Anaerobic Treatment of Low Strength Wastewater

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ABSTRACT

The anaerobic filter has been shown to be able to treat diluted domestic wastewater at ambient temperature. However, the production of ammonia and hydrogen sulfide as by-products often prevents the effluent from meeting secondary treatment requirements set by the PL92-500. In this study, the performance of an anaerobic filter followed by an aerobic filter was evaluated. Domestic wastewater was used in continuously fed pilot scale reactors (2' X 8') to achieve a hydraulic retention time of 24 hours and an organic loading of 10 lbCOD/1000 cu ft/day. Both biological and physical responses of the reactors were monitored.

Based on the data collected in the pilot studies, a suggested design and costs for an anaerobic filter unit were developed. These costs were compared with costs of other small treatment alternatives such as pressure sewers. Results of this study show significant promise of the anaerobic filter process for on-site treatment of domestic wastewater and other low strength wastes.

CHAPTER 1.

INTRODUCTION

Wastewater treatment is essential to maintain environmental quality and to protect public health. Most treatment systems in this country are highly centralized and developed, requiring the use of large and sophisticated treatment processes such as activated sludge, digestion, trickling filters, and clarification. The operational cost of these facilities can be quite high. At this time, the total energy consumed in treatment costs more than $130 million annually (Jewell, 1979). If secondary and tertiary treatment are required, then this amount may be increased by nearly 10 times. This is an energy equivalent of 15.9 billon BTU per year (100 million barrels/yr) of oil, or about 1% of total U.S. energy consumption (Jewell, 1979). Wastewater from 10,000 people, treated aerobically, can generate 500 kg/day of sludge that is also difficult to dispose. Therefore, even though these wastewater treatment systems may be optimal for large cities where land is expensive, the cost may not justify the benefit for other areas, and rural areas in particular.

The current energy situation has stressed the need for more energy conservation as well as more domestic energy sources. A recent study by Stobaugh (1979) concluded that energy conservation provides the only practical short-term solution to reducing oil imports and becoming more energy self-sufficient. Due to the large capital costs, increased energy consumption, and the sometimes questionable treatment efficiency that results, alternative wastewater processing methods are sought. This is the primary stimulus for investigating the potential of using

anaerobic wastewater treatment.

Anaerobic treatment is ideal in the respect that the process allows energy conservation, and normally produces a usable by-product, methane gas. The advantages of anaerobic treatment can be best summarized as the following five points: 1) A high degree of water stabilization approaching 80 to 90 percent has been reported; 2) Biogas containing 55% to 65% methane, with a heating value of 500-600 BTU/cu ft is produced; 3) Low production of waste biological sludge, because anaerobic metabolism is inefficient; 4) Low nutrient requirements as all needs of the microorganisms are proportionally reduced, and 5) No oxygen requirements, thus removing the need of expensive and energy consumptive aeration systems. A list of the advantages and disadvantages of the anaerobic process is summarized in Table I.

Although the concept of the modern anaerobic filter has been studied for approximately 15 years, most of the work has been on laboratory scale systems, with little pilot or full-scale information being developed. Of these, very few addressed were low strength domestic wastewater treatment. Anaerobic treatment has not been used in low strength waste stabilization for many reasons. Primarily, it is because of our ability to obtain 'cheap energy', which reduces the incentive to develop conservative measures. Other reasons include a general misunderstanding or lack of knowledge of anaerobic processes.

Historically, anaerobic systems have been used to stabilize solids produced by aerobic processes such as the activated sludge process. The process has always been restricted to treatment of wastes with high potential for gas production, as some type of process heating was

Table 1 Characteristics of Anaerobic Process

ADVANTAGES:

- Less production of waste biological solids
- Conversion of biodegradable COD into methane, a useful by-product
- Low nutrient requirements
- No aeration equipment required
- High degree of waste stabilization

DISADVANTAGES:

- Relatively long periods are required to start-up the process
- Sensitivity to variable loads and possible toxicity problems
- Little practical experience has been gained in full scale operations
- Anaerobic processes have been traditionally limited to pretreatment applications; additional treatment could be required to meet most water quality standards
- Temperature sensitivity

usually required. The conventional anaerobic process also requires a long hydraulic detention time. However, significant advances on a pilot scale have been made, extending the process to successfully treat domestic wastewater at ambient temperature. At this time, the main drawbacks are the production of hydrogen sulphide as a by-product, and failure to remove ammonia. These short-comings often make the effluent unsuitable for direct discharge to the environment.

The objective of this project is to develop a low cost, energy producing, and technically simple wastewater treatment process which will be capable of treating wastewater for recycling or discharge to the environment. A key constraint of the treatment process is that it must be reliable and can be safely operated by individuals who have no technical expertise. The treated wastewater quality is to be sufficiently high to allow it to be used for secondary recycling (non-potable) purposes such as irrigation, industrial use or groundwater recharge, as it is anticipated that water recycling will play a major role in meeting California's upcoming water shortages. To understand the severity of the State's water problem, it is useful to review California water resources and demand. Asano, Girelli, and Wasserman (1979) have compiled some useful statistics projecting a 6.2 million acre-ft of groundwater overdraft by the year 2000.

In this report the next step in the logical development of the anaerobic filter is examined. A combination of anaerobic/aerobic filter systems is tested on low strength domestic wastewater. Raw wastewater is treated through two columns; first anaerobic and then aerobic. The latter is for the removal of the ammonia concentration

and the reduction of hydrogen sulfide by oxidizing the by-products to sulphur dioxides and nitrates. The objective is to obtain secondary treatment effluent standards. At the conclusion of this paper, the technology to implement the anaerobic/aerobic filtration system is developed.

CHAPTER 2.

LITERATURE REVIEW

2.1 Anaerobic Treatment Process

Anaerobic treatment requires a unique microbial balance of the fast growing acid-forming bacteria and the slower growing methane-producing bacteria. In general, the efficiency of the anaerobic processes is dependent on the solids retention time (SRT). The required solids retention time controls the specific growth rate of the organisms desired, with slower growing organisms requiring a longer solids retention time. Theoretically, this is equal to the residence time of the biomass in the reactor.

Development studies on anaerobic treatment were conducted with the emphasis on four points that were thought to limit the application of anaerobic methane fermentation: (1) ineffectiveness when treating low strength wastewaters; (2) inability to be used at low temperatures (less than 95°F); (3) sensitivity to shock loadings; and (4) sensitivity to toxic materials. These restrictions exist because the growth rate of the methane bacteria is the controlling step. Consequently, the anaerobic treatment process has always been restricted to high strength wastes, which can be economically heated. Thus, most anaerobic processes are operated at elevated temperatures in mesophilic (95-100°F) or thermophilic (125-130°F) ranges. The methane gas produced by the process is normally used for heating to achieve efficient and economical operation. This situation is changed by two significant developments: the anaerobic contact process by Fullen (1953), and the anaerobic filter process, originally proposed by Coulter (1957) with developments

by McCarty (1964, 1968).

The anaerobic processes available to date include the conventional, completely-mixed digester, the anaerobic contact process, the anaerobic filter, the anaerobic fixed-film expanded-bed process and the anaerobic sludge blanket process. A schematic flow diagram of each system is shown in Figure 1. The conventional digester is the most common anaerobic process with its use mainly in digestion of municipal sludges. The process is designed as a complete mixed reactor. However, its use is limited since the hydraulic retention time (HRT) is equal to the solids retention time, resulting in low volumetric loading rates and large reactor volumes. The minimum practicable solids retention time (SRT) required for adequate digestion and process stability is in the range of 10-15 days at 95°F.

The first significant improvement was the anaerobic contact process. In this process, reactor effluent solids are recycled, which allows the solids retention time to be controlled by the amount of solids wasted, and not by hydraulic retention time. By so doing, a reduction in the hydraulic retention time is made possible. The pilot studies on the anaerobic contact process were first reported by Fullen (1953) and later in more detail by Schreopfer, et al. (1955, 1959). Steffen and Bedker (1961) reported a full-scale anaerobic contact process treating a meatpacking waste at a loading of 2.5 kg/cu m day BOD with hydraulic retention time of 13.3 hr producing a 90% BOD removal efficiency. The primary disadvantage of the contact process was the need for a degasifier in order to recycle reactor effluent solids.

The two processes that allow high solids retention time without

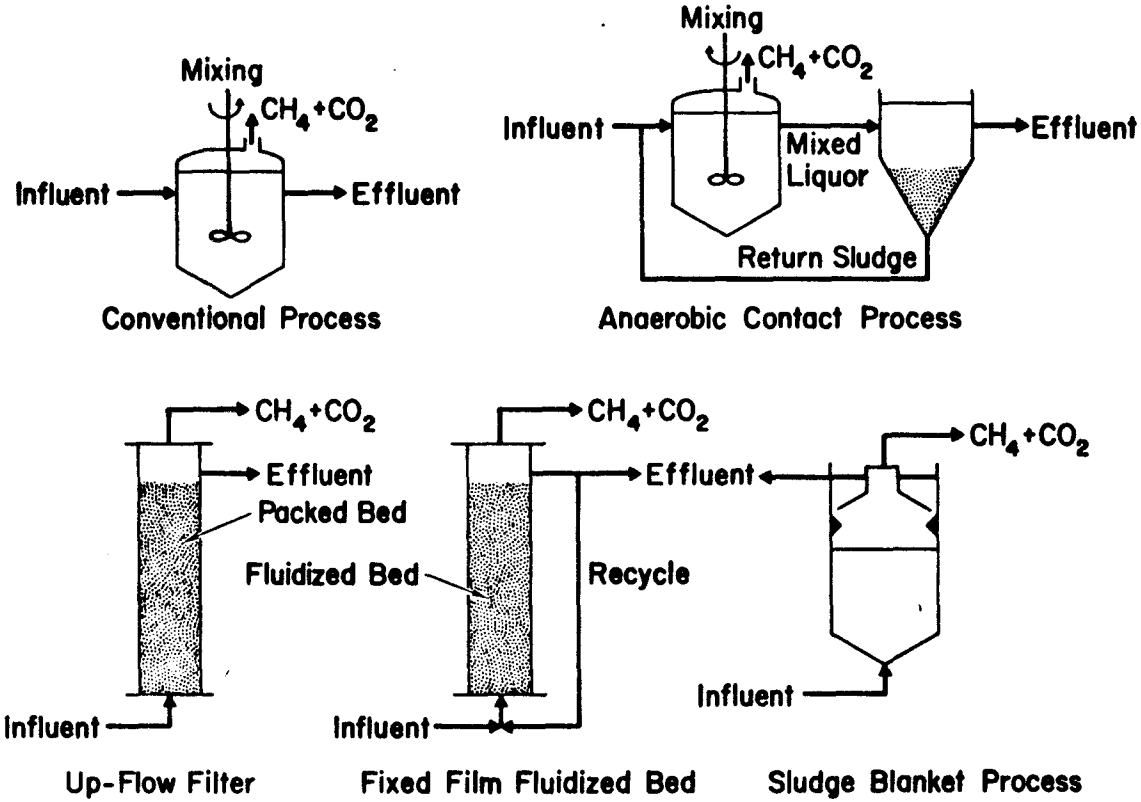


Figure 1. Various Anaerobic Treatment Processes

clarification are the anaerobic filter and the fluidized-bed upflow reactor. The former retains biomass on the surfaces of rocks or other fixed media and the latter on small particles which are kept in suspension by recycling effluent. The latest process developed by Lettinga and his co-workers (1980) in Holland is the upflow sludge blanket process. This relies on the development of a highly concentrated bed (up to 80 gm/l) of microorganisms. A large biomass population is possible due to the gas collector/separator which sits above the reactor. In the pilot plant studies, organic loadings of 15 to 40 kg COD/cu m/day at residence times of 3-8 hours were reported. Frostell (1980) evaluated both upflow sludge blanket process and the filter system. He found 90% COD removal for both systems at organic loading rates up to 10 kg COD/cu m/day. The results also indicated that the anaerobic filter was superior, although the differences were small.

2.2 Development of the Anaerobic Filter

The anaerobic filter process was first developed by Coulter (1957) but was virtually forgotten until 1969 when Young and McCarty (1969) renewed interest by demonstrating the process's ability to treat a medium to high strength carbohydrate/protein wastewater. Young and McCarty demonstrated the importance and potential of the anaerobic filter process by successfully treating a medium strength (1500-6000 mg/l COD) synthetic waste at 25°C, at loading rates ranging from 0.06 to 0.212 lb COD/cu ft of filter volume.

The theory and kinetics of the anaerobic filter process were examined by several investigators. The original work of McCarty has been extended. Mueller (1975) presented a mathematical model incor-

porating a two stage reaction sequence. Two types of reaction kinetics were analyzed with his model. One employing Monod kinetics and the other, first-order kinetics. Both were shown to adequately approximate the filter performance. A model formulated by DeWalle and Chian (1976) showed that at high substrate concentrations, the substrate removal rate is proportional to the square root of the substrate concentration used and the specific area of the filter medium. Jennings (1976) developed a mathematical model for percent removal of a nonabsorbable biogradable substrate in a submerged biological filter. He used non-linear Monod expression for the substrate utilization rate and theoretically investigated the effects of diffusion in a plug flow reactor under steady-state conditions.

The performance of the anaerobic filter has also been tested on a variety types of wastewater. This includes landfill leachates, high strength acid wastewater by Chian (1977), heavy metals by DeWalle (1979), wheat starch-gluten plant waste by Taylor (1972), pharmaceutical wastes by Jennett and Dennis (1975), shellfish processing wastes by Hudson, et al. (1978), brewery wastes by Lovan and Fores (1971), food processing wastes by Plummer, et al. (1968) and the removal of organic particulates by Morris (1981). Almost all of these investigations were directed at the treatment of medium to high strength wastes (1000 mg/l or greater COD). Table 2 summarizes the recent findings of investigations using anaerobic filtration.

There have been other investigations of the anaerobic filter to further ascertain its usefulness. The pH tolerance of anaerobic digestion was presented by Clark and Speece (1970) while Shafie and Blood-

Table 2 Studies Using Anaerobic Filtration

Waste	Scale and Temperature	Organic Loading Rate (lb COD/1000 cu ft day)	Efficiency %	Retention Time (hr)	Reference
domestic sewage 179 mg/l BOD	lab scale, 9ℓ 25°C	103	50-65	2.5-8.5	Coulter (1957)
food processing 8500 mg/l	lab scale 11-16" high 35°C	100-640	30-68	13-83	Plummer (1968)
sodium acetate 6000 mg/l COD pH tolerance	lab scale 3' X 5.5" 33°C	750	-	12	Clark (1970
brewery press 6000-24000 mg/l	lab scale 6" X 6" 34°C	50	90	15-330	Lovan (1971)
domestic sewage 500 mg/l	lab scale, 2.5l 20°C	0.03	90	24-45	Pretorious (1971)
wheat starch 8800 mg/l	full scale	237	64	22	Taylor (1972)
metrecal 10000 mg/l COD multi-stages	lab scale 18" X 5.5" 30°C	2560	>70.5	13-18	El-Shafie (1973)
synthetic organic alcohols, acids, amines, phenol 2000 mg/l	lab scale 25" X 35" 34°C	35-130	64-76	17-46	Hovius (1972)
petrochemical	Pilot scale	40-145	10-13	72	Hovius (1972)

(Table 2 Continued)

Waste	Scale and Temperature	Organic Loading Rate (lbCOD/1000 cu ft day)	Efficiency %	Retention Time (hr)	Reference
pharmaceutical 95% methanol 1250-16000 mg/l	lab scale 3' X 5.5" 37°C	14-220	94-98	12-48	Dennis (1974)
sulfite liquor 1300-5300 mg/l	lab scale 19' X 5.7" 35°C	125-375 BOD	27-58	89-95	Meuller (1975)
potato processing 3000 mg/l	lab scale 4' X 8' 19-22°C	33-145	41-79	13-59	Mueller (1975)
high strength leachate 6200-62000 mg/l	lab scale 8' X 8" room temp.	80	94-98	4.3-74	Chian (1976)
leachate from solid waste landfill 30000 mg/l	lab scale 7' X 7" 25°C	49	95	84	DeWalle (1976)
effluent from heat treatment of acti- vated sludge 9500 mg/l	lab scale 6' X 5.5" 32°C	300	76	48	Haug (1977)
synthetic wastewater 3800-9400 mg/l	lab scale 33°C	27-141	68-94	162-816	Norrman (1977)
shellfish process wastewater 466-121 mg/l	lab scale 5' X 6" 918-26°C	2-23	46-81	8-74	Hudson (1978)
domestic sewage 134-209 mg/l	5' X 10' 13-25°C	17-38 BOD	45-55	12-48	Koon (1979)
sewage 60-220 mg/l	pilot scale 18.3' X 5' 15-20°C	3-38 BOD	55	2.5-10.5	Genung (1979)

(Table 2 Continued)

Waste	Scale and Temperature	Organic Loading Rate (lbCOD/1000 cu ft day)	Efficiency %	Retention Time (hr)	Reference
bean blanching waste 600 mg/l VA	lab scale 3.6' X 3" 35°C	5.3-936 volatile solids	83-93	2.5-62.4	Van den Berg (1979)
leachate 11628 mg/l heavy metal removal	lab scale 8' X 8" room temp.	---	75	4.2-34	DeWalle (1979)
primary domestic sewage	lab scale 1.6' X 2.5" 10-30°C	62.4-1498	--	0.13-0.5	Jewell (1979)
synthetic wastewater 32.3-45 mg/l	lab scale 24°C	5-9 lb NH_3 N/day/1000cf	75-79	0.14-0.3	Young (1979)
synthetic wastewater 6000 mg/l	lab scale 6' X 6" 25°C	26.5-212	79-98	4.5-72	Young (1979)
guar 9140 mg/l	full scale 30' X 40' 36.6°C	470	60	24	Witt (1979)
acetate + formate + 2-butanone 5000-10000 mg/l	lab scale	380-500	86-94	--	Witt (1979)
formate + actate + methanol + formaldehyde 17999-24000 mg/l	lab scale	690-910	72-92	--	Witt (1979)
acrylic acid + acrylate esters 79000-85000 mg/l	lab scale	500-600	94-97	--	Witt (1979)
evaporated milk	lab scale	450-550	80-90	--	Witt (1979)

(Table 2 Continued)

Waste	Scale and Temperature	Organic Loading (lbCOD/1000 cu ft day)	Efficiency %	Retention Time (hr)	Reference
acetate + aldehyde + glycol + vinyl adetate 700-10000 mg/l	lab scale	380-500	86-94	--	Witt (1979)
synthetic substrate t0-600 mg/l	lab scale, 1L 10, 20, 30°C	500	80	0.3-6	Switzenbaum (1980)
glucose, domestic sewage compare aerobic & anaerobic	lab scale 22°C	624.2	80	0.16-0.5	Jewell (1980)
palm oil sludge 45000 mg/l	pilot scale 20L 50°C	--	5	--	Chin (1980)
heat treat liquor 10750 mg/l	11.5' X 2" 35°C	97-590	17-68	16.5-98	Donovan (1981)
synthetic wastewater 8700 mg/l	lab scale 6.7L 30°C	43.7-62	79-93	7.2-29	Frostell (1981)
primary settled sewage 186 mg/l	1.6 X 2.5" 20°C	40.6-2184	45-80	0.08-3	Jewell (1981)
synthetic substrate 333-1833 mg/l	lab scale, 1L 30°C	300-500	75-85	1.66	Morris (1981)
synthetic substrate 200-1000 mg/l phenol	lab scale	31.2-582	80-97.5	9.3	Khan (1981)
domestic sewage 44-573 mg/l	lab scale	3-34	76	24	Kobayashi (1981)

good (1973) investigated the progressive breakdown of a synthetic waste to volatile fatty acids by analyzing samples from a multiple upflow filter system.

2.3 Anaerobic Filter for Low Strength Wastewater

As indicated previously, anaerobic treatment has not been regarded as an effective process for the treatment of low strength wastes, those with a biochemical oxygen demand (BOD) of less than 1000 mg/l such as domestic wastewater.

The key concept that allows the application of anaerobic process to low strength wastewater lies in the ability to control solids retention time independently of the hydraulic retention time. Since heating is not required at lower waste temperatures due to long SRT, low strength wastes, which produces only small quantities of gas per unit volume of incoming waste, can now be effectively treated. This feature permits anaerobic treatment at lower temperatures than previously thought economically feasible.

Treatment of low strength wastes, such as domestic wastewater, was first investigated by Coulter (1957). He showed an 84% of solids and over 50% of BOD removals in a 1640 gallon pilot plant at Loveland, Ohio. Pretorius (1971) tested a modified digester/anaerobic filter with a multi-media, size-graded upflow static filter. A 36 to 52 percent soluble COD reduction at a retention time of 24 hours was achieved. Jeris (1974, 1975, 1977) demonstrated that a fluidized bed type of process is effective in wastewater treatment for the removal of nitrates, BOD and nitrogen, while Leuschner (1976) showed that an anaerobic attached film was capable of treating dilute wastes at relatively short

retention times at ambient temperature. Conceptual designs for processing up to 1MGD of raw sewage were presented by Genung et al. (1979) based on the performance of these anaerobic filter pilot plant investigations.

At the same time, anaerobic fixed-film expanded bed (AAFEB) was investigated by Jewell et al. (1979, 1980, 1981). Even at hydraulic retention periods shorter than 30 minutes it was shown that most of the biodegradable organics were removed from a dilute primary settled wastewater, leaving less than 40 mg/l for the total COD and 5 mg/l suspended solids in the effluent. Maximum organic removal efficiency occurred up to an organic volumetric loading of 4 kg/cu m of reactor per day. Switzenbaum (1980) presented two simplified first-order equations relating the process efficiency to the net specific growth rate of the anaerobic film and specific substrate utilization rate. A list of the latest studies using anaerobic treatment for low strength wastewater is presented in Table 3. All the investigations have shown that the anaerobic filter can be used at ambient temperatures to produce treated wastewaters that are approximately equal in quality to secondary effluent conditions standards defined by PL92-500 (greater than 85% BOD removal, or less than 30 mg/l BOD and TSS, whichever is less), except for the by-products of hydrogen sulfide and ammonia.

The latest studies on the combination of aerobic/anaerobic systems include studies by Norrman (1980) on treatment of wastewater from food and fermentation industries, Parker (1981), on food processing waste by anaerobic/aerobic lagoons, McFarlane (1980) on anaerobic photosynthetic lagoons for fellmongery wastewater and Lindley (1981) on milking center

Table 3. Recent Studies of Anaerobic Treatment on Low Strength Wastewater

Investigator	Type	Waste Strength	Efficiency
Coulter (1957)	Anaerobic Contact Process	180 mg/l BOD	50%
Pretorious (1971)	Modified Digester/Filter	500 mg/l COD	90%
Jeris (1974, 1975, 1977)	Fluidized Bed	24,45 ($NaNO_3$); 24 mg/l BOD	86,99,93%
Genung (1979)	Anaerobic Attached Film	220 mg/l BOD	55%
Koon (1979)	Up-flow Fixed Film	200 mg/l BOD	40-55%
Jewell (1979, 1980, 1981)	AAFEB	186 mg/l COD	45-80%
Switzenbaum (1980)	AAFEB	50-600 mg/l COD	40-90%
Kobiyashi (1980)	Anaerobic Filter	44-570 mg/l BOD	70%
This Study (1981)	Anaerobic Filter	90 mg/l BOD	90%

*AAFEB - Anaerobic Attached-Film Expanded Bed Process

waste. BOD removals of over 90% and nitrogen 92% were reported.

In order to upgrade anaerobic filter effluent to meet secondary standards, an experimental investigation using a second filter, operated aerobically, was made. That is, the anaerobic filter was followed by a small aerobic filter. Alternatively a biodisc (RBC) could have been used. This has been shown to be suitable for secondary treatment standards by Stenstrom and Chan (1979), without final clarification.

CHAPTER 3

THEORETICAL CONSIDERATION OF ANAEROBIC TREATMENT PROCESS

3.1 Introduction

Despite widespread use of the anaerobic treatment process, optimum performance is seldom achieved. A high degree of empiricism still prevails in the design and operation of such systems. It is therefore essential to have a rational basis for process analysis so that the full potential of anaerobic treatment can be realized.

Prior to the widespread use of digital computers, most of the mathematical models for the description of biological processes were steady-state models. It was not until 1969 when Andrews (1969) published his work on dynamic modelling of anaerobic digestion that the commonly observed performance of the process during start-up or failure could be predicted. Such models are useful in obtaining a semi-quantitative measure of process stability and control which, in turn, is of considerable importance in selecting a process for a specific application.

The success of the mathematical modelling approach depends heavily on the understanding of the microbiology and biochemistry of the anaerobic process. In considering the application of process kinetics to anaerobic system design, this section describes: 1) anaerobic pathways and 2) the kinetic model employed.

I Description of Anaerobic Pathways

Methane producing bacteria are very difficult to isolate and study; consequently, relatively little was known of their basic biochemistry until recently. In the conversion of organic matter into

methane, tracer studies have indicated that the major sources of methane are from acetic acid cleavage and carbon dioxide reduction as shown in Table 4. Figure 2 presents pathways elucidated by studies in anaerobic fermentation of complex wastes. Anaerobic treatment of complex organic materials has been considered to be a two-stages process, The first stage being production of short-chain fatty acids and the second stage being methane production

In the case of digestion of insoluble material, e.g. sludges, they must first be solubilized or reduced in size to facilitate transport across the cell membrane. The reactions responsible for these are usually hydrolytic and are catalyzed by enzymes which have been released to the medium by the bacteria. The small molecules which are produced can then be used as carbon and energy sources by bacteria which carry out fermentations. The final end products of the acid fermentation step are primary short-chain volatile acids such as acetic, propionic, etc.. Their production is referred to as acidogenesis and the responsible organisms are called acid-forming bacteria. In addition, there exist other groups of acid formers which possess a specialized enzyme system that allows them to oxidize reduced compounds without using an organic electron acceptor. They release hydrogen gas. The collective activity of these hydrogen-producing bacteria is called hydrogenesis.

During the second stage, the hydrogen and the organic acids are converted by methanogenic bacteria into gaseous end products: carbon dioxide and methane. There are several different groups of methane formers; each group is characterized by its ability to ferment a

Table 4 Major Mechanisms of Methane Formation

I Acetic Acid Cleavage:

$$C^{*}H_3COOH \longrightarrow C^{*}H_4 + CO_2$$

II Carbon Dioxide Reduction:

$$CO_2 + 8H \longrightarrow CH_4 + 2H_2O$$

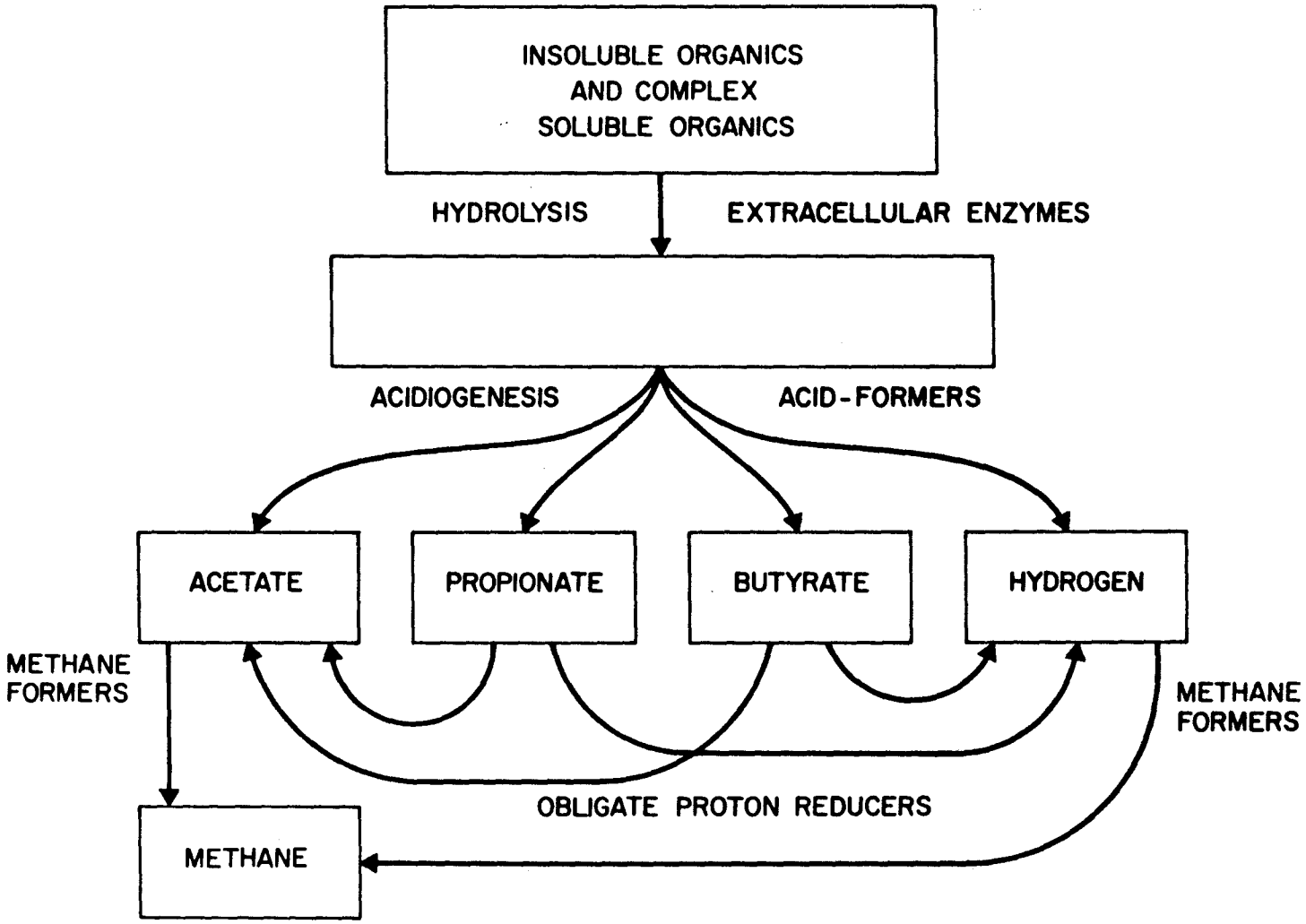


Figure 2. Pathways

relatively limited number of organic compounds. Factors favoring methane production are listed in Table 5.

The production of methane from complex organic compounds involves the interaction of many microorganisms. Due to the narrow substrate range of the methanogenic bacteria, researchers are in support of the theory that there exists a mutualistic relationship between the methanogens and the nonmethanogens. Some of the latest research on methanogenesis has been investigated by Mah and his co-workers (1977, 1978a, 1978b). They have shown that acetate can be used as the sole carbon source for methane production.

Methane production may be estimated from waste strength by the use of the following experienced formula (McCarty, 1964):

$$C = 5.62(eF-1.42A) \quad \text{.....} \quad (1)$$

where C = cubic feet of methane produced per day, STP

e = efficiency of waste utilization, (%)

F = pounds of BOD added per day, (lb/day)

A = pounds volatile biological solids produced per day, (lb/day)

The value of 5.62 is the theoretical methane production from stabilization of one pound of BOD. The constant 1.42 is the factor for conversion of pounds of volatile biological solids to ultimate BOD.

Methane and CO_2 are produced at equal molar rates; however, the biogas of a properly operating anaerobic digester almost always contains in excess of 50% methane. This is due to the high solubility of CO_2 and the low solubility of methane. A major fractions of the gaseous CO_2 becomes dissolved in the digesting liquid, producing carbonic acid.

Table 5 Factors Favoring Methane Production

1. Temperature control within 1 to 2°C of operating temperature.
2. Adequate mixing.
3. Frequent addition of feed solids.
4. Maintenance of pH within range 6.4 to 7.6.
5. Anaerobic conditions: $O_2 = 0$.
6. Sufficient nutrients.
7. Absence of toxic materials
   a. Heavy metals (copper, zinc, nickel and hexavalent chromium): < 1 mg/L.
   b. Soluble sulfides (HS^-): < 100 mg/L.
   c. Free ammonia (NH_3): < 150 mg/L.
   d. Alkali and alkaline earth metals (Na^+, K^+, Ca^{2+}, Mg^{2+}): < 1000 mg/L.

The carbonic acid is neutralized by the alkalinity for high rate digestion liquid, and the major component of the alkalinity for high rate digestion is usually ammonia. The normal biogas composition of a conventional digester is approximately 65% methane and 35% carbon dioxide, with traces of nitrogen and hydrogen sulfides. For lower systems operating at lower rates, the methane percent can become much higher. For example, Kobayashi, et al. (1981) found biogas compositions of 92-98% methane for a low loading rate anaerobic filter. Figure 3, which was adapted from McCarty (1964) shows the relationship of alkalinity, pH and carbon dioxide percent. Anaerobic filters treating low strength waste usually operate in very low alkalinity, which results in high methane composition in the biogas.

II Basic Kinetic Equations

The relationship between microbial growth and substrate utilization can be formulated in two basic equations. Equation 2 describes the relationship between the net ratio of growth of microorganisms and the rate of utilization of the growth limiting substrate.

$$\frac{dX}{dt} = \mu X - KdX \qquad (2)$$

where $\frac{dX}{dt}$ = net growth rate of microorganisms per unit volume of reactor, (mass/vol-time)

μ = Monod specific growth rate, (1/time)

Kd = microorganism decay coefficient, (1/time)

and X = microbial mass concentration, (mass/volume)

Equation 3 relates the rate of substrate utilization to the maximum growth rate and the concentration of the growth limiting substrate

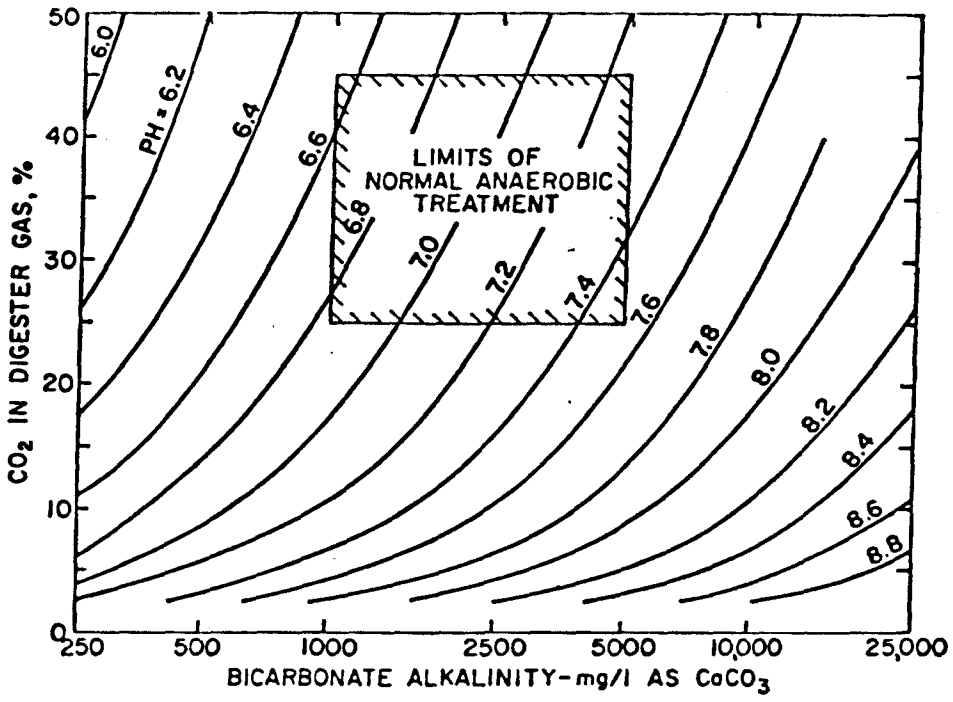


Figure 3 Relationship Between pH and Bicarbonate near 95°F. (McCarty, 1964)

surrounding the organisms.

$$\mu = \frac{\hat{\mu}S}{K_s+S} \tag{3}$$

$\hat{\mu}$ = maximum rate of substrate utilization per unit weight of microorganism, (1/time)

S = concentration of the growth limiting substrate surrounding the microorganisms, (mass/volume)

and K_s = half saturation coefficient, that is the substrate concentration at half the maximum growth rate ($\hat{\mu}/2$), (mass/volume)

One critical assumption in this model is that the microbial growth is assumed to be limited by the availability of one substrate and that all other nutrients are present in excess amounts.

From these equations, the design parameters for the steady-state operation of biological systems can be developed. These parameters include specific utilization rate (U), i.e., the rate of substrate utilization per unit mass of microorganisms and biological solids retention time (SRT), which is approximately equal to the reciprocal of net specific growth rate (μ). A material balance on a biological reactor yields the following two governing equations:

$$\frac{1}{\Theta_c} = \mu - K_d \tag{4}$$

$$\frac{\mu}{Y} = \frac{Q(S_o-S_1)}{X\,V} \tag{5}$$

For systems without provisions to retain the biological solids (without recycle or retention on fixed films), the hydraulic retention time is equal to the net organism growth rate. It is this ability to control solids retention time independently of hydraulic retention time that

made possible the treatment of domestic wastewater at ambient temperature and smaller reactor volume. Selection of a value of Θ_c, in essence the microbial growth rate, determines the concentration of growth limiting substrate surrounding the microorganisms and in the system effluent. Figure 4 illustrates this relation between Θ_c and effluent concentration (S_1) and also treatment efficiency.

The efficiency of a waste treatment process is defined as:

$$E = \frac{100(S_o - S_1)}{S_o} \qquad (6)$$

in which E = treatment efficiency, (%)

S_o = influent waste concentration, (mass/volume)

and S_1 = effluent waste concentration, (mass/volume)

It is observed from the figure that the higher the solids retention time, the better is the treatment efficiency. Table 6 summarizes the steady-state relationships for completely mixed treatment process.

For fixed-film reactors, such as the anaerobic filter, the non steady-state governing equations become partial differential equations, and are much more complex to solve. The implications of this difference have been discussed by others (DeWalle, 1976; Jennings, 1976) are beyond the scope of this study.

In the case of predicting process performance during start-up operations or under transient conditions resulting from changes in process loading or treatment failure, dynamic modelling is required. An inhibition function was used by Andrews (1969) to relate volatile acids concentration and specific growth rate for methane bacteria. This inhibition function may be expressed as follows:

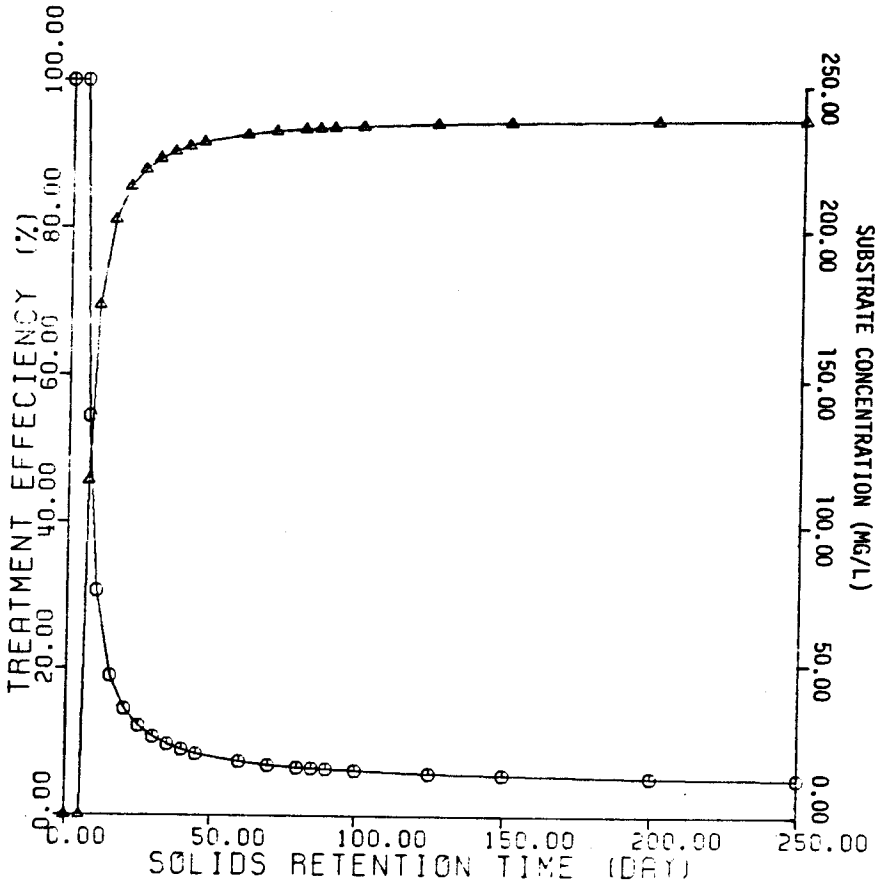


Figure 4 Steady-State Relationships Between Solids Retention Time, Effluent Waste Concentration and Specific Treatment Efficiency for Completely Mixed, Continuous Flow Biological Treatment Process

Table 6. Summary of Steady-State Relationships for Completely Mixed Biological Waste Treatment Process

Characteristics	Without Recycle
Specific Efficiency	$E_1 = \frac{100(S_o - S_1)}{S_o}$
Effluent Waste Concentration	$S_1 = \frac{K_1[1 + K_d(\theta_c)]}{\theta_c(Yk - K_d(\theta_c)}$
Microorganism Concentration in Reactor	$X = \frac{Y(S_o - S_1)}{1 + K_d\theta_c}$
Hydraulic Retention Time	$\theta_h = \frac{V}{Q}$
Net Growth Rate	$\frac{dX}{dt} = \mu X - K_d X$
Substrate Utilization Rate	$\mu = \frac{\hat{\mu}S}{K_s + S}$
Solid Retention Time	
General	$(\theta_c)^{-1} = \frac{YkS_1}{K_1 + S_1} - K_d$
Limiting Minimum	$[\theta_c^m]_{11m} = (Yk - K_d)^{-1}$
Sludge Production Rate	$S = Q(S_1 - S_o)Y - K_d XV$

$$\mu = \frac{\hat{\mu}}{1 + \frac{K_s}{S} + \frac{S}{K_I}} \qquad (7)$$

in which μ = specific growth rate, (1/time)

$\hat{\mu}$ = maximum specific growth rate in the absence of inhibition, (1/time)

S = limiting substrate concentration, mass/volume

K_s = saturation constant, numerically equals lowest concentration of substrate at which the specific growth rate is equal to 1/2 the maximum specific growth rate in the absence of inhibition, (mass/volume)

K_I = inhibition constant, (mass/volume)

The inhibition function for several values of K_I is shown in Figure 5.

For a given waste treatment process configuration, models can be developed by combining the basic kinetic expressions from this section. While the mass balances are written in differential forms, it is common to use the steady-state form of the equations, i.e., $\frac{dX}{dt} = 0$, $\frac{dS}{dt} = 0$. This approach is considered to be quite satisfactory for design considerations.

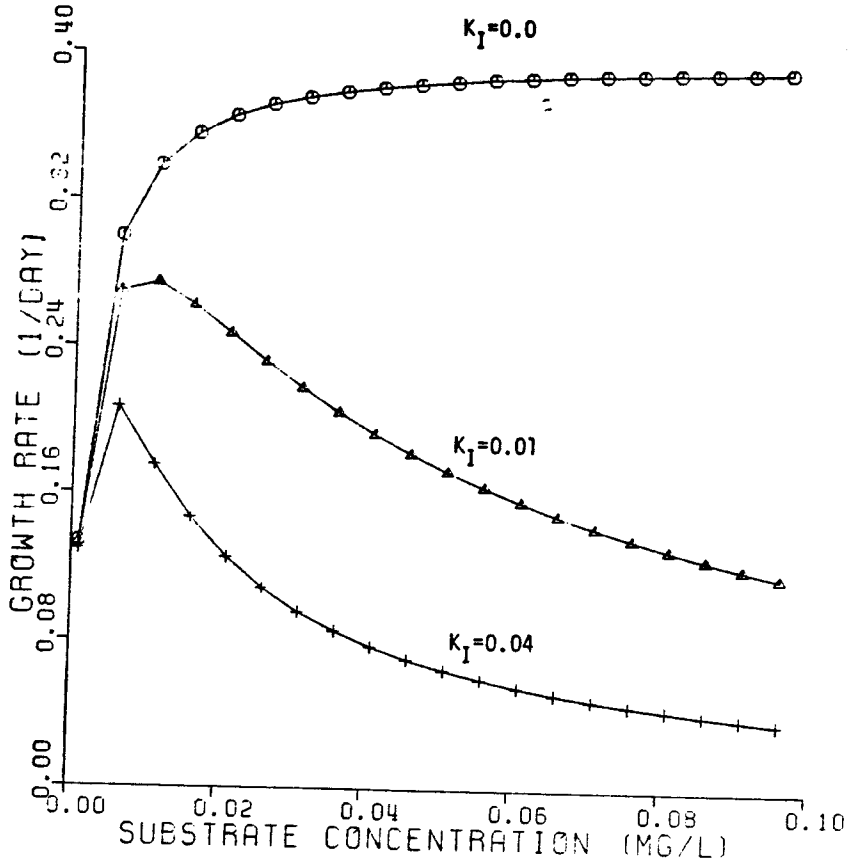


Figure 5 The Effects of Inhibition Function on Specific Growth Rate

CHAPTER 4.

RESEARCH PROCEDURE

4.1 Experimental Procedure

Experimental Apparatus:

The experimental setup included one holding tank approximately 5 feet in diameter and 6 feet tall, one anaerobic filter 2 feet in diameter by 8 feet tall, coupled with an aerobic filter approximately in the same dimensions. The total volume of the empty cylinders were 117, 30 and 25 cu. ft. respectively.

Figure 6a depicts the schematic flow pattern of the facility. As can be seen from the diagram, the raw sewage was pumped from the Westwood Blvd. sewer to the holding tank at approximately 10 gpm. The Westwood Boulevard sewer receives a combination of wastewaters from the Bel Air region above the UCLA campus, dining hall facilities the Campus, and sanitary wastes from the Engineering I Building. Solids were kept in suspension by a 1/4 hp mixer. Wastewater was pumped to the anaerobic filter at a rate of 0.11 gal/min producing a hydraulic retention time in the anaerobic filter of 24 hours. Both the columns were filled with a plastic media, Koro-Z, manufactured by the B. F. Goodrich Co. The plastic media has an effective surface area of 44 square ft/cu. ft. and has a 97% of void volume. The Koro-Z first arrived as rectangular blocks of 2 feet by 2 feet by 4 feet. This was cut into circular blocks to fit into the columns. A cross-sectional view of the anaerobic filter tank is shown in Figure 7.

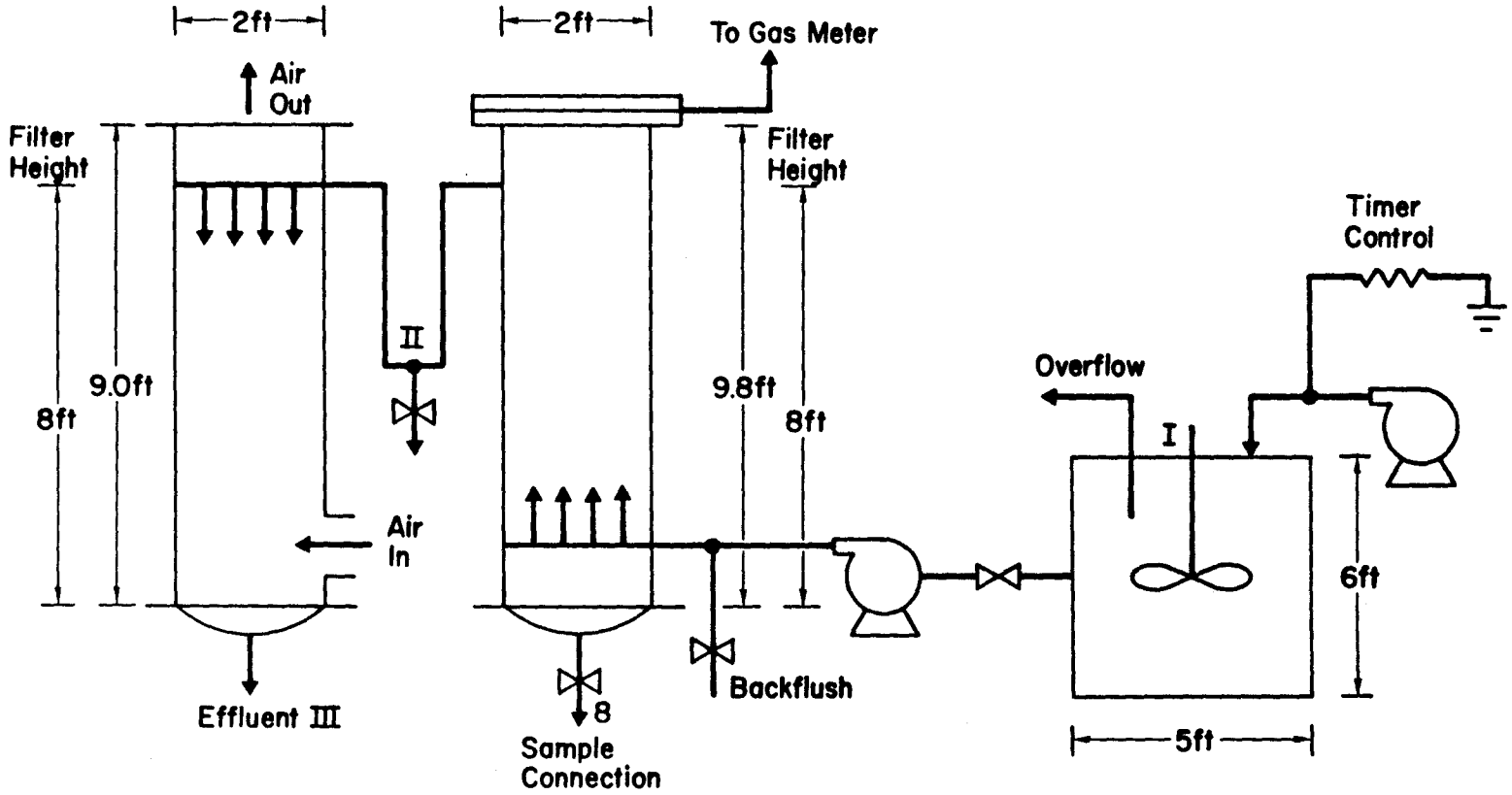


Figure 6a. Schematic Diagram of Experimental Equipment

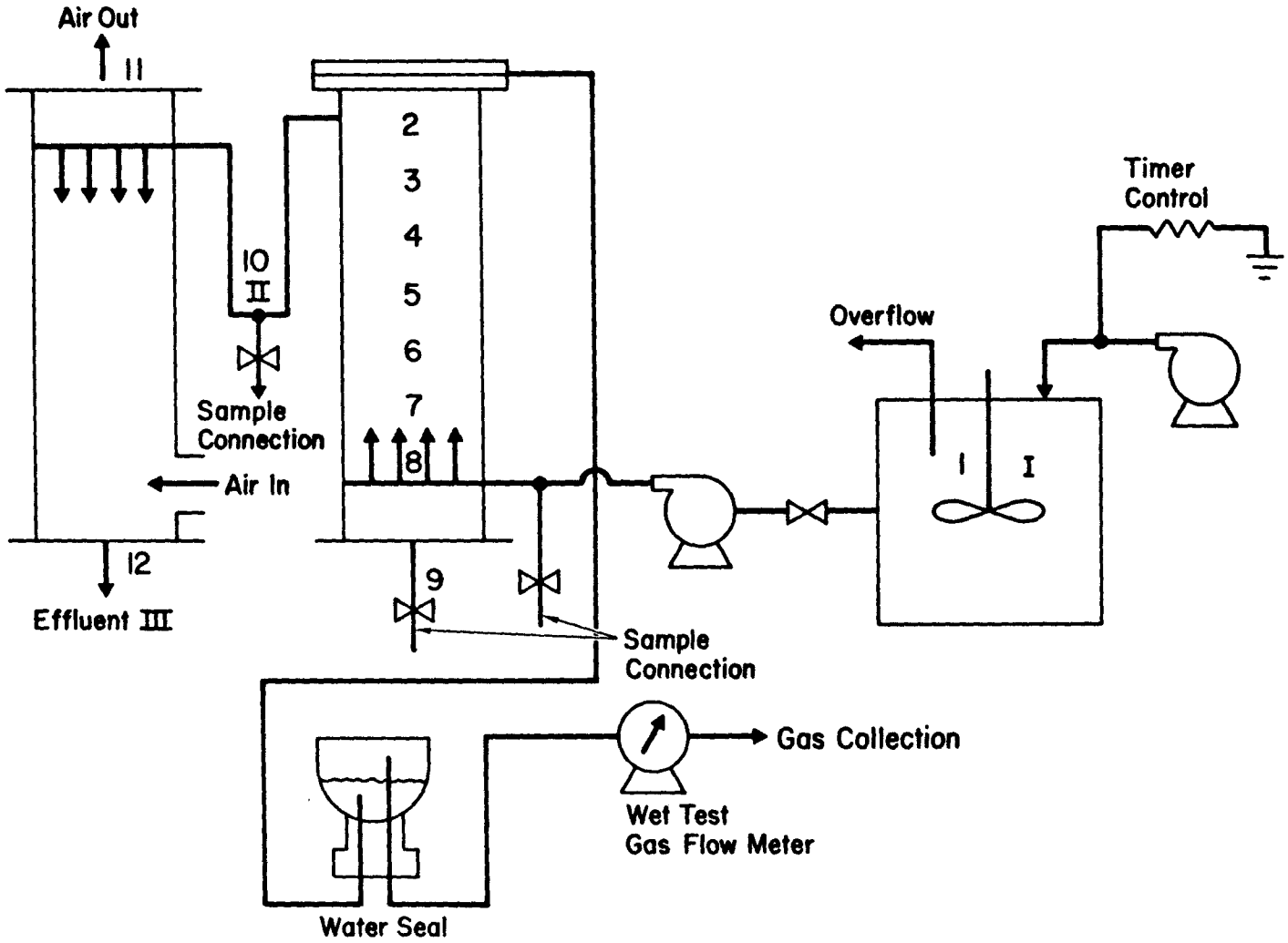
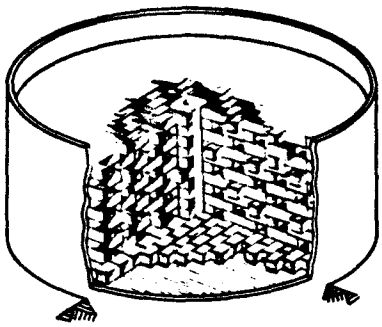
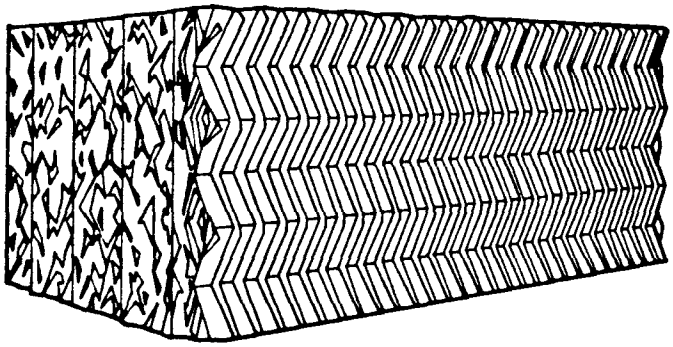


Figure 6b. Flow Pattern with Sample Collection Taps



(a)



(b)

(a) Cross-Sectional View of Filter Tank
(b) Vinyl Core - KORO-Z TM

Figure 7 Cross-Sectional View of Anaerobic Media

For a profile analysis of the wastewaters, samples could be withdrawn from the sample taps shown in Figure 6b. During the experimental period, samples were withdrawn from Station I, II, and III representing the influent, mid-point which was the point after anaerobic treatment and before aerobic treatment, and the final effluent. The gas was collected from the top of the anaerobic filter and was metered by a Wet Test Meter, Precision Model #T110. A sludge withdrawal part also allowed the drainage of sludge when necessary. This was approximately once every two months.

The anaerobic filter was seeded by 30 gallons of sludge from a pilot scale 50-gallon digester on May 15, 1981. The digester was treating raw primary sludge obtained from the City of Los Angeles' Hyperion Treatment Plant. The columns were then operated on domestic wastewater until steady state was obtained. Domestic sewage was pumped continuously from the holding tank by a Gorman Rupp pump model #M1 4251 7 X 15. This was later replaced by the 7017 Masterflex Brand variable speed drive unit due to frequent clogging of the check valves in the Gorman Rupp pump. At several points, the operation was also turned off due to the breakdown of the main pump used to lift wastewater from the Westwood Blvd. sewer. Originally, two Gould's 3884 pumps were used, but were later replaced due to seal failure and impellor hub failures (Day 187, Day 190, Day 194, Day 215, Day 265). A Hydromatic grinder pump, model SPG-200 was used as replacement. The flow was intermittent during those times and a change of the influent quality was reflected in the data. This occurred due to the reduced flow rate which changed the ratio of Engineering I

building wastewater to Bel Air and dining hall facility wastewater.

The feed system and filter were all installed in the open air on the roof of Engineering I and were thus maintained at ambient temperature.

4.2 Waste Description

Raw domestic waste was chosen to test the ability of the filter to treat low strength wastewater at ambient temperature. Table 7 presents a comparison of the characteristics of typical raw sewage and the influent in this experiment. Although the influent was of a quality superior to that of the typical sewage, changes in the quality of the influent were also noted with the changing of the pumps and after the new quarter began when students returned to campus.

Laboratory analyses were conducted on samples withdrawn to determine flow rate, temperature, pH, and turbidity according to Standard Methods. These were done daily while ammonia, total suspended solids and total organic carbon (TOC) (which replaced BOD) were tested 3 times a week. The probe method Orion #95 10 was used on ammonia while total organic carbon was determined by the Ionic 1270 using acetic acid as a standard. The turbidity meter used is from the HG Instruments Co. Model #DRT 100. Gas production was monitored daily through the Wet Test Gas Flow Meter Precision Scientific (Model T110) and determinations for methane and carbon content were made using gas chromotography.

Table 7 A Comparison Between the Characteristics of Typical Raw Sewage and Experimental Influent

Constituents	Concentration			
	Strong	Medium	Weak	This Study
Total Suspended Solids (mg/l)	1200	720	350	50
Total Dissolved Solids (μM HOS)	850	500	250	380
Biochemical Oxygen Demand (BOD_5, 20°C)	400	200	110	90
Total Organic Carbon (TOC)	290	160	80	40
Chemical Oxygen Demand (COD)	1000	500	250	190
Nitrogen:				
Free Ammonia	50	25	12	6
Nitrites	0	0	0	0
Nitrates	0	0	0	0
Alkalinity ($CaCO_3$)	200	100	50	130

* All values except otherwise noted are expressed in mg/L

CHAPTER 5.

EXPERIMENTAL RESULTS

5.1 Data Collected from the Pilot Plant

The anaerobic filter concept was investigated using raw domestic wastewater a a hydraulic retention time of 1 day and an organic rate of 10 lb COD/1000 cu ft/day under steady-state conditions. The results of the experiments are shown in Figures 8-17 and all data are listed in Appendix A.

After 5 weeks of operation at approximately the same loading rate, a steady-state removal efficiency was obtained. Although the quality of the raw sewage varied from hour to hour, the holding tank kept the influent quality stable thus avoiding shock loading and allowing equalization of flow. This was indicated by the data collected every two hours throughout the day (Table 8). The flow in the system was kept fairly constant except during pump failure. A change in the influent characteristics was reflected on Day 266 after the Hydromatic pump was used. The reduced flow rate of the Hydromatic pump changed the wastewater blend.

The pH at any point in the system seemed to be well controlled and was usually in the favorable range. Although the pH did vary from as low as 6.8 to as high as 8.8 in the influent, no ill effect was noted. The effluent remained at 7.0 to 7.9 during the entire time of operation. The highest value of pH was obtained usually at the final effluent while the pH of the mid-point (effluent from the anaerobic filter) stayed within the optimum range of anaerobic treatment.

During the course of the experiment, TOC was used to replace

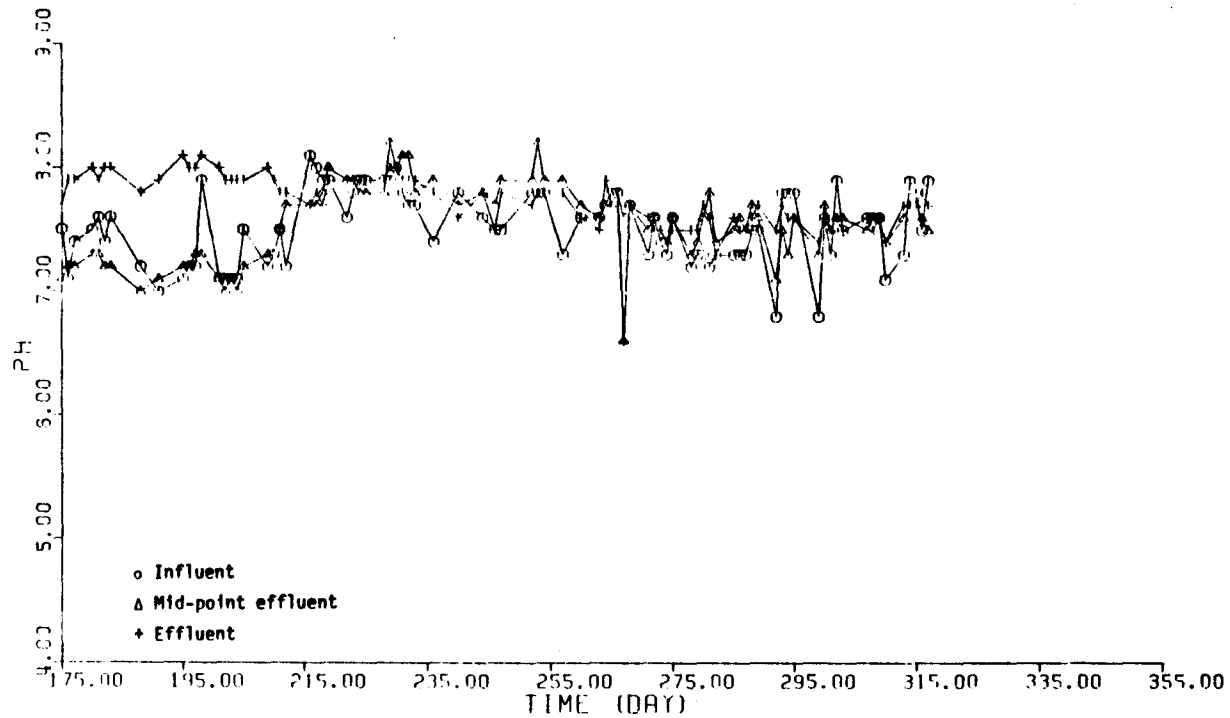


Figure 8 pH of Influent, Mid-point and Effluent

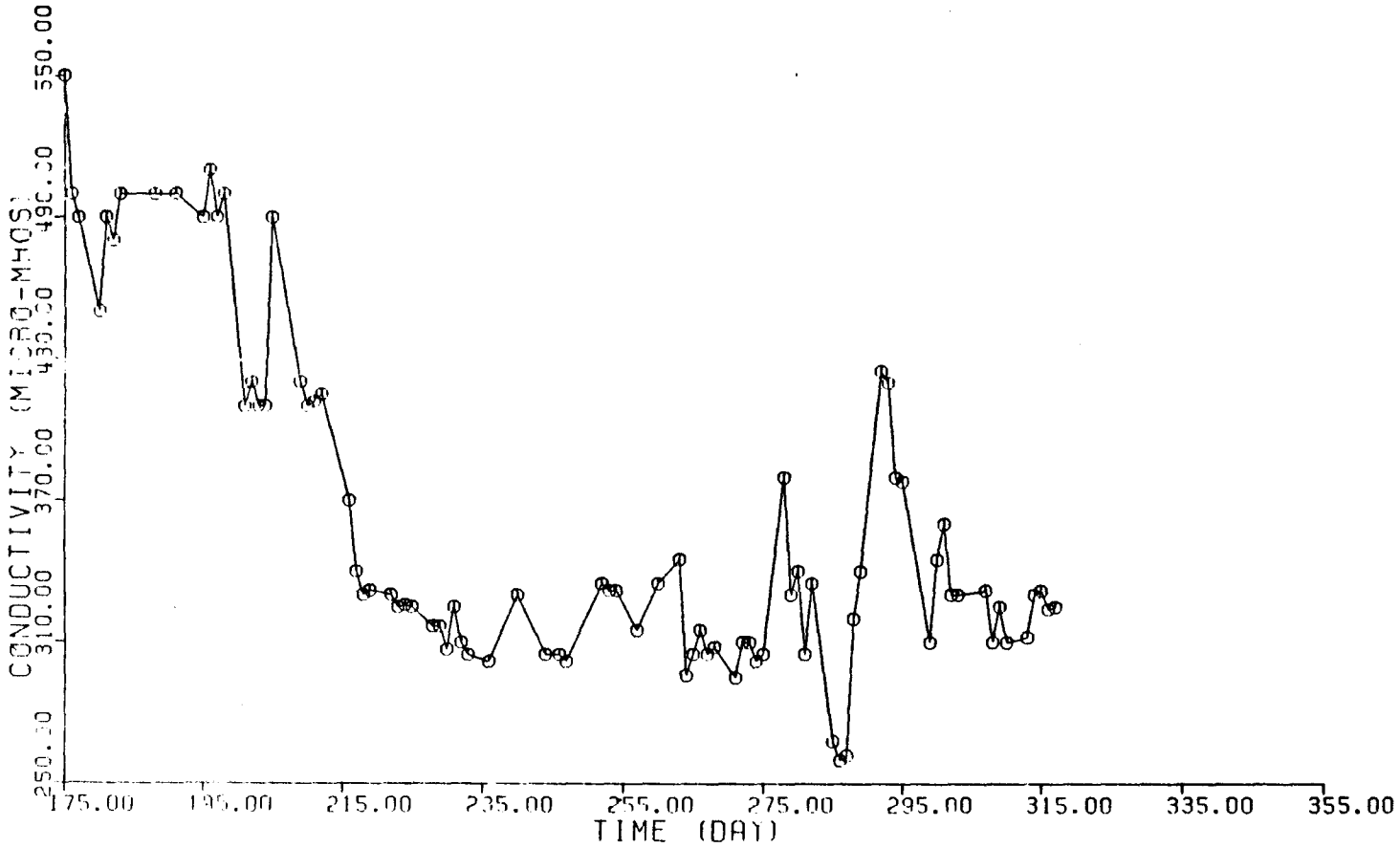


Figure 9 Total Dissolved Solids of Effluent

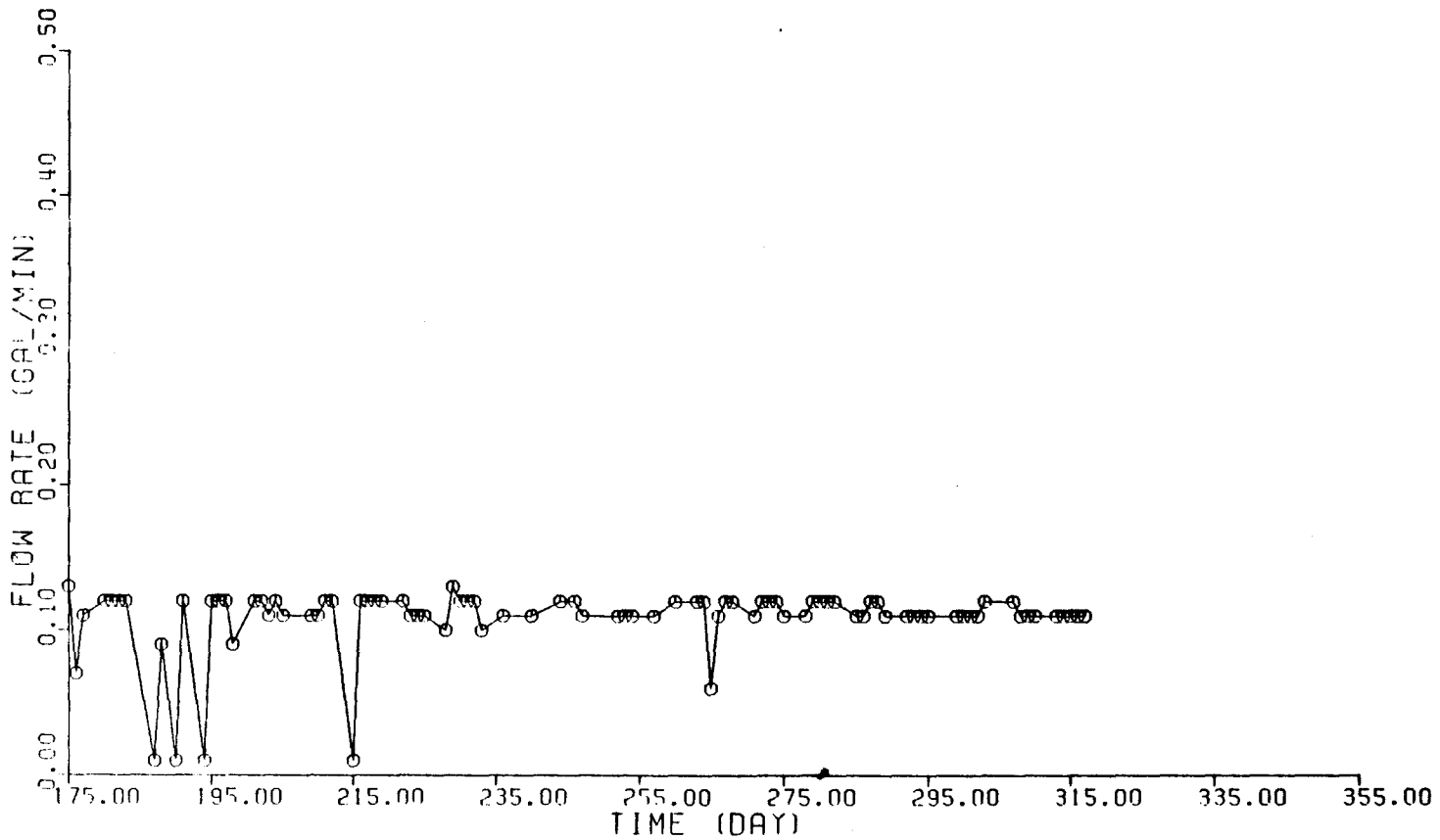


Figure 10 Flow Rate Versus Time

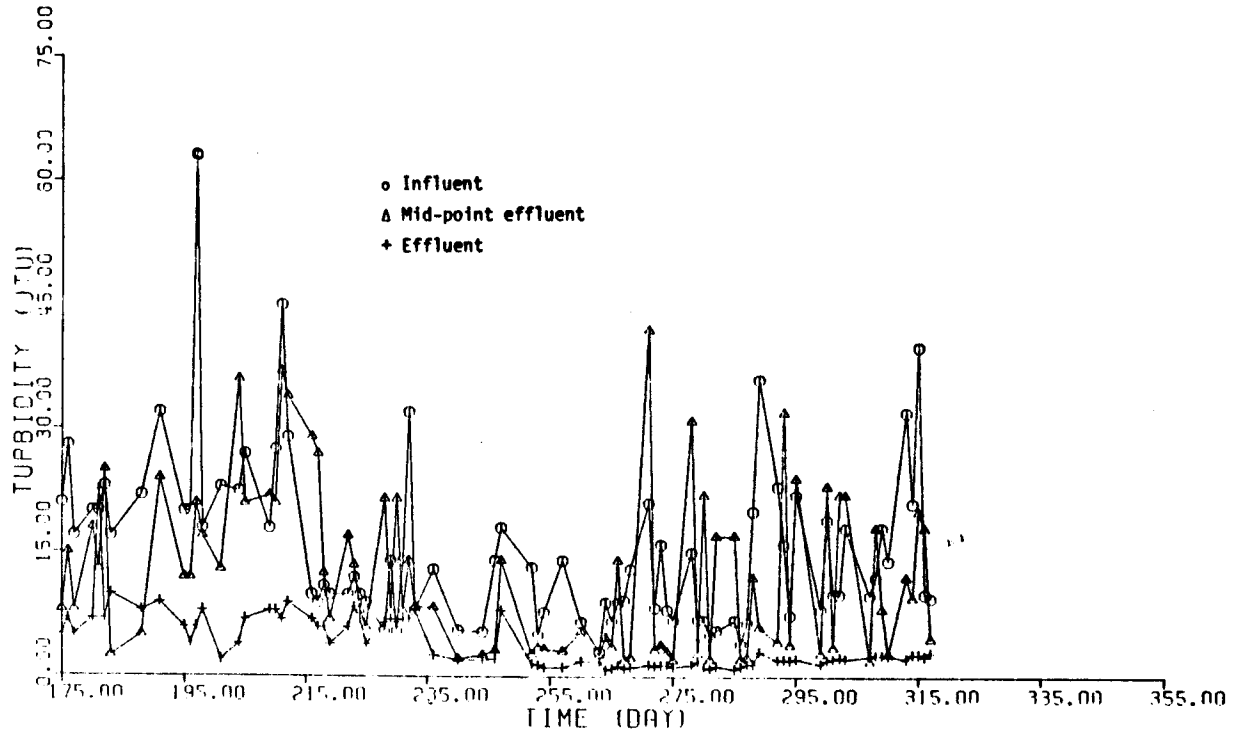


Figure 11 Turbidity of Influent, Mid-point and Effluent

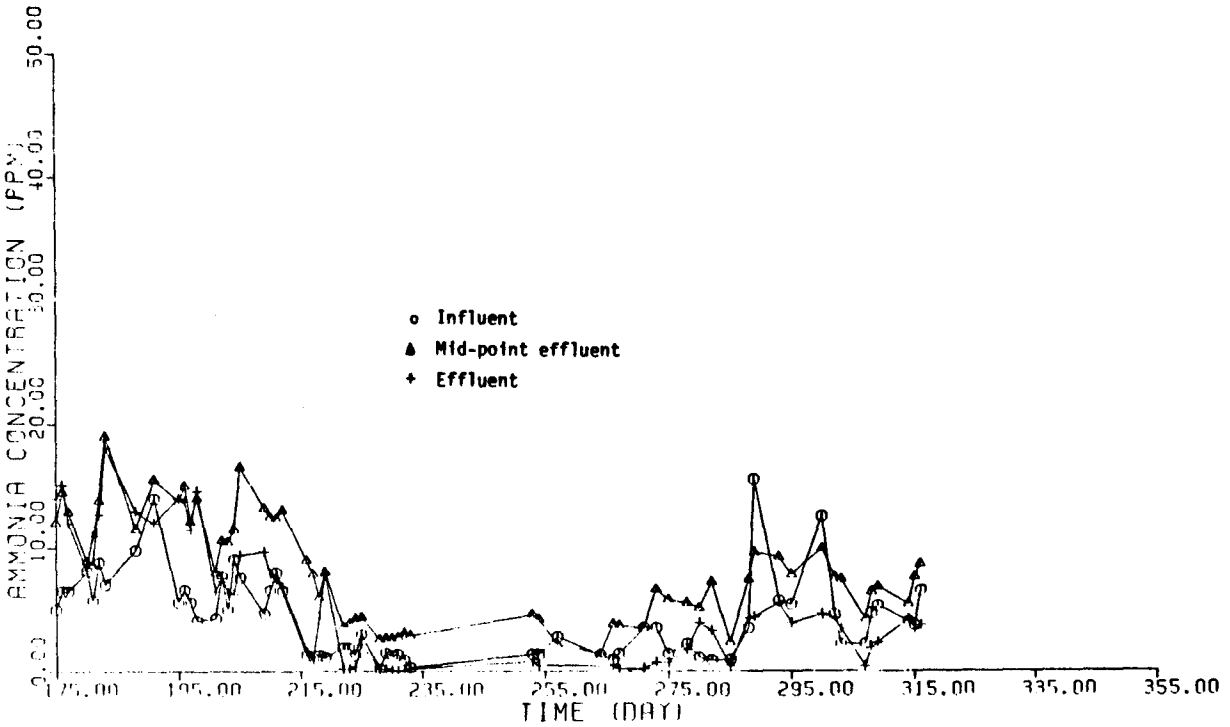


Figure 12 Ammonia Concentration of Influent, Mid-point and Effluent

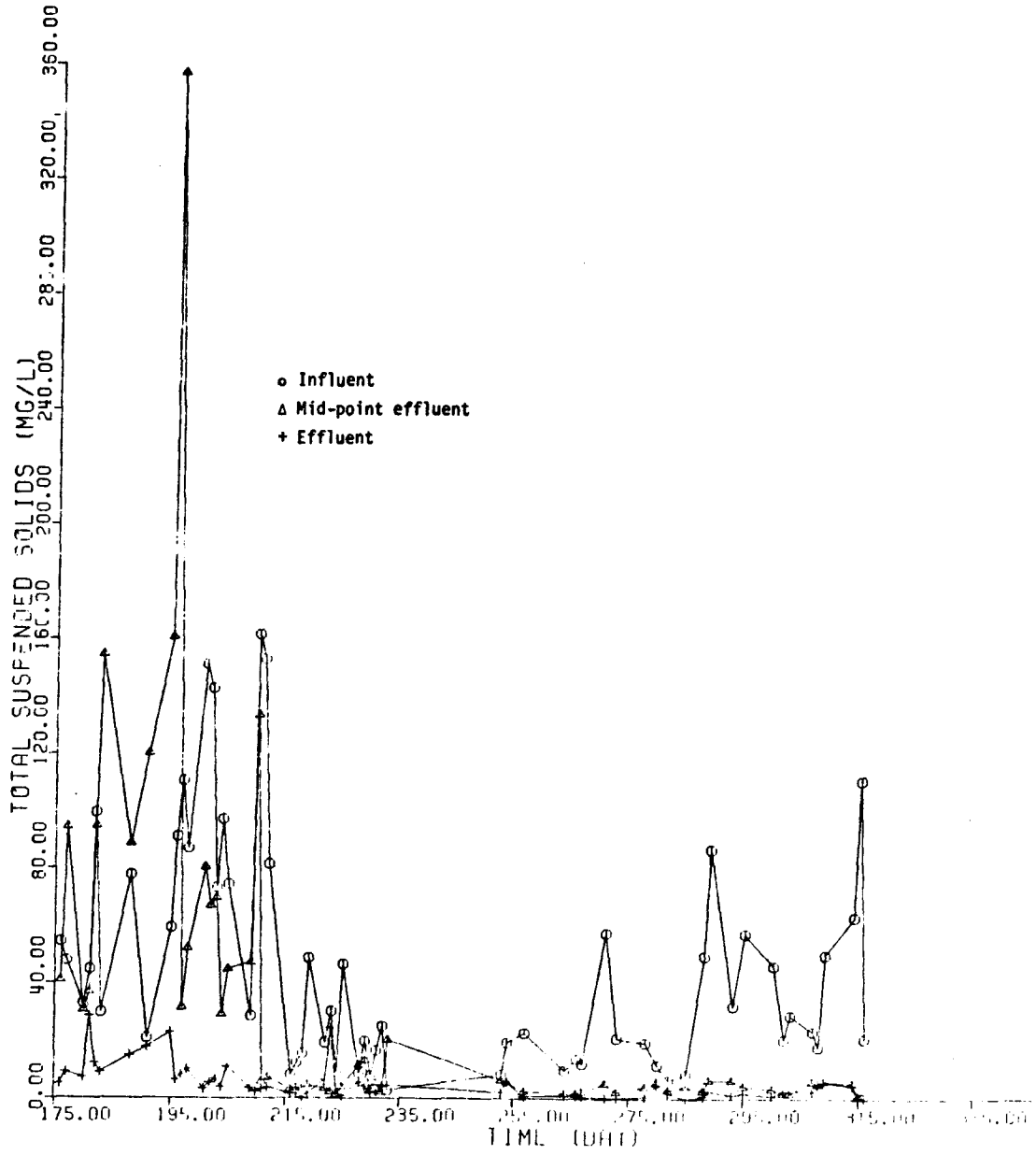


Figure 13 Total Suspended Solids of Influent, Mid-point and Effluent

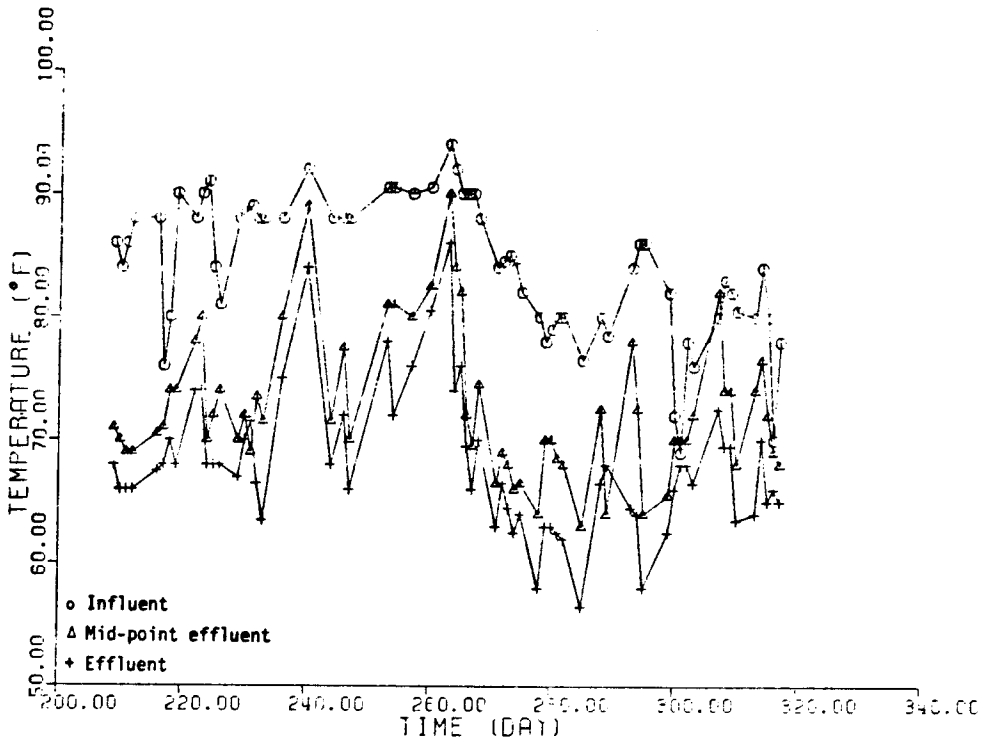


Figure 14 Temperature of Influent, Mid-point and Effluent

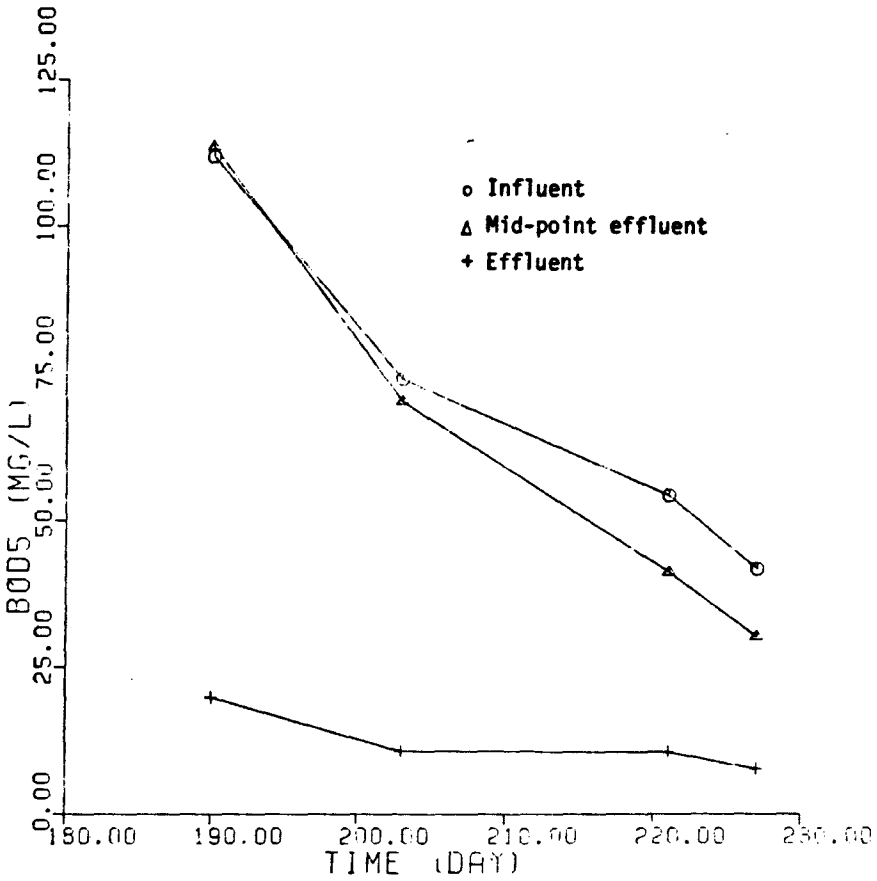


Figure 15 BOD Ultimate of Influent, Mid-point and Effluent

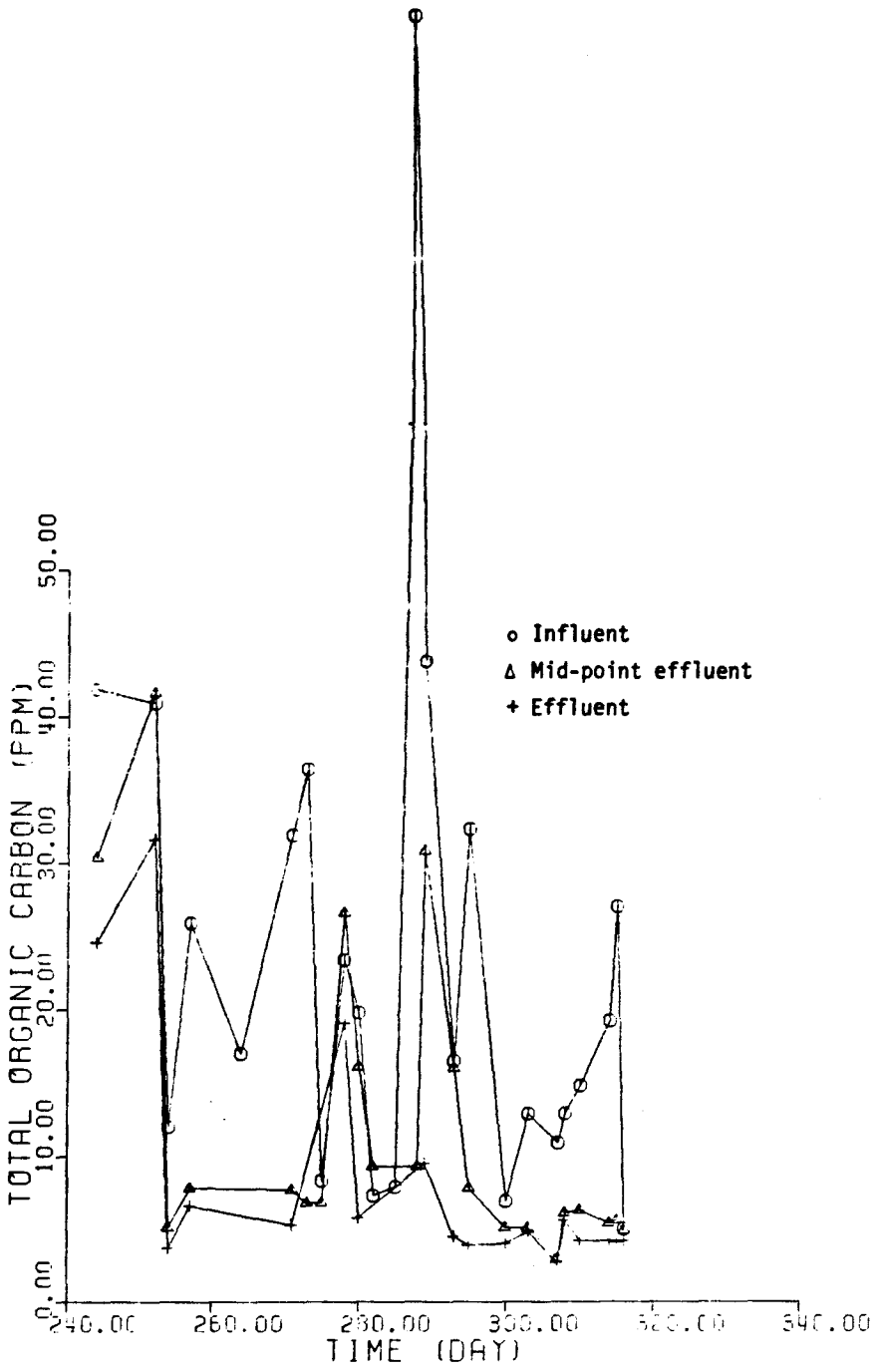


Figure 16 TOC of Influent, Mid-point and Effluent

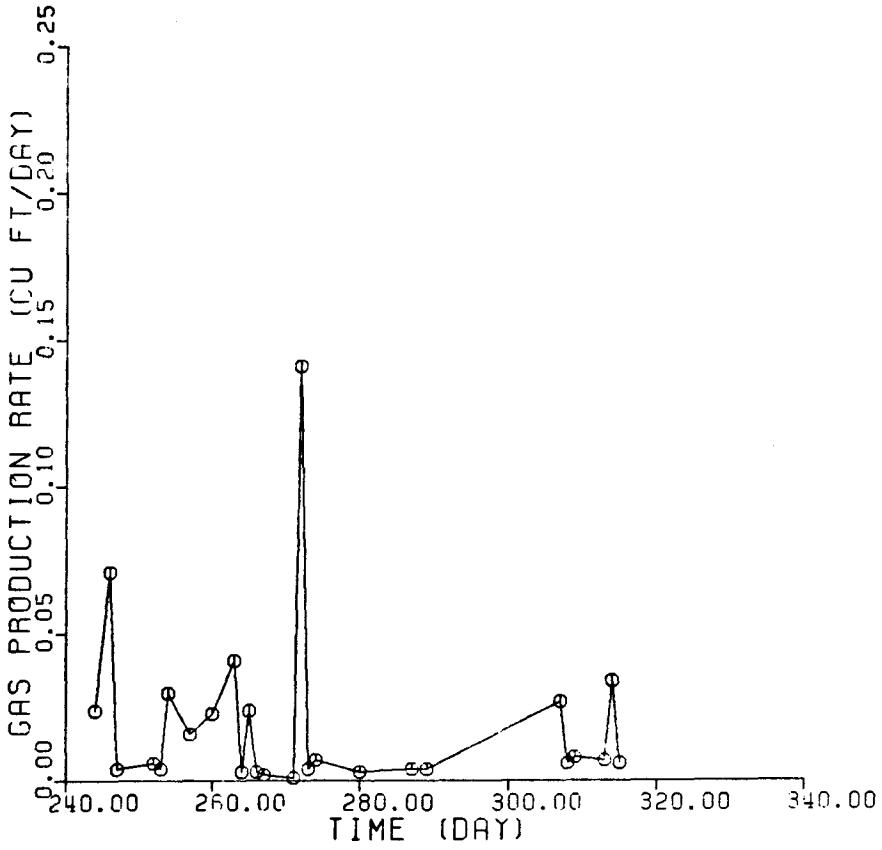


Figure 17 Gas Production Rate Versus Time

Table 8 Diurnal Fluctuation in Wastewater Quality

TIME		DAY 289			DAY 316			DAY 327		
		I*	M	E	I	M	E	I	M	E
8:00 a.m.	Temp. (°F)	78.0	60.5	54.0	69.0	61.0	55.0	66.0	62.0	60.0
	TOC (ppm)	40.8	31.7	12.2	5.8	6.5	5.2	12.4	4.9	2.3
	TSS (mg/l)	88.7	6.2	1.6	21.2	4.7	3.0	79.9	5.1	3.2
	Turbidity (JTU)	22.0	3.4	1.5	10.0	21.0	5.4	28.0	4.8	3.0
	TDS, effluent (µMHOS)			300.0			320.0			312.0
1:00	Turbidity (JTU)	35.0	3.8	2.2	10.0	15.0	2.9	28.0	5.2	3.2
12:00	Turbidity (JTU)	29.0	4.6	3.0	10.0	3.9	2.8	42.0	4.8	3.2
2:00	Temp. (°F)	78.5	64.0	68.0	70.0	69.0	66.0	70.0	69.0	65.0
	TOC (ppm)	43.6	30.8	9.5	5.0	4.9	4.2	15.3	4.7	3.2
	TSS (mg/l)	86.6	6.4	2.6	20.9	2.9	0.3	152.3	3.9	1.5
	Turbidity (JTU)	36.0	6.0	3.0	10.0	18.0	2.5	45.0	21.0	3.0
	TDS, effluent (µMHOS)			340.0			324.0			340.0
4:00	Turbidity (JTU)	36.0	6.6	3.6	9.5	4.2	2.7	44.0	5.0	3.2
6:00	Turbidity (JTU)	35.0	6.0	4.0	28.0	3.2	2.6	44.0	5.4	3.3
8:00	Temp (°F)	78.0	68.0	61.5	70.5	64.0	62.5	74.0	64.0	60.0
	TOC (ppm)	54.2	39.5	12.2	8.4	6.5	6.2	61.4	14.4	2.9
	TSS (mg/l)	56.3	7.7	3.3	95.3	3.3	3.3	101.5	7.0	2.4
	Turbidity (JTU)	30.0	8.2	3.8	24.0	4.4	3.2	44.0	9.0	4.0
	TDS, effluent (µMHOS)			350.0			322.0			340.0
10:00	Turbidity	30.0	8.2	4.0	12.0	4.3	3.1	44.0	8.8	4.8

* I - Influent, M - Mid-point, E - Effluent

BOD as the former allows higher correlation and more data collection. A TOC removal efficiency of 70% to as high as 99% was observed. Equally high efficiency was recorded with total suspended solids removal. High quality effluent was obtained with low turbidity, total dissolved solids and suspended solids during the latter part of August throughout September.

The influent temperature varied from 69° F to 94° F and seemed to have no correlation with the treatment efficiency. The temperature of the anaerobic filter was higher than the influent at only one point, on Day 301.

During the period of study, very little methane gas was collected. The low production rate results primarily from the extremely low influent TOC. Also, the solubility of methane in water is such that much of the gas produced in this study is lost in the effluent. The percentage of methane gas was found to be approximately 6.5%, with the remainder being nitrogen.

5.2 Discussion

The ability of the anaerobic system to efficiently treat low strength domestic wastewater (10 lb COD/1000 cu ft/day) at ambient temperature to meet secondary effluent conditions is demonstrated by the efficiency and stability of the anaerobic system: 1) The development of a dense and concentrated biomass with concentrations as high as 30 kg/cu m to perhaps a value approaching 100 kg/cu m (Jewell, 1980); 2) Entrapment and filtration of fine particles producing effluent suspended solids concentration of less than 10 mg/l. This combination of exceptionally high biomass concentration and efficient

solids capture provides the basic requirements that results in high efficiency biological treatment process. This high rate of treatment and removal efficiency compares favorably with even high rate aerobic systems.

During the entire experimental period the effluent was shown to reach secondary effluent conditions with a TSS concentration of less than 30 mg/l. This occurred even when the suspended solids in the mid-point were as high as 160 mg/l. At the latter stage of data collection, turbidity of the effluent was consistently below 4 JTU. The effluent was observed to be clear, containing only tiny air bubbles which would clear up if the sample was allowed to stand for a few minutes. During the start of the experiment, when solids were still being washed out of the anaerobic column, the mid-point effluent was usually blackish in color with high concentration of suspended solids. The color gradually changed to a greenish tint. Both the final and the mid-point effluent were shown to have very low amounts of organic carbon content (TOC) indicating that treatment was near to completion.

The aerobic filter was also shown to be effective in treating the oxygen demanding forms of nitrogen and sulfides produced during anaerobic fermentation. As shown in Figure 12 and Table 9 (respectively), both compounds were oxidized without the need of mechanical aeration systems as evidenced by their final concentration in the effluent. The sulfides were shown to be more effectively removed than ammonia as the latter was observed to vary with the influent concentration. However, the ammonia concentration was still lower than the incoming influent ammonia concentration.

During the whole time of the study, sludge was partially drained

Table 9. Sulfides Concentration of Influent, Mid-Point and Effluent.

Date	Influent	Mid-point	Effluent
Day 313	1.5	11.2	---
Day 314	0.68	3.88	---
Day 316	0.45	3.50	0.18
Day 317	0.78	9.68	0.24
Day 320	1.09	4.84	0.82
Day 322	0.68	4.48	0.61
Day 327	1.11	4.79	0.64
Day 329	1.36	6.42	0.42

All concentrations are in mg/l.

for a total of three times, once every two months. An increase in the total suspended solids in the mid-point effluent can be used as a good indicator for when the sludge should be drained. Presented below are the amounts drained each time:

DATE	VOLUME(ℓ)	CONC.(mg/l)	TOTAL SOLIDS DRAINED
7/15	6.91	1148.9	10.3 gm
9/22	5.81	1944.3	11.3 gm
11/24	4.31	2710.6	11.6 gm

A total of 5971.9 gm of COD were removed, resulting in an apparent yield of 0.0019 gm VSS/gm COD removed. The values reported by Chain (1976) and Young and McCarty (1968) were 0.012 gm and 0.015 gm VSS/gm COD for fatty acid wastes respectively. The value calculated in this study is low comparatively because the sludge could only be partially drained. A higher yield would also be obtained if the accumulation of the biological solids onto the plastic media were also measured.

Several factors are recognized to determine the amount of solids leaving the filter. A gradual accumulation of solids is usually observed in the anaerobic filter during which the effluent suspended solids remained low. It is only after the filter has reached its maximum storage capacity would the effluent solids show an increase (Chain, 1976).

Finally, the reason for the low production of biogas can be accounted for by two factors: 1) low organic loading rates (an average of less than 40 ppm of TOC); and 2) a certain amount of methane was lost through the effluent even though methane gas is considered to be insoluble. Figure 18 presents the amount of methane that can be

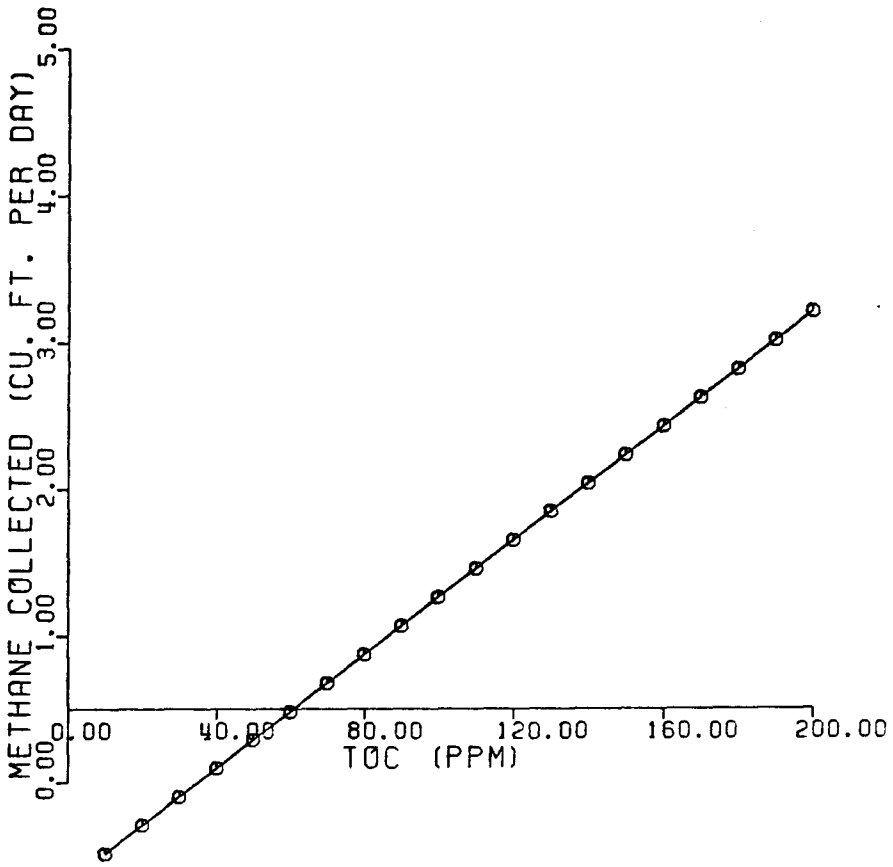


Figure 18 Theoretical Amount of Methane Production

theoretically collected versus the amount of incoming total organic carbon (TOC, ppm) at 435 ml/min. (This is the theoretical flow rate for 1 day retention time used in this study.) It is observed that no methane is to be collected when total organic carbon is under 60 ppm. The values shown are calculated using an effluent total organic carbon as 5 ppm and 90% methane concentration at 1.05 atmosphere.

Although not within the scope of this study, in the future there is an interest to determine the lifetime, the toxic effects, and the frequency of clogging for such a system, even though these have not occurred throughout the six months of this study. Table 10 summarizes these steps and presents some control measures.

Table 10 Indicators, Causes and Control of Unbalanced Anaerobic Stabilization

Indicators of imbalance

1. Drop in methane gas percent.
2. Increase in volatile acid concentration.
3. Drop in methane production.
4. Drop in pH.

Factors causing imbalance

Temporary imbalance

1. Sudden change in temperature.
2. Sudden change in organic loading.
3. Sudden change in nature of organics.

Prolonged imbalance

1. Extreme drop in pH.
2. Presence of toxic materials.

Steps to control imbalance

1. Add alkaline material to maintain pH at about 6.8.
2. Determine and correct cause of imbalance.
3. Provide pH control until treatment returns to normal.
4. Temporary terminate feeding.
5. Reseed.

CHAPTER 6.

ENGINEERING SIGNIFICANCE

6.1 Introduction

In the planning of wastewater treatment facilities, small scale or for rural and semirural communities, the system offered should be inexpensive, reliable and environmentally sound to meet user's needs. According to the 1970 U.S. census, 71.1% of U.S. housing units are sewered, 24.5% are served septic tank systems and 4.3% use other methods (Department of Commerce, 1972). Numerous smaller communities and rural areas fall in the unsewered category.

In rural or semirural areas, homes are typically scattered. This causes sewer costs to be dramatically higher than those in larger communities. This effect is demonstrated in Figure 19. From the study of 16 community wastewater facilities in Oklahoma (Krishnan, (1978), the cost of construction per customer decreases as the number of customers per mile increases. Thus, if outlying sections of the community are to be served only by extending sewers, the cost of conventional facilities would become prohibitive, exceeding $10,000 per household in capital costs (Kreissl, 1976).

The wastewater generated from rural homes has been characterized by several investigators (Bennett, 1975; Witt, 1975). Rural wastewater flows are measured to be between 500-1000 gallons per capita per day. A 7 to 14-day study at five homes showed a tremendous variability both in flow and in the organic matters concentration (Figure 20 and 21). Since state and local codes prohibit surface discharge of

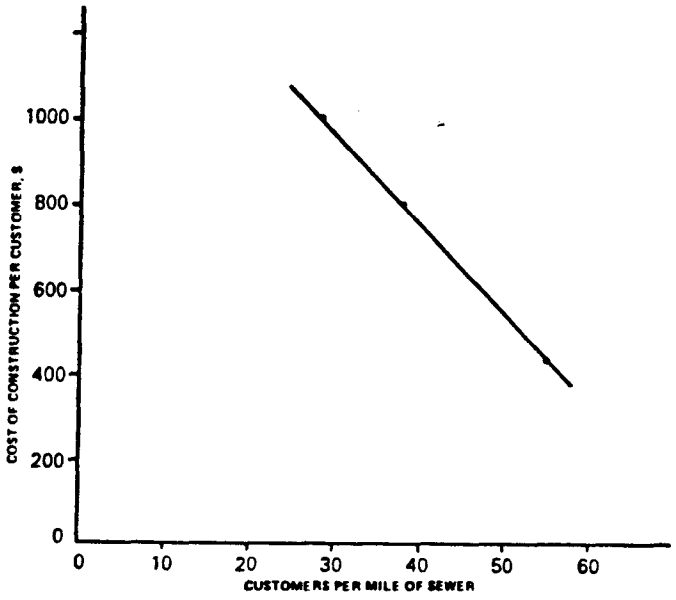


Figure 19 Cost of Sewer Construction Per Customer. (Study of 16 Community Wastewater Facilities in Oklahoma) (Krishnan, 1978)

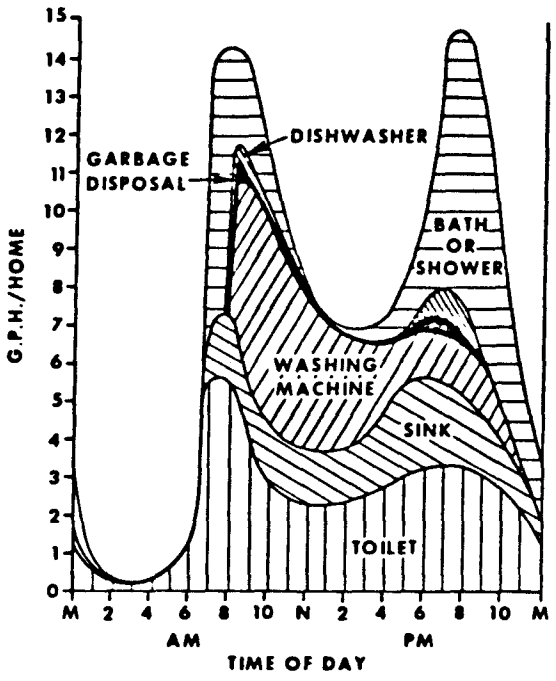


Figure 20 Daily Household Water Use.
(Bennett, 1975)

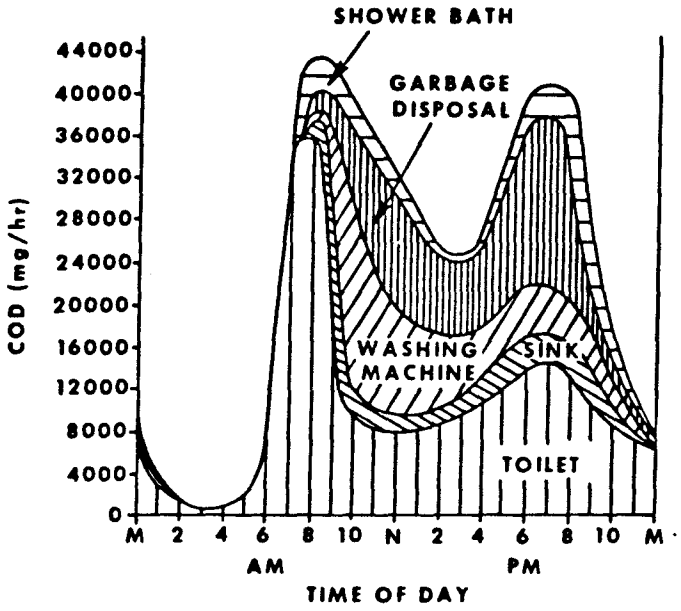


Figure 21 COD Profile. (Bennett, 1975)

wastewater from individual homes, the treatment problem thus becomes one of designing an economical treatment facility of high reliability, low maintenance and easy operation.

The traditional forms for such systems are septic tank-soil adsorption systems, mounds, sand filters, and evapotranspiration systems. However, these systems have not always performed as well as anticipated; the selection, design, installation and maintenance of these systems are significant problems.

Pollutants removed in septic-tanks accumulate in the form of scum and sludge. Since these accumulations occupy increasingly greater portions of the total volume, they eventually reduce the tank's effectiveness by causing an efflux of soil-clogging solids to the soil absorption system. An additional drawback with septic-tanks is the pumpout 'septage'. This extremely offensive sludge is estimated to be four billion gallons annually and the fact that it must be treated and disposed of in an acceptable manner further enhance its undesirability. Improper operations of septic tanks have been implicated in the pollution of many recreational lakes and waterways. As for the other systems, substantial space is usually required in addition to the septic tank and pump. Application is also limited for the evapotranspiration system, since it is climatic dependent. Table 11 presents a comparative cost analysis of the various systems for a single household in Glide, Oregon in a subdivision of 48 hourse (Browne, 1976).

At the present time, there are two basic alternatives available to subsurface disposal and conventional sewage treatment plant disposal. They are the gravity sewer and the pressure sewer. Pressure sewers

Table 11. Comparative Cost for Single Household in Glide, Oregon

PROCESS	INITIAL INVESTMENT COST
Septic Tank	$1550
Mound (average installation)	$3000-$5500
Sand Filter	$3000-$4000
Evapotranspiration	$3000-$7500

for small flows usually offer substantial savings over gravity sewers however, pressure sewers may still be very expensive.

For example, a recent study by General Development Utilities, reported by Cooper, indicates that substantial reductions in capital cost can be obtained by using pressure sewers in lieu of conventional gravity sewers. They evaluated housing developments of 79, 101, and 37 units in Florida, and found that the pressure sewers reduced capital costs by 56%. Operating costs increased by $1640/year due to pumping costs.

To demonstrate how a small community can benefit economically by treating their wastewater with the anaerobic filter, a cost comparison and a basic conceptual design of such a system are presented here. The process design parameters are based on the needs of a small town with 500 residents, having a wastewater generation rate of 100,000 gpd. Treatment objectives are based on EPA secondary treatment standards which require 30-day average BODS and TSS concentrations of less than 30 mg/l. In addition, a minimum removal of 85% is required for both constituents based on a conservative 30-day average.

6.2 Design Basis

Data from the pilot studies and other recent anaerobic filter studies are used as a basis for developing the anaerobic filter process system. The initial evaluation includes an estimation of the influent quality from domestic sewage as defined in Table 12.

The basic design developed includes pretreatment, the anaerobic filtration process and effluent polishing facilities. The latter vary

Table 12 Estimation of Influent Domestic Sewage Quality

Constituent	Concentration
Solids, total:	720
Dissolved total	500
Fixed	300
Volatile	200
Suspended, total	220
Fixed	55
Volatile	165
Settleable solids, mL/L	10
Biochemical oxygen demand, (BOD_5, 20 C)	220
Total organic carbon (TOC)	160
Chemical oxygen demand (COD)	500
Nitrogen (total as N):	40
Organic	15
Free ammonia	25
Nitrates	0
Nitrates	0
Phosphorus (total as P):	8
Organic	3
Inorganic	5
Chlorides	50
Alkalinity (as $CaCO_3$)	100
Grease	100

(All values except settleable solids are expressed in mg/L.)

depending on whether the effluent is used for direct discharge or recycle. A summary of the process parameters is presented in Table 13. A flow diagram of the actual design is illustrated in Figure 22.

Pretreatment:

Pretreatment facilities included in the design are bar screening, grit removal and a holding tank for equalization of influent and dampening of peak flow variations. Requirements for the bar screening and grit removal are typical of a conventional treatment plant. It is assumed that they are required to ensure the filter will not be prematurely clogged due to the presence of large incoming solids. A key factor to consider in this process is the cost of additional facilities, the power requirements and the technical expertise to operate the system. The system should be reliable and allow easy detection of malfunction.

Anaerobic Filter System:

The system design is based on a hydraulic retention time of one day and an organic loading rate of 15.5 lb/ 1000 cu ft/day. As wastewater rises continuously in an up-flow mode, treatment is accomplished by three basic mechanisms: gravitational settling, bio-precipitation and biological decomposition of carbonaeceous wastes. Gases produced by the anaerobic processes are expected to be 65-80% methane, 30-10% carbon dioxide and other mixtures. These gases are to be collected and can be used for heating purposes. A probable application of waste gas or heat is to raise the influent temperature to 20-25C in the winter time in order to increase treatment efficiency. that of the ambient summer temperature.

Table 13 Process Design Parameters of Anaerobic Filter System

Parameter	Value
Pretreatment Facilities	
Bar Screen (optional)	
Volume of Screenings, cu ft/mil gal	3.5
Method of Cleanings, small plant	Manual
Grit Chamber (Optional)	
Minimum Size of Particles to be Removed, mm	0.2
Volume of Grit, cu ft/mil gal	4.0
Equalization Tank	
Volume, Percent of Average Daily Flow	25.0
Anaerobic Filter System	
Packing Media	Koro-Z
Average Hydraulic Loading Rate, gpm	
Solids Yield Coefficient, gm TSS/ gm BOD Removed	0.06
Effluent TSS, mg/l	30.0
Effluent Polishing Facilities	
Aerobic Column, Up-flow, Volume, Fraction of Filter Volume	1
Chlorination (Optional)	
Cl_2 Dosage, lb/mil gal	50.0
Mixing Time, sec	15-30
Minimum Contact Time, min	30
Sludge Handling (Optional)	
Sludge Surge Tank	
Volume, Fraction of Filter Bed Volume	0.7
Sludge Drying Beds	
Solids Loading lb/sq ft - yr	25
Substrate	Domestic Sewage
Feed Condition	Continuously
Hydraulic Retention	1 Day
Temperature	Ambient
Reactor Volume (Anaerobic Filter, cu ft)	14000

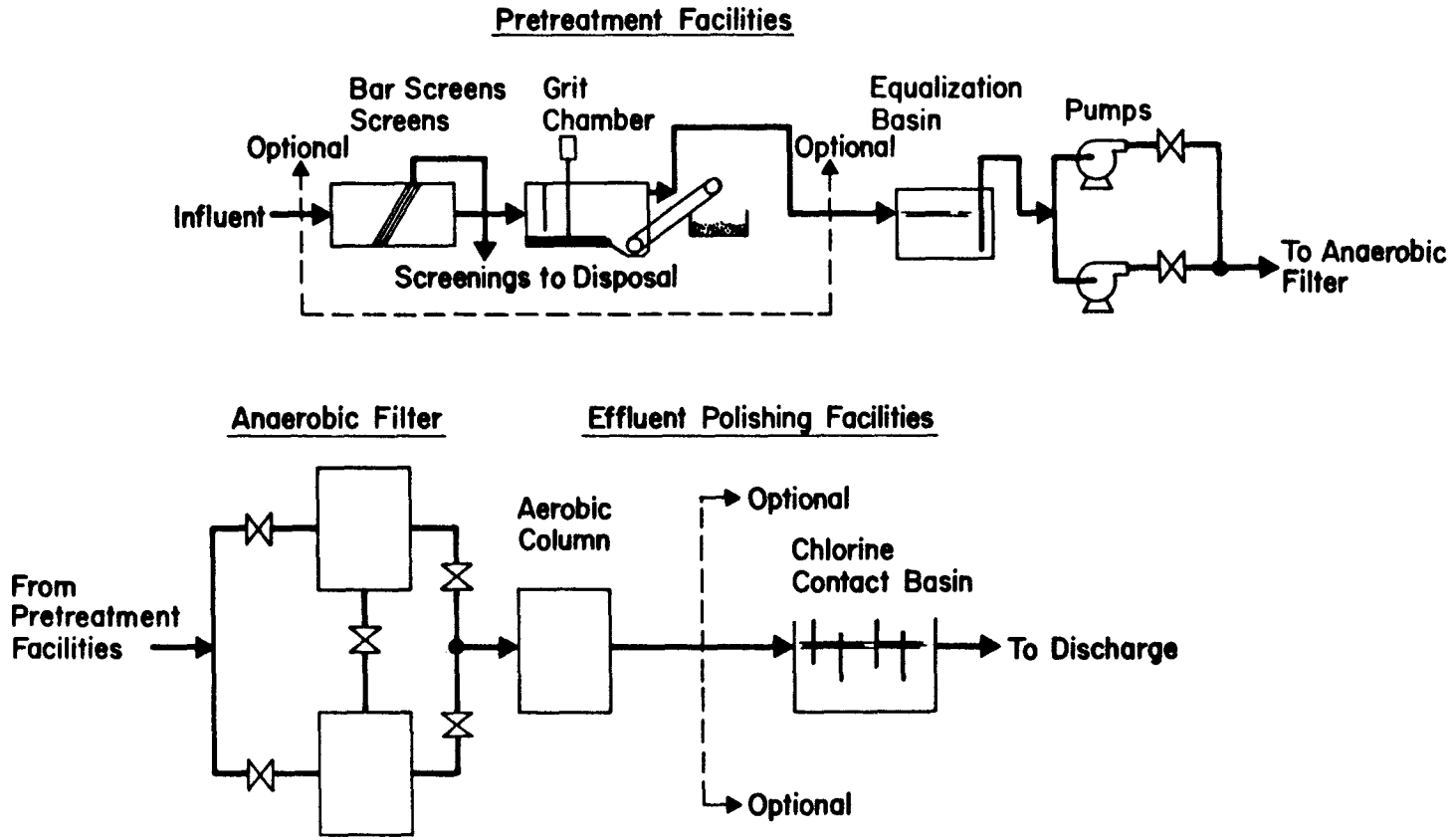


Figure 22 Flow Diagram

Effluent Polishing Facilities:

A second filter of similar dimensions is used to operate aerobically in an downflow mode as in the pilot study. It serves to oxidize the sulfides and ammonia. The pilot plant results indicated that no additional filtration system is needed to further polish the effluent. It is also assumed that no aeration system is needed.

Backwash water is to be recycled back to the filter when necessary. Chlorine can then be used at various locations to disinfect the effluent, depending on the type of recycling use.

Sludge Handling Facilities:

Sludge is wasted from the system intermittently as observed from the pilot study. From the calculations presented in the section below, it is estimated that approximately 3.8 kg/l of solids is generated each day. Since the quantity of solids is small compared to aerobic systems, and since the sludge has undergone anaerobic digestion, relatively inexpensive sand drying beds can be used under these conditions. The dried sludge can also be reapplied back to neighboring soil as nutrient supplements. Another probable solution is to withdraw once every 3 months and transport the sludge to a nearby treatment center or landfill.

6.3 Calculations:

Design Volume: 100,000 gpd (378500 ℓ/day)
Assumed Retention Time: 1 day
K_d, decay Rate Constant: 0.03 day^{-1}
Y, yield: 0.05
θ_c, specific growth rate: 50 days (Taken from Figure 4 at 10% treatment efficiency)

A, flow rate.

S_o, influent substrate concentration

S_1, effluent substrate concentration

Flow Rate:

$$100{,}000 \text{ gpd} \times \frac{1 \text{ day}}{24 \text{ hr}} \times \frac{1 \text{ hr}}{60 \text{ min}} = \underline{69.4 \text{ gal/min}} \ (262.9 \text{ ℓ/min})$$

Radius of Anaerobic Filter:

(Assumed 12 ft in height)

$$\text{Volume} = \pi r^2 h = 100{,}000 \text{ gpd} = 100{,}000 \text{ gal} \times 0.1337 \frac{\text{cu ft}}{\text{gal}}$$

$$= 13370 \text{ cu ft}$$

$$\text{Radius} = \sqrt{\frac{\text{volume}}{\pi \times h}}$$

$$= \sqrt{\frac{13370 \text{ cu ft}}{\pi \times 12 \text{ ft}}}$$

$$= \underline{18.9 \text{ ft}}$$

Organic Loading Rate:

(assumed an average of 250 mg/ℓ of BOD)

$$\text{Organic Loading Rate} = \frac{\text{conc. (mg/ℓ)} \times \text{flow rate (ℓ/min)} \times \text{(min/day)}}{1000000 \text{ (mg/kg)}}$$

$$= \frac{250 \text{ mg/ℓ} \times 262.9 \text{ ℓ/min} \times 1440 \text{ min/day}}{1000000 \text{ mg/kg}}$$

$$= 94 \text{ kg/d} \quad (207 \text{ lb/day}$$

or 0.248 kg/cu m/day (15.48 lb/1000 cu ft/day)

Methane Production Rate:

Same as the theoretical methane gas production used in Figure 18.

Concentration of Microorganisms:

Equation used: $XV = \frac{QY\ (S_o - S_1)}{(1 + Kd\ \theta_c)}$

$$X = \frac{262.8\ \frac{\ell}{min} \times 1440\ \frac{min}{day} \times 0.05(250-30)mg/\ell}{(1 + 0.03(50))day \times 378500}$$

$$= \underline{4.4\ mg/\ell}$$

Sludge Production Rate (per day):

Equation used: $X_w = Q(S_o - S_1)Y - KdXV$

$$X_w = 262.8\ \frac{\ell}{min} \times 1440\ \frac{min}{day}\ (250-30)\ \frac{mg}{\ell}\ (0.05) - (0.03)(4.4\frac{mg}{\ell})(378500\ell)$$

$$= (4163860-378500)^{mg}$$

$$= 3785360\ mg = 3.78\ kg = 8.3\ lb$$

Sludge Volume:

(assumed 1 litre of sludge at 5% solids)

$$= \frac{3785360^{mg}}{50000\ mg/\ell}$$

$$= 75.7\ \ell/day$$

6.4 Comparative Cost Analysis

It is difficult to compare the cost of alternative treatment systems without detailed site-specific estimates; however, it is convenient to compare process requirements, as shown in Table 14. From this table it is observed that the systems are comparable for collection and headworks. The difference in costs will result from the difference in aeration requirements, volume of reactors, and need for plastic packing material. Also the maintenance requirements will be quite different for the two systems, with the activated sludge plant requiring much more maintenance, due to the use of more rotating equipment (i.e., blowers and sludge pumps).

The disadvantages of the anaerobic filter system will be the greater tankage requirements. The anaerobic filter will require approximately 24 hour hydraulic retention time while most activated sludge package plants require 12 or less hour hydraulic retention time. The anaerobic filter system will also require plastic packing or other type of packing. These costs would be offset by the costs of aeration system, secondary clarifier, and sludge digester, which would be needed in the activated sludge system. In fact, by comparison, the anaerobic filter system appears to be the cheapest of all on-site wastewater systems available to date for the quality of effluent produced. Only the minimal amount of energy is required for a modest lift of 12 feet to the filter. At the same time, the anaerobic filter can essentially run itself with infrequent operator attendance. Since there are no highly mechanized parts, further savings can be recognized on man-power and technical expertise attention on the system.

Table 14. Comparative Analysis of the Anaerobic Filter System versus Conventional Package Treatment Plants*

Item	Anaerobic Filter	Conventional Filter
Home Owner Property	——— Same ———	
Collection System	——— Same ———	
Grinding and Grit Removed	——— Same ———	
Influent Pumping	——— Same ———	
Primary Clarification	——— Same ———	
Aeration	Not needed	Provided at an energy cost of approximately \$4.00/lb O_2 transferred (1 lb O_2/hp hr)
Sludge Recycle	Not needed	Required at a pumping rate of 50% to 200% of design flow
Aerobic Filtration for Polishing	Required, depending upon desired degree of treatment	Not needed
Final Clarification	Not needed	Required at 600 - 1000 gal/ft^2 day
Disinfection	——— Same ———	
Sludge Digestion	Provided by the process	Aerobic digestion often needed
Sludge Disposal	Required, but only 80 - 90% less sludge mass provided	Required
Maintenance	Less for anaerobic filtration system due to less equipment (i.e., blowers, recycle pumps, or clarifier mechanisms)	

*Activated Sludge Package Plant

Substantial energy benefits can also result from the offgas production. Although it would not be as high as 420 Btu/day/person due to low influent BOD concentration, the gas produced is still a directly usable energy source for almost any purpose. Finally, implementing this technology will be very low in cost. Due to the low cost of construction, two or more anaerobic columns can be built at the same site with both parallel and in series pipings. This would allow for growth in the community as well as continual treatment of sewage in case of filter failure.

CHAPTER 7

CONCLUSIONS AND RECOMMENDATIONS

An anaerobic filtration process is shown to be successful in treating low strength domestic wastewater at ambient temperature. From the experimental study and the economic analysis, the following conclusions are made:

1) The anaerobic filtration system is effective in treating low strength domestic wastewater at an organic loading rate of 10 BOD lb/1000 cu ft/day at ambient temperature to secondary effluent conditions.

2) Although little methane gas was collected, this could be explained due to the low loading and substrate concentrations.

3) The aerobic filter appears to be promising for effluent polishing especially in oxidizing ammonia and sulfides. Further testing is required for ammonia at higher loading rates.

The important aspect of this investigation is to recognize the potential of a technically simple anaerobic filtration and its effectiveness in treating domestic wastewater.

It is hoped that the existing filter will be operated for at least another year. During this period, shock loading and variation of hydraulic retention times will be tested to find the optimum conditions and the maximum capacity of the filters. Investigations into the degree of treatment versus height are planned and possible reaction kinetics can be investigated.

Appendix A

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Appendix B

Pilot Plant Data:

Table 1

Data: Date Versus Temperature and TOC of Influent, Mid-point and Effluent

OBS	DATE	TEMPIN	TEMPM	TEMPEFF	TOCIN	TOCM	TOCEFF
1	175	.	.	.	.	.	.
2	176	.	.	.	.	.	.
3	177	.	.	.	.	.	.
4	180	.	.	.	.	.	.
5	181	.	.	.	.	.	.
6	182	.	.	.	.	.	.
7	183	.	.	.	.	.	.
8	187	.	.	.	.	.	.
9	188	.	.	.	.	.	.
10	190	.	.	.	.	.	.
11	191	.	.	.	.	.	.
12	194	.	.	.	.	.	.
13	195	.	.	.	.	.	.
14	196	.	.	.	.	.	.
15	197	.	.	.	.	.	.
16	198	.	.	.	.	.	.
17	201	.	.	.	.	.	.
18	202	.	.	.	.	.	.
19	203	.	.	.	.	.	.
20	204	.	.	.	.	.	.
21	205	.	.	.	.	.	.
22	209	86.0	71.0	68.0	.	.	.
23	210	84.0	70.0	66.0	.	.	.
24	211	86.0	69.0	66.0	.	.	.
25	212	88.0	69.0	66.0	.	.	.
26	215	.	.	.	.	.	.
27	216	88.0	70.5	67.5	.	.	.
28	217	76.0	71.0	68.0	.	.	.
29	218	80.0	74.0	70.0	.	.	.
30	219	90.0	74.0	68.0	.	.	.
31	221	.	.	.	.	.	.
32	222	88.0	78.0	74.0	.	.	.
33	223	90.0	80.0	74.0	.	.	.
34	224	91.0	70.0	68.0	.	.	.
35	225	84.0	72.0	68.0	.	.	.
36	226	81.0	74.0	68.0	.	.	.
37	227	.	.	.	.	.	.
38	228	.	.	.	.	.	.
39	229	88.0	70.0	67.0	.	.	.
40	230	88.5	72.0	70.0	.	.	.
41	231	89.0	69.0	71.5	.	.	.
42	232	88.0	73.5	66.5	.	.	.
43	233	88.0	71.5	63.5	.	.	.
44	236	88.0	80.0	75.0	.	.	.
45	240	92.0	89.0	84.0	.	.	.
46	244	88.0	71.5	68.0	41.9	30.4	24.6
47	246	88.0	77.5	72.0	.	.	.
48	247	88.0	70.0	66.0	.	.	.
49	252	.	.	.	40.9	41.6	31.6
50	253	90.5	81.0	78.0	.	.	.
51	254	90.5	81.0	72.0	12.0	5.1	3.7
52	257	90.0	80.0	76.0	25.9	7.8	6.6
53	260	90.5	82.5	80.5	.	.	.
54	263	94.0	90.0	86.0	.	.	.
55	264	92.0	84.0	78.0	17.0	0.0	0.0
56	265	90.0	82.0	76.0	.	.	.

Table 1 (continued)

OBS	DATE	TEMPIN	TEMPS	TEMPEFF	TOCIN	TOCS	TOCEFF
57	266	90.0	72.0	69.5	.	.	.
58	267	93.0	69.5	66.0	.	.	.
59	268	88.0	74.5	70.0	.	.	.
60	271	84.0	66.5	63.0	31.9	7.7	5.3
61	272	84.5	69.0	66.5	.	.	.
62	273	85.0	68.0	64.5	36.4	6.8	0.0
63	274	84.0	66.0	62.5	.	.	.
64	275	82.0	66.5	64.0	8.3	6.8	0.0
65	278	80.0	64.0	58.0	23.4	26.6	19.1
66	279	78.0	70.0	63.0	.	.	.
67	280	79.0	70.0	63.0	19.8	16.1	5.8
68	281	80.0	68.5	62.5	.	.	.
69	282	80.0	68.0	62.0	7.3	9.3	0.0
70	285	76.5	63.0	56.5	7.9	0.0	0.0
71	286	.	.	.	.	.	.
72	287	.	.	.	.	.	.
73	288	80.0	72.5	66.5	87.7	9.3	0.0
74	289	78.5	64.0	68.0	43.7	30.8	9.5
75	292	.	.	.	.	.	.
76	293	84.0	78.0	64.5	16.5	16.0	4.5
77	294	86.0	72.5	64.0	.	.	.
78	295	86.0	64.0	58.0	32.3	7.8	3.9
79	299	82.0	65.5	62.5	.	.	.
80	300	72.0	70.0	66.0	6.9	5.1	4.0
81	301	69.0	70.0	68.0	.	.	.
82	302	78.0	70.0	68.0	.	.	.
83	303	76.0	72.0	66.5	12.9	5.1	4.8
84	307	80.0	82.0	72.5	10.9	3.0	2.7
85	308	83.0	74.0	69.5	12.9	6.1	5.6
86	309	82.0	74.0	69.5	.	.	.
87	310	80.5	68.0	63.5	14.8	6.3	4.2
88	313	80.0	74.0	64.0	.	.	.
89	314	84.0	76.5	70.0	19.2	5.4	4.2
90	315	80.0	72.0	65.0	27.0	5.6	4.1
91	316	70.0	69.0	66.0	5.0	4.9	4.2
92	317	78.0	68.0	65.0	.	.	.
93	320	76.0	64.5	60.5	.	.	.
94	321	76.5	70.0	66.5	16.1	3.7	2.4
95	322	76.0	68.0	60.0	.	.	.
96	323	81.5	80.0	69.5	27.2	8.1	3.9
97	327	76.0	66.0	61.0	57.4	5.4	2.3
98	328	70.0	69.0	65.0	52.4	3.9	1.4
99	329	78.0	63.0	55.0	.	.	.

Table 2

Data: Date Versus Turbidity, Total Suspended Solids of Influent, Mid-point and Effluent and Total Dissolved Solids of Effluent

OBS	DATE	TDS	TURBIN	TURM	TUREFF	TSSIN	TSSM	TSSEFF
1	175	550	21.0	8.0	5.0	.	.	.
2	176	500	28.0	15.0	7.0	54.4	40.9	5.1
3	177	490	17.0	8.0	5.0	47.8	94.5	9.1
4	180	450	20.0	18.0	7.0	32.8	30.6	7.2
5	181	490	20.0	13.0	22.5	44.9	36.9	28.6
6	182	480	23.0	25.0	7.0	99.7	94.8	12.2
7	183	500	17.0	2.5	10.0	29.9	154.7	9.0
8	187	.	.	.	.	.	.	.
9	188	530	22.0	5.0	8.0	77.8	88.9	14.8
10	190	.	.	.	.	.	.	.
11	191	500	32.0	24.0	9.0	20.8	120.3	17.6
12	194	.	.	.	.	.	.	.
13	195	490	20.0	12.3	6.0	59.4	160.6	22.8
14	196	510	20.0	12.0	4.0	91.3	356.6	6.5
15	197	490	63.0	21.0	6.0	10.6	31.5	8.2
16	198	500	18.0	17.0	8.0	87.0	52.1	10.0
17	201	410	23.0	13.0	2.0	51.1	80.4	3.4
18	202	420	.	.	.	42.9	67.1	5.5
19	203	410	.	.	.	73.4	69.9	6.8
20	204	410	22.5	36.0	4.0	97.3	29.2	4.0
21	205	490	27.0	21.0	7.0	74.4	44.9	11.1
22	209	420	18.0	22.0	8.0	28.6	47.3	3.6
23	210	410	27.5	21.0	8.0	61.5	133.4	2.4
24	211	412	45.0	37.0	7.0	52.9	5.8	3.2
25	212	415	29.0	34.0	9.0	81.5	7.2	3.9
26	215	.	.	.	.	.	.	.
27	216	370	10.0	29.0	7.0	8.2	3.3	1.7
28	217	340	9.0	27.0	6.0	12.5	3.6	1.8
29	218	330	11.0	12.5	6.0	15.6	3.4	0.3
30	219	332	10.0	7.0	4.0	48.9	4.8	1.8
31	221	.	.	.	.	.	.	.
32	222	330	10.0	17.0	6.0	19.3	3.4	6.4
33	223	325	12.0	13.5	8.5	30.2	2.2	24.4
34	224	326	10.0	8.0	7.0	9.2	1.3	2.9
35	225	325	9.0	6.0	4.0	46.7	4.1	0.8
36	226	.	.	.	.	.	.	.
37	227	.	.	.	.	.	.	.
38	228	317	5.5	21.5	6.5	8.9	11.6	5.7
39	229	317	14.0	8.0	7.0	19.8	13.2	4.3
40	230	307	5.5	21.5	6.5	7.9	2.9	1.8
41	231	325	14.0	8.0	7.0	16.7	5.5	2.3
42	232	310	32.0	14.0	7.0	25.1	2.3	4.2
43	233	335	8.0	8.5	8.0	2.9	20.5	4.6
44	236	302	13.0	8.4	2.6	.	.	.
45	240	330	5.6	2.2	2.0	.	.	.
46	244	305	5.4	2.6	2.0	.	.	.
47	246	305	14.0	3.2	2.2	.	.	.
48	247	302	18.0	14.0	8.0	.	.	.
49	252	335	13.2	2.7	1.6	.	.	.
50	253	332	5.0	3.5	1.4	8.5	6.5	2.3
51	254	332	7.8	3.4	1.1	19.5	5.9	6.1
52	257	315	14.0	3.1	1.1	22.9	2.4	0.8
53	260	335	6.6	5.5	1.9	.	.	.
54	263	345	3.0	2.6	1.8	.	.	.
55	264	296	8.9	4.7	0.8	9.8	1.8	1.1

Table 2 (continued)

OBS	DATE	TDS	TURIN	TURN	TUREFF	TSSIN	TSSN	TSSEFF
56	265	305	6.7	3.6	1.0	.	.	.
57	266	315	8.9	14.0	1.3	13.8	1.7	0.9
58	267	305	9.2	2.8	1.0	12.1	2.3	0.7
59	268	308	13.0	2.3	1.1	.	.	.
60	271	295	21.0	42.0	1.4	57.5	5.3	0.4
61	272	310	8.3	3.2	1.3	.	.	.
62	273	310	16.0	3.8	1.4	20.9	2.2	0.2
63	274	302	8.1	3.0	1.4	.	.	.
64	275	305	6.8	2.0	1.2	.	.	.
65	278	380	15.0	31.0	1.4	19.4	4.2	0.8
66	279	330	7.0	2.8	1.8	.	.	.
67	280	340	6.8	22.0	1.1	11.9	5.8	4.8
68	281	305	5.0	2.1	1.0	.	.	.
69	282	335	5.6	17.0	1.3	6.4	2.9	2.1
70	285	268	6.8	17.0	1.0	8.4	4.4	0.1
71	286	260	4.0	2.1	1.2	.	.	.
72	287	262	6.5	2.3	1.5	.	.	.
73	288	320	20.0	12.0	1.6	49.4	2.6	0.8
74	289	340	36.0	6.0	3.0	86.6	6.4	2.6
75	292	425	23.0	4.2	2.0	.	.	.
76	293	420	16.0	32.0	2.1	32.3	6.4	1.5
77	294	380	7.4	3.8	2.0	.	.	.
78	295	378	22.0	24.0	2.2	57.2	4.3	1.9
79	299	310	8.2	2.8	1.6	.	.	.
80	300	345	19.0	23.0	2.0	46.1	3.4	1.1
81	301	360	10.0	3.5	2.2	.	.	.
82	302	330	10.0	22.0	2.3	20.4	2.0	2.7
83	303	330	18.0	22.0	2.2	28.8	1.8	2.8
84	307	332	9.8	1.9	2.5	23.5	6.0	2.6
85	308	310	12.0	18.0	2.6	18.1	5.4	4.5
86	309	325	18.0	8.2	2.6	49.9	6.0	5.8
87	310	310	14.0	2.7	2.6	.	.	.
88	313	312	32.0	12.0	2.2	.	.	.
89	314	330	21.0	9.5	2.7	63.1	5.5	4.9
90	315	332	40.0	20.0	2.7	10.6	1.1	0.7
91	316	324	10.0	18.0	2.5	20.9	2.9	0.3
92	317	325	9.5	4.6	2.9	.	.	.
93	320	282	14.0	3.6	1.8	.	.	.
94	321	300	20.0	3.8	2.4	55.8	4.7	2.6
95	322	292	21.0	3.2	2.6	.	.	.
96	323	350	24.0	24.0	2.5	58.3	5.4	2.7
97	327	302	44.0	5.6	2.7	77.5	4.1	1.6
98	328	340	45.0	21.0	3.0	52.4	3.9	1.5
99	329	430	82.0	21.0	7.8	0.3	15.8	8.1

Table 3

Data: Date Versus Flow Rate, pH and Ammonia Concentration of Influent, Mid-point and Effluent

OBS	DATE	FLOWRATE	PHIN	PHM	PHEFF	AMMIN	AMMM	AMMEFF
1	175	0.13	7.5	7.1	7.7	5.0	12.0	14.0
2	176	0.07	7.1	7.2	7.9	6.5	14.5	15.1
3	177	0.11	7.4	7.2	7.9	6.5	12.9	12.0
4	180	0.12	7.5	7.3	8.0	8.1	8.9	8.2
5	181	0.12	7.6	7.3	7.9	5.7	8.6	11.0
6	182	0.12	7.4	7.2	8.0	8.8	13.8	12.7
7	183	0.12	7.6	7.2	8.0	7.0	19.0	18.1
8	187	0.01	.	.	.	.	.	.
9	188	0.09	7.2	7.0	7.8	9.8	11.5	12.9
10	190	0.01	.	.	.	.	.	.
11	191	0.12	7.0	7.1	7.9	14.0	15.5	12.0
12	194	0.01	.	.	.	.	.	.
13	195	0.12	7.1	7.2	8.1	5.6	13.8	14.0
14	196	0.12	7.2	7.2	8.0	6.6	15.0	13.8
15	197	0.12	7.2	7.3	8.0	5.6	12.1	11.5
16	198	0.09	7.9	7.3	8.1	4.1	14.0	14.6
17	201	0.12	7.1	7.1	8.0	4.3	8.0	6.5
18	202	0.12	7.1	7.0	7.9	7.7	10.6	7.6
19	203	0.11	7.1	7.1	7.9	5.0	10.5	6.4
20	204	0.12	7.1	7.0	7.9	9.1	11.5	6.3
21	205	0.11	7.5	7.2	7.9	7.6	16.5	9.4
22	209	0.11	7.2	7.3	8.0	4.7	13.2	9.7
23	210	0.11	7.3	7.3	7.9	6.6	12.5	7.9
24	211	0.12	7.5	7.5	7.8	8.0	12.4	7.3
25	212	0.12	7.2	7.7	7.8	6.5	13.0	6.8
26	215	0.01	.	.	.	.	.	.
27	216	0.12	8.1	7.7	7.7	1.3	9.0	1.6
28	217	0.12	8.0	7.7	7.8	1.1	7.9	0.9
29	218	0.12	7.9	7.8	7.7	1.3	6.0	0.0
30	219	0.12	7.9	8.0	7.9	1.1	8.0	7.9
31	221	.	.	.	.	.	.	.
32	222	0.12	7.6	7.9	7.9	1.9	3.6	0.2
33	223	0.11	7.8	7.9	7.9	1.9	3.9	0.2
34	224	0.11	7.9	7.8	7.9	1.3	4.3	0.4
35	225	0.11	7.9	7.8	7.9	3.0	4.4	2.2
36	226	.	.	.	.	.	.	.
37	227	.	.	.	.	.	.	.
38	228	0.10	7.9	7.8	7.9	0.2	2.6	0.2
39	229	0.13	7.9	8.2	8.0	1.5	2.7	0.2
40	230	0.12	8.0	8.0	8.0	1.3	2.7	0.1
41	231	0.12	7.8	8.1	7.9	1.4	2.8	0.1
42	232	0.12	7.7	8.1	7.9	0.9	3.1	0.1
43	233	0.10	7.7	7.8	7.9	0.3	2.9	0.2
44	236	0.11	7.4	7.9	7.8	.	.	.
45	240	0.11	7.8	7.7	7.6	.	.	.
46	244	0.12	7.6	7.8	7.8	.	.	.
47	246	0.12	7.5	7.7	7.5	.	.	.
48	247	0.11	7.5	7.9	7.8	.	.	.
49	252	0.11	7.8	7.9	7.7	.	.	.
50	253	0.11	7.8	8.2	7.8	1.4	4.6	0.8
51	254	0.11	7.8	7.9	7.8	1.4	4.2	0.5
52	257	0.11	7.3	7.9	7.8	2.8	2.2	0.0
53	260	0.12	7.6	7.7	7.6	.	.	.
54	263	0.12	7.6	7.6	7.5	.	.	.
55	264	0.12	7.7	7.7	7.9	1.3	1.4	0.0

Table 3 (continued)

OBS	DATE	FLOWRATE	PHIN	PHR	PHEFF	AMMIN	AMMR	AMMEFF
56	265	0.06	7.8	7.7	7.7	.	.	.
57	266	0.11	7.8	7.8	7.7	0.9	3.8	0.4
58	267	0.12	7.6	6.6	6.6	1.4	3.7	0.2
59	268	0.12	7.7	7.7	7.7	.	.	.
60	271	0.11	7.3	7.5	7.6	3.5	3.5	0.2
61	272	0.12	7.6	7.6	7.5	.	.	.
62	273	0.12	7.4	7.4	7.5	3.5	6.6	0.7
63	274	0.12	7.3	7.4	7.4	.	.	.
64	275	0.11	7.6	7.6	7.5	1.4	5.8	0.7
65	278	0.11	7.2	7.3	7.5	2.2	5.5	1.7
66	279	0.12	7.3	7.4	7.5	.	.	.
67	280	0.12	7.3	7.6	7.7	1.1	5.1	3.9
68	281	0.12	7.2	7.8	7.6	.	.	.
69	282	0.12	7.3	7.4	7.3	0.9	7.2	3.3
70	285	0.11	7.3	7.5	7.6	0.9	2.3	0.4
71	286	0.11	7.3	7.6	7.5	.	.	.
72	287	0.12	7.3	7.5	7.5	.	.	.
73	288	0.12	7.5	7.7	7.6	3.5	7.4	4.3
74	289	0.11	7.5	7.6	7.7	15.5	9.6	4.4
75	292	0.11	6.8	7.1	7.5	.	.	.
76	293	0.11	7.8	7.5	7.7	5.7	9.2	5.5
77	294	0.11	7.8	7.3	7.6	.	.	.
78	295	0.11	7.8	7.6	7.6	5.4	7.8	3.9
79	299	0.11	6.8	7.3	7.4	.	.	.
80	300	0.11	7.6	7.7	7.6	12.5	9.9	4.6
81	301	0.11	7.3	7.5	7.5	.	.	.
82	302	0.11	7.9	7.6	7.6	4.6	7.6	4.3
83	303	0.12	7.5	7.6	7.5	2.2	7.4	3.4
84	307	0.12	7.6	7.5	7.6	2.2	4.2	0.4
85	308	0.11	7.6	7.6	7.5	4.7	6.4	1.9
86	309	0.11	7.6	7.6	7.6	5.3	6.8	2.3
87	310	0.11	7.1	7.4	7.4	.	.	.
88	313	0.11	7.3	7.6	7.7	.	.	.
89	314	0.11	7.9	7.7	7.7	4.0	5.4	4.2
90	315	0.11	7.8	7.7	7.7	3.7	7.6	3.4
91	316	0.11	7.5	7.6	7.8	6.5	8.6	3.7
92	317	0.11	7.9	7.5	7.7	.	.	.
93	320	0.11	7.7	7.5	7.7	.	.	.
94	321	0.11	7.9	7.7	7.7	3.5	4.8	1.3
95	322	0.12	7.6	7.5	7.6	.	.	.
96	323	0.11	7.9	7.8	7.7	4.8	4.6	0.8
97	327	0.11	8.8	7.7	7.7	2.4	3.1	1.1
98	328	0.11	7.7	7.9	7.9	9.2	8.2	3.5
99	329	0.12	7.8	7.6	7.8	6.0	10.8	6.2

Table 4 Statistical Analysis of the Parameters

VARIABLE	N	MEAN	STANDARD DEVIATION	MINIMUM VALUE	MAXIMUM VALUE	STD ERROR OF MEAN	SUM	VARIANCE	C.V.
FLOWRATE	96	0.10895833	0.02291766	0.01000000	0.13000000	0.00233902	10.460000	0.0005252	21.033
PHIN	92	7.55000000	0.31951199	6.80000000	8.80000000	0.03331143	694.600000	0.1020879	4.232
PHM	92	7.55543478	0.29921661	6.60000000	8.20000000	0.03119549	695.100000	0.0895306	3.960
PHEFF	92	7.72608696	0.21680475	6.60000000	8.10000000	0.02260346	710.800000	0.0470043	2.806
TDS	92	358.79347826	69.58740856	260.00000000	550.00000000	7.25498886	33009.000000	4842.4074295	19.395
TURIN	90	17.20111111	12.78218119	3.00000000	82.00000000	1.34736020	1548.100000	163.3841561	74.310
TURM	90	12.51666667	9.89284442	1.90000000	42.00000000	1.04279736	1126.500000	97.8683708	79.037
TUREFF	90	3.96888889	3.26908036	0.60000000	22.50000000	0.34459133	357.200000	10.6868864	82.368
AMMIN	67	8.41343284	3.31506641	0.20000000	15.50000000	0.40499982	295.700000	10.9896653	75.113
AMM	67	7.89552239	4.19738132	1.40000000	19.00000000	0.51279174	529.000000	17.6180100	53.162
AMMEFF	67	4.83731343	4.90463095	0.00000000	18.10000000	0.59919604	324.100000	24.0554048	101.392
TSSIN	65	37.75692308	26.76672356	0.30000000	99.70000000	3.32000345	2454.200000	716.4574904	70.892
TSSM	65	30.19692308	56.74773665	1.10000000	356.60000000	7.03869045	1962.800000	3220.3056154	187.926
TSSEFF	65	5.03076923	5.75456132	0.10000000	28.60000000	0.71376549	327.000000	33.1149760	114.387
TEMPIN	70	83.41428571	5.89212901	69.00000000	94.00000000	0.70424412	5839.000000	34.7171843	7.064
TEMPM	70	72.27142857	5.91956086	63.00000000	90.00000000	0.70752285	5059.000000	35.0412008	8.191
TEMPEFF	70	67.94285714	5.75750478	55.00000000	86.00000000	0.68815344	4716.000000	33.1488613	8.550
TOCIN	28	25.55714286	18.76492845	5.00000001	87.70000000	3.54623815	715.600000	352.1225397	73.423
TOCM	28	10.16785714	10.09198632	0.00000001	41.60000000	1.90720615	284.700000	101.8481678	99.254
TOCEFF	28	5.65714286	7.43557159	0.00000000	31.60000000	1.40519095	158.400000	55.2877249	131.437
GAS	31	0.01638710	0.02792571	0.00000000	0.14100000	0.00501561	0.508000	0.0007798	170.413