jank, and B.A. Monaghan (1977), "Reliability of Nitrification Systems With Integrated Phosphorus Precipitation", Canada-Ontario Agreement Research Report No. 64, Project No. 75-3-21, Ottawa.

Torpey, W.H., W.T. Heukelekian, A.J. Kaplovsky, and L. Epstein (1972), "Effects of Exposing Slimes on Rotating Discs to Atmospheres Enriched with Oxygen", Advances in Water Pollution Research, Jerusalem, pp. 405-417.

U.S. Environmental Protection Agency (1976), "Process Design Manual for Phosphorus Removal", Office of Technology Transfer, Washington, D.C.

Weng, C.N., and A.H. Molof (1974) , "Nitrification in the Biological Fixed-Film Disc System", Jour. Wat. Poll. Contr. Fed., 46(7), pp. 1674-1685.

Wilkey, R.C., and A.A. Friedman (1975), "Response of Rotating Biological Contactors to Shock Loadings", Presented at the 5th Annual Environmental Science and Engineering Conference, University of Kentucky, Louisville, March 3-4.

Willard, K.H., W.F. Eckerle, and R.H. Scott (1972), "Feasibility of Rotating Disc Treatment Process for Hardboard and Insulation Board Treatment", conducted for the Product Research Association, Dallas, Texas.

Williamson, K., and P.L. McCarty (1976), "A Model of Substrate Utilization by Bacterial Films", Jour. Wat. Poll. Contr. Fed. 48(1), pp. 9-23.

Wilson, R.W. (1975), "Continuous Fixed Film Biological Nitrification and Denitrification of Wastewater", M. Eng. Thesis, McMaster University, Hamilton, Ontario.

APPENDIX I

SUMMARY OF TIME SERIES ANALYSES

APPENDIX I

SUMMARY OF TIME SERIES ANALYSES

1 BACKGROUND

A time series is a set of observations generated sequentially in time. The time series generated during this study are discrete, as observations were made at one-half hour intervals, rather than continuously.

Time series analysis is a body of techniques, which allows the characterization of dynamic processes by relating observed time series of process outputs, to time series of process inputs. Predictive models called transfer function-noise models are developed and their statistical validity is checked using these techniques. Data analysis for building dynamic transfer function-noise models involves an iterative procedure to identify tentative models, to estimate model parameters and to test residuals to determine the adequacy of the fit. These models have the forms:

$$Y_t = \frac{(\omega_0 - \omega_1 \beta - \ldots - \omega_s \beta^s)\, \beta^b}{(1 - \delta_1 \beta - \delta_2 \beta^2 - \ldots - \delta_r \beta^r)} X_t + N_t$$

where:

X_t	=	input deviation from the mean at time t
Y_t	=	output deviation from the mean at time t
δ, ω	=	transfer function model parameters
r,s	=	transfer function model orders
b	=	system delay period
β	=	backward shift operator

and

$$N_t = \frac{(1 - \Theta_1 \beta - \ldots - \Theta_q \beta^q)}{(1-\beta)^d \, (1 - \phi_1 \beta - \ldots - \phi_p \beta^p)} a_t$$

where:

N_t	=	disturbances in the output which are not explained by X_t
Θ, ϕ	=	noise model parameters
p,d,q	=	noise model orders, and
a_t	=	a white noise sequence of independent random variables.

The general procedures used in the development of these models involves an iterative procedure whereby:

1) the transfer function polynomial orders (r, s) and the delay period (b) are identified by examining the cross correlation function $\rho_{xy}(k)$ and/or the impulse response weights ν_j,
2) the parameters (δ, ω) of the tentatively identified transfer functions model are estimated,
3) the form of the noise model (N_t) and its parameters are identified,
4) the parameters of the combined transfer function-noise model are re-estimated, and
5) model adequacy is verified through diagnostic checks.

<u>The Cross Correlation Function</u>

The analytical tool employed for the identification of the transfer function models is the cross correlation function between input and output, which is used to indicate the model's orders r and s, and the delay period b.

The cross correlation function between two input-output series (x and y) separated by a constant lag, k, is given by:

$$\rho_{xy}(k) = \frac{\gamma_{xy}(k)}{\sigma_x \sigma_y}, \quad k = 0, \pm 1, \pm 2, \ldots \pm K$$

where: $\gamma_{xy}(k)$ = cross covariance function between x and y, and
σ_x and σ_y = standard deviations of the x and y series.

An estimate $C_{xy}(k)$ of the cross covariance function $\gamma_{xy}(k)$ at lag k is provided by:

$$C_{xy}(k) \begin{cases} \frac{1}{n} \sum_{t=1}^{n-k} (X_t - \bar{X})(Y_{t+k} - \bar{Y}) & k = 0, 1, 2 \ldots \\ \frac{1}{n} \sum_{t=1}^{n+k} (Y_t - \bar{Y})(X_{t-k} - \bar{X}) & k = 0, -1, -2 \ldots \end{cases}$$

where: $\bar{X}$, $\bar{Y}$ = means of X,Y series.

Similarly, the estimate $r_{xy}(k)$ of the cross correlation coefficient $\rho_{xy}(k)$ at lag k may be obtained from:

$$r_{xy}(k) = \frac{C_{xy}(k)}{S_x S_y} \qquad k = 0, \pm 1, \pm 2 \ldots$$

where S_x, S_y = estimate for ρ_x, ρ_y.

Matrix inversion techniques are used to obtain estimates of the impulse response function, $\gamma(B)$, which may be obtained by solving:

$$\gamma_{xy} = \Gamma_{xx} \nu$$

where γ_{xy} = cross covariance function = $\begin{bmatrix} \gamma_{xy}(0) \\ \gamma_{xy}(1) \\ \cdot \\ \cdot \\ \cdot \\ \gamma_{xy}(k) \end{bmatrix}$

Γ_{xx} = autocorrelation function of the differenced data = $\begin{bmatrix} \gamma_{xx}(0) & \gamma_{xx}(1)\ldots & \gamma_{xx}(k) \\ \gamma_{xx}(1) & \gamma_{xx}(0)\ldots & \gamma_{xx}(k-1) \\ \cdot & & \\ \cdot & & \\ \cdot & & \\ \gamma_{xx}(k) & \gamma_{xx}(k-1)\ldots & \gamma_{xx}(0) \end{bmatrix}$

ν = impulse response function = $\begin{bmatrix} \nu_0 \\ \nu_1 \\ \cdot \\ \cdot \\ \cdot \\ \nu_k \end{bmatrix}$

To identify the transfer function models, the cross correlation function between influent and effluent variables is used. Analysis of the cross correlation function allows identification of potential transfer function models, the model orders (r, s), the delay periods (b), and the initial estimates of the parameter values (ω, δ). The autocorrelation functions of the residuals of the fitted transfer function models are used to identify the noise models, N_t. Once the parameters of the combined transfer function-noise models are estimated efficiently, using a non-linear least squares technique, model adequacies can be verified through diagnostic checks.

The transfer function model is verified by calculating the cross correlations $r_{x'\hat{a}}$ between the residuals and the input, and using these values to calculate the S-statistic. This is then compared to the Chi-Square value with K + 1 - (r+s-1) degrees of freedom.

$$S = m \sum_{k=0}^{K} r^2_{x'\hat{a}}(k)$$

Noise model verification is obtained by autocorrelating the residuals of the transfer function noise model. Again, a general lack of fit test is to calculate the Q-statistic, which may compare to Chi-Square with K - p - q degrees of freedom.

$$Q = m \sum_{k=1}^{K} r^2_{\hat{a}\hat{a}}(k)$$

2 SAMPLE CALCULATION

2.1 Identification of the Transfer Function Model: TOC Load vs TOC Effluent Concentration

After one differencing to induce stationarity in the data, the impulse response weights of the cross correlation function were examined in order to identify the transfer function model orders (r, s, b). The impulse response weights are presented in Figure I-1. In this case a review of Figure 10.6 of Box and Jenkins (1970), identified the model as a (0, 1,0) process. This simply means that the output is proportional to the input, with a zero time delay, and the response will be spread over s + 1 or 2 lags in proportion to $\nu_{b=0} = w_o$, and $\nu_{b=1} = -w_1$. All other ν_j weights are not significant, and for all practical purposes are considered equal to zero. Parameter estimates were then obtained using Table 10.1 of Box and Jenkins (1970).

The preliminary transfer function model may therefore be identified as:

$$Y_t = (w_o - w_1 \beta)\ X_t$$

with $w_o \sim \nu_o = 0.000512$

$-w_1 = \nu_1 = 0.0009918$

(Note: Y_t and X_t are differenced).

<u>Identification of the Noise Model</u>

The model under consideration is:

$$\hat{Y}_t = \frac{w(\beta)}{\delta(\beta)} X_{t-b} + N_t$$

where N_t is to be parsimoniously represented as

$$N_t = \frac{\theta(\beta)}{\nabla^d \phi(\beta)} \cdot a_t$$

Having already identified the transfer function model, N_t may be identified from:

$$N_t = Y_t - w_o X_t + w_1 X_{t-1}$$

Plots of the autocorrelation function and partial autocorrelation function of the noise sequence are given in Figure I-2. Examination of these functions reveals a

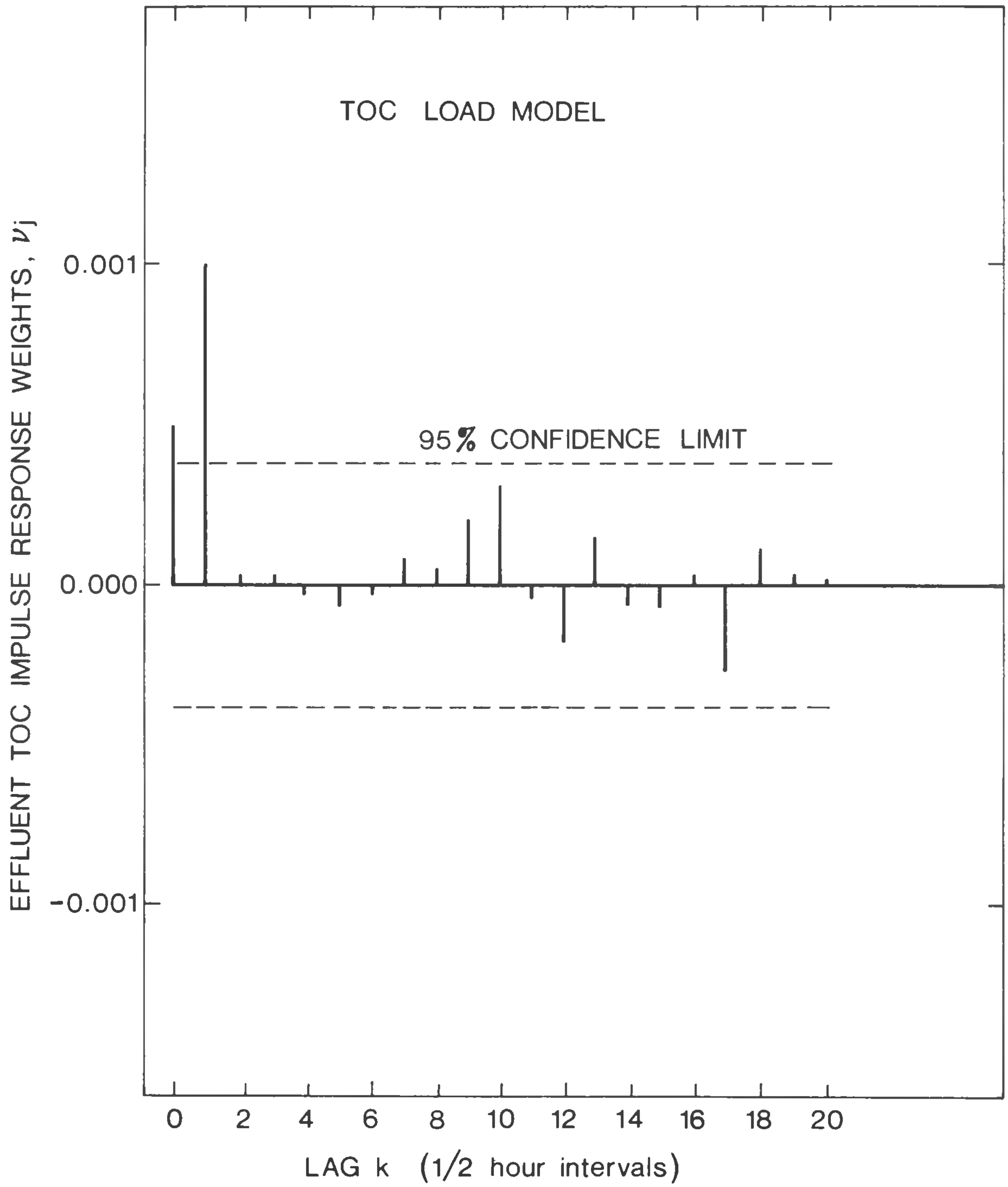

FIGURE I-1 IMPULSE RESPONSE WEIGHTS FOR EFFLUENT TOC CONCENTRATION (Carbon Removal Mode)

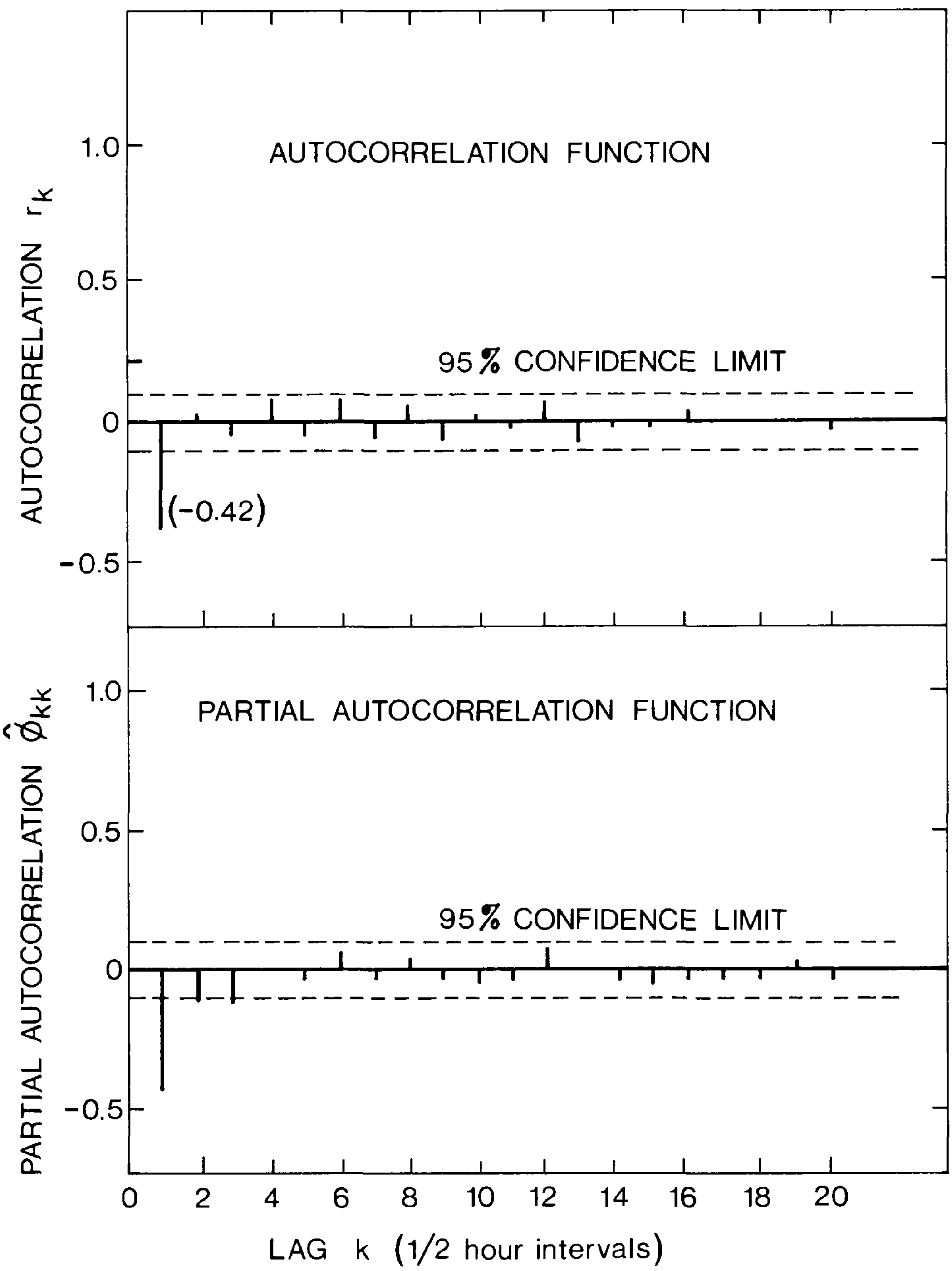

FIGURE I-2 AUTOCORRELATION FUNCTION AND PARTIAL AUTOCORRELATION FUNCTION OF TIME SERIES DATA (Carbon Removal Mode) decaying partial autocorrelation function which becomes negligible after lag 3. A single significant spike at lag 1 is noted for the autocorrelation function. This is indicative of a moving average (MA) process of order 1, or

$$N_t = (1 - \theta_1 \beta) \cdot a_t$$

The value of θ_1 can be estimated from r_k in Figure I-2, where r_k is an estimate of the autocorrelation coefficient ρ_k and using the following expression:

$$r_k \simeq \rho_k = \theta_1/(1 + \theta_1^2)$$

$$\theta_1 = 0.55 \text{ for } r_k = -0.422$$

The complete transfer function-noise model has now been tentatively identified as having the following form.

$$Y_t = (w_o - w_1 \beta) X_t + (1 - \theta_1 \beta)\, a_t$$

2.2 Refining Parameter Estimates and Applying Diagnostic Checks of Model Adequacy

The initial parameter estimates obtained in the previous section were used as starting values in a non-linear least squares estimating procedure to optimize the parameter values. This was accomplished using a computer program (TS HAUS) to provide simultaneous estimates of the dynamic and the noise parameters including 95 percent confidence limits for each. These parameter estimates appear in Table I-1.

TABLE I-1 PARAMETER ESTIMATES FROM NON-LINEAR LEAST SQUARES ANALYSIS

Model	Parameters	Initial Estimates	Final Estimates	95% Confidence Interval
Transfer	w_o	5.12×10^{-4}	4.703×10^{-4}	7.504×10^{-4}, 1.91×10^{-4}
Function	w_1	9.917×10^{-4}	1.065×10^{-3}	1.345×10^{-3}, 7.856×10^{-4}
Noise	θ_1	5.5×10^{-1}	5.889×10^{-1}	7.378×10^{-1}, 4.4×10^{-1}

The two most frequently used checks applied to diagnose transfer function-noise model adequacy (or inadequacy as the case may be) involves the autocorrelation function $r_{\hat{a}\hat{a}}(k)$ of the residuals from the fitted model, and the cross correlation function between pre-whitened input (or in our case the first difference ∇x_t) and the model residuals. Assuming the form of the transfer function-noise model is correct, and that the two parameter values are known, then the estimated autocorrelation function of the residuals should be uncorrelated and distributed normally about zero with variance n^{-1}, where n is the number of data points in the series. These checks are presented in Figures I-3 and I-4. The results in Figures I-3 and I-4 indicate that there is no significant transfer function model or noise model inadequacy and that the chosen models adequately represent the data. The significant peak is of no concern. Due to the experimental design employed for this work, it is not uncommon to detect significant peaks at lags ≥ 10, which is the point at which the step changes took place.

A further assessment of the residuals involves taking the first k autocorrelations and computing the Q-statistic. Comparing the result to the Chi-Square distribution with k-p-q degrees of freedom, where p+q is the total number of parameters of the noise model, determines the significance of the residuals. In a similar manner, the significance of the cross correlations between residuals and the stationary input series of each variable, can be computed by comparing the S-statistic to the Chi-Square distribution. The results given in Table I-2 show that there is no apparent lack of fit for the model, as the Q-and S-statistics are smaller than the respective Chi-Square values.

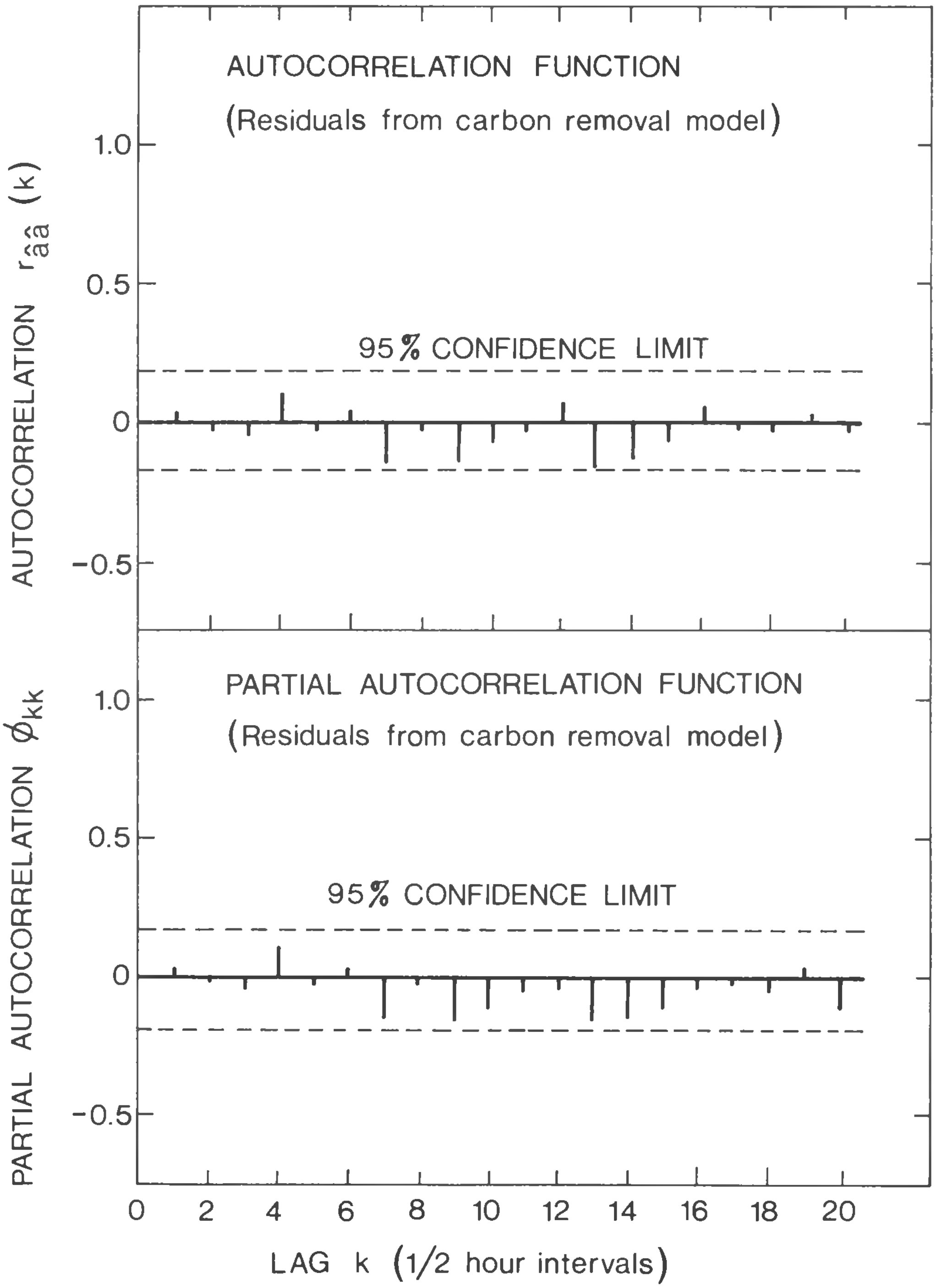

FIGURE I-3 AUTOCORRELATION AND PARTIAL AUTOCORRELATION OF TRANSFER FUNCTION-NOISE MODEL RESIDUALS (Carbon Removal Mode)

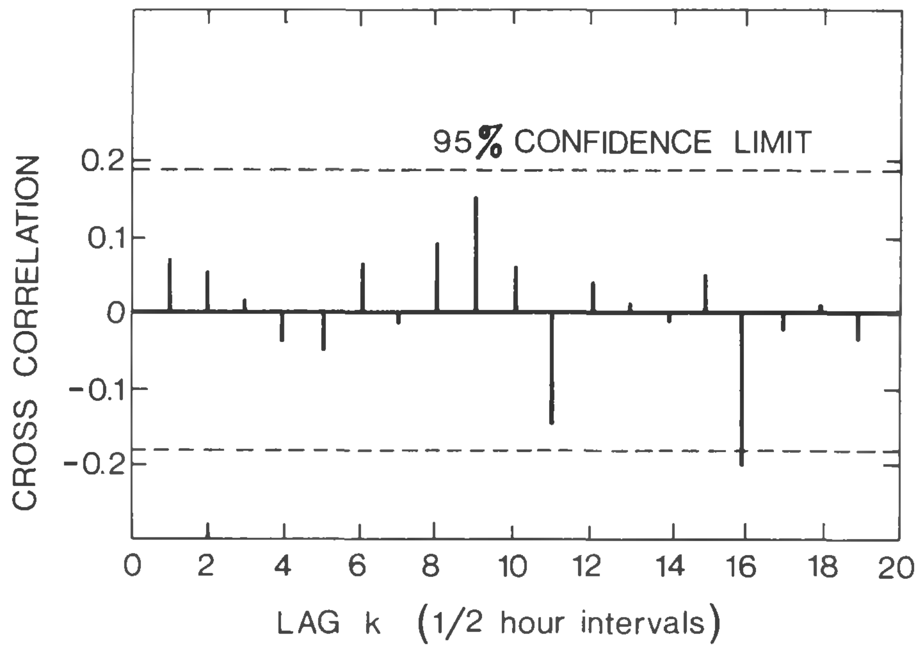

FIGURE I-4 CROSS CORRELATION BETWEEN PREWHITENED INPUT AND MODEL RESIDUALS (Carbon Removal Mode)

TABLE I-2 DIAGNOSTIC CHECKS TO TEST MODEL ADEQUACY

Autocorrelation of Model Residuals	Cross Correlation of Pre-whitened Input and Model Residuals
If Q-statistic $= n \sum_{k=1}^{20} r^2_{\hat{a}\hat{a}}(k)$	If S-statistic $= n \sum_{k=1}^{20} r^2_{x'\hat{a}}(k)$
where n = no. of observations	where n = no. of observations
k = lag	$x' = (1-\beta)X$
$\hat{a}$ = model residuals	
AND Chi-Square $= X^2_{\nu,\alpha}$	AND Chi-Square $= X^2_{\nu,\alpha}$
where ν = K-r-s	where ν = K-p-q
= 20-1-0	= 20-1-1
$\alpha = 0.95$	$\alpha = 0.95$
THEN $Q = 15.0$; $X^2 = 31.4$	THEN $S = 13.9$; $X^2 = 32.7$
AND $Q < X^2_{\nu,\alpha}$	AND $S < X^2_{\nu,\alpha}$

APPENDIX 2

SELECTION OF DESIGN PARAMETERS

APPENDIX 2

SELECTION OF DESIGN PARAMETERS

General Design Basis:

1) short hydraulic detention system
2) diurnal flow of peak to mean to minimum = 2:1:0.5
3) final effluent suspended solids contain 1/3 part BOD_5 per 1 part SS
4) system includes primary solids removal

A. Organic Carbon Removal: (average 15 mg/L SS:15 mg/L carbonaceous BOD_5)

Calculate Filtrable BOD_5 In Effluent

BOD_{5f} = 15 mg total BOD_5/L - 1/3 (15) mg suspended BOD_5/L
= 10 mg filtrable BOD_5/L

Determine Basic BOD_5 Loading At 12°C

BOD_5 Loading = 10 kg total $BOD_5/1000\ m^2 \cdot d$ for an effluent of 10 mg BOD_{5f}/L (Figure 4-6, Section 4.3.2)

Scale-up Factor

Assume Scale-up Factor = 1.10 (Section 8.3.3).

∴ revised BOD_5 Loading = $\frac{10}{1.1}$ kg total $BOD_5/1000\ m^2 \cdot d$
= 9.1 kg total $BOD_5/1000\ m^2 \cdot d$

Compensation for Diurnal Variations

Assume Diurnal Factor = 1.25 (Section 5.3)

∴ Revised BOD_5 loading = $\frac{9.1}{1.25}$ kg total $BOD_5/1000\ m^2 \cdot d$
= 7.3 kg total $BOD_5/1000\ m^2 \cdot d$

Temperature Correction

Assume temperature coefficient = 1.05 (Section 4.4)

	5°C	10°C	15°C
Loading ($kg/1000\ m^2 \cdot d$)	$7.3 \times 1.05^{(5-12)}$	$7.3 \times 1.05^{(10-12)}$	$7.3 \times 1.05^{(15-12)}$
Loading ($kg/1000\ m^2 \cdot d$)	5.2	6.6	8.5
Say	5.0	6.5	8.5

B. Organic Carbon Removal: (average 30 mg/L SS: 30 mg/L carbonaceous BOD_5)

Calculate filtrable BOD_5 in effluent

BOD_{5f} = 30 mg total BOD_5/L - 1/3 (30) mg suspended BOD_5/L
= 20 mg filtrable BOD_5/L

Determine Basic BOD_5 Loading at 12°C

BOD_5 Loading = 20 kg total BOD_5/1000 m^2•d for an effluent with 20 mg/L filtrable BOD_5 (Figure 4-6).

Scale-Up Factor

Assume Scale-Up Factor = 1.10 (Section 8.3.3)

∴ Revised BOD_5 Loading = $\frac{20}{1.1}$ kg total BOD_5/1000 m^2•d
= 18 kg total BOD_5/1000 m^2•d

Compensation of Diurnal Variations

Assume Diurnal Factor = 1.25 (Section 5.3)

∴ Revised BOD_5 Loading = $\frac{1.8}{1.25}$ kg total BOD_5/1000 m^2•d
= 14.5 kg total BOD_5/1000 m^2•d

Temperature Correction:

Assume temperature coefficient = 1.05 (Section 4.4)

	5°C	10°C	15°C
Loading (kg/1000 m^2•d)	$14.5 \times 1.05^{(5-12)}$	$14.5 \times 1.05^{(10-12)}$	$14.5 \times 1.05^{(15-12)}$
Loading (kg/1000 m^2•d)	10.4	13.2	17
Say	10	13	17

C. Combined Carbon Removal/Nitrification (5 mg TKN/L)

Select Base TKN Loading at 16°C

TKN Loading = 1.0 kg filtrable TKN/1000 m^2•d for nitrification (Section 4.5)

The pseudo steady-state data in Figure 4-3 indicate consistent filtrable TKN removals of 80-90% at a loading of 1.0 kg filtrable TKN/1000 m^2•d. This will produce an effluent with a filtrable TKN of 5 mg/L or less, assuming maximum influent concentration of about 30 mg/L.

This applies only to wastewater with a ratio of total BOD_5 to filtrable TKN of approximately 6:1. Lower loadings may be required at ratios significantly higher than this.

Scale-Up Factor

Assume Scale-up Factor	=	0 (Section 8.3.3)
∴ Revised TKN loading	=	1.0 kg filtrable TKN/1000 m^2•d

Compensation for Diurnal Flow

Assume Diurnal Factor	=	1.35 (Section 5.3)
∴ Revised TKN Loading	=	$\frac{1.0}{1.35}$ kg filtrable TKN/1000 m^2•d
	=	0.74 kg filtrable TKN/1000 m^2•d

Temperature Correction

Assume temperature coefficient = 1.1 (Section 4.4)

	5°C	10°C	15°C	20°C
Loading (kg/1000 m^2•d)	$0.74 \times 1.1^{(5-16)}$	$0.74 \times 1.1^{(10-16)}$	$0.74 \times 1.1^{(15-16)}$	$0.74 \times 1.1^{(20-16)}$
Loading (kg/1000 m^2•d)	0.25	0.42	0.67	1.07
Say	0.25	0.40	0.65	1.1