Effects of Hydraulic Loading and Laundry Detergent on the Operation of Aerobic Package Treatment Systems

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(Abstract)

This study focused on three potential problems with the operation of aerobic package treatment systems: hydraulic retention time, laundry detergents, and hydraulic surges.

To determine the effect of hydraulic retention time on system performance, six bench scale activated sludge systems were constructed. Wastewater from an actual residence was collected twice per week and fed to the small activated sludge systems. Two of the systems had a hydraulic retention time of 2 days, two had a hydraulic retention time of 1 day and two had a hydraulic retention time of 0.5 days.

Effluent quality was stable and good with regard to chemical oxygen demand (COD) and ammonia (NH_3-N) and seemed to be independent of hydraulic retention time. All of the systems performed well, despite considerable variability in influent strength.

To study the effect of high concentrations of laundry detergents on the operation of package treatment systems, three of the six laboratory systems were fed high concentrations of detergent. Other than some residual COD

from the detergent, no effect on system performance was observed.

The final component of the study was the modification of an existing package treatment system to equalize flows from an automatic washing machine.

After a month of operation the modified system produced a more constant effluent quality, than did the unmodified system. The field system, with and without modification, had a low mixed liquor suspended solids (MLSS) concentration (35 mg/L). This was probably the result of the long hydraulic retention time.

As a result of the low MLSS the system, with and without modification, did not meet Virginia effluent requirements for BOD_5, TSS or D.O. or generally accepted levels of NH_3-N.

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1. Introduction

Traditionally, rural wastewater has been disposed of through some type of soil absorption system. In most cases, conventional septic tanks with leach fields are still considered to be the preferred method of waste disposal when population density is too low to allow for centralized sewage collection and disposal. In many cases, however, site conditions preclude the use of soil absorption. In these cases housing development is dependent on the use of alternative disposal systems, such as aerobic package treatment systems.

Aerobic package treatment systems are generally designed as small self-contained extended aeration treatment plants. Theoretically, these systems should be able to produce a high quality effluent, similar to municipal activated sludge plants, and suitable for surface water discharge. Unfortunately, numerous studies of field performance have shown highly variable effluent quality. Though much of the poor performance could be attributed to improper maintenance and mechanical failure, poorly treated effluent was often produced by mechanically sound systems. There seems to be a problem with the transfer of design assumption from full scale municipal plants to individual household systems.

The objective of this study was to identify factors that were causing poor field performance of aerobic package treatment systems. Specifically this project focused on three factors:

1) The effect of hydraulic retention time on the performance of small extended aeration systems;
2) The effect of hydraulic surges on system performance; and
3) The effect of high concentrations of laundry detergent on system performance.

The onsite systems often operate with a hydraulic retention time (HRT) of three days or longer. Suggested HRTs for full scale extended aeration systems range from 18 to 36 hours (Metcalf and Eddy, 1991). To determine the effect of shorter HRTs on system performance, six bench scale activated sludge systems were constructed. Two of the systems had a HRT of 2 days, two had a HRT of 1 day and two had a HRT of 0.5 days. Wastewater from an actual residence was collected twice per week and used as influent.

Upon completion of the HRT experiment the laboratory systems were used to assess the effect of laundry detergent on extended aeration systems. Laundry detergent was added to the feed of 3 of the laboratory systems, while 3 were operated as controls.

Throughout the HRT experiment and the laundry detergent experiment, the laboratory systems were monitored for effluent quality .

The final component of the study was the modification of an existing package treatment system to attenuate plug flows from an automatic washing machine. Wastewater from the washing machine was stored in a large tank and pumped into the treatment system at a steady rate. This eliminated the most significant hydraulic surge from the waste stream.

2. Literature Review

This section will establish some of the operational problems encountered in previous studies of aerobic treatment systems. The National Science Foundation (NSF) Standard 40, which reported laboratory testing of the system used in this study will be presented along with field studies. Additionally, studies of activated sludge and extended aeration processes will be presented to provide a theoretical framework for the system modifications proposed by this study.

2.1 Virginia Regulations

The Virginia State Water Control Board has issued a general Virginia Pollutant Discharge Elimination System (VPDES) permit for individual home sewage treatment systems. Systems discharging less than 1,000 gallons per day are permitted to discharge treated effluent to surface waters provided that a yearly grab sample does not exceed 30 milligrams per liter (mg/L) 5 day biochemical oxygen demand (BOD_5), or 30 mg/L total suspended solids (TSS). Additionally, effluent must be between pH 6.0 and pH 9.0, have a dissolved oxygen content greater than 5 mg/L, and contain no more than 200 fecal coliform bacteria per 100 milliliters (mL). (Commonwealth of Virginia, 1992)

The Virginia Department of Health requires that all onsite treatment systems be approved by the National Sanitation Foundation (Cooley, 1993).

2.2 NSF Standard 40

The National Sanitation Foundation is an independent organization which tests and certifies technologies to be safe with regard to public health and the environment. Specifically, the NSF Standard 40 is a certification of individual aerobic wastewater treatment plants. The treatment plant used for this study was tested and certified under the provisions of NSF Standard 40 in 1984. (NSF, 1984)

The NSF Standard 40 consisted of two parts. The first was an evaluation of effluent quality under normal operational conditions. A typical daily wastewater flow pattern was achieved by distributing 80 equal doses to simulate an average day. The system received 500 gallons per day, which is the maximum rated capacity of the system. All wastewater used for this testing was comminuted municipal wastewater. Effluent and aeration chamber samples were tested for dissolved oxygen (D.O.), total and volatile suspended solids (TSS and VSS), pH and temperature. Additionally, effluent samples were tested for BOD_5, color, odor, and the presence of oily film and foam. No testing was reported for nitrogen species. (NSF, 1984)

The second part of the NSF Standard 40 was stress testing. Under this protocol, the system was put through a series of stressful flow conditions. These included a simulated wash day, working mother load, equipment failure, and return from nine day vacation. (NSF, 1984)

The influent during testing had a BOD_5 concentration of between 35 and 275 mg/L with a median value of 139 mg/L. Influent TSS ranged from 50 to 590 mg/L with a median value of 196 mg/L. (NSF, 1984)

During normal operation the system produced a high quality effluent in terms of the parameters tested under the NSF Standard 40. BOD_5 was between 2 and 31 mg/l with a median value of 11 mg/l. Effluent TSS was between 2 and 51 mg/l with a median value of 15 mg/l. Of interest in this study, the mixed liquor volatile suspended solids (MLVSS) in the system ranged from 25 to 88 mg/l with a median of 65 mg/l. This is a very low concentration for activated sludge, and especially for extended aeration. (NSF, 1984)

Effluent D.O. averaged 4.2 mg/L with a range of 1.1 to 10 mg/L. Effluent pH averaged 7.4 with a range of 7.0 to 8.2. Color was less than 0.5 units, threshold odor number was less than 1 and there was no oily film or foam.

Stress testing requirements were that the system produce BOD_5 and TSS of 60 and 100 mg/l, respectively, 24 hours after stress conditions were completed (resumption of normal conditions), these conditions were met. (NSF, 1984)

2.3 Previous Field Studies

Many field studies have been performed to evaluate aerobic package treatment systems. These studies were conducted on many different models, and generally did not report specific design parameters. This limits the comparisons which can be made to design problems addressed in this study. In order to establish that poor performance is not limited to the model under study here, and to show that it is, in fact, an industry wide problem, a summary table is presented. This table, taken from Kellam *et al.* 1992, shows inconsistent treatment with regard to BOD_5 and TSS, and generally poor performance with regard to nitrification. Only 4 of the 24 listed mean effluent BOD_5 values were less than the Virginia maximum of 30 mg/L. None of the systems would have met the Virginia limit of 30 mg/L TSS.

The only study which specifically evaluated the performance of the system used for this study was Kellam *et al.*, 1992. Kellam *et al.* tested the effluent of five of these system in use in Southwest Virginia. Kellam's results are presented in Table 2. He found highly variable, and generally poor effluent quality with regard to BOD_5, TSS, and nitrification. It is also interesting to note that MLVSS was consistently very low and that the HRTs for the aeration chambers averaged 3.8 days. (Kellam *et al.*, 1992)

Table 1. Field Data of BOD_5, TSS, NO_3^--N, and NH_3-N (from Kellam *et al.*, 1992)

Description	BOD_5 (mg/L) mean	TSS (mg/L) mean	NO_3^--N (mg/L) mean	NH_4^+-N (mg/L) mean	TN (mg/L) mean
Bernhart 1967[1]	47	94			
Bennett *et al.* 1973	150	150			
Glasser 1975					
Bi-A-Robi[2]	47	75	1.06	50	
Chromaglass[2]	52	83	12	27	
Flygt[2]	27	56	69	10	
Jet[2]	45	83	0.73	31	
Nayadic[2]	70	104	4.4	73	
Otis *et al.* 1975	55	38	19.2	0.74	32.2
	55	65	29.8	0.02	39.11
	36	59	37.1	0.00	40.2
Tipton 1974					
Chromaglass[2]	207	139			
CT86[2]	150	204			
Jet[2]	33	41			
PCD[2]	83	100			
Sanicell[2]	279	126			
Voell and Vance 1974	92	94			
McClelland 1976[3]	13	57			
Sauer 1977	26	48	33.8	0.4	
ARC[4]	37	62			
Brewer *et al.* 1978	31	49	8.9	40.6	
SSWMP[5]	37	39	30	0.9	36
Effert *et al.* 1985	28	38	7.4	-	17.8
	27	39	9.3	-	17.3
	31	62	3.7	-	33.3

[1] Bernhart 1967 as reported by Hutzler *et al.* 1978
[2] systems' names as reported by author
[3] McClelland 1976 as reported by EPA 1978
[4] Appalachian Regional Commission as reported by Hutzler *et al.* 1978
[5] Small Scale Waste Management Project as reported by Hutzler *et al.* 1978

Kellam *et al.* concluded that "the aeration chamber did not perform as a biological reactor". Though significant mechanical and maintenance failures were noted during the study, poor performance seemed to be due, at least in part, to basic system design. (Kellam *et al.*, 1992)

Table 2. Field Results for Aerobic Package Treatment System from Kellam *et al.*, 1992

Site #	BOD_5 (mg/L) (mean)	TSS (mg/L) (mean)	NO_3^--N (mg/L) (mean)	NH_4^+-N (mg/L)	MLVSS (mg/L)
1	60	30	3.2	10.4	96
2	64	135	15.9	6.9	2160
3	85	160	8.1	37.7	202
4	56	17	1.8	20.6	45
5	82	72	13	2.3	29

2.4 Design Considerations

Several different design parameters have been proposed for the design of activated sludge systems in general, and extended aeration systems in specific. These parameters are of two basic types: theoretical and empirical. The theoretical parameters include food to microorganism ratio (F/M) and sludge age (Θ_c). The empirical parameters include HRT and organic loading rates (pounds of BOD_5/cubic foot of aeration volume).

2.4.1 Theoretical Design Criteria

In the 1950s F/M ratio was developed as a control parameter for activated sludge (Garret and Sawyer, 1952; McCabe and Eckenfelder, 1961). F/M ratio was defined as the amount of substrate available for biodegradation (influent BOD_5) per day divided by the mass of organisms in the reactor (MLVSS) (Stall and Sherrard, 1978). This relationship indicated the point on the microbial growth curve at which the system was operating (ei constant growth phase, declining growth phase or auto-oxidation phase). For the extended aeration process a range of F/M ratios of 0.05 to 0.15 is suggested by Metcalf and Eddy (1991).

Generally, in the case of extended aeration, models based upon F/M ratios assume complete oxidation of biological solids. That is that since no solids are wasted, the growth of degradable microorganisms due to substrate digestion is equal to the loss of biomass due to auto-oxidation. (Eckenfelder, 1970)

Eckenfelder (1970) presented a design procedure based on this assumption. The amount of biomass (total mass) required to oxidize all available organics is given by:

$$X_{tm}=\frac{Y_o(S_{otm}-S_{etm})}{fb}$$

Where:
X_{tm} = Total mass of volatile solids in system (Pounds)
Y_o = Growth yield coefficient of biodegradable MLVSS (mg of biodegradable MLVSS/mg of BOD_5)
S_{otm} = Influent BOD_5 (Pounds/Day)
S_{etm} = Effluent BOD_5 (Pounds/Day)
f = biodegradable fraction of MLVSS
b = Microbial decay coefficient, day^{-1}.

Reactor volume was then calculated based on a desired MLVSS concentration (3500 mg/L).

$$V=\frac{X}{(3500)(8.34)}$$

Where:
V = Reactor volume (Million gallons)
8.34 = Conversion from mg/L to pounds/million gallons

For the purposes of this study the proportion of MLVSS which is biodegradable is assumed to be equivalent to the proportion of biodegradable MLVSS yielded from the oxidation of BOD_5. Y_o will be replaced by Y (growth yield coefficient of total MLVSS) and f will be disregarded.

Benefield and Randall (1980) presented a similar model with the addition of a calculation of effluent substrate concentration based on kinetic parameters.

$$S_e = \frac{b}{YK}$$

Where:
S_e = Effluent substrate concentration (mg/L)
K = Specific substrate utilization rate constant (L/mg-day)

In 1970, Lawrence and McCarty proposed a model for the design of biological wastewater treatment. In this model the retention time of biological solids (microorganisms) within the process was stressed as having the greatest impact on system performance and effluent quality. (Lawrence and McCarty, 1970)

$$S_e = \frac{K_s[1+b(\Theta_c)]}{\Theta_c(Yk-b)-1}$$

Where:

K_s = Substrate concentration at one half maximum microbial growth rate, mg/L
Θ_c = Biological solids retention time (sludge age), days
k = Maximum specific utilization rate, day^{-1}

In this model all of the parameters except sludge age are properties of the waste and its associated microbial population. Only sludge age can be used as a process control parameter. Final effluent quality is not a function of either HRT or influent substrate concentration. These parameters do, however, determine the concentration of solids in the aeration chamber. (Lawrence and McCarty, 1970)

$$X=\frac{Y(S_0-S_e)\Theta_c}{(1+b\Theta_c)\Theta}$$

Where:

S_0 = Influent substrate concentration, mg/L
Θ = Hydraulic retention time

This model can be applied to nitrification as well as reduction of organic constituents. Using this model the Water Pollution Control Federation (1983) suggested a minimum sludge age of 6.2 days for nitrification. At sludge ages less than this the health of nitrifying bacterial populations could not be assured. (Water Pollution Control Federation, 1983)

In order to apply this model to extended aeration, complete oxidation of biological solids cannot be assumed. Some solids must leave the system, either through intentional wasting or as effluent suspended solids in order to have a meaningful sludge age. Grady and Lim (1980) reported the normal range of sludge age for extended aeration systems to be 20 to 30 days.

At these long sludge ages there is some question as to whether the parameters calculated for conventional activated sludge are still valid. Goodman and Englande (1974), found that the microbial decay coefficient (b) was not constant with respect to sludge age. As sludge age increased (and the proportion of viable cells decreased) the rate of microbial decay (auto-oxidation) decreased. Goodman and Englande (1974) presented the relationship:

$$b_{\theta_c}=b(0.75^{Ln\theta_c/Ln2})$$

Where:
$b_{\theta c}$ = microbial decay coefficient at extended sludge age
b = conventional activated sludge decay coefficient

An important characteristic of both the relationship presented by Benefield and Randall (1980) and that presented by Lawrence and McCarty (1970), is that effluent substrate concentration is independent of influent concentration. Several studies (Grady and Williams, 1974; Daigger and Grady, 1977a,b; Chudoba, 1983; Gaudy *et al.* 1986; Hao and Lau, 1988) have found that this is not the case. They have found that effluent Chemical Oxygen Demand (COD) increases with increasing influent substrate concentration. Many of these studies (Daigger and Grady, 1977b; Chudoba, 1983; Gaudy and Blachly, 1985; Hao and Lau, 1988) also reported that most of the effluent COD was not untreated influent substrate, but, rather, organic compounds produced by the system. These refractory compounds generally are relatively nonbiodegradeable and do not affect effluent BOD_5.

Researchers have also found that the rate of refractory COD production (mg COD/mg MLVSS/day) is affected by sludge age. At high sludge ages the rate of production was shown to increase with increasing sludge age. (Daigger and Grady, 1977b; Hao and Lau, 1988).

2.4.2 Empirical Design Criteria

In addition to these theoretical models for the design of extended aeration systems, empirical parameters are often suggested.

Barnes and Wilson (1976) proposed a minimum HRT of 1.3 days. Viessman and Hammer (1985) indicated a similar minimum aeration time of one day.

In its 1958 "Report on Individual Household Aerobic Sewage Treatment Systems" the National Academy of Sciences-National Research Council established some basic design assumptions and criteria for aerobic package systems. Long HRTs (2 to 5 times those of conventional activated sludge), would reduce the level of solids in the activated sludge chamber such that excess sludge would not become a problem. Design parameters were based upon a minimum reactor volume (400 gallons per pound of influent BOD_5 per day). (National Academy of Sciences - National Research Council, 1958)

In 1960 the U.S. Public Health Service published a preliminary evaluation of extended aeration systems. It reported both HRT and organic loading rate as design criteria required by states at that time. The most common requirement was a minimum 24 hour HRT in the aeration basin. One state required a maximum of 15 pounds of BOD_5 per day per 1,000 gallons of aeration volume (500 gallons per pound of BOD_5 per day). (U.S.P.H., 1960)

2.5 Laboratory Studies

Several studies have been conducted to discover the effect of long sludge ages (and resulting high concentrations of biological solids) on the efficiency and stability of extended aeration units.

Ludzack (1965) operated a bench scale extended aeration plant at a HRT of approximately one day. The influent consisted of a relatively concentrated feed containing fish meal (BOD_5 = 227 - 308 mg/L). No solids were removed from the system other than those lost as effluent suspended solids. Mixed liquor suspended solids increased steadily throughout the experiment reaching 10,000 mg/L after 24 weeks, and finally exceeding 17,000 mg/L. Though there were occasions of poor effluent quality, the system generally performed well. Low effluent COD and readily settleable solids were observed throughout the experiment. (Ludzack, 1965)

In March of 1967, Gaudy *et al.* (1971) began an experiment to determine the ultimate stability of extended aeration processes. In order to avoid any loss of solids from the system, the effluent was centrifuged, and all solids were returned to the system. In this way researches were assured that the concentration of biological solids would be controlled by internal processes (ie microbial decay) and not by loss of solids during occasions of poor settling. Hydraulic retention time in the aeration chamber was 0.7 days, and the synthetic feed had a COD of 525 mg/L. The researchers were able to operate the system for over 1000 days. (Gaudy *et al.*, 1971)

The MLVSS generally increased through the first 600 days of operation, reaching a level of nearly 18,000 mg/L. As might be expected, effluent suspended were often high (occasionally exceeding 200 mg/L). Despite the fact that these effluent solids were returned to the aeration tank, the system displayed an ability to control its own biological population. After reaching its high point of 18,000 mg/L, the MLVSS dropped off and remained between 12,000 and 16,000 mg/L for the final 400 days of the study. (Gaudy *et al.*, 1971)

2.6 Hydraulic Surges

Among the problems often cited that might explain the poor performance of aerobic package plants are hydraulic surges. Hutzler *et al.* (1978) identified hydraulic surges as a cause for the loss of biological solids in the aeration basin. Bennett *et al.* (1973) found hydraulic surges to be one of two reasons, along with homeowner neglect, for poor system performance. He found that wastewater surges of 60 gallons could enter the system in a period as short as seven minutes.

One of the greatest sources of hydraulic surges in the average household is the automatic washing machine. Bennett, *et al.* (1973) found that 26 percent of total household wastewater use was produced to the washing machine. Siegrist *et al.* (1976), in a study of rural households, found 25 percent of water was used by the washing machine.

2.7 Laundry Detergents

In addition to these surges, washing machines also discharge a significant level of surfactant. Siegrist *et al.* (1976) found that nearly 30 percent of the BOD_5 produced by a household was from the washing machine. Bennett *et al.* (1973) attributed 40 percent of the methylene blue active substances (surfactant) to the washing machine.

McClelland and Mancy (1969) studied the effect of common surfactant on activated sludge. Concentrations of 10 and 20 mg/l linear alkylate sulfonate (LAS), the most commonly used surfactant in laundry detergents, had almost no effect on activated sludge systems. These concentrations were chosen as representative of average domestic wastewater. It is likely that the wastewater flow from an individual household could have short-term concentrations many times this. It is unclear what effect very high concentrations of surfactant would have on small extended aeration systems.

3. Methods and Materials

This research project consisted of three separate studies. The first was a lab-scale investigation of the effect of HRT on the effluent quality of aerobic systems. In the second study the same laboratory apparatus was used to determine the possible effect of high concentrations of laundry detergents on the operation of small systems. Finally, an experiment was performed to assess the effect of flow equalization on a full-scale system under normal operating conditions.

3.1 Treatment System Design

In each study, the field site, refers to a household in Montgomery County, Virginia with an operating aerobic treatment system. This was the same household referred to as APTS I by Kellam *et al.*, 1992. The system had been in continuous operation since installation in October of 1988. The household consisted of two adults and one child. Neither adult worked outside of the home. There was a clothes washing machine and a dishwasher in the home, but no garbage disposal unit.

The package treatment system was an extended aeration treatment system consisting of a primary clarifier, aeration chamber, secondary clarifier and a submersible pump chamber

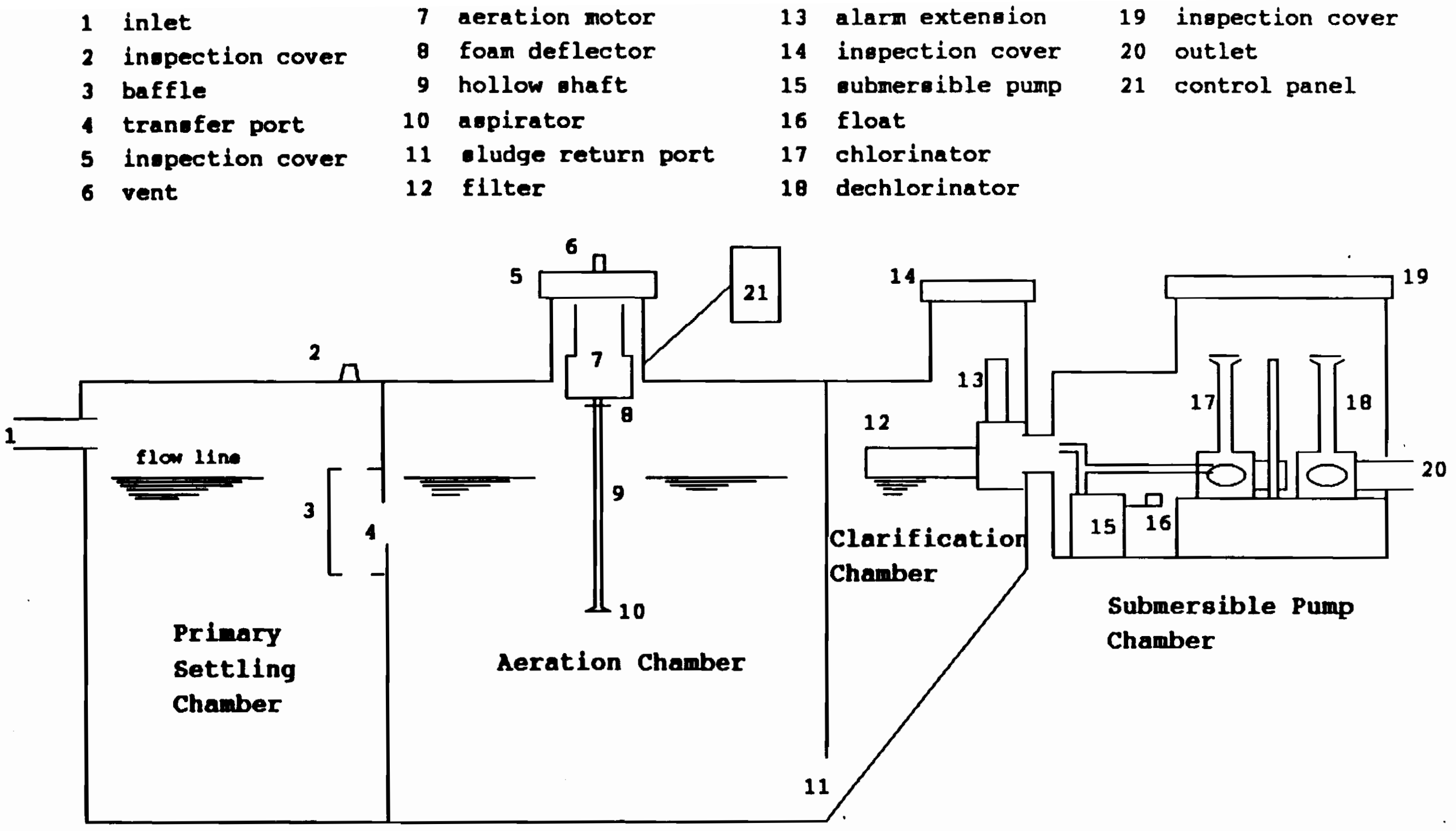


Figure 1: APTS Schematic (from Kellam *et al.*, 1992)

(Figure 1). Wastewater from the household entered the 450 gallon primary compartment where solids were settled and organic material was initially broken down by anaerobic organisms. Fluid then passed through a tee baffle into the 600 gallon aeration chamber. An electrically driven mixer/aspirator kept the waste mixed and aerated to allow for aerobic treatment of organic material. The treated wastewater along with biological solids then passed into a 250 gallon hopper-shaped secondary clarifier. The quiescent conditions in the clarifier allowed biological solids to flocculate and settle through the sludge return port back into the aeration chamber. (NSF, 1984)

Clarified effluent then passed through a synthetic filter to remove any unsettled solids and into the submersible pump chamber. Wastewater flow through the system was by hydraulic displacement until reaching the submersible pump chamber. Water was pumped from the submersible pump chamber by a float-activated, submersible pump. Approximately 11 gallons was discharged from the system through tablet feeders containing chlorination/dechlorination tablets. Approximately 5 gallons in addition to the 11 gallon discharge was pumped back through the synthetic filter to act as backwash. (Kellam *et al.*, 1992) Finally, the treated wastewater was discharged to a small tributary to the North Fork of the Roanoke River.

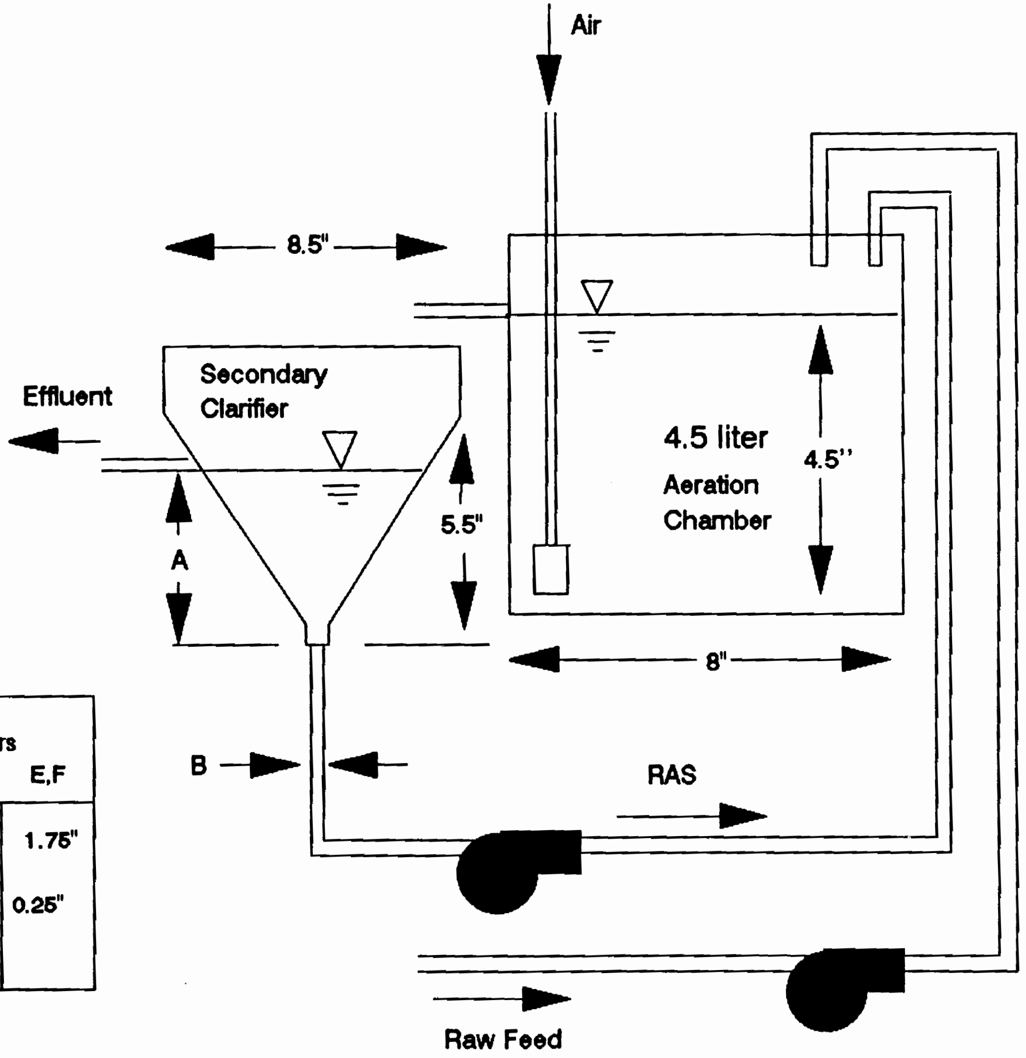
3.2 System Operation

3.2.1 Hydraulic Retention Time

Six, bench-scale, extended aeration systems (labeled A-F) were constructed and operated for a period of 228 days. The six systems were operated at three different hydraulic retention times. Two systems (A and B) were operated at a half day HRT (with respect to the aeration chamber), two (C and D) were operated at a one day HRT, and two (E and F) were operated at a two day HRT.

Each system consisted of a aeration chamber and separate clarifier. The aeration chamber was a square (8 inch by 8 inch) plexiglass tank, with an outlet hole positioned to provide approximately 4.5 L of aeration volume (Figure 2). Aeration was provided by four aeration stones placed in the bottom corners of the aeration chambers. Compressed air was supplied to the aeration stones through glass tubes. The air from the aeration stones also provided mixing to keep the activated sludge in suspension. In order to reduce evaporation from the aeration tanks, the air supply was passed through water bottles to presaturate the air, and the tanks were covered.

Mixed liquor flowed through a 3/8 inch inside diameter (ID) effluent tube into the separate clarifier constructed from a plastic funnel. The inside surface of the clarifier



Dimension	Reactors A,B	 C,D	 E,F
A	4.5"	3.5"	1.75"
B (all tubing, ID)	0.375"	0.375"	0.25"

Figure 2: Schematic of Laboratory System

was continuously scraped by a plastic tube attached to the end of a stainless steel rod. The rod was rotated at 1 revolution per minute (rpm) by an electrical motor. This prevented sludge from forming a bridge in the clarifier, which would interfere with settling.

Initially, all clarifiers were the same volume: 1.3 L. After several months of operation, the systems with the shortest HRTs (A and B) began to exhibit problems with biomass floating to the surface of the clarifier. To prevent this,the retention time within the clarifiers was reduced. Clarifiers with hydraulic retention times of four hours were installed on all six systems over a period of three weeks, starting on day 147. No difference in effluent quality or system performance was noticed, other than the elimination of floating biomass in systems A and B.

No solids were intentionally wasted from the systems. As with the full scale systems, sludge age was determined by solids lost as effluent suspended solids.

Raw feed and return activated sludge (RAS) were both pumped into the aeration chamber by a peristaltic pump. Because of the relatively low flow rates desired for this study, pumps were on timers which allowed operation two minutes out of every twelve. During those two minutes the pumps were adjusted to operate at a flow rate of 28 ml/min for systems A and B, 14 ml/min for systems C and D, and 7 ml/min for systems E and F. Within each system the raw feed and the RAS were pumped at the same rate.

Feed for the systems was collected from the primary settling chamber at the field site. Supernatant was pumped from the chamber into five gallon carboys for transport to the laboratory. Feed was collected twice per week between 5:00 pm and 9:00 pm. Because of foaming problems experienced early in the study, an agreement was made with the homeowner to avoid use of the washing machine on feed collection days. Feed that was stored for more than two days was refrigerated.

Effluent, mixed liquor, and influent samples were periodically taken throughout the study period. Initially, all samples were grab samples. From day 150 until the end of the study, effluent samples were cumulative over a period long enough to collect 500 to 1000 ml (approximately 1.5 hours for systems A and B, 6 hours for systems E and F). Effluent samples were tested for soluble COD, TSS, VSS, Total Kjeldahl Nitrogen (TKN), Ammonia (NH_3-N), Nitrate (NO_3-N), Nitrite (NO_2-N), Orthophosphorus (OP-P), and Total Phosphorus(TP-P). Several of the samples were also tested for both soluble and total BOD_5. Mixed liquor samples were tested for mixed liquor suspended solids (MLSS) and MLVSS. Influent samples were tested for total COD.

Grab samples were taken from carboys of raw feed on twelve separate occasions and tested for total BOD_5, COD, total organic carbon (TOC), TKN, NH_3-N, NO_3-N, NO_2-N, OP, TP, and pH. Total, rather than soluble, COD tests were performed on raw samples, under the assumption that the majority of organic solids in the feed would be available for biological degradation.

3.2.2 Laundry Detergent

On day 228 of operation an experiment was begun to assess the effect of high concentrations of laundry detergents on the operation of the laboratory systems. While maintaining all other operational parameters, laundry detergent was added to the feed of systems A, C, and E. Systems B, D and F were operated without detergent addition, to act as controls. The detergent was a common consumer brand, used by the household (Cheer Free®, containing anionic surfactants). It contained no artificial colors or fragrances. Detergent was mixed into the raw feed to produce a concentration of 400 mg of detergent per liter of feed. This concentration was estimated to be equivalent to that which would be produced by four full loads of laundry per day. This calculation was based on one scoop of detergent per load and the average daily waste flow from this household (182 gal/day) as reported by Kellam *et al.*, 1992.

Sampling from the systems was performed as it had been prior to day 228. The experiment continued through day 282.

3.2.3 Flow Equalization

In order to assess the effect of hydraulic surges from the washing machine on the operation of the field system, a system of flow equalization was installed at the field site. Effluent from the washing machine was collected in a 140 gal tank situated next to the homeowners' washing machine. Water from this tank was periodically pumped into a 280 gal tank located in a small shed built next to the treatment system. A peristaltic pump, operating continuously, pumped wastewater from the 280 gal tank into the primary settling chamber. The pumping rate was adjusted during the first few days to maximize flow equalization, without allowing the storage tanks to overflow. The final flow rate was 400 ml/min.

The system was operated for four weeks to allow the biomass to adjust to the new conditions. At the end of this time samples were taken on four separate days over a period of five weeks. The sampling procedure reported by Kellam *et al.*, 1992, was reproduced as much as possible, to allow for direct comparison of results. On sampling days, samples were taken during the morning (8 am - 10 am), during mid-afternoon (3 pm - 5 pm), and during the evening (9 pm - 11 pm). Grab samples were taken from the submersible pump chamber prior to chlorination. These were tested for: soluble and total BOD_5, COD, TKN, NH_3-N, and TP; soluble NO_3-N, NO_2-N and OP; TSS, VSS, Dissolved Oxygen (D.O.), pH and temperature. In addition, samples were taken from the aeration chamber and tested for MLSS and MLVSS. D.O. measurements were taken in the aeration chamber and the secondary clarifier.

D.O., temperature and pH measurements were taken on-site. All other tests were performed in the laboratory.

3.3 Testing Procedures

3.3.1 Preservation

All samples were preserved as indicated in Standard Methods for the Examination of Water and Wastewater, section 1060B, and EPA document SW846, Test Methods for Evaluation of Solid Waste. The only exceptions to this were those tests which were performed on the Ion Chromatograph (IC, OP and NO_3), and the TOC Analyzer. Samples that were analyzed on the IC and TOC analyzer were filtered and frozen.

3.3.2 Solids

TSS, VSS, MLSS and MLVSS were determined as outlined by Standard Methods for the Examination of Water and Wastewater (2540 D, 2540 E), using Whatman 934-AH glass microfibre, 5.5 cm filters (Hillsboro, Oregon). All solids testing was performed immediately after sampling. In the case of field samples, testing was performed immediately after return to the laboratory (within two hours of sampling).

3.3.3 Organic Constituents

BOD_5 testing was performed in accordance with Standard Methods for the Examination of Water and Wastewater section 5210B. Total BOD_5 tests were performed on unfiltered samples and were not seeded. Soluble BOD_5 tests were performed on samples which had been filtered through a 0.45 µm filter. The dilution water was seeded with primary effluent from the Pepper's Ferry Wastewater Treatment Plant, near Blacksburg, Virginia.

For determination of COD, the closed reflux, titrimetric method was used (Standard Methods for the Examination of Water and Wastewater, section 5220 C). In order to increase precision, the digested samples were titrated with 0.05 molar (M) ferrous ammonium sulfate (FAS) rather than the 0.10 M FAS indicated for the standard method. All effluent samples were filtered through 0.45 µm filters. Raw sewage samples were not filtered.

Sample TOC was determined through the use of a Dohrmann, DC-80 TOC Analyzer (Santa Barbara, CA), (Standard Methods for the Examination of Water and Wastewater section 5310B). Samples were acidified and purged with oxygen to remove any inorganic carbon prior to injection.

3.3.4 Nutrients

Ammonia concentrations were determined by the distillation/titrimetric method (<u>Standard Methods for the Examination of Water and Wastewater</u> section 4500-NH_3 E.). Fifty mL aliquots of sample were used.

The semi-micro-Kjeldahl method (<u>Standard Methods for the Examination of Water and Wastewater</u>, section 4500-N_{org} C.) was used to digest 25 ml samples for the determination of TKN. Final NH_3-N concentrations were determined by the distillation/titrimetric method (<u>Standard Methods for the Examination of Water and Wastewater</u> section 4500-NH_3 E.).

Orthophosphorus concentrations in the raw waste and TP-P concentration in all samples were determined using the ascorbic acid method (<u>Standard Methods for the Examination of Water and Wastewater</u> section 4500-P E.). The persulfate digestion method (<u>Standard Methods for the Examination of Water and Wastewater</u> section 4500-P B.5) was used to digest samples for TP-P determination.

Orthophosphorus concentrations in the effluent samples and nitrate and nitrite concentrations in all samples were determined using a Dionex IC (Sunnyvale, CA) (<u>Standard Methods for the Examination of Water and Wastewater</u> section 4110 B).

3.3.5 D.O. and pH

A Fisher Accumet Model 156 portable pH meter was used for the determination of pH. Dissolved oxygen concentrations were determined with a Yellow Springs INC. (YSI) Model 54, D.O. Meter.

3.4 Statistical Analysis

Mean differences between the operation of systems during the HRT experiment were estimated with the signed rank procedure. This procedure was performed on the Minitab® computer package.

Box plots of data were produced using the Systat ® computer package. These plots include population mean (center line), 25 percent of population above mean (upper hinge), 25 percent of population below mean (lower hinge), maximum value and minimum value. Outliers were considered to be any values that were more than 1.5 times the difference between the hinge values above the upper hinge, or below the lower hinge. Extreme outliers were considered to be any values that were more than 3 times the distance between hinges above the upper hinge or below the lower hinge.

4. Results

4.1 Characteristics of Untreated Waste

The results of tests performed on the untreated primary effluent are presented in Table 3. The tests were performed on grab samples taken during the first two months of laboratory system operation. Presented for comparison are data from Kellam *et al.* (1992) and studies of septic tank effluent.

Table 3. Constituents of Primary Effluent

	Mean	Standard Dev.	Previous Study at this Site[1]	Reported Values for Septic Tank Effluent	
				Viraraghavan[2]	Brandes[3]
COD	281 (257)[4]	89 (53)	411 (275)	568	870
BOD_5	126	40	161	280	223
TOC	66	31		73	310
NH_3^+-N	14	3	15.5	97	75
TKN	29	4	19	-	105
TP	6.3	2.3	24.7 (7.3)	11.6	17
OP	3.4	2.1	3.1	-	-
pH	6.8	6.8-7.0[5]		6.53-7.45	7.4

[1] Kellam *et al.*, 1992

[2] Viraraghavan, T and Warnock, R. G., 1976

[3] Brandes, M., 1978

[4] Several data sets contained a single outlying point which significantly affected the average. Since these may be the result of nonrepresentative samples, an average value for the data with the outlying point removed is presented in parenthesis.

[5] Range

4.2 Hydraulic Retention Time

The laboratory systems reached a state of relatively stable operation on day 34, and remained fairly stable until day 228, when the detergent study was begun. Reactor A was typical of the systems (Figure 3). Though there was a period during which there was loss of biomass (days 150 - 189), followed by a period of high biological solids (217 - 228), these seemed to be a result of changing feed strength, and occurred simultaneously in all six systems. This does not meet the conventional definition of steady state, but still represents stable operation of a system treating variable influent. Therefore, the entire period (day 34 to 228) will be included in analysis of system performance.

All systems performed well throughout the study. There were no problems with bulking or filamentous bacteria. Minor problems did occur with some of the mechanical parts of the secondary clarifiers. These were repaired quickly, and did not affect the overall operation of the systems, other than increasing effluent TSS for a period of one to two days.

On several occasions, high levels of laundry detergent in the raw feed caused problems with foaming in the reactors. These were not planned experiments, but occurred because of washing machine use in the household. Though the resident attempted to avoid use of the washing machine on days in which

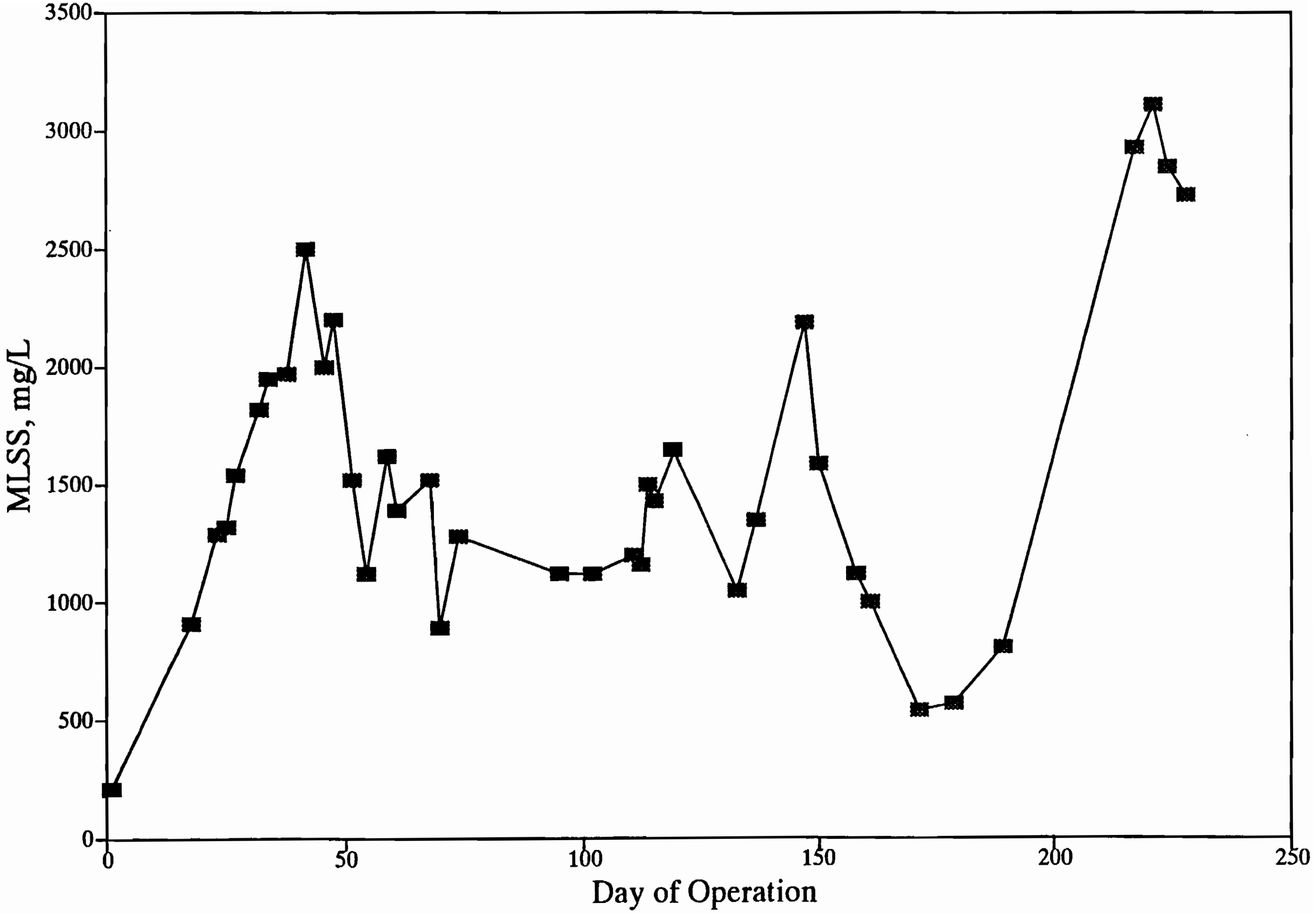


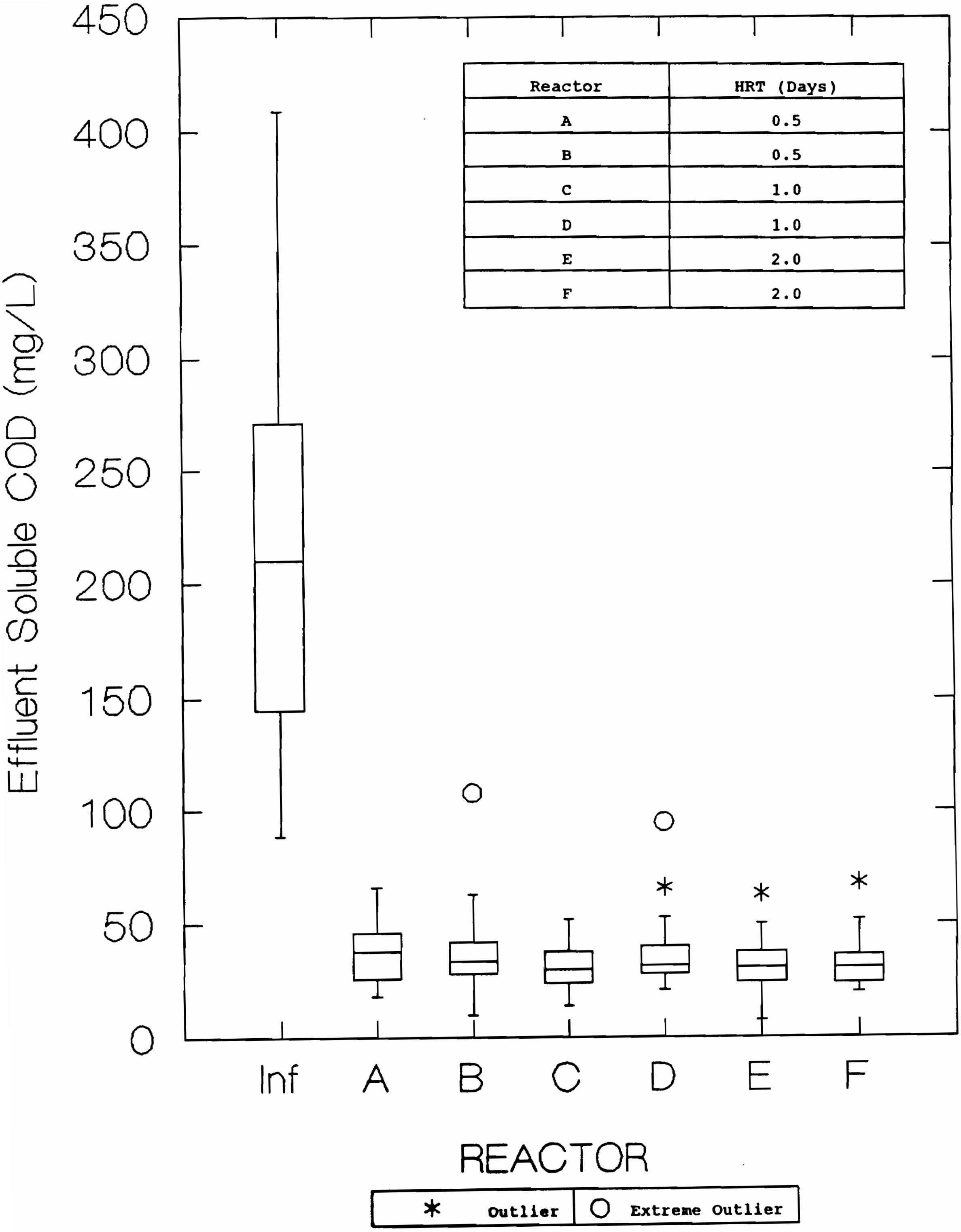
Figure 3. Mixed Liquor Suspended Solids, Reactor A, Days 0 - 228

feed was to be collected, it was sometimes unavoidable. On one occasion (days 128-130) laundry detergent caused foaming over the sides of reactors A, B and E. Though some biomass was lost, all reactors recovered quickly and continued to produce high quality effluent.

4.2.1 Organic Constituents

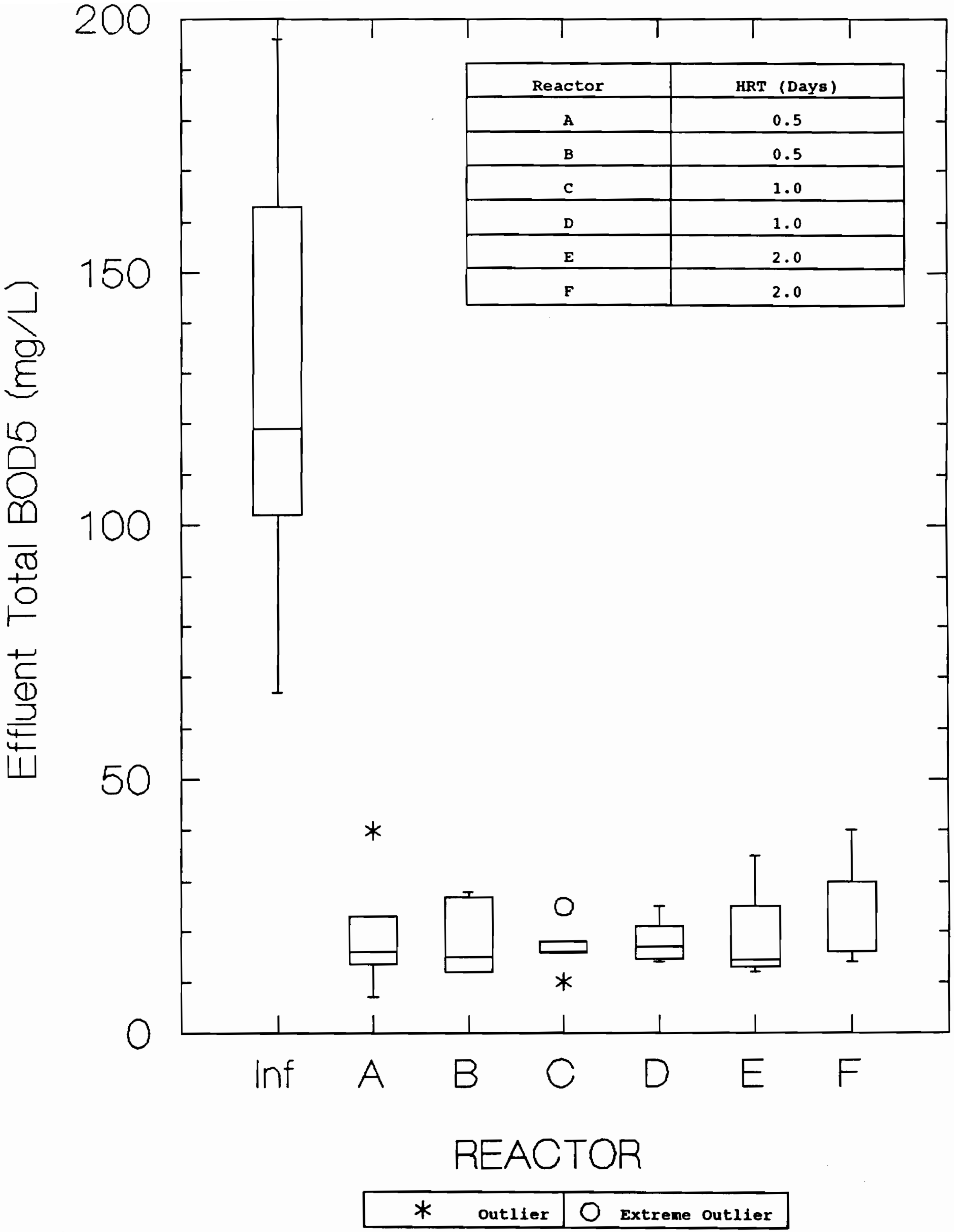
Effluent soluble COD data is presented in Figure 4. All six systems performed well, averaging 36 mg/L COD. Systems A and B (HRT = 0.5 days) averaged 37 mg/L, systems C and D (HRT = 1.0 day) averaged 33 mg/L, and systems E and F (HRT = 2.0 days) averaged 32 mg/L. Standard deviations ranged from 11 mg/L (systems C and E) to 17 mg/L (system B). Though there was a small difference between these average values, differences were not statistically significant (see below).

Effluent total BOD_5 data are presented in Figure 5. The average total BOD_5 for each reactor was less than the 30 mg/l allowed by Virginia regulations. Effluent samples were tested for total BOD_5 on five separate occasions. Three systems (A, E and F) produced an effluent with a BOD_5 higher than 30 mg/L on one occasion. All six reactors provided effective treatment of BOD_5, and there seemed to be no difference between the three HRTs. Systems A and B averaged 19 mg/L, systems C and D averaged 18 mg/L, and systems E and F averaged 21 mg/L. Apparent differences between reactors are probably



Reactor	HRT (Days)
A	0.5
B	0.5
C	1.0
D	1.0
E	2.0
F	2.0

Figure 4. Effluent Soluble Chemical Oxygen Demand, Days 34 - 228



Reactor	HRT (Days)
A	0.5
B	0.5
C	1.0
D	1.0
E	2.0
F	2.0

Figure 5. Effluent Total 5-Day Biochemical Oxygen Demand, Days 34 - 228

attributable to the relatively small number of samples analyzed for BOD_5.

4.2.2 Suspended Solids

Effluent suspended solids levels were often higher than the 30 mg/L maximum allowed by Virginia regulations (Figure 6). Averages for systems with HRTs equal to 0.5, 1.0 and 2.0 were 29, 32 and 26 mg/L, respectively. Though reactors A and E averaged well below 30 mg/L TSS, there were significant periods during the study when the effluent from these reactors contained more than the legal limit. Once again, HRT had no effect on effluent quality.

Testing for MLVSS was begun on day 111. Prior to that, testing was done for MLSS only. MLVSS/MLSS ratios were computed for days 111 to 234 for each reactor. MLVSS for samples taken prior to day 111 were estimated by multiplying MLSS for those samples by the computed MLVSS/MLSS ratio. These ratios are shown in Table 4.

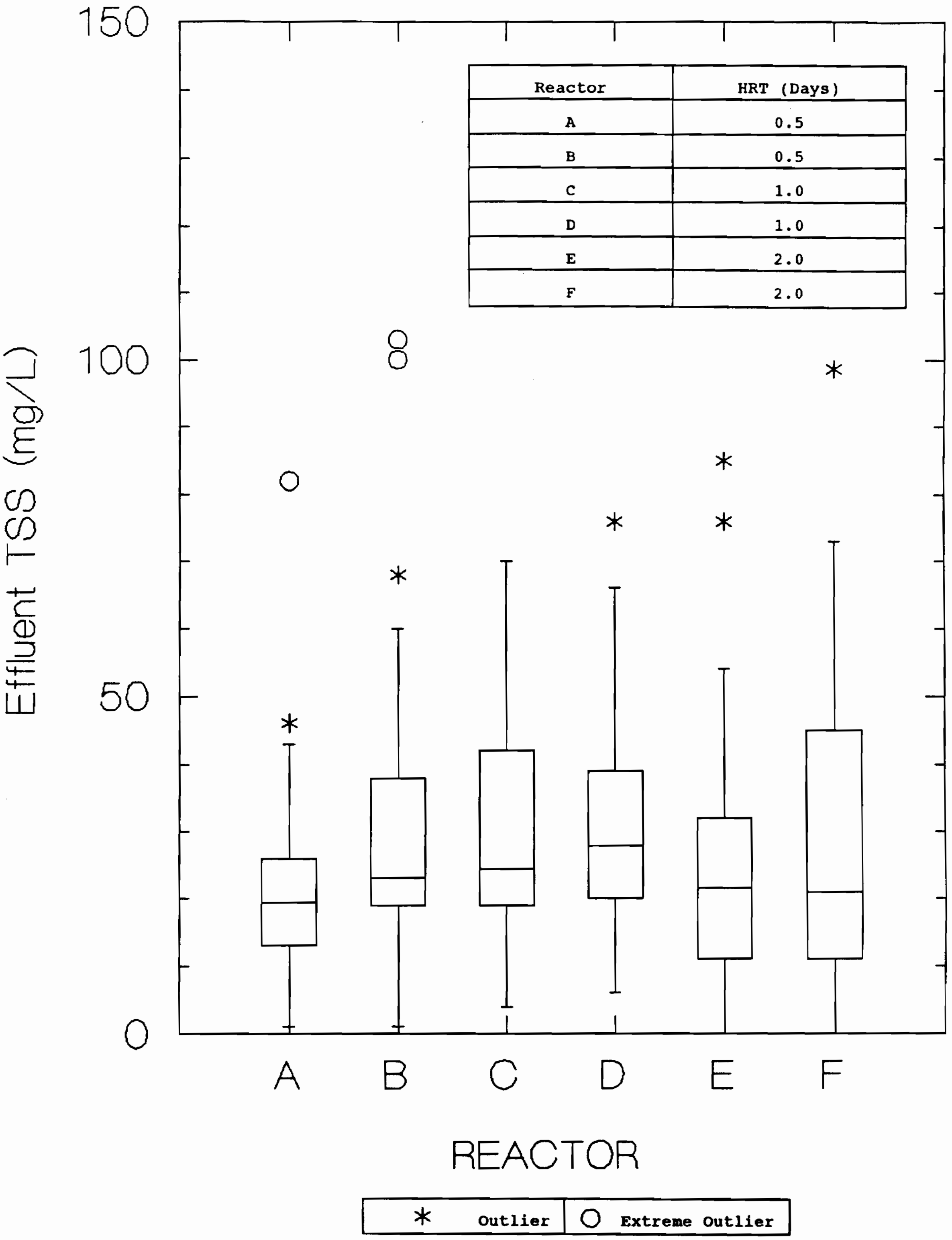
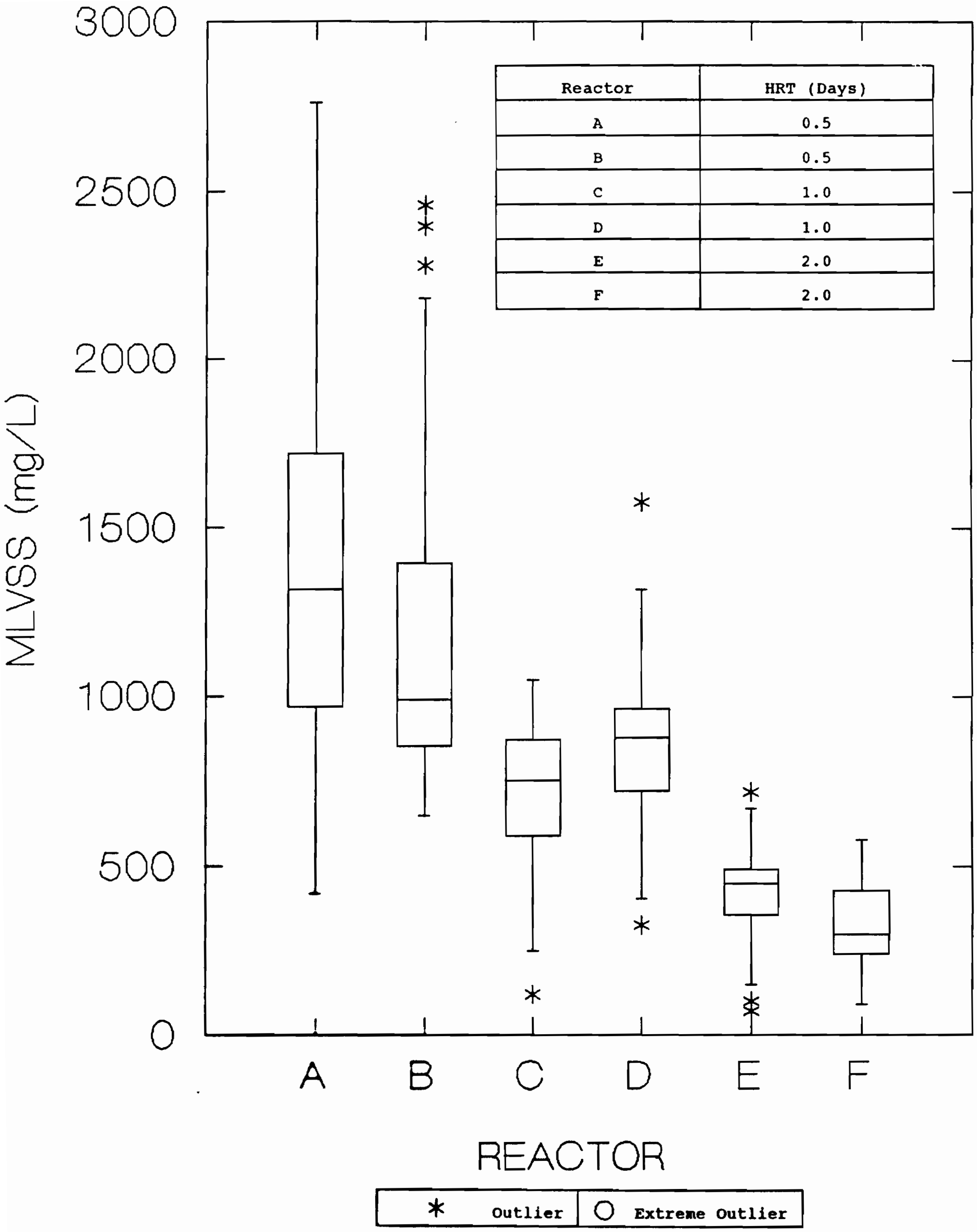


Figure 6. Effluent Total Suspended Solids, Days 34 - 228



Reactor	HRT (Days)
A	0.5
B	0.5
C	1.0
D	1.0
E	2.0
F	2.0

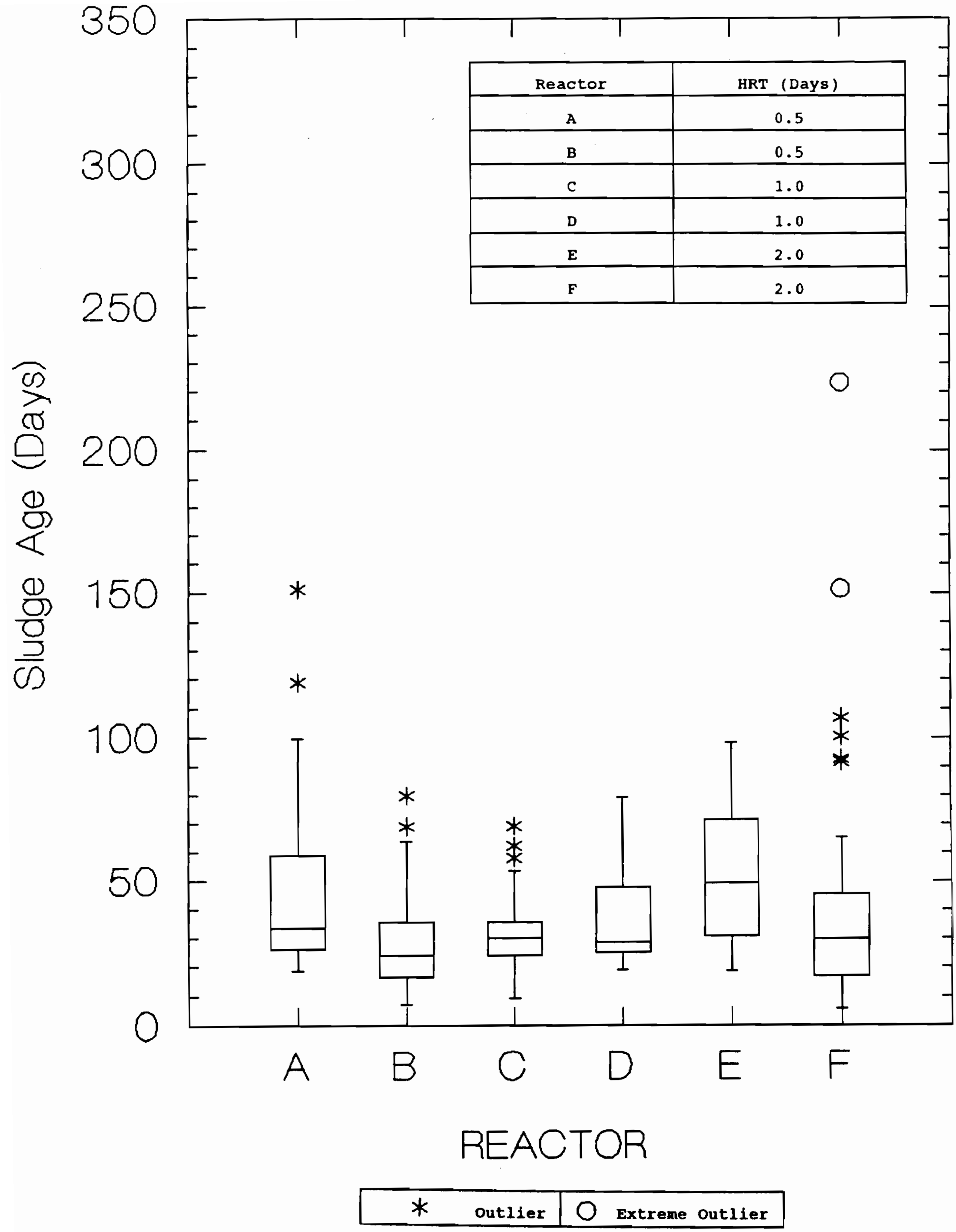
Figure 7. Mixed Liquor Volatile Suspended Solids, Days 34 - 228

Table 4. Mixed Liquor Suspended Solids and MLVSS/MLSS Ratios, Days 34 - 228

Reactor	MLSS (mg/L) (mean)	MLVSS/MLSS Ratio (mean)
A	1665	0.88
B	1415	0.88
C	803	0.87
D	997	0.85
E	495	0.84
F	367	0.88

Mixed liquor volatile suspended solids were higher in the reactors with the lower HRTs (Figure 7). Reactors A and B (HRT = 0.5 days) had an average MLVSS of 1367 mg/L, reactors C and D (HRT = 1.0 days) had an average MLVSS of 779 mg/L, and reactors E and F (HRT = 2.0 days) had an average MLVSS of 411 mg/L. There was a significant amount of variation in the data, resulting in high standard deviations for all of the reactors.

Sludge age for each reactor was calculated from five sample running averages of MLVSS and effluent VSS (Figure 8). Sludge age was not correlated with HRT. Reactors A and B averaged 31 days, reactors C and D averaged 27 days, and reactors E and F averaged 36 days. This is the high end of the range reported by Grady and Lim, 1980, for extended aeration (20 - 30 days).



Reactor	HRT (Days)
A	0.5
B	0.5
C	1.0
D	1.0
E	2.0
F	2.0

Figure 8. Sludge Age, Days 34 - 228

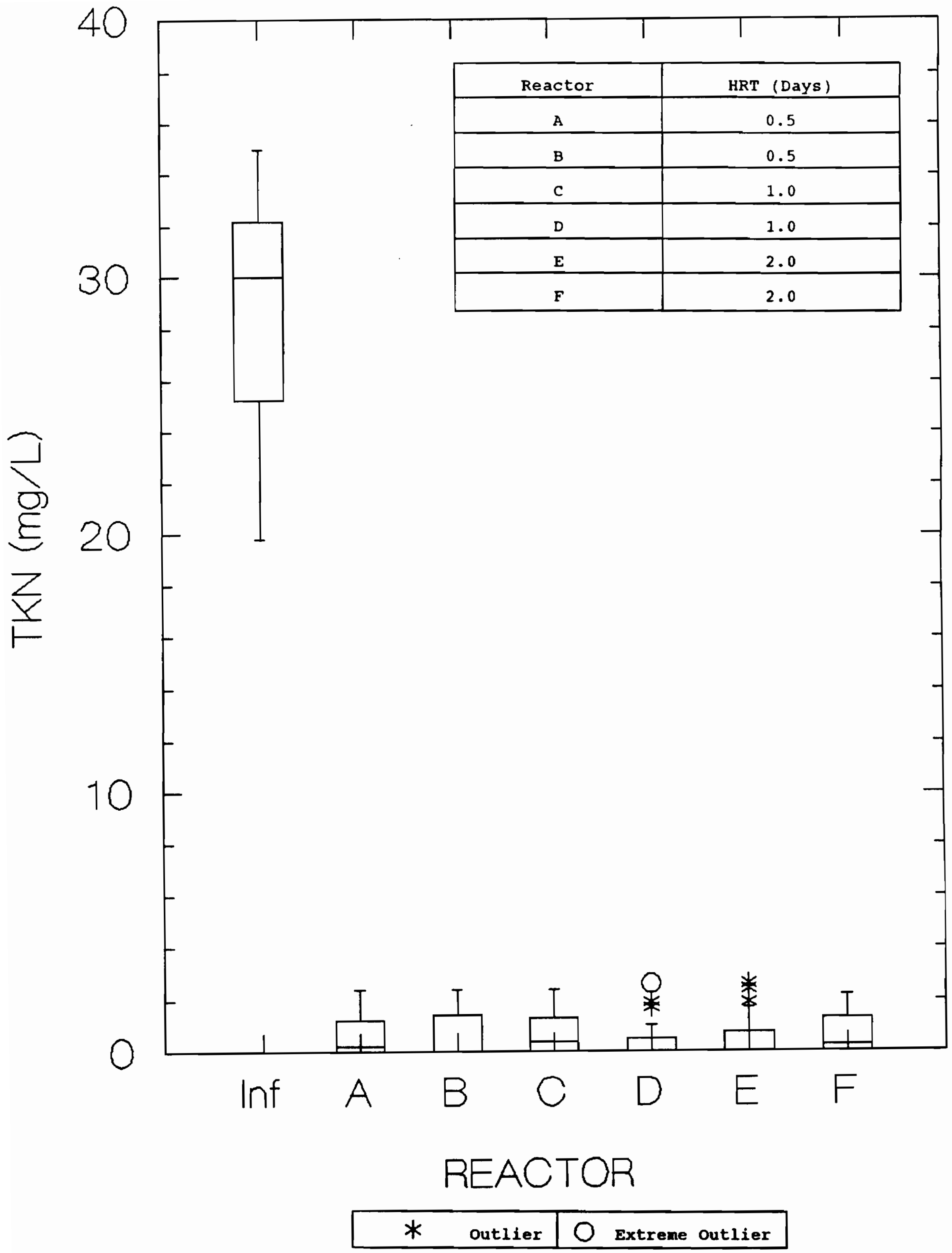
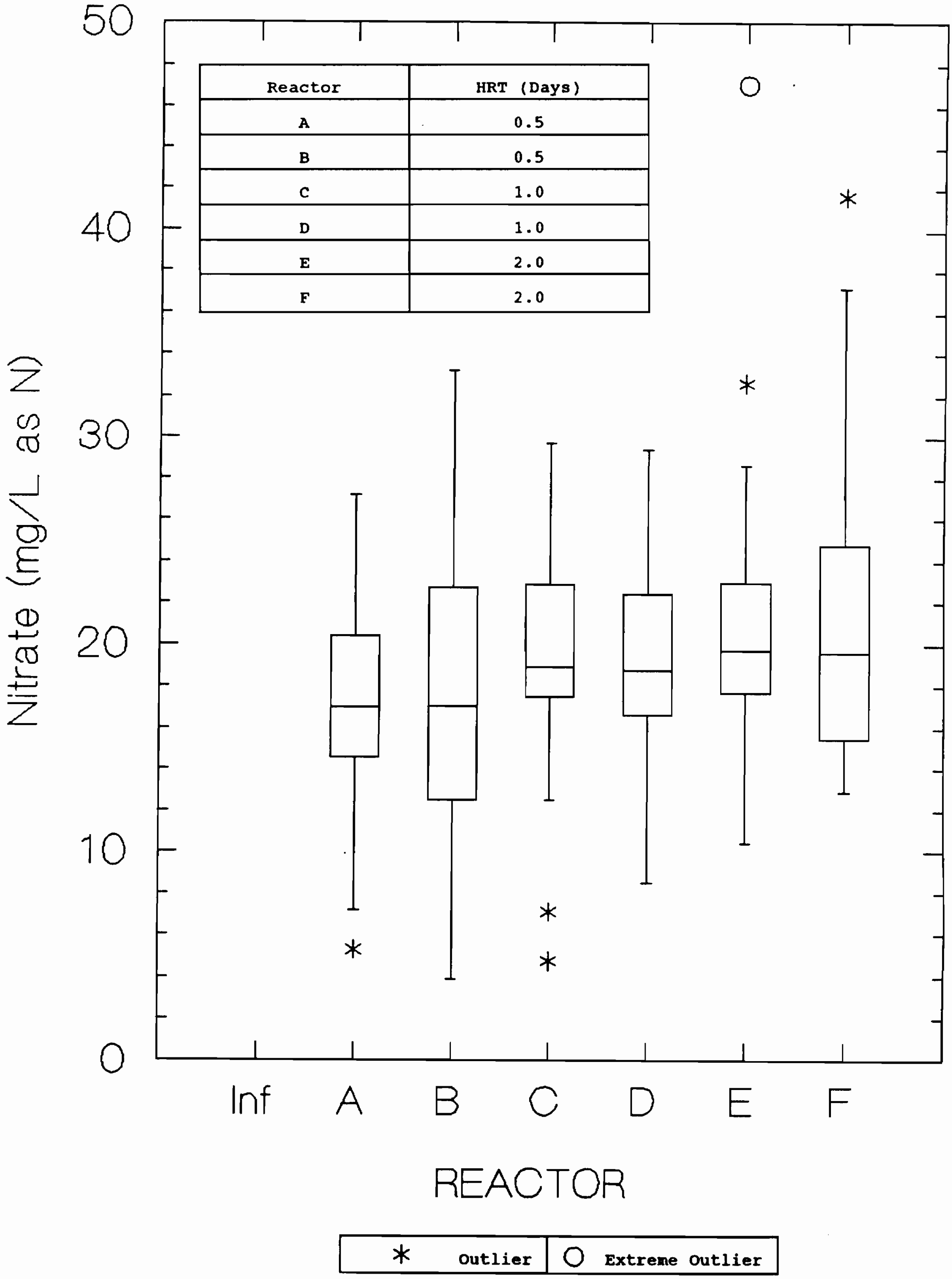


Figure 9. Effluent Soluble Total Kjeldahl Nirtrogen, Days 34 - 228



Reactor	HRT (Days)
A	0.5
B	0.5
C	1.0
D	1.0
E	2.0
F	2.0

Figure 10. **Effluent Soluble Nitrate, Days 34 - 228**

4.2.3 Nitrogen

Effluent TKN and NO_3^--N data are presented in Figures 9 and 10. Nitrification was essentially complete in all reactors. Soluble TKN was consistently below 2 mg/L and was often below 0.5 mg/L. Because of these low TKN concentrations, NH_3-N concentrations were not determined and assumed to be negligible. No NO_2-N was detected in any sample. Effluent nitrogen was in the form of NO_3^-.

4.2.4 Statistical Analysis

The data sets for effluent COD, TSS, TKN and NO_3-N were not all normally distributed or symmetrical, and therefore a nonparametric sign test was used to determine significant differences in system performance (Ryan *et al.*, 1991). Results of the analysis are presented in Table 5.

Though several comparisons between reactors indicated some statistically significant differences, none seemed to be consistently related to HRT. For instance, reactor A (HRT = 0.5 days) produced effluent which was significantly lower in TSS then did reactor D (HRT = 1.0 day). However, since there was no significant difference between reactors B and D, or between reactors A and C, it is unlikely that the difference between reactors A and D was a result of HRT.

Table 5. Results of Nonparametric Signed Rank Test of the Differences Between Reactor Performances[1]

Reactor Variables Compared	COD		TSS		NO_3-N		TKN	
	P Value	η[2]	P Value	η	P Value	η	P Value	η
A - B	0.850	2.0	0.185	-7.0	0.096	1.2	0.774	0.0
A - C	0.052	8.5	0.061	-4.0	**0.008**	**-2.0**	0.289	0.0
A - D	0.572	5.0	**0.043**	**-6.0**	0.144	-1.7	1.000	0.0
A - E	0.071	7.0	1.000	1.0	**0.031**	**-2.6**	0.774	0.0
A - F	0.088	3.0	1.000	1.0	0.332	-2.1	0.581	0.0
B - C	0.265	5.0	0.585	-4.5	**0.001**	**-3.2**	1.000	0.0
B - D	0.701	1.5	0.442	-2.0	0.238	-2.5	0.754	0.0
B - E	0.136	4.5	0.099	8.8	**0.019**	**-2.9**	1.000	0.0
B - F	0.458	7.0	0.572	8.0	0.238	-3.1	0.774	0.0
C - D	**0.043**	**-2.0**	1.000	-0.5	0.238	1.7	1.000	0.0
C - E	0.851	0.0	**0.016**	**11.0**	1.000	0.1	0.508	0.0
C - F	0.839	0.0	0.265	8.0	0.481	0.7	0.774	0.0
D - E	0.572	2.0	**0.008**	**6.5**	**0.049**	**-1.2**	0.289	0.0
D - F	0.185	4.0	0.185	6.0	0.144	-1.1	0.344	0.0
E - F	0.856	-2.0	0.345	-3.0	1.000	0.1	1.000	0.0

[1] **Note: Bold values indicate statistical significance at a 95% confidence level**
[2] Estimated median difference

4.3 Laundry Detergent

During the period in which laundry detergent was added to reactors A, C and F (days 235-284) all reactors continued to perform well. Though there was some foaming in reactors A, E and F during the first two days of detergent feed, it did not present any operational problems, and there was none evident throughout the rest of the study. There were mechanical problems with the secondary clarifier for reactor F, resulting in two effluent samples with very high TSS. The problem was corrected and those data points were disregarded.

4.3.1 Organic Constituents

All of the reactors continued to provide good treatment of COD through the laundry detergent experiment (Figure 11). COD of the influent was more than twice as strong for the detergent reactors (418 mg/L) than it was for the nondetergent reactors (175 mg/L). This was due entirely to the addition of laundry detergent. Effluent COD also tended to be higher in the detergent reactors. Systems A, C and E produced effluent with a combined average COD of 56 mg/L, as compared to 25 mg/L for systems B, D and F. HRT did not affect effluent COD in either the detergent or the nondetergent reactors.

Total and soluble effluent BOD_5 are presented in Table 6. Though total BOD_5 data were variable, soluble BOD_5 was consistently low for all reactors. BOD_5 to COD ratios were also very low.

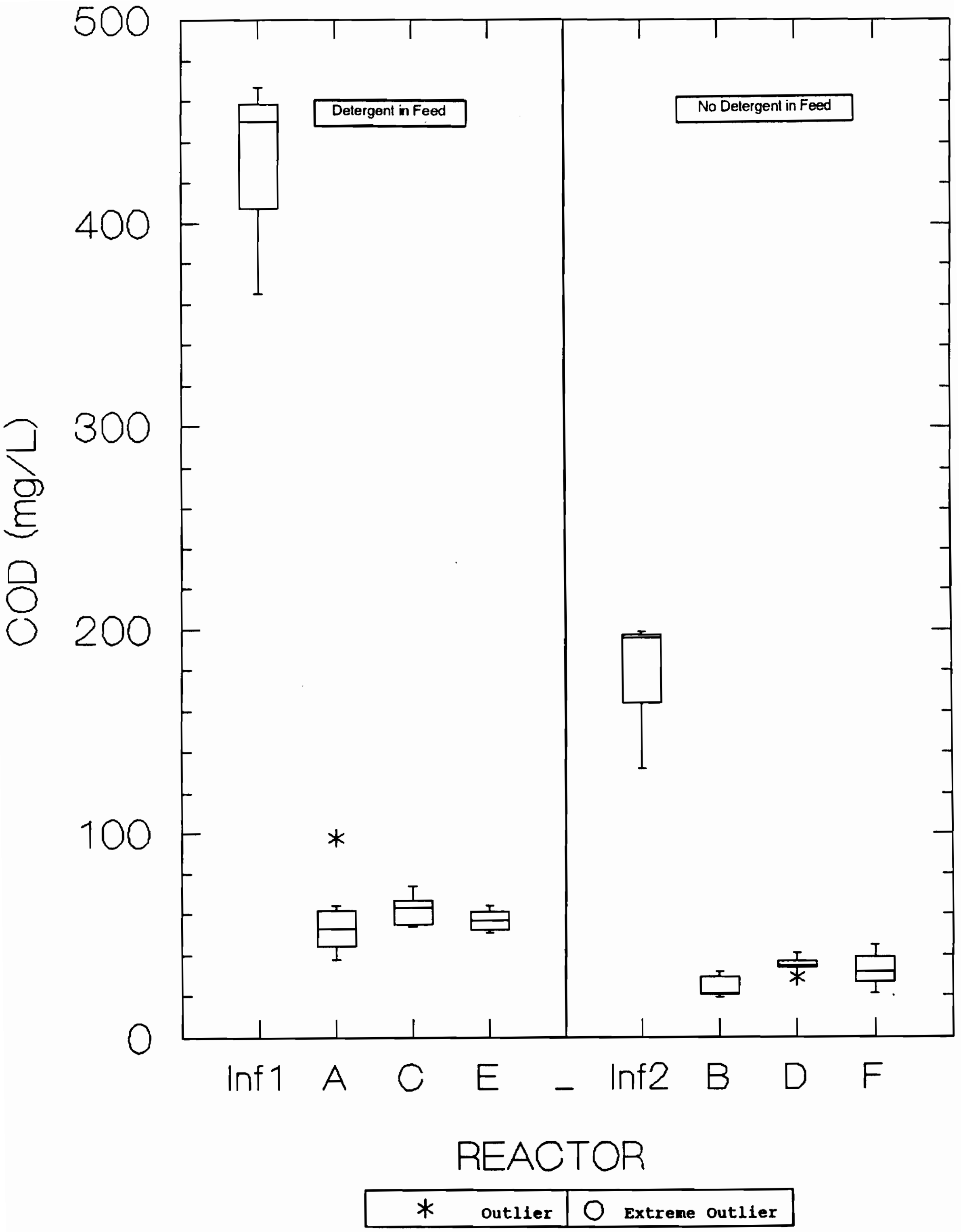


Figure 11. **Effluent Soluble Chemical Oxygen Demand, With and Without Detergent, Days 228 -282**

Table 6. **Effluent BOD_5 and COD During Laundry Detergent Experiment**[1]

Reactor	Total BOD_5 (mean)	Soluble BOD_5 (mean)	Soluble COD (mean)	BOD_5/COD Ratio
A	29	2	57	0.04
B	18	2	23	0.09
C	16	3	62	0.05
D	9	2	35	0.06
E	13	3	56	0.06
F	53	4	30	0.14

[1] Detergent added to reactors A, C and E

4.3.2 Suspended Solids

Effluent TSS was not effected by either detergent or HRT (Figure 12). Though the settling was generally good, none of the systems would have been consistently in compliance with state regulations (30 mg/L). Apparent high variability in the TSS data for reactor F was the result of mechanical problems with the secondary clarifier.

As with the HRT experiment, the MLVSS was higher in the reactors with the lower HRT (Figure 13). MLVSS was also higher in the reactors that had detergent feed than it was in those that did not. This difference increased steadily throughout the experiment (Figure 14).

4.3.3 Nitrogen

Addition of detergent to the wastewater had no effect on

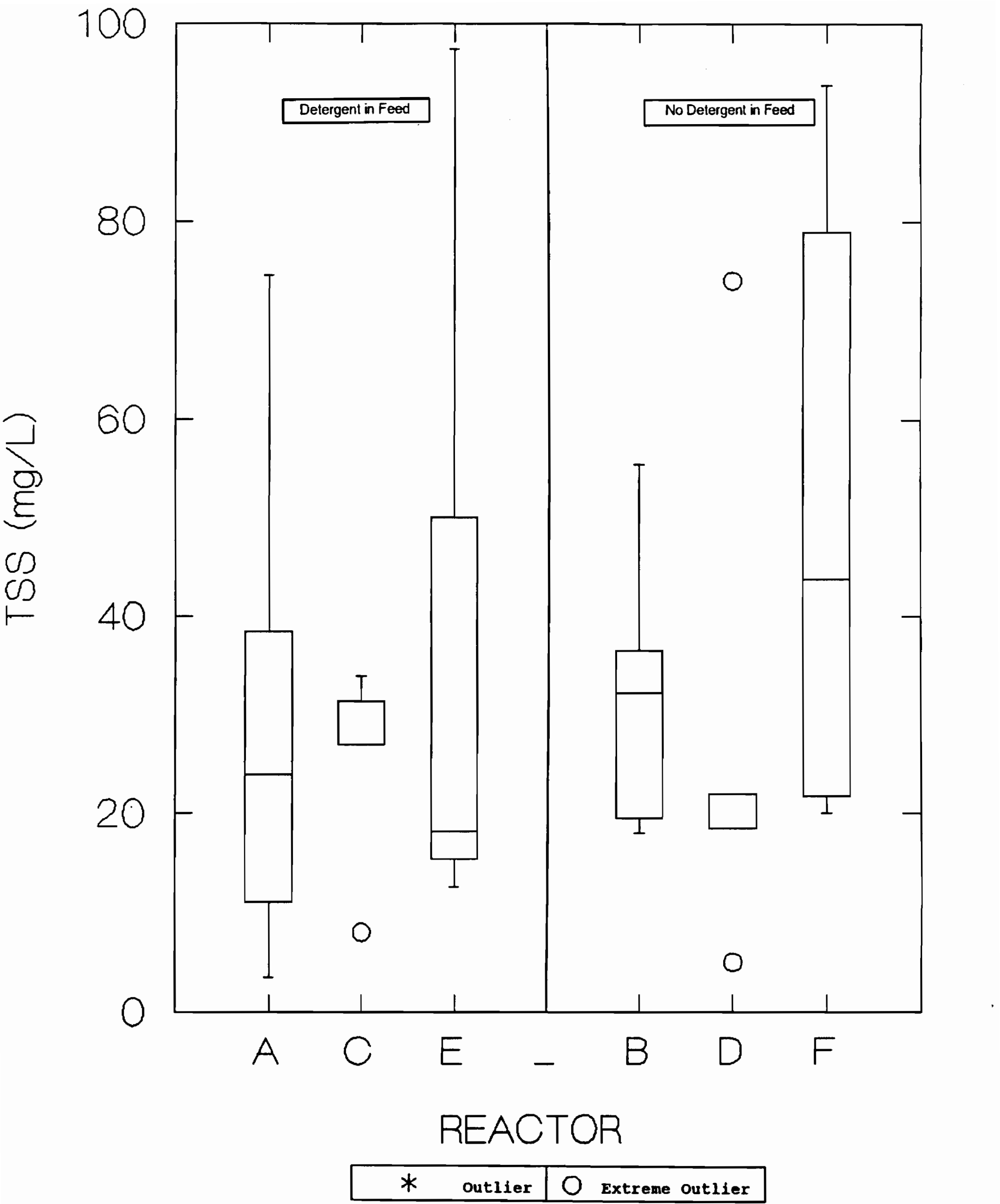


Figure 12. **Effluent Total Suspended Solids, With and Without Detergent, Days 228 - 282**

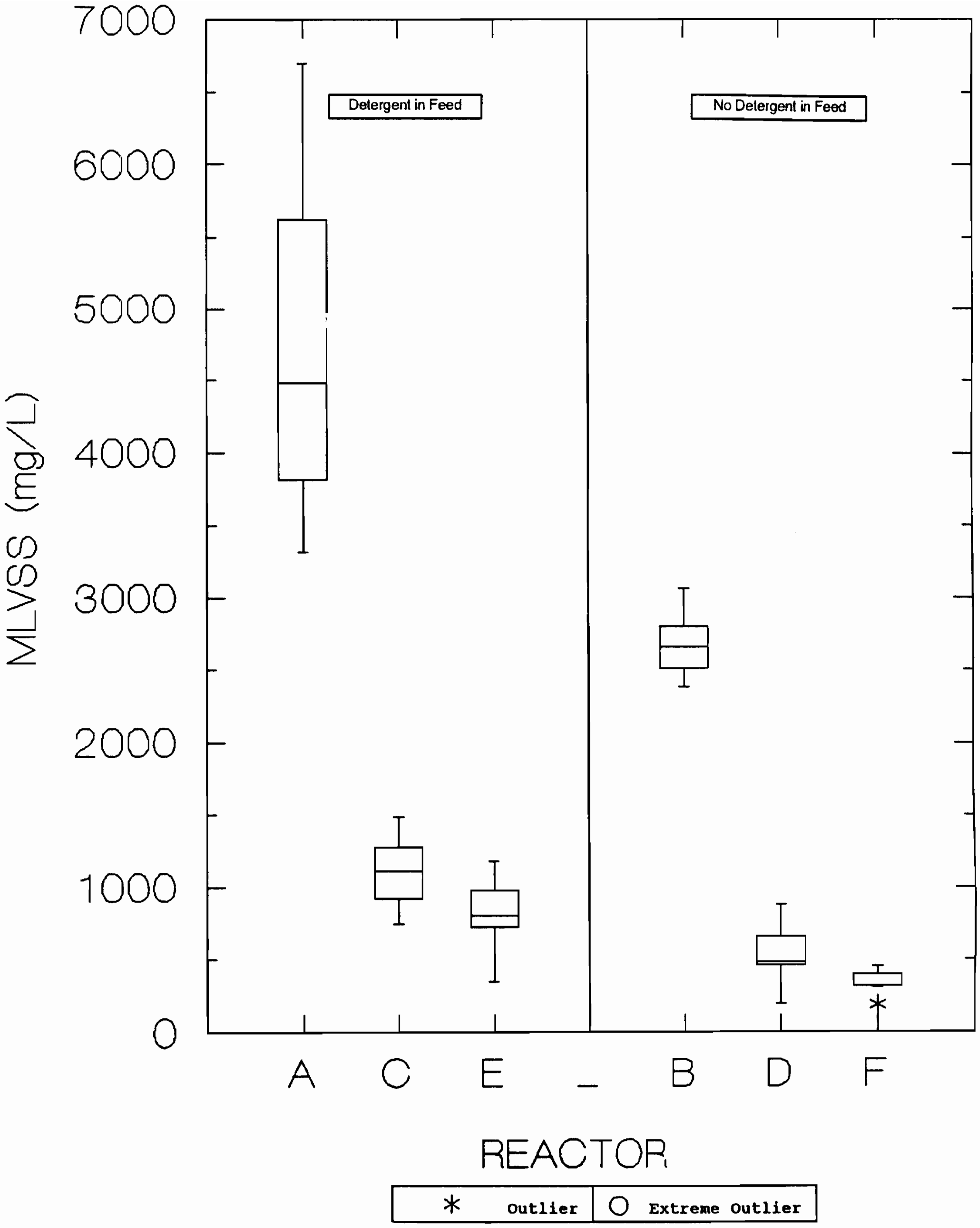


Figure 13. **Mixed Liquor Volatile Suspended Solids, With and Without Detergent, Days 228 - 282**

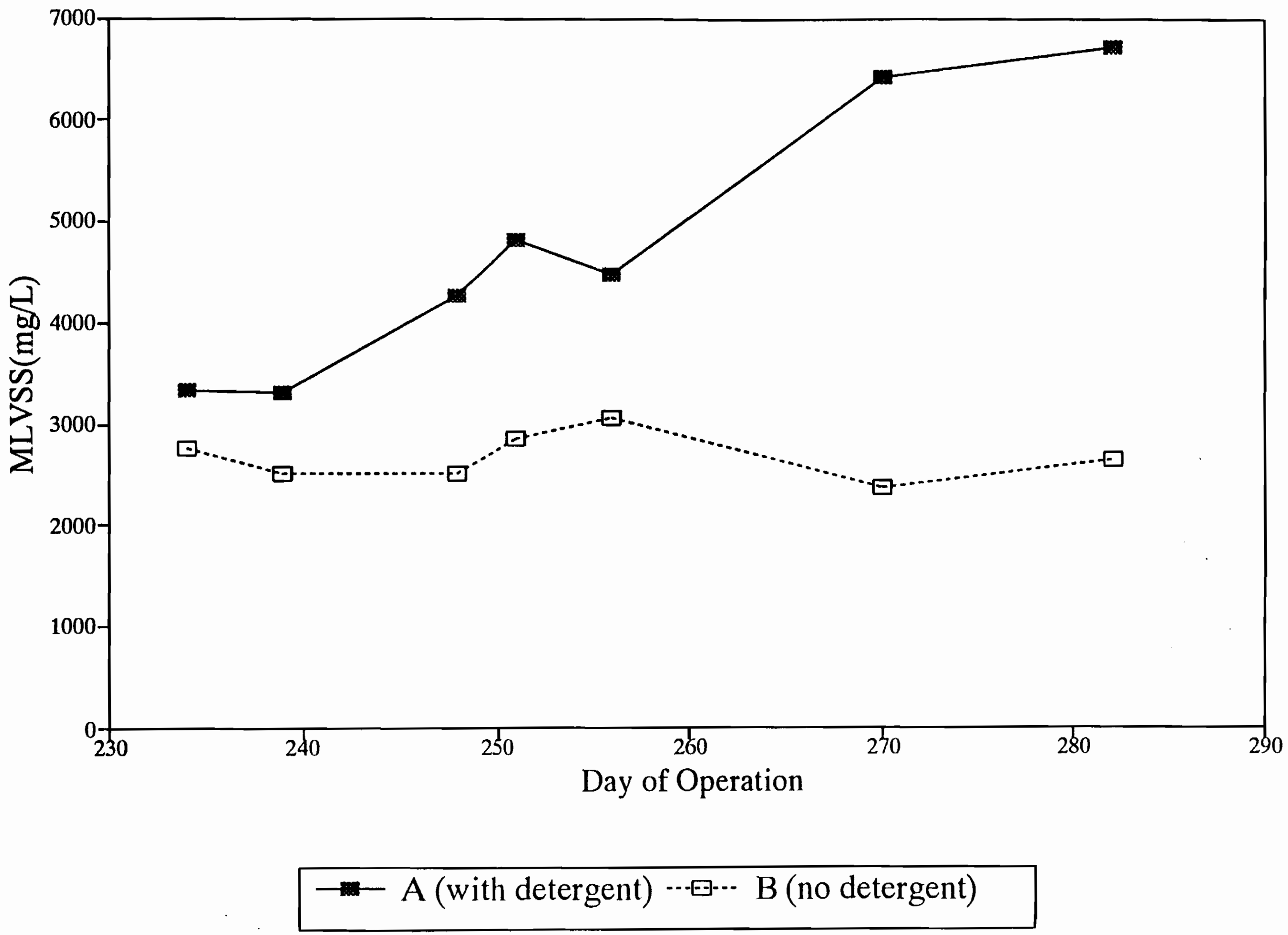


Figure 14. Mixed Liquor Volatile Suspended Solids, Reactors A and B, Days 234 - 282

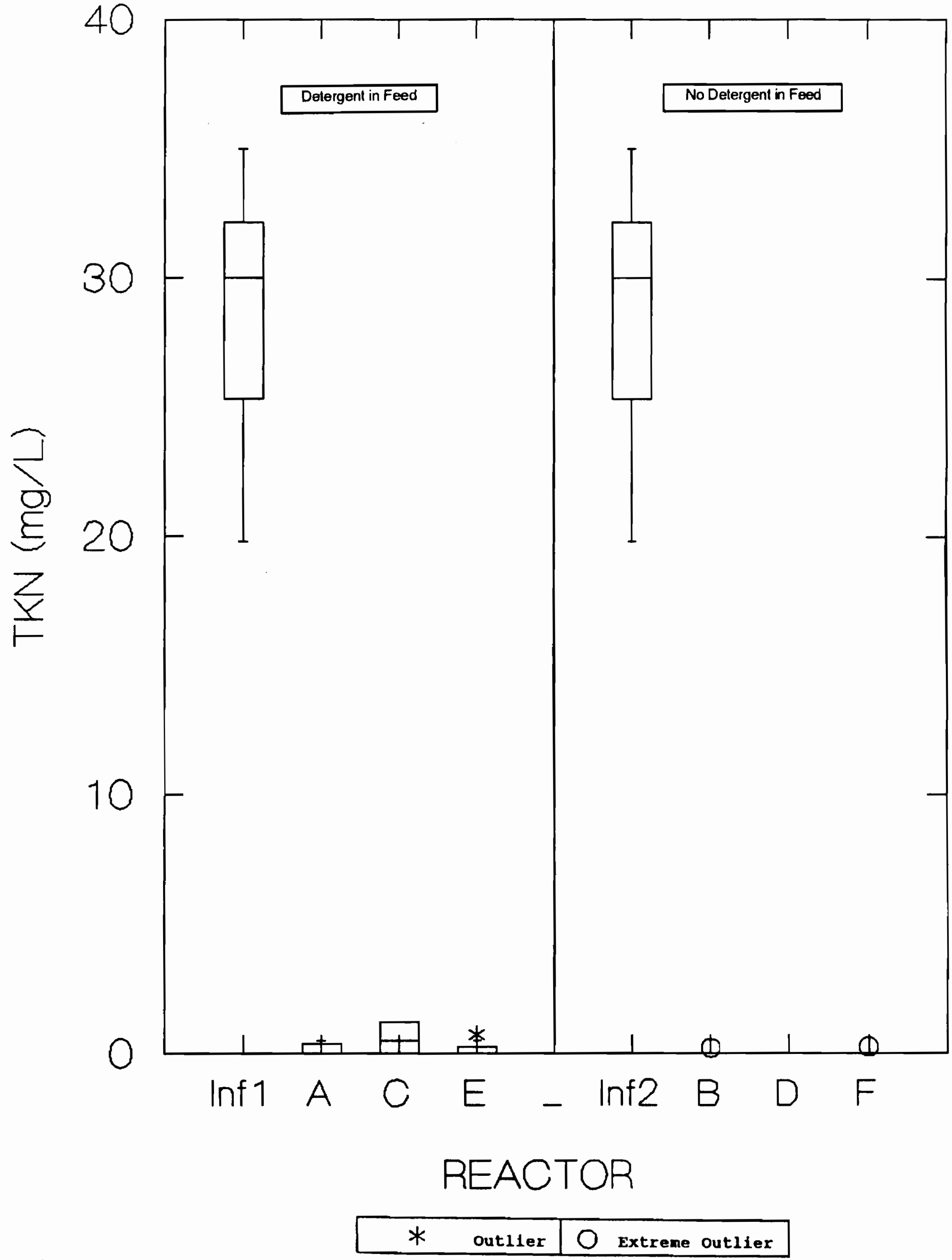


Figure 15. **Effluent Soluble Total Kjeldahl Nitrogen, With and Without Detergent, Days 228 - 282**

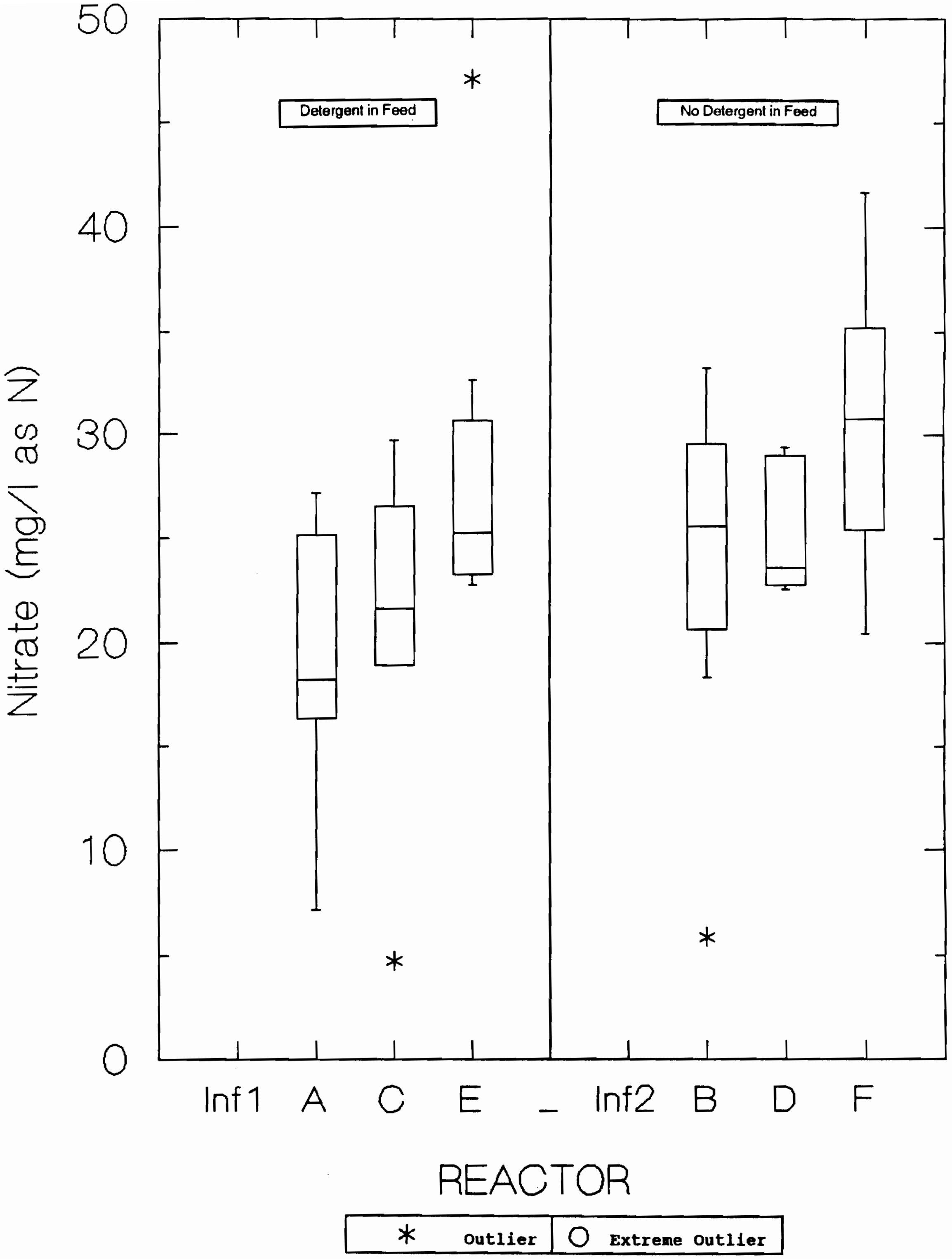


Figure 16. Effluent Soluble Nitrate, With and Without Detergent, Days 228 - 282

nitrification (Figures 15 and 16). TKN values were consistently low for all reactors. No NO_2-N was detected. All effluent nitrogen was in the form of NO_3^-.

Effluent NO_3^--N concentrations were lower for reactors with lower HRT. Additionally, reactors with detergent produced lower levels of NO_3^--N then did the reactors with no detergent.

4.4 Field Experiment: Flow Equalization

The flow equalization system performed very well, mechanically. On several occasions a clogged line reduced flow from the equalization tank to the primary clarifier. On each occasion the line was cleared within a few hours. It is unlikely that this would have had any effect on overall system operations.

There were no mechanical problems with the aerobic package treatment system. All of the effluent from the secondary clarifier was passing through the synthetic filter into the pump chamber, rather than bypassing the filter as had been noted by Kellam *et al.*, 1992. However, the filter, and all other surfaces within the secondary clarifier, were caked with organic solids, which could cause clogging in the future.

The results from the flow equalization field study are presented in Table 7. The flow equalized system was in compliance with state regulations in terms of TSS for 11 out of 12 samples. BOD_5 values were not as good, averaging 33 mg/L. Almost all of the nitrogen in the effluent was in the form of NH_3. The system was providing complete ammonification, but no nitrification.

Generally, the results from the system with flow equalization appeared to be slightly better than those from the same system without flow equalization. Average effluent BOD_5 was lower for the flow equalized system (33 mg/L compared to 60 mg/L) as was effluent TSS (18 mg/L compared to 30 mg/L).

These results may be misleading, however. The average for the flow equalized system is based on 12 samples taken on 4 days (Appendix). The average for the system without equalization was based on 9 samples taken on 3 days (Kellam *et al.*, 1992; Appendix). Both data sets show very little variation between samples taken on the same day (Figures 17, 18, 19 and 20). In effect, samples taken on the same day can be treated as a single data point for the purposes of comparing overall system performance. Table 7, therefore represents the comparison of an average based on 4 data points to one based on 3. The apparent differences between the effluent quality with and without flow equalization is the result of a single day of poor effluent during the previous study (Kellam *et al.*, 1992).

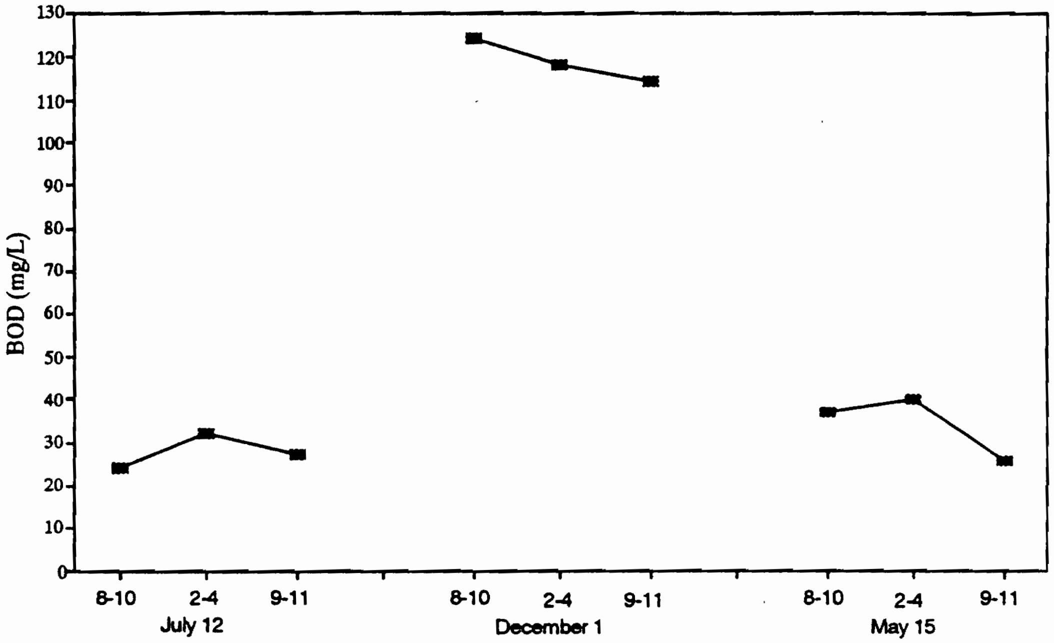


Figure 17. Effluent 5-Day Biochemical Oxygen Demand from Field System Without Attenuation of Laundry Effluent (Kellam *et al.*, 1993)

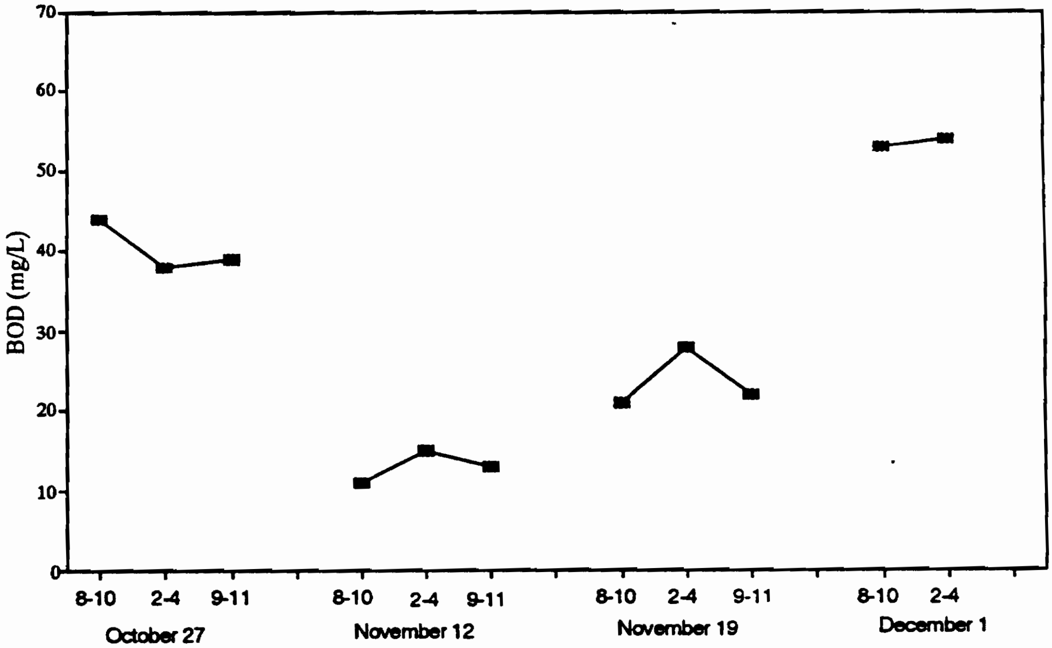


Figure 18. Effluent 5-Day Biochemical Oxygen Demand from Field System With Attenuation of Laundry Effluent

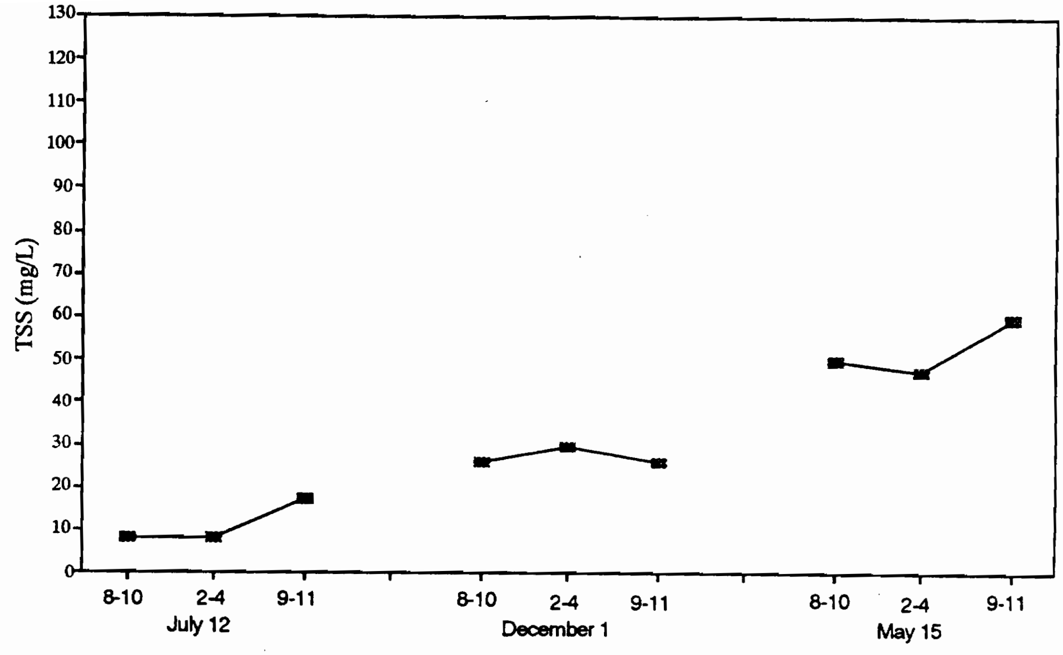


Figure 19. Effluent Total Suspended Solids from Field System Without Attenuation of Laundry Effluent (Kellam *et al.*, 1993)

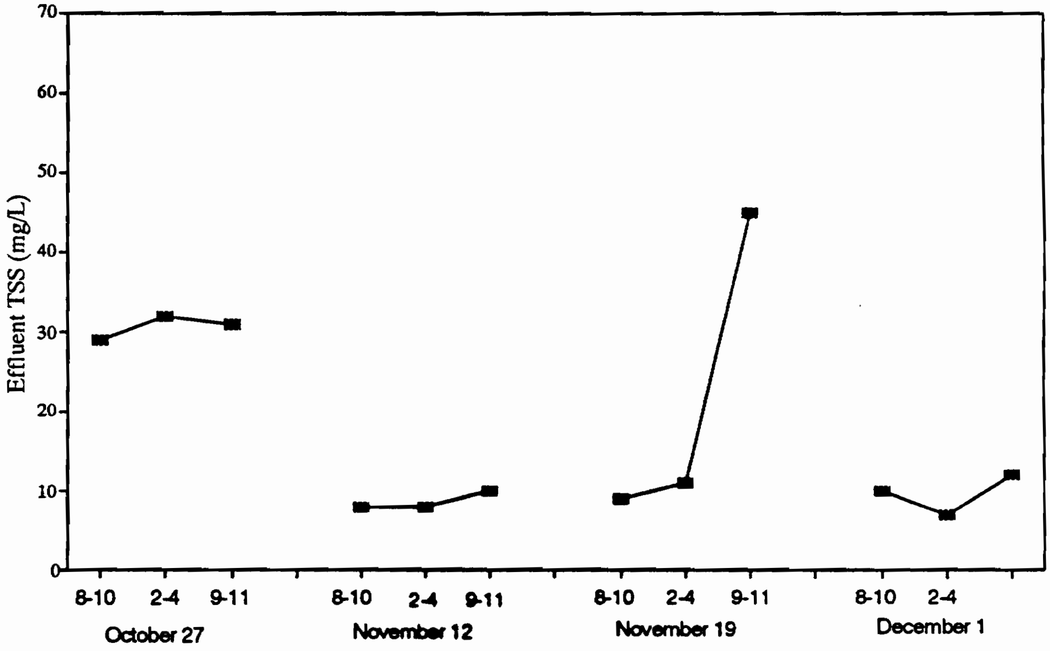


Figure 20. Effluent Total Suspended Solids from Field System With Attenuation of Laundry Effluent

Table 7. Comparison of Effluent System Performance with Equalization (This Study) and without Equalization (Previous Study)

	This Study		Previous Study at This Site[1]		
	Average	Standard Deviation	Average	Standard Deviation	Regulatory Limits
TSS	18	13	30	19	30
VSS	12	8	29	10	
MLSS	35	22	152	76	
MLVSS	24	16	100	53	
Total					
BOD_5	33	6	60	44	30
COD	112	37	148	90	
TKN	30	9	19	14	
TP	5.7	1.3	6.4	1.6	
Soluble					
BOD_5	10	3			
COD	74	15			
TKN	23	3			
NH_3-N	22	8	10.4	6.2	
NO_3^--N	0.22	0.14	3.2	4.3	
TP	5.4	1.3			
OP	4.7	1.8	2.9	2.2	
pH	7.2	6.7-7.8[2]	7.2	6.8-7.5[2]	

[1] Kellam *et al.*, 1992

[2] Range

Average D.O. concentrations for this study are presented in Figure 21. Though the aeration chamber had relatively high levels of D.O., the oxygen concentration was much lower in the secondary clarifier and the pump tank. Effluent D. O. from the pump tank, and therefore from the system averaged 1.2 mg/L, far below the Virginia regulatory minimum of 5 mg/L (Commonwealth of Virginia, 1992).

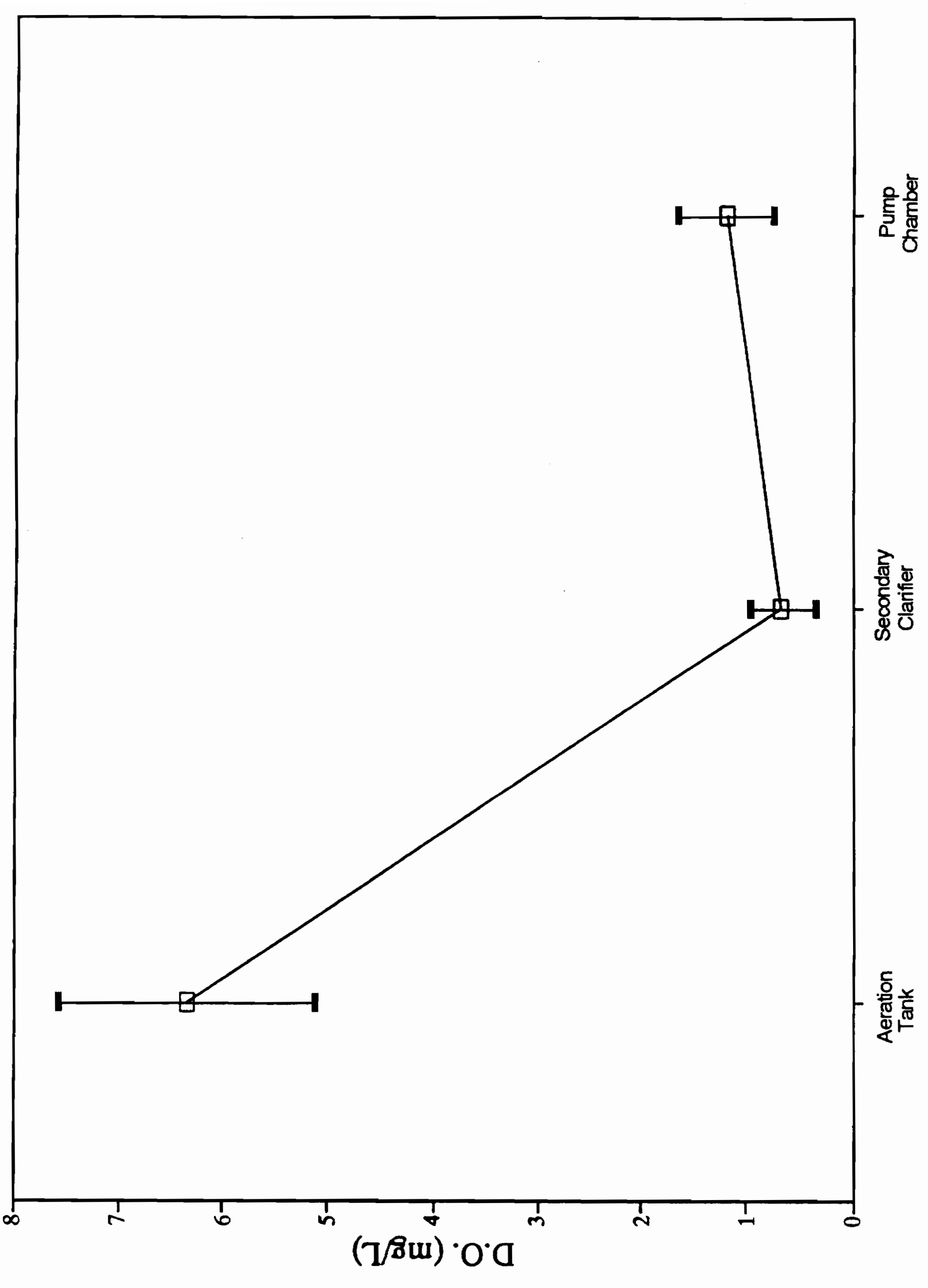


Figure 21. Dissolved Oxygen Concentrations in Aerobic Package Treatment System

5. Discussion

5.1 Hydraulic Retention Time

5.1.1 System Performance

The most important implication of this experiment was that extended aeration systems with steady influent flow perform as well at low HRTs (0.5 days) as they do at higher HRTs (2 days). All systems performed well in terms of treating BOD_5 and NH_3, as would be expected in systems with such long sludge ages, but performed poorly in terms of effluent TSS. Generally the systems would have complied with the Virginia effluent requirement for BOD_5 (30 mg/L), but would not have complied with the state TSS limit (30 mg/L). On the few occasions when one of the systems produced effluent BOD_5 concentrations greater than 30 mg/L, the excess was attributable to high effluent TSS.

Though soluble BOD_5 was not tested during the HRT experiment, it was measured during the laundry detergent experiment. The control reactors (B, D and F) were operated under the same conditions during the detergent experiment as they had been during the HRT experiment. Average soluble and total effluent BOD_5 are presented in Figure 22 for the 2 days on which both tests were performed. About 80 - 90 percent of the effluent total BOD_5 was attributable to suspended solids

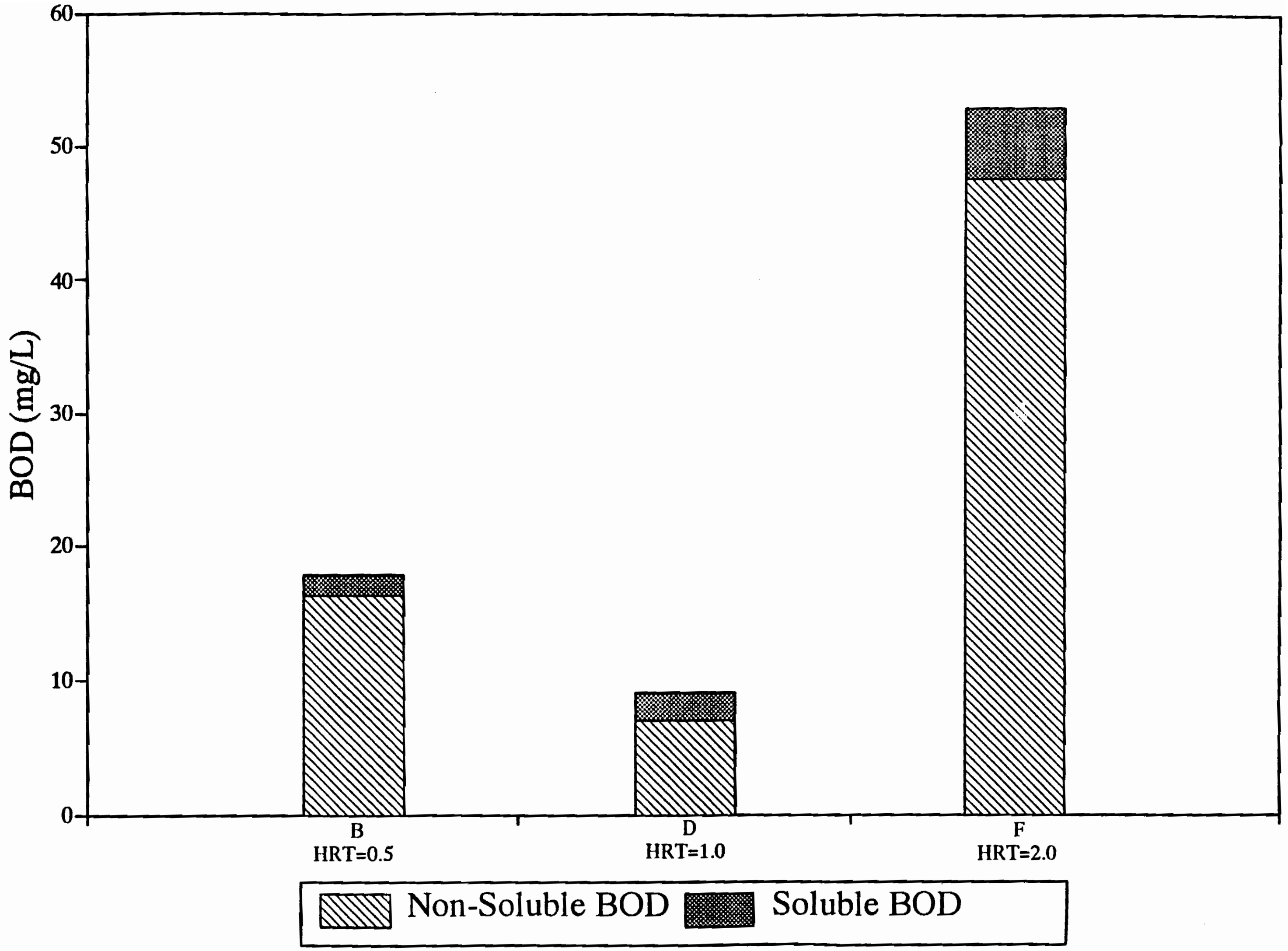


Figure 22. Soluble and Non-soluble 5-Day Biochemical Oxygen Demand, Reactors B, D and F, Days 228 - 282

(Table 6).

The laboratory systems depended on secondary clarification for solids removal. In most full-scale systems some other device is used in addition to secondary clarification to ensure low levels of effluent TSS. The package treatment system studied during the flow equalization portion of this project was designed with a synthetic filter to remove unsettled solids. In other systems effluent is passed through a final sand filter. Had the effluent from the laboratory systems been passed through some type of filter, it would have been well within Virginia state effluent requirements for both BOD_5 and TSS.

Though no conclusions can be made as to how these systems would have handled variations in hydraulic loading, all systems responded well to variations in feed strength. MLVSS increased and decreased in response to changing feed strength, but no change was discernable in effluent quality. MLVSS and effluent COD from system C are shown in Figure 23. Though there was a high variability in MLVSS, effluent COD generally remained between 20 and 50 mg/L. This was typical of all systems.

Figure 24 is a comparison of influent and effluent COD for all systems. Despite a significant range of influent strength, effluent COD remained steady throughout the study.

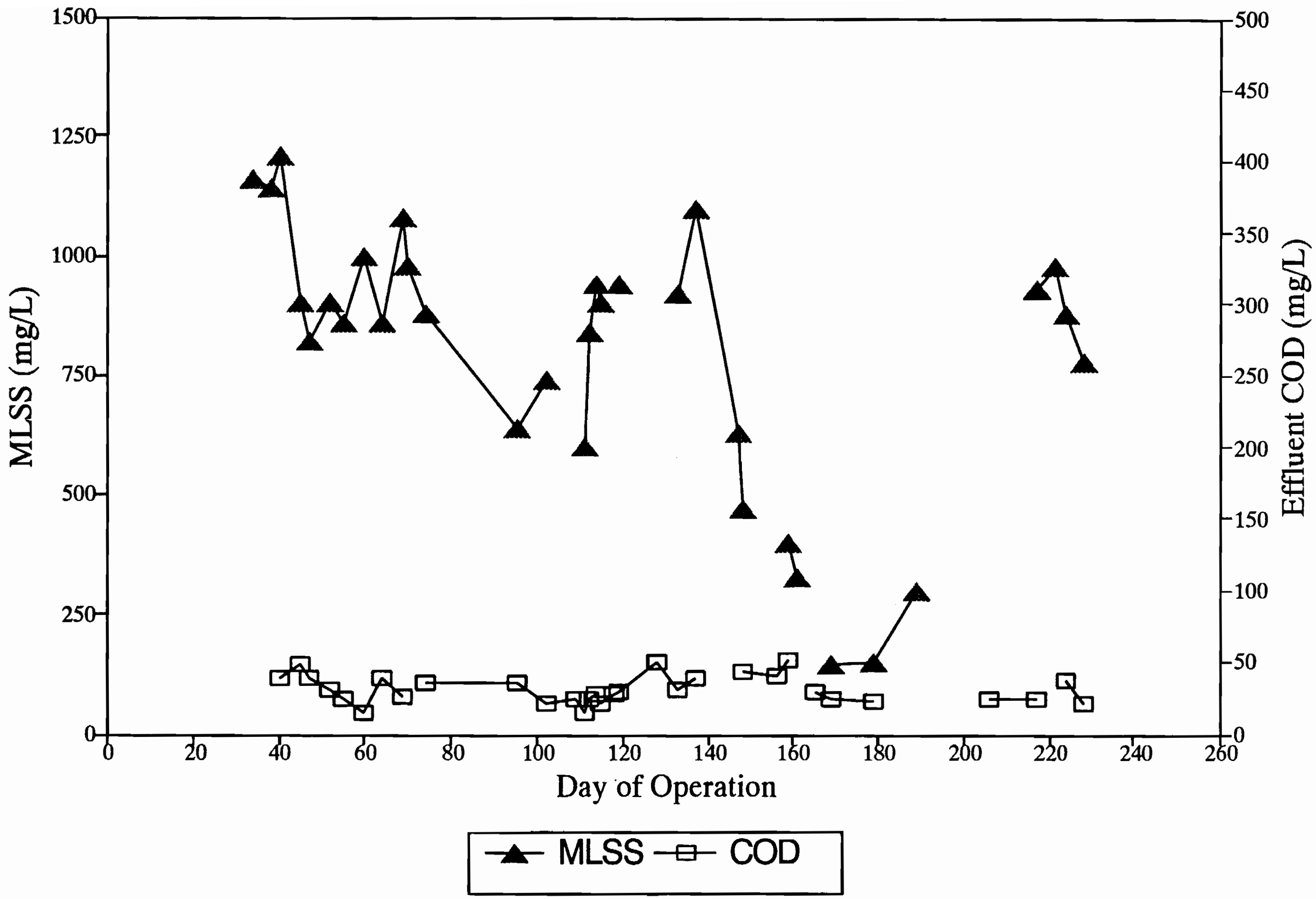


Figure 23. Mixed Liquor Suspended Solids and Effluent Soluble Chemical Oxygen Demand, Reactor C, Days 34 - 228

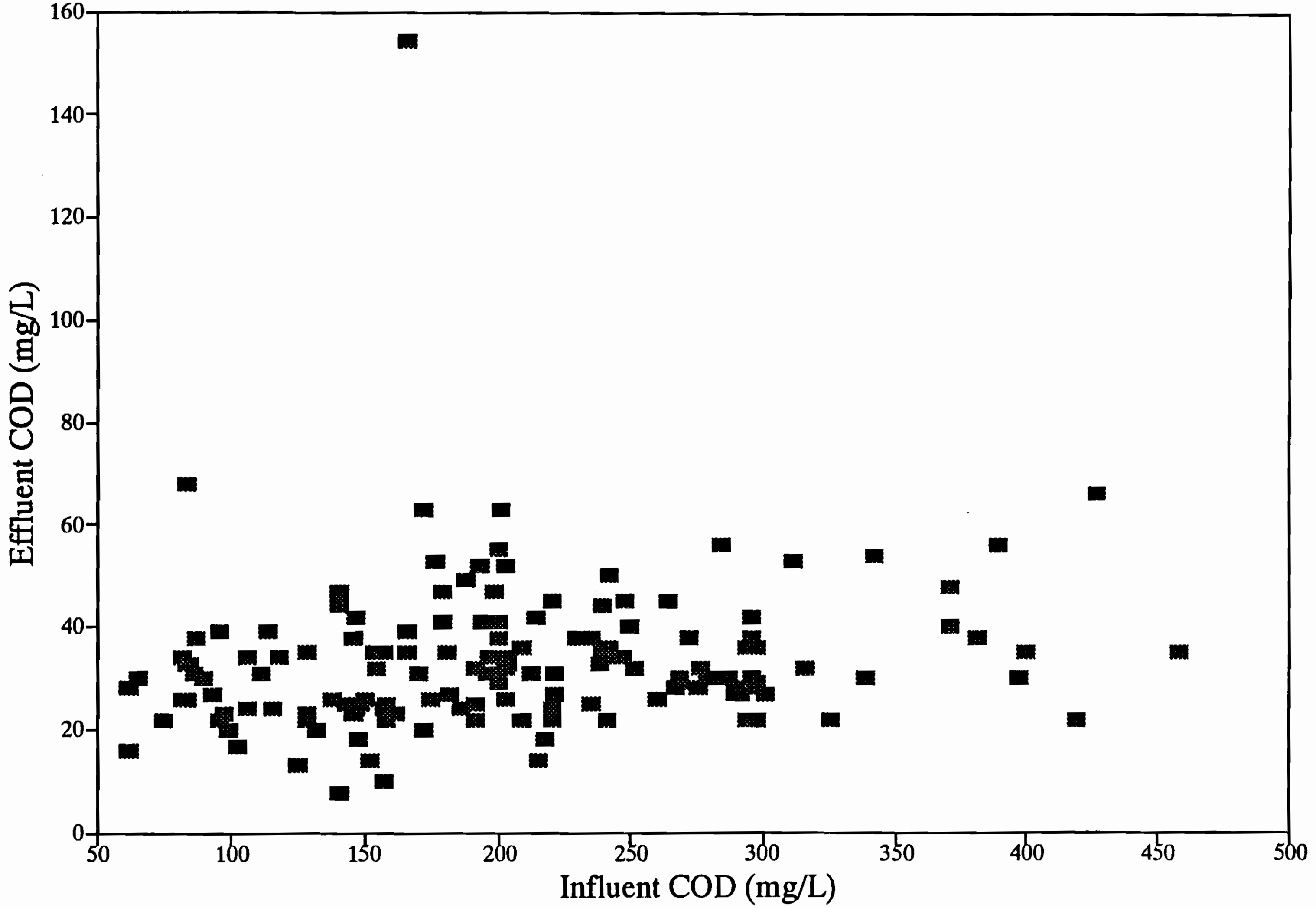


Figure 24. Influent and Effluent Chemical Oxygen Demand, All Reactors, Days 34 - 228

5.1.2 Comparison to Design Models

F/M ratios are presented in Figure 25. All six systems were operating at a higher F/M ratio than was suggested by Metcalf and Eddy (1991). This would indicate that the MLVSS in the systems did not reach as high a concentration as would be recommended for good system operation. Despite this, all systems performed well. The average values for systems with the same HRT were 0.195, 0.172 and 0.185 for HRTs of 0.5, 1.0 and 2.0 days, respectively. It did not appear that HRT had any affect on F/M ratios.

Predicted levels of MLVSS are compared to measured values in Figure 26 and 27. In Figure 26, commonly reported values for kinetic parameters ($Y=0.4$, $b=0.06$ days^{-1}; COD basis) were used in theoretical models proposed by Eckenfelder (1970) and Lawrence and McCarty (1970). Measured values of effluent COD were used rather than values predicted by the models. The model presented by Lawrence and McCarty did a much better job of predicting MLVSS. This would be expected, since the Eckenfelder model did not take into account solids lost as effluent TSS.

Several adjustments to the kinetic parameters were considered. The first was based on previously published observations that the autoxidation rate (b) decreases with increased sludge age. The standard value for activated sludge was modified using the relationship presented by Goodman and

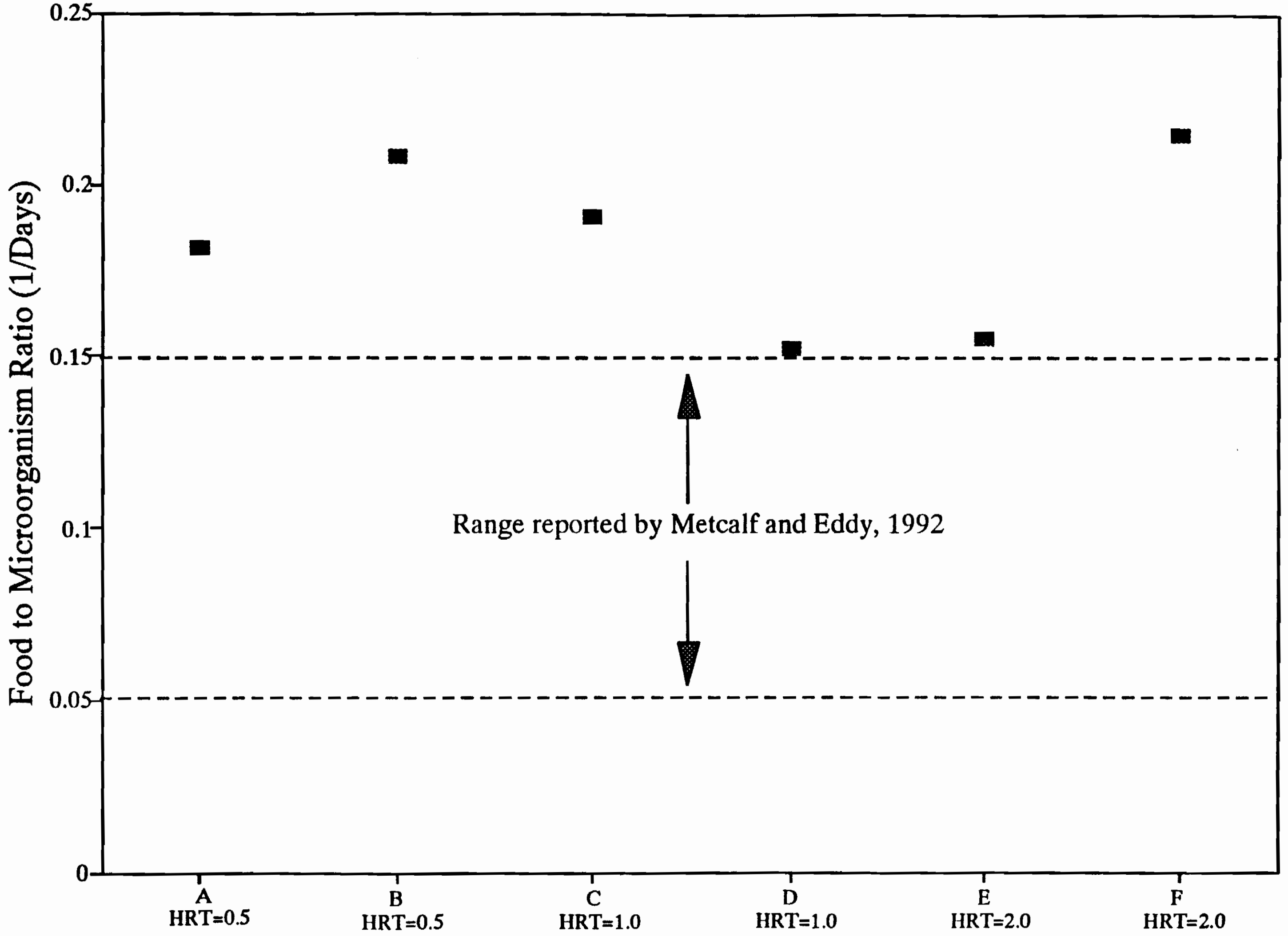


Figure 25. Average Food to Microorganism Ratios, Days 34 - 228, as Compared to Typical Range Reported by Metcalf and Eddy, 1992

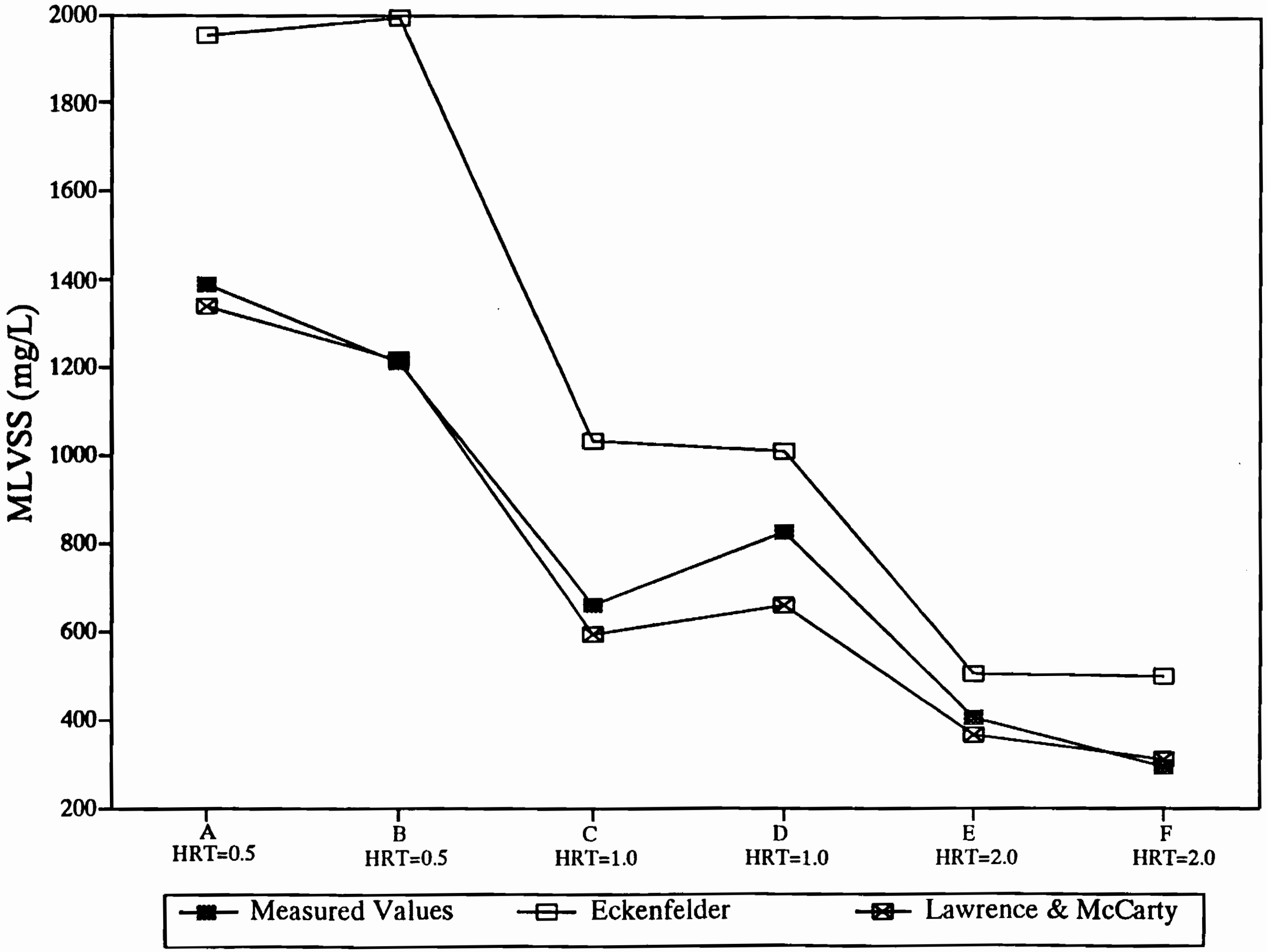


Figure 26. Actual Values of Mixed Liquor Volatile Suspended Solids as Compared to Theoretical Values ($Y=0.4$, $b= 0.06$ $days^{-1}$)

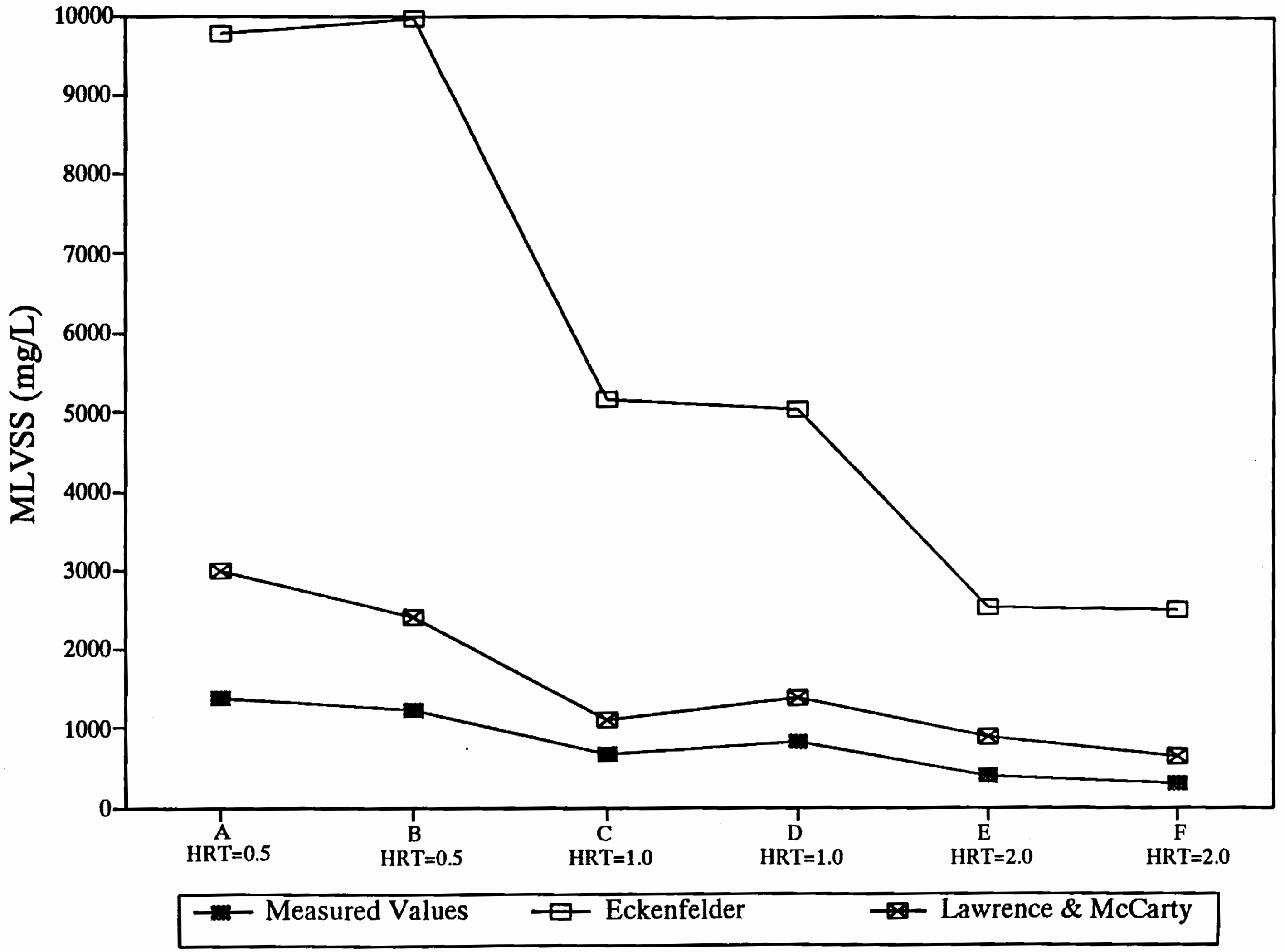


Figure 27. Actual Values of Mixed Liquor Volatile Suspended Solids as Compared to Theoretical Values (Y = 0.4, b = 0.012 days^{-1})

Englande (1974):

$$b_{\theta_c}=b(0.75^{Ln\theta_c/Ln2})$$

The new value of b yielded Figure 27. Adjusting b by itself worsens the performance of both models. The Eckenfelder model was especially sensitive to changes in the autoxidation rate. The predicted MLVSS were, on average, 2.2 times the measured MLVSS using the reported value of b, but were 11 times the measured MLVSS using the adjusted value of b.

The second adjustment considered was to calculate all four parameters (k, K_s, Y and b) based upon the measured data and the Lawrence and McCarty model.

To accomplish this, untreated influent COD needed to be distinguished from nondegradable COD. When specific substrate utilization (mg COD metabolized per mg MLVSS per day) was plotted against effluent COD, 4 reactors (B, D, E and F) yielded a relatively straight line (Figure 28). The points for the other two reactors (A and C) were disregarded. Though there is no independent reason for concluding that these two reactors were functioning differently then the other four, Figure 28 indicates that there was a difference. In order to complete the exercise of estimating kinetic parameters, the

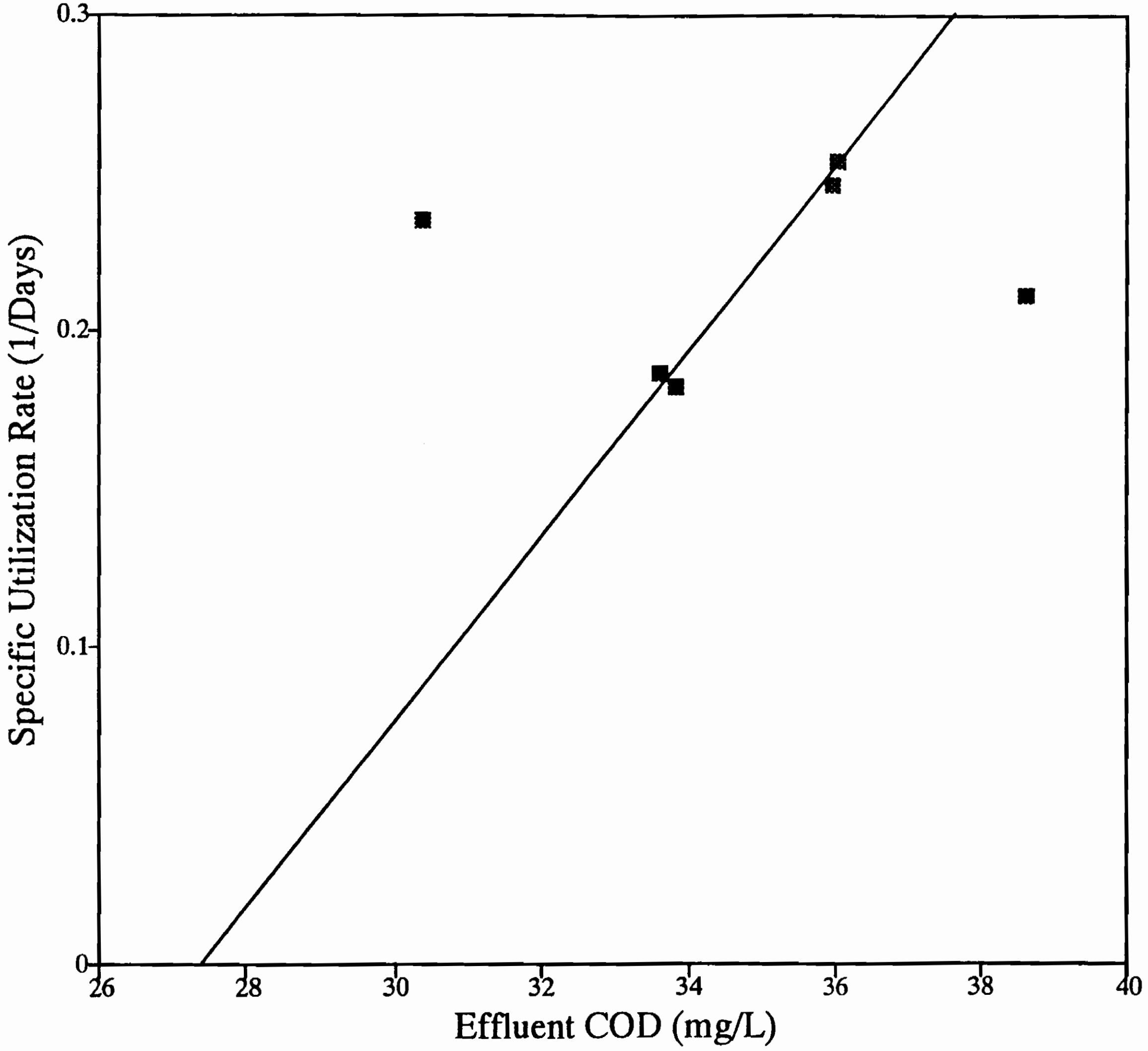


Figure 28. Specific Utilization Rate Versus Effluent Soluble Chemical Oxygen Demand

assumption was made that systems B, D, E and F represent the average system performance and that systems A and C are deviations from the norm.

Figure 28 was based on the theory that, at these concentrations, substrate utilization is influenced by substrate concentration. Complete mix reactors with a higher substrate concentration (reflected by effluent COD) would have a higher rate of substrate utilization. Additionally, this plot should show a theoretical point at which substrate utilization ceases. This point represents the amount of nondegradable COD in the effluent. If COD levels in the reactor drop to this point all biological activity would cease, because remaining COD would be unavailable to microorganisms.

This theory presupposes that residual nondegradable COD was the same in each biological reactor; that it was a characteristic of the influent. This may not be the case for these systems. In Figure 28, the two outlying points represent data for systems A and C. Reactor A had a greater level of COD in solution, but that increased concentration did not result in an increase in the rate of substrate utilization. Reactor C had less COD in solution, yet it was still able to maintain the same substrate utilization rate as the other systems. This may indicate a difference in the proportion of COD in each systems available for microbial degradation. Whether this was a result of differences in the rate of production of nondegradable compounds by the microbial populations, or some other factor can only be a matter of speculation. Previous studies of activated sludge would suggest that influent COD and sludge age could both affect refractory COD (Daigger and Grady, 1977a,b; Chudoba, 1983; Gaudy and Blachly, 1985; Hao and Lau, 1988). Attempts to correlate effluent COD with influent COD, sludge age or MLVSS were unsuccessful (see Figures 24, A.1, A.2 and A.3). There was no relationship evident between effluent COD and any of the other factors.

In order to obtain kinetic parameters, data from systems A and C were disregarded and calculations were based on systems B, D, E and F. Estimates of kinetic parameters were based on linearizations of the Lawrence and McCarty model. The predictions of the model using the new kinetic parameters are plotted in Figure 29. Though the model seems to have done well at predicting system performance, the kinetic parameters themselves raise questions about the validity of the exercise. Table 8 presents the calculated kinetic values along with commonly reported values for activated sludge. The value for b was actually calculated to be negative, but was very close to 0. Though some differences between these systems and standard activated sludge might be expected, the calculated values seemed to be unreasonable. This was probably the

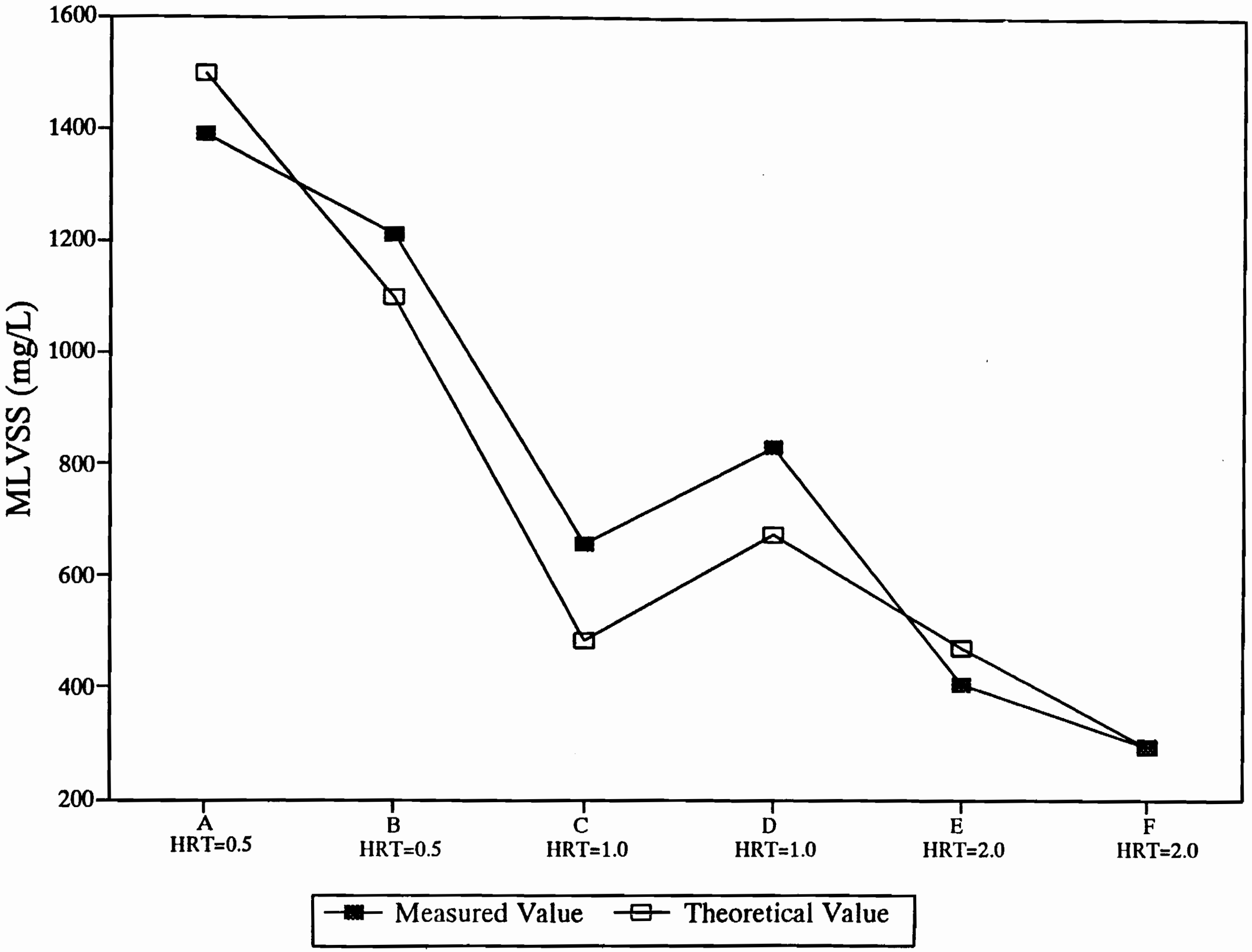


Figure 29. Actual Values of Mixed Liquor Volatile Suspended Solids as Compared to Theoretical Values Predicted by Lawrence and McCarty, 1970, ($Y=0.14$, $b=0$ days^{-1})

result of all systems running at fairly similar sludge ages. Good estimations of kinetic parameters require data from systems operating at a wide range of sludge ages.

Table 8. Comparison of Calculated and Reported Kinetic Parameters

Parameter	Calculated Values	Reported by Benefield and Randall (1980)	Calculation Based on Goodman and Englande (1974)
k	8.8	6-8	
K_s	298	25-100	
Y	0.14	0.35-0.45	
b	≈0	0.05-0.1	0.012-0.024

5.1.3 Concerns with Nitrogen Data

There seemed to be a discrepancy between influent and effluent data, especially TKN and NO_3^--N. Figures 9 and 10 seems to show a net loss in total nitrogen (TKN plus NO_3^--N) between influent and effluent in all six reactors. Assuming that NO_3^--N concentrations in the raw waste were negligible, influent total nitrogen (TN) averaged 29 mg/L. Effluent TN for all six reactors averaged 20 mg/L. It is unclear as to whether this represents a true loss in nitrogen, or a discrepancy between influent and effluent data.

Analysis of raw waste characteristics was performed on 12 samples taken over a one month period (days 39-70 of system operation). These were grab samples taken from carboys of raw feed immediately after they had been brought in from the field site. The only influent parameter to be monitored through the remainder of the study was COD. Unfortunately, the data from days 39-70 do not indicate that there was any correlation between COD and TKN. Conclusions about TN concentrations cannot be made by observing variations in feed strength in terms of COD.

The discrepancy in nitrogen data may have been the result of the raw feed having lower TN concentration during the period in which effluent nitrogen data were taken (days 111 - 228) then during the period when influent data was taken (days 39-70).

Another explanation may have been a loss of NH_3-N to the atmosphere during aeration. Though the pH of the systems was relatively low for NH_3-N stripping (less then 8.5), the long HRTs may have allowed for some volatilization. However, if this had been the most important mechanism for NH_3-N loss, the loss would have been greatest in the reactors with the highest HRTs. The opposite was true.

A third explanation for the loss of nitrogen was the possibility of denitrification in the secondary clarifiers. This seems especially likely in systems A and B, which tended to produce less NO_3-N then the other systems. Anoxic conditions in the secondary clarifiers of systems A and B resulted in floating biomass early in the experiment. A reduction in clarifier volume solved the problem of floating biomass. However, it is likely that some small amount of denitrification continued throughout the experiment. If this was also happening to a lesser extent in the other systems, it would explain the discrepancy in nitrogen data.

5.2 Laundry Detergent

5.2.1 System Performance

Generally the addition of laundry detergent had no effect on the ability of the systems to meet effluent requirements. The three systems which received laundry detergent in their feed continued to produce effluent with low BOD_5 and TKN. As with the previous experiment, effluent TSS would not have been in compliance with Virginia regulations. The occurrences of BOD_5 which would have violated state regulations were also attributable to high TSS.

The only parameter to be affected by detergent was effluent COD, which is not regulated. Effluent COD concentrations from reactors A, C and E were more than twice as high as those from reactors B, D and F. Since effluent soluble BOD_5 did not show a similar relationship, it can be concluded that the additional COD was due to nondegradable organics in the laundry detergent, and not an indication of poor system performance.

As with the HRT study, there seems to have been a loss of total nitrogen, between influent and effluent. This loss seems to have been greater in systems A and C (which received detergent) than it was in systems B and D (Figures 15 and 16). This would tend to agree with the theory that denitrification was occurring in the secondary clarifiers. Because of the increased influent COD to the detergent systems, they also had an increased level of MLVSS. This would have caused increased levels of sludge in the clarifiers, and an increased opportunity for denitrification.

5.2.2 Observations

An interesting qualitative observation during this experiment was the absence of foaming in the reactors. On several occasions during the HRT experiment, foaming was a significant problem when feed with high levels of detergents was unintentionally added to the systems. Because of this, precautions were taken prior to the beginning of the detergent experiment to guard against loss of biomass during periods of high foam. During the first two days of the detergent experiment, some foaming was observed. For the remainder of the experiment the reactors receiving detergent were indistinguishable from the control systems.

This lack of foaming may have been related to several factors. One is the formation of micelles. At high concentrations, detergent molecules form groups. These micelles reduce the amount of individual detergent molecules in solution, and reduce the ability of the detergent to cause foaming. The foaming experienced during the first two days of operation may have been caused by detergent concentrations in the systems that had not yet reached the level of micelle formation.

Another factor may have been an adjustment by the microorganism populations in their ability either to digest or adsorb the detergents before foaming became a problem. During the HRT experiment a specific effort was made to avoid feed that contained laundry detergents. The homeowner was asked not to do laundry on days on which feed would be collected. During the HRT experiment the microbial populations had very little opportunity to acclimate themselves to the laundry detergents; so, when high concentrations of detergents were accidently introduced, foaming became a significant problem.

Another observation was the increase in biomass which occurred when detergent was added to the influent. This was to be expected, since the addition of detergent doubled the COD of the influent. What was interesting, however, was how well system A performed despite the large increase in MLVSS. At the end of the experiment reactor A had attained a MLVSS concentration of 6700 mg/L. The system was still performing very well. Solids were settling well and the system showed no sign of upset.

5.3 Field Experiment: Flow Equalization

5.3.1 System Performance

Because of the small amount of data taken in the previous experiment it was difficult to make any concrete conclusions as to the effect that flow equalization had on the field system. The data from the previous study and the data from this study both showed very little variation between samples taken on the same day. In the previous study samples were taken on three separate days during the morning, afternoon and evening. Since morning, afternoon and evening samples were similar, it is not reasonable to treat the data as 9 separate samples. The values should probably be viewed, rather, as data from three separate days. This limits the analysis to a comparison of 3 data points (from the previous study) to 4 data points (from the present study). Although BOD_5 and TSS average values were higher during the previous study, they were both influenced by a single day of high values. With only three days of data, it is impossible to know whether that one day was an unusual event.

The stability of the system over a short period of time (within a single day) may indicate an ability to handle changes in hydraulic loading. The long HRTs throughout the system may help to buffer it against upset due to sudden hydraulic loading. Attenuation of the washing machine effluent seemed to improve system stability more over the long term (week to week) than it did over the short term (hour to hour) (Figures 17, 18, 19 and 20).

Some general statements can be made about the performance of the field system during this experiment. The system provided some treatment of BOD_5 (total effluent BOD_5 averaged 33 mg/L), but did not meet the Virginia standard of 30 mg/L, despite fairly low TSS. The low effluent TSS (18 mg/L) was probably the result of the synthetic filter, which performed well throughout the experiment (Figure 1). The system provided no nitrification, though it had converted nearly all the organic nitrogen to NH_3-N. Despite the system's theoretically long sludge age there was no nitrification occurring. In addition, the MLVSS in the system was so low (35 mg/L) that it could hardly have been expected to behave like an extended aeration system. Despite the attenuation of washing machine effluent the field system did not provide acceptable treatment of household waste.

5.3.2 Comparisons to Design Models

Table 9 is a comparison of actual and theoretical system performance. The model presented by Benefield and Randall

(1980), was basically the same model proposed by Eckenfelder (1970), with the addition of a relationship for effluent BOD_5. In order to use the Lawrence and McCarty model, sludge age was calculated based on MLVSS, effluent VSS and HRT as reported by Kellam *et al.* (1992). The calculated sludge age (6.6 days) is very low for a system designed to operate as an extended aeration process. Commonly cited kinetic parameters (BOD_5 basis) were used for these calculations ($k=5$, $K_s=60$, $Y=0.6$ and $b=0.06$).

Table 9. Measured and Theoretical Soluble BOD_5 and MLVSS

	Measured Value (mean)	Benefield and Randall (1980)	Lawrence and McCarty (1970)
Soluble BOD_5 (mg/L)	10	1	2
MLVSS (mg/L)	24	380	104

Both models predicted very low MLVSS. The model presented by Benefield and Randall predicted more than that presented by Lawrence and McCarty, because it did not take into account solids lost as effluent TSS. None of these MLVSS values, either predicted by theory or those measured in the field, were close to the range generally cited as a healthy range for extended aeration processes. Metcalf and Eddy (1991) proposed 1,500-6,000 mg/L MLSS, and though the corresponding MLVSS range would be slightly lower than this (by ~30 %), it would still be well above the values presented in Table 9.

Table 10 is a summary of the aeration basin volumes which would be recommended by several different design methods. The models suggested by Benefield and Randall (1980) and Lawrence and McCarty (1970) were both theoretical designs based on standard kinetic parameters. A design MLVSS concentration of 3,500 mg/L was chosen, based on commonly reported values (Metcalf and Eddy, 1991; Benefield and Randall, 1980). A 25 day sludge age was chosen for the Lawrence and McCarty model based upon the assumption that a properly operating extended aeration system would fall within reported values (20 to 30 days; Grady and Lim, 1980). The final three designs were all based upon empirical criteria.

All of the designs used the influent BOD_5 value obtained during this study and the average flow rate reported by Kellam *et al.*, 1992.

Table 10. Design Values for Aeration Basin Volume and HRT

	Governing Assumption	Aeration Basin Volume (Gal.)	Hydraulic Retention Time (Days)
Actual Field System		600	3.3[1]
Benefield and Randall[2]	No Solids Lost	65	1.8
Lawrence and McCarty[3]	Sludge Age = 25 days[7]	39	0.2
Barnes and Wilson[4]	HRT = 1.3 Days	237	1.3
USPHS[5]	Organic Loading = .002 #BOD$_5$/Gal/Day	95	0.5
NAS-NRA[6]	Organic Loading = .0025 #BOD$_5$/Gal/Day	76	0.4

[1] Kellam, *et al.*, 1992
[2] Benefield and Randall, 1980
[3] Lawrence and McCarty, 1970
[4] Barnes and Wilson, 1976
[5] U. S. Public Health Service, 1960
[6] National Academy of Sciences-National Research Council, 1958
[7] Sludge Age based on average sludge age for extended aeration, Grady and Lim, 1980

All of the designs listed in Table 10 gave HRTs and aeration basin volumes much smaller than those observed in the field. It is standard practice when designing on site waste treatment systems to use "conservative" flowrate estimates for the household, based on a maximum flow per capita at full occupancy. Designs based on this criteria would have produced larger HRTs and aeration basin volumes than are listed in Table 10. This may not, however, be the best design for an extended aeration system.

If a system were designed for the maximum flowrate possible from a residence, most of the time it would be treating much less than the design flow rate, with resulting long HRTs. In the case of the system used in this study the long HRT (3.3 days) resulted in a very low MLVSS concentration (Table 9). Theoretically (based both on the Eckenfelder model and the Lawrence and McCarty model), long HRTs and low MLVSS should not affect final effluent quality. This, however, assumes that at this high HRT, a system would still be functioning as an extended aeration system, with a long sludge age.

The Lawrence and McCarty model was used to estimate the sludge age of the field system. Based upon the measured MLVSS the field system appeared to have been operating at a sludge age between 0.5 and 4 days depending on the kinetic parameters used in the calculations.

Estimates of sludge age based upon MLVSS and effluent VSS were higher (6.6 days). However, this assumes that all solids not lost as VSS were retained in the aeration chamber. This was probably not the case. A large accumulation of biomass was observed in the secondary clarifier during sampling. It is likely that the lower sludge age estimates derived from the Lawrence and McCarty model were more accurate.

These extremely low sludge ages would explain the complete lack of nitrification found in this system. The Water Pollution Control Federation (1983), recommended a minimum sludge age of greater than 6 days to insure the health of nitrifying bacteria. (Nitrifying bacteria have a relatively slow growth rate and must be kept in the system for a longer time then other bacteria to maintain healthy populations). Consistent nitrification cannot be expected at sludge ages less then 4 days.

The system tested during this study was not operating as an extended aeration system. It was not even operating as an activated sludge system. It was, rather, an aerobic chemostat. Raw waste was aerated for several days in the aeration chamber without any accumulation of microbial population. Microbes that did grow during aeration were either lost as effluent solids, or were trapped on the synthetic filter or on the walls of the secondary clarifier.

The poor system performance seems to have been a result of the long HRT. Samples taken from the aeration chamber showed no sign of flocculation. At such low concentrations of MLSS, solids were completely dispersed. No settling occurred, even after samples were allowed to sit for several hours. Even if the system had been able to attain a long sludge age, the concentration of solids predicted by the Lawrence and McCarty model (220 mg/L) would still not have been high enough to insure good flocculation and settling.

6. Conclusions

As presently designed and operated, onsite aerobic package treatment systems perform poorly. This poor performance is primarily a result of extremely long hydraulic retention times. Systems operated at HRTs of 3 days and longer are unable to maintain biological populations of high enough concentrations to insure good flocculation and settling. Such systems are unstable, and are likely to violate state effluent limits.

Hydraulic surges affect the systems to a lesser degree. Even with the largest surge flow attenuated, the field system did not perform well. Though there was some improvement compared to the unattenuated system, the package treatment system was still unable to maintain a healthy biological population. However, since the systems relative insensitivity to surge flows is at least, in part, a result of long HRT, both must be considered in any proposed design modifications.

It is recommended that package treatment systems be designed such that the *average* (rather than the maximum) flow expected from a household will produce an HRT of 1 day. Though this may result in periods of lower HRT, extended aeration systems were shown to function well at these low HRTs. This should improve overall system performance and stability.

Flow equalization, especially of grey water sources, should be included in the low HRT design, as the smaller aeration chamber may be more sensitive to hydraulic surges.

Expected concentrations of the laundry detergent considered in this study (with anionic surfactants) did not have a significant effect on the performance of extended aeration systems, except as an additional source of organic substrate.

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Appendix

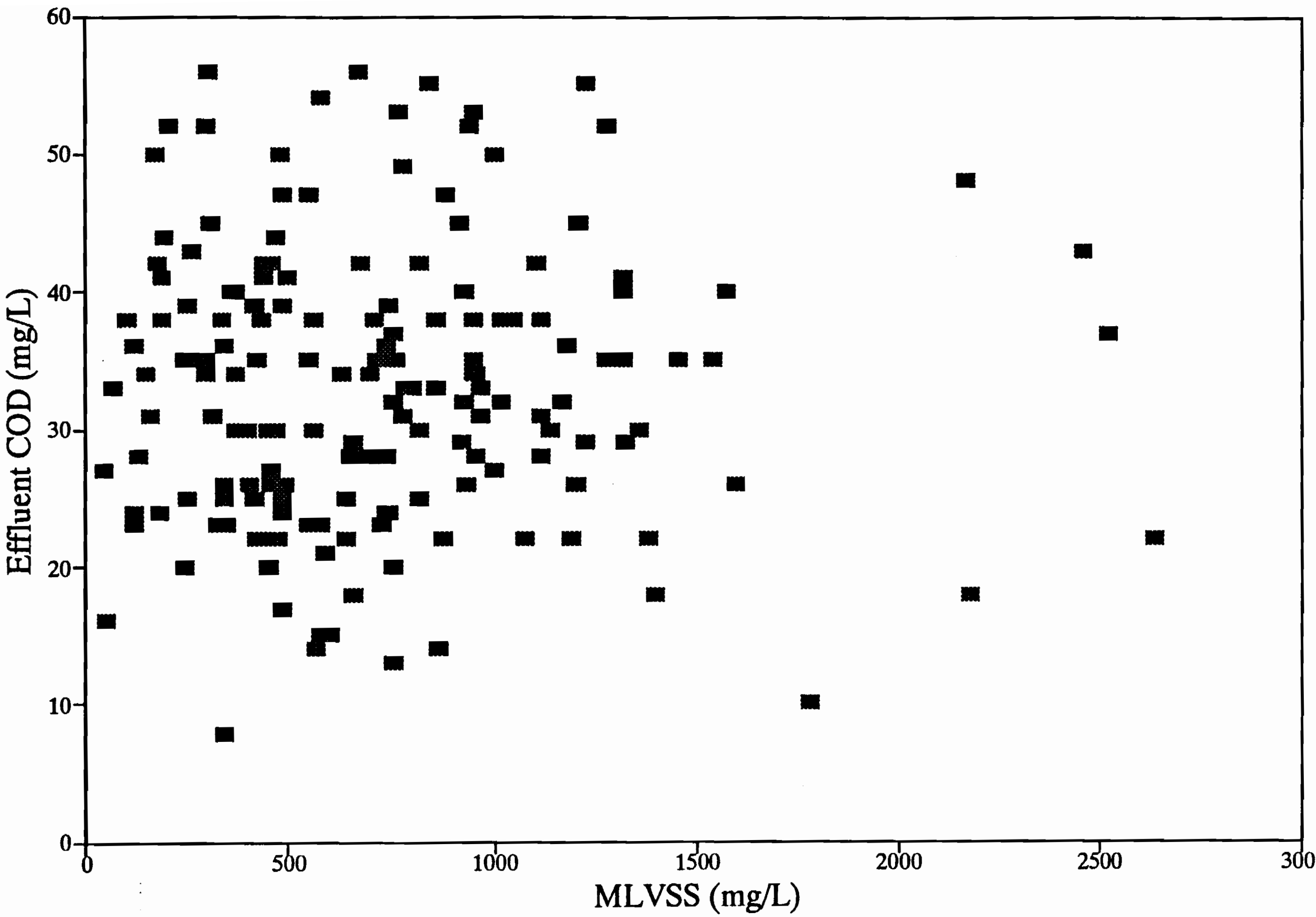

Figure A1. Effluent Chemical Oxygen Demand Versus Mixed Liquor Volatile Suspended Solids, All Reactors, Days 34 - 228 (HRT Experiment)

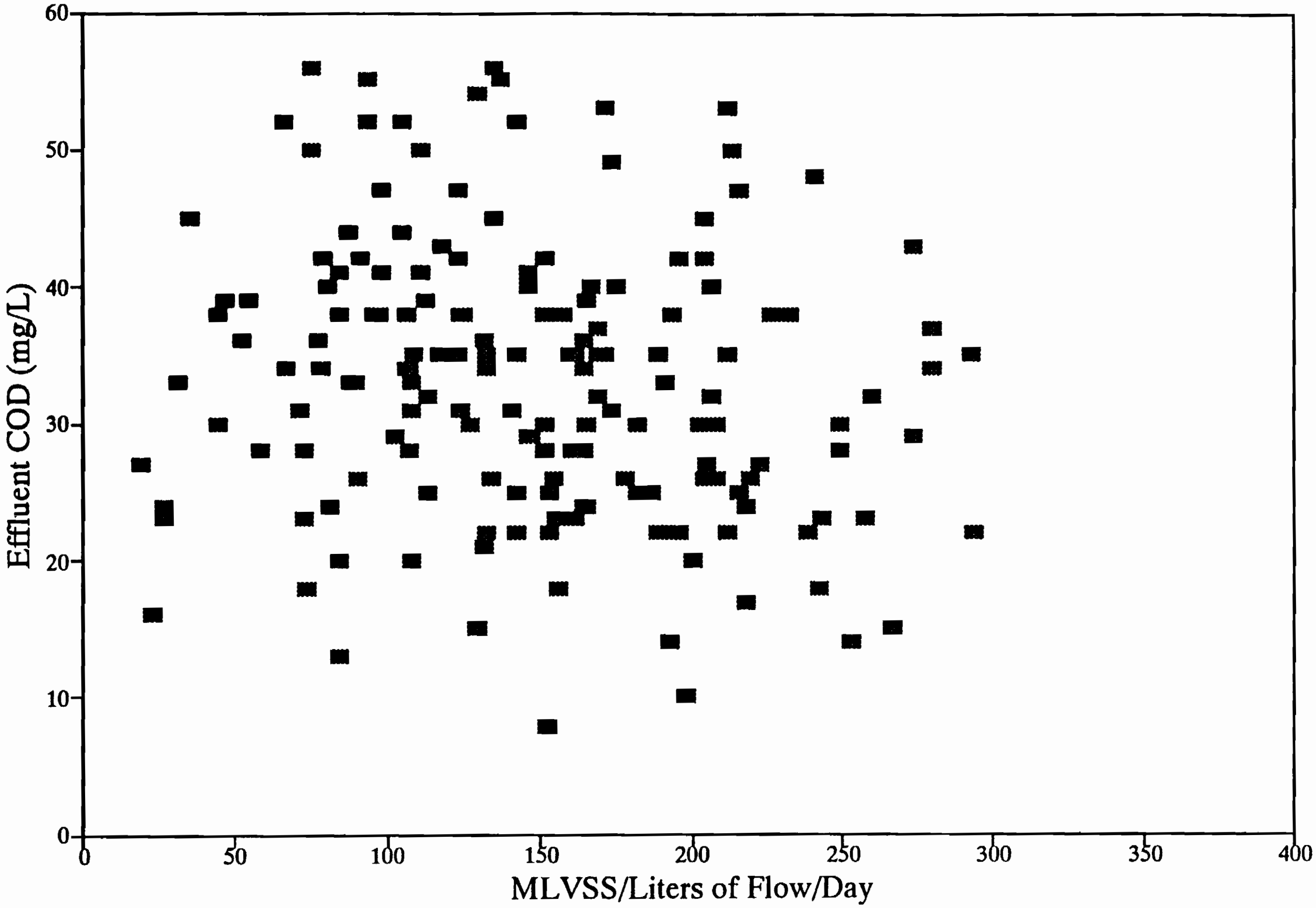

Figure A2. Effluent Chemical Oxygen Demand Versus Mixed Liquor Volatile Suspended Solids, (Equalized for Flow Rate), All Reactors, Days 34 - 228 (HRT Experiment)

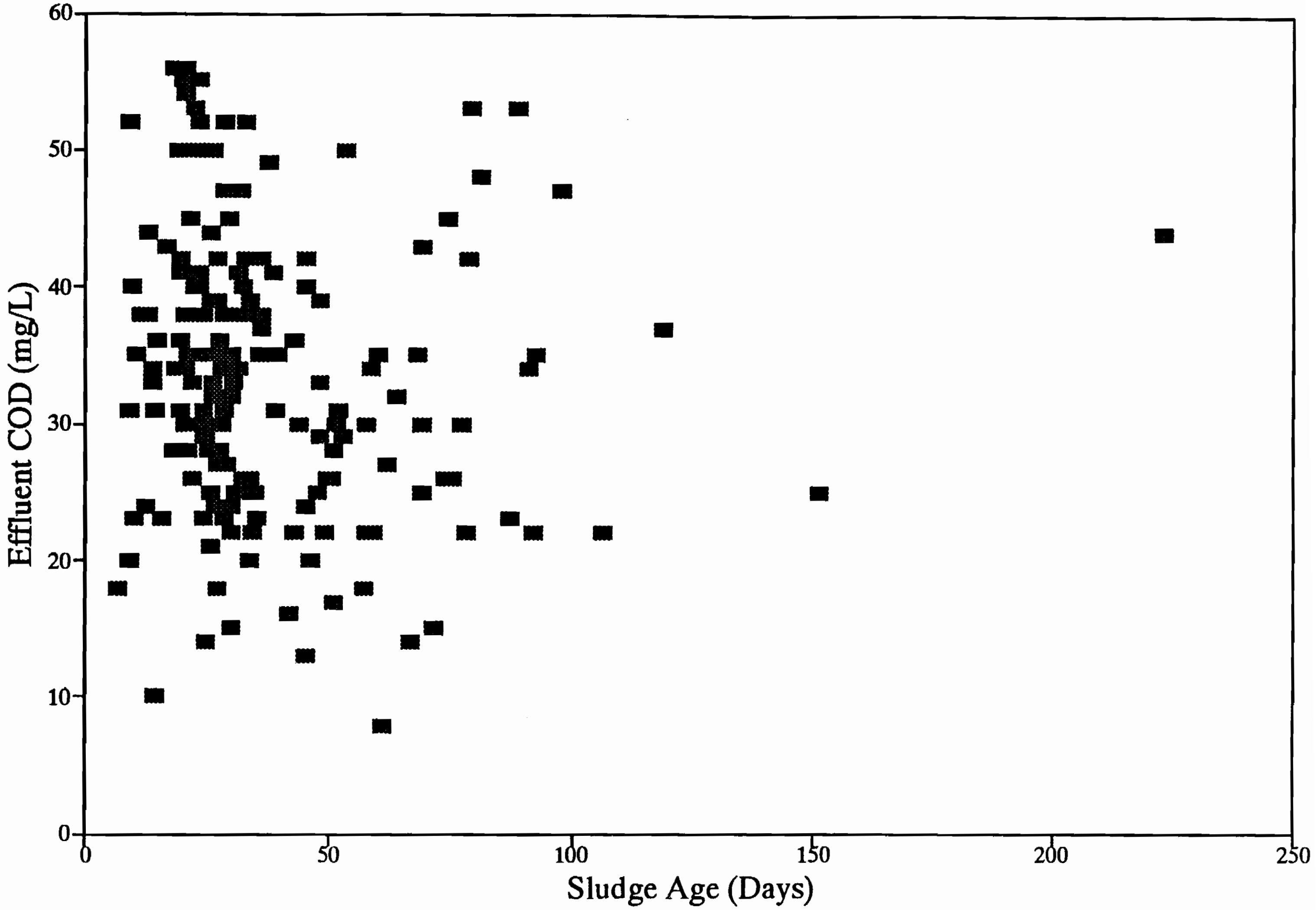

Figure A3. Effluent Chemical Oxygen Demand Versus Sludge Age, All Reactors, Days 34 - 228 (HRT Experiment)

Table A1. Effluent COD (mg/L), HRT Experiment

	System					
Day	A	B	C	D	E	F
40	48	35	38	66	30	22
45	63	63	49	53	47	44
47	63	34	38	45	22	25
52	41	55	31	32	26	35
55	33	45	24	22	25	34
60	18	32	14	29	26	20
64	22	47	39	*	7	24
69	40	35	26	35	30	22
74	42	10	35	35	35	35
95	34	38	35	35	23	31
102	31	*	22	23	26	22
109	24	31	24	29	22	24
111	38	18	15	28	15	23
112	38	20	25	42	14	25
114	26	33	28	28	17	20
115	30	28	22	36	34	30
118	26	35	27	24	24	27
119	22	29	30	32	40	30
128	53	108	50	42	63	47
133	44	31	31	*	50	52
137	52	36	38	33	39	50
148	66	55	44	28	42	52
156	50	28	40	95	38	43
159	52	42	52	41	38	34
165	20	26	30	31	34	*
169	39	13	24	26	34	38
179	39	33	23	23	33	68
189	34	*	*	21	38	31
206	22	36	24	22	32	31
217	22	18	25	32	25	23
224	37	43	37	40	42	35
228	*	27	22	32	30	28

Table A2. Total Suspended Solids (mg/L) - HRT Experiment

	System					
Date	A	B	C	D	E	F
34	16	28	60	66	48	20
38	13	20	37	20	26	3
42	12	21	32	15	10	0
48	26	43	29	19	5	6
52	46	24	37	28	11	6
55	25	36	51	28	32	11
59	15	35	42	24	24	10
61	19	8	4	6	0	0
68	29	29	58	40	27	47
70	22	19	26	11	7	11
74	21	21	22	24	11	14
95	40	103	23	39	18	15
102	43	100	47	26	24	28
111	29	60	13	28	12	21
112	26	45	17	45	23	30
114	21	39	20	32	25	27
115	23	22	15	29	23	45
119	8	21	8	26	76	47
137	6	38	37	7	11	15
147	82	68	65	76	54	68
150	1	12	70	30	11	24
158	32	9	33	24	20	11
161	15	1	19	19	35	14
171	20	24	10	42	33	73
179	14	20	22	20	15	70
189	7	6	19	20	32	28
217	14	22	43	36	12	34
221	19	32	20	33	10	99
224	13	11	22	49	85	
228	10	4	15	40	12	49

Table A3. Effluent Total BOD_5 (mg/L) - HRT Experiment

	System					
Date	A	B	C	D	E	F
112	16	28	16	14	12	16
120	7	27	18	25		40
161	23	15	25	21	35	16
173	40	12	10	17	15	14
181	14	12	16	15	14	30

Table A4. Effluent NO_3 (mg/L) - HRT Experiment

	System					
Date	A	B	C	D	E	F
111	22.8	21.4	26.5	24.3	25.3	26.6
112	17.0	17.7	18.0	18.8	20.4	12.9
114	10.6	12.5	12.5	11.4	12.7	14.2
115	7.5	6.3	28.8	16.8	19.9	20.9
118	17.7	6.8	18.2	11.6	11.5	14.9
119	*	*	*	*	*	*
128	*	16.4	14.2	8.5	15.9	15.5
133	5.3	3.9	7.1	*	10.4	14.6
137	24.1	19.4	22.9	15.9	17.7	17.4
148	18.7	8.7	15.3	16.8	17.6	15.0
156	17.4	14.5	19.5	18.2	22.5	18.9
159	14.6	14.4	17.5	16.5	18.0	15.6
165	15.9	12.7	17.3	17.6	16.6	*
169	14.7	17.9	19.0	19.1	19.1	16.8
177	20.4	25.1	25.2	22.3	23.0	24.4
206	13.0	12.0	18.7	19.0	19.6	19.6
216	19.1	18.0	20.6	17.9	20.2	19.0
220	15.6	15.5	21.1	19.0	17.8	21.4
224	24.4	22.7	18.1	15.8	18.0	14.5
228	14.0	15.6	18.7	21.1	18.5	22.0

Table A5. Effluent TKN - HRT Experiment

	System					
Date	A	B	C	D	E	F
111	0.2	0.0	0.0	0.0	0.0	0.0
112	0.0	0.0	0.0	0.0	0.0	1.4
114	0.5	1.4	0.5	0.0	0.0	0.0
115	0.0	1.4	0.0	0.0	0.7	1.0
118	0.0	1.2	0.0	0.0	0.7	1.0
119	0.0	0.0	1.4	0.7	1.7	1.4
128	1.7	1.4	1.7	1.9	1.9	0.2
133	1.2	1.7	0.7	1.0	0.7	1.2
137	2.2	0.0	0.5	0.2	1.2	1.2
148	2.2	1.9	1.9	0.0	0.5	1.4
156	1.7	0.0	0.0	*	2.6	1.4
159	2.4	2.4	2.4	2.6	2.4	2.2
165	1.2	1.4	1.7	1.7	1.7	1.4
169	1.7	1.4	1.7	1.7	1.7	1.4
177	0.0	0.0	0.0	0.0	0.0	0.0
206	0.0	0.0	0.0	0.0	0.0	0.0
216	0.2	0.0	0.0	0.5	0.5	0.5
220	*	*	*	*	*	*
224	0.0	0.0	0.0	0.0	0.0	0.0
228	*	*	*	*	*	*

Table A6. Effluent OrthoP (mg/L) - HRT Experiment

	System					
Date	A	B	C	D	E	F
111	7.1	7.2	7.4	7.9	6.9	6.4
112	4.9	5.1	5.5	2.4	5.5	4.0
114	4.7	4.5	3.9	3.1	4.0	3.5
115	2.6	2.2	7.9	5.2	4.2	3.9
118	4.3	2.2	4.7	4.3	3.2	3.0
119	*	*	*	*	*	*
128	*	5.1	2.4	2.3	4.1	3.9
133	3.1	3.0	2.9	*	3.5	3.5
137	4.1	3.7	4.0	4.2	4.5	4.7
148	4.5	3.6	4.4	4.3	4.3	4.7
156	3.4	2.9	3.5	3.3	3.5	3.4
159	3.2	2.8	3.2	3.0	2.7	2.6
165	3.3	3.3	3.3	3.3	3.5	*
169	3.8	3.3	3.5	3.1	3.6	3.3
177	3.1	3.2	3.3	2.9	3.3	3.3
206	1.8	1.4	2.3	2.6	3.0	3.1
216	4.2	3.9	4.4	4.4	4.3	4.2
220	7.3	7.4	6.4	5.9	5.3	6.3
224	7.8	9.4	7.0	5.4	6.7	5.6
228	6.3	6.3	6.3	7.1	6.1	7.5

Table A7. Effluent TotalP (mg/L) - HRT Experiment

	System					
Date	A	B	C	D	E	F
111	7.0	7.0	7.1	7.7	6.5	7.0
112	7.6	5.9	6.5	6.7	6.8	6.5
114	5.3	5.5	5.8	6.2	6.2	6.5
115	5.2	5.4	6.0	6.1	6.1	6.4
118	5.0	5.1	5.7	5.8	7.0	7.6
119	5.5	5.8	6.0	*	*	3.9
128	1.8	4.7	1.4	3.1	5.2	5.8
133	4.1	2.7	3.9	*	4.7	4.5
137	5.6	4.5	4.8	4.8	4.7	5.2
148	4.8	3.9	7.9	5.2	4.6	5.0
156	1.7	2.3	1.8	6.6	4.1	4.1
159	1.8	2.0	2.3	2.9	0.7	2.7
165	*	2.9	3.2	3.7	2.8	*
169	2.5	2.4	3.0	2.4	*	*
177	*	*	*	*	*	*
206	2.4	2.6	3.3	3.3	3.5	3.3
216	5.2	4.0	5.2	5.4	4.7	4.7
220	7.7	8.5	7.5	4.7	5.6	6.1
224	*	*	*	*	*	*
228	*	*	*	*	*	*

Table A8. Effluent COD (mg/L) - Detergent Experiment

	System					
Date	A	B	C	D	E	F
234	59	32	67	35	61	45
239	64	21	62	34	64	29
248	98	21	64	41	57	39
251	38	20	55	29	51	21
256	51	29	74	37	51	*
270	53	30	*	*	61	27
282	38	19	54	*	54	35

Table A9. Effluent TSS (mg/L) - Detergent Experiment

	System					
Date	A	B	C	D	E	F
234	14	37	27	74	19	94
239	4	35	27	22	98	*
248	11	20	8	5	13	*
251	75	30	32	19	17	64
256	39	56	34	22	15	20
270	34	18	*	*	50	24

Table A10. Effluent Total BOD_5 (mg/L) - Detergent Experiment

	System					
Date	A	B	C	D	E	F
248	13	18	13	8	15	*
252	45	18	19	10	11	53

Table A11. Effluent Soluble BOD_5 (mg/L) - Detergent Experiment

	System					
Date	A	B	C	D	E	F
248	1.0	1.7	2.4	2.4	4.6	8.0
252	3.3	1.8	3.1	1.7	2.7	2.7
270	2.1	2.8	*	*	2.4	1.5

Table A12. Effluent NO_3 (mg/L)- Detergent Experiment

	System					
Date	A	B	C	D	E	F
235	18.2	26.4	26.5	29.0	23.6	24.8
239	25.8	32.7	29.7	29.4	28.7	33.2
248	27.2	33.2	18.9	23.6	22.9	26.0
251	24.4	25.6	23.9	22.8	22.8	30.8
256	15.9	18.3	19.5	22.6	25.3	20.5
270	16.7	23.0	*	*	47.1	41.6
284	7.2	5.9	4.7	*	32.7	37.2

Table A13. Effluent TKN (mg/L) - Detergent Experiment

	System					
Date	A	B	C	D	E	F
235	0.0	0.0	0.00	0	0.0	0.00
239	0.5	0.0	0.25	0	0.5	0.25
248	0.0	0.0	1.20	0	0.0	0.00
251	0.0	0.0	0.00	0	0.0	0.00
256	0.0	0.0	0.70	0	0.0	0.00
270	0.5	0.0	*	*	0.0	0.00
284	0.2	0.2	1.20	*	0.7	0.00

Table A14. Effluent OrthoP (mg/L) - Detergent Experiment

	System					
Date	A	B	C	D	E	F
235	6.0	8.4	7.1	6.9	8.6	11.5
239	6.9	8.4	8.1	7.0	8.9	9.9
248	4.8	5.3	7.1	4.6	6.3	7.2
251	5.3	5.1	7.8	4.8	4.8	6.5
256	4.2	4.7	6.5	5.5	6.0	5.3
270	4.7	5.5	*	*	9.5	8.3
284	3.0	2.7	2.4	*	7.2	6.1

Table A15. Filtered Effluent Characteristics with Attenuation of Laundry Effluent (mg/L) - Field Experiment

Date	Time	COD	BOD_5	TKN	NO_3-N	TKN	OP-P	TP-P
10/27/92	8-10	67	11	21	0.54	17	2.05	3.7
	2-4	63	12		0.27	17	1.59	3.1
	9-11	61	11		0.31	18	2.13	3.7
11/12/92	8-10	62	8	27	0.20	24	5.44	7.1
	2-4	78	4	26	0.18	25	5.81	6.8
	9-11	60	5	26	0.19	25	6.21	6.8
11/19/92	8-10	70	10	27	0.09	24	5.62	6.3
	2-4	70	13	27	0.06	25	6.08	6.1
	9-11	67	11	54	0.07	24	5.48	5.5
12/1/92	8-10	94	12	31		25		5.1
	2-4	106	14	31	0.35	25	5.47	4.9
	9-11	90	14	31	0.20	25	5.74	5.5
	Ave	74.0	10.4	30.1	0.22	22.8	4.69	5.4
	Min	60	4	21	0.06	17	1.59	3.1
	Max	106	14	54	.54	25	6.21	7.1

Table A16. Unfiltered Effluent Characteristics with Attenuation of Laundry Effluent (mg/L) - Field Experiment

Date	Time	COD	BOD_5	TKN	TP-P
10/27/92	8-10	132	44	21	5.3
	2-4		38		
	9-11		39		
11/12/92	8-10	67	11	27	6.6
	2-4	73	15	26	6.6
	9-11	78	13	26	2.3
11/19/92	8-10	97	21	27	4.9
	2-4	104	28	27	6.2
	9-11	95	22	54	5.7
12/1/92	8-10	165	53	31	6.5
	2-4	158	54	31	6.9
	9-11	160		31	5.9
	Ave	113	30.7	30.1	5.69
	Min	67	11	21	2.3
	Max	165	54	54	6.9

Table A17. Dissolved Oxygen in Aerobic Treatment System with Attenuation of Laundry Effluent (mg/L) - Field Experiment

Date	Time	Aeration Basin	Secondary Settling Basin	Pump Chamber
10/27/92	8-10			1.0
	2-4			
	9-11			1.3
11/12/92	8-10	6	0.5	1.5
	2-4			1.5
	9-11		1.3	2.2
11/19/92	8-10	5.1	0.5	1.3
	2-4	6.1	0.6	0.8
	9-11			1.3
12/1/92	8-10	6.1	0.5	0.6
	2-4	8.4	0.6	0.9
	9-11			0.8
	Ave	6.34	0.67	1.20
	Min	5.1	0.5	0.6
	Max	8.4	1.3	2.2

Table A18. Effluent and Mixed Liquor Suspended Solids with Attenuation of Laundry Effluent (mg/L) - Field Experiment

Date	Time	TSS	VSS	MLSS	MLVSS
10/27/92	8-10	29	25	53	44
	2-4	32	26	69	55
	9-11	31	24	60	47
11/12/92	8-10	8	7	12	11
	2-4	8	7	11	8
	9-11	10	7	12	8
11/19/92	8-10	9	7	32	25
	2-4	11	9	58	19
	9-11	45	7	49	12
12/1/92	8-10	10	10	17	17
	2-4	7	5	19	14
	9-11	12	11	27	23
	Ave	17.7	12.1	34.9	23.6
	Min	7	5	11	8
	Max	45	26	69	55

Table A19. Temperature and pH in Aerobic Treatment System with Attenuation of Laundry Effluent - Field Experiment

Date	Time	pH	Temperature (C)
10/27/92	8-10	7.3	20
	2-4		
	9-11	7.4	20
11/12/92	8-10	7.4	20
	2-4	7.8	20
	9-11		20
11/19/92	8-10	7.5	19
	2-4	7.5	21
	9-11	7.3	20
12/1/92	8-10	7.3	18
	2-4	6.7	18
	9-11	7.0	18
	Ave	7.2	
	Min	6.7	18
	Max	7.8	21

Table A20. BOD$_5$ and COD from Field System without Attenuation of Laundry Effluent (mg/L), (from Kellam *et al.*, 1992)

SITE & DATE	TIME OF DAY	PRIMARY SETTLING CHAM		FILTRATION	EFFLUENT	
		BOD$_5$ (mg/L)	COD (mg/L)	BOD$_5$ (mg/L)	BOD$_5$ (mg/L)	COD (mg/L)
APTS I	8-10 AM	96	111	19	24	58
(7/12/90)	2-4 PM	168	282	36	32	56
	9-11 PM	309	885	29	27	67
APTS II	8-10 AM	666	3771	16	33	84
(9/02/90)	2-4 PM	590	3755	55	81	311
	9-11 PM	714	7614	115	55	139
APTS III	8-10 AM	145	238	44	46	86
(9/23/90)	2-4 PM[a]	119	173	42	110	307
	9-11 PM[a]	144	268	47	36	386
APTS IV	8-10 AM	38	112	16	9[b]	83
(10/21/90)	2-4 PM	44	155	15	-[b]	96
	9-11 PM	34	126	21	-[b]	68
APTS V	8-10 AM	33	91	9	15	40
(10/06/90)	2-4 PM[c]	39	116	10	22	116
	9-11 PM	65	81	17	20	46
APTS I	8-10 AM	181	313	103[d]	124[d]	264
(12/01/90)	2-4 PM	171	274	115[d]	118[d]	262
	9-11 PM	300	422	114[d]	114[d]	260
APTS II	8-10 AM	843	4407	36	121	208
(2/04/91)	2-4 PM	795	4603	43	69	161
	9-11 PM	906	7814	64	107	178
APTS III	8-10 AM	152	344	103	108	190
(1/05/91)	2-4 PM	167	624	119	89	290
	9-11 PM	140	316	130	100	255
APTS IV	8-10 AM	65	174	89	58	110
(1/18/91)	2-4 PM	104	250	68	62	123
	9-11 PM	187	414	88	96	150
APTS V	8-10 AM	118	118	76	49	128
(2/27/91)	2-4 PM	118	110	42	79	77
	9-11 PM	159	202	54	82	80

continued

Table A20. continued

SITE & DATE	TIME OF DAY	PRIMARY SETTLING CHAM		FILTRATION	EFFLUENT	
		BOD$_5$ (mg/L)	COD (mg/L)	BOD$_5$ (mg/L)	BOD$_5$ (mg/L)	COD (mg/L)
APTS I	8-10 AM	33	207	6	37	145
(5/12/91)	2-4 PM	87	267	19	40	94
	9-11 PM	102	321	20	26	129
APTS II	8-10 AM	119	694	37	29[e]	119
(5/30/91)	2-4 PM	88	1135	11	40[e]	173
	9-11 PM	91	988	22	40[e]	163
APTS III	8-10 AM	352	2575	94	83	162
(5/11/91)	2-4 PM	208	4148	96	103	242
	9-11 PM	186	3211	86	90	232
APTS IV	8-10 AM	116	345	58	53	174
(5/26/91)	2-4 PM	-	-	52	58	151
	9-11 PM	-	-	47	56	159
APTS V	8-10 AM	130	144	72	103	148
(6/05/91)	2-4 PM	95	132	79	150	115
	9-11 PM	122	144	61	222	131

[a] low volume in submersible pump when activated

[b] BOD$_5$ could be influenced by residual chlorine; not enough $Na_2S_2O_3$ to neutralize residual chlorine.

[c] possible contamination with ditch water and/or sediments in the effluent pipe

[d] Residual D.O. < 0.5 mg/L; thus, calculated BOD$_5$ for the dilutions with the least sample volume.

[e] Used 7.00 mg/L as D.O.$_i$ reading in calculations since experienced problems with the probe.

Table A21. TSS and VSS from Field System without Attenuation of Laundry Effluent (mg/L), (from Kellam *et al.*, 1992)

SITE & DATE	TIME OF DAY	PRIMARY SETTLING CHAM		EFFLUENT	
		TSS (mg/L)	VSS (mg/L)	TSS (mg/L)	VSS (mg/L)
APTS I	8-10 AM	104	-	8	-
(7/12/90)	2-4 PM	101	-	8	-
	9-11 PM	620	-	17	-
APTS II	8-10 AM	4240	-	77	-
(9/02/90)	2-4 PM	4160	-	336	-
	9-11 PM	6200	-	130	-
APTS III	8-10 AM	182	-	46	-
(9/23/90)	2-4 PM[a]	146	-	244	-
	9-11 PM[a]	134	-	465	-
APTS IV	8-10 AM	37	-	16	-
(10/21/90)	2-4 PM	83	-	20	-
	9-11 PM	21	-	11	-
APTS V	8-10 AM	20	-	6	-
(10/06/90)	2-4 PM[b]	34	-	426	-
	9-11 PM	24	-	12	-
APTS I	8-10 AM	34	20	26	21
(12/01/90)	2-4 PM	34	20	30	21
	9-11 PM	51	37	26	20
APTS II	8-10 AM	4470	3090	188	127
(2/04/91)	2-4 PM	4650	3140	138	88
	9-11 PM	7580	5170	157	100
APTS III	8-10 AM	243	163	84	58
(1/05/91)	2-4 PM	625	385	185	115
	9-11 PM	247	160	164	106
APTS IV	8-10 AM	20	17	7	5
(1/18/91)	2-4 PM	29	15	6	4
	9-11 PM	32	22	12	9
APTS V	8-10 AM[b]	14	11	445	63
(2/27/91)	2-4 PM[b]	39	28	304	38
	9-11 PM	81	60	27	16

continued

Table A21. continued

SITE & DATE	TIME OF DAY	PRIMARY SETTLING CHAM TSS (mg/L)	PRIMARY SETTLING CHAM VSS (mg/L)	EFFLUENT TSS (mg/L)	EFFLUENT VSS (mg/L)
APTS I (5/12/91)	8-10 AM	92	71	50	37
	2-4 PM	127	93	48	35
	9-11 PM	116	73	60	41
APTS II (5/30/91)	8-10 AM	530	360	47	30
	2-4 PM	820	533	74	54
	9-11 PM	746	546	71	52
APTS III (5/11/91)	8-10 AM	2000	1270	71	62
	2-4 PM	3500	2114	94	78
	9-11 PM	1770	1100	91	77
APTS IV (5/26/91)	8-10 AM	164	105	25	21
	2-4 PM	-	-	36	31
	9-11 PM	-	-	22	19
APTS V (6/05/91)	8-10 AM	61	46	41	22
	2-4 PM	46	36	59	35
	9-11 PM	51	41	53	28

[a] low volume in submersible pump when activated
[b] possible contamination with ditch water and/or sediments in the effluent pipe

Table A22. **MLSS and MLVSS from Field System without Attenuation of Laundry Effluent (mg/L), (from Kellam *et al.*, 1992)**

SITE & DATE	TIME OF DAY	4' BELOW WATER LEVEL MLSS (mg/L)	MLVSS (mg/L)	2' BELOW WATER LEVEL MLSS (mg/L)	MLVSS (mg/L)
APTS I	8-10 AM	205	152	-	-
(7/12/90)	2-4 PM	167	117	-	-
	9-11 PM	243	167	-	-
APTS II	8-10 AM	4067	2587	-	-
(9/02/90)	2-4 PM	4227	2747	-	-
	9-11 PM	4587	2947	-	-
APTS III	8-10 AM	203	130	-	-
(9/23/90)	2-4 PM	182	120	-	-
	9-11 PM	220	143	-	-
APTS IV	8-10 AM	14	13	-	-
(10/21/90)	2-4 PM	16	14	-	-
	9-11 PM	14	13	-	-
APTS V	8-10 AM	9	8	-	-
(10/06/90)	2-4 PM	5	5	-	-
	9-11 PM	9	7	-	-
APTS I	8-10 AM	36	23	67	40
(12/01/90)	2-4 PM	67	40	71	45
	9-11 PM	63	36	60	33
APTS II	8-10 AM	4860	3270	4600	3100
(2/04/91)	2-4 PM	4350	2910	4350	2930
	9-11 PM	4570	3040	4560	3070
APTS III	8-10 AM	247	87	267	173
(1/05/91)	2-4 PM	403	250	420	245
	9-11 PM	317	200	316	200
APTS IV	8-10 AM	10	2	14	10
(1/18/91)	2-4 PM	27	18	19	13
	9-11 PM	24	16	22	15
APTS V	8-10 AM	12	8	14	9
(2/27/91)	2-4 PM	12	9	15	10
	9-11 PM	20	13	36	25

continued

Table A22. continued

SITE & DATE	TIME OF DAY	4' BELOW WATER LEVEL		2' BELOW WATER LEVEL	
		MLSS (mg/L)	MLVSS (mg/L)	MLSS (mg/L)	MLVSS (mg/L)
APTS I	8-10 AM	200	120	-	-
(5/12/91)	2-4 PM	192	118	-	-
	9-11 PM	198	128	-	-
APTS II	8-10 AM	715	508	-	-
(5/30/91)	2-4 PM	817	550	-	-
	9-11 PM	833	575	-	-
APTS III	8-10 AM	565	355	-	-
(5/11/91)	2-4 PM	460	287	-	-
	9-11 PM	380	235	-	-
APTS IV	8-10 AM	167	118	-	-
(5/26/91)	2-4 PM	166	118	-	-
	9-11 PM	148	108	-	-
APTS V	8-10 AM	113	80	-	-
(6/05/91)	2-4 PM	82	61	-	-
	9-11 PM	91	64	-	-

Table A23. Temperature, pH and D.O. from Field System without Attenuation of Laundry Effluent (mg/L), (from Kellam *et al.*, 1992)

SITE & DATE	TIME OF DAY	4' BELOW WATER LEVEL			2' BELOW WATER LEVEL		
		pH	TEMP (C)	D.O. (mg/L)	pH	TEMP (C)	D.O. (mg/L)
APTS I	8-10 AM	7.2	28.7	4.6	-	-	-
(7/12/90)	2-4 PM	7.5	29.4	6.1	-	-	-
	9-11 PM	7.1	27.4	4.8	-	-	-
APTS II	8-10 AM	6.7	27.2	2.6	-	-	-
(9/02/90)	2-4 PM	6.8	28.3	2.5	-	-	-
	9-11 PM	6.8	27.2	4.1	-	-	-
APTS III	8-10 AM	7.3	23.8	5.9	-	-	-
(9/23/90)	2-4 PM	7.1	23.4	4.9	-	-	-
	9-11 PM	7.1	22.4	5.3	-	-	-
APTS IV	8-10 AM	7.5	21.9	6.0	-	-	-
(10/21/90)	2-4 PM	7.3	23.0	6.5	-	-	-
	9-11 PM	7.1	21.1	6.1	-	-	-
APTS V	8-10 AM	7.8	21.5	5.8	-	-	-
(10/06/90)	2-4 PM	7.9	23.6	5.6	-	-	-
	9-11 PM	7.8	21.2	6.0	-	-	-
APTS I	8-10 AM[a]	6.8	18.0	0.9	6.8	18.4	1.0
(12/01/90)	2-4 PM	7.2	19.9	6.2	7.3	20.2	6.5
	9-11 PM	7.2	16.4	8.4	7.3	18.2	8.0
APTS II	8-10 AM	6.8	16.4	1.4	6.7	16.0	1.1
(2/04/91)	2-4 PM	6.8	19.5	2.4	6.7	19.8	2.2
	9-11 PM	6.9	15.1	4.8	6.8	15.6	4.8
APTS III	8-10 AM	7.1	11.1	8.0	7.2	10.9	8.0
(1/05/91)	2-4 PM	7.2	12.1	7.1	7.2	12.3	7.5
	9-11 PM	7.2	11.5	7.5	7.2	11.8	7.4
APTS IV	8-10 AM	7.0	13.2	6.2	7.0	13.7	5.8
(1/18/91)	2-4 PM	6.9	13.6	4.0	7.0	14.3	3.9
	9-11 PM	7.0	13.4	4.4	7.1	11.9	4.2
APTS V	8-10 AM	6.5	12.0	8.4	6.5	12.0	8.4
(2/27/91)	2-4 PM	6.5	13.5	8.1	6.5	13.5	8.0
	9-11 PM	6.5	11.8	8.4	6.5	11.9	8.4

continued

Table A23. continued

SITE & DATE	TIME OF DAY	4' BELOW WATER LEVEL			2' BELOW WATER LEVEL		
		pH	TEMP (C)	D.O. (mg/L)	pH	TEMP (C)	D.O. (mg/L)
APTS I	8-10 AM	7.5	24.6	4.4	-	-	-
(5/12/91)	2-4 PM	7.4	23.5	4.7	-	-	-
	9-11 PM	7.3	23.5	4.1	-	-	-
APTS II	8-10 AM	7.1	26.1	1.8	-	-	-
(5/30/91)	2-4 PM	7.3	26.6	2.5	-	-	-
	9-11 PM	7.3	26.1	2.1	-	-	-
APTS III	8-10 AM	7.4	20.8	-	-	-	-
(5/11/91)	2-4 PM	7.4	20.8	4.5	-	-	-
	9-11 PM	7.4	20.5	4.5	-	-	-
APTS IV	8-10 AM	8.1	22.7	4.5	-	-	-
(5/26/91)	2-4 PM	8.1	22.6	4.6	-	-	-
	9-11 PM	8.2	21.9	5.1	-	-	-
APTS V	8-10 AM	7.2	22.7	4.6	-	-	-
(6/05/91)	2-4 PM	7.4	23.1	4.7	-	-	-
	9-11 PM	7.3	22.6	5.2	-	-	-

[a] aeration motor found unplugged

Table A24. Nitrate, Ammonia and TKN from Field System without Attenuation of Laundry Effluent (mg/L), (from Kellam *et al.*, 1992)

SITE & DATE	TIME OF DAY	PRIMARY SETTLING CHAMBER NITRATE (mg/L)	PRIMARY SETTLING CHAMBER AMMONIA (mg/L)	PRIMARY SETTLING CHAMBER TKN (mg/L)	EFFLUENT NITRATE (mg/L)	EFFLUENT AMMONIA (mg/L)	EFFLUENT TKN (mg/L)
APTS I	8-10 AM	1.6	13.9	12.4[a]	8.4	2.4	3.9
(7/12/90)	2-4 PM	0.0	6.3	7.1	8.6	2.2	10.3
	9-11 PM	0.1	7.6	9.2	9.8	2.6	24.2
APTS II	8-10 AM	18.0	4.6	154.5	23.3	0.5	4.5
(9/02/90)	2-4 PM	20.4	4.1	145.5	24.3	0.5	6.7
	9-11 PM	0.2	14.7	970.0	30.1	1.1	16.0
APTS III	8-10 AM	10.8	10.1	124.5	18.5	3.3	9.8
(9/23/90)	2-4 PM	15.6	5.1	9.9	20.4	3.4	4.5
	9-11 PM	10.8	6.6	10.3	18.5	3.7	4.5
APTS IV	8-10 AM	0.0	4.6	5.8	3.6	0.2	3.1
(10/21/90)	2-4 PM	0.0	2.4	4.5	3.6	0.2	4.3
	9-11 PM	4.2	1.7	4.9	3.7	0.1	4.3
APTS V	8-10 AM	13.6	2.0	111.2	17.9	0.3	7.6
(10/06/90)	2-4 PM	15.0	0.9	102.3	18.2	0.1	4.5
	9-11 PM	12.7	1.2	59.1	18.1	0.3	8.2
APTS I	8-10 AM	0.3	20.8	22.6	0.4	13.1	19.1
(12/01/90)	2-4 PM	0.4	15.8	29.3	0.4	14.0	17.6
	9-11 PM	0.4	19.0	24.2	0.4	11.7	54.3
APTS II	8-10 AM	3.2	21.7	271.0	19.6	11.4	22.3
(2/04/91)	2-4 PM	12.1	30.5	210.0	22.3	18.4	15.3[a]
	9-11 PM	8.0	43.9	381.0	22.3	17.8	13.0[a]
APTS III	8-10 AM	2.3	56.3	120.1	2.9	54.5	72.7
(1/05/91)	2-4 PM	2.0	52.1	96.1	3.0	54.9	77.7
	9-11 PM	2.5	54.5	55.9	3.0	56.3	93.4
APTS IV	8-10 AM	1.6	30.4	33.9	1.6	27.4	31.6
(1/18/91)	2-4 PM	1.6	30.3	36.2	1.6	26.6	31.8
	9-11 PM	1.6	24.2	30.4	1.0	28.6	33.4
APTS V	8-10 AM	8.0	1.8	11.2	13.4	0.3	6.4
(2/27/91)	2-4 PM	17.9	0.5	10.6	14.8	0.6	5.3
	9-11 PM	24.2	2.4	12.5	20.0	0.6	4.3

continued

Table A24. continued

SITE & DATE	TIME OF DAY	PRIMARY SETTLING CHAMBER			EFFLUENT		
		NITRATE (mg/L)	AMMONIA (mg/L)	TKN (mg/L)	NITRATE (mg/L)	AMMONIA (mg/L)	TKN (mg/L)
APTS I	8-10 AM	180.3	25.5	25.0[a]	0.3	15.2	15.0[a]
(7/12/90)	2-4 PM	115.3	15.6	21.5	0.3	15.0	13.5[a]
	9-11 PM	19.0	15.0	19.5	0.3	17.5	14.0[a]
APTS II	8-10 AM	0.0	25.2	47.0	0.3	21.4	18.5[a]
(9/02/90)	2-4 PM	0.0	21.7	87.5	0.3	21.0	21.5
	9-11 PM	0.0	16.2	34.0	0.4	19.3	18.5[a]
APTS III	8-10 AM	0.1	58.1	430.0	2.7	52.4	45.5[a]
(9/23/90)	2-4 PM	0.1	60.0	145.0	1.8	54.4	51.5[a]
	9-11 PM	0.2	56.2	135.1	2.3	56.2	55.0[a]
APTS IV	8-10 AM	0.4	34.9	45.5	0.2	33.2	32.0[a]
(10/21/90)	2-4 PM	-	-	-	0.3	34.9	41.0
	9-11 PM	-	-	-	0.4	34.5	43.0
APTS V	8-10 AM	2.3	8.3	15.0	5.9	5.6	9.0
(10/06/90)	2-4 PM	1.1	8.5	10.0	4.4	6.6	5.5[a]
	9-11 PM	1.1	9.5	11.5	4.1	6.2	6.5

[a] Denotes impossible reading, TKN < Ammonia; more confidence with ammonia reading since that test is calibrated and more direct.

Table A25. Orthophosphate and Total Phosphate from Field System without Attenuation of Laundry Effluent (mg/L), (from Kellam *et al.*, 1992)

SITE & DATE	TIME OF DAY	PRIMARY SETTLING CHAM ORTHO-P (mg/L)	PRIMARY SETTLING CHAM TP (mg/L)	EFFLUENT ORTHO-P (mg/L)	EFFLUENT TP (mg/L)
APTS I	8-10 AM	4.3	164.0	3.4	6.1
(7/12/90)	2-4 PM	1.7	3.6	3.6	7.6
	9-11 PM	2.6	8.4	3.6	6.6
APTS II	8-10 AM	6.1	152.8	5.8	8.4
(9/02/90)	2-4 PM	7.0	139.0	6.0	7.2
	9-11 PM	8.8	542.5	6.1	10.5
APTS III	8-10 AM	3.0	11.2	1.9	5.6
(9/23/90)	2-4 PM	2.3	7.3	2.1	6.6
	9-11 PM	2.2	11.4	2.1	23.5
APTS IV	8-10 AM	4.8	5.6	3.8	4.5
(10/21/90)	2-4 PM	4.6	8.4	3.9	4.5
	9-11 PM	4.5	-	3.9	4.4
APTS V	8-10 AM	5.4	5.9	5.5	5.5
(10/06/90)	2-4 PM	5.3	5.6	5.5	5.5
	9-11 PM	5.6	5.8	5.6	5.6
APTS I	8-10 AM	3.3	5.6	5.0	7.8
(12/01/90)	2-4 PM	3.0	8.9	5.0	8.1
	9-11 PM	7.0	15.0	4.9	8.0
APTS II	8-10 AM	6.4	255.2	5.8	11.2
(2/04/91)	2-4 PM	5.7	195.2	5.6	11.5
	9-11 PM	6.6	421.7	5.6	9.7
APTS III	8-10 AM	3.3	9.2	3.3	5.6
(1/05/91)	2-4 PM	2.9	13.4	3.4	13.0
	9-11 PM	3.0	13.6	3.2	11.1
APTS IV	8-10 AM	6.1	10.2	5.6	9.1
(1/18/91)	2-4 PM	5.5	9.0	5.5	9.4
	9-11 PM	3.4	4.4	5.7	9.2
APTS V	8-10 AM	4.1	6.6	3.6	6.1
(2/27/91)	2-4 PM	3.8	4.6	3.8	2.7[a]
	9-11 PM	3.8	7.5	3.8	4.1

continued

Table A25. continued

SITE & DATE	TIME OF DAY	PRIMARY SETTLING CHAM ORTHO-P (mg/L)	PRIMARY SETTLING CHAM TP (mg/L)	EFFLUENT ORTHO-P (mg/L)	EFFLUENT TP (mg/L)
APTS I (5/12/91)	8-10 AM	3.0	5.2	0.0	4.9
	2-4 PM	1.7	5.9	0.0	3.6
	9-11 PM	1.7	6.1	0.0	4.8
APTS II (5/30/91)	8-10 AM	6.2	19.9	5.8	7.0
	2-4 PM	5.9	62.4	5.6	7.0
	9-11 PM	6.6	13.0	5.2	5.6
APTS III (5/11/91)	8-10 AM	7.3	219.4	2.7	4.1
	2-4 PM	3.3	58.1	2.7	6.3
	9-11 PM	3.3	41.7	2.8	3.9
APTS IV (5/26/91)	8-10 AM	4.3	6.9	4.2	6.4
	2-4 PM	-	-	4.4	7.0
	9-11 PM	-	-	4.5	3.2[a]
APTS V (6/05/91)	8-10 AM	6.2	4.2[a]	5.8	7.2
	2-4 PM	6.4	3.0[a]	6.0	5.3[a]
	9-11 PM	6.5	4.9[a]	5.9	5.1[a]

[a] Denotes impossible reading, TP < Ortho-P; more confidence is placed in ortho-P reading since that test is calibrated and more direct.

Vita

The author was born in Tempe, Arizona in 1964. He was raised in Falls Church, Virginia, were he graduated from Falls Church High School in 1982. He received a Bachelors of Science degree in Agricultural Engineering in 1987 from Virginia Polytechnic Institute and State University.

The author worked as a Full Time Volunteer in rural eastern Kentucky, as an Extension Technician at VPI&SU, and as an Environmental Planner for the Northern Neck Planning District Commission before returning to VPI&SU to pursue a Masters of Science degree in Environmental Engineering. He completed his M.S. in May of 1993.

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