MAY 1993

ONSITE TREATMENT OF SEPTIC TANK EFFLUENT: AN EVALUATION OF ROTATING BIOLOGICAL CONTACTOR CAPABILITIES

WRRI Report No. 277

NEW MEXICO WATER RESOURCES RESEARCH INSTITUTE
New Mexico State University
Box 30001, Dept. 3167
Las Cruces, New Mexico 88003-0001

ONSITE TREATMENT OF SEPTIC TANK EFFLUENT: AN EVALUATION OF ROTATING BIOLOGICAL CONTACTOR CAPABILITIES

By
Ricardo B. Jacquez
Principal Investigator
Department of Civil, Agricultural, and Geological Engineering
New Mexico State University

Firojbhai Vora
Graduate Research Assistant
Department of Civil, Agricultural, and Geological Engineering
New Mexico State University

and

Nehal Abdul-Kareem Zaidan
Graduate Research Assistant
Department of Civil, Agricultural, and Geological Engineering
New Mexico State University

TECHNICAL COMPLETION REPORT

Account Number 1423632

May 1993

New Mexico Water Resources Research Institute

in cooperation with

Department of Civil, Agricultural, and Geological Engineering
New Mexico State University

The research on which this report is based was financed in part by the U.S. Department of Interior, Geological Survey, through the New Mexico Water Resources Research Institute.

The purpose of Water Resources Research Institute technical reports is to provide a timely outlet for research results obtained on projects supported in whole or in part by the institute. Through these reports, we are promoting the free exchange of information and ideas, and hope to stimulate thoughtful discussion and actions that may lead to resolution of water problems. The WRRI, through peer review of draft reports, attempts to substantiate the accuracy of information contained in its reports, but the views expressed are those of the author(s) and do not necessarily reflect those of the WRRI or its reviewers. Contents of this publication do not necessarily reflect the views and policies of the U.S. Department of the Interior, nor does mention of trade names or commercial products constitute their endorsement by the United States government.

ACKNOWLEDGMENTS

The authors wish to express their appreciation to Mr. R.P. Lathis for his cooperation in allowing Val Verde Mobile Home Park to be used as a test site for the project.

Special thanks are also extended to Mr. Don Richardson for his assistance in the construction of the rotating biological contactor system used by the research project.

Gratitude is also expressed to the Civil, Agricultural, and Geological Engineering Department for its support in providing research facilities.

ABSTRACT

A rotating biological contactor (RBC) was evaluated with respect to nitrogen removal and practicality of use in tandem with a septic tank at a single family dwelling. The unit was used to treat wastewater generated by five persons and which was discharged into a 1500-gallon septic tank. Principal operational variables evaluated included hydraulic loading rate, disc rotational speed, anoxic stages, and varying recycle of effluent. Two modifications were made to promote nitrogen removal in the system; establishing anoxic conditions in the third stage, and making the first stage anoxic and recycling final effluent. Effluent was recycled at a ratio (recycle: influent) of 3:1, 2:1, 1:1, and zero. The results showed that the RBC was capable of achieving nitrification/denitrification while simultaneously reducing carbonaceous oxygen demand. The optimum treatment efficiency was achieved at a hydraulic loading rate of 1.0 gpd/ft^2 and disc rotational speed of 9 rpm conditions. The RBC showed 70%-90% Chemical Oxygen Demand (COD) removal and 60%-70% Total Suspended Solids (TSS) removal efficiency. Even after a four-day power failure, the biofilm was able to regain a stable operation within 24-48 hours and provide a COD removal efficiency in the range of 60%-70%. Total nitrogen removal was maximized (82%) by establishing anoxic conditions in the first stage and eliminating the recycle of effluent. The RBC appears to be particularly attractive for small communities and/or for an individual home because of its high treatment efficiency and low energy demand. Although the system required frequent labor attention to prevent hydraulic operational problems the use of an RBC is recommended where nitrogen contamination of groundwater is problematic.

Keywords: water pollution, groundwater, septic tanks, nitrogen, rotating biological contactor.

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

INTRODUCTION

Groundwater is a resource of immense value, comprising 25% of all the fresh water available for use in the United States. It supplies the needs of 50% of the nation's population and 75% of the public water systems. Groundwater supplies 95% of the drinking water in rural areas (Bitton and Gerba 1984).

Groundwater contamination has occurred in many states in the nation and is being detected with increasing frequency especially in the areas having high densities of septic tank systems (Bitton and Gerba 1984, Canter and Knox 1986, McQuillan 1987). Septic tank systems have been identified as a source of localized and regional groundwater pollution. Approximately 1/3 of all existing housing units and about 25% of all new homes being constructed in the United States dispose of domestic wastewater through septic tank systems. They are the most widely used method of treatment in rural and suburban areas because they are economical and simple to operate. However, because many existing systems have exceeded their design life by several fold, septic tank systems rank highest in the total volume of wastewater discharged directly into the groundwater and the most frequently reported source of groundwater contamination.

Although historical concerns have focused on bacterial and nitrate pollution, more recently, synthetic organic chemicals such as trichloroethylene (TCE), benzene, and methylene chloride have been used as degreasing agents to clean septic systems, and identified as potential contaminants (Canter and Knox 1986, Bitton and Gerba 1984). When the soil capacity to absorb effluent from the septic tank has been exceeded, pollutants, especially nitrates, move rapidly through the soil.

Nitrogen contamination of groundwater by septic tank systems is a frequent occurrence (Canter and Knox 1986). The two nitrogen forms of major concern relative to groundwater pollution are ammonium ions (NH_4^+) and nitrates (NO_3^-) (Canter and Knox 1986). Ammonium ions are discharged directly to a soil absorption system and into the subsurface environment. Adsorption is the major mechanism of NH_4^+ removal in the soil. The adsorption capacity of the leach field can be exceeded as the soil becomes saturated. Therefore, ammonia can migrate into groundwater if the effluent is transmitted through a continuously saturated soil into the aquifer. Nitrate ions can also be discharged directly from soil absorption systems into the subsurface environment, or they can be generated within the upper layers of the soil through the nitrification process. The transport and fate of the nitrate ions may involve movement with the water phase, uptake in plants or denitrification (Canter and Knox 1986). Since nitrate ions have a negative charge, they are not attracted to soils which generally possess a negative charge. Accordingly, nitrates are more mobile than ammonium ions in both unsaturated and saturated soils. Nitrogen in the form of nitrate usually reaches groundwater, and becomes very mobile because of its solubility and anionic form. It can migrate long distances from input areas and can reach the groundwater with minimal transformation.

Historical concerns also have focused on bacterial and nitrate pollution because they can be a health hazard. Nitrate levels more than 10 mg/L can cause infantile methemoglobinemia, a rare but potentially serious and sometimes fatal disease affecting infants. Several recent studies have reported on the extent of nitrate contamination of groundwater adjacent to septic tank seepage beds (Canter and Knox 1986). Groundwater nitrate levels resulting from household septic tank contamination can be as high as 30 mg/L NO_3- N, which is three times the health standard (10 mg/L) established by Environmental Protection Agency (EPA) for drinking water (McQuillan and Keller 1988). This level of nitrate contamination is present in New Mexico and other regions of the United States (McQuillan and Keller 1988).

From this evidence, it is clear that septic tanks represent a potential threat to groundwater quality. The following methods have been proposed to minimize the possibility of groundwater pollution resulting from septic tank system use, especially those experiencing problems with overloaded soil absorption fields:

- Improving operational practices to decrease the strength of the septage, such as more frequent inspection and pumping of the septic tank.

- Connecting to a centralized sewerage system and treating the waste at a wastewater treatment plant
- Reducing the volume and strength entering the septic tank by segregating the water originating from different sources in the household, such as toilet waste (black water) and other household wastewater (gray water). The larger volume of gray water could be used for other beneficial purposes such as irrigation, gardening, etc. if state law permits. Removal of black water from the household waste stream through the use of nonconventional toilet systems can reduce the loading on the septic tank and absorption system (Canter and Knox 1986)
- Reducing the effluents nitrate content by using it to grow crops having a long growing season and a high nitrogen requirement (Canter and Knox 1986)
- Constructing septic tank mounds to limit groundwater pollution by enhancing microbial degradation

These approaches may minimize the chance of groundwater pollution but they may not be economically feasible and/or practical to implement. A more beneficial alternative is to modify the existing septic tank. A typical septic tank system consists of two units: a buried tank to collect the wastewater and separate the scum, grease, and settleable solids, and a soil absorption system where clarified effluent percolates into the soil (EPA 1980). The most critical stage is the soil absorption field because any failure in this system will cause the contaminants to reach the groundwater. The removal efficiencies of the soil are correlated with the depth of unsaturated soil. It has been estimated that as many as one-half of all septic tank soil absorption systems are not operating satisfactorily (Canter and Knox 1986). Therefore, the best approach to protect groundwater may be the treatment of the septic tank effluent before discharging it to the leach field. The supplemental treatment technology should be economical, efficient, and operationally reliable to serve both as a retrofit to existing septic tanks and as an integral component of a new septic tank system.

Using a rotating biological contactor (RBC) as a second-stage treatment to remove the contaminants from the wastewater will allow for direct disposal of the effluent into the leach field without threatening the groundwater. The advantages for the RBC system include flexibility, efficient treatment, process stability, low maintenance and power consumption, provision for nitrification/denitrification, and good sludge settling characteristics (Antonie 1978). Also, the construction and operation of the RBC enables it to be readily scaled up or down over a wide range of equipment sizes with no loss of process efficiency or change in effect of process variables. This flexibility allows the process to be applied to various wastewater treatment applications including the treatment requirements of an individual residence (Antonie 1978).

Numerous studies have evaluated the treatability and performance of pilot-scale RBC units. It is well established that RBCs can provide high levels of organic removal and are efficient in simultaneous nitrification and denitrification (Atwater and Bradshaw 1981; Masuda et al. 1982, Pano and Middlebrooks 1983, Rusten and Odegaard 1982). Pano and Middlebrooks (1983) found that RBCs are able to achieve removal efficiencies 80% or higher for both carbon and ammonia oxidation. Masuda and Rusten (1982) and Odegaard (1982) reported that nitrogen removal higher than 90% can be achieved along with biochemical oxygen demand (BOD) removal in the RBC system. These authors confirmed the RBC's capability to be used to nitrify and denitrify domestic wastewater.

RESEARCH OBJECTIVE

Despite the relatively wide use of RBC s in municipal systems, presently there are insufficient data available to determine the capabilities of an RBC system to treat organic contaminants from a domestic septic tank discharge. The objective of this research was to evaluate the practicality and performance of a small-scale RBC system, placed in tandem with a single family dwelling septic tank, as an economical on-site treatment system to stabilize and reduce carbon and nitrogen contaminants from septic tank effluent.

MATERIALS AND METHODS

DESCRIPTION OF THE PILOT-SCALE RBC

The RBC was installed at a mobile home park in Dona Ana, NM. The pilot-scale RBC served five persons living in two mobile homes. One mobile home included a family of two adults with two children and the second included one single adult. A flow diagram of the pilot-scale RBC system is shown in Figure 1. Wastewater was pumped on a continuous basis from the secondary compartment of the septic tank to the RBC by means of a 0.5 HP (0.37 kW) submersible pump. The discs were held on a central shaft system which was driven independently by a 0.75 HP (0.56 kW) variable speed electric motor. A summary of the physical properties of the RBC unit is presented in Table 1.

Table 1. Physical Properties of the Pilot-Scale RBC Unit.

Item	Description
Disc Media	0.125 in. Thick Plexiglas
Spacing of Discs	0.5 in. Clear Space Disc to Disc
Diameter of Discs	19 in.
Number of Discs per Stage	13
Number of Stages	4
Total Surface Area	205 ft^2
Submerged Area (%)	42
Net Liquid Volume of Reactor	18.5 gal.
Volume to Surface Area Ratio	0.09 gal/ft^2

Effluent from the RBC flowed to a 15-gallon, conical shaped, secondary clarifier which removed the biological solids produced by the RBC through gravity settling. Secondary sludge was drawn from the clarifier by a manually operated valve. The sludge collected from the clarifier was quantified on a regular basis to determine the sludge yield coefficient. The final effluent from the secondary clarifier was discharged directly to the septic tank leach field. The RBC was housed in a metal tool sheed (8 ft x 10 ft x 6 ft) to protect the unit from vandals and adverse elements.

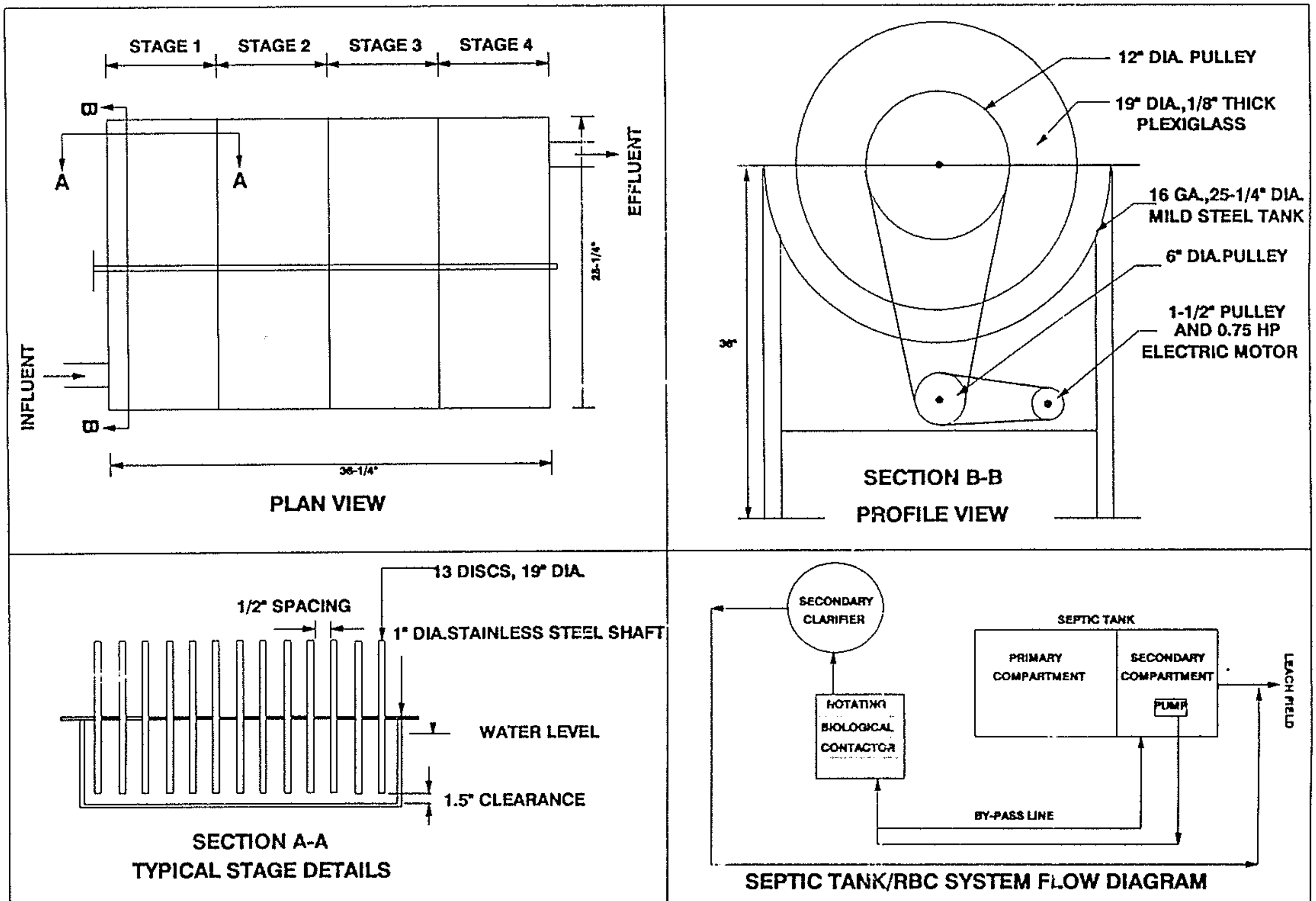

Figure 1. Construction Details of RBC Unit And Experimental Flow Diagram.

RESEARCH PROGRAM

The study included a 24-month period extending from July 1989 through June 1991. The operational program included three phases of study. To monitor the performance of the RBC, grab samples were taken from the influent at the four stages of the reactor and from the effluent of the secondary clarifier, typically during the period from 9:00 to 11:00 am. The experimental program and sampling and analytical procedures for each phase are described below.

Phase I

Phase I included an eight-month period extending from July 1989 through February 1990. During this period the four stages of the RBC were maintained aerobic to study removal of carbonaceous oxygen demand by the RBC. During this preliminary stage, optimum operational conditions for carbonaceous removal were determined. Sampling and analyses were performed three times per week. Each sample was analyzed for total and soluble chemical oxygen demand (COD), total suspended solids (TSS), volatile suspended solids (VSS), ammonia nitrogen (NH_3-N), and nitrate nitrogen (NO_3-N). Influent and effluent samples were also analyzed for total Kjeldahal nitrogen (TKN).

Optimum operating conditions for the RBC were determined by an experimental program performed in three operational periods. Each operational mode continued until the steady-state condition, a relatively constant soluble effluent COD concentration, was established. Whenever one or more of the operational parameters were altered in operation, the RBC was allowed at least three days to adjust to the operational change.

Influence of Hydraulic Loading Rate

This experimental phase evaluated the effect of hydraulic loading on RBC performance. During this study the discs were turned at a constant rotational speed of 6 rpm. The RBC was operated at hydraulic loading rates of 0.5, 1.0, 2.0, and 4.0 gpd/ft^2. The optimal rate was determined based on total and soluble COD removal efficiency.

Influence of Rotational Speed

The next experimental phase evaluated the effect of disc rotational speed on RBC performance. The optimal hydraulic loading determined in the first phase was used to test COD removal efficiencies corresponding to variations in disc rotational speeds of 6, 9, and 12 rpm.

Influence of Intermittent Power Supply Outage

This experiment evaluated the effect of intermittent operation of the RBC due to power loss. Power to the RBC was turned off for a period of four consecutive days. When the power supply was activated, influent and final effluent samples were collected after 2, 4, 6, and 24 hours of elapsed time. The samples were analyzed for soluble and total COD concentration.

Phase II

Phase II included a five-month period extending from March through July 1990. This phase studied the RBC's capability to nitrify and denitrify the wastewater. Denitrification was implemented by making the third stage anaerobic by stopping the disks from rotating. Sampling and laboratory testing were performed three times per week. Influent and effluent samples were analyzed for ammonia, nitrate nitrogen, and TKN concentrations, while the samples from the four stages were tested for ammonia and nitrate species only. Total and soluble COD were analyzed infrequently during this period to check accuracy.

Phase III

Phase III included an 11-month period extending from August 1990 through June 1991. During this phase, a second modification was made to maximize the biological nitrification/denitrification processes. The principle factor evaluated was the creation of an anaerobic environment with raw wastewater a carbon source to promote denitrification. The first stage was maintained anaerobically by preventing the disks from rotating. In addition, effluent from the secondary clarifier was recycled to stage one at four ratios (recycle: influent) 3:1, 2:1, 1:1 and 0. Secondary clarifier effluent was pumped continuously (0.024 hp, 0.018 kW pump) to a 36-gallon mix tank located between the septic tank and the RBC. The septic tank effluent was pumped by means of a 0.125 hp (0.0925 kW) submersible pump to the mix tank six minutes per hour to maintain a 200 gpd flow rate. The influent mixture flowed to the RBC by gravity. Sludge and scum portions were pumped from the septic tank at the beginning of this phase (Figure 2). The following analyses were performed: NH_3-N for all samples twice per week, NO_3-N for all samples one to two times per week, TKN for influent and effluent samples once per week, and total and soluble COD one to two times per week for all samples. TSS was measured periodically.

ANALYTICAL PROCEDURES

Total and Soluble Chemical Oxygen Demand

Total and soluble COD concentrations were determined by method 508 C, the Closed Reflux Colorimetric Technique, outlined in Standard Methods (AWWA et al. 1985). Suspended solids interference was eliminated from the soluble COD measurement by filtering the sample through a 0.45 μm glass fiber filter.

Biochemical Oxygen Demand

Periodic measurements of BOD_5 were performed in accordance with Standard Methods section 507 (AWWA et al. 1985).

Total and Volatile Suspended Solids

TSS and VSS measurements were made according to modified procedures outlined in Standard Methods sections 209 C and 209 D, respectively (AWWA et al. 1985). TSS concentrations of the secondary clarifier sludge were measured frequently. A 50 ml sample was centrifuged at 9000 rpm for ten minutes to separate solids and liquid. Subsequently, the solids were resuspended in distilled water to a final volume of 50 ml. The centrifuging step was repeated two additional times. After the third centrifuge step, the solids were oven dried for 10 hours at 103° C before weighing.

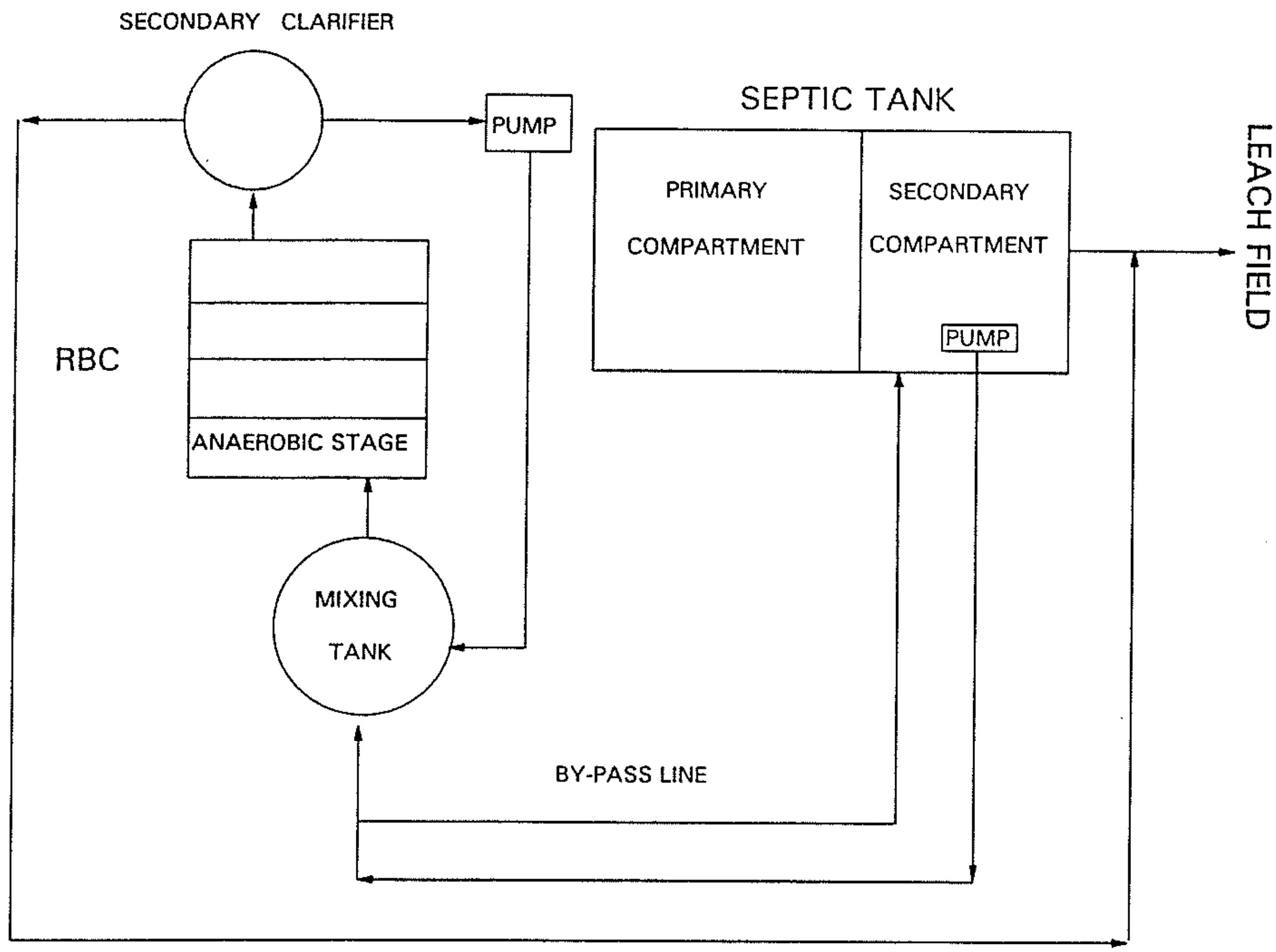

Figure 2. Septic Tank/RBC System Flow Diagram During Phase III

Dissolved Oxygen (DO) and pH

Dissolved oxygen and pH measurements were performed periodically on the influent, effluent, and in each RBC stage. A YSI model 54A DO meter and YSI 5439 DO probe were used to measure oxygen concentrations. A Beckman D 40 pH meter was used for pH measurements.

Power Consumption

The electric current consumed by the RBC and pump was metered periodically by a Fluke 75 Multimeter and LCA-10 Line Current Test Adapter. The motor turning the RBC discs and the submersible pump were connected in series during the electric power measurement. Using the voltage power measurements, kilowatt demand was calculated using power factors provided by the local electric company.

Ammonia-Nitrogen (NH3-N)

Ammonia-nitrogen was analyzed according to Standard Methods Section 417 A (AWWA et al. 1985). Preliminary distillation was accomplished using a Buchi 320 nitrogen distillation unit. The final NH3-N concentration was determined by the titrimetric method, Section 417 D (AWWA et al. 1985).

Total Kjeldhal Nitrogen (TKN)

TKN was measured according to Standard Methods Section 420 A (AWWA et al. 1985). Sample digestion was accomplished using a Buchi 425 Digester.

Nitrate-Nitrogen (NO3-N)

Nitrate analyses were performed using the Hach high-range nitrate (0-30 mg/L as NO3-N) procedure (Nitra Ver 5 Nitrate Reagent powder pillows for 25-ml sample). The samples were filtered through 0.45 μm glass fiber filter prior to the analysis. A Bausch and Lomb Spectronic 21 spectrophotometer was used for absorbance measurements.

Temperature

For Phase I, water temperature was measured in stage one of the RBC and the air temperature was obtained from the New Mexico State University (NMSU) weather station. For phases II and III water and ambient high and low temperatures inside and outside the protective shed were measured in the field using a high and low temperature mercury thermometer.

RESULTS

WASTEWATER CHARACTERISTICS

The characteristics of the wastewater discharged from the septic tank are shown in Table 2. These results show that the total COD concentration of the wastewater in the study's first two years was approximately half of that reported as average for a typical septic tank. The TSS concentration for the wastewater at the test site in the first year was approximately 50% higher than the reported average. Total and ammonia nitrogen concentrations were similar to the reported average in the first year. However, in the second year, the total nitrogen was 25% lower than the reported average and the ammonia concentration was 50% less than the reported average. Nitrate nitrogen was significantly higher than the reported average throughout the period studied. Comparison with typical primary effluent at a municipal treatment plant indicates that the wastewater used during the first year of study was equivalent to weak domestic wastewater with respect to the COD content, and equivalent to strong domestic wastewater with respect to the total and ammonia nitrogen concentrations. In the second year, the water quality was equivalent to weak domestic wastewater with respect to COD and medium strength with respect to the nitrogen content (Metcalf and Eddy 1979).

Table 2. Septic Tank Water Quality Characteristics

Parameter	First Year Range	First Year Average	Second Year Range	Second Year Average	Typical Average[a]
Total COD	160 - 200	170	130 - 160	145	327
Soluble COD	99 - 130	100	67 - 108	92	---
BOD_5	95 - 105[b]	95	--	--	138
TSS	70 - 90	80	--	--	49
Total-N	27 - 74[c]	35	19 - 41	32	45
NH_3-N	20 - 40[d]	30	4 - 27	16	31
NO_3-N	0 - 17		0 - 24	7	.4

[a]Summary of effluent quality from various septic tank studies (Canter and Knox 1985).
[b]Based on analysis of four samples.
[c]Based on analysis of ten samples.
[d]Based on analysis of thirteen samples.

START-UP

The first three weeks of the study were uneventful, except that an area resident voiced frequent complaints about odors from the treatment system due to hydrogen sulfide and volatile decomposing organics from the septic tank. Within a two-week period the odors were essentially eliminated a bacterial film developed on the discs.

Two weeks after start-up, daily sampling of influent and effluent to the RBC was initiated to evaluate operating conditions. The RBC quickly reached a stable, efficient level of performance. After four weeks, the analytical results showed that the effluent-soluble COD concentration ranged between 10-20 mg/L. Consistent results at this level for thirteen consecutive days indicated of relatively steady-state operating conditions.

EFFECT OF HYDRAULIC LOADING RATE

To evaluate the effect of the hydraulic loading variation on the RBC's performance four different loadings were analyzed by determining the soluble and total COD removal efficiency at each loading. The disc rotational speed was set at 6 rpm throughout this first study. A summary of the results obtained during this test is presented in Table 3 and Figure 3.

At an initial hydraulic loading rate of 1.0 gpd/ft^2 (0.95-1.26 lb COD/day/1,000 ft^2), the average total and soluble removal efficiencies were 88% and 91%, respectively. At a higher loading rate of 2.0 gpd/ft^2 (2.58-3.62 lb COD/day/1000 ft^2), the RBC achieved total and soluble COD removal efficiencies of 70% and 81%, respectively. The hydraulic loading rate was increased to 4.0 gpd/ft^2, however, sufficient wastewater (820 gpd) was not available to maintain the hydraulic loading rate. Consequently, the hydraulic loading rate was reduced to 0.5 gpd/ft^2 (0.81-0.90 lb COD/day/1,000 ft^2) which yielded average total and soluble COD removal efficiencies of 65% and 84%, respectively.

TSS concentrations varied considerably throughout the operational changes. The results obtained from monitoring suspended solids are summarized in Table 3 and Figure 4. During the hydraulic loading rate of 1.0 gpd/ft^2, the influent TSS concentration ranged from

Table 3. Influence of Hydraulic Loading Rate and Disc Rotational Speed on COD and TSS Reduction

Hydraulic Loading Rate (gpd/ft^2)	Disc Rotation Speed (rpm)	RBC Detention Time (hr)	Clarifier Detention Time (hr)	COD Removal Efficiency (%)		TSS Removal Efficiency (%)	Temperature, °F		
							Waste-water	Ambient[b]	
				Total	Soluble			High	Low
0.5	6	4.3	3.5	62-67 (65)[a]	80-87 (84)	46-71 (59)	72-77 (75)	49-72 (60)	26-39 (32)
1.0	6	2.2	1.8	86-90 (88)	90-93 (91)	43-83 (67)	72-77 (75)	81-90 (84)	46-58 (50)
2.0	6	1.1	0.9	70-83 (80)	78-84 (81)	10-74 (58)	72-77 (75)	68-76 (72)	30-40 (34)
1.0	9	2.2	1.8	65-82 (75)	81-89 (86)	38-76 (58)	48-59 (50)	47-68 (56)	21-35 (29)
1.0	12	2.2	1.8	77-80 (78)	84-87 (87)	55-58 (56)	48-59 (50)	61 (61)	22-42 (30)

a. Brackets( ) indicate average for the period.
b. Temperature recorded at NMSU weather station (10 miles south of test site).

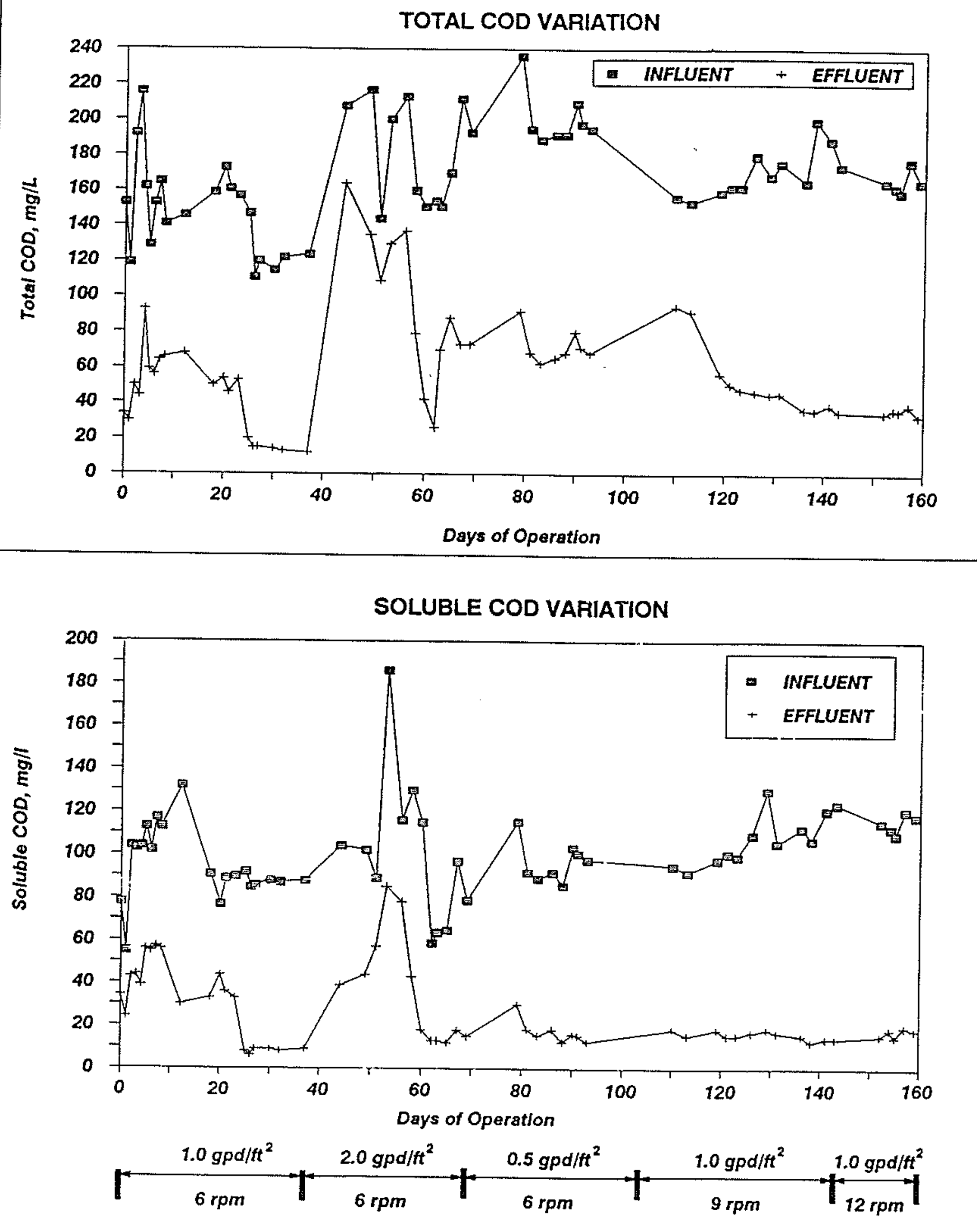

Figure 3 Total and Soluble COD Variation Resulting from RBC Treatment of Septic Tank Effluent.

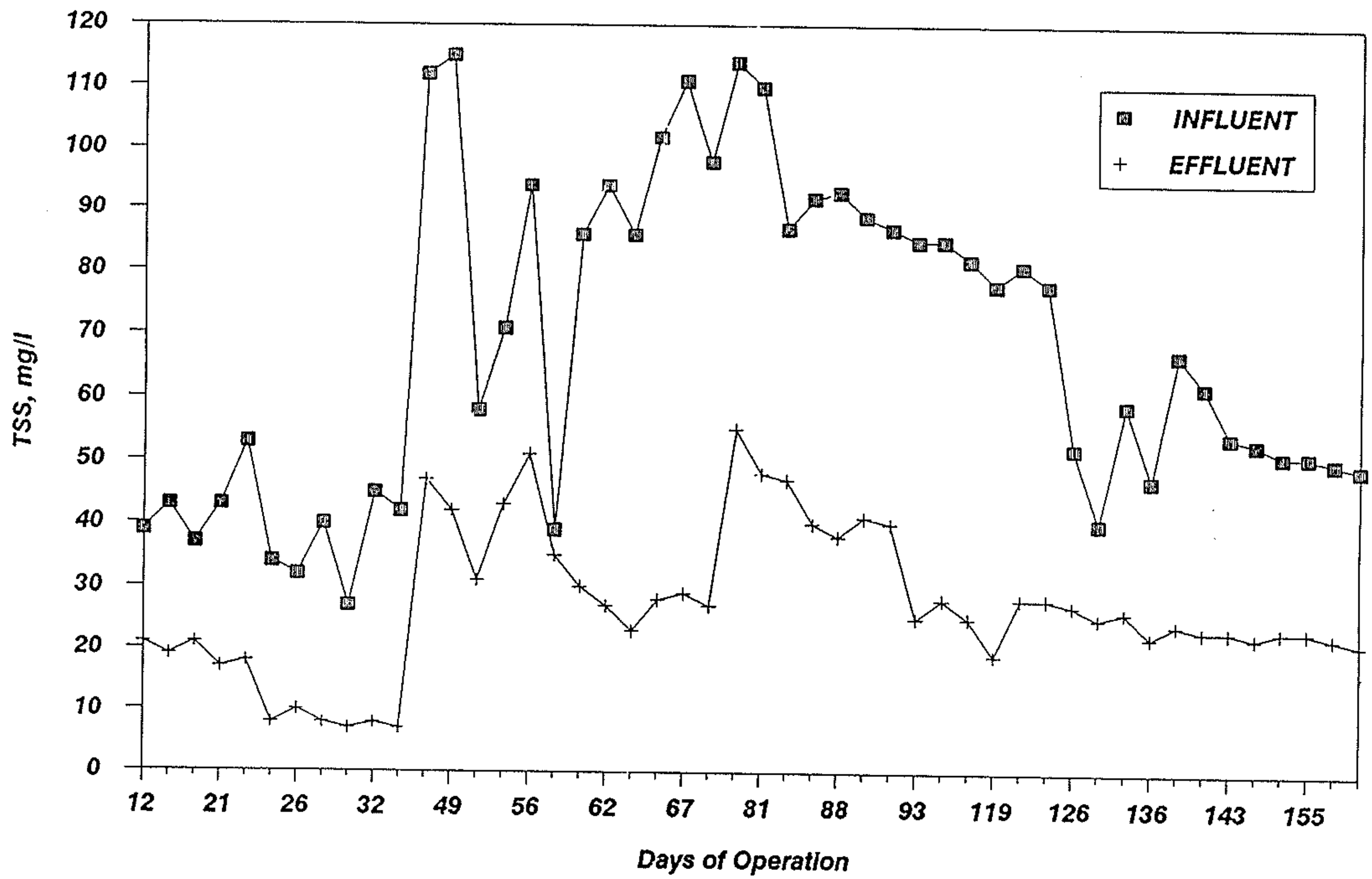

Figure 4. TSS Variation Resulting from RBC Treatment of Septic Tank Effluent.

27-53 mg/L. During this time, the overall average TSS removal efficiency of 67% yielded an average effluent TSS concentration of 13 mg/L (range: 7-21 mg/L). Increasing the hydraulic loading rate to 2.0 gpd/ft^2 decreased the TSS removal efficiency to 58% resulting in an effluent TSS concentration of 34 mg/L. At a hydraulic loading rate of 0.5 gpd/ft^2, the average influent TSS concentration was 92 mg/L. Meanwhile, the average TSS removal efficiency was 59% while the average effluent TSS concentration was 39 mg/L.

Biofilm densities were not measured at any stage, but were noted visually. The dense white biomass was heaviest in the first stage and continually decreased in density in successive stages. The biofilm also became increasingly splotchy brown in the latter stages. The biomass growth was so thick in the first stage that the clearances between discs (0.5 in) were virtually filled. The biomass's overdevelopment and its subsequent bridging over the disc clearance reduced the surface area available for organic substrate to reach the bulk of the biomass (Bates 1978).

Previous studies by Antonie (1976) have shown that four-stage treatment is superior to two-stage treatment. This feature was evaluated in the present study by monitoring the COD reduction efficiency across the four stages during variation of the hydraulic loading rate (Figure 5). Conversion of soluble COD is a basic phenomenon in biologically based wastewater treatment which need not be applied to removal of colloidal and suspended organics. Consequently, this study underlines soluble COD removal to describe the treatment efficiency of the RBC.

At a hydraulic loading rate of 1.0 gpd/ft^2, soluble COD decreased from 88 mg/L in the influent to 9 mg/L in stage four. Soluble COD removal in stage one was 64% and increased to 74% in stage two. The soluble COD removal efficiency increased to 90% in stage four and was essentially completed in stage three (89%). Increasing the hydraulic loading rate to 2.0 gpd/ft^2, decreased the overall soluble COD removal efficiency by 14%. The soluble COD decreased from 80 mg/L in the influent to 19 mg/L in stage four. The pattern of soluble COD removal efficiency was similar to the pattern observed at 1.0 gpd/ft^2. Decreasing the

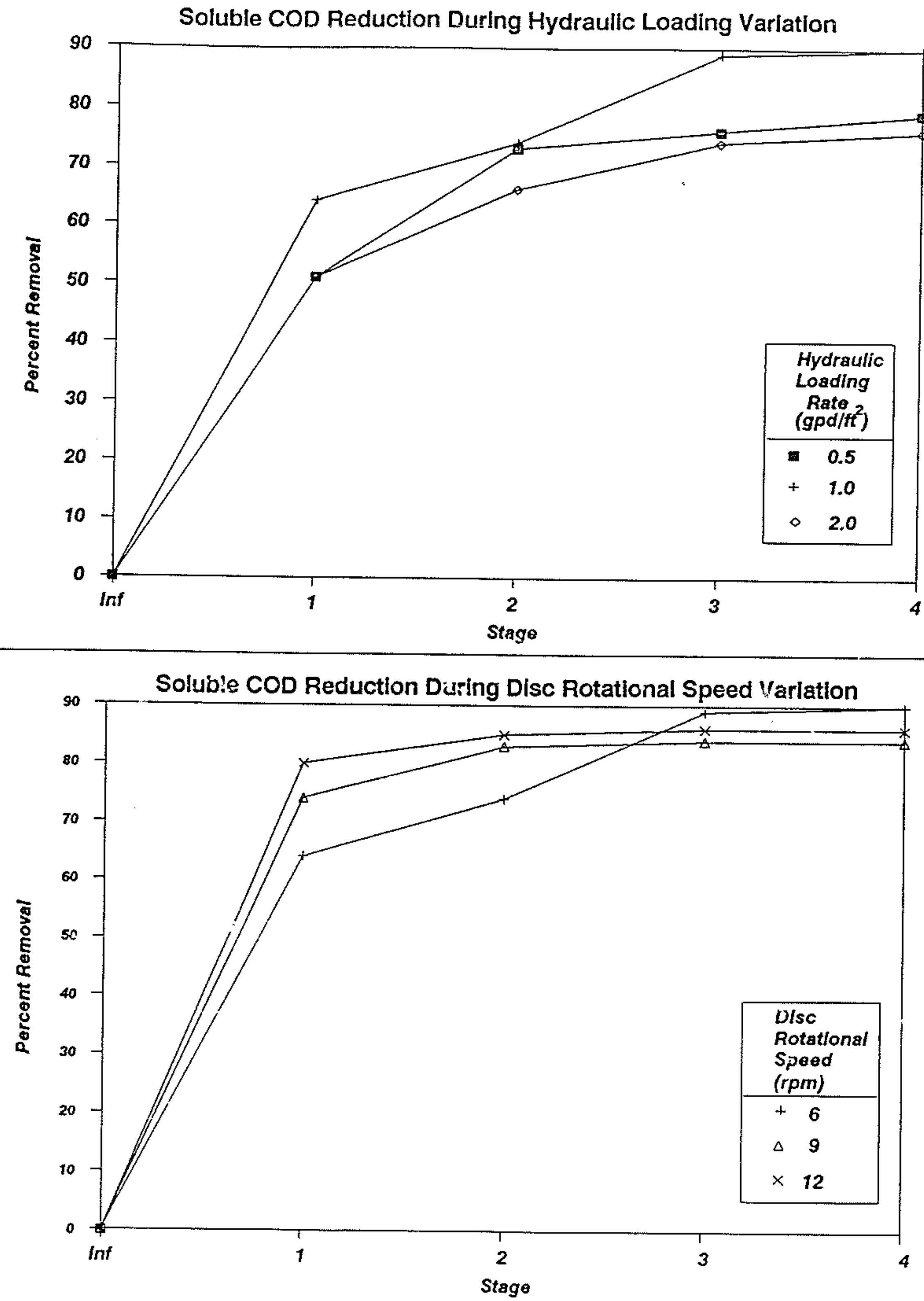

Figure 5 Influence of Hydraulic Loading Rate and Disc Rotational Speed on Total and Soluble COD Reduction Across the RBC

hydraulic loading rate to 0.5 gpd/ft^2 showed a similar pattern of soluble COD removal efficiency. However, it was observed that soluble COD removal efficiency essentially reached a maximum in stage two. Soluble COD removal efficiency was 73% in stage two and only increased to 79% in stage four.

Examination of COD removal efficiencies during the hydraulic loading variation study indicated that the highest total and soluble COD removal efficiencies of 88 and 91%, respectively, were obtained at a hydraulic loading rate of 1.0 gpd/ft^2. Considering the disc surface available in the pilot scale unit (205 ft^2) this loading rate seems more realistic for a typical single family dwelling. Typical rural homes where septic tanks generally are used include a population density of three to four persons producing an average wastewater contribution of 45-75 gpcd (Canter and Knox 1986, EPA 1980). The hydraulic loading rate study also showed that stages one and two effectively reduced total and soluble COD to a near minimum. Through this rational, 1.0 gpd/ft^2 was selected as the hydraulic loading rate for the study's second phase.

INFLUENCE OF DISC ROTATIONAL SPEED

During this study, the effect of varying rotational disc velocity on RBC performance was examined. Disc speeds of 6, 9, and 12 rpm were tested producing peripheral velocities of approximately 30, 45, and 60 ft/min, respectively. The RBC was operated at a hydraulic loading rate of 1.0 gpd/ft^2. The wastewater temperature ranged between 46°-59°F (9°-15°C).

The results are summarized in Table 3 and Figures 3, 4 and 5. At a disc rotational speed of 6 rpm, the TSS removal efficiency was 60%-70%. Increasing the disc rotational speed to 9 and 12 rpm showed similar results. A summary of the soluble COD reduction across the RBC under varying rotational speed is presented in Figure 5. Soluble COD removal in stage one ranged from 74%-80% and increased to 83%-85% in stage two. At the end of stage two the soluble COD concentration ranged from 13-20 mg/L. In other words, soluble COD removal essentially was completed at the end of the first stage, and there was no further significant increase in soluble COD removal efficiency in subsequent stages.

Throughout the disc speed variation study, dissolved oxygen was monitored on a regular basis. A typical dissolved oxygen profile across the RBC by stage is summarized in Figure 6. During the evaluation of a disc rotational speed of 6 rpm, the wastewater temperature range was 72°-77°F (22°-25°C). At disc rotation speeds of 9 and 12 rpm, the wastewater temperature decreased to the range of 48°-59°F (9°-15°C). As shown in Figure 6, due to the lower wastewater temperature and higher oxygen solubility, the influent DO concentration ranged between 1.2-1.4 mg/L. At disc speeds of 9 and 12 rpm, the DO concentration decreased to 3.5 mg/L during the time the final effluent was contained in the secondary clarifier.

EFFECT OF INTERMITTENT POWER SUPPLY OUTAGE

During Phase I, the effect of power loss on RBC treatment efficiency was evaluated. This evaluation was made based on total and soluble COD removal efficiencies. The RBC and pump were turned off for a four-day period. Upon reactivating power, effluent samples were collected and analyzed at 2, 4, 6, and 24 hours. During the first six-hour period the effluent total COD progressively decreased from 165 to 115 mg/L. The total and soluble COD removal efficiencies for the first six-hour period were approximately 30% and 40%, respectively. At the end of the two-hour period, the total and soluble COD removal efficiencies were in the range 70% yielding effluent concentration of 50 and 25 mg/L, respectively.

SLUDGE PRODUCTION

The solids collected in the secondary clarifier were quantified on a regular basis to determine the rate at which the RBC produces microbial mass. The volume and mass of sludge produced and the corresponding yield coefficient were determined. At a hydraulic loading rate of 1.0 gpd/ft^2 and a 6 rpm disc rotational speed, the yield coefficient ranged from 0.1 to 0.12 mg of solids per mg of COD removed. At the higher speeds of 9 and 12 rpm, the yield coefficient ranged from 0.15 to 0.20.

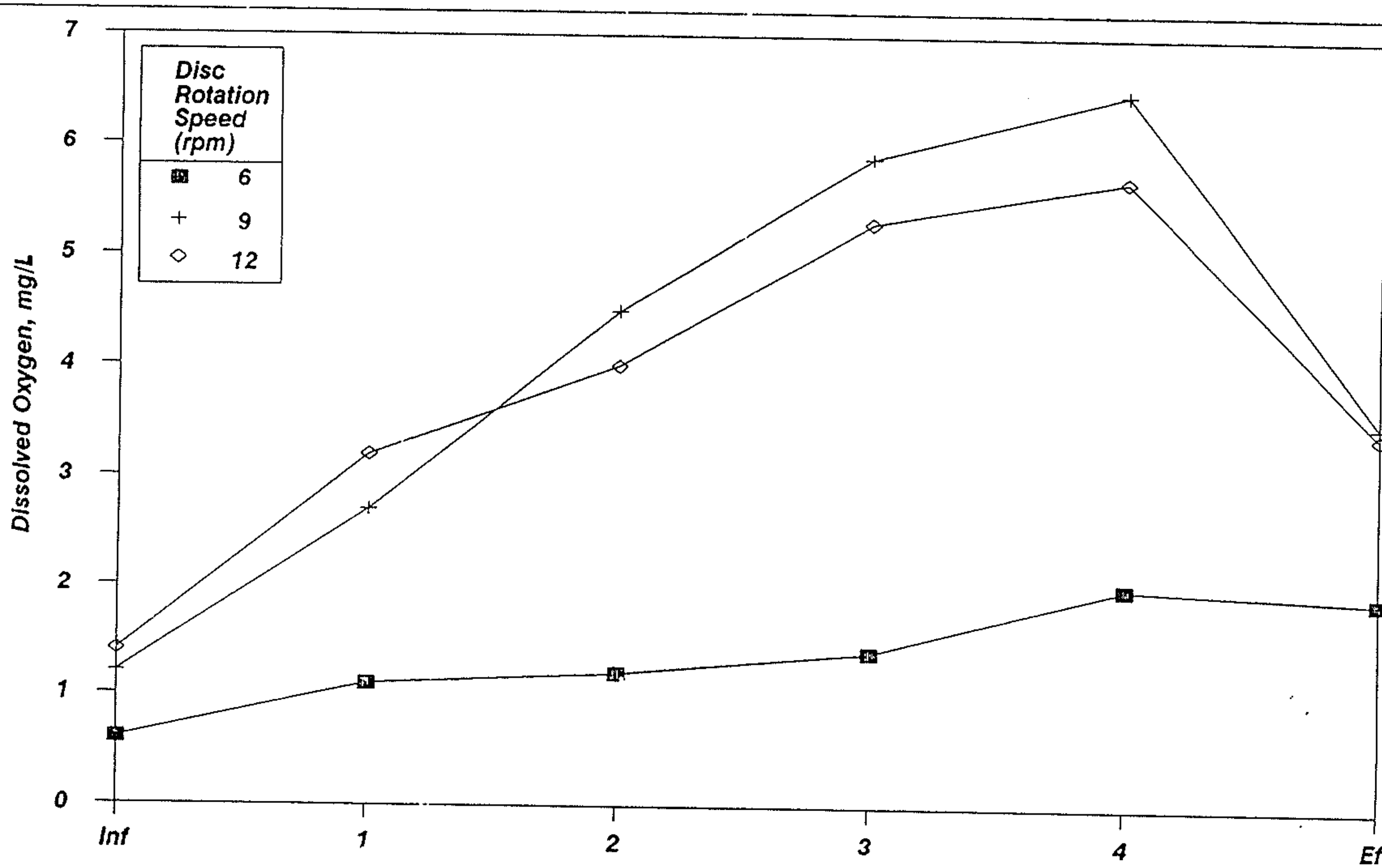

Figure 6: Dissolved Oxygen Profile During Disc Rotational Speed Variation.

During the hydraulic loading study (rotational speed 6 rpm), sludge production was relatively uniform. Four liters of secondary sludge were removed from the clarifier over a 16-day period (1.8 liters/week). At a disc rotational speed of 9 rpm, approximately 7.0 liters of sludge were produced on a weekly basis. During the 12 rpm study, approximately 6.5-7.0 liters of sludge were removed during a five-day time period (9.1-9.8 liters/week).

TEMPERATURE EFFECT

Wastewater temperature is a potential influence to the operational efficiency of the RBC system (Grady 1980). Higher temperatures increase the microbial growth rate (Gaudy and Gaudy 1988) which ultimately increases the oxygen demand removal efficiency. To assess the effect of temperature variation on the RBC's performance, ambient (high and low) and wastewater temperature, and corresponding COD removal efficiencies were monitored. This analysis is based on soluble COD removal efficiencies because this parameter best represents microbial degradation of organics. Wastewater temperatures below 55°F (13°C) did not result in a decrease in COD removal. During the time period of early September to mid November, the wastewater temperature range was 72°-77°F (22°-25°C). During this time (1 gpd/ft^2 and 6 rpm) the RBC achieved a COD removal efficiency of 88-91%. As the study progressed from late November to late January, the wastewater temperature decreased to about 48°-59°F (9°-15°C). The corresponding COD removal efficiency was 85%. These results indicated that as the temperature decreased, with an increase in the disc rotational speed and increased oxygen concentration, the overall COD removal efficiency in the RBC did not decrease significantly.

NITROGEN DYNAMICS

The performance period monitoring nitrogen removal included two modifications made to the RBC to maximize biological nitrification/denitrification processes. The total data base covered winter and summer months. Figure 7 shows the temperature variation of the wastewater and the air inside the metal shed during the experimental periods. Wastewater temperature ranged between 5° to 29°C while the ambient temperature inside the shed ranged

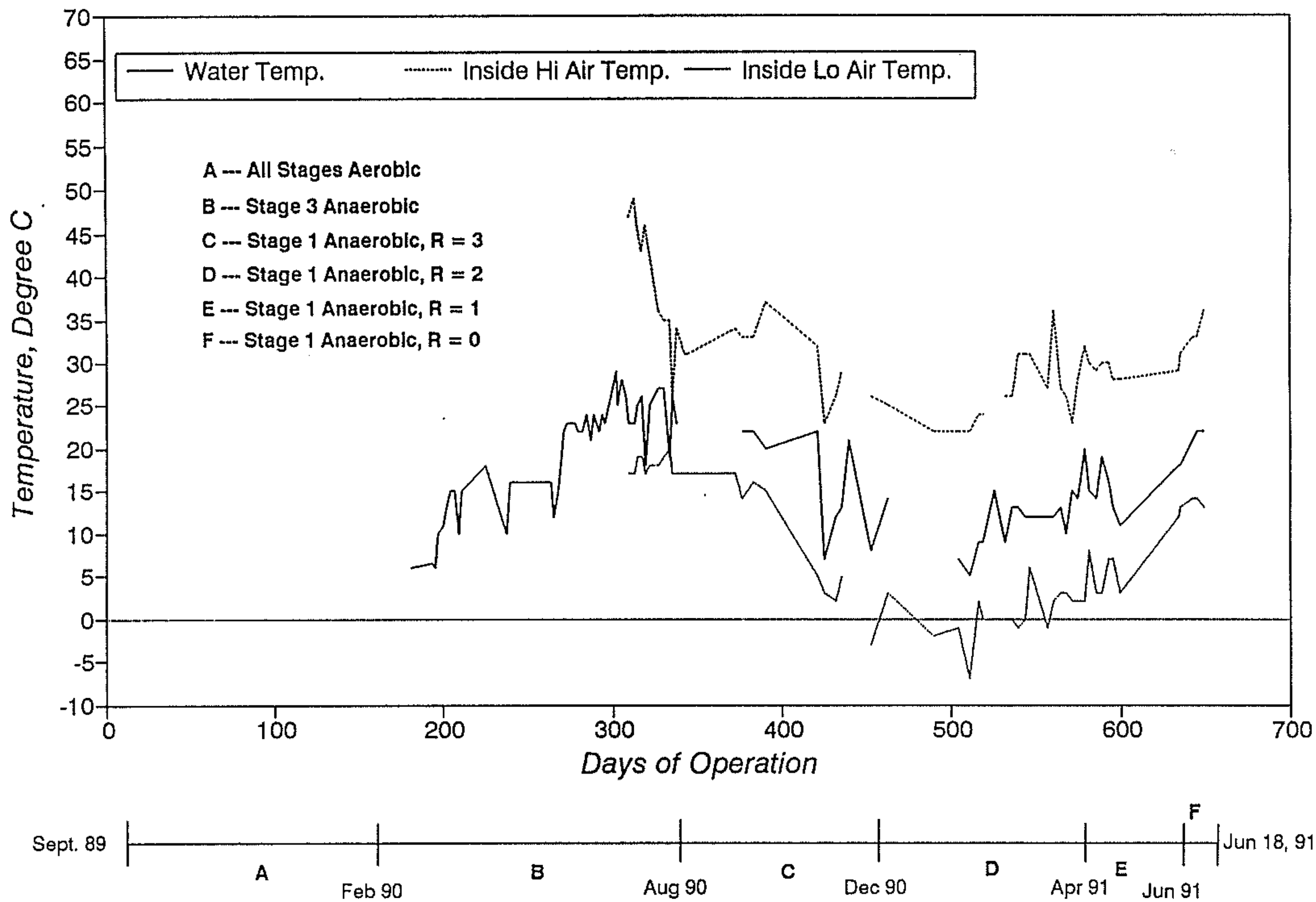

Figure 7. Temperature During the Experimental Period

from -7° to 49°C. Air temperatures inside the shed varied ±1° to 16°C compared to outside ambient temperature.

Table 4 summarizes water temperature, pH, and DO for the research phases. The lowest average water temperature occurred during Phase I (10°C) which delayed the development and establishment of the nitrifying and denitrifying bacteria. The average influent pH was 8.0 and the trend of the pH was to increase slightly across the RBC stages.

Figures 8, 9, and 10 show the respective variation of influent and effluent NH_3-N, NO_3-N, and Org-N concentrations during the study periods. During the second year of operation, septic tank effluent ammonia dropped by 53% following pumping the septic tank sludge (Figure 8). The average influent ammonia concentration to the RBC during the first year was 36 mg/L, ranging from 20 to 48 mg/L, while during the second year the average was 17 mg/L with a range of 4 to 27 mg/L.

Figure 9 shows that nitrate was always present in the influent at an average level of 6 mg/L ranging from 0.3 to 17 mg/L. Depending on the influent nitrate levels and the operational configuration, either a net increase or net decrease of nitrate occurred in parallel with oxidation of carbonaceous material oxidation.

Figure 11 shows variation of the total nitrogen entering and leaving the RBC system during the experimental periods. This data clearly indicate that the RBC system was able to achieve nitrogen reduction during all phases.

Table 5 shows the mass balance for nitrogen across the RBC during the study periods. The overall nitrogen decrease ranged from 29% to 82%. Nitrogen removal efficiency was determined based on the mixed influent concentration. Corresponding total N decreases based on influent concentration are 55%, 56%, 72%, and 82% for recycle ratios 3:1, 2:1, 1:1, 0, respectively. Nitrogen removal can be achieved through four possible mechanisms: 1) ammonia volatilization, 2) biomass synthesis, 3) organic nitrogen adsorption, and 4) denitrification. The respective contribution by each mechanism is summarized in Table 6. Table 6 shows the fraction of nitrogen lost by each mechanism during the experimental periods. The volatilized NH_3-N

Table 4

Wastewater Temperature, Dissolved Oxygen, and pH Data
During the Experimental Periods

Experimental Period	Wastewater Temperature (°C)		Influent	Stage 1	Stage 2	Stage 3	Stage 4	Effluent
	Average	Range						
Phase I	10	9-15						
DO (mg/L)			1.2	2.7	4.5	5.9	6.5	3.5
Phase II								
No Nitrification	11	6-15						
Low Nitrification	15	10-18						
High Nirtification	23	12-29						
DO (mg/L)			1.2	1.2	2.8	1.2	4.4	1.4
Phase III								
R=3	17	7-22						
DO (mg/L)			0.1	2.4	2.5	4.2	5.3	5.8
R=2	11	5-15						
DO (mg/L)			0.1	1.8	2.8	4.0	4.4	4.1
pH (average)			7.9	8.1	8.1	8.1	8.2	8.1
pH (range)			7.4-8.2	7.6-8.3	7.7-8.6	7.4-8.5	7.8-8.5	7.6-8.5
R=1	16	11-20						
pH (average)			8.1	8.2	8.3	8.3	8.3	8.3
pH (range)			7.7-8.5	7.9-8.4	8.2-8.5	8.0-8.5	8.2-8.6	8.1-8.7
R=0	22	21-22						

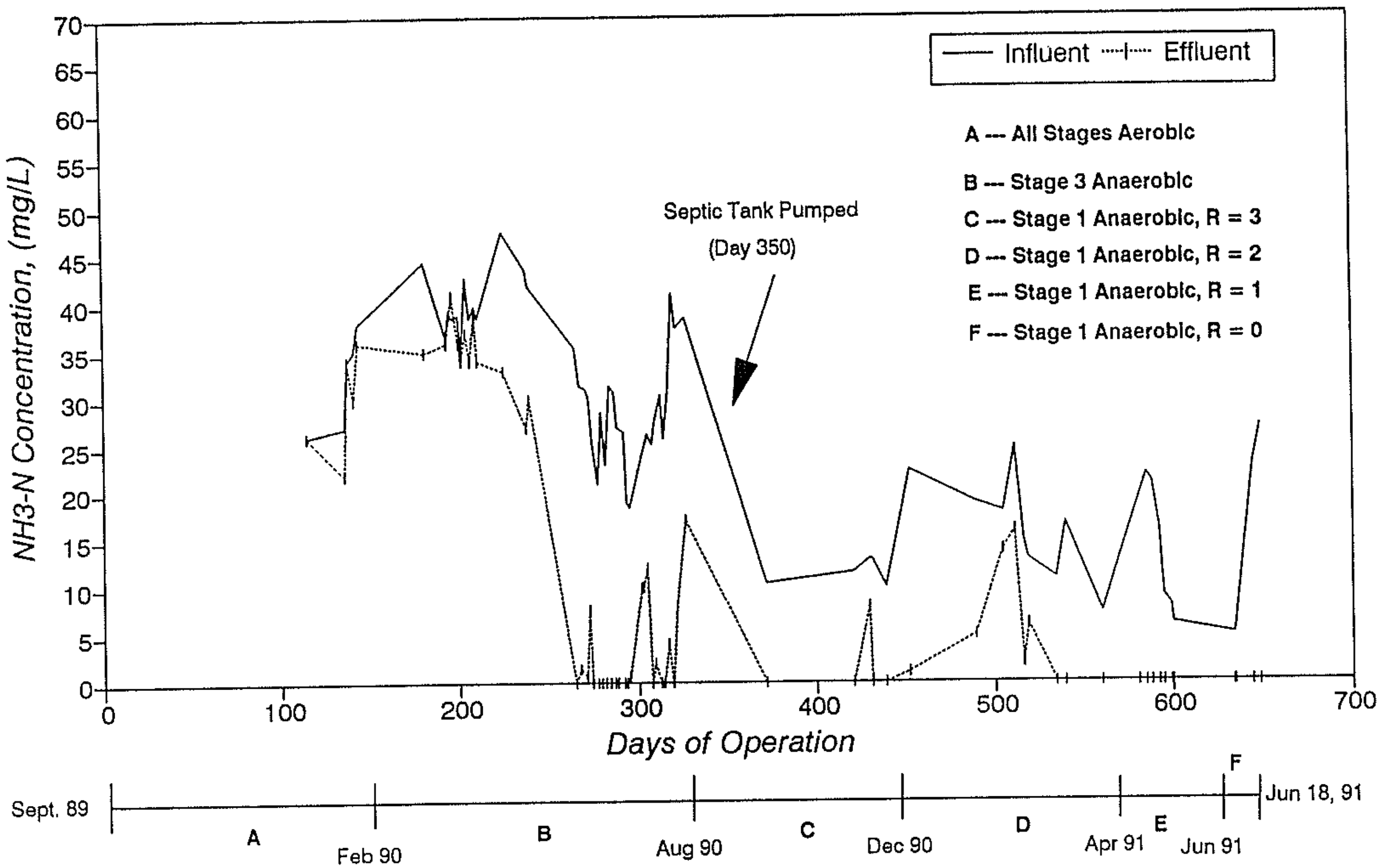

Figure 8. Influent and Effluent Ammonia Nitrogen Variation Across the RBC During the Experimental Period

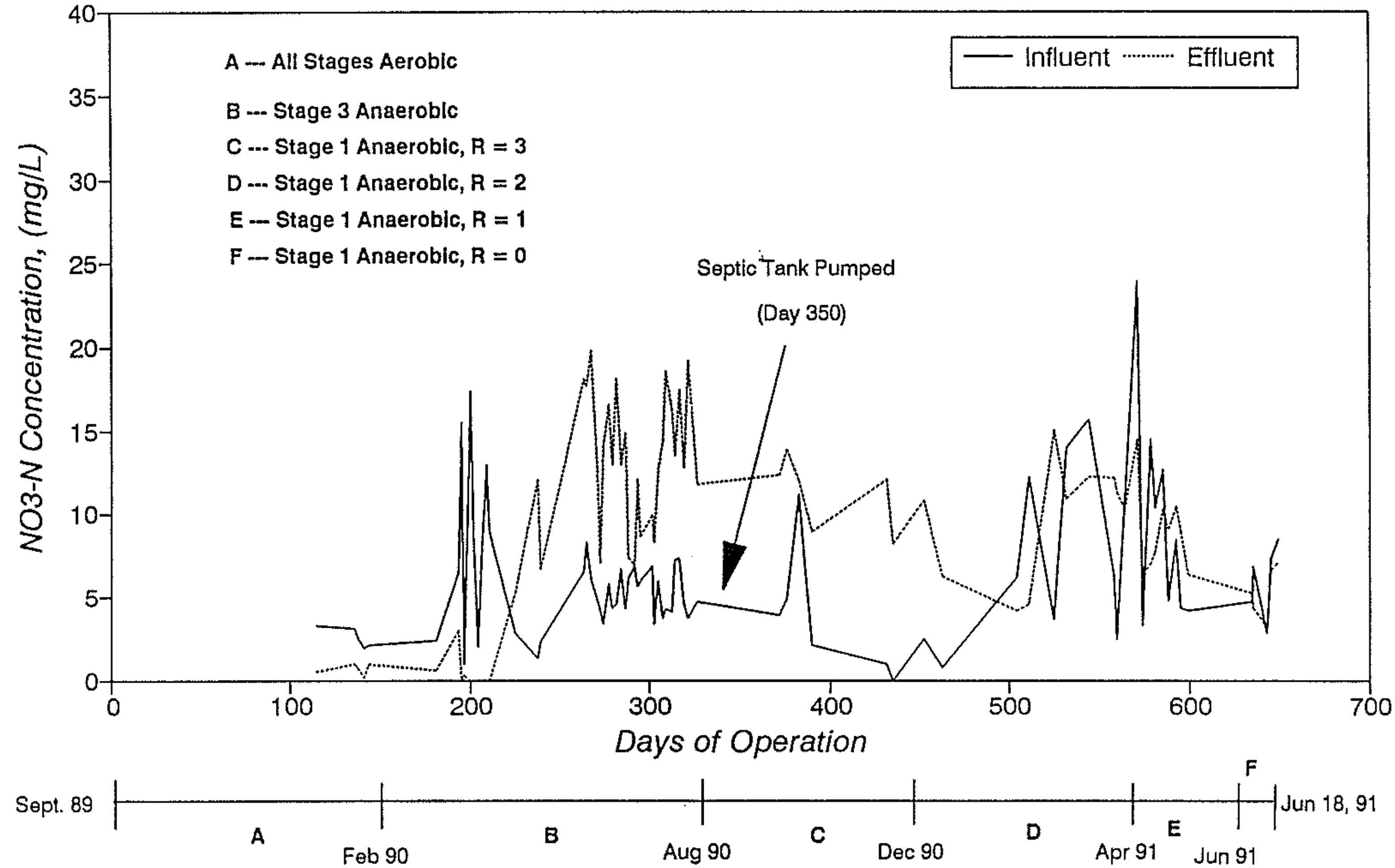

Figure 9. Influent and Effluent Nitrate Nitrogen Variation Across the RBC During the Experimental Period

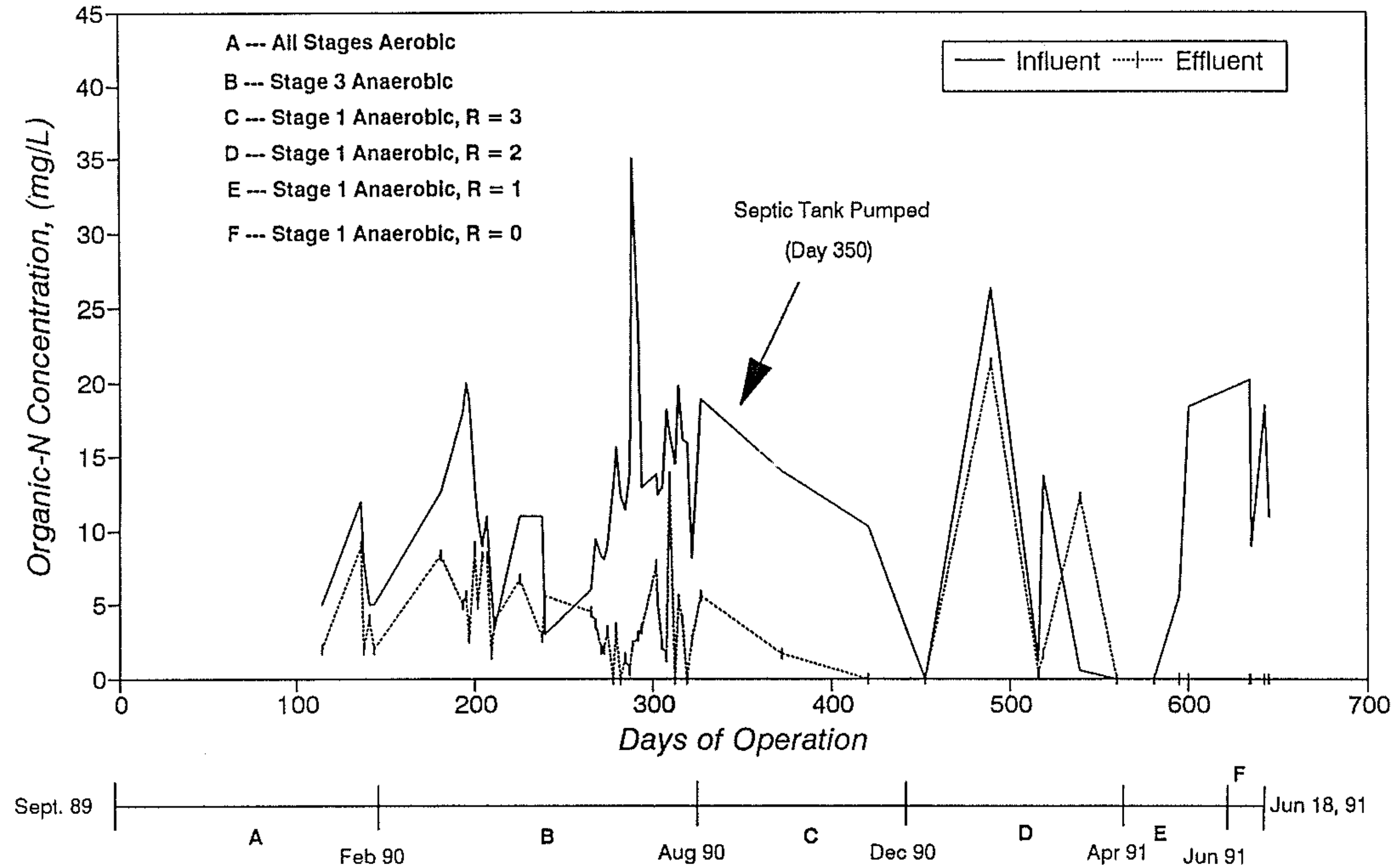

Figure 10. Influent and Effluent Organic Nitrogen Variation Across the RBC During the Experimental Period

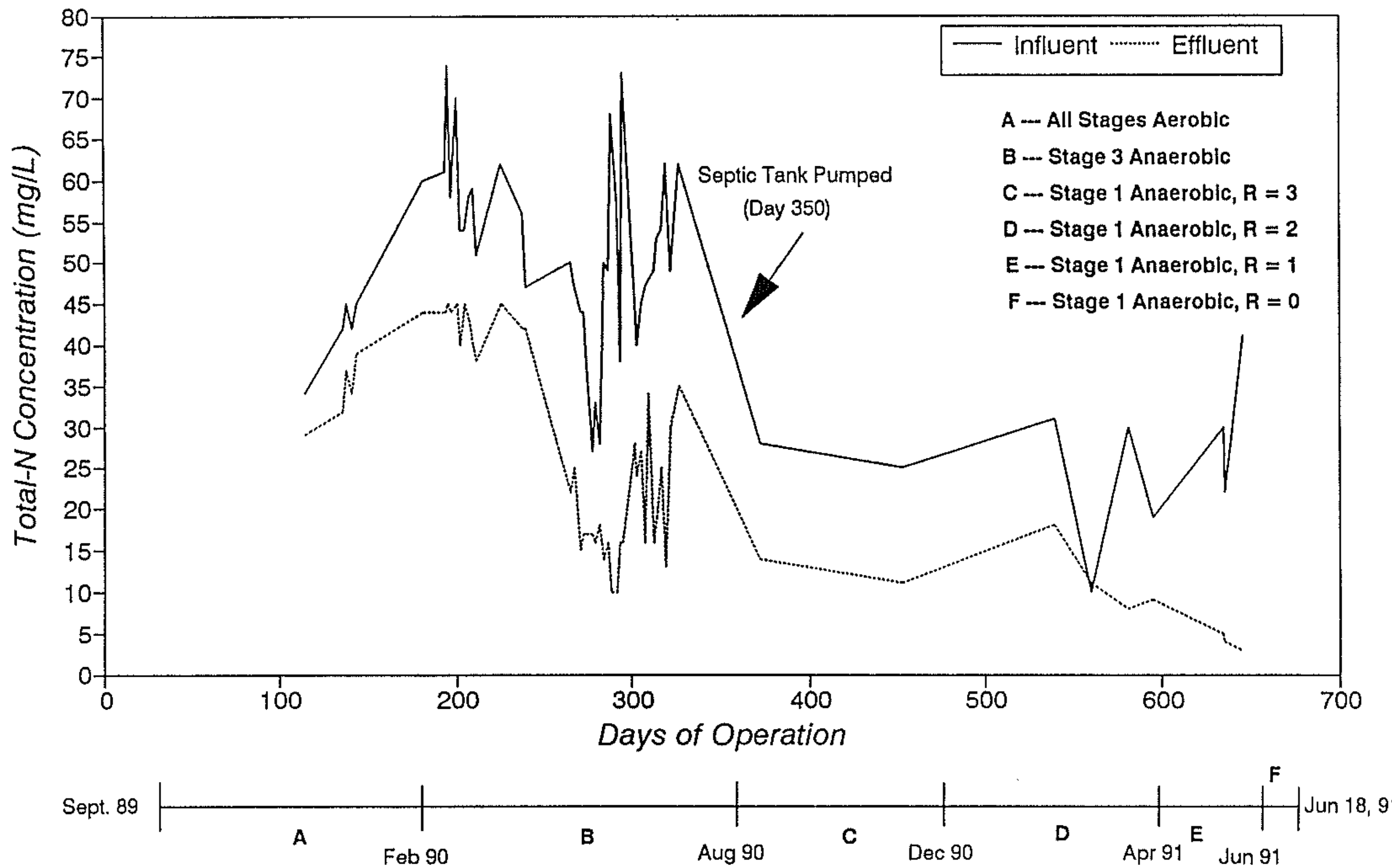

Figure 11. Influent and Effluent Total Nitrogen Variation Across the RBC During the Experimental Period

Table 5

Nitrogen Mass Balance Across the RBC During the Experimental Periods

Condition	Average Water Temp. °C	Raw Influent Total-N mg/L	Mixed Influent[a] Total-N mg/L	Effluent Total-N mg/L	Total-N Decrease mg/L	Total-N Decrease %	Effluent Total-N %	NH_3-N Volatilized %	Sludge -N %	Organic-N Adsorption %	Denitrification N-Loss %
Phase I:											
All Stages Aerobic	10	42	N/A	34	8	19	81	1.4	8	9.5	0.1
Phase II: 3rd Stage Anaerobic,											
No Nitrification	11	59	N/A	42	17	29	71	1.3	4	11.9	11.8
Low Nitrification	15	55	N/A	43	12	22	78	2.2	5	7.3	7.5
High Nitrification	23	50	N/A	20	30	60	40	2.9	5	22	30.1
Phase III: 1st Stage Anaerobic,											
Recycle Ratio= 3:1	17	29	19	13	6[b]	32	68	0.6	4.3	16	11.1
Recycle Ratio= 2:1	11	34	21	15	6[b]	29	71	0.8	2.9	10	15.3
Recycle Ratio= 1:1	16	29	19	8	11[b]	58	42	0.9	6.2	21	29.9
Recycle Ratio= 0	22	33	N/A	6	27	82	18	3.6	7.0	18.2	53.2

[a] Influent concentration calculated based on mixture of raw influent and recycle
[b] Mass balance based on mixed influent concentration

Table 6

Fractional Distribution of Nitrogen Removal During the Periods of Study

Period	Total-N Removed mg/L	NH_3-N Volatilized %	Biomass Nitrogen %	Organic-N Adsorption %	Denitrification N-Loss %
Phase I:					
All Stages Aerobic	8	7	42	50	1
Phase II:					
3rd Stage Anaerobic, No Nitrification	17	5	14	41	40
Low Nitrification	12	10	23	36	31
High Nitrification	30	5	8	37	50
Phase III:					
1st Stage Anaerobic, R=3:1	6*	2	14	51	33
R=2:1	6*	3	10	35	52
R=1:1	11*	2	11	36	51
R=0	27	4	9	22	65

* Total-N removed based on mixed influent concentrations (see Table 4)

was calculated with the conservative assumption that all the free ammonia was stripped out during the rotation of the disks. The amount of the unionized ammonia was computed based on the average pH and the temperature of the wastewater during each period (Piper et al. 1982). Ammonia volatilization was low during the study periods (0.6 to 3.6%) with an average of 1.5% (Table 5). This analysis clearly indicates that the nitrogen loss through ammonia stripping was insignificant. The percentages sludge for nitrogen represent the nitrogen lost due to cellular incorporation during biomass synthesis. The nitrogen fraction in typical biomass is about 14% by dry weight (Gaudy and Gaudy 1988). Nitrogen uptake was computed based on the amount of nitrogen assimilated during oxidation of carbonaceous material. It is generally established that nitrogen is assimilated at a BOD:N ratio of 20:1 (Hitdlebaugh and Miller 1981). For this study the BOD:COD correlation was assumed to be that of typical septic tank effluent, 0.42 (Canter and Knox 1986). The COD removed for biomass synthesis was calculated as the difference between the influent total COD and the effluent soluble COD. The biomass nitrogen ranged from 4% to 8% and averaged 5% (Table 4). The nitrogen lost due to organic-N reduction (adsorption of the suspended solids) ranged from 7.3% to 22% and averaged 14.5% (Table 5).

The most important mechanism in total nitrogen removal is denitrification. The mass balance across the RBC showed that denitrification occurred throughout the study. Since nitrogen loss by the first three mechanisms was quantifiable, denitrification could be determined by the difference between the total nitrogen entering and leaving the RBC system and the sum of the contribution by the three mechanisms. Denitrification ranged from 0.1% to 53.2% (Table 5).

Table 7 summarizes the RBC performance data including nitrogen and COD analysis during the experimental periods. The following sections present the results of the individual study periods.

Phase I

Phase I covered a period from July 1989 through February 1990 during which time all RBC stages were maintained aerobically. This phase was considered a preliminary study for the nitrogen removal evaluation. Optimum operation conditions were found to be 1.0 gpd/ft^2 for the

Table 7

Summary of the RBC Performance Data During the Experimental Periods

Experimental Period	Average Water Temp. (^{o} C)	NO_3-N Average (mg/L)	NO_3-N Range (mg/L)	NO_3-N Increase (Net) (%)	NH_3-N Average (mg/L)	NH_3-N Range (mg/L)	NH_3-N Reduction (%)
Phase I	10						
Influent		2.6	1.9-3.3	-80	32	26-38	6
Effluent		0.6	0.1-0.9		30	22-36	
Phase II							
No Nitrification	11						
Influent		8	0.9-17	-100	39	34-45	5
Effluent		0	0.0-3		37	34-41	
Low Nitrification	15						
Influent		2	1.3-3	67	44	42-48	32
Effluent		6	5.3-12		32	27-33	
High Nitrification	23						
Influent		5.5	3-8	59	30	19-41	90
Effluent		13.5	7-20		3	0-17	
Phase III							
R=3	17						
Influent*		5	0-11	58	12	4-13	100
Effluent		12	8-14		0	0	
R=2	11						
Influent*		8.3	0.8-16	17	14	7-22	100
Effluent		10	4-15		0	0	
R=1	16						
Influent*		9	3-24	11	12	6-22	100
Effluent		8	4-15		0	0	
R=0	22						
Influent		6	3-9	0	23	18-27	100
Effluent		6	3-7		0	0	

* Influent concentrations before mixing with recycle

Table 7

(continued)

Experimental Period	Organic-N			Total-N		
	Average (mg/L)	Range (mg/L)	Reduction (%)	Average (mg/L)	Range (mg/L)	Reduction (%)
Phase I						
Influent	7	5-12	57	42	33-53	19
Effluent	3	2-9		34	24-46	
Phase II						
No Nitrification						
Influent	12	3-20	58	59	38-82	29
Effluent	5	2-9		42	36-53	
Low Nitrification						
Influent	9	3-11	38	55	46-62	22
Effluent	5	3-7		43	35-52	
High Nitrification						
Influent	14	6-49	79	50	28-98	60
Effluent	3	0-14		20	7-51	
Phase III						
R=3						
Influent*	12	10-14	93	29	14-38	55
Effluent	0.8	0-1.7		13	8-16	
R=2						
Influent*	12	0-26	58	34	8-64	56
Effluent	5	4-15		15	8-30	
R=1						
Influent*	8	0-18	100	29	9-64	72
Effluent	0	0		9	4-33	
R=0						
Influent	6	1-11	100	35	22-47	83
Effluent	0	0		6	3-7	

* Influent concentrations before mixing with recycle

Table 7

(continued)

Experimental Period	COD Total (mg/L)	COD Soluble (mg/L)	Total/Soluble Reduction (%)
Phase I			
Influent	175	110	74/86
Effluent	45	15	
Phase II			
No Nitrification			
Influent			
Effluent			
Low Nitrification			
Influent			
Effluent			
High Nitrification			
Influent	170	110	67/54
Effluent	56	51	
Phase III			
R=3			
Influent*	160	67	84/70
Effluent	25	20	
R=2			
Influent*	130	100	65/58
Effluent	46	42	
R=1			
Influent*	145	108	77/69
Effluent	34	33	
R=0			
Influent			
Effluent			

* Influent concentrations before mixing with recycle

hydraulic loading rate and 9 rpm for disk rotational speed. These operational conditions were maintained throughout the study period. Phase I performance data presented here covered a period from December 1989 to January 1990 (29 days) operating under the optimum conditions cited above. The nitrate build-up during this period was negligible and the ammonia reduction was very low (6%, Table 7) indicating the inability of the RBC to achieve significant nitrification. However, based on the material balance around the RBC, the overall nitrogen removal across the RBC was 19% (Table 5).

Phase II

Phase II operations data presented here covered a period from March 1990 through July 1990 (113 days). During this phase, mixing was eliminated in the third stage which reduced the DO from 5.9 to 1.2 mg/L. This modification was intended to induce denitrification in the system. Based on the degree of nitrification that was measured, the 113 days of operation were separated into three periods, designated as the no-nitrification period (33 days), low-nitrification period (24 days), and high-nitrification period (56 days).

During the first period (March-April 1990), the RBC was incapable of achieving nitrification. The ammonia reduction was low (5%) and no net nitrate build-up was measured (Table 7). However, the nitrate that was present in the influent was reduced by 100% across the RBC. This result indicated denitrification was occurring in the system. The nitrogen lost due to the denitrification process was 11.8% (Table 5), and the overall nitrogen reduction was 29%, which is approximately 1.5 times higher than the previous period.

During the second period (April 1990-May 1990), the RBC's capability to achieve nitrification/denitrification improved. Ammonia reduction was 32%, and the net nitrate increase was 67%. However, the overall nitrogen decrease was only 22% (Table 5).

During the third period (May-July 1990), the unit achieved more significant nitrification/denitrification. Ammonia reduction was 90% and the net nitrate build-up was 59% (Table 7). Figure 12 shows the ammonia, nitrate, and TKN profile across the RBC system during this period. Approximately 61% of the ammonia was reduced across stage 1. Nitrate decreased slightly (10%), while the ammonia did not decrease across stage 3. This result was expected because of the near anaerobic condition in stage 3. Nitrate decreased by 10% across the secondary clarifier indicating signs of denitrification due to sludge anaerobisis. The overall nitrogen removal was 60% (Table 5) which is three times higher than the removal in Phase I.

Phase III

Phase III data presented here covered a period from September 1990 through June 1991 (277 days). This phase evaluated the effect of recycling the effluent from the secondary clarifier at four ratios 3:1, 2:1, 1:1, and 0, and establishing low oxygen conditions in the first stage of the RBC. Figures 13, 14, 15, and 16 show the typical profile of NH_3-N, NO_3-N, and TKN across the RBC during the recycle ratios of 3:1, 2:1, 1:1, and 0, respectively. The influent quality represented by these figures is for the raw septic tank effluent (i.e., RBC influent before mixing with recycle). Ammonia reduction reached 100% at all recycle ratios, an indication that the nitrifying bacteria were developed (Table 7). The net nitrate increase declined (Table 7) while the nitrogen lost due to denitrification increased (Table 5) as the recycle ratio decreased. This result indicated development of denitrifying organisms in the system. During operation at zero recycle the overall nitrogen removal was highest at 82% (Table 5).

Dilution effect at the mix tank due to recycle was considered. Dilution occurred during each blending cycle. The cycle's duration cycle was one hour. Raw influent was pumped from the septic tank every hour for only six minutes while effluent was recycled on a continuous basis. Accordingly, the blended influent (raw influent + recycle) would have maximum concentrations at the beginning of the blending cycle (during pumping of the raw influent) and would reach minimum concentration (secondary clarifier effluent quality) at the end of the cycle (Table 8) At the beginning of the blending cycle, nitrate concentrations were either equal to or higher than the raw influent because both the raw influent and recycle contained high nitrate concentrations. Ammonia and COD concentrations were always lower in the blended influent because of the recycle dilution effect. At the end of the blending cycle, the mixed influent was almost the same quality as the secondary clarifier effluent. The contact time for the RBC reactor without recycle

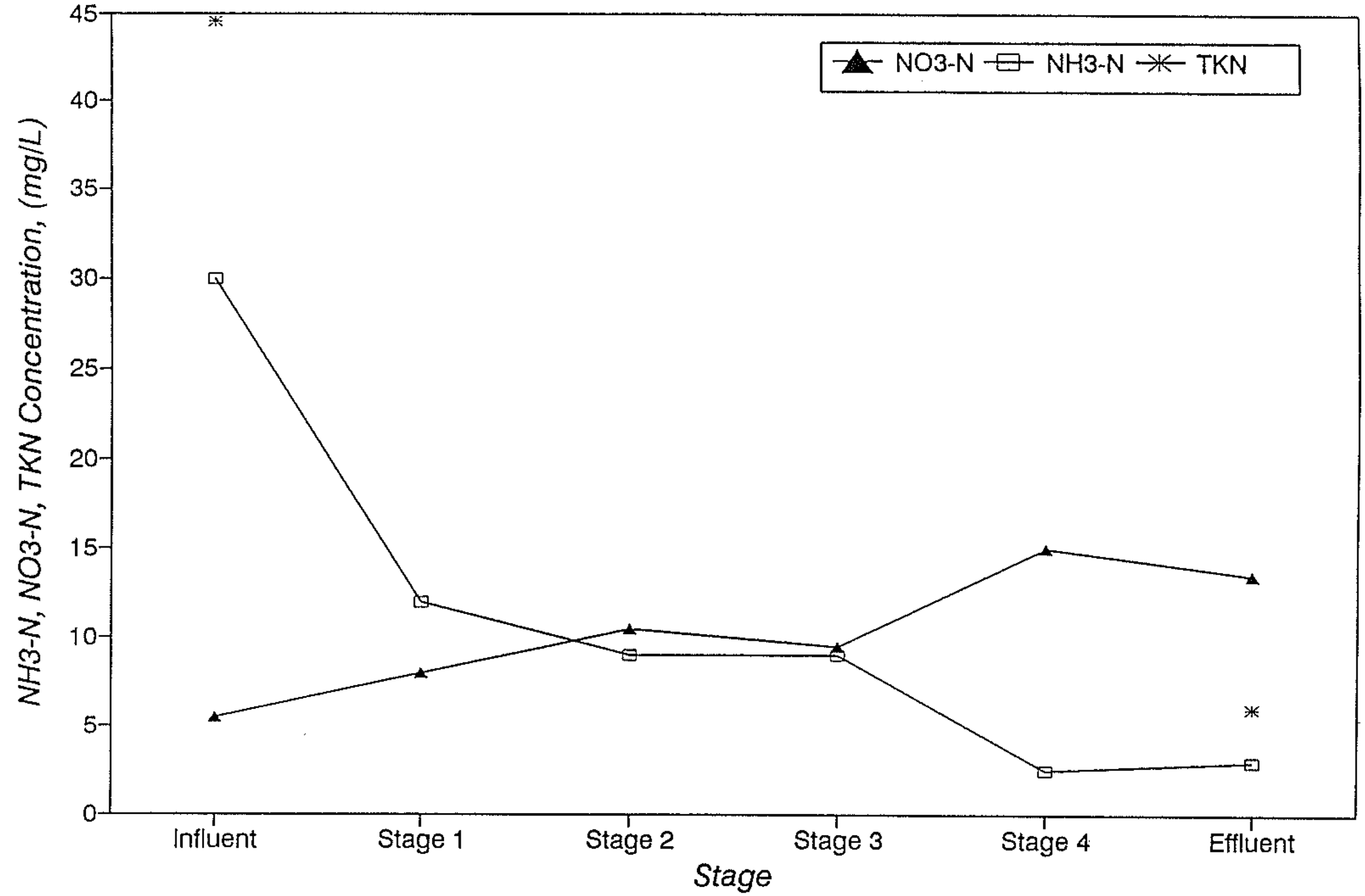

Figure 12. Typical NH3-N, NO3-N, and TKN Profile Across the RBC with 3rd Stage Anaerobic

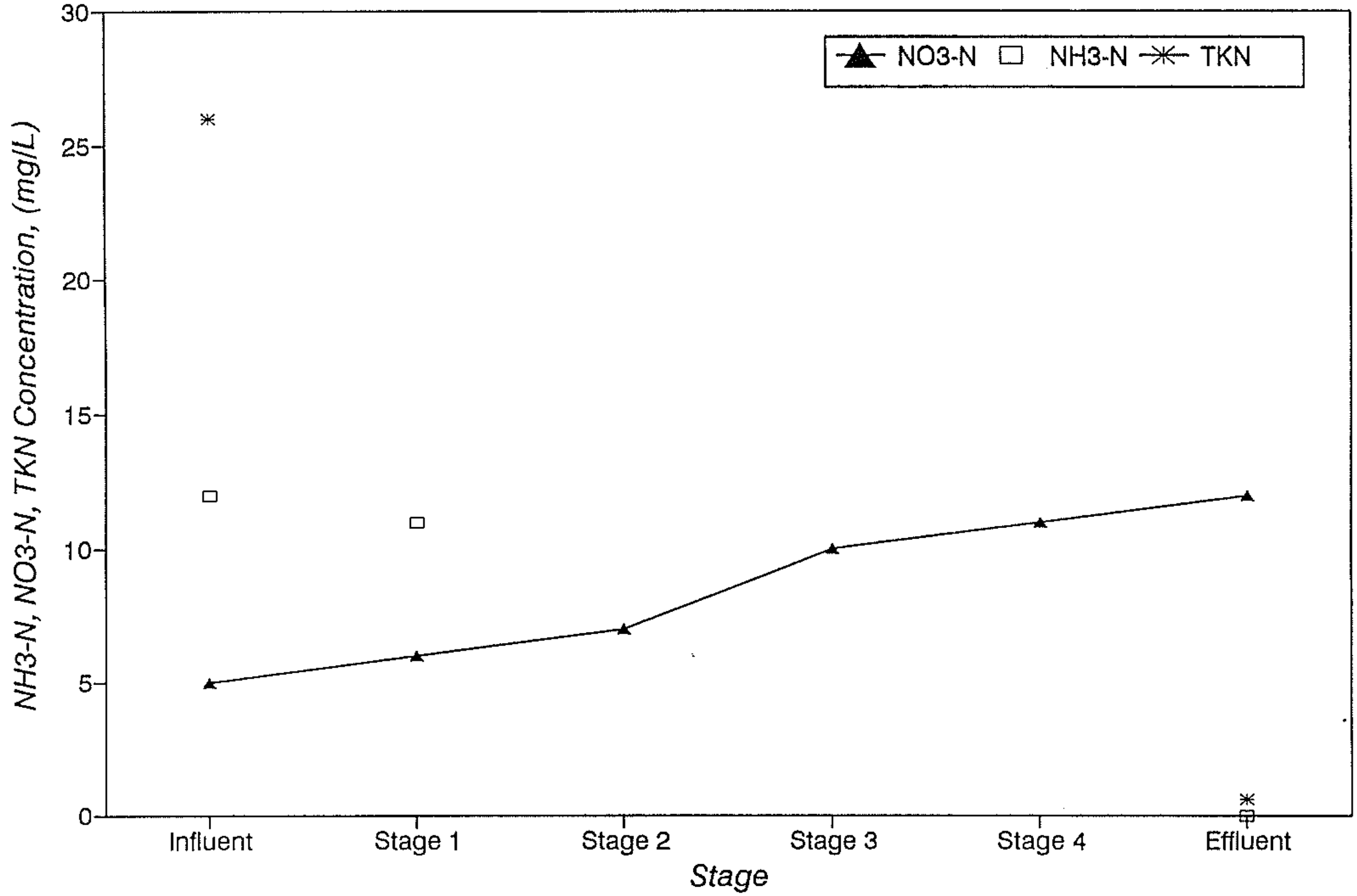

Figure 13. Typical NH3-N, NO3-N, and TKN Profile Across the RBC with 1st Stage Anaerobic and Recycle Ratio = 3:1

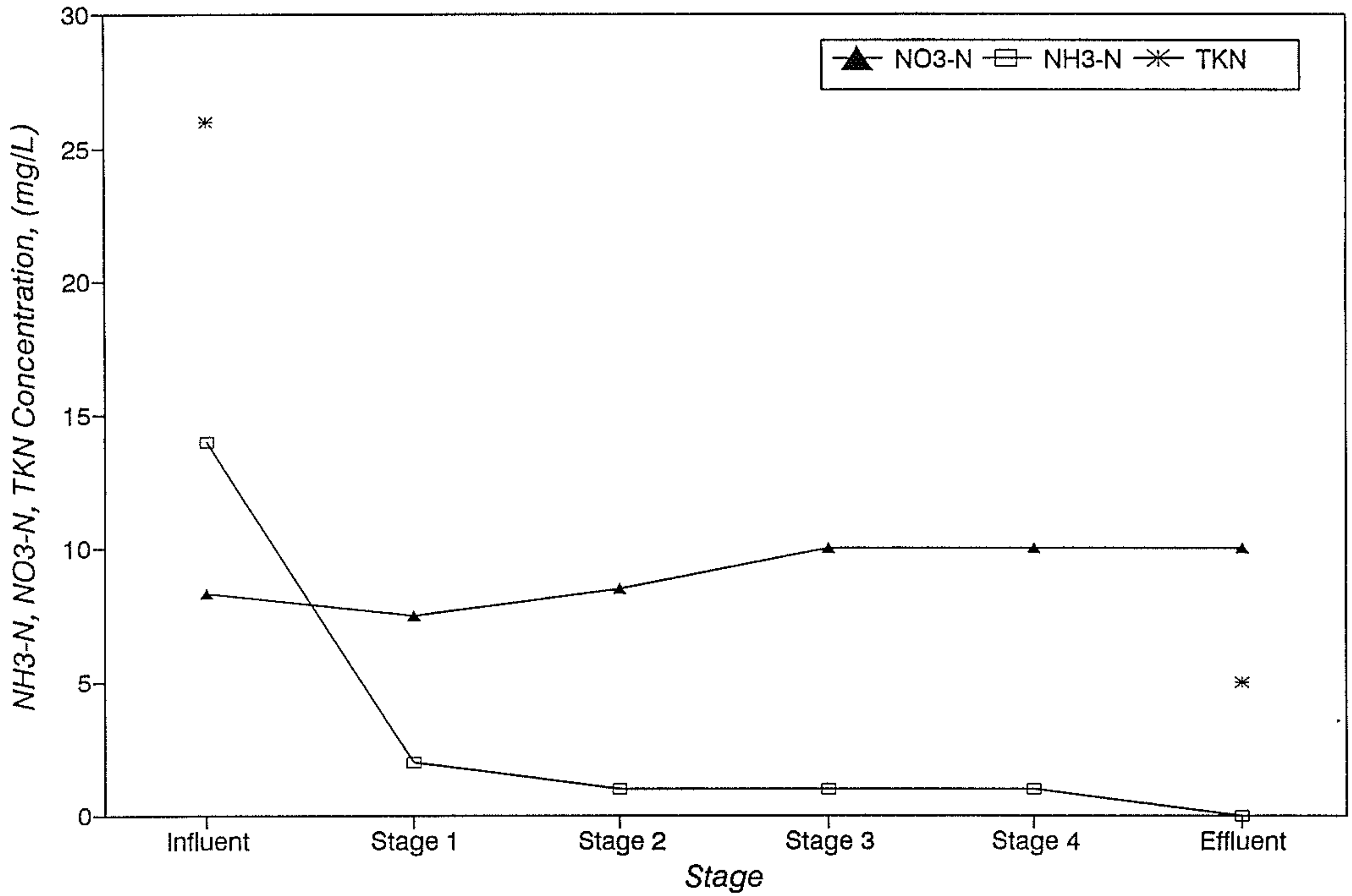

Figure 14. Typical NH3-N, NO3-N, and TKN Profile Across the RBC with 1st Stage Anaerobic and Recycle Ratio = 2:1

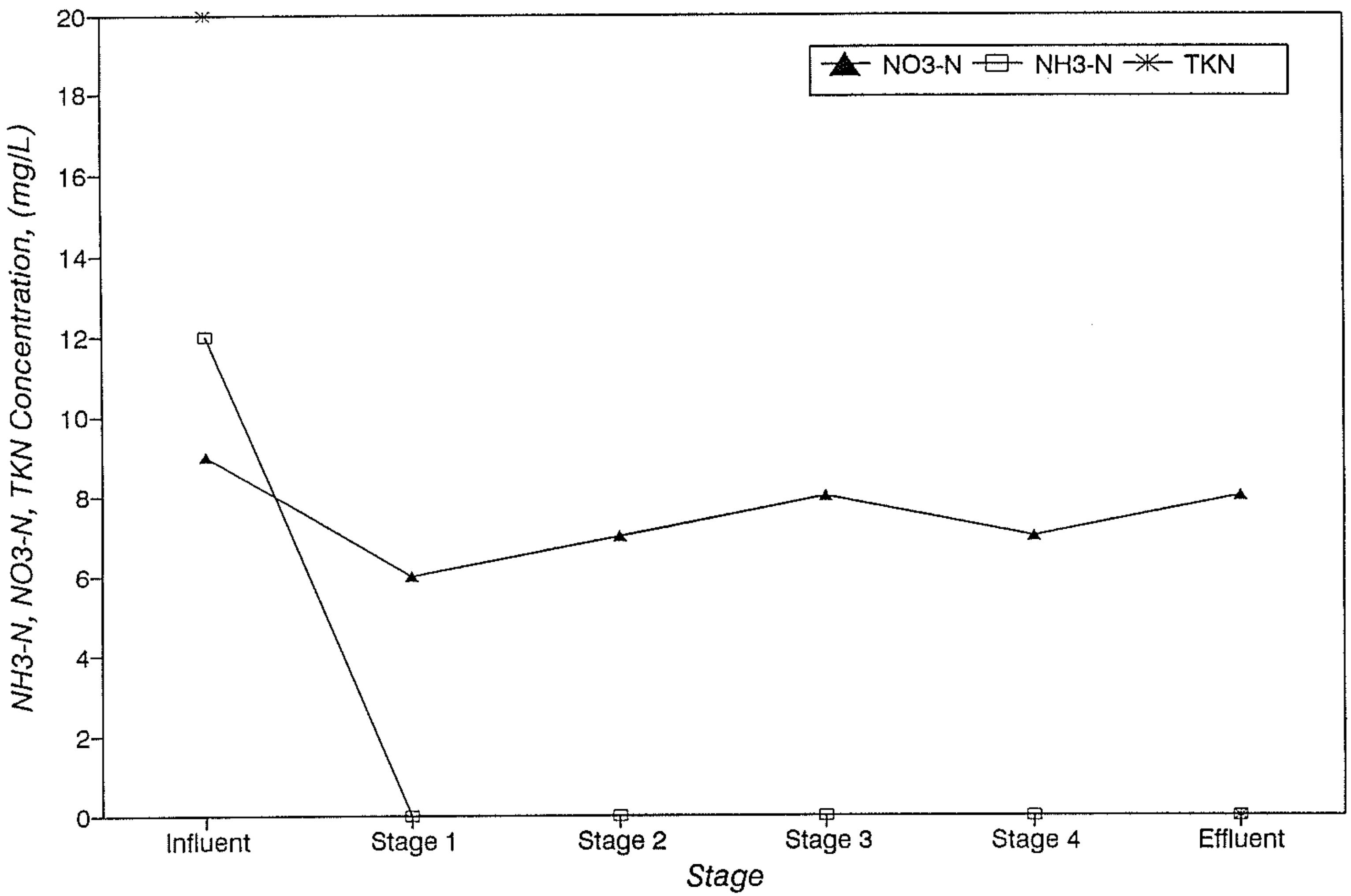

Figure 15. Typical NH3-N, NO3-N, and TKN Profile Across the RBC with 1st Stage Anaerobic and Recycle Ratio = 1:1

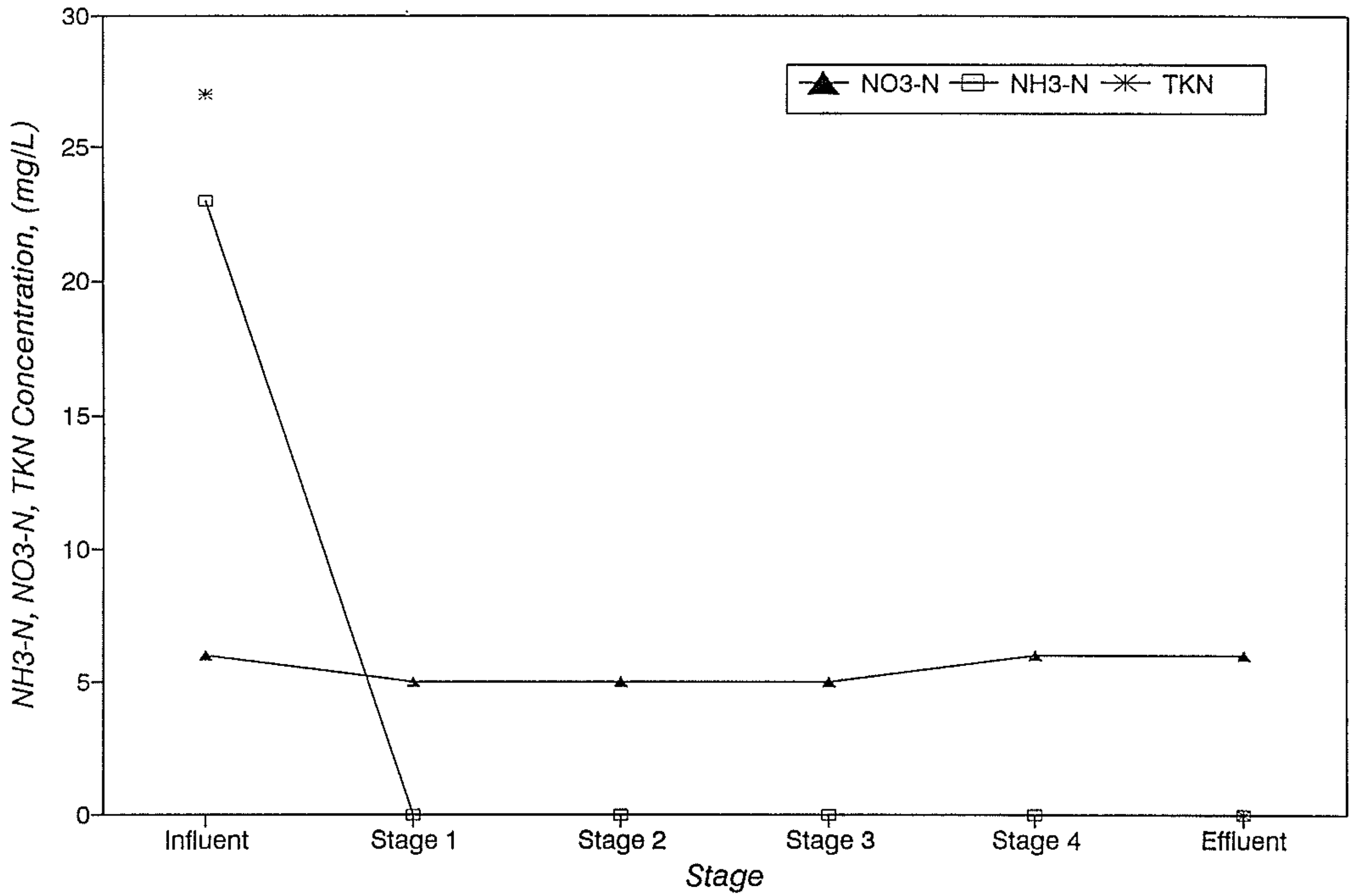

Figure 16. Typical NH3-N, NO3-N, and TKN Profile Across the RBC with 1st Stage Anaerobic and Zero Recycle

Table 8

Dilution Effect in the Mix Tank Due to Recycle

Parameter	Recycle Ratio 2:1			Recycle Ratio 1:1		
	Raw Influent	Mix 1[a]	Mix 2[b]	Raw Influent	Mix 1[a]	Mix 2[b]
COD (mg/L)	130	50	120	145	35	140
NH_3-N (mg/L)	14	0	12	12	0	11
NO_3-N (mg/L)	9	10	8	9	9	9

[a]Mix 1 - Concentration measured in the mix tank at end of blending cycle (Concentrations for recycle ratio 3:1 were not determined)

[b]Mix 2 - Concentration estimated in the mix tank at beginning of blending cycle

was 2.22 hours which changed during the recycling phases. The contact time at the recycle ratios 3:1, 2:1, and 1:1 were 0.56 hour, 0.74 hour, and 1.11 hours, respectively.

OPERATION AND MAINTENANCE

Since the RBC is essentially stable under conditions of hydraulic and organic load variations, and does not require the recycle of sludge or effluent for proper operation, no special operating flexibility or sophisticated instrumentation are required (Antonie 1976). In this study, using simple mechanical components for the process hardware resulted in very low and simple mechanical maintenance. The maintenance was limited to greasing bearings on a monthly basis, and inspecting the V-belts, pulley, and sprockets for wear and slack biweekly. As noted earlier, increasing the rotational speed from 6 to 9 and 12 rpm also increased the amount of sludge produced weekly from 1.8 liters to 7 and 9 liters, respectively. With increased sludge production, the disposal frequency was also increased. At disc speeds of 6, 9, and 12 rpm, the sludge disposal frequencies were 16,7, and 5 days, respectively.

CAPITAL COST OF THE RBC

The capital cost of the RBC system used in this study included the purchase of Plexiglas disc material, the shaft, bearings, fabrication of a 16 gauge metal tank, 3/4 hp motor and pulleys, 1/4 hp pump, PVC valves and piping, and the metal tool shed. The total capital cost of the materials was $900. The average power consumption throughout the study was $19 per month.

DISCUSSION

WASTEWATER CHARACTERISTICS

Nitrates were always present in the septic tank effluent at an average level of 6 mg/L and ranging from 0-24 mg/L. This high nitrate concentration is unusual compared to typical water quality reported for septic tanks (Canter and Knox 1986, Table 2). Although the raw water supply was the suspected source, tests revealed that nitrates were absent. A possible explanation for this unusual phenomena is that nitrification occurred near the water surface of the septic tank where oxygen was available in the range of 0.1-1.2 mg/L DO (Table 4). This level of DO promoted the growth of nitrifying bacteria on the tank surface at the water air interface. The growth of these organisms increased the nitrate content of the septic tank effluent. Similarly, Atwater and Bradshaw (1981) reported that nitrates were always present in the septic tank effluent at levels normally greater than 3.0 mg/L ranging from 3.5 to 20 mg/L. They assumed that the nitrates source was the raw water supply but they did not verify their assumption. During the study's second year, ammonia concentrations dropped by 53% after pumping the accumulated sludge from the septic tank. Cleaning the septic tank reduced the solids which had previously been decomposing to NH_3-N.

EFFECT OF HYDRAULIC LOADING RATE VARIATION

A comparative study conducted under three different hydraulic loading rates showed a significant effect on the total and soluble COD removal efficiency. Increase in the hydraulic loading from 1.0 gpd/ft^2 to 2.0 gpd/ft^2 reduced the RBC detention time by 50% (see Table 3). As a result, the total and soluble COD removal efficiencies in stage one were reduced by 12% and 13%, respectively. Increase in the effluent COD concentration during higher hydraulic loading of 2.0 gpd/ft^2 was due to the corresponding increase in the influent COD concentration (172 mg/L). At the outset, the higher loading rate was expected to be less efficient.

An increase in the available biological reaction time due to decreasing the hydraulic loading rate to 0.5 gpd/ft^2 resulted in 65% and 84% total and soluble COD removal efficiencies, respectively (Table 3). Although improved COD removal at 0.5 gpd/ft^2 was anticipated, the results clearly demonstrated a decline. This decline could be attributed to an increase of suspended solids in the effluent due to heavier sloughing of the biofilm which occurs during substrate-limited growth. This hypothesis is supported by the measurement of a higher effluent TSS concentration (39 mg/L) during the evaluation of the lower loading rate. It was noticed that at a hydraulic loading rate of 0.5 gpd/ft^2, the influent TSS concentration (92 mg/L) was higher than that of loading rates 1.0 and 2.0 gpd/ft^2. It is believed that the increase in suspended solids was due to an increased rate of water returning to the septic tank in the by-pass which, in turn, caused resuspension of settled solids. Increased sloughing of the biofilm also acts to reduce the biomass available for COD reduction.

The overall TSS removal efficiency (67%) during the hydraulic loading rate of 1.0 gpd/ft^2 was low compared to results from other studies. Antonie (1976) achieved TSS removal efficiency of 81% at a hydraulic loading rate of 1.06 gpd/ft^2 for typical domestic wastewater. In general, the effluent TSS concentration was below 30 mg/L which is considered to be a good quality effluent for drain field disposal. Since solids are a major factor responsible for clogging a drain field, an effluent TSS concentration less than 30 mg/L would significantly reduce the chances for drain field failure. This decrease in potential for failure ultimately reduces replacement and maintenance cost of the leach field system. Conclusively, a hydraulic loading rate of 1.0 gpd/ft^2 seems appropriate for onsite treatment of septic tank effluent using a pilot-scale RBC.

The RBC unit used in this study was designed for a 200 gpd flow rate. Under these conditions (hydraulic loading rate of 1.0 gpd/ft^2) the effluent COD and TSS concentrations were generally less than 30 mg/L. Study results also indicate that the unit can handle up to 400 gpd; equivalent to a single family dwelling population of five to six persons. Under these conditions, the RBC has the capability to achieve 85% COD removal and 60% TSS removal efficiency producing an effluent quality of 30-40 mg/L COD and TSS. A hydraulic loading rate less than 1.0 gpd/ft^2 is not recommended because the increase in surface area of the rotating discs as well as the size of entire unit may make installation of the RBC cost prohibitive. Excessive biofilm growth on the discs also reduces the effective surface area available for organic substrate to diffuse into the bulk of the biomass. To reduce this effect, it is suggested that a disc of 3/4" in stage one and 1" in stage two can be provided.

EFFECT OF DISC ROTATIONAL SPEED VARIATION

Potential effects of increased rotational speed and therefore peripheral speed, on the performance of an RBC include the following:

- increasing the rate of oxygen transfer,
- increasing the mass transfer rate of substrate to and through the biofilm, and
- increasing biomass shear (Friedman, et al. 1979).

Based on their operational data, Autotrol Corporation (1978) identified the most effective peripheral speed for an RBC unit as 60 ft/min. Results have shown that during low hydraulic loading (1.0 gpd/ft^2) and low temperature conditions, increasing the disc rotational speed demonstrated an improvement for removing carbonaceous oxygen demand. Through the present study, it was determined that increasing the disc rotational speed to 9 and 12 rpm maximized the removal of organics in the first stage of the treatment process.

By comparing Figures 5 and 6, it is clear that the COD removal was not limited by dissolved oxygen at the higher disc rotational speeds. On the other hand, at a speed of 6 rpm, COD removal in the first stage was limited by dissolved oxygen. Reduced biological activity and increased oxygen solubility at lower temperatures further justified the higher rotational speed.

TEMPERATURE EFFECT

The COD removal efficiencies achieved by the RBC during this study demonstrated that performance was not affected adversely by low wastewater temperature. The temperature data and soluble COD removal efficiencies reported in Table 3 support this conclusion. Soluble COD removal efficiency was not affected adversely because the ambient temperature inside the metal tool shed was very likely higher than the temperature recorded at NMSU (56°-61°F). This warmer air temperature counteracted the colder water temperature (50°F). This situation was possible because the metal tool shed adsorbed and radiated solar heat. The benefits of this heating effect was realized because the attached bacterial film were exposed to the warmer air temperature approximately 60% of the time.

As previously noted, a removal efficiency due to low temperature can be counteracted by increasing the disc rotational speed. Under extreme conditions, expanding the available surface area by increasing the number or diameter of the discs in the RBC may be required to counteract low temperature effectively. Therefore, flexibility is essential in designing an efficient onsite scale RBC unit. For ambient temperatures less than 55°F (13°C), especially in the northern U.S. and Canada, temperature correlation factors, previously described should be used to determine the need for increasing disc area to compensate for the decrease in the biological activity (Forgie 1975). In addition, the heat produced by the biological activity can be conserved by placing an insulated cover directly over the reactor.

EFFECT OF INTERMITTENT POWER SUPPLY OUTAGE

The results of the power interruption study indicated that the RBC is a very stable and durable biological process which can withstand extended periods of no operation. Even after four days of not operating, the biofilm returned to a stable state of metabolism and provided a COD removal efficiency of 70%. Consequently, if the RBC process must be interrupted for repair or service, the downtime could be extended for up to four days without serious consequences. The RBC system was able to produce an effluent COD under 50 mg/L within 24 hours of startup after an extended shut down period. Based on these short-term results, it is anticipated that within a seven-day period or less, the effluent quality from the RBC would return to a stable COD level of less than 35 mg/L.

SLUDGE PRODUCTION

Increasing the disc rotational speed was found to be a major factor which influenced sludge production. The higher speed increased shear forces between the water and biofilm resulting in greater sloughing of biomass and therefore, increased sludge production. Compared to a disc speed of 6 rpm, speeds of 9 and 12 rpm increased sludge mass production 73% and 83%, respectively. This observation may have been a short-term phenomenon because under steady-state conditions the COD should reach a relatively constant level and sludge production would stabilize. The results of this study clearly demonstrated that operation of the RBC unit at a hydraulic loading rate of 1.0 gpd/ft^2 and disc rotational speed of 9 rpm would provide for efficient treatment as well as minimize sludge production. These operating conditions would also reduce sludge disposal maintenance in the clarifier to approximately once per week. Reduced sludge production would decrease the septic-tank cleaning frequency and ultimately reduce the maintenance costs of the entire treatment system. These two major incentives could help to persuade the average homeowner to use the RBC as an alternative for the treatment of septic-tank effluent.

CAPITAL AND OPERATING COSTS

According to Ronald Antonie (1976), the capital cost for a package RBC plant with steel tankage and a surface area of 205 ft^2 would range from \$700-\$750. This cost compares favorably with the materials cost for the unit used in this study.

In this study, power consumption was a major cost; approximately \$3.00 per 1,000 gallons of wastewater treated. Since the RBC showed satisfactory treatment efficiency at a disc speed of 9 rpm, operating costs could be reduced further by reducing the motor size from 1/2 to 1/4 hp. A low operation (power) cost should greatly reduce the owner's incentive to interrupt power supplied to the treatment system in order to reduce operating costs.

The high-quality effluent discharged by the RBC greatly reduced the potential for leach field failure. This feature will reduce the need and cost associated with leach field replacement. The high effluent quality from the RBC unit could also be used to convince county and other government agencies of the acceptability for installing relatively high density septic-tank/RBC systems in areas of shallow groundwater or highly permeable soils. As the possibility of groundwater contamination decreases through the installation of onsite RBC units, the county's and state's minimum lot size requirement possibly could be reduced. At present, for off-site and on-site water sources, the minimum required lot sizes in New Mexico is and 0.75 acres (NMEID 1990). With an RBC system in place the lot sizes could potentially be reduced to 0.25 to 0.5

acres depending on the type of soil. A decrease in minimum lot size may be possible because the potential for transport of nitrogen is reduced due to effective nitrogen treatment provided by the RBC. A second factor which is not influenced by nitrogen treatment is soil permeability. Reduced permeability will tend to negate the effects of nitrogen treatment by increasing the minimum lot size. Influence to pathogen reduction by the leach field is not considered in this analysis because monitoring of indicator organisms around the RBC was not included in the scope of the project.

EVALUATION OF THE OPERATIONAL SCHEME FOR NITROGEN REMOVAL

Phase I

During this phase all RBC stages were maintained aerobic. The COD removal was high and ranged from 74% to 86% (Table 6). However, the overall nitrogen reduction was low (19%; Table 5) because the RBC was not capable of achieving nitrification/denitrification. Nitrification/denitrification activity was very limited due to temperature. The development of the nitrifying/denitrifying bacteria was inhibited by the low water temperature (10 ^{o}C). Nitrification and denitrification rates in RBCs are more temperature sensitive than carbonaceous oxidation rates (Antonie 1978). The two mechanisms effective in removing nitrogen were biomass synthesis and organic-N adsorption (Table 7). Both mechanisms are feasible because high COD and SS removal were achieved during this phase.

Phase II

During this phase the third stage was maintained near anaerobic or at reduced oxygen concentration to induce denitrification in the system. Because of the low temperature (Table 4) the nitrifying/denitrifying bacteria did not become well established until the last two months of the phase when the average water temperature reached 23 ^{o}C. The data for this period show that nitrification was improved significantly, increasing from 0.1% in Phase I to 30.1% during Phase II. The nitrate increased from -80% to 59% and ammonia reduction increased from 6% to 90% (Table 7). The overall nitrogen removal increased to 60%. Denitrification was not more extensive because the carbon source was limited (soluble COD 75 mg/L) in the third stage.

Previous studies indicated that 7 mg COD or 5.2 mg BOD are required per mg NO_3-N removed (Argaman and Brenner 1986, Piluk and Hao 1989).

Phase III

Phase III evaluated the effect of recycling effluent and establishing low oxygen conditions in the first stage of the RBC. Ammonia conversion was complete throughout this phase because nitrifying bacteria were so well developed that the low water temperature (11°C) at a recycle ratio of 2:1 did not affect the nitrification process. Denitrification improved progressively with the decrease of recirculation because a higher soluble COD was available and less oxygen was present in the first stage. The greatest nitrogen loss through denitrification was 53.2% and occurred at zero recycle. Under this condition denitrification represented about two-thirds of the total nitrogen lost (Table 6). As result of this improvement the overall total nitrogen removal was increased to a high of 82% at zero recycle.

Mechanisms for Nitrogen Loss

Ammonia Volatilization

Ammonia volatilization is not easy to accomplish, especially in a system like the RBC where the pH of the septic tank effluent was relatively low (8.0) and disk rotation was not aggressive enough to strip out the free ammonia. In spite of its limited potential, ammonia volatilization was considered. All free ammonia (0.6 to 3.6%, Table 5) was assumed lost due to volatilization. Theoretical calculations showed that NH_3-N loss by stripping was very low in all phases (Table 5) and accounted for an average of 1.5% of the total nitrogen entering the RBC. In general ammonia volatilization accounted for 3% for the total nitrogen removed (Table 6). Thus, the loss of nitrogen by ammonia stripping was basically insignificant.

Biomass Incorporation

Nitrogen lost due to biomass assimilation accounted for 5% of the total nitrogen entering the system (Table 5). In Phase I this mechanism accounted for 42% of the total nitrogen decrease (Table 6). Biomass incorporation represented an average of 13% of the total nitrogen lost in Phases II and III. This variation occurred because almost no nitrification/denitrification occurred during Phase I while different levels of nitrification/denitrification occurred throughout Phases II and III.

Organic-N Adsorption

The reduction of the organic-N across the RBC occurred through adsorption of suspended solids by the biofilm. Organic-N removed by adsorption averaged 14.5% (Table 5) throughout the study. Excluding Phase I, the nitrogen lost this way accounted for an average of 37% of the nitrogen lost (Table 5). Accordingly, Organic-N adsorption was an important mechanism in nitrogen removal in the RBC system.

Nitrification/Denitrification

The principle of biologically induced nitrogen removal in wastewater treatment facilities is based on the activity of populations of autotrophic nitrifying and heterotrophic denitrifying bacteria and their capability to oxidize and reduce nitrogen from ammonia to nitrate and to nitrogen gas (Hitdlebaugh and Miller 1981). Therefore, to control total nitrogen levels in the wastewater both nitrification and denitrification must be achieved. In the RBC system carbonaceous oxidation and nitrogen removal can be achieved in the same unit. Many studies have reported that simultaneous nitrification/denitrification occurs along with carbonaceous oxidation in an RBC (Atwater and Bradshaw 1981, Gujer and Boller 1990, Masuda et al. 1982, Rusten and Odegaard 1982). Sometimes nitrification activity is evidenced by the color of the biofilm. The nitrifying biofilm is reported to be tan to bronze in color and darkens with aging (Brenner et al. 1984). This color and color change were observed during the latter part of the first year and throughout the second year. In the RBC process, oxygen seldom penetrates into the deepest part of the biofilm. Therefore, denitrifying bacteria can exist in the anaerobic inner portions within the biofilm. This portion of the film utilizes organic carbon to reduce nitrate nitrogen to gaseous nitrogen (N_2 or N_2O). The results demonstrate that the RBC system was capable of achieving nitrification/denitrification in addition to the carbonaceous oxidation. By difference in nitrogen mass balance (Table 5) the only other mechanism which accounts for nitrogen removal is denitrification. The change in the nitrate concentrations across the RBC is described as net change because nitrification/denitrification reactions can occur concurrently (Table 7).

Rusten and Odegaard (1982) found that with raw wastewater, denitrification can be achieved down to 5°C. In the present study, denitrification increased from 30.1% to 53.2% when the anoxic stage was changed from the third to the first stage where sufficient COD would be available for the bacteria. A previous study has shown that nitrogen removal can be obtained in the RBC without using an external carbon source by making the first stage anaerobic and recycling the final effluent containing nitrate to the first stage (Odegaard and Rusten 1980); a similar modification was used in Phase III. This study showed that as the recycle ratio decreased, denitrification increased causing the overall total nitrogen removal to increase as well. This finding agreed with Gujer and Boller (1990) in their modeling of an RBC system. Using a series combination of rock and sand filters, the results of Piluk and Hao (1989) agreed with this finding. They also found that denitrification improved with the reduction of recirculation because higher dissolved COD, less oxygen, and therefore, lower nitrate, were present in the first reactor. However, Odegaard and Rusten (using a pilot scale RBC), and Qasim (using a pilot scale suspended growth reactor) found that nitrogen removal was enhanced with the increase of the recycle ratio (Odegaard and Rusten 1980, Qasim et al. 1990).

Best Operational Scheme

The results of the study show that 100% ammonia conversion and the highest level of denitrification were achieved during Phase III at zero recycle (Table 5). Therefore, to maximize the capabilities of the RBC regarding nitrogen removal, the first stage of the RBC should be maintained anaerobic while operating the RBC at a hydraulic loading rate of 1 gpd/ft^2 and disk rotational speed of 9 rpm. If necessary, low wastewater temperatures can be compensated for by decreasing the hydraulic loading rate to 0.5 gpd/ft^2 (Antonie 1978). The treated wastewater will be free of ammonia and organic nitrogen, with nitrate-N in the range 3 to 7 mg/L, total and soluble COD around 35 mg/L, and DO in the range 2 to 3 mg/L DO. Under these conditions the nitrate content of the treated water will also be less than the required health limit (10 mg/L NO_3-N). Therefore, the effluent will not be a threat to groundwater .

OPERATION AND MAINTENANCE

The use of simple mechanical and electrical components in the pilot RBC resulted in very low maintenance, meaning that minimal skills will be required for effective operation of the treatment unit. However, the RBC system did have some hydraulic operational problems, including:

- Clogging of the effluent pipe by the large solids that slough from the discs caused flooding in the RBC unit.
- Clogging of the septic tank pump interrupted the flow to the RBC. However, interrupting the operation of the RBC could be extended for a few days without serious consequences in performance
- Pump problems occurred especially during the recycle phase.
- Manual removal of the sludge from the secondary clarifier was required.

As a result, the RBC unit requires frequent labor attention to deal with these problems. This requirement represents a major drawback or individual households. The typical homeower does not have the desire or technical knowledge to deal with these types of problems. A potential solution ot this situation is for septic tank installation and pumping businesses to provide a monthly or as needed maintenance service on a contract or cost basis. In larger RBC systems these operational problems would not occur (Atwater and Bradshaw 1981). Atwater and Bradshaw have reported that a septic tank-RBC configuration (at a much larger scale) for a flow of 4,000 gpd is also a practical choice due to its efficient treatment and operational simplicity. Other positive aspects of using the RBC were that no odors or health problems were associated with its operation. Also, the high quality of the effluent disposed to the soil will protect the leach field from failure due to saturation.

Operating the RBC at a disc rotational speed of 9 rpm was found to control excessive growth of biomass on the slides of the RBC tank effectively. Thus, the final design of the RBC should provide for effective transmittal of suspended solids through each stage to the final effluent. Increasing the opening at the bottom of each stage divider from 3/4" -1.0" will allow sufficient space for solids to flow easily through the entire unit. This detail will reduce significantly the requirement for scraping solids trapped within a single stage.

CONCLUSIONS

Based on the results obtained through this study, the following conclusions are drawn:

1. Efficient treatment characteristics provide good incentives for the RBC to be used as an alternative technology for onsite treatment of wastewater generated by a single family dwelling.
2. The RBC demonstrated the capability to consistently achieve 70%-90% COD removal and 60%-70% removal of TSS. A typical effluent quality included 35 mg/L COD and 20 mg/L TSS.
3. Treatment efficiency for an onsite unit can be optimized by operating the RBC at a hydraulic loading rate of 1.0 gpd/ft^2 and a disc rotational speed of 9 rpm.
4. A disc rotational speed of 9 rpm maximized oxygen availability to the biofilm and minimized the requirement for scraping solids but increased the frequency for removal of solids from the secondary clarifier in the RBC system.
5. The RBC was able to regain a stable operation and produce a COD removal efficiency of 70% even after a four-day power interruption.
6. Temperatures lower than 10 ^oC affected the development of the nitrifying/denitrifying bacteria while higher than 80% carbonaceous removal was achieved under this low temperature condition.
7. Without recycle or any other modifications the RBC system was capable of achieving a minimum nitrogen removal of 19%.
8. Nitrification/denitrification were the major mechanisms responsible for nitrogen removal.
9. Ammonia volatilization and nitrogen incorporation into the biomass were insignificant mechanisms for nitrogen removal.
10. The best operational scheme for nitrogen removal occurred when the first stage was operated at near anaerobic conditions with zero recycle. The highest total nitrogen removed under this condition was 82%.

11. The RBC system has significant maintenance needs, but no odors or health problems were associated with the operation except during the first three weeks of the study when a neighbor complained of odor. Accordingly, the use of the RBC may not be attractive for an individual home because it requires frequent labor attention and monitoring to prevent overflowing, pump malfunction, and sludge removal. However, the RBC can be considered the technology of choice in situations where protection of the groundwater is critical such as in areas of shallow groundwater or high permeability soils.

LITERATURE CITED

American Water Works Association, American Public Health Association, Water Pollution Control Federation, Standard Methods for the Examination of Water and Wastewater. 16th Edition. 1985.

Antonie, R.L., Fixed Biological Surfaces Wastewater Treatment-The Rotating Biological Contactor. CRC Press, Inc., Cleveland, Ohio. 1978.

Argaman, Y. and Brenner, A., "Single-Sludge Nitrogen Removal: Modeling and Experimental Results." Journal of Water Pollution Control Federation, **58**, 8, 853. 1986.

Autotrol Wastewater Treatment Systems Design Manual, Autotrol Corporation, Bio Systems Division, Milwaukee, WI. 1978.

Atwater, J.W., and Bradshaw, G., "Operational Performance of an RBC Unit in Conjunction with a Septic Tank." Canadian Journal of Civil Engineering, **8**, 4, 477. 1981.

Bates, V., Comparative Performance of Rotating Biological Contactors Using Air and Pure Oxygen, Master's Thesis, University of Missouri-Rolla. 1978.

Bitton, G., and Gebra, C.P., Groundwater Pollution Microbiology. John Wily and Sons, New York, NY. 1984.

Brenner, R.C., Heidman, J.A., Opatken, E.J., and Petrasek, A.C., "Design Information on Rotating Biological Contactors." Wastewater Research Division Municipal Environmental Research Laboratory, Office of Research and Development, U.S. Environmental Protection Agency, Cincinnati, Ohio 45268. 1984.

Canter, L.W., and Knox, R.C., Septic Tank System Effects on Ground Water Quality. Lewis Publishers, Inc., Chelsea, MI. 1986.

Chittenden, J.A., and Wells, W.J., "Rotating Biological Contactors Following Anaerobic Lagoons." Journal of Water Pollution Control Federation, 43, 746. 1971.

"Design Manual, Onsite Wastewater Treatment and Disposal Systems." Office of Research and Development Municipal Environmental Research Laboratory, Cincinnati, OH 45268. 1980.

Environmental Protection Agency, "Design Manual, Onsite Wastewater Treatment and Disposal Systems." Office of Research and Development Municipal Environmental Research Laboratory, Cincinnati, OH 45268.

Forgie, D.J.L, Application of the Rotating Biological Contactor to Individual Household Wastewater Treatment. Master's Thesis, University of Saskatchewan. 1975.

Friedman, A.A. 1979 "Effect of Disk Rotational Speed on Biological Contactor Efficiency." Journal of Water Pollution Control Federation, 51, 2678. 1979.

Gaudy, A.F., and Gaudy, E.T., Element of Bioenvironmental Engineering. Engineering Press, Inc., San Jose, CA. 1988.

Grady, C.P.L., Jr., and Lim, H.C., Biological Wastewater Treatment. Marcel Dekker, Inc. New York, NY. 1980.

Gujer, W., and Boller, M., "A Mathematical Model for Rotating Biological Contactors," Water Science Technology, **22**, 1/2, 53. 1990.

Hitdlebaugh, J.A., and Miller, R.D., "Operational Problems with Rotating Biological Contactors." Journal of Water Pollution Control Federation, **53**, 8, 1283. 1981.

Kreissl, J.F., "A Statistical Abstract of the Unsewered U.S. Population." Water Engineering Research Laboratory, Office of Research and Development, U.S. EPA, Cincinnati, OH. 1982.

Masuda, S., Watanabe, Y., and Ishiguro, M., "Simultaneous Nitrification and Denitrification in a Rotating Biological Contactor." Proceeding of the First International Conference on Fixed-Film Biological Processes, Kings Island, OH. 1982.

Metcalf and Eddy, Inc., Wastewater Engineering: Treatment/Disposal/Reuse. McGraw-Hill, Inc. 1979.

Miller, R.D., Hitdlebaugh, J.A., Noss, C.I., and Smith, E.D., "Nitrification Using the Rotating Biological Contactor." The American Institute of Chemical Engineers, **77**, 209, 186. 1981.

McQuillan, D. M., and Keller, N. S.,"Ground Water Contamination in New Mexico 1927-1986." Proceedings of the 32nd Annual New Mexico Water Conference, New Mexico State University, Las Cruces, New Mexico. 1987.

Moody, D.W. 1986 National Water Summary-Hydrologic Events and Ground Water Quality, U.S. Geological Survey, Water Supply Paper 2325, Washington, D.C. 1986

New Mexico Environment Improvement Division (NMEID), "Application for the Treatment of Unsewered Systems Form." Page 2, NMEID, Santa Fe, NM. 1990.

Odegaard, H., and Rusten, B., "Nitrogen Removal in Rotating Biological Contactors without the Use of External Carbon Source." Proceedings of the First National Symposium on Rotating Biological Contactor Technology, University of Pittsburg, Vol II, 1301. 1980.

Pano, A., and Middlebrooks, E.J., "Kinetics of Carbon and Ammonia Nitrogen Removal in RBCs." Journal of Water Pollution Control Federation, **55**, 7, 956. 1983.

Piluk, R.J., and Hao,O.J., "Evaluation of Onsite Waste Disposal System for Nitrogen Reduction." Journal of Environmental Engineering, **115**, 4, 725. 1989.

Piper, R.G., McElwain, I.B., Orme, L.E., McCraren, J.P., Fowler, L.G., and Leonard, J. R., Fish Hatchery Management. United States Department of the Interior, Fish and Wildlife Service, Washington, D.C., 1982.

Poon, Calvin P.C. 1981. "Upgrading with Rotating Biological Contactor for BOD Removal." Journal of Water Pollution Control Federation, 53, 474. 1981

Power, S. E., Collins, A. G., Edzwald, J. K., and Dietrich, J. M., "Modeling an Aerated Bubble Ammonia Stripping Process." Journal of Water Pollution Control Federation, **59**, 2, 92. 1987.

Prakasam, T.B.S., "Denitrification." Journal of Water Pollution Control Federation, **56**, 6, 582. 1984.

"Process Design Manual for Nitrogen Control." U.S. Environmental Protection Agency, Technology Transfer. 1975.

Qasim, S.R., Parker, C.E., and McMillon, R.T., "Biological Nutrient Removal a Pilot Plant Study." Proceedings of the 45th Purdue Industrial Waste Conference, Purdue University, Ann Arbor Science Publishers. May 1990.

Roesler, J. F., Smith, R., and Eilers, R. G., "Simulation of Ammonia Stripping from Wastewater." Journal of the Sanitary Engineering Division, **97**, SA3, 269. 1971.

Rusten, B., and Odegaard, H., "Denitrification in a Submerged Bio-Disc System with Raw Sewage As Carbon Source." Proceedings First International Conference on Fixed-Film Biological Processes, Kings Island, Ohio. 1982.

Shieh, W. K., Chen, C. Y., Chang, S. D., "Preliminary Study of a Suspended Growth Predenitrification System." Journal of Biotechnology and Bioengineering, **24**, 11, 2557. 1982.

Weng, C.N., and Molof, A.H., "Nitrification in the Biological Fixed-Film Rotating Disk System." Journal of Water Pollution Control Federation, **46**, 7, 1674. 1974.