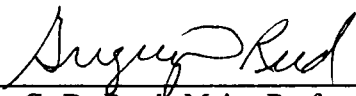


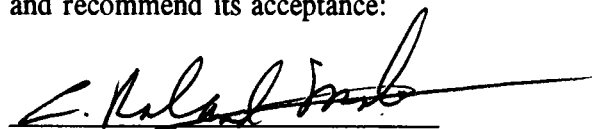
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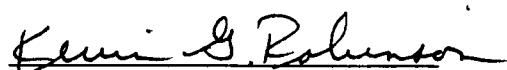
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**DEVELOPMENT OF AN ON-SITE SUBSURFACE HORIZONTAL
FLOW WASTEWATER TREATMENT SYSTEM**

A Thesis

Presented for the

Master of Science

Degree

The University of Tennessee, Knoxville

Christine M. Rudolph

December 1992

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DEDICATION

This thesis is dedicated to my parents,

Mr. Giles A. Rudolph and Mrs. Christine Rudolph

Without their love, confidence, and support, this would not have been achievable.

I love you both.

ACKNOWLEDGMENTS

I would like to thank my major professor, Dr. Gregory D. Reed, for his guidance and enduring patience over the last six years. I would also like to thank the other committee members, Dr. Roland Mote and Dr. Kevin Robinson, for their comments and assistance. I would like to express my thanks to my parents for their faith and confidence in me to finish. You gave me the drive and determination to achieve whatever goals I set for myself. I would also like to thank LeRoy for his help in collecting the valuable data for this research. LeRoy did an outstanding job. Finally, I would like to thank my new husband, Mike Goddard, for his love, patience, and help.

ABSTRACT

This research attempted to determine the performance characteristics of an anaerobic up-flow fixed-film biological reactor (ANFLOW), in series with a standard septic tank and a subsurface horizontal flow (SSHF) bed, for use as an on-site wastewater treatment system. Two wastewater treatment systems were studied in Pigeon Forge, Tennessee. The performance evaluation was based on the collection of wastewater samples. The samples were analyzed for pH, temperature, biological oxygen demand (BOD), chemical oxygen demand (COD), total suspended solids (TSS), ammonia, and fecal coliforms. The data collected showed that the ANFLOW unit in conjunction with the SSHF bed proved to be a very effective wastewater treatment system. The ANFLOW unit enhanced the septic tanks performance such that the SSHF bed could be reduced in size. System start-up was rapid for the SSHF bed because the ANFLOW unit served as pretreatment for the SSHF bed. The data collected showed that the SSHF beds studied tended to follow first-order reaction kinetics, where the organic removal efficiency of the SSHF bed tended to be independent of the influent concentrations and dependent on the length (directly) and flow (inversely). It was concluded that the SSHF bed was most likely an anaerobically controlled system that is not sensitive to the amount of oxygen diffusion into the SSHF bed for BOD and COD removal. In conjunction with the ANFLOW unit, the SSHF bed will provide an energy-effective on-site wastewater treatment process for residential areas and small commercial businesses.

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LIST OF ABBREVIATIONS

anaerobic upflow fixed-film reactor	ANFLOW
biological oxygen demand	BOD
chemical oxygen demand	COD
Environmental Protection Agency	EPA
gallons per day	gpd
hydraulic retention time	HRT
kilograms per million gallons	Kg/MG
mean cell residence time	MCRT
million gallons per day	MGD
solids retention time	SRT
suspended solids	SS
subsurface horizontal flow	SSHf
total suspended solids	TSS
Tennessee Valley Authority	TVA
United Kingdom	UK
Water Research Center	WRC

CHAPTER I

INTRODUCTION

A considerable need exists to find effective on-site wastewater treatment processes for residential and small commercial properties located in areas unsuitable for standard septic tank leaching fields. The oldest, simplest, and most widely used treatment process for on-site treatment of wastewaters is the septic tank system. With a suitable soil, the disposal of sewage with a septic tank system can be simple, economical, and inoffensive. Where rock or ground water is close to the surface or the soil is a tight clay, some other form of treatment may be required. Research continues to develop feasible modifications to the standard septic tank system such that on-site treatment is still an option with the above restrictions. (1) This thesis is a performance study to determine the feasibility of using an anaerobic upflow fixed-film biological reactor (ANFLOW unit) in series with a standard septic tank and a subsurface horizontal flow (SSHF) bed as a wastewater treatment system for the treatment of domestic sewage.

The proposed treatment system is a passive system which requires little or no operational or maintenance costs. The ANFLOW component was developed by researchers at the Oak Ridge National Laboratories and is discussed in Chapter II. The SSHF bed is a modification of an artificial wetland bed which uses characteristics of both an artificial wetland and a fixed-media biofilter. The SSHF bed is discussed in detail in Chapter II.

Two demonstration projects were studied for this thesis. The Shagbark System and the Haynes House System were both located in Pigeon Forge, Tennessee. Both systems used a septic tank for the capture of solids and oil and grease, an equalization basin (Shagbark system only), an ANFLOW unit, and a SSHF bed. The Shagbark system's SSHF bed was 50 feet long. It was followed by a small chlorination system and a holding pond so that the final effluent from the system could be used for irrigation purposes. The Haynes House system's SSHF bed was 60 feet long and was followed by an existing drain field that accompanied the original wastewater treatment system. These systems are discussed in Chapter III.

Wastewater samples were collected at both locations and were analyzed for pH; temperature; biological oxygen demand (BOD); chemical oxygen demand (COD); total suspended solids (TSS); and ammonia, nitrate, nitrite, and fecal coliforms. The Shagbark samples were collected in three locations: influent to the ANFLOW unit, influent to the SSHF bed and effluent from the SSHF bed. The Haynes House samples were collected in six locations in the treatment system: influent to the ANFLOW unit, influent to the SSHF bed, 15 ft into the SSHF bed, 30 ft into the SSHF bed, 45 ft into the SSHF bed, and effluent from the SSHF bed. The Shagbark and Haynes data were used to determine performance characteristics for the treatment system. The data are presented in Chapter IV.

Chapter V discusses the overall performance of the treatment system and how its individual components' performance compares to the literature. Typical design equations for treatment technologies that are similar to the SSHF bed and the way the SSHF bed's performance is related to these equations are also discussed in Chapter V.

The effectiveness of the whole treatment system and its future applications are briefly discussed in Chapter VI. Additional research is needed to help solidify the data base required to develop and design SSHF bed treatment systems for the treatment of wastewaters from residential and small commercial properties.

CHAPTER II

LITERATURE REVIEW

ANAEROBIC FIXED-FILM TREATMENT

HISTORY

By far the most common disposal system employed in on-site sewage disposal is the subsurface leaching field. The substitution of a horizontal-flow subsurface system for a down-flow leaching field might require more area. One method of minimizing this area would be to improve the pretreatment. Since the septic tank operates anaerobically, additional pretreatment by an anaerobic process would be automatically compatible. Several anaerobic alternatives are available for consideration. Until recently, the professional wastewater treatment community perceived anaerobic wastewater treatment as a sensitive biological process that had a limited role in generic wastewater treatment and no place at all in the treatment of domestic sewage. Now a rapidly growing body of knowledge suggests that most applications of anaerobic treatment of wastewaters offers greater advantages than its aerobic counterpart. (2, 3)

Anaerobic processes have been used for the treatment of municipal and industrial wastes for more than a century. (3, 4, 5) Progress in the development of anaerobic treatment systems has been characterized by increased control over the process variables, especially microbial solids. The oldest, simplest, and most widely used treatment process for on-site treatment of wastewaters is the septic tank system, used as early as 1895. The anaerobic septic tank process has been used extensively in both the United States and Europe. This process, without temperature or solids control, can be expected to remove up to 75% of the BOD and 70 to 80%

of the suspended solids (SS). (2, 3) With a suitable soil, the disposal of sewage with a septic tank system could be simple, economical, and inoffensive. Where rock or ground water is close to the surface, the soil is a tight clay, or land availability for a drain field is minimal, some other form of treatment system may be required. (1)

In the 1950s, research efforts were made to incorporate the development of the activated sludge system into an anaerobic process that used sludge recycle. Also during this period, the biolytic tank was tested and is now commonly referred to as the Upflow Anaerobic Sludge Blanket. This process has been used in combination with clarifiers and rock filters. The possibility of using submerged rocks as an anaerobic filter was further tested in the 1960s. This possibility was followed by other processes, such as the expanded bed process, which immobilizes microbes on small inert particles and activated sludge systems. During the 1960s and 1970s, the fixed-film concept was generally overshadowed by these other systems. (2, 3) These anaerobic treatment processes are shown in Figure 2.1.

MICROBIOLOGY

The anaerobic treatment of complex wastes involves two stages. The first stage is referred to as acid fermentation or acetogenesis. In this stage, organic material is converted to organic acids, alcohols, and new bacterial cells. The second stage is referred to as methane fermentation or methanogenesis. This second stage uses the end products of the first stage and converts them to gases (mainly methane and carbon dioxide). The two stages of anaerobic waste treatment and its sequential nature are shown in Figure 2.2. It should be noted that even though the anaerobic processes are presented in sequential nature, both stages occur simultaneously and synchronously in an active, well-buffered system.

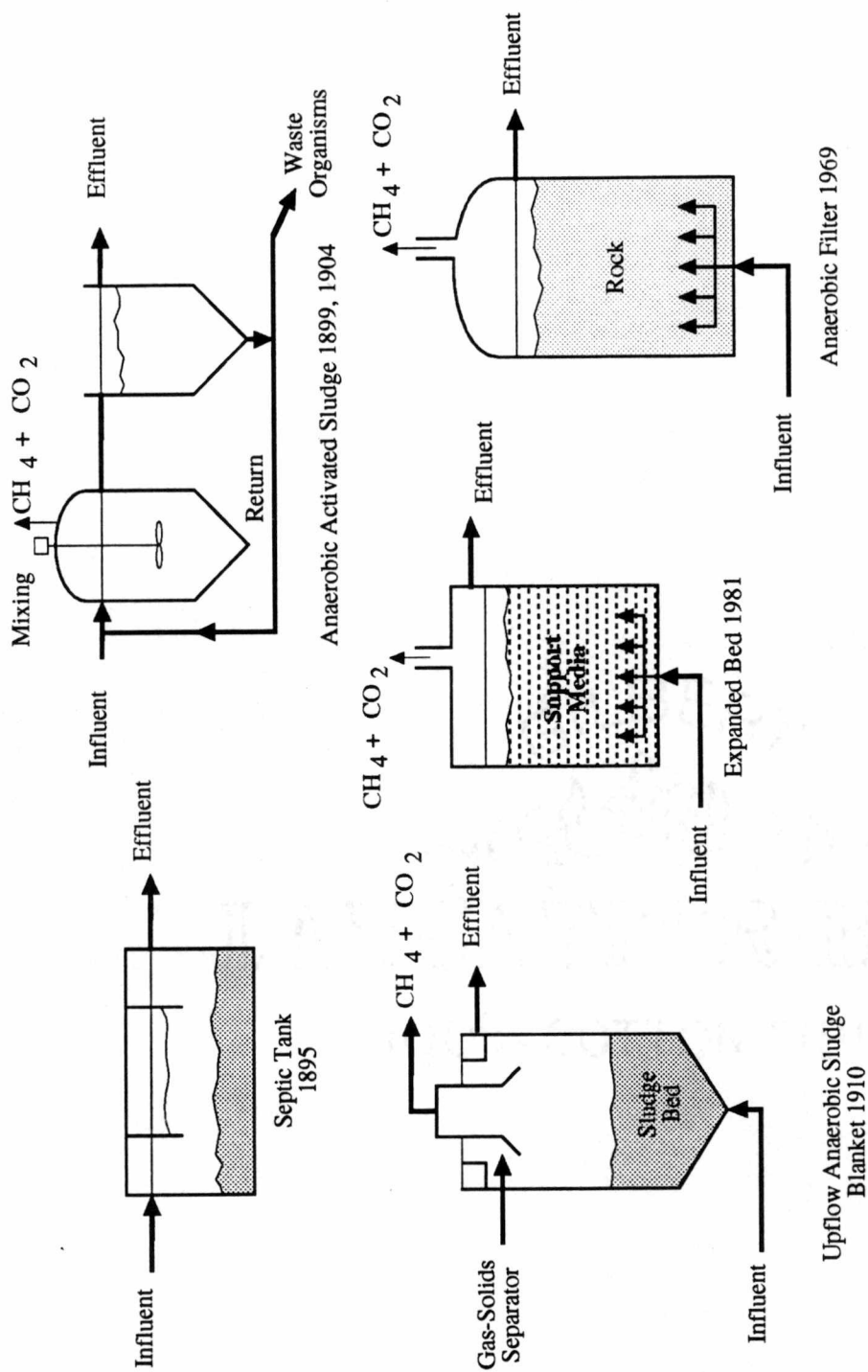


Figure 2.1
Anaerobic Treatment Processes (2)

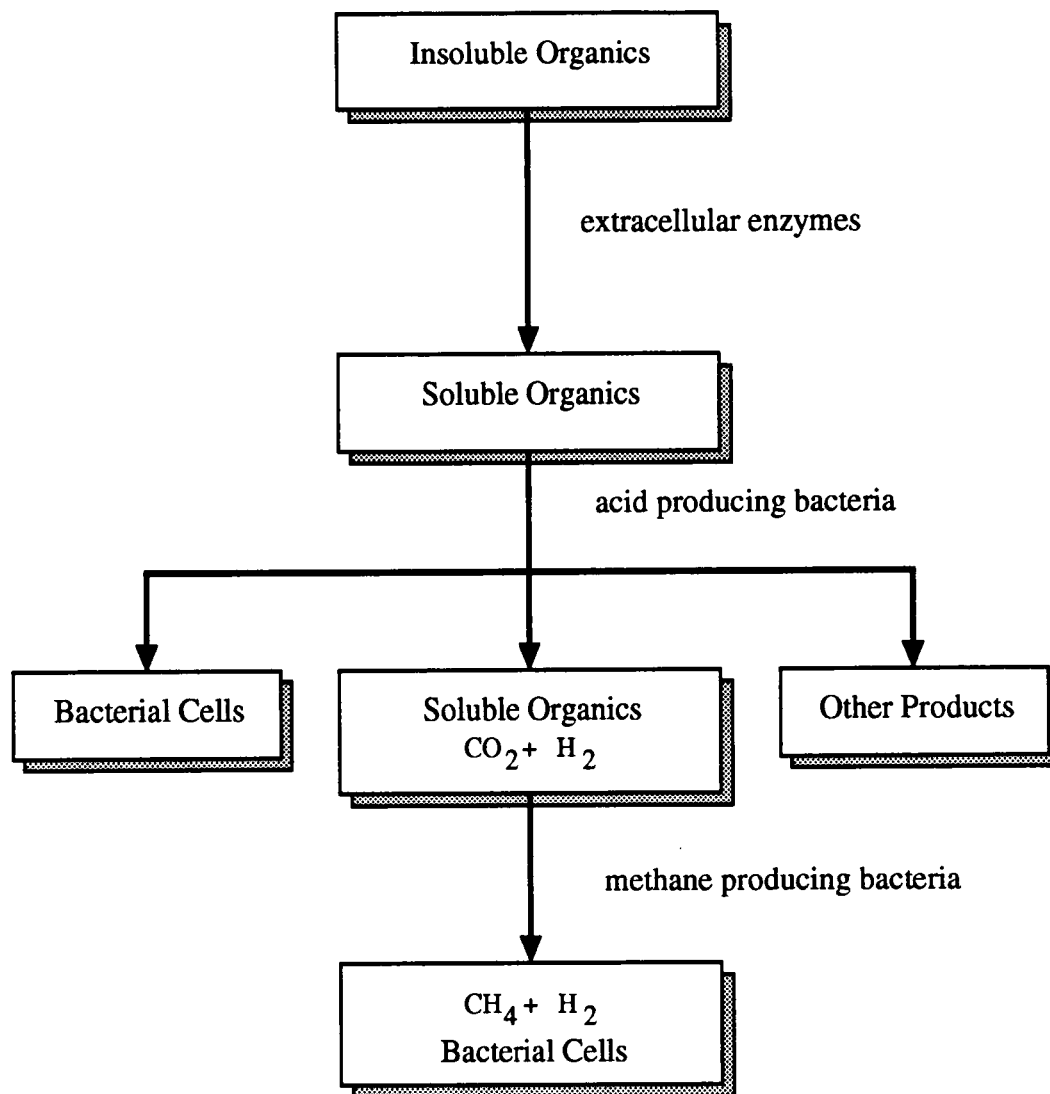


Figure 2.2
Stages of Anaerobic Waste Treatment (8)

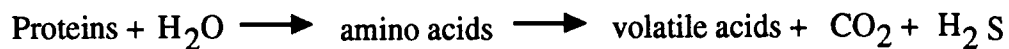
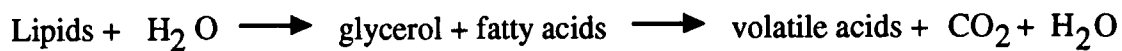
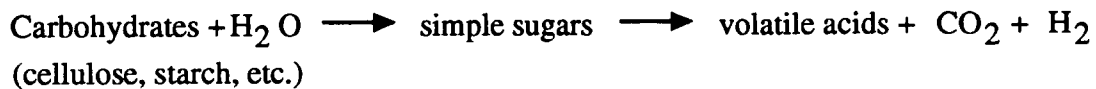
Figure 2.3 represents some of the typical reactions that take place during acetogenesis and methanogenesis. Figure 2.4 illustrates the significance of primary acids produced during acid fermentation (propionic and acetic) as precursors for methane formation. The percentages will vary according to the strength and composition of the incoming waste. It should also be noted that while acetogenesis breaks down organics to acids and CO_2 , little stabilization of BOD or COD is realized. It is not until methanogenesis occurs that true stabilization of the organic material occurs and the overall BOD or COD is reduced. (6)

ADVANTAGES/DISADVANTAGES

Anaerobic treatment of organic wastes has certain advantages that make it more desirable as a wastewater treatment process than aerobic treatment. The major advantages include the treatability of stronger wastes, lower sludge production rates, the possibility of intermittent operation, and a resilience to toxic effects. (2, 3, 4, 7, 8, 9, 10)

By definition, anaerobic treatment takes place in the absence of oxygen, offering certain advantages over aerobic processes. With strong wastes, aerobic processes are often unstable; intense mixing is needed for high oxygen transfer, which consumes energy and produces much more sludge. (3, 4) For anaerobic processes, no oxygen is required, and thus no energy is required for oxygen transfer. More energy efficient than aerobic processes, anaerobic treatment can save as much as 20 million BTUs per ton of aerobically oxidized COD. With a well-planned and well-designed fixed-film anaerobic system, sludge production could be approximately 30% of that of an aerobic fixed film system and less than 10% of that of a conventional system. (6) This is by far the largest difference between aerobic and anaerobic systems. More than half of the substrate removed from aerobic systems yields new

Acetogenesis



Methanogenesis

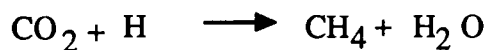
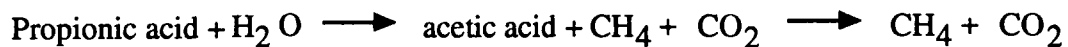


Figure 2.3
Typical Microbial Reactions (6)*

*Equations are not stoichiometrically balanced.

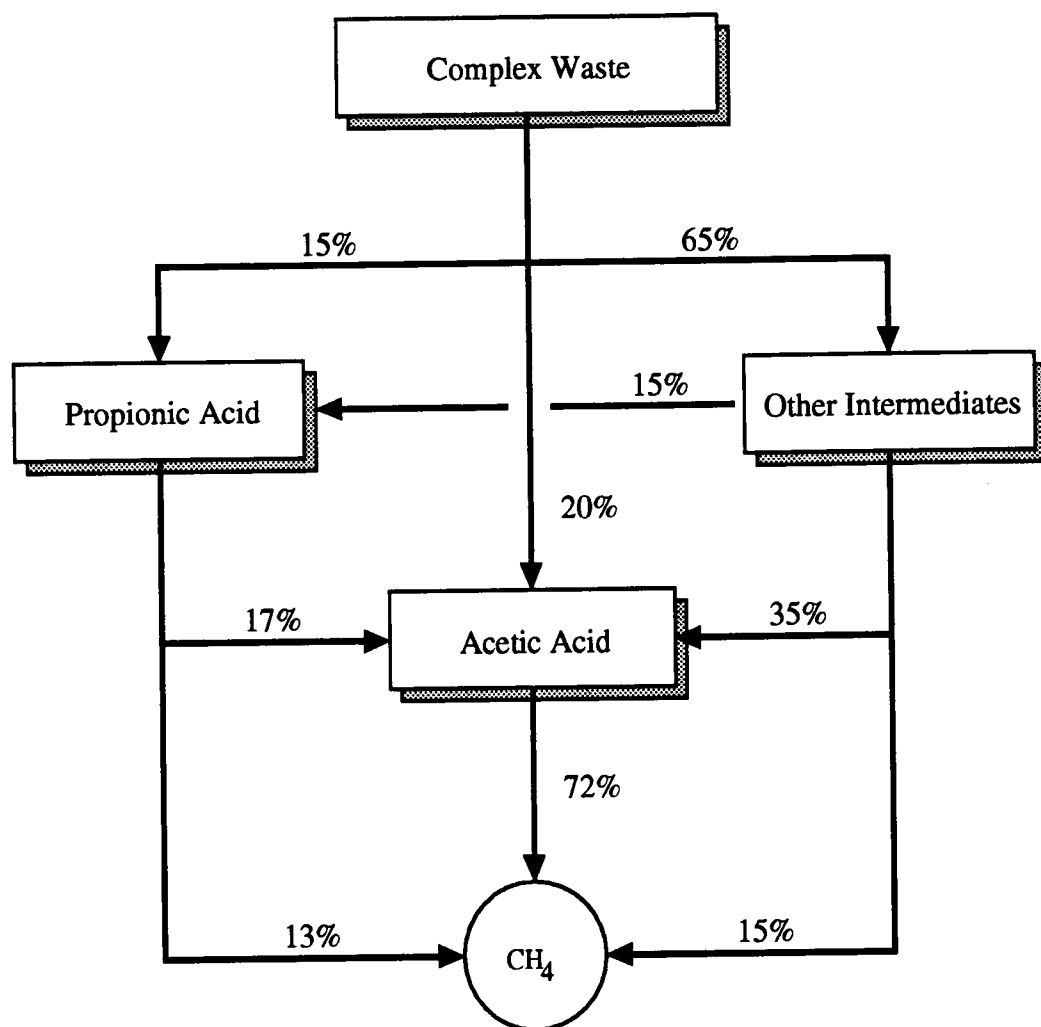


Figure 2.4
Percent COD Conversion in Methane Fermentation of Wastes (8)

microbial mass or sludge, whereas the sludge yield under anaerobic conditions is usually less than 15% of the organic substances removed. (2, 3, 8) With lower sludge production rates come lower nutrient requirements. Anaerobic treatment not only reduces sludge that is troublesome to dispose of; it also results in the production of a natural gas, methane. (2, 3, 6, 7)

Due to rising operational costs of suspended growth systems and recent developments of lightweight, low cost-improved filter media, interest has been revived in fixed-film systems. It is now possible to construct fixed-film biological systems that can be smaller in size but otherwise technologically and economically competitive with activated sludge systems. The operational principle of anaerobic fixed-film reactors is based on the fact that their attachment to the media prevents them from being washed out of the reactor. The capability of having a long solids retention time (SRT) independent of hydraulic retention time (HRT) translates into improved space-loading rates and COD removal capabilities. (9) With a long SRT, the net production of biological solids is so slow that solids wasting may not be required for relatively long periods of time. The process is relatively insensitive to variable organic loading rates, can accept severe shock loads without experiencing upsets, and is capable of rapid restart after relatively long periods (weeks to months) without feed. (2, 10) Re-establishment of normal reactor performance usually occurs 12 to 48 hours after upset. (11) Anaerobic systems are also capable of intermittent operation due to some of their resistance to environmental factors or effects. (7)

Fixed-film biological processes may be better able to withstand and recover from attacks by toxic substances than other processes. A slug of toxic material might kill all of the bacteria in an activated sludge system, requiring it to be taken down, cleaned, and refurbished; but a with fixed-film biological system, the toxic material could kill even a large portion of the biomass,

which would slough off the medium and leave a much thinner film or even some bare spots. After the end of the toxic shock, the biofilm would begin to regenerate. Both suspended and attached growth systems are basically resilient to toxic effects. The fixed-film biological system, however, comes back faster after an acute toxic episode. (12)

If the toxicity is more chronic and if properly designed and built, fixed-film systems, can provide solids retention necessary to prevent biomass sloughing or washout before acclimation. It was pointed out that fixed-film anaerobic systems have two major advantages over completely mixed reactors for treating toxic wastewater: a higher inherent SRT and a hydraulic mode (plug flow) that can pass slug doses of toxicants quickly. (3, 7)

There are some disadvantages to anaerobic treatment systems. These disadvantages include difficulties with startup, sensitivity to temperature, and a need for post-aeration. (3, 8, 9, 12) One of the serious problems limiting the full-scale application of anaerobic fixed-film processes is the reactor startup. Startup of these systems is generally time-consuming and often difficult because of the slow growth rate of acetate-using methanogens, an occasional inadequate nutrient supply in the wastewater, and the unsteady nature of startup. Fixed-film biological systems need a long time, sometimes as long as three months, to achieve steady state. (3, 9)

Anaerobic fixed-film processes are usually used to treat wastewaters low in suspended solids because of problems associated with clogging of the filter media. Flow distribution becomes a problem when the biological solids concentration increases to a point where channelization develops. If wastewaters with high solids concentrations are to be treated by a fixed-film system, some form of pretreatment may be required. In combining the ANFLOW unit

with the septic tank, a solids separation process (such as the septic tank used in this study) could serve as a source of pretreatment to remove large solids and oil and grease that may clog the packing media of an anaerobic fixed-film unit. (8)

Anaerobic systems are also sensitive to temperature and pH changes, due to the inherent nature of the biomass. Close control of pH by chemical additions or dilutions and/or additions of buffering solutions usually serve to control any pH problems. (3, 6) Temperatures around 25 degrees C are generally recommended for anaerobic systems. Because of the high SRT of fixed-film systems, they have been successfully operated in the 15 to 25 degree C range. (3, 6)

Odors represent a potentially large public relations problem for wastewater treatment plants, and a great deal of effort is expended to eliminate or mask the odors. There are three sources of odors from anaerobic treatment processes: off gas, effluent, and waste solids. (12) Because of the greater mass of the effluent stream, special care will have to be taken to control the odors. The most reasonable solution to this problem appears to be aeration of the effluent immediately upon its exiting the biological reactor. (12)

PERFORMANCE

Septic Tank

A septic tank is a watertight tank designed to slow down the movement of raw sewage and wastes passing through so that solids can separate or settle out and be broken down by the liquefaction and bacterial action. The tank does not purify the sewage, eliminate odors, or destroy all of the solid matter. The septic tank simply conditions the sewage so that it can be disposed of to a subsurface leaching system without prematurely clogging the system. (1)

Septic tank volume is estimated using a flow rate of 75 gal per capita per day or 150 gal per bedroom per day, a detention time of 1 to 3 days, and a sludge accumulation volume. The minimum septic volume for any household is 750 gal. A septic tank for a typical home generally requires cleaning or removal of the sludge every 3 to 5 years. The septic tank should be designed with a liquid depth of approximately 4 ft and a length-to-width ratio of 1, 2, or 3. (1, 13)

Typical influent concentrations for domestic sewage to a septic tank average 200 mg/l for BOD, 400 mg/l for COD, 200 mg/l for TSS, 30 mg/l for ammonia, and 20 mg/l for organic nitrogen. These figures represent an organic loading rate of approximately 0.2 lb of BOD per day per person and 0.2 lb TSS per day per person. Since the septic tank functions as a primary sedimentation tank and the expected removal efficiencies average between 30 and 35% for the sedimentation tank, it is expected that removal efficiencies would be the same for the septic tank. (1, 13)

Effluent from the septic tank is typically discharged to a subsurface leaching field if the soil is suitable. This is the topic of discussion in the section on the SSHF bed. In an effort to determine if fixed-film technology could enhance the septic tank's performance before discharge to the subsurface leaching field or a surface release, an ANFLOW unit was combined with the septic tank system. If the septic tank's performance could be enhanced, then the organic loading to the receiving system may be reduced, and thus the size of the receiving system may be reduced. The ANFLOW units performance is discussed below.

ANFLOW

The ANFLOW system is a fixed-film biological wastewater treatment process in which a fixed media matrix is used to retain anaerobic biological solids within a reaction vessel so that treatment occurs as wastes flow up through the unit and come into contact with the microorganisms contained therein. The original tests conducted by Young and McCarty and by other investigators between 1968 and 1980 suggest that a significant part of the biological solids in upflow anaerobic filters was not attached to the media but was trapped within the void spaces. (10, 14) Thus, COD removal efficiency performance is related more to the way in which wastes flow through the media than to the unit surface area of the media. Therefore, an open tubular media would capture solids without hindering the hydraulic flow. In addition, test data showed that a significant plug flow component exists in most operational anaerobic filters. (10, 14)

Pilot plant studies were conducted at Oak Ridge and Knoxville, Tennessee, by Oak Ridge National Laboratories to evaluate the performance of ANFLOW. The pilot plant at Oak Ridge treated municipal wastewaters at a flow rate of 5,000 gal/day (gpd). The Knoxville pilot plant operated at 50,000 gpd, also treating municipal wastes. Both studies demonstrated that an ANFLOW bioreactor could be used to treat raw wastewater at ambient temperatures while achieving significant reductions in total suspended solids (TSS) and BOD. TSS removal efficiencies ranged from 78 to 80%, with BOD removal efficiencies of 63 to 70%. Approximately one-half to two-thirds of the BOD was removed in the first 3 ft of the ANFLOW unit. The HRTs ranged from 9 to 10 hours for both pilot plants. Their performance was maintained in ambient cold weather tests from 12 to 25 degrees C. (12, 15)

Twenty percent of the influent carbon was converted to methane, and 30% remained in the reactor as sludge. The balance of the carbon was converted to CO₂ or remained in the liquid effluent as TSS and BOD. The off gas produced was recovered and found to be 60 to 70% methane by volume. More than half of the methane was dissolved in the effluent; this methane could be recovered along with the methane in the off gas. The gas production rate increased with higher-strength wastes. (12, 15)

Dry solids accumulation in the reactor was approximately 150 Kg/MG of wastewater treated. This amount varied according to temperature changes. An increase in sludge production occurred with a decrease in temperature due to reduced biological activity, which normally converts solids to off gas. The solids accumulation represents a 75% reduction in solids production compared to primary sedimentation, activated sludge, and trickling filters. With the fixed-film process, the bacteria are retained in the packing material and are not washed off in the effluent. This retention provides for a long mean cell residence time (MCRT), approximately 100 days. It was also demonstrated that the ANFLOW reactor could be operated under the above conditions for extended periods without plugging. Discharge of the accumulated solids in the reactor was accomplished easily by a gentle stream of water from a hose. (12, 15)

As discussed in the previous section, ANFLOW's tolerance to and recovery from toxicants is fairly rapid and may not require reinoculation, because only the surface of the film appears to be affected. Odor control caused by ammonia in the effluents poses a problem that needs to be handled carefully and efficiently.

It appears that the ANFLOW process will require some additional processes to meet secondary standards. Preliminary treatment will be required to remove grit, solids, and grease that may clog the packing media. The use of a septic tank before the use of the ANFLOW unit should accomplish this. To further reduce the TSS and BOD and to take care of the odor problem, a polishing step will be required for the effluent from the reactor. (12) It should be noted that the increased cost to provide an effluent polishing step to the ANFLOW process will reduce the design flow rate where break-even capital costs were estimated. (16) These observations lead to the next topic of discussion, Subsurface Horizontal Flow (SSHf) Bed.

SUBSURFACE HORIZONTAL FLOW BED

By far the most common disposal system employed in on-site sewage disposal is the subsurface leaching field. It consists of excavations or trenches filled with gravel through which distribution pipes run. Effluent from the septic tank comes out of holes, slots, or joints in the distribution pipe and runs over and through the gravel into the soil matrix. The effluent percolates down through the soil and is cleansed by physical and biological mechanisms. Typical dimensions of the trenches are from 18 in. to 3 ft wide and from 18 in. to 4 ft deep. Several methods are available for determining the amount of bottom area required for the trenches, which, in turn, determine the length of the trenches. Local or state health departments may have their own methods, but the three most common methods are the *Uniform Plumbing Code*, *The Manual of Septic Tank Practice* and the Environmental Protection Agency's design manual, *Onsite Wastewater Treatment and Disposal Systems*. All three methods are based on the characteristics of the soil and its percolation rate. Where a high water table, steep slopes, tight soils, very coarse soils, or shallow bedrock prevents the soil in the surrounding area from being

suitable for a drain field, other forms of treatment of the effluent will have to be considered. The investigation of the septic tank and ANFLOW unit and their modification to be more suitable for an on-site treatment system spawned the idea of using an artificial wetland instead of the standard drain field. (1, 13)

ARTIFICIAL WETLAND HISTORY AND PERFORMANCE

In conjunction with microorganisms, plants recycle waste and produce food. These fundamental facts have been known and taken for granted for hundreds of years. What authorities have not known, but are beginning to realize, is that natural biological processes involving relationships between plants and microorganisms can be used to correct environmental problems caused by waste disposal, industrial development, and environmental abuse. The acceptance and use of these natural biological processes will be essential for the future. (17)

The term "wetland" is used to describe a number of diverse environments that are typically of high productivity and share characteristics of both aquatic and terrestrial habitats. (18, 19) The U.S. Fish and Wildlife Service has developed a definition and classification system for all types of wetland habitats. It recognizes wetlands as a transition between terrestrial and aquatic systems. The dominant factor in determining the development of soils and associated biological communities where the water table is at or near the soil surface is water. The definition states that a system is a wetland if it fulfills one of these three conditions:

- The area should support a predominance of hydrophytes, at least periodically.
- The area should have a predominately undrained hydric soil. It must be wet enough for long enough to produce anaerobic conditions that limit the types of plants that can grow there.
- The area could be an area with a nonsoil substrate (such as rock or gravel) that is saturated or covered by shallow water at sometime during the growing season.

It is this last condition that applies to the modified artificial wetland bed termed Subsurface Horizontal Flow (SSHF) bed being used for this research. (20, 21)

Wetland uses include flood peak reduction, food and habitat for game and non-game animals, shoreline stabilization, groundwater recharge, and water quality improvement. (18, 19) The use of wetlands for wastewater treatment has been documented in the general literature over the last century. Most of the literature, however, deals with natural wetland systems. Interestingly, the use of wetlands for low-cost and energy-efficient treatment of municipal and agricultural wastewater has increased tremendously in recent years.

Why use an artificial wetland system instead of a natural wetland? Artificial wetlands - or rather, engineered wetlands -- offer several advantages over their natural counterpart. The physical processes of an artificial system are more readily managed. The type of vegetation and the ability to manipulate the flow path allow for more efficient use of space and for more flexibility in size and site selection. The artificial wetland offers an economically sound alternative while preserving aesthetic values and providing for enhancement of wildlife. (23, 24)

The concept of using wetlands for cost and energy-effective treatment of municipal wastewaters has been demonstrated in Europe and the United States with some success. However, to date, little literature has been published on the operational parameters and associated treatment efficiency of wetlands specifically designed for wastewater treatment. (25)

There are three major types of artificial wetlands currently in use today for wastewater treatment: marsh system, marsh-pond-meadow system, and root zone system. Each of these

systems has two types of discharge to the environment: seepage and discharge. Seepage wetlands have no final discharge point; instead, the water percolates into the soil, evaporates, or transpires into the atmosphere. Discharge wetlands have one or more well defined discharge points. (23) A few studies have been conducted in the United States using the marsh-pond-meadow system and the marsh system. The results of these studies are shown in Table 2.1.

The use of wetlands for treatment of wastewaters has been practiced in the United Kingdom (UK) for more than a century. As early as 1877, six cubic meters of sewage were reported to be applied daily per square meter of land. This practice resulted in the

Table 2.1 Wetland Studies

Wetland Type	Influent Type	Loading Rate	Removal Efficiencies:			Ref
			BOD	ammonia	TSS	
marsh	raw sewage	-----	96	85	--	(23)
M-P-M ^a	aerated raw sewage	420-855 ^c	92	86	89	(23)
M-P ^b	aerated raw sewage	420-855 ^c	89	58	88	(23)
M-P-M ^a	raw sewage	50,000 ^d	97	77	89	(28, 29, 30)
M-P-M ^a	raw sewage	30,000 ^d	96	87	94	(31)

^a M-P-M: marsh-pond-meadow system

^b M-P: marsh-pond system

^c m/ha-day

^d gal/acre-day

production of an offensive-smelling swamp. From this practice, other forms of land treatment of wastewaters were developed. The possibility of using aquatic plants to assist with the treatment by maintaining aerobic conditions in the soil was not understood until studies were started in Europe, the United States, and Canada 15 to 20 years ago. (26)

A method termed root zone has been scientifically developed by Dr. Reinhold Kickuth at the University of Hessen in Germany. In the root zone method, the purification of the wastewater takes place within the soil matrix, which is saturated with sewage and has been thoroughly penetrated by the plant roots. The wastewater enters the system below the soil surface and proceeds through the bed mainly horizontally. The sewage components in the soil matrix are degraded, fixed, and released to the atmosphere or assimilated by the vegetation. (26, 27) As shown in Figure 2.5, the root zone system is similar to a horizontal leaching field. A leaching field, however, does not have an impermeable liner or vegetative cover, as does the wetland bed.

Application rates of about 0.1m/m-day for the root zone system have been used to treat screened and degrittied sewage in Germany since 1974. The original design population for this system was 2700. The system has been extended in stages to treat wastewater for a population of 10,000 in 1985. The root zone method was first introduced in Denmark in 1983. (33)

Interest in the method was first aroused in the UK following a visit to Germany in 1984 by the Water Research Center (WRC). The visit concluded that the root zone method could be a cost-effective method to treat screened and degrittied sewage to produce a high- quality effluent. (26) The root zone method is being used in the UK for treatment of industrial wastewaters,

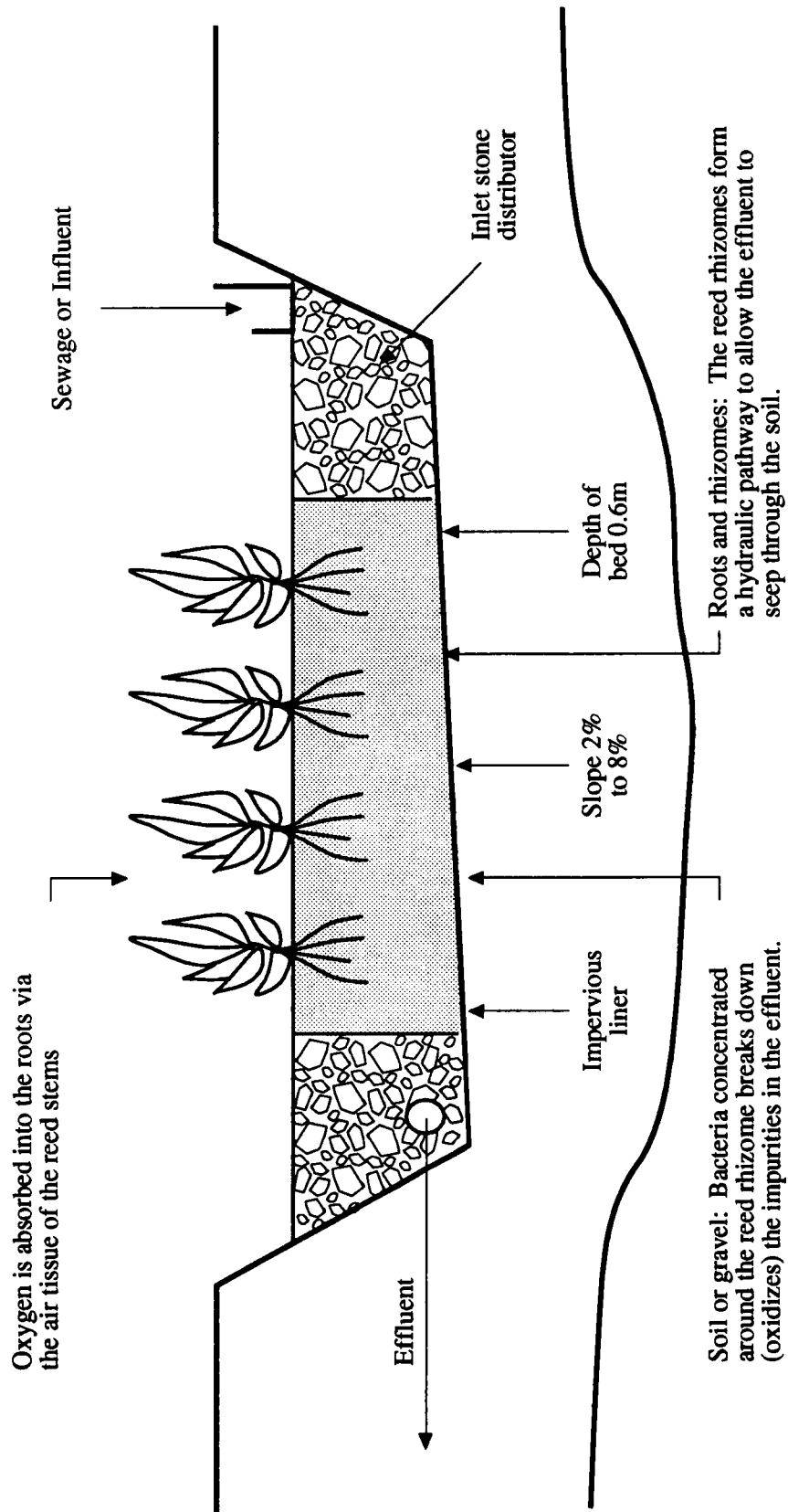


Figure 2.5
Typical Root Zone Artificial Wetland (26, 32)

leachate, and raw sewage and primary sewage sludge. The reasons for applying the root zone method for the applications mentioned above were that:

1. The root zone method is an intensive process on a relatively small area of land capable of producing a high-quality, fully nitrified effluent.
2. The root zone method is capable of producing a treated sludge with a good quality filtrate and without odor problems. (27)

Currently, six root zone method systems are in operation in the UK, with others in the process of being put into operation. Table 2.2 shows some of the removal efficiencies experienced at a plant in Germany at Othfresen. (27)

Table 2.2 Wetland Efficiencies (27)

Parameter	Inflow Average (mg/l)	Removal Efficiencies Percentage
1977		
BOD-5	488	96
COD	337	87
Total N	82	94
Total P	16	98
1983, at an overloaded state		
BOD	399	99
COD	469	92
Total N	48	75
Total P	22	99

The Tennessee Valley Authority (TVA) has been investigating the use of wetlands for wastewater treatment for several years. TVA has accepted wetland treatment of sewage as a viable alternative to conventional processes because of the encouraging results from their studies

of marsh and marsh-pond-meadow systems. A site in Hardin, Kentucky, has been identified and selected for the operation and evaluation of the root zone method in the United States. (33)

It is the root zone artificial wetland that was modified for this research, since it closely resembles a horizontal leaching field. The modification is that the SSHF bed used area vegetation -- basically grass. No extensive root system is characteristic of the root zone method. Why modify the root zone artificial wetland only to have area vegetation or grass? Because of their extensive root system, bulrushes and the common reed are types of wetland vegetation that are needed for the root zone system. However, establishing a well-vegetated bed with a well-defined root system is difficult. The literature sites several recommendations or guidelines to follow in developing the wetland vegetation. These guidelines point out the drawbacks in using this type of vegetation. The guidelines are as follows:

- For survival, the vegetation should be planted in the spring (March/April) or in the autumn (Sept./Oct.), avoiding frosts. (27, 27, 33)
- After the reeds have been planted, the bed should be flooded with water or sewage effluent. The water level should be kept 25 mm below the bed surface. (26, 27, 33)
- The water level in the bed should be dropped between August and October for the first 3 years to encourage the downward penetration of the roots and rhizomes. (26, 27, 33)
- A plant density of 3 to 11 plants per square meter is suggested for higher efficiencies. (33, 34, 35, 36)
- It takes approximately 3 years for the wetland to be fully capable of handling its ultimate loading. (35, 36)
- At normal peak flow rates, the maximum flow rate through a cross-sectional area of the bed should not exceed $10E-4$ m/s, or damage to the secondary structure will occur. (33)

The wetland plant species vary in their requirements and performance efficiencies and are limited by physical conditions such as sunshine, temperature, and water depth in the bed. Thus, developing this type of root system would take approximately 3 years before the wetland bed could be used to its potential. Enough vegetation would have to be planted and carefully cultivated so that the vegetation would not die off. The water level in the bed would have to be watched and regulated. If the wetland bed is in an area where the weather is cold in the fall and winter months, the vegetation in the bed could be lost to frosts and snow. Anything that leads to the loss of vegetation in the wetland bed also leads to the decrease in removal efficiencies of the bed. This type of vegetation (bulrushes, etc.), may be somewhat awkward and unsightly in a homeowner's yard, whereas grass cover would be more readily acceptable.

ARTIFICIAL WETLANDS MICROBIOLOGY

Wetlands remove aquatic pollutants through a complex variety of biological, physical, and chemical processes. The mechanisms include both bacterial transformations and physicochemical processing, including adsorption, precipitation, and sedimentation. The major system components affecting wastewater treatment include bacteria, soils, and plants. (19, 23, 33, 37) Bacteria and fungi serve to decompose organic matter and perform many of the steps in the nitrogen cycle.

The plant rhizome provides surfaces for bacterial growth as well as for filtration of solids. Most importantly, plants translocate oxygen from the shoots to the roots. The rhizosphere or root zone offers an oxidized microenvironment in an otherwise anaerobic environment, stimulating the decomposition of organic matter and the growth of nitrifying bacteria, the latter of which converts ammonia to nitrate. (19, 23, 33, 37) A study conducted in Santee, California, showed

that the presence and type of vegetation in the wetland can make a significant difference in the ammonia-removal efficiency. The bulrush had the highest removal efficiency, 94%. It was followed by reeds, with a removal efficiency of 78%. The cattails had the lowest removal efficiency, 28%. The unvegetated bed showed removal efficiencies of 11%. The differences in the efficiencies was due to the plants' ability to transport oxygen down to the roots, thereby establishing an oxidized rhizosphere necessary for nitrification/denitrification. The cattail may have proved to be the poorest performer, because it has the shallowest root zone, most of which is within the top 30 cm of the soil. The root zones of the bulrushes and reeds, however, extend to > 60 cm and > 76 cm, respectively. These plants also showed significant differences in their BOD removal efficiencies. The bulrush once again showed the highest removal efficiency, at 96%. The reed, cattail, and unvegetated bed had lower efficiencies of 81%, 74%, and 69% respectively. The differences here are also due to the plants' ability to translocate oxygen to the root zone since, BOD removal is enhanced under aerobic conditions. (37) Without an extensive root system to translocate oxygen down into the SSHF bed, oxygen transfer into the bed may depend on surface diffusion. The SSHF bed being tested would be expected to obtain at least the removal efficiencies cited for an unvegetated bed.

The pollutant parameters most widely considered in wastewater treatment are BOD, SS, bacteria and viruses, phosphorus, nitrogen, heavy metals, and refractory organics. BOD and SS in the wetland system are removed through two basic processes: a long retention time and low flow velocities. These allow heavier solids to physically settle and decompose. The smaller solids and soluble portion that will not settle are metabolized by the microorganisms within the wastewater and on the roots of the vegetation. (19, 23, 33)

Little is known about the ability of the wetlands to remove disease causing viruses from municipal wastewater. A study performed in Santee, California, found that the ability of higher aquatic plants such as bulrushes to translocate oxygen into the rhizome may account for some of the enhanced virus removal in the vegetated wetlands. It appears that adsorptive mechanisms play a major role in the removal of viruses. Mean removal efficiencies were 95%. It was found that the virus-removal capacity of the wetland did not decrease even after high continuous wastewater loading rates were applied for over a year. (38) These data suggest that either the capacity for virus adsorption is extremely large in the wetlands or that significant inactivation of the viruses occurs during retention in the wetlands because the viruses are away from suitable hosts.

Bacteria are lost from wetland systems by chemical reactions, natural die-off, and predation. Bacterial removal efficiencies in the California study on bulrushes was found to be 99.0 to 99.9%. Quantifying bacterial removal efficiencies is difficult when coliforms are used as an indicator organism because organics in the wetlands will contribute to the coliform count. However, the use of coliforms as an indicator organism is widely used. (23, 38)

Nitrogen enters the wetland in various forms: ammonia, organic nitrogen, nitrite, and nitrate. Some of the vegetation will assimilate approximately half of the applied nitrogen as nitrate and ammonia. Unless the vegetation is harvested, much of this removal is temporary. Harvesting can generally remove 10% of the nitrogen applied to the system. Volatilization provides another means of nitrogen removal from the system. In this process, ammonia is lost to the atmosphere at a pH greater than 8. Elevated pH values such as this may occur during the warm summer months. Nitrification/denitrification, however, constitutes the major mechanism by which elemental nitrogen is returned to the atmosphere. Ammonia is the predominant

inorganic form of nitrogen in the oxygen-deficient flooded systems, and denitrification cannot take place unless conditions exist such that nitrification of the ammonia can occur first. Dissolved oxygen penetrates down the shoots and rhizomes into the saturated soil before being consumed. The resulting development of a thin oxygenated surface layer on the plant roots and soil allows the bacteria to oxidize ammonia to nitrate. Nitrate produced from this reaction diffuses down to the underlying oxygen-free layer, where it is denitrified. It is then converted into N_2O and N_2 gases, which are lost to the atmosphere. (19, 23, 33, 39, 40) Figure 2.6 illustrates this process.

The nitrification/denitrification process may still occur in the SSHF bed, where oxygen transfer into the bed may only be accomplished through surface diffusion and not through the plant's roots. However, nitrification/denitrification generally occurs when the oxygen uptake rate for BOD removal is less than the oxygen transfer or diffusion rate into the soil. It is at this point when oxygen can become available for nitrification. It is expected that, as the BOD is removed down the length of the SSHF bed, oxygen will become available for nitrogen removal. (19, 23, 33, 39)

ARTIFICIAL WETLAND ADVANTAGES/DISADVANTAGES

Numerous advantages and disadvantages exist for using artificial wetlands for the treatment of wastewater. Conventional treatment plants are technically complex, difficult to operate, expensive, and often unaffordable, especially to small communities. On-site treatment systems that are effective, have relatively low capital and operational costs, and are easily operated and maintained are needed. (28)

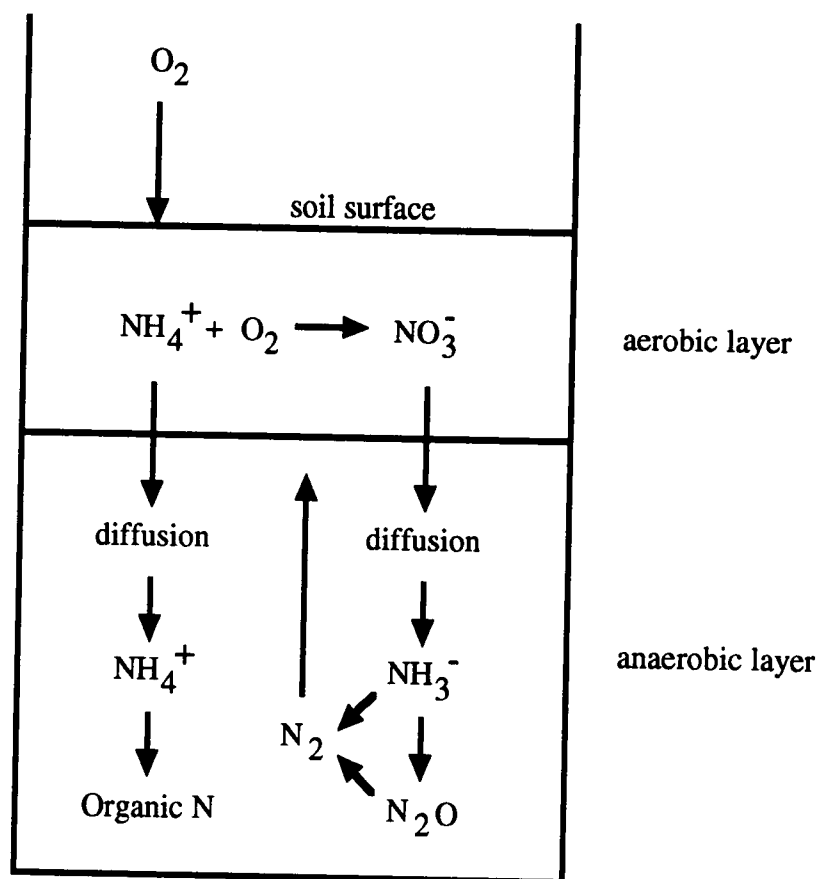


Figure 2.6
Nitrification / Denitrification Process in Soils (37)

The major advantage or benefit of the artificial wetland system is cost. Capital costs are 25 to 75% of the costs of conventional systems, and operating costs are typically 10 to 25% of conventional processes. The artificial wetland's simplicity in operation and maintenance and low operational and capital costs have accelerated interest in its application for the treatment of wastewater. One of the major cost savings is due to the fact that forced aeration, as in activated sludge systems, is not necessary with wetland treatment. The vegetation translocates the required oxygen into the underlying soil profile. Such cost and energy savings make artificial wetland systems an attractive approach for the treatment of wastewater. (23, 26, 28, 29, 32, 37)

Using wetland systems holds other advantages. The treated effluent can be reused and may be highly compatible for projects involving aquaculture. The treatment system is relatively stable, allowing the wetland to withstand shock loadings and a wide range of operating conditions. The wetlands also produce a consistent quality effluent. (23, 26, 29) The absence of overland flow with the root zone method reduces problems caused by freezing and reduces the overall land requirement. Wetlands offer an alternative land treatment system for reducing the load of disease-causing viruses to the environment. (33, 38) The use of artificial wetlands for the treatment of wastewater is environmentally acceptable (appearance, flies, and odors are not a problem) and offers considerable wildlife conservation potential. (24, 26, 29)

The one major disadvantage of the wetland system is that it is neither widely known nor accepted by engineering firms or regulatory agencies. With this disadvantage comes institutional problems relating to obtaining discharge permits. A few other disadvantages to using a wetland system are related to the lack of information on the design parameters and performance efficiencies of these systems. The amount of land required and the requirement to locate the beds

of plants may prohibit the use of wetlands in highly urban areas. The wetland plant species may vary in their requirements and performance efficiencies and may be limited by physical conditions such as sunshine, temperature, and water depth in the bed. (26, 27, 33)

As mentioned above, the treatment process efficiencies are not completely defined, making precise design criteria difficult to establish. Potential problems affecting the root zone method's efficiency are accidental burning of the vegetation; the need for a 3 year period before the bed becomes fully operational; the effects of parasites; the lack of a full assessment of plant disease and severe chemical contamination; and the possibility of a loss of oxygen supply to the rhizomes if the bed becomes totally covered with ice. (23, 26, 28, 29)

Because of these disadvantages, the SSHF bed was designed to use area vegetation. The SSHF bed was tested to see if this system was as dependable and as reliable as wetlands, with acceptable results within feasible space requirements.

TREATMENT SYSTEM

The treatment system is a modification of the standard septic tank treatment system. An ANFLOW unit was added to the septic tank in order to enhance its performance and to reduce the load on the SSHF bed. As discussed in the literature, the standard ANFLOW unit was designed with 10 ft of packing material, a hydraulic retention time of approximately 9 hours, and a BOD removal efficiency of 63 to 70%. The ANFLOW unit is commonly installed in the ground so that the natural insulation properties of the earth will keep it warm in winter and cool in summer. Since the ANFLOW unit was installed at a residence in series with a standard septic

tank and since the cost of digging is high, the unit was modified to only have 3 ft of packing material, which corresponds to a hydraulic detention time of approximately 3 hours and a BOD removal efficiency of approximately 50%. The septic tank should serve as pretreatment for the ANFLOW unit to remove large solids and grease that might clog the packing media. The septic tank and ANFLOW unit serve as pretreatment for the SSHF bed.

The SSHF bed is similar to the root zone bed. The major difference between the two is their vegetative cover. During this performance study, the SSHF bed had little or no vegetative cover. The vegetative cover present consisted of grasses and area plants that do not have extensive root systems. Oxygen transfer into the bed did not occur by the extensive root system characteristic of the root zone method but possibly by surface diffusion. It is expected that the SSHF bed should perform at least as well as the unvegetated bed cited in the literature. The average BOD removal efficiency was 69% and the average ammonia removal efficiency was 11% for the unvegetated bed. (37) It is expected that as the oxygen requirement for BOD removal decreases, it will become less than the oxygen diffusion rate into the soil, and the available free oxygen can be used for the nitrification process. The main substrate for the SSHF bed was gravel. The bed was comprised of 18 in. of coarse gravel, 6 in. of pea gravel, and 6 in. of soil cover. These modifications were done so that this type of treatment system could be constructed in different communities with area vegetation.

These systems were modified and combined into one treatment system for residential properties and small commercial properties. The system -- a septic tank, a modified ANFLOW unit, and a SSHF bed, -- was monitored to determine the adequacy and feasibility of using this combination of treatment technology to treat domestic sewage. Because the SSHF bed used in

this research is a modification of the artificial root zone wetland system, the standard design equations may not apply. The next section discusses the typical design equations used for artificial wetland systems and a typical design equation for fixed media filters that may also be applicable to the SSHF bed, since it closely resembles a horizontal trickling filter.

TYPICAL DESIGN EQUATIONS

ARTIFICIAL WETLANDS

According to the literature, first-order reaction equations can be used to describe an artificial wetland system. Equation 1 below is a first-order reaction equation. First-order reactions proceed at a rate directly proportional to the concentration of one reactant. The rate of the reaction for first-order reactions depends on the concentration of the reactant, which changes with time; thus, a plot of the natural log of the concentration with time will give a linear response. The slope of the line is the dynamic kinetic constant (K). Thus, equation 1 postulates that the organic removal efficiency (C_i/C_o) of the system is proportional to the time of contact in the system. If the wetland bed has a uniform cross-section and the flow is a uniform constant plug flow, the length of the bed can be substituted for time. This equation then postulates that the organic removal is directly proportional to bed length. Equation 2 uses the relationship between time and distance due to a uniform flow rate and postulates that with a constant flow rate and a constant bed depth, the organic removal is directly proportional to bed area (length x width). Equation 3 postulates that with a constant flow rate and bed depth, the width of the bed is directly proportional to the hydraulic conductivity of the bed and the slope of the bed.

$$1. \quad C_t = C_o e^{-KT} \quad (26, 41)$$

where:

C_t = Outlet BOD

C_o = Inlet BOD

T = Retention period in vegetated bed

K = Function of temperature and treatability @ 8 deg C, porosity of the soil equal to 42 %, $K = 0.032$

$$2. \quad A_b = 5.2 Q_d (\ln C_o - \ln C_t) \quad (26, 41)$$

where:

A_b = The area of the bed in square meters with a corresponding bed depth of 0.6 m

Q_d = The average flow rate in m³/d

5.2 = Relates to BOD removal. 9 should be used for COD and 14.5 for industrial wastes.

$$3. \quad A_c = Q_s / (K_f * dH/dS) \quad (26, 41)$$

where:

A_c = Cross-sectional area in square meters

Q_s = Average sewage flow in m³/s

K_f = Hydraulic conductivity of a fully developed root zone bed equal to 3x10E-5 m³/m²-s

dH/dS = Slope of the bed in m/m

FIXED MEDIA FILTERS

Early fixed media filters used beds of rock sprayed with presettled wastewater from fixed nozzles. Rotary distributors were introduced to allow for continued feed while alternately dosing different radial areas of the filter. Alternatives to rock media have also been developed to provide more surface area per unit volume, to improve ventilation, and to reduce clogging. In principle, the fixed media or trickling filter process is one in which the biological growth is attached to a non-moving medium. The wastewater is applied to the top of the trickling filter and percolates downward in a plug-flow mode. Soon after a filter is placed into operation, the surface of the media becomes coated with bacteria. The wastewater applied then percolates through the filter. Organic removal occurs by adsorption and assimilation by the biota of the soluble and suspended constituents. (42)

Trickling filters are typically aerobic systems where oxygen for metabolism is supplied partially from circulation of air through the filter media and partially from the dissolved oxygen in the wastewater. After initial startup of these systems, the microbial buildup creates an anaerobic interface, with some of the filter media causing the growth of facultative and anaerobic organisms along with the aerobic organisms. (42)

Numerous empirical trickling filter formulations are available. The one that best depicts the SSHF bed is the Eckenfelder formula. The SSHF bed physically resembles a horizontal trickling filter where the flow through the gravel media is horizontal. The Eckenfelder formula is based on the Schulze formula, which postulates that, since the system is in a plug-flow mode, the time of liquid contact with the biological mass is directly proportional to the filter depth and inversely proportional to the hydraulic loading rate. Schulze then combined the time of contact

with the first order reaction equation for BOD removal. This equation then postulates that the organic removal is directly proportional to the filter-media depth. In 1963, Eckenfelder modified the Schulze equation to include the effect of changes in filter depth on BOD removal per unit of depth. This was an attempt to achieve a formula applicable to all filter media types. (42) The equation is as follows:

$$4. \quad C_t/C_o = \exp [\{ kD^{(1-m)} \} / (Q/A)^n] \quad (42)$$

where:	C_t	=	effluent BOD at time t, mg/l
	C_o	=	influent BOD, mg/l
	k	=	treatability constant
	D	=	filter depth, feet
	m	=	An exponent on filter depth, indicative of biological slime distribution with filter depth; for uniform distribution, (1-m) is equal to 1.
	Q	=	hydraulic loading, gpm
	A	=	filter area, square feet
	n	=	exponent characteristic to filter media

This equation shows the relationship of BOD removal to hydraulic loading and filter media depth. It does not directly incorporate temperature and oxygen transfer into the analysis. (42)

This equation can be compared to equation 1 above for artificial wetlands, since Eckenfelder defines residence time in the filter as related to operating and filter media characteristics. (42) It is as follows:

$$5. \quad t = CD / (Q/A)^n = D^{(1-n)} / (Q/A)^n \quad (42)$$

where: t = residence time in the filter

$$C \sim 1 / D^n$$

Substituting equation 5 back into equation 4, equation 4 fundamentally becomes:

$$6. \quad C_t = C_o e^{-kt}$$

This is the same as equation 1. As stated previously, time can be interchanged with bed length using a uniform cross-section and a uniform plug flow.

CHAPTER III

METHODOLOGY

Rural areas that are not always suitable for septic tank use have posed a persistent problem for the development of good wastewater treatment systems. The two demonstration systems studied may solve to this problem. A passive bioreactor was combined with a standard septic tank and a SSHF bed was used instead of the standard drain field on an experimental basis at the Shagbark development near Pigeon Forge, Tennessee, and on a demonstration basis at the Haynes House in Pigeon Forge, Tennessee.

The ANFLOW components used for the two research projects were monitored for their performance characteristics. The effluent from the ANFLOW unit requires a polishing step (because it is an anaerobic process) to meet secondary effluent limits. A SSHF bed was added to the treatment system to provide the required polishing step for the septic tank/ANFLOW unit. The SSHF bed is an equally passive treatment process, as presented in Chapter II. The SSHF beds used for the two research systems should function similar to an artificial wetland or trickling filter. It is desirable for the SSHF bed to remove the BOD at the front end and nitrify ammonia at the back end. Both research systems were designed to achieve discharge quality water before the effluent was allowed to be released to the environment.

This chapter will discuss the design and setup of each system, sampling procedures, field and laboratory tests, and performance data analysis procedures used to determine the performance characteristics of the treatment systems.

DESIGN PARAMETERS AND SETUP: SHAGBARK SYSTEM

The Shagbark treatment system was designed to handle a combination of single and multi-unit buildings with a total population of 240 persons. The design hydraulic loading was 60 gpd per person and the design organic loading rate was 0.23 lbs BOD per person per day. These values are typical for domestic sewage.

In designing the SSHF bed, a conservative 50% efficiency was used for the ANFLOW component. Specific design parameters were also used in designing the SSHF bed, as follows and as presented in the Literature Review for the root zone method. The design hydraulic loading rate was approximately 15 acres/MGD. The expected flow rate to the system was 5,400 gpd. The inlet distribution pipe was designed to provide uniform distribution of flow at the inlet. It is located on the bottom of the SSHF bed above the impermeable liner and spans the entire width. A dosing siphon was used to flood the distribution pipe to help ensure uniform flow distribution at the inlet. The outlet collector pipe is a perforated pipe located at the bottom of the pea gravel layer of the SSHF bed. The SSHF bed depth is approximately 2.5 ft with a bed slope of 0.15%. The SSHF bed is made up of 18 in. of 3/4 to 1 in. gravel as a base layer, followed by 6 in. of pea gravel and 6 in. of soil cover as a top layer. The bed liner was constructed of synthetic (plastic swimming pool liner) material, with a permeability of $1.0\text{E}-8$ m/s or less. Oxygen transfer into the soil during this research was probably due to surface diffusion instead of translocation by the plant roots as described by the root zone method, since the bed had no vegetative cover.

As shown in Figure 3.1, the Shagbark treatment system flow scheme consisted of a septic tank at each housing unit, a 3,800-gal equalization tank and a 3,800-gal ANFLOW unit in series.

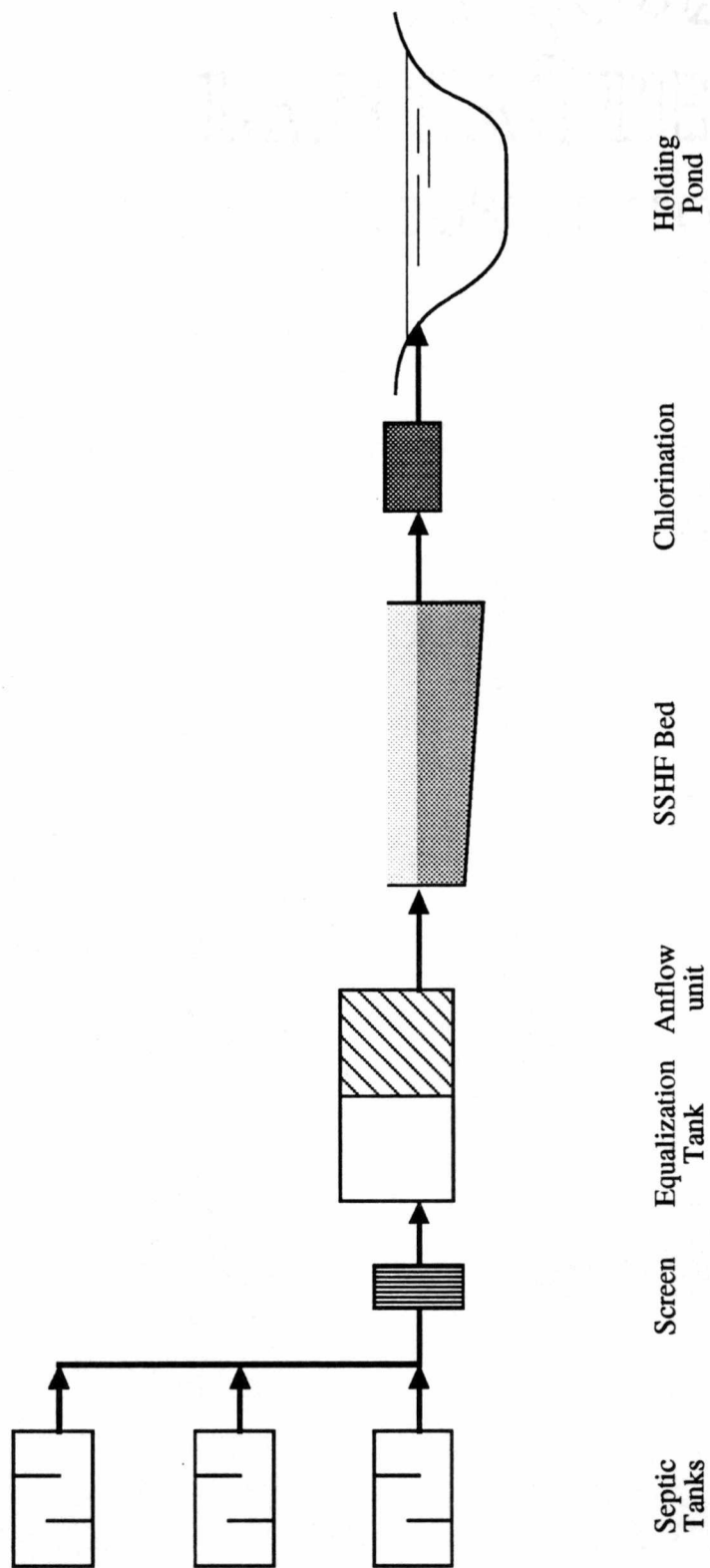


Figure 3.1
Shagbark System

The ANFLOW unit and equalization basin were each 7.5 ft x 7.5 ft x 8 ft deep. The operating volume of the ANFLOW unit was approximately 2300 gal. The ANFLOW unit was to be followed by 0.4 acres of SSHF beds to handle the full wastewater load. Upon installation of the wastewater treatment system, only two buildings were constructed, with the other housing units to be constructed in the near future. These two buildings accounted for only a small fraction of the design wastewater flow and loading rate to the treatment system. Because of this initial loading, only one SSHF bed was constructed, as shown in Figure 3.2, with additional beds to be constructed as required by increased loading rates as the other buildings are constructed and occupied. Thus, the completed wastewater system consisted of a full-size equalization basin followed by a full-size ANFLOW unit and a single SSHF bed 50 ft long and 4 ft wide. This resulted in a non-uniform hydraulic loading to the system. The SSHF bed was followed by a small chlorination system, consisting of a small synthetic container with two columns containing chlorine tablets. The chlorination system provided disinfection of the effluent by allowing the flow to come into contact with the chlorine tablets held in the columns. Following the chlorine contact system was a 0.3-acre holding pond.

To act as a solids and grease trap, the septic tanks were placed at each unit or cluster of units. The equalization tank was designed and constructed to allow for a more uniform loading to the treatment system, eliminating diurnal fluctuations. The ANFLOW unit should provide biological treatment and solids digestion, and the SSHF bed should provide polishing of the anaerobic effluent by bio-oxidation. The chlorination unit should provide any chlorination required for the disinfection of the effluent. The pond was incorporated in the design for minot polishing of the effluent and for holding of the effluent for irrigation.

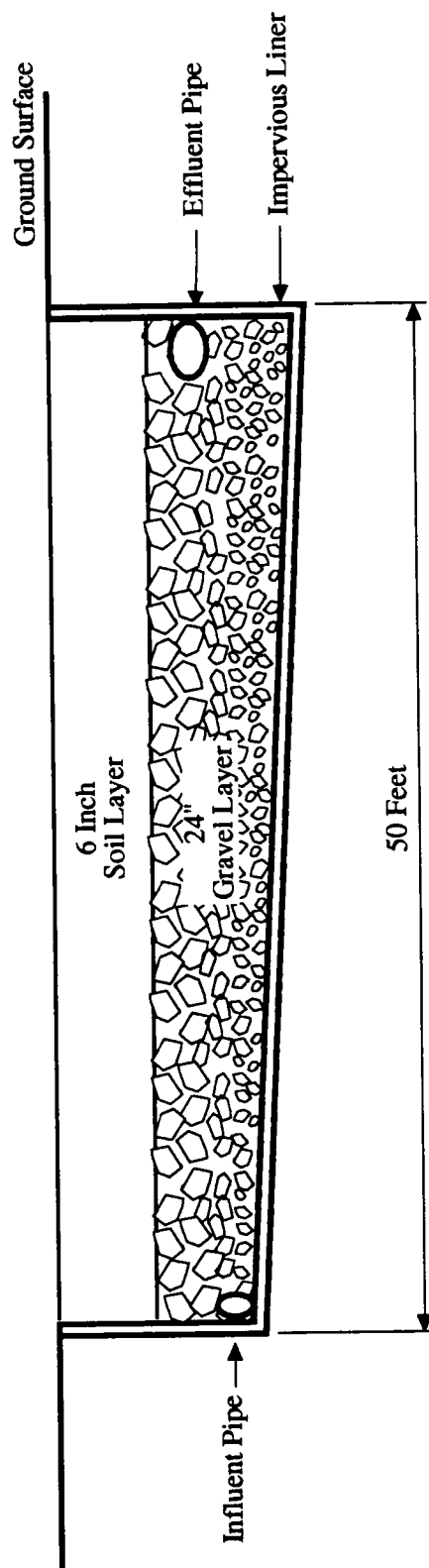


Figure 3.2
Shagbark SSHF Bed

The above arrangement of a full-sized equalization tank, a full-sized ANFLOW unit, and a single SSHF bed produced a problem for the monitoring program. The small flow into the equalization basin resulted in a single daily pumping cycle of 30 to 45 minutes. This cycle produced a single slug loading to the treatment system instead of a continuous uniform loading rate as designed. This problem was identified and an attempt was made to correct it. The influent line to the ANFLOW unit from the equalization tank was modified, to include recycle back to the equalization tank, in order to lengthen the pump cycle. Two ball valves were installed on the line with a tee and additional piping, as shown in Figure 3.3. Even with this, the flow rate to the treatment system could not be regulated because of the fluctuation in the static head on the pump, which fluctuated the pump's discharge flow rate.

The system was constructed early in 1987, and sampling began at the end of May in 1987. Sampling continued through March of 1988.

DESIGN PARAMETERS AND SETUP: HAYNES HOUSE SYSTEM

At the Haynes house, a septic tank and leaching field were used for wastewater treatment. This system failed; the drain field plugged, leading to standing water in the yard. The owner of the home needed a new treatment system and invested in an ANFLOW/SSHF bed system.

The treatment system was designed based on a single-family home with 4 occupants. The treatment system consisted of a septic tank with an ANFLOW component, a SSHF bed, and a new drain field. The wastewater treatment system flow scheme is shown in Figure 3.4. The system was designed to handle a BOD loading of 200 mg/l and a hydraulic loading of 60 gpd per person. Based on these parameters, an ANFLOW unit was installed within a septic tank with a

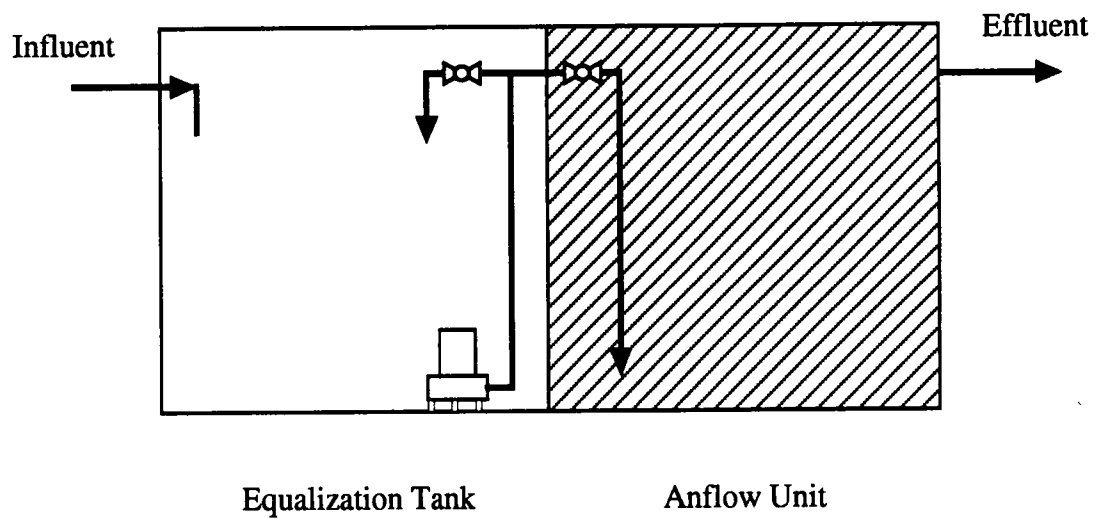


Figure 3.3
Modified Piping for Recycle

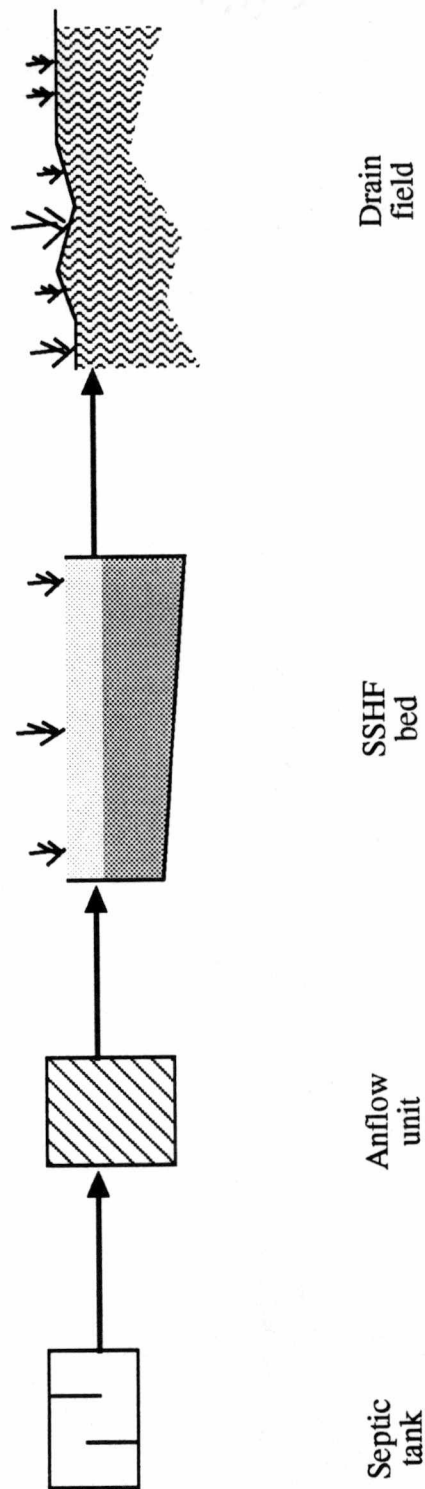


Figure 3.4
Haynes House System

total volume of 1000 gal. The septic tank volume was approximately 320 gal, and the ANFLOW volume was approximately 720 gal. As shown in Figure 3.5, the SSHF bed was designed with the same parameters as the Shagbark system discussed previously in the chapter. The SSHF bed was as described in Chapter II; influent concentrations based on an ANFLOW efficiency of 50%, an inlet perforated pipe located at the bottom of the bed and above the impermeable liner spanning the entire width of the bed, an outlet perforated pipe located at the bottom of the pea gravel layer of the bed, a bed slope of 0.15%, and bed vegetation that was mostly grass.

Oxygen transfer in the SSHF bed was most likely accomplished by surface diffusion and not by the plants roots as described by the root zone method in Chapter II because the bed vegetation was basically a grass cover. The SSHF bed was 4 ft wide, 60 ft long, and 30 in. deep. The SSHF bed was made up of an impermeable liner followed by 18 in. of coarse gravel covered with 6 in. of pea gravel, and 6 in. of top soil. The SSHF bed was followed by the existing leaching field for final disposal of the effluent.

The septic tank should function as a grease and solids trap for the ANFLOW unit. The ANFLOW unit should provide BOD and solids reduction, and the SSHF bed should provide the polishing step for the anaerobic effluent from the ANFLOW unit. The leaching field followed the SSHF bed to provide further polishing of the effluent if needed.

The system was constructed in April of 1988, and sampling began in May of 1988 and continued through September of 1988. In late May of 1988 and early June of 1988, the yard at the Haynes house was regraded to level it. With the regrading of the yard, an additional 6 in. of soil cover was added to the SSHF bed depth. So instead of the recommended 6 in. soil cover

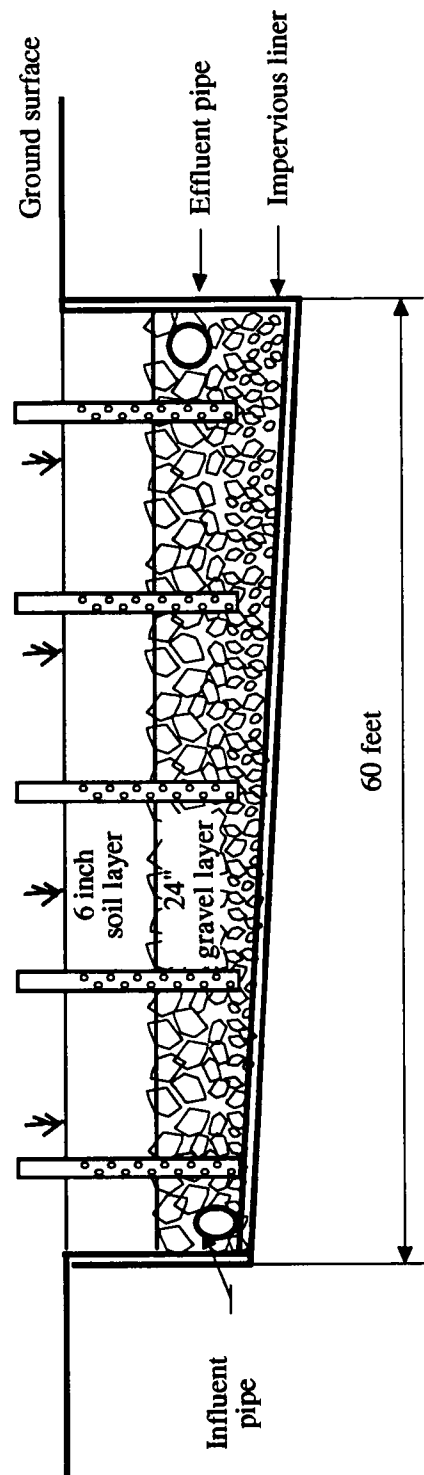


Figure 3.5
Haynes House SSHF Bed

for the SSHF bed, 12 in. of soil cover resulted. For analysis of the data from the Haynes home, the data were split into two time periods, based on soil cover depth.

SAMPLING PROCEDURES

Sampling at Shagbark was performed weekly. The samples were taken as grab samples. They were collected at three locations in the treatment system: at the equalization tank, at a sampling box that was effluent to the ANFLOW unit and influent to the SSHF bed, and at a sampling box which immediately followed the chlorination system. The samples were labeled Infl, I/E, and Effl, respectively.

Weekly grab samples were collected at the Haynes home at six locations in the treatment system. The first sample was the influent to the ANFLOW unit. The second was taken at a sample port influent to the SSHF bed and effluent to the ANFLOW unit. The next three samples were collected from sample ports within the SSHF bed at distances of 15, 30, and 45 ft from the influent end of the SSHF bed. The last sample was taken at the end of the SSHF bed before the drain field. These samples were labeled INFL, I/E, 15, 30, 45, and 60/out, respectively.

Samples from both locations, Shagbark and the Haynes home, were collected in 1-liter and 250-ml bottles. The samples from the two treatment systems were iced down in an ice chest and transported back to the Environmental Engineering lab at the University of Tennessee. The 1-liter bottles were analyzed upon arrival at the lab for BOD, COD, and TSS, and the 250-ml samples were frozen for later analysis at the Agricultural Engineering lab at the University of Tennessee for ammonia and fecal coliforms.

FIELD AND LABORATORY TESTS

The same field and laboratory tests were conducted on the samples collected from each treatment system. During each sampling visit, any observable changes in the treatment systems were recorded. Changes included snow coverage, changes in the SSHF bed vegetation, any unusual growth or build up in the sampling ports, and the presence of flies. Immediately after the samples were collected, they were analyzed in the field for temperature and pH. Temperature was measured in degrees Celcius and pH was measured using a pH probe that was field- and laboratory-calibrated. Upon completion of these measurements, the samples were iced down for transportation back to the labs. The samples were placed in ice to retard any further biological activity.

At the Environmental Engineering lab, the samples were immediately prepared for testing. The samples were analyzed for BOD, COD, TSS, and residual chlorine. Only the Shagbark samples were tested for chlorine. The samples were analyzed at the Agricultural Engineering lab for ammonia, nitrates, nitrites, and fecal coliforms.

The 5-day BOD test is used to reflect the oxygen demand that the effluent wastewater would place on the receiving waters. Measuring the biodegradable organics indicates the efficiency of the treatment system in stabilizing organic matter by measuring the biodegradable organics. Standard Method 507, the Winkler Method, was used for the 5-day BOD determination. (43, 44) The BOD was reported in mg/l. Because the BOD test is based on the use of available free oxygen, and since the effluent from the ANFLOW unit is anaerobic, the effluent samples from the ANFLOW unit had to be seeded with wastewater collected from the Knoxville Wastewater Treatment Plant. At Shagbark, the effluent from the SSHF bed was

chlorinated. Chlorine in the sample will kill or inhibit the bacteria in the sample. Because of this chlorine, the samples were tested for residual chlorine prior to the BOD test. Chlorine testing is discussed below in the next paragraph. If no residual chlorine was detected, the sample was seeded to make sure there was a viable bacterial population for the BOD test and if residual chlorine was found, the sample was dechlorinated following Standard Method 507 and then seeded for the BOD test. (43, 44) The samples were seeded with 1 ml of primary effluent from the Knoxville Wastewater Treatment Plant. Most of the Shagbark effluent samples had no detectible chlorine.

The total chlorine and free chlorine were measured using a portable Hach Field Kit. Free chlorine was subtracted from the total chlorine to give a value for the residual chlorine. The chlorine tests were a DPD colorimetric test. (43, 44) Chlorine was reported in mg/l.

The COD test measures the oxygen required to oxidize organics. The closed-reflux titrimetric method from Standard Method 508B was followed. (43, 44) The low-range (0 to 150 mg/l) COD vials and the heating block were obtained from Hach Chemical Company. The test results were found to be more reproducible when the samples were diluted and low-range COD vials were used. Hach's procedure was identical to the above-mentioned Standard Method and was followed for this testing.

Total suspended solids refer to matter suspended in the wastewater. This material may adversely affect the quality of the wastewater. Solids analyses are important for controlling the biological treatment process and for assessing compliance with effluent limitations. Standard

Method 209C, total suspended solids dried at 103 to 105 degrees C, was used for analysis. (43, 44)

The ammonia test was first conducted using Standard Method 418B, the nitrate electrode screening method. (43, 44) Analysis by this method proved to be inaccurate and the results non-reproducible. The samples were then sent to the Agricultural Engineering lab, where the analysis was performed using Standard Method 418C, cadmium reduction method. (43, 44) The importance of the presence and amounts of ammonia, nitrates, and nitrites in the system is to determine whether or not the SSHF bed is nitrifying the ammonia present in the wastewater being treated.

Standard Method 909C, fecal coliform membrane filter procedure, was used for fecal coliform analysis. (43, 44) In wastewater, fecal coliforms are used as indicator organisms to determine the quality of the wastewater and the degree of contamination. The membrane filter technique involves direct plating for the detection and estimation of coliform densities.

PERFORMANCE DATA ANALYSIS

For performance purposes, the data were analyzed for observable trends and compared to the literature. Data was collected at Shagbark for 1 year and at the Haynes home for approximately 5 months. The data are presented in tabular form in Appendix A. Due to the regrading of the yard, the data collected for the Haynes treatment system were divided into two parts for analysis. The first data section is typical of a 6-in. soil cover and the second section is typical of a 12-in. soil cover for the SSHF bed.

The Haynes House data were collected as profile data for the SSHF bed. The data were collected such that analysis could be conducted by comparing the BOD and COD concentrations throughout the length of the SSHF bed. These data could then be compared to the typical design equations discussed in the literature.

How the treatment system and its individual component's performance compares to the literature is discussed in Chapter V. This chapter will also discuss how the typical design equations for similar treatment technologies might be used to describe this treatment system.

CHAPTER IV

RESULTS

The data from the Shagbark and the Haynes home are tabulated and graphed in this chapter so that general trends can be noted and explored in the next chapter.

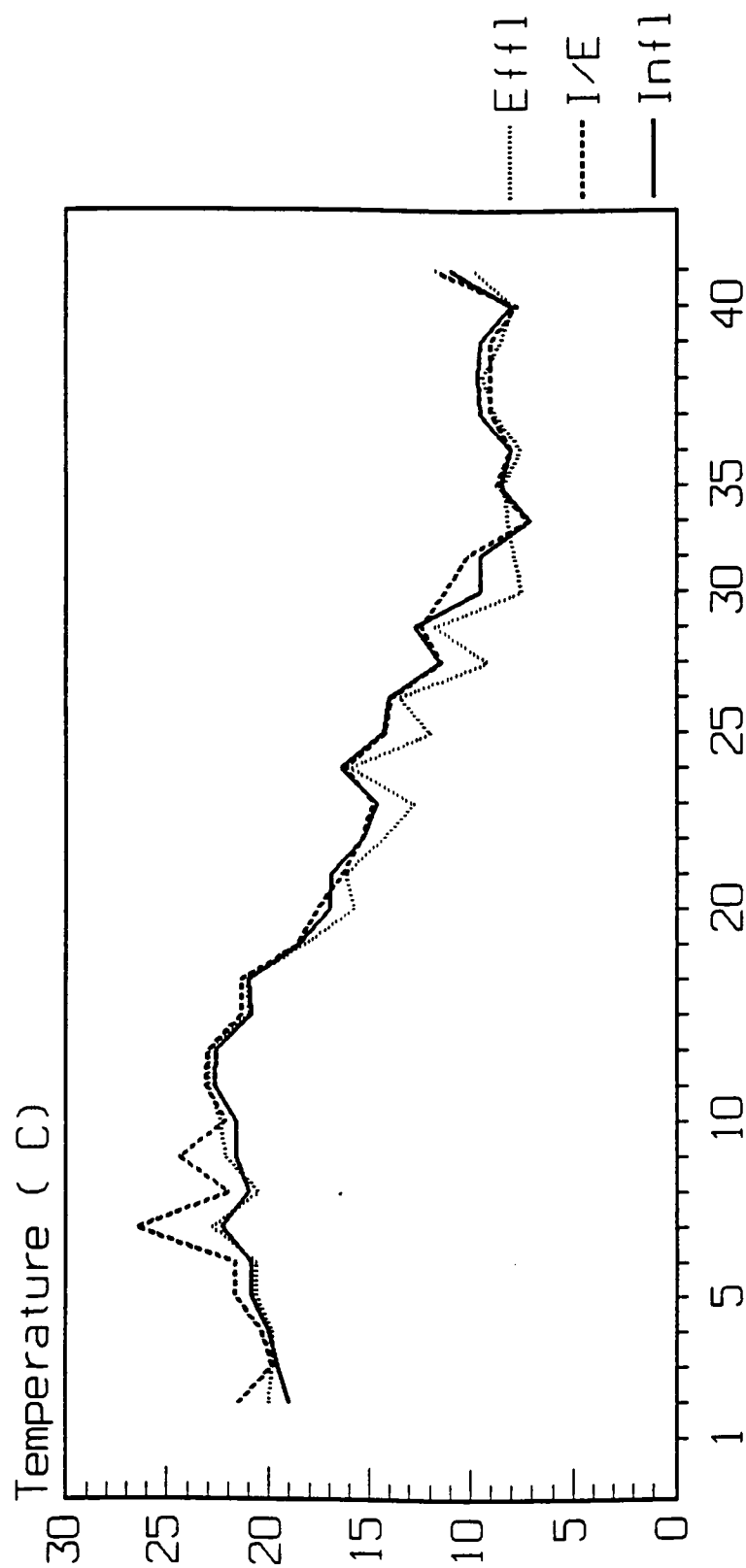
SHAGBARK SYSTEM RESULTS

Data were collected at Shagbark from May 28, 1987, to March 3, 1988. The raw field and lab data for the Shagbark system are presented in tabular form in Appendix A.

As shown in Figure 4.1, the temperature data show the normal expected seasonal variations with the temperatures ranging from 7.1 degrees C in the winter months to a high of 24.4 degrees C in the summer months. The temperature difference across the three samples was generally less than 1 degree C for any one set of samples.

The pH of the three samples remained virtually constant throughout the testing period. With a few exceptions, the pH stayed between 6 and 8 as expected. The pH data taken on Nov. 12, 1987 and Nov. 19, 1987 were very low; however, the other data collected on these two days were normal. The bad data have been attributed to a faulty pH meter. The pH electrode was replaced. The pH data are shown in Figure 4.2.

May 28, 1987 to March 3, 1988



Week
Figure 4.1
Shagbark Temperature Data
ANFLOW and SSHF Bed

May 28, 1987 to March 3, 1988

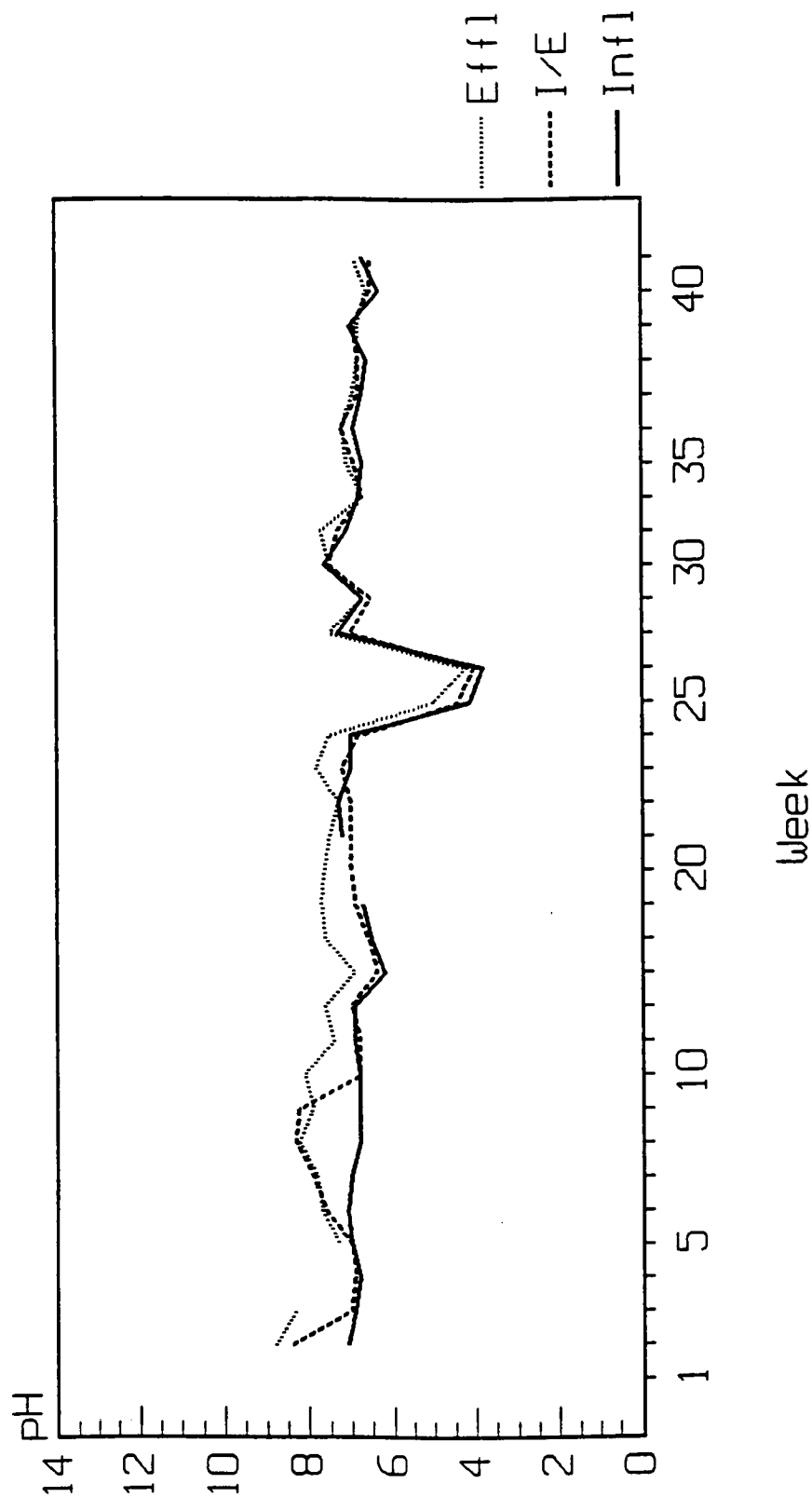


Figure 4.2
Shagbark pH Data
ANFLOW and SSHF Bed

The BOD and COD data are presented in graphical form in Figures 4.3 and 4.4, respectively. The BOD- and COD-removal efficiency data are presented in Figures 4.5 and 4.6, respectively. The average influent to the ANFLOW unit was 143 mg/l and 279 mg/l for BOD and COD, respectively. The effluent from the ANFLOW unit that was influent to the SSHF bed had an average BOD and COD of 81 mg/l and 180 mg/l, respectively. The effluent from the SSHF bed had an average BOD and COD of 31 mg/l and 91 mg/l, respectively. The ANFLOW efficiencies were 43% for BOD removal and 35% for COD removal. The SSHF bed removal efficiencies were 62% for BOD and 49% for COD. These removal efficiencies for the individual systems resulted in overall removal efficiencies for the system of 78% and 67% for BOD and COD, respectively. The data are presented in Tables 4.1 and 4.2.

**Table 4.1 Shagbark Data
Mean/Standard Deviation Values**

Parameter	Anflow Infl.	Anflow Effl.	SSHF Bed Effl.
BOD (mg/l)	143/49	81/32	31/16
COD (mg/l)	279/50	180/39	91/26
TSS (mg/l)	44/22	21/19	10/7
pH	Between 6.0 and 8.0		
Temperature	Seasonal between 7.1 C and 24.4 C		
Fecal Coliforms	reduction of 2 to 3 orders of magnitude		

May 28, 1987 to March 3, 1988

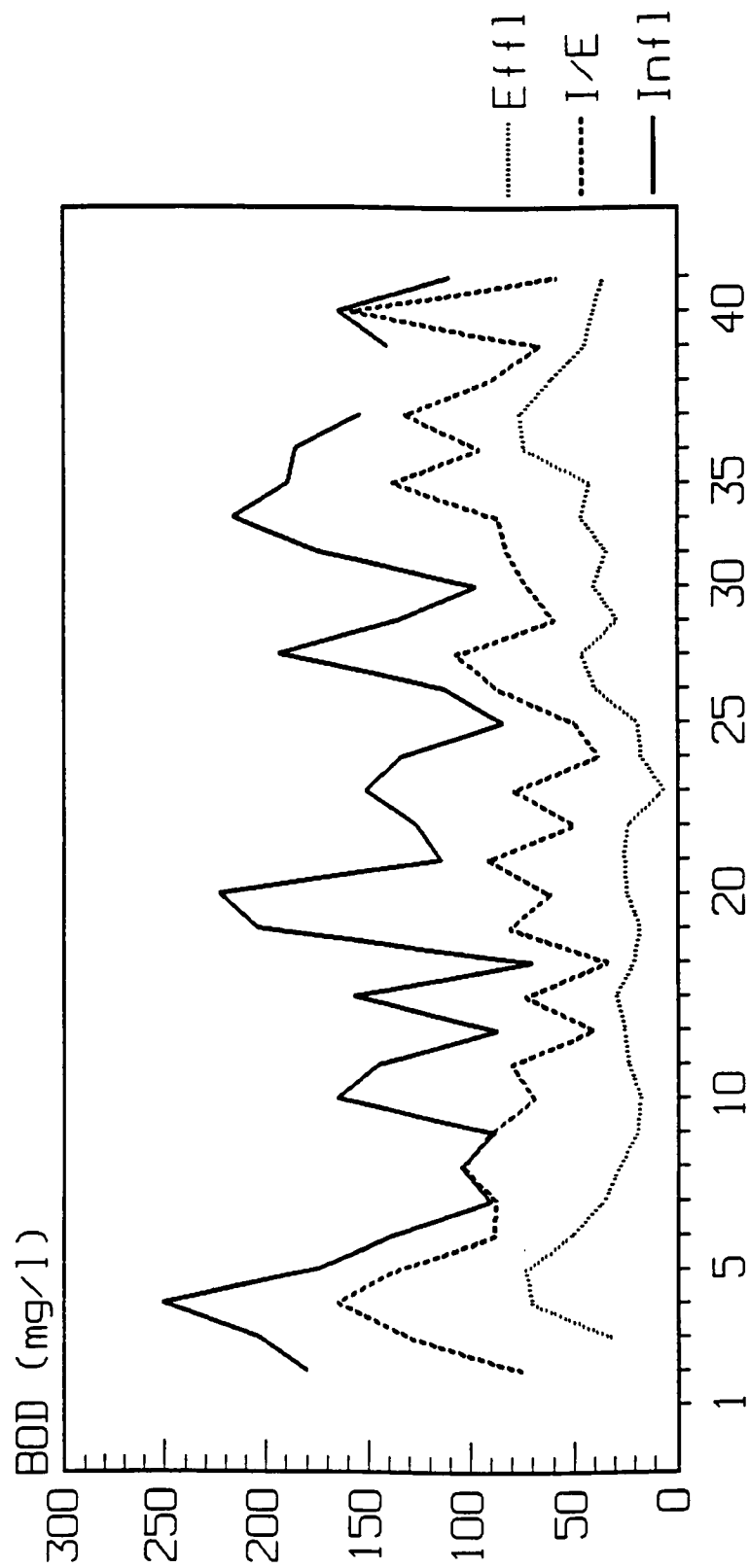
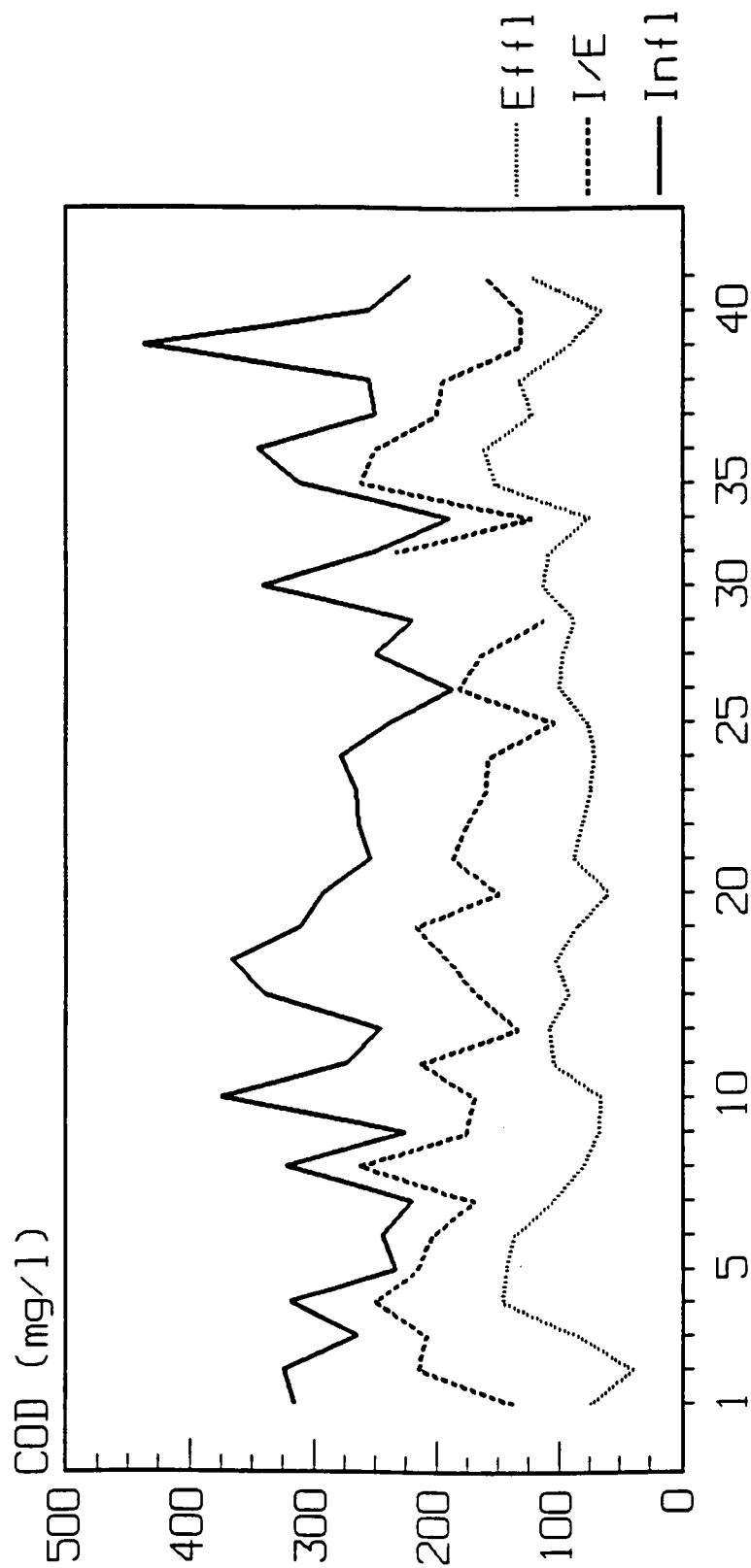


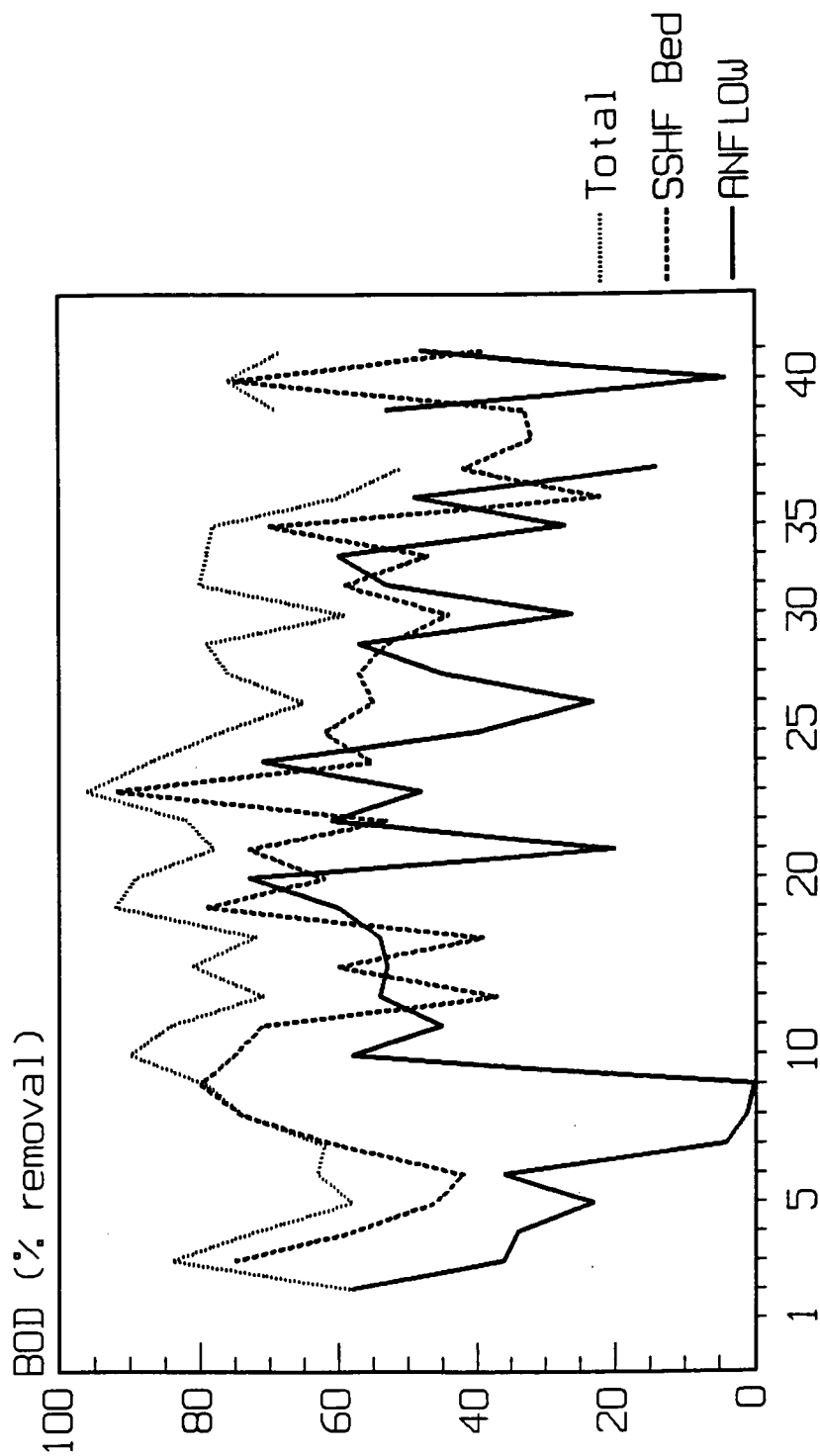
Figure 4.3
Shagbark BOD Data
ANFLOW and SSHF Bed

May 28, 1987 to March 3, 1988



Week
Figure 4.4
Shagbark COD Data
ANFLOW and SSHF Bed

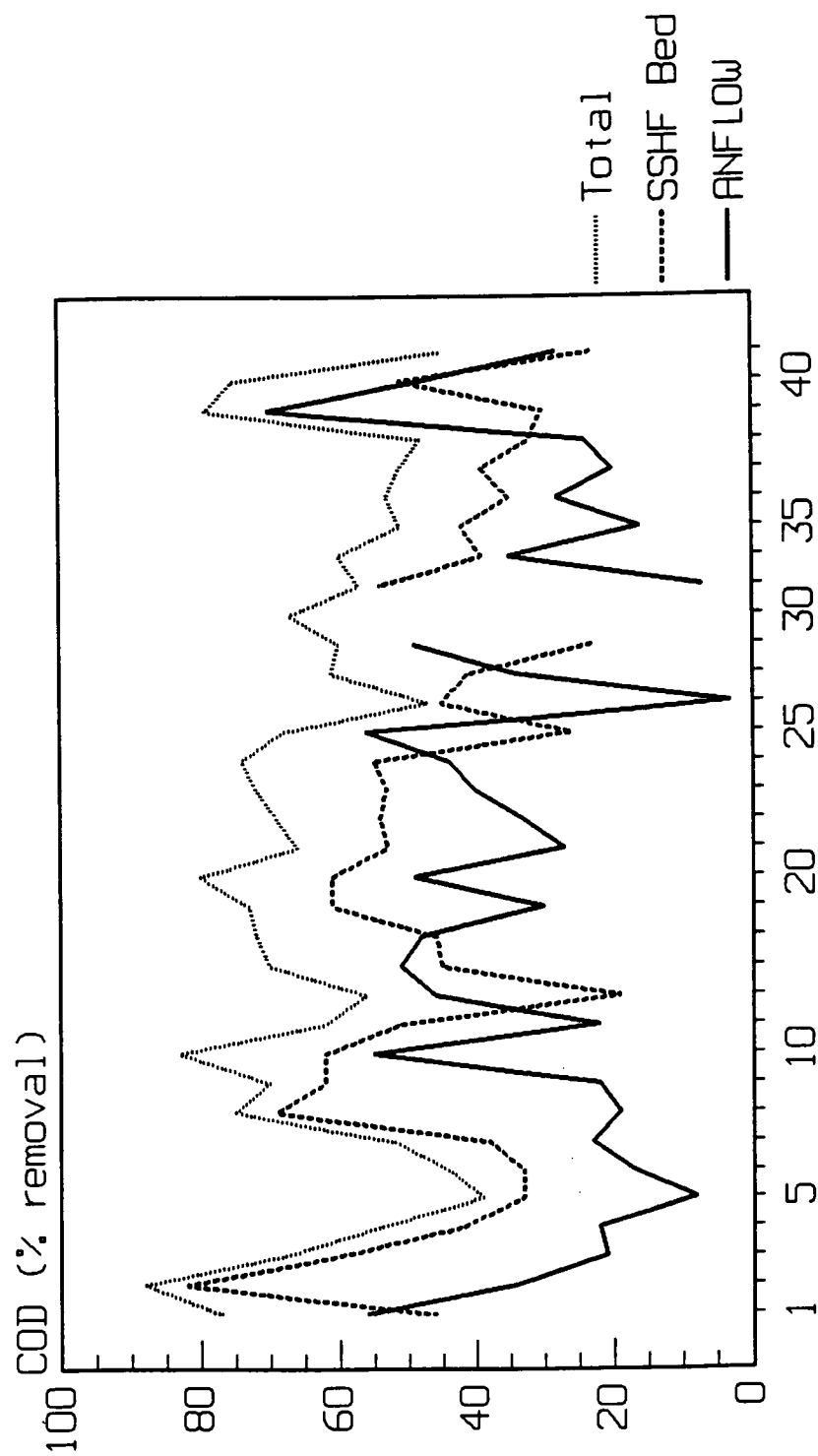
May 28, 1987 to March 3, 1988



Week

Figure 4.5
Shagbark BOD Removal Data
ANFLOW and SSHF Bed Efficiencies

May 28, 1987 to March 3, 1988



Week

Figure 4.6
Shagbark COD Removal Data
ANFLOW and SSHF Bed Efficiencies

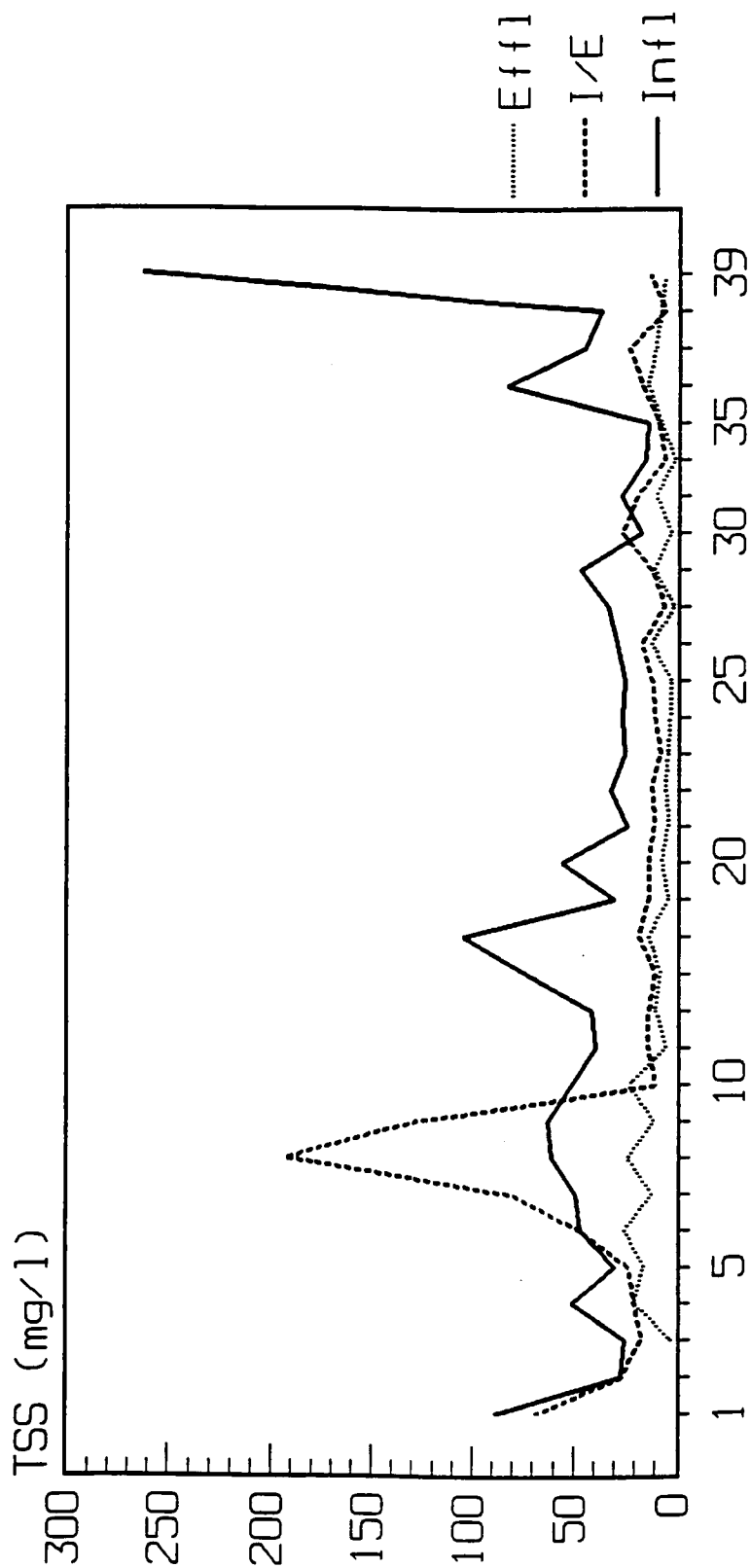
**Table 4.2 Shagbark Data
Average Removal Efficiency Percentage**

Parameter	ANFLOW	SSHF Bed	Total
BOD	43	62	78
COD	35	49	67
TSS	52	52	77

The total suspended solids (TSS) data are presented in Figures 4.7 and 4.8. The raw data presented in graphical form in Figure 4.7 show that the TSS remained relatively constant throughout the testing period. Figure 4.8 shows the removal efficiencies. The mean influent TSS to the system was 44 mg/l, the mean influent TSS to the SSHF bed was 21 mg/l, and the effluent TSS was 10 mg/l. The ANFLOW TSS removal efficiency and the SSHF bed removal efficiency were both 52%. The TSS removal efficiency for the total system was 77%. Tables 4.1 and 4.2 show this data.

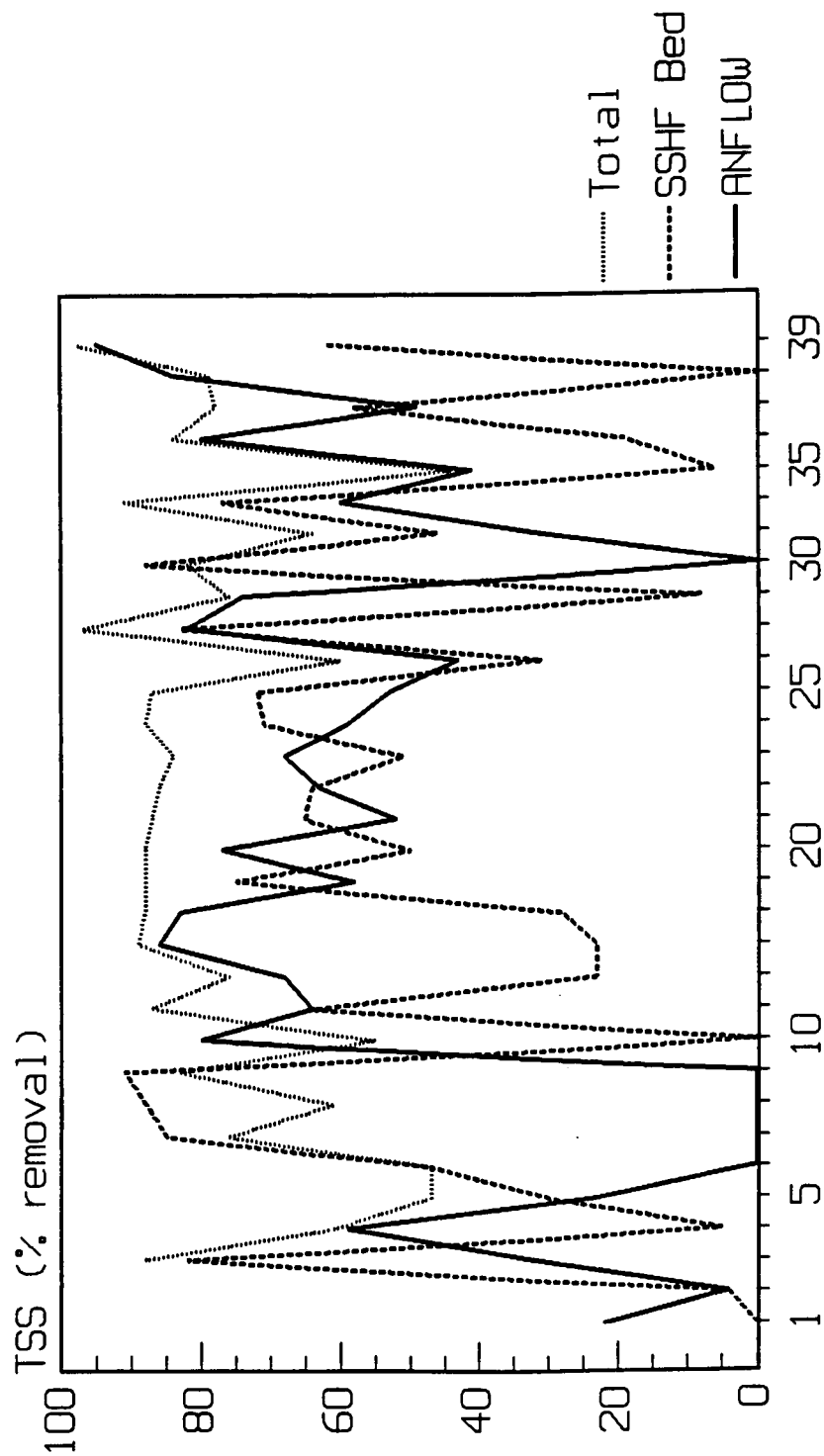
The ammonia data are presented in Figures 4.9 and 4.10. Figure 4.9 shows the raw data and Figure 4.10 shows the removal efficiencies for both the ANFLOW unit and the SSHF bed. The average influent ammonia concentration to the system was 24 mg/l. The average influent concentration to the SSHF bed was 23 mg/l and the average effluent concentration was 20 mg/l. No noticeable trends appear in the data.

May 28, 1987 to March 3, 1988



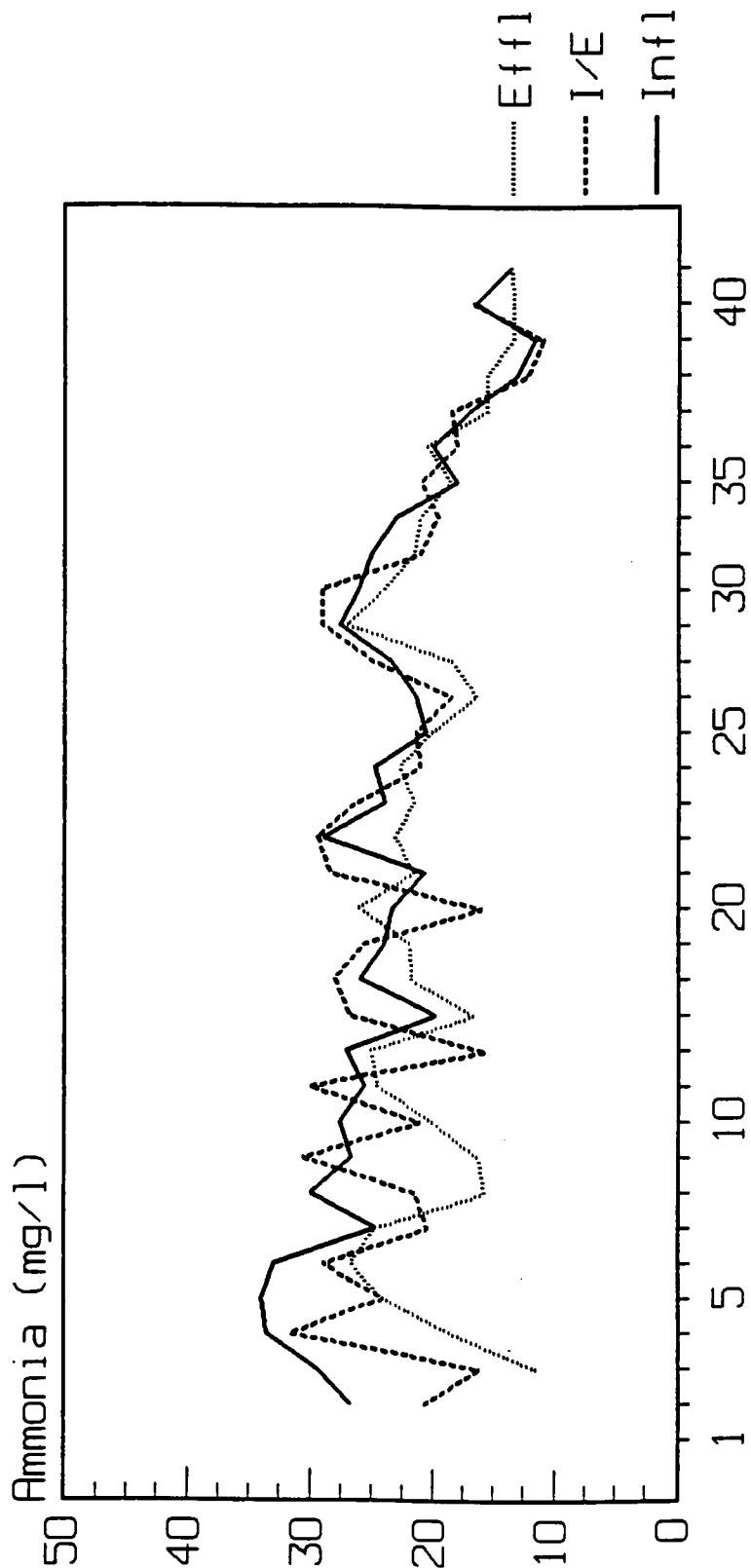
Week
Figure 4.7
Shagbark TSS Data
ANFLOW and SSHF Bed

May 28, 1987 to March 3, 1988



Week
Figure 4.8
 Shagbark TSS Removal Data
 ANFLOW and SSHF Bed Efficiencies

May 28, 1987 to March 3, 1988



Week
Figure 4.9
Shagbark Ammonia Data
ANFLOW and SSHF Bed

May 28, 1987 to March 3, 1988

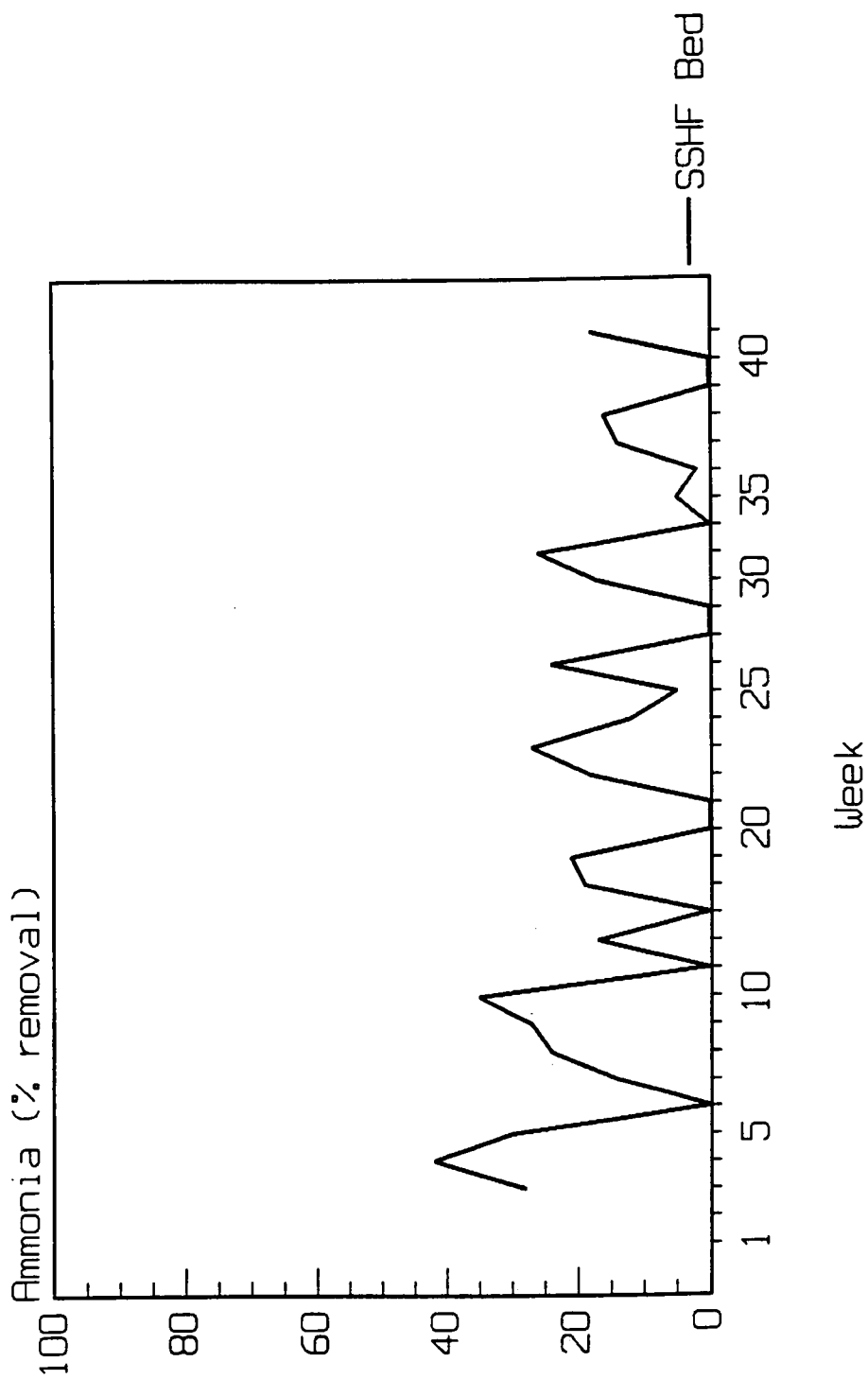


Figure 4.10
Shagbark Ammonia Removal Data
SSH F Bed Efficiency

The fecal coliform data are only presented in Appendix A and are summarized in Table 4.1. Removal efficiencies or fecal coliform reduction throughout the system was within 2 to 3 orders of magnitude.

HAYNES HOME RESULTS:

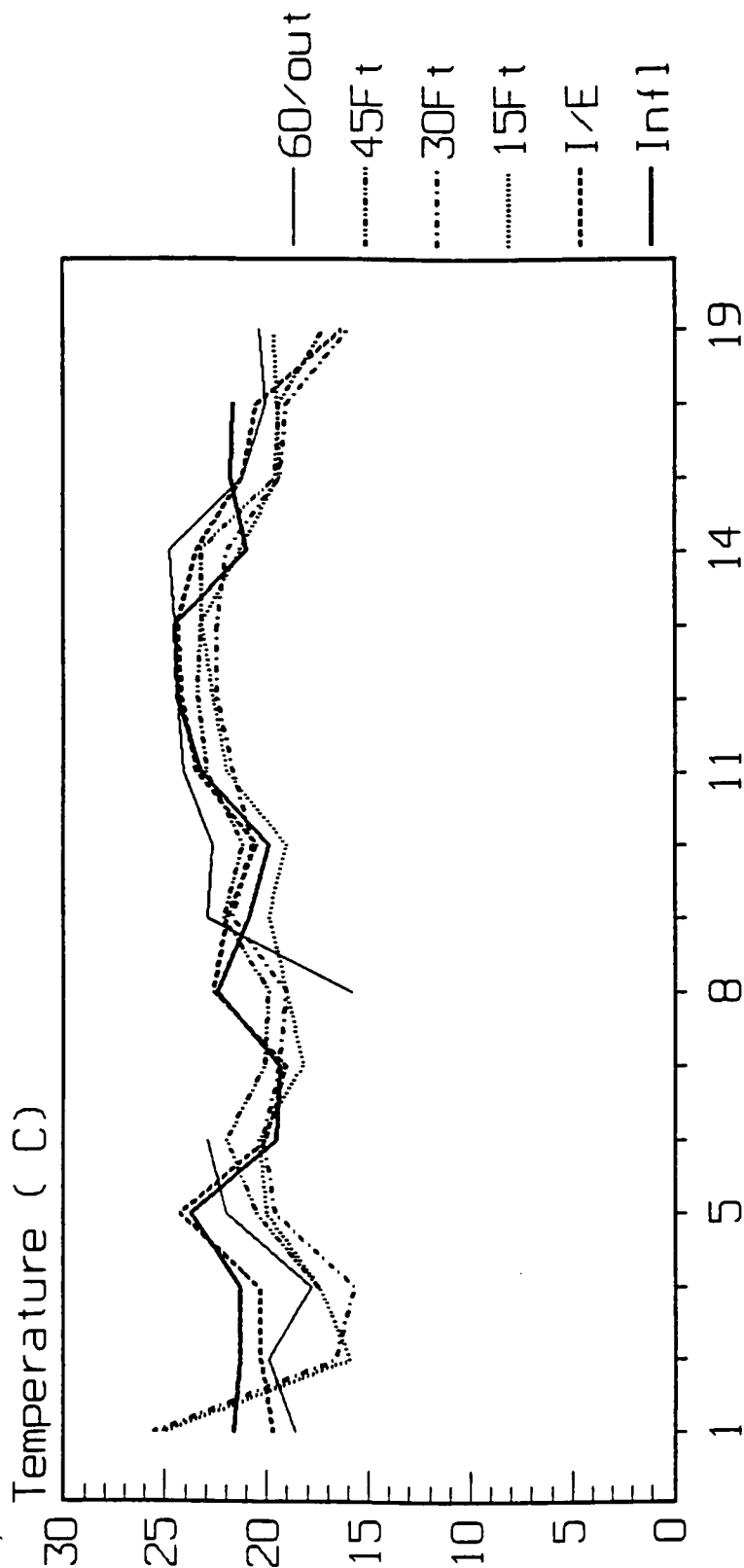
Data were collected at the Haynes demonstration project from May 10, 1988, to September 30, 1988. The data were divided into three sections for evaluation of the SSHF bed: 6 in. soil cover, transition, and 12 in. soil cover. When the system was constructed in April, the SSHF bed had a 6 in. soil cover. The yard was regraded in June to level it, and with this an additional 6 in. of soil cover was added to the SSHF bed.

The three weeks between the 6- and 12-in. cover periods provided the transition period allowing the SSHF bed to get accustomed to the 12 in. soil cover. The data for the SSHF bed were analyzed only during the 6- and 12-in. periods and not during the transition period. The raw data are presented in tabular form in Appendix A.

The temperature data for the ANFLOW unit and the SSHF bed showed the normal seasonal variations as expected and ranged from 15.7 degrees C in the spring to 27 degrees C in the summer. The temperature across the six samples was 1 to 2 degrees C for any one sampling date. The data are shown in Figure 4.11.

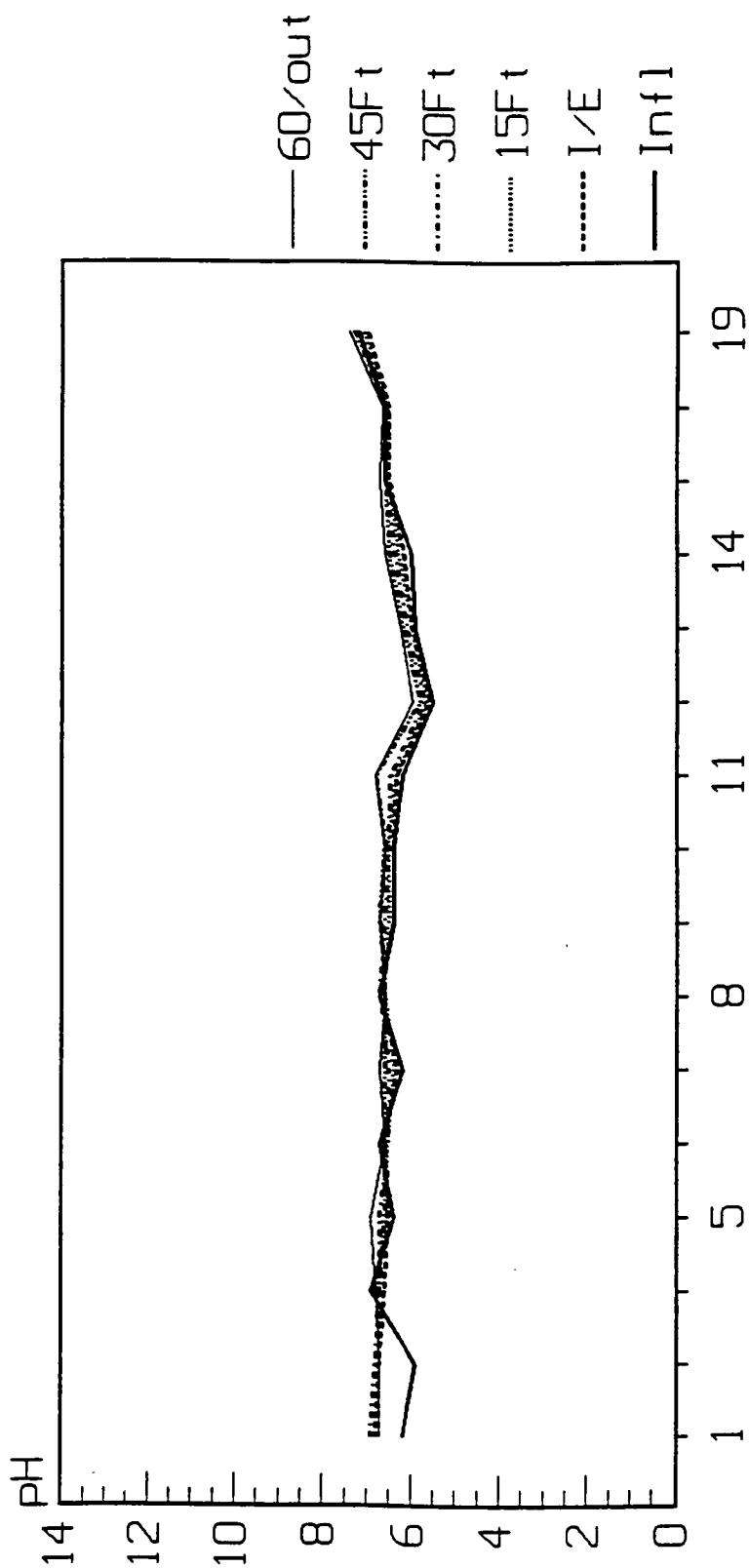
The pH for the ANFLOW unit and the SSHF bed ranged from 5.4 to 7.4. It remained virtually constant for the six samples throughout the sampling period. The data is presented in Figure 4.12.

May 10, 1988 to Sept. 30, 1988



Week
Figure 4.11
Haynes House Temperature Data
ANFLOW and SSHF Bed

May 10, 1988 to Sept. 30, 1988

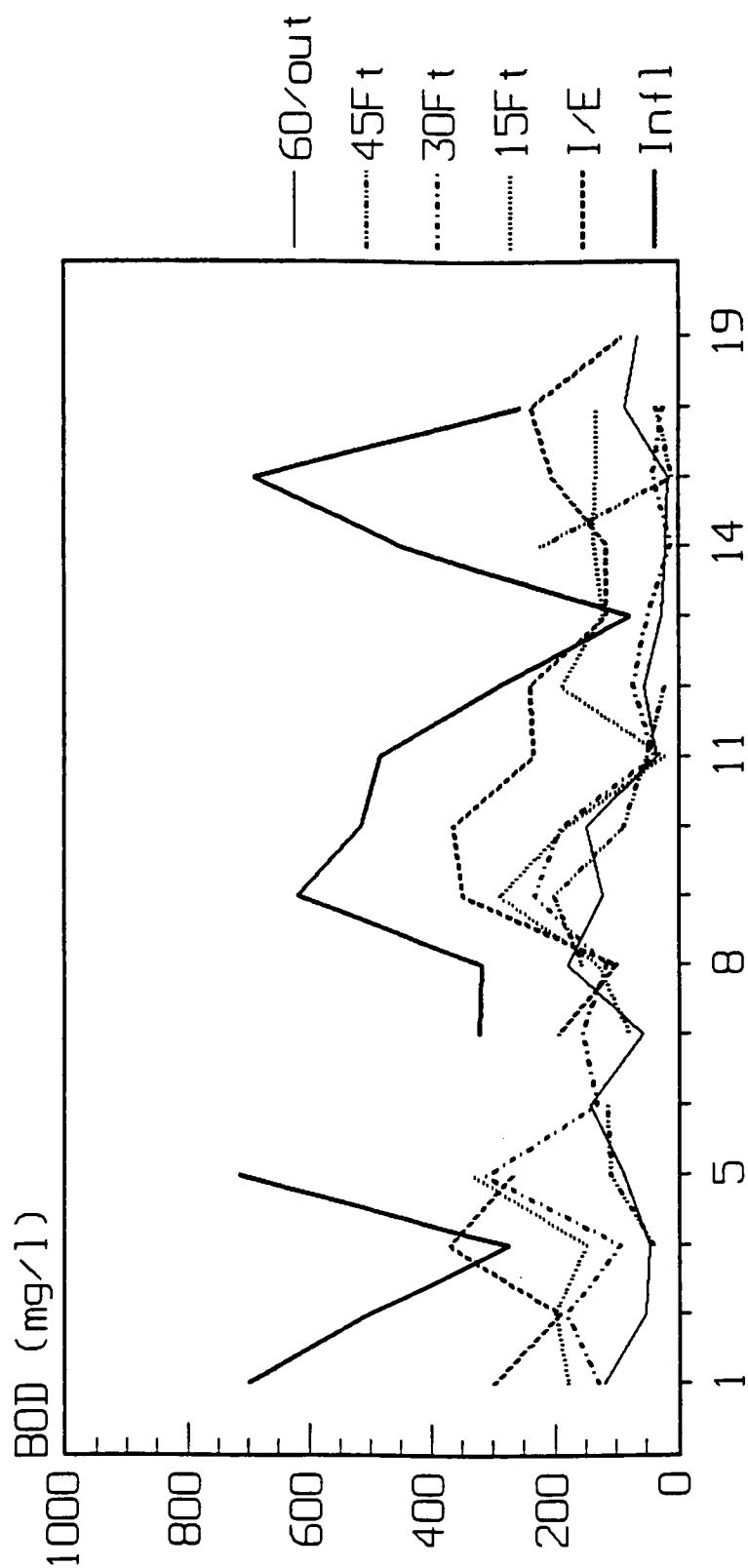


Week
Figure 4.12
Haynes House pH Data
ANFLOW and SSHF Bed

The raw BOD data is presented in Figure 4.13 for the ANFLOW unit and SSHF bed and the raw BOD profile data in Figures 4.14 and 4.15 for the SSHF bed with 6- and 12-in. soil cover. The raw COD data is presented in Figure 4.16 for the ANFLOW unit and SSHF bed and in Figures 4.17 and 4.18 as profile data for the SSHF bed with 6- and 12-in. soil cover. The influent BOD to the system was very high and ranged between 300 mg/l and 700 mg/l. The average effluent from the ANFLOW that was influent to the SSHF bed ranged between 232 mg/l and 281 mg/l of BOD and 417 mg/l and 453 mg/l of COD. The ANFLOW unit exhibited BOD removal efficiencies averaging between 45 and 49% and COD removal efficiencies averaging 27 and 36%. The SSHF bed with 6 in. of soil cover had an average BOD effluent of 75 mg/l with a removal efficiency of 73% and an average COD effluent of 85 mg/l and a removal efficiency of 81%. With 12 in. of soil cover, the BOD effluent was 59 mg/l with a BOD removal efficiency of 75%, and the COD effluent was 125 mg/l with a COD removal efficiency of 83%. The data are summarized in Tables 4.3, 4.4, and 4.5. The removal efficiencies for the whole system is shown graphically in Figures 4.19 and 4.20.

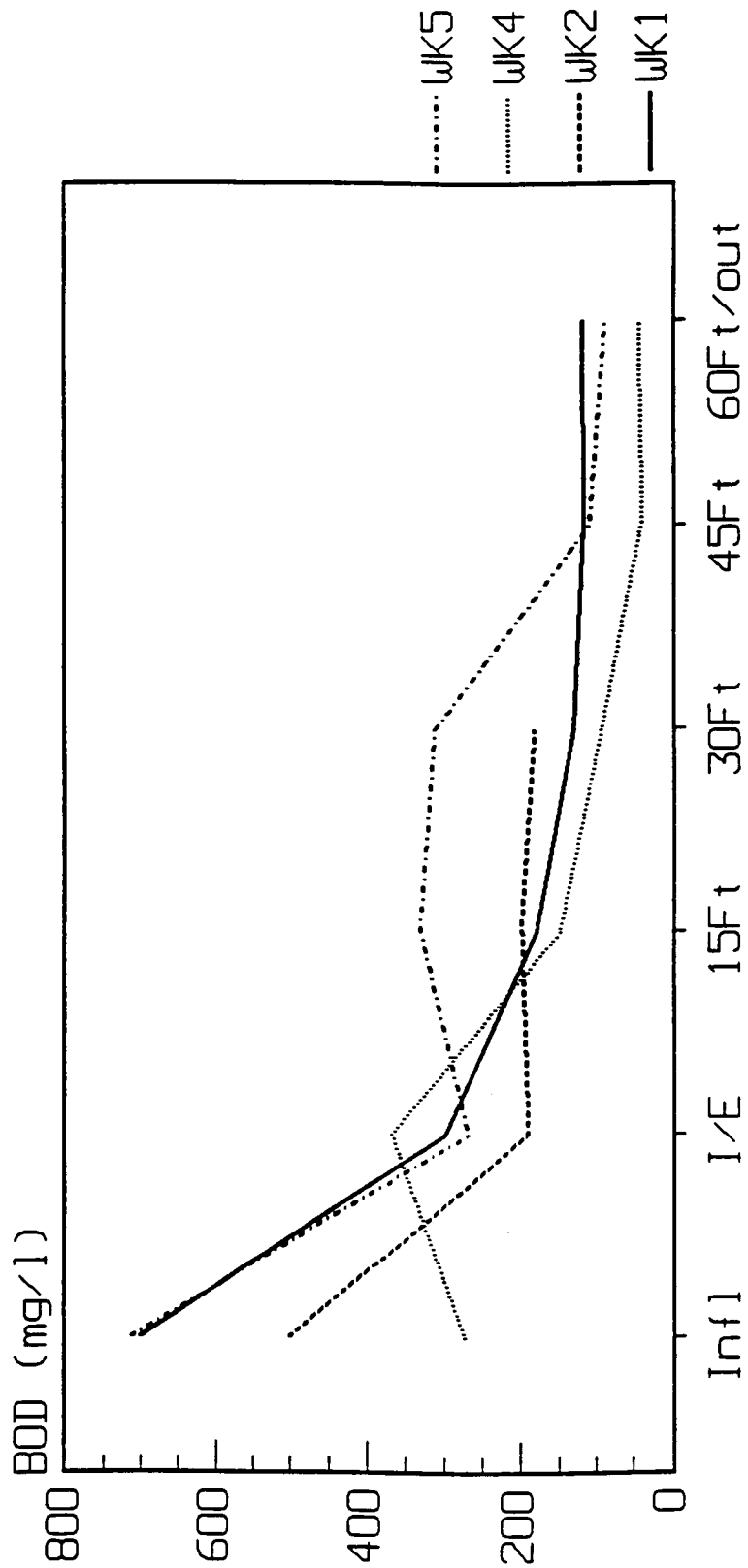
The TSS data are presented in Figures 4.21 for the ANFLOW unit and SSHF bed. The TSS for the influent to the system averaged between 154 mg/l and 161 mg/l. The effluent from the system averaged between 20 mg/l and 23 mg/l. The average ANFLOW TSS removal efficiencies ranged between 58 and 67%. The SSHF-bed TSS removal efficiencies were 61% with 6 in. soil cover and 66% with 12 in. soil cover. The removal efficiencies for the whole treatment system were 87% for the ANFLOW 6 in. soil cover bed and 86% for the ANFLOW 12 in. soil cover bed. The removal efficiencies are shown graphically in Figure 4.22. The data are presented in tabular form in Tables 4.3, 4.4, and 4.5.

May 10, 1988 to Sept. 30, 1988



Week
Figure 4.13
Haynes House BOD Data
ANFLOW and SSHF Bed

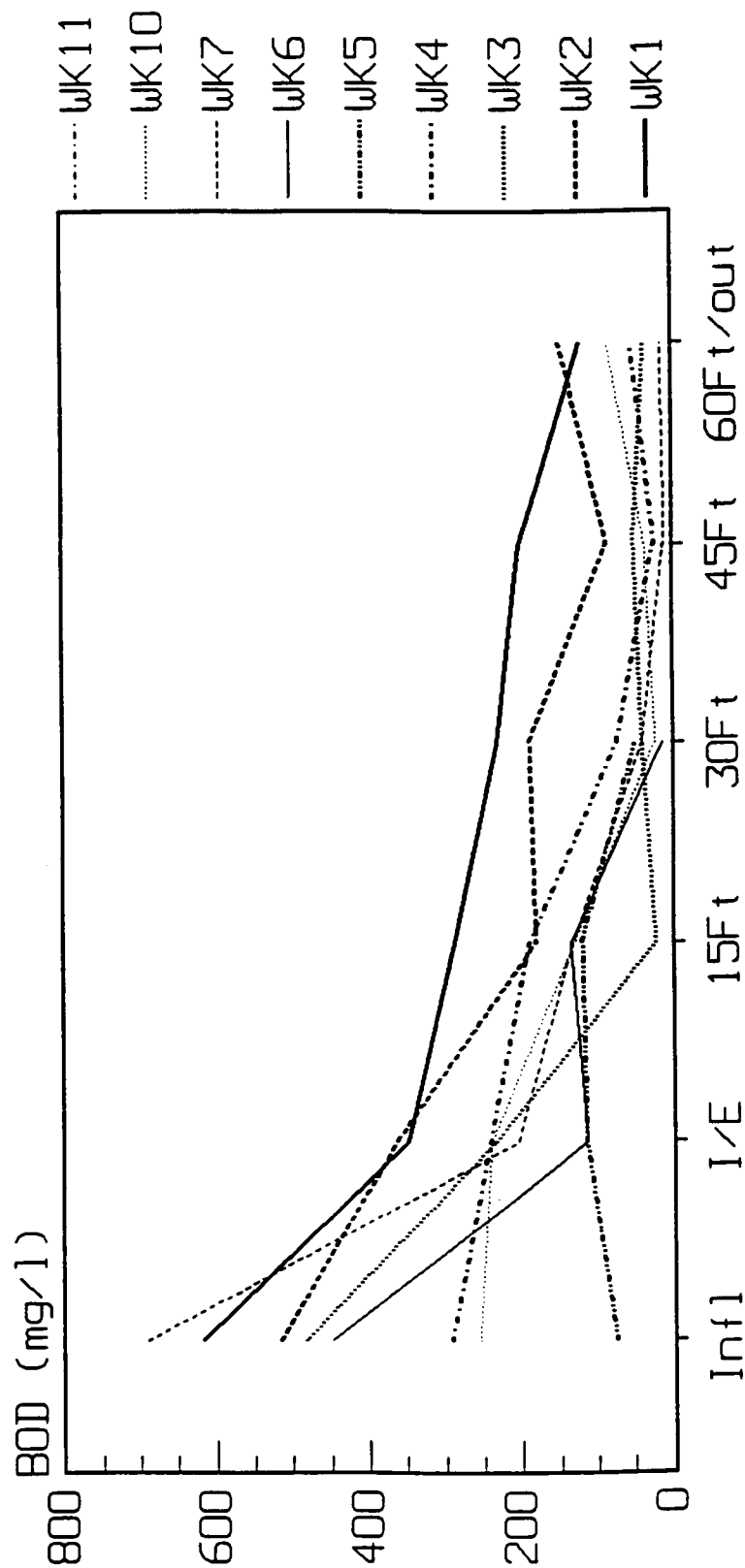
May 10, 1988 to June 17, 1988
 Profile Data, 6in. Soil Cover



Position in Treatment System

Figure 4.14
 Haynes House 6" BOD Profile Data

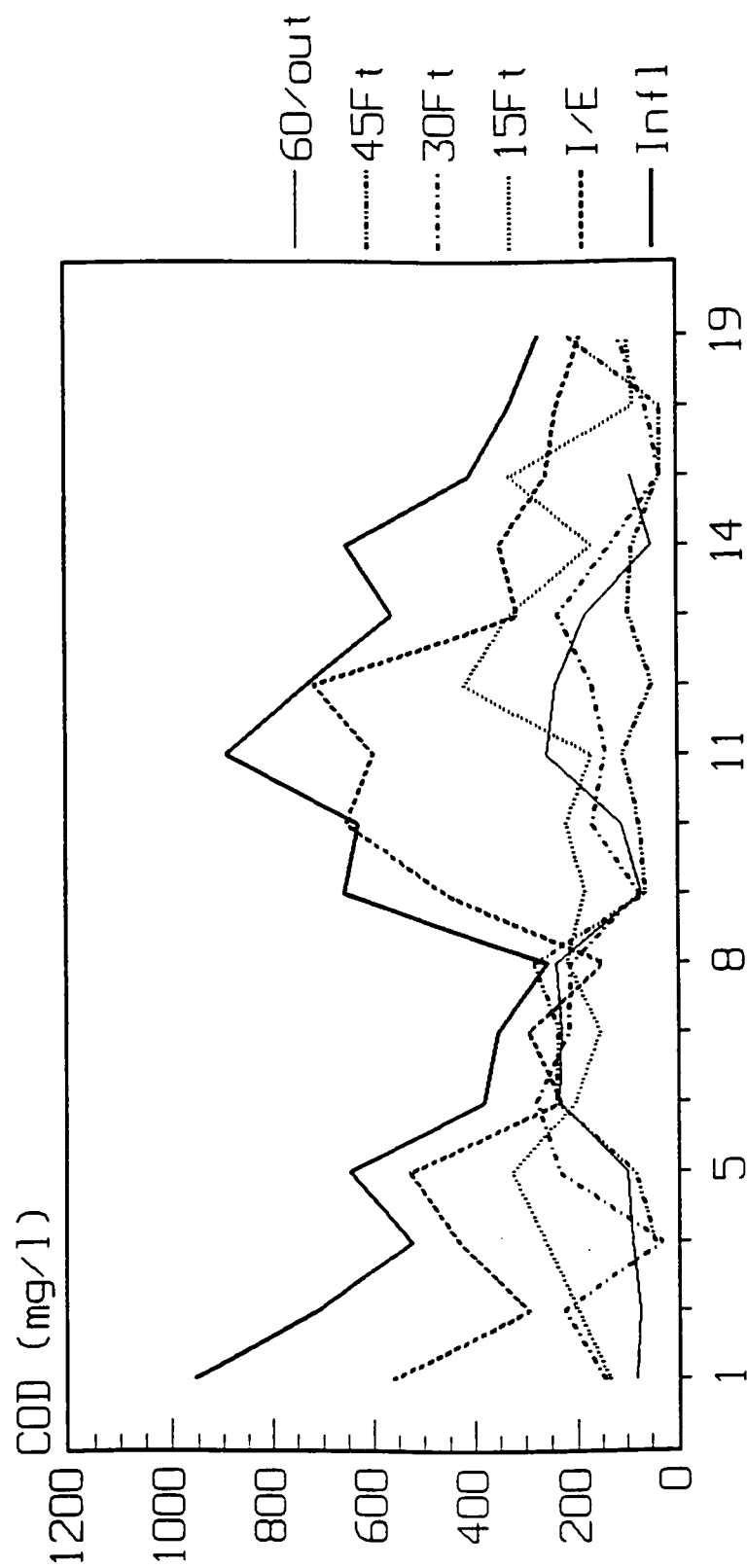
July 15, 1988 to Sept. 30, 1988
Profile Data, 12 in. Soil Cover



Position in Treatment System

Figure 4.15
Haynes House 12" BOD Profile Data

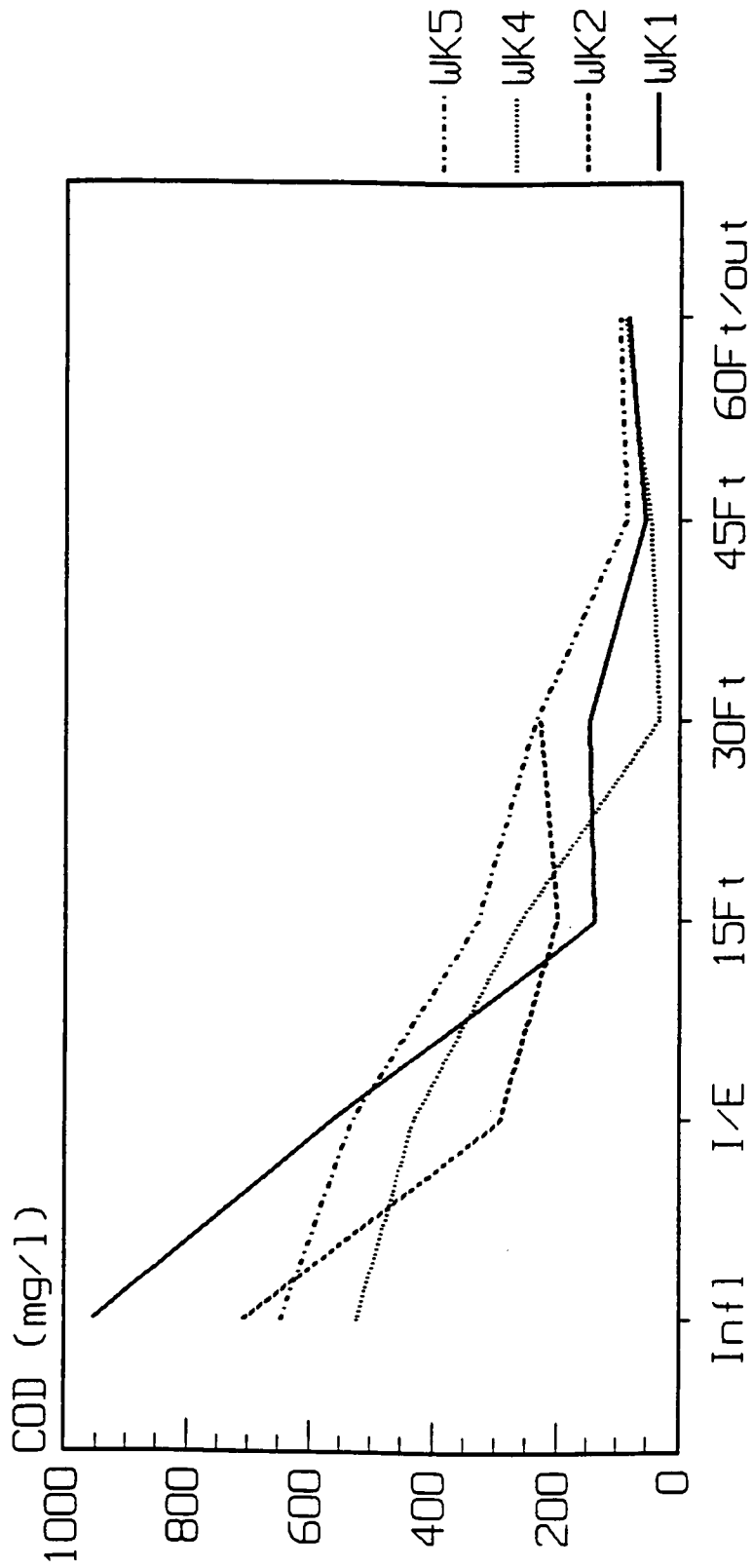
May 10, 1988 to Sept. 30, 1988
ANFLOW and Wetland Data



Week

Figure 4.16
Haynes House COD Data

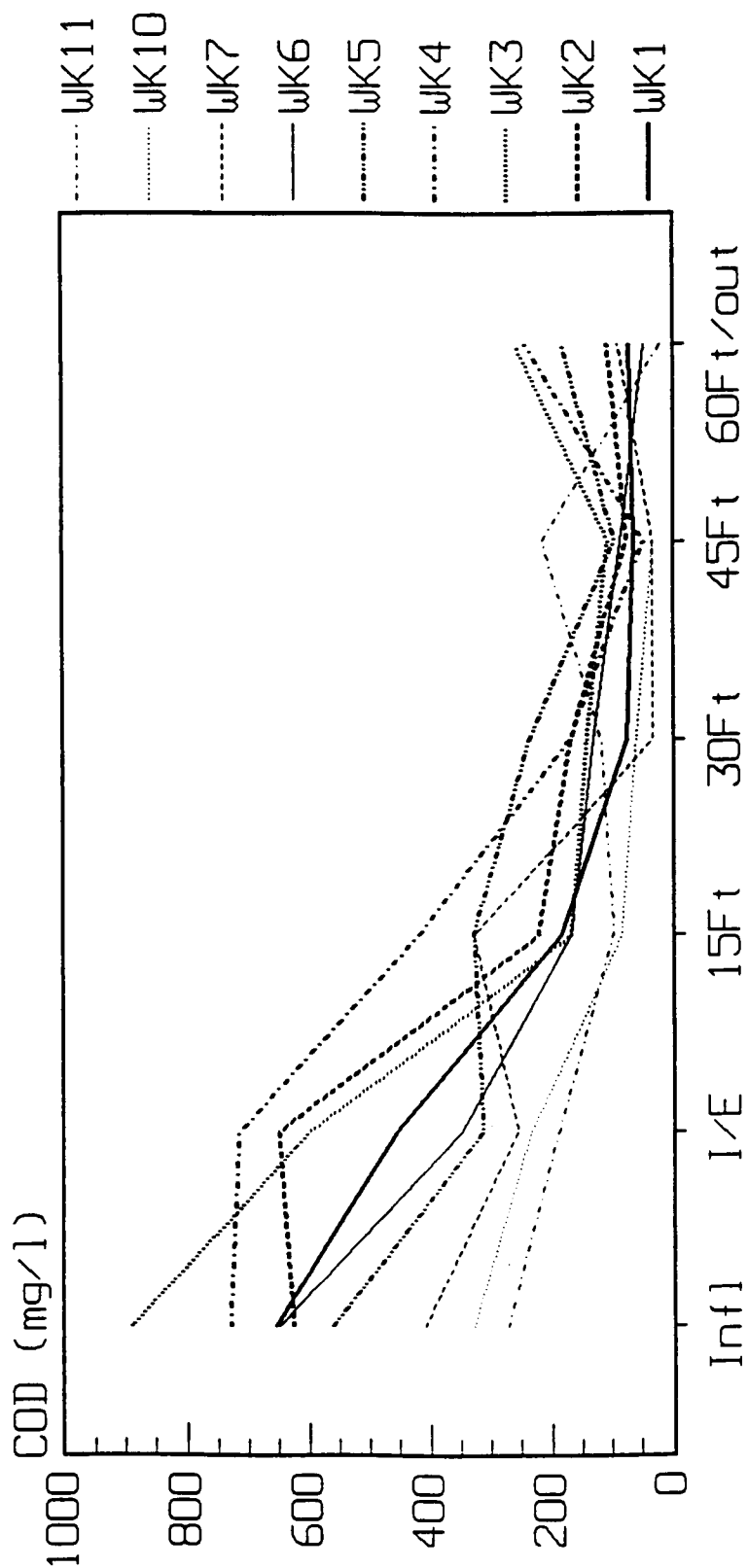
May 10, 1988 to June 17, 1988
 Profile Data, 6 in. Soil Cover



Position in Treatment System

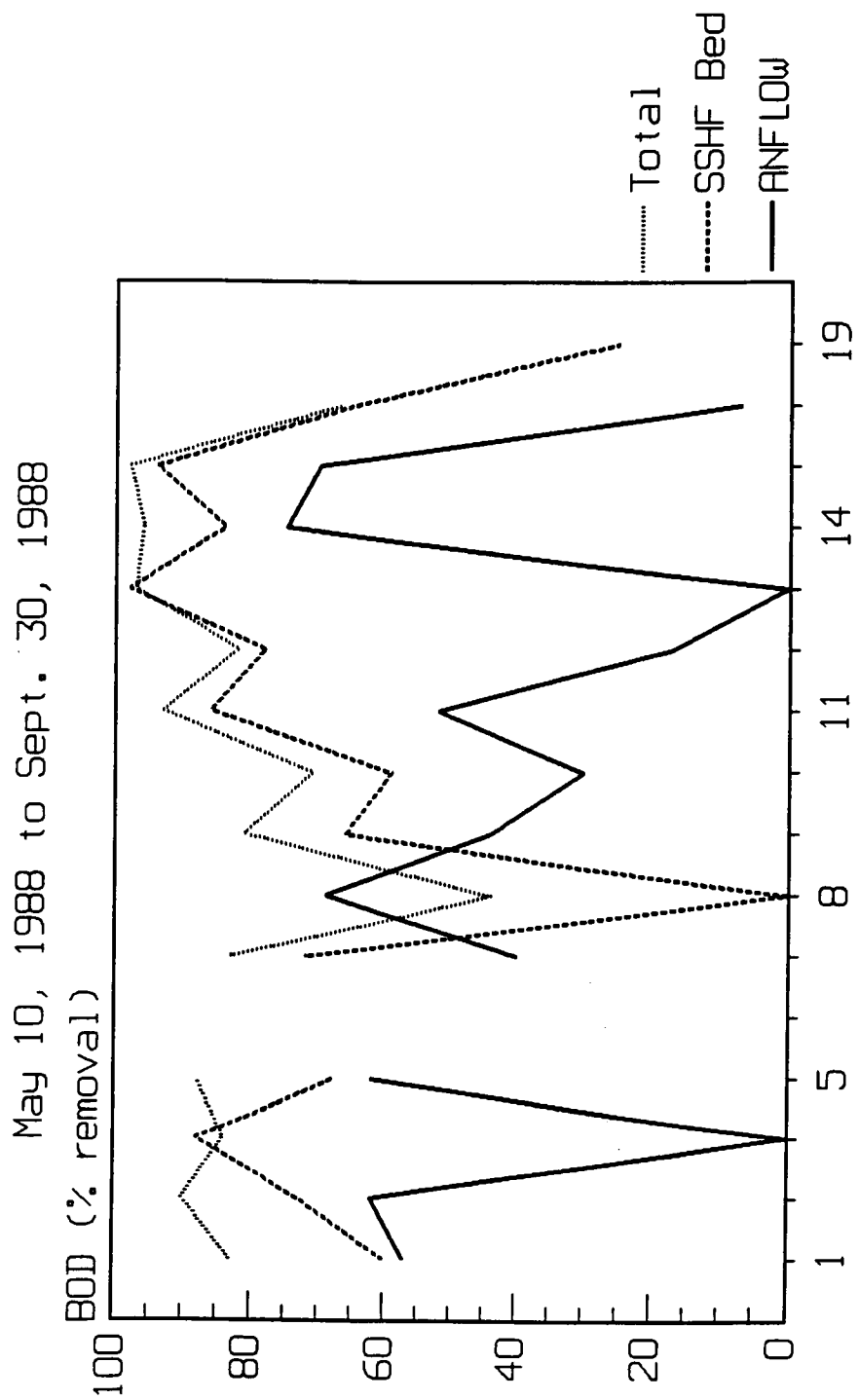
Figure 4.17
 Haynes House 6" COD Profile Data

July 15, 1988 to Sept. 30, 1988
 Profile Data, 12 in. Soil Cover



Position in Treatment System

Figure 4.18
 Haynes House 12" COD Profile Data



Week

Figure 4.19
Haynes House BOD Removal Data
ANFLOW and SSHF Bed Efficiencies

May 10, 1988 to Sept. 30, 1988

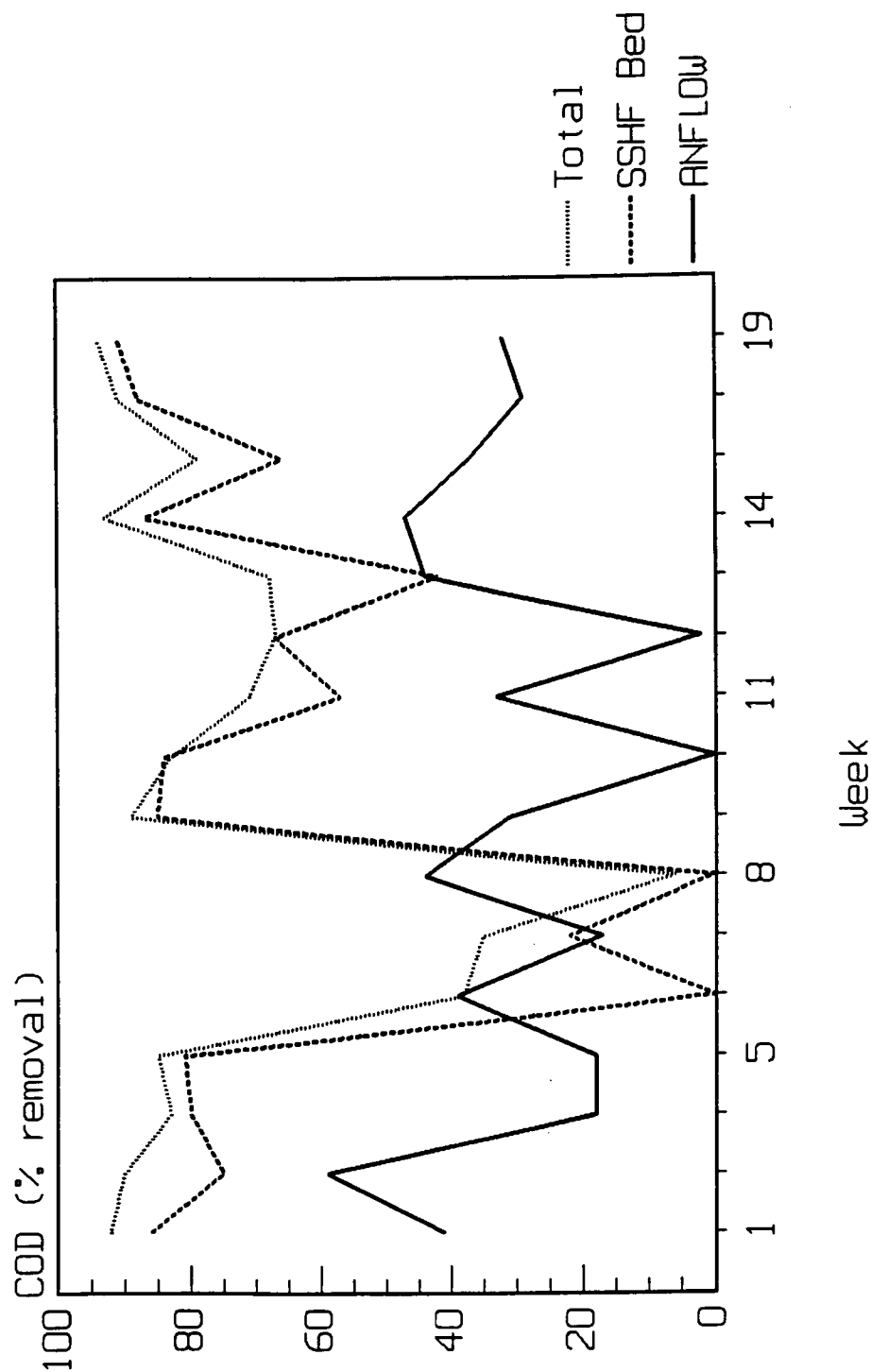
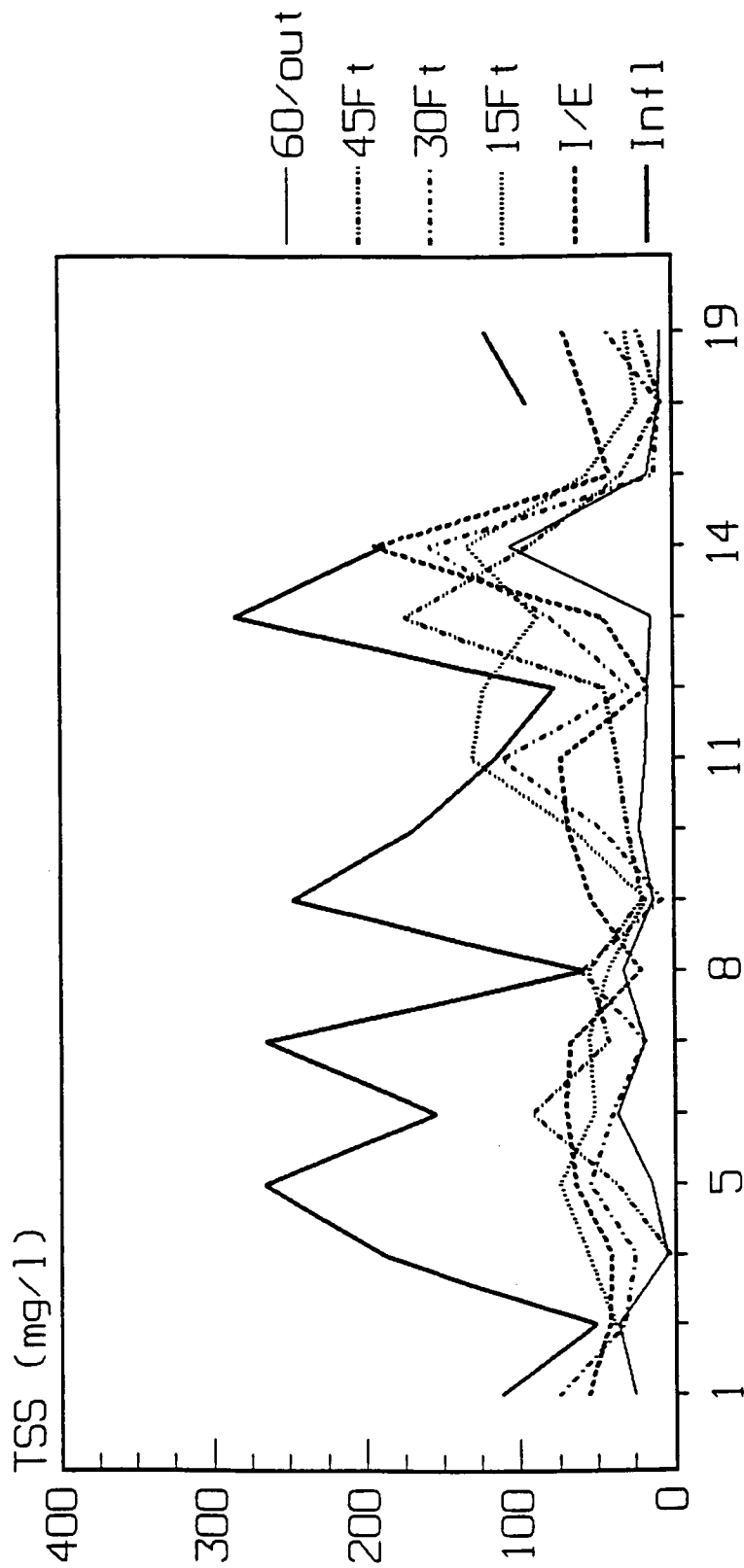


Figure 4.20
Haynes House COD Removal Data
ANFLOW and SSHF Bed Efficiencies

May 10, 1988 to Sept. 30, 1988



Week
Figure 4.21
Haynes House TSS Data
ANFLOW and SSHF Bed

May 10, 1988 to Sept. 30, 1988

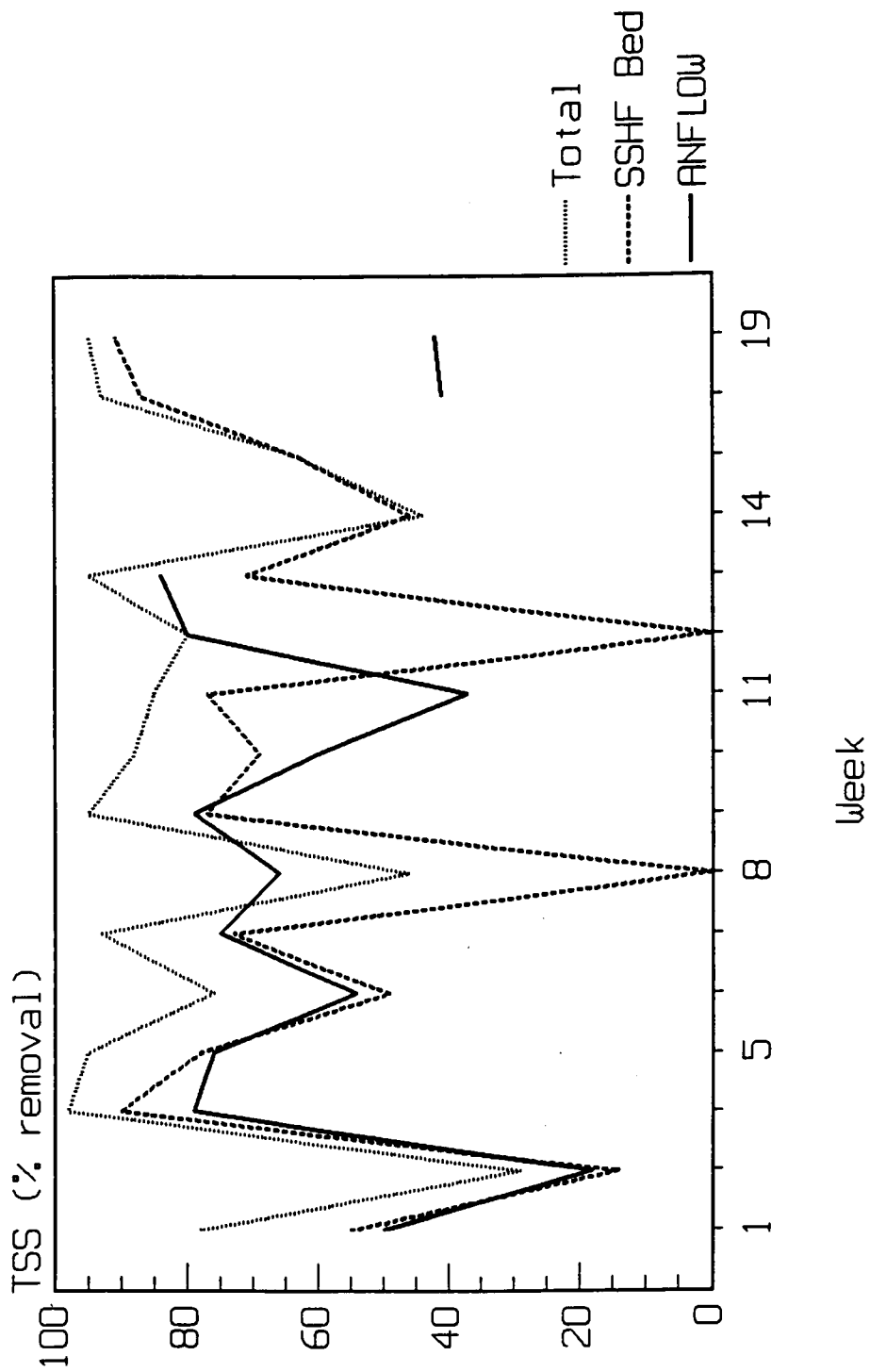


Figure 4.22
Haynes House TSS Removal Data
ANFLOW and SSHF Bed Efficiencies

**Table 4.3 Haynes 6 In. Data
Mean / Standard Deviation Values**

PARAMETER INFL	I/E	15	30	45	60/OUT	
BOD (mg/l)	548/ 207	281/ 74	214/ 80	178/ 96	88/ 42	75/ 34
COD (mg/l)	709/ 182	453/ 121	231/ 82	159/ 95	60/ 21	85/ 11
TSS (mg/l)	154/ 93	51/ 12	56/ 18	46/ 23	36/ 33	20/ 14
pH	Between 5.4 and 7.4					
Temperature	Seasonal between 15.7 C and 27 C					
Fecal Coliforms	reduction of 2 to 3 orders of magnitude					

**Table 4.4 Haynes 12 In. Data
Mean / Standard Deviation Values**

PARAMETER INFL	I/E	15	30	45	60/OUT	
BOD (mg/l)	422/ 203	232/ 92	149/ 76	81/ 83	88/ 88	59/ 53
COD (mg/l)	569/ 200	417/ 195	221/ 114	123/ 63	81/ 56	125/ 90
TSS (mg/l)	161/ 75	68/ 50	73/ 47	54/ 52	50/ 52	23/ 31
pH	Between 5.4 and 7.4					
Temperature	Seasonal between 15.7 C and 27 C					
Fecal Coliforms	reduction of 2 to 3 orders of magnitude					

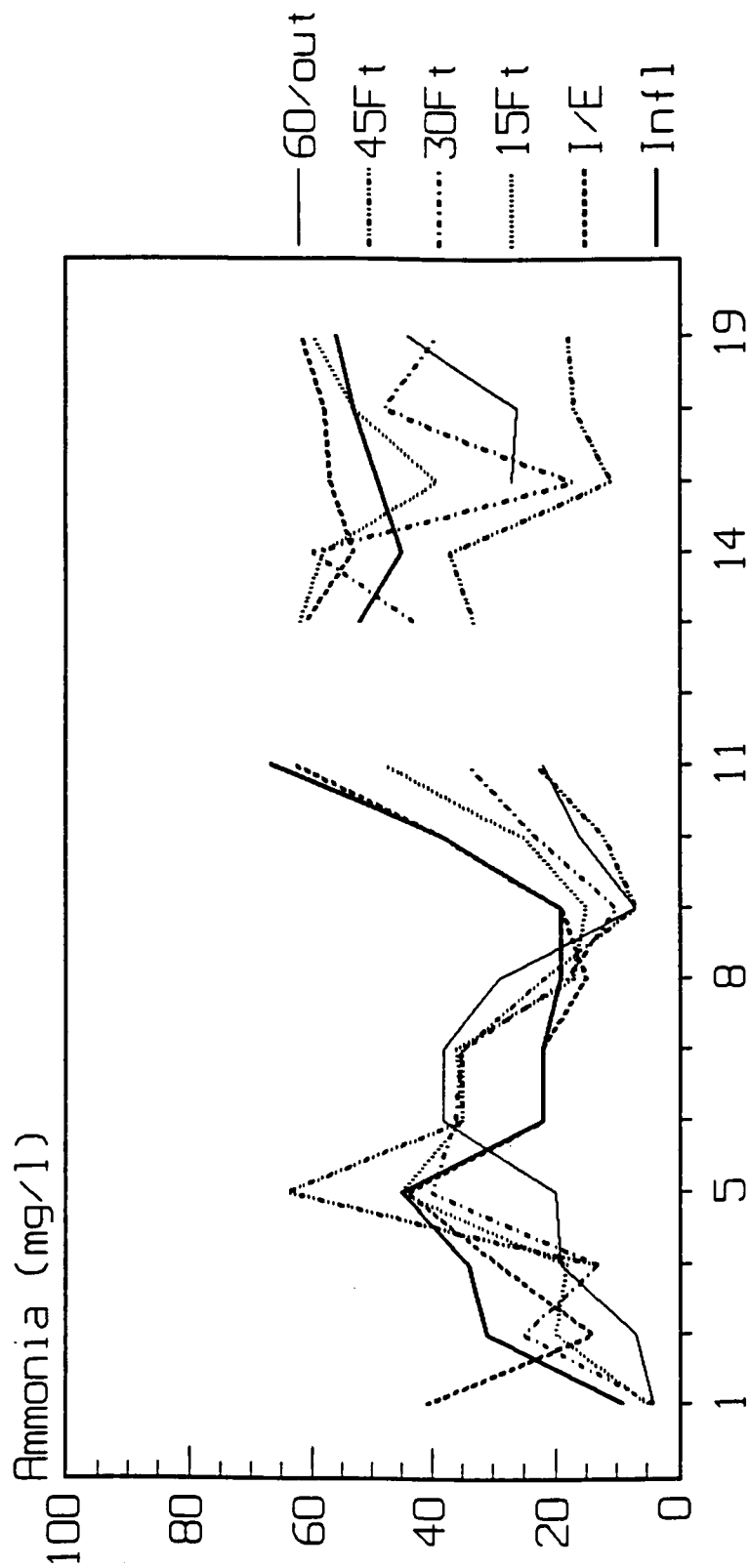
Table 4.5 Haynes Data
Average Removal Efficiency Percentage

Parameter	ANFLOW	SSHF Bed	Total
6-INCH SOIL COVER:			
BOD	49	73	86
COD	36	81	88
TSS	67	61	87
12-INCH SOIL COVER:			
BOD	45	75	86
COD	27	83	78
TSS	58	66	86

The ammonia data are presented in Figures 4.23, 4.24, 4.25, and 4.26. Figure 4.23 shows the raw ammonia data for both the ANFLOW unit and the SSHF bed. Figures 4.24 and 4.25 show the profile data for the SSHF bed with 6- and 12-in. soil covers, and Figure 4.26 shows the removal efficiencies for the SSHF bed. The mean influent ammonia concentration to the ANFLOW unit was 30 mg/l, with an influent concentration of 32 mg/l to the SSHF bed, and a system effluent concentration of 13 mg/l for the 6-in. data. The mean ammonia influent concentration to the ANFLOW unit was 48 mg/l, with an influent concentration of 52 mg/l to the SSHF bed, and a system effluent concentration of 25 mg/l for the 12-in. data. The ammonia removal efficiency averaged 57% with the 6-in. soil cover and 47% with the 12-in. soil cover. No ammonia removal in the ANFLOW unit occurred, since it is an anaerobic process.

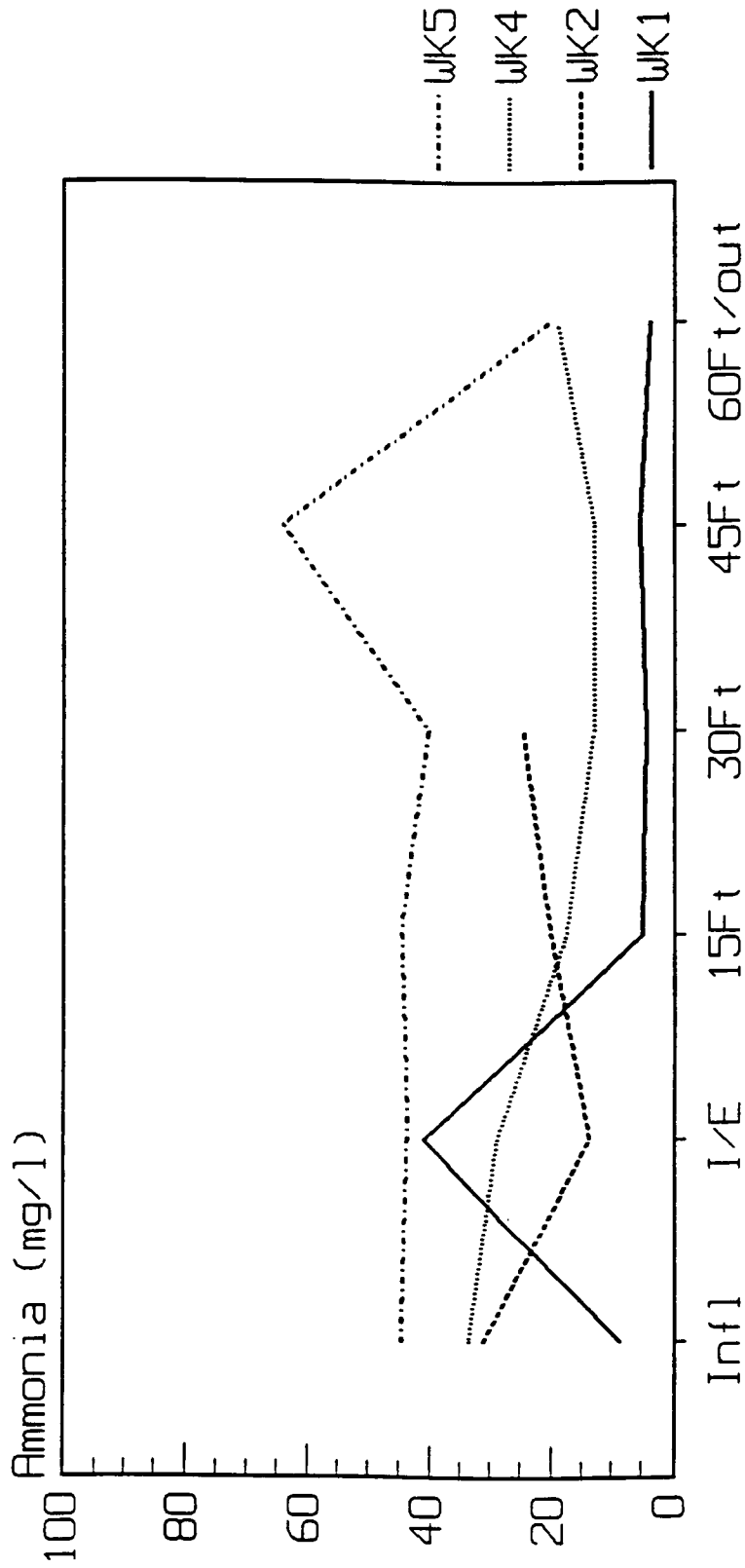
The fecal coliform data are presented in Appendix A and are summarized in Tables 4.3 and 4.4. Fecal coliform reduction in the total system was within 2 to 3 orders of magnitude.

May 10, 1988 to Sept. 30, 1988



Week
Figure 4.23
Haynes House Ammonia Data
ANFLOW and SSHF Bed

May 10, 1988 to June 17, 1988
 Profile Data, 6 in. Soil Cover



Position in Treatment System

Figure 4.24
 Haynes House 6" Ammonia Profile Data

July 15, 1988 to Sept. 30, 1988
 Profile Data, 12 in. Soil Cover

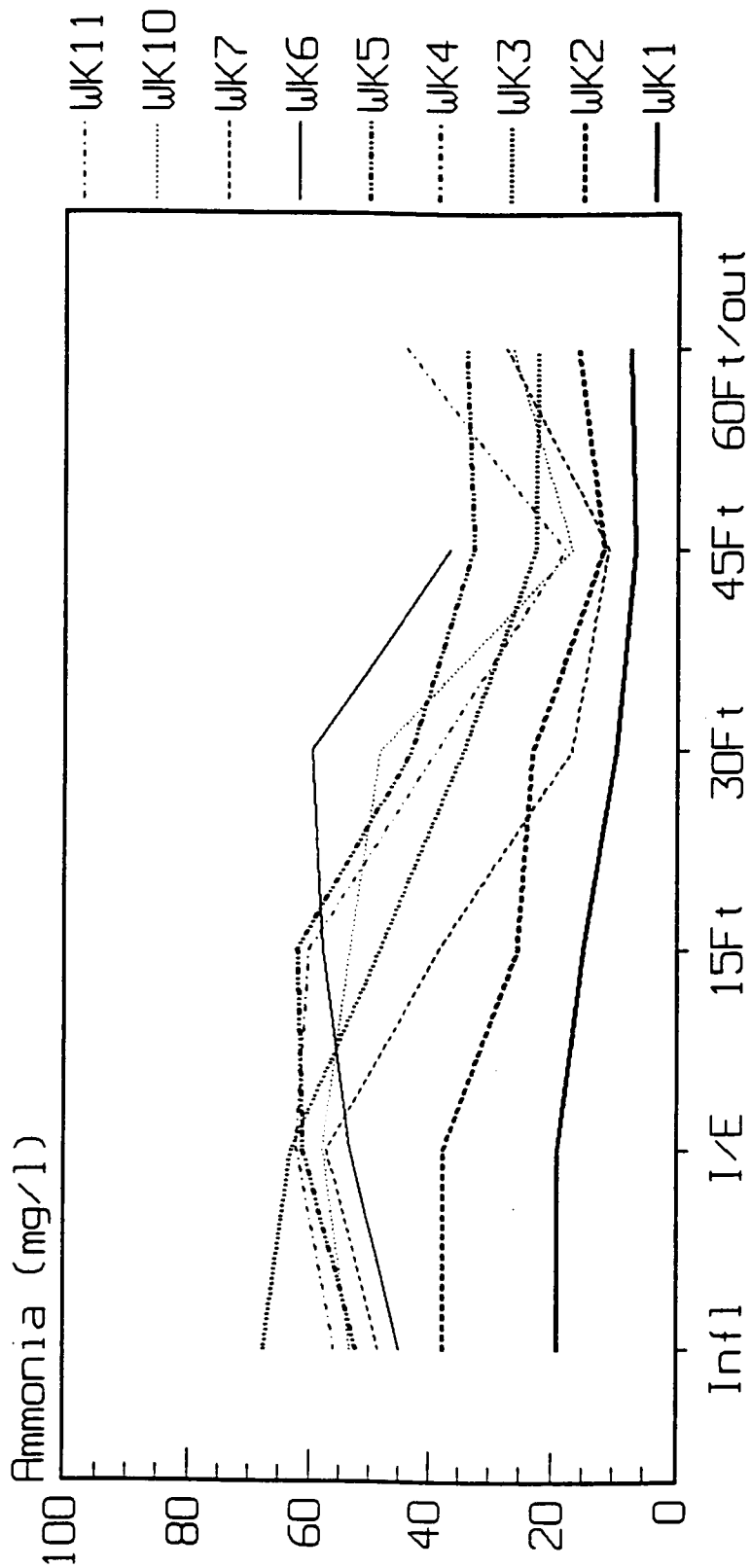
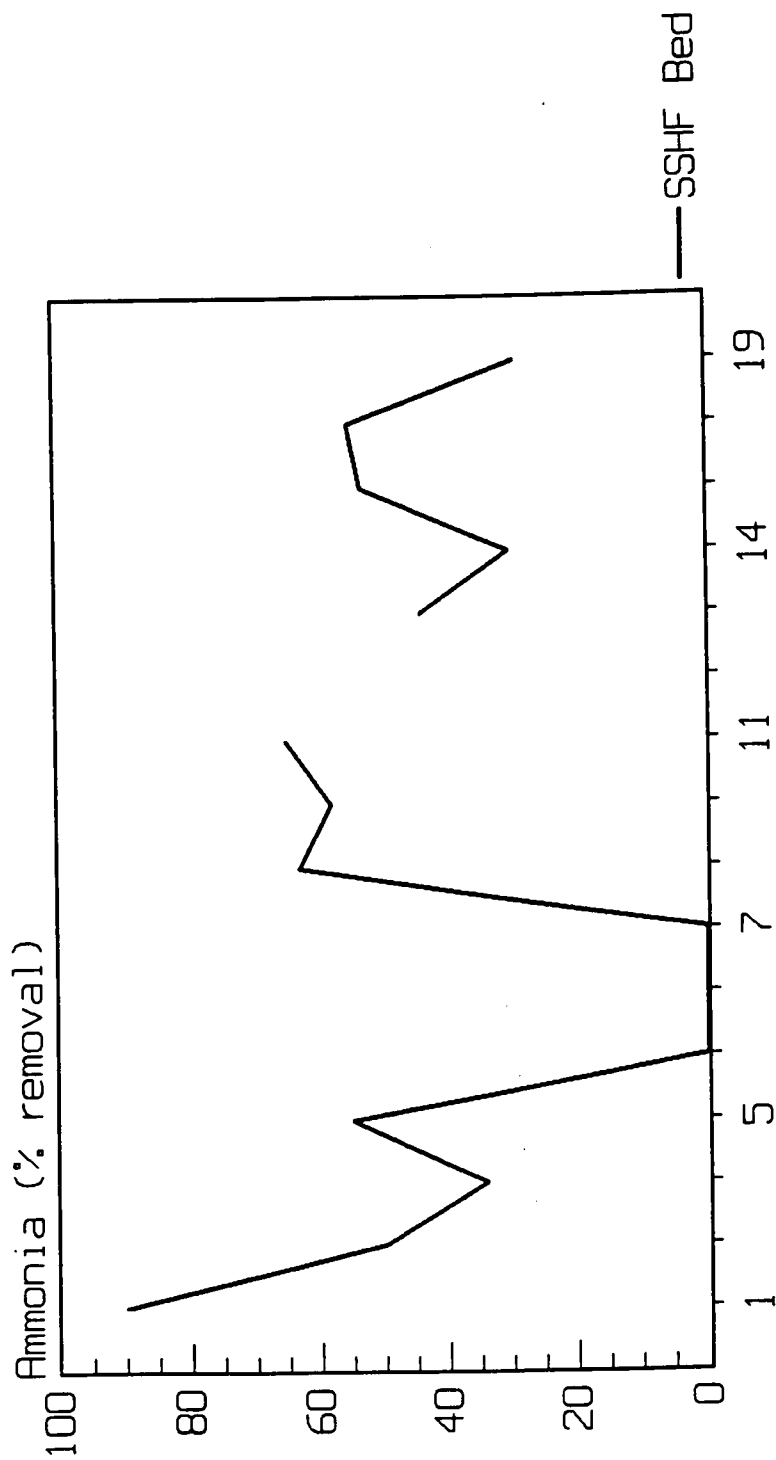


Figure 4.25
 Haynes House 12" Ammonia Profile Data

May 10, 1988 to Sept. 30, 1988
6 in. and 12 in. Soil Cover



Week
Figure 4.26
Haynes House Ammonia Removal Data
SSHF Bed Efficiency

CHAPTER V

DISCUSSION

SHAGBARK SYSTEM PERFORMANCE

The BOD and COD removal efficiencies for the ANFLOW unit averaged 43% and 35%, respectively. The ANFLOW units performance was lower than the expected 50% BOD-removal efficiency. As discussed in the literature, the removal efficiency for the ANFLOW unit should have averaged 50% based on the design of the unit with the 3 feet of packing media. The reduced BOD and COD removal efficiencies for the ANFLOW unit were most likely due to the hydraulics of the system. As discussed in Chapter III, the hydraulics of the Shagbark system allowed only a single 30- to 45-minute pump cycle per day and not the uniform steady plug flow for which the system was designed. The SSHF bed removal efficiencies averaged 62% for BOD and 49% for COD. These values are also lower than the values reported in the literature review for the similar artificial wetland systems, which were 69 to 81% for BOD removal. The low percent removal efficiencies of BOD and COD for the SSHF bed were also most likely due to the non-uniform hydraulic loading to the system. Additionally, as discussed in the Literature Review, first-order reaction equations may be used to describe the organic removal efficiency of this system. First-order reaction equations postulate that organic removal is directly proportional to detention time in the system. Therefore, as the detention time decreases, so will the organic removal. This relationship may explain why the BOD removal in this system is lower than expected and reported in the literature. Thus, BOD- and COD-removal efficiencies for the ANFLOW unit and the SSHF bed might have been higher if a uniform low flow rate had been applied to the system over a longer period of time as designed. The overall removal efficiencies

for the wastewater treatment system averaged 78% for BOD and 67% for COD. The ANFLOW/SSHf bed system performed very well, even though it did not have a uniform hydraulic loading.

The ANFLOW TSS-removal efficiency and the SSHf bed-TSS removal efficiency averaged 52% each. The average TSS removal efficiency for the total system was 77%. Here again, the percent removal of TSS was as expected for the ANFLOW unit and lower than the values reported in the literature review for artificial wetland system's, of which the SSHf bed is a modification. The TSS-removal efficiencies for the wetland cited in the literature were 89 to 94%, and the expected removal efficiency for the ANFLOW unit was 50%. Suspended solids removal is a physical process where removal of solids due to gravity settling depends on detention time in the system and flow rate through the system. The slower the flow rate through a system and the longer the time allowed for settling, the greater the settling and the greater the removal efficiency. The TSS-removal efficiency for the ANFLOW unit was better than expected, based on the hydraulic loading of the Shagbark system. The lower TSS removal efficiency of the SSHf bed compared to that of the literature may be attributed to the hydraulic loading of the system or to the suspended biomass.

No noticeable trends were observed in the ammonia data, due to the fact that little or no measurable nitrification was taking place in the treatment system. No nitrification appears to have taken place in the ANFLOW unit as expected because it is an anaerobic system where ammonia may form from any organic nitrogen present. Neither did any measurable nitrification appear to have taken place in the SSHf bed. The literature reports ammonia removal efficiencies of 11 to 94%, based on the type and density of vegetation in an artificial wetland bed, with 11%

removal for an unvegetated bed and 94% removal for a bed with bulrushes. Since the SSHF bed for this study had little or no vegetation and is similar to the artificial wetland with no vegetative cover, an ammonia removal efficiency of 11% for the bed would be expected. For nitrification to occur in the SSHF bed, the oxygen diffusion rate into the bed would have to exceed the oxygen uptake rate for BOD removal so that there would be oxygen available for the nitrification process to occur. Therefore, with the reduction of the detention time in the ANFLOW unit and consequently the reduction of the detention time in the SSHF bed due to the hydraulics of the system, it is postulated that the cumulative BOD removal in the system was not great enough to allow the nitrification process to occur.

Removal efficiencies or fecal coliform reduction throughout the system was within 2 to 3 orders of magnitude, as expected. This same reduction is reported in the literature review.

The Shagbark performance data are summarized in Table 5.1 along with the Haynes House performance data.

HAYNES HOME SYSTEM PERFORMANCE

Startup for the wastewater treatment system was very rapid, as is evident by the data presented in Chapter IV. The strength of the influent wastewater to the system was a surprise. The influent BOD ranged from 300 to 700 mg/l. The system was designed assuming an influent BOD of 150 to 250 mg/l (a typical domestic strength waste) and a 50% BOD-removal efficiency for the ANFLOW unit. The inhabitants of the house were questioned about their diets and activities in an effort to determine the cause of the high-strength organic waste. However, the cause of the high strength waste was never determined. As evident by the data presented, both

Table 5.1 Shagbark and Haynes House Summary Data

Removal Efficiencies (%)

	ANFLOW		SSHFB Bed		System Performance	
	Shagbark	Haynes House	Shagbark	Haynes House	Shagbark	Haynes House
			6" cover	12" cover	6" cover	12" cover
BOD	43	45 - 49	62	73	78	86
COD	35	27 - 36	49	81	67	88
TSS	52	58 - 67	52	61	77	86
Ammonia	0	0	0	57	0	57
				47		47

the ANFLOW unit and the SSHF bed performed very well. The ANFLOW unit performed as expected, with BOD-removal efficiencies of 45 to 49% and COD-removal efficiencies of 27 to 36%. These values are consistent with the reported Shagbark data and very close to the BOD removal efficiency of 50% cited in the literature. The BOD-removal efficiency for the SSHF bed with 6-in. soil cover averaged 73% and 81% for COD removal. The 12-in. soil cover resulted in an average BOD-removal efficiency of 75% and an average COD-removal efficiency of 83%. The literature reports BOD-removal efficiencies for similar artificial wetland systems of 69 to 81%. The SSHF bed for this study performed as expected. Once again, if first-order reaction kinetics are used to describe this system (as stated in the literature), organic removal can be considered a function of detention time in the system such that, even though the organic loading rate to the system may increase, the removal efficiency of the system is expected to remain the same. As expected, these removal efficiencies are higher than the Shagbark data because of the increased detention time in the Haynes House system. The increased detention time in this system relative to the Shagbark system may be attributed to the better hydraulics of the system and the increased length of the SSHF bed. The system-removal efficiencies for BOD and COD averaged 86% and 88%, respectively for the system with 6-in. of soil cover. The system-removal efficiency for BOD and COD averaged 86% and 78%, respectively, for the system with 12-in. of soil cover.

The TSS removal efficiencies averaged between 58 and 67% for the ANFLOW unit. These values were higher than the Shagbark data, as expected, because of the improved hydraulics of the system. The average TSS removal efficiencies were also slightly higher than those cited in the literature, which was 50% for the ANFLOW unit. The SSHF-bed TSS-removal efficiencies averaged 61% for the 6-in. soil cover period and 66% for the 12-in. soil cover

period. The TSS removal efficiencies for the SSHF bed were as expected and did not change much with the addition of 6-in. of soil cover. These values once again were higher than those reported for the Shagbark system because of the improved hydraulic loading of this system, since TSS removal is a physical process. The system-removal efficiency for TSS averaged 87% with the 6-in. soil cover and 86% with the 12-in. soil cover.

As stated previously, little or no measurable nitrification was taking place in the ANFLOW unit, because it is an anaerobic system where ammonia may form from any organic nitrogen present. This phenomena is shown in the data presented in Chapter IV. The ammonia-removal efficiencies for the SSHF bed averaged 57% with 6 in. of soil cover and 47% with 12 in. of soil cover. The literature reports ammonia-removal efficiencies of 11 to 94% for artificial wetland systems, depending on the density and type of vegetation, with a value of 11% for an unvegetated bed. Since the SSHF bed is similar to an artificial wetland with no vegetative cover, an ammonia removal efficiency of 11% is expected. For the Haynes House system, ammonia was being removed from the system at a rate higher than expected.

The fecal coliform reduction throughout the whole treatment system was within 2 to 3 orders of magnitude, as expected and reported in the literature.

The Haynes House performance data are summarized in Table 5.1 along with the Shagbark performance data.

TYPICAL DESIGN EQUATIONS

As discussed in the Literature Review, typical design equations for artificial wetland

systems and fixed media filter systems may be used to describe and help evaluate the SSHF bed, since it is similar to these technologies.

Artificial wetland systems -- specifically, the root zone system -- uses the first-order reaction equation to describe the system, wherein organic removal efficiency of the system is directly proportional to the time of contact in the system. Fixed-media filter technology also uses the same philosophy for describing its system. The Eckenfelder equation presented in the Literature Review also makes use of first-order reaction kinetics. The Eckenfelder equation postulates that organic removal is directly proportional to the filter media depth. As shown in the Literature Review, Eckenfelder defines residence time in the filter as related to operating and filter media characteristics. If residence time is substituted back into the equation, then essentially the same equation used for artificial wetland systems appears. This equation as presented in Chapter II is as follows:

$$1. \quad C_1 = C_o e^{-KL}$$

where:

$$C_1 = \text{Desired effluent concentration (BOD or COD)}$$

$$C_o = \text{Influent concentration (BOD or COD)}$$

$$K = \text{Dynamic Kinetic Constant}$$

$$L = \text{Length of wetland bed}$$

In evaluating the data for the Shagbark and the Haynes Home SSHF beds, the beds tend to follow first-order reaction kinetics. If a system follows first-order reaction kinetics, then the organic removal efficiency of the system would be directly proportional to the residence time in the system. As the residence time in the bed increases, so should the organic removal efficiency.

The Shagbark system did not have a uniform hydraulic loading during this study, which decreased the detention time in the SSHF bed. The SSHF bed exhibited organic removal efficiencies of 62% for BOD and 49% for COD. The Haynes House data showed increased removal efficiencies for BOD and COD in the SSHF bed compared to the Shagbark data. The SSHF bed removal efficiencies for the Haynes Home, as shown in Table 5.1, were 73% for BOD and 81% for COD removal with the 6-in. soil cover and 75% for BOD and 83% for COD removal with the 12-in. soil cover. The Haynes system had more of a uniformly constant flow than the Shagbark system. The SSHF bed for the Haynes House was also 10 ft longer than the Shagbark SSHF bed. This increased length of bed, along with an essentially uniform plug flow to the system, should have increased the detention time in the SSHF bed (compared to the Shagbark bed). The data seem to show that the organic removal efficiency in the SSHF bed tended to be independent of the influent concentrations and dependent on the length (directly) and flow (inversely), consistent with first-order kinetics.

The Haynes House system also experienced higher-than-normal organic loading to the system. If the system follows first-order reaction kinetics, the higher organic loading should not measurably affect the organic removal efficiencies. As shown in the data presented in the previous chapter, the SSHF bed tended to have higher organic removal efficiencies when it experienced a high organic loading than when it experienced a non-uniform hydraulic loading to the system, a finding consistent with first-order reaction kinetics.

To further evaluate the research data in terms of the typical design equations for artificial wetland systems and fixed media filter systems, the data were graphed. The Haynes House data were collected in the form of concentration versus length in the SSHF bed and thus were

appropriate for this evaluation. The Shagbark data were only collected as influent and effluent samples. The BOD and COD data were substituted for concentration, and position in the SSHF bed was substituted for time in the first-order reaction equation presented above and discussed in the Literature Review. An exponential regression was performed on the Haynes House data by taking the natural log of the BODs and CODs versus position in the SSHF bed for both 6-in. and 12-in. soil cover data. The slope of the line for the regression equation is the dynamic kinetic constant as described by the first-order reaction equation. The kinetic constants and the correlation coefficients for the four equations developed are presented in Table 5.2. The statistical data used for the formulation of these equations are shown in Appendix B. As shown in Table 5.2, the dynamic kinetic constants for BOD and COD are similar for both the 6-in. and 12-in. soil cover periods. The correlation coefficients are somewhat low but may be due to the wide variability of the influent waste stream. This problem is one that is commonly encountered when monitoring wastewater systems and using grab sampling procedures. Figures 5.1, 5.2, 5.3, and 5.4 show plots of the raw BOD and COD data versus position in the SSHF bed for both 6-in. and 12-in. soil cover, with the regression line through the data, the 95% confidence interval around the regression line, and the regression equation. Substituting the kinetic constants into Equation 1, the equations become:

2. $L = \ln (C_i / C_o) / -0.0244$
3. $L = \ln (C_i / C_o) / -0.0298$
4. $L = \ln (C_i / C_o) / -0.0302$
5. $L = \ln (C_i / C_o) / -0.0315$

**Table 5.2 Haynes Data
Design Equation Statistics**

EQUATION	KINETIC CONSTANT	r VALUE
BOD Data		
6-in. soil cover	- 0.0244	- 0.7958
12-in. soil cover	- 0.0298	- 0.5697
COD Data		
6-in. soil cover	- 0.0302	- 0.7741
12-in. soil cover	- 0.0315	- 0.7249

If proven to be valid, these equations could be used to predict bed lengths for the SSHF bed using influent BOD or COD concentrations and desired BOD and COD effluent concentrations at the same hydraulic loading as tested in this study.

The average influent BOD concentrations to the Haynes House system, as reported in Chapter IV, were 548 mg/l for the 6-in. soil cover period and 422 mg/l for the 12-in. soil cover period. The average SSHF bed effluent BOD concentrations for the Haynes House data were 75 and 59 mg/l for the 6- and 12-in. soil cover periods, respectively. The predicted flow rate to the system was 240 gpd. If these concentrations were substituted back into the prediction equations for a root zone system in Chapter II, the predicted land requirements for the artificial wetland root zone bed would be an area 34 to 35 ft by 96 ft, with a wetland bed depth of 2 ft for both soil cover periods. These calculations are shown in Appendix C. If, based on the Haynes Home BOD data, the equations formulated in this chapter were used to predict the area requirement of the SSHF bed, the SSHF beds predicted dimensions would be 55 ft by 4 ft for the 6-in. BOD

95% Prediction Confidence Band
 $Y = 287.8535 * \text{Exp}(-0.0244 * X)$

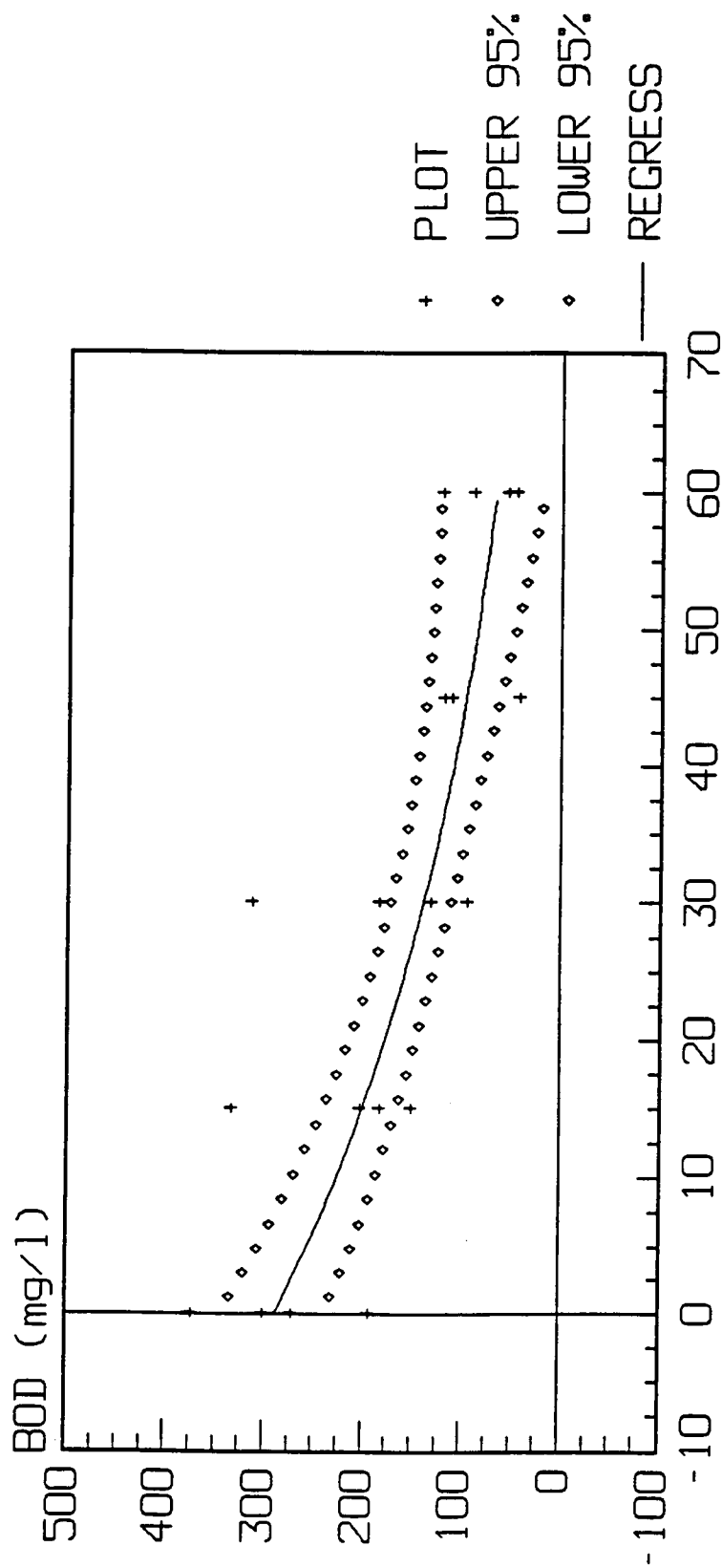


Figure 5.1
 Haynes House 6" Soil Cover BOD Prediction Equation

95% Prediction Confidence Band
 $Y = 358.6752 * \text{Exp}(-0.0302 * X)$

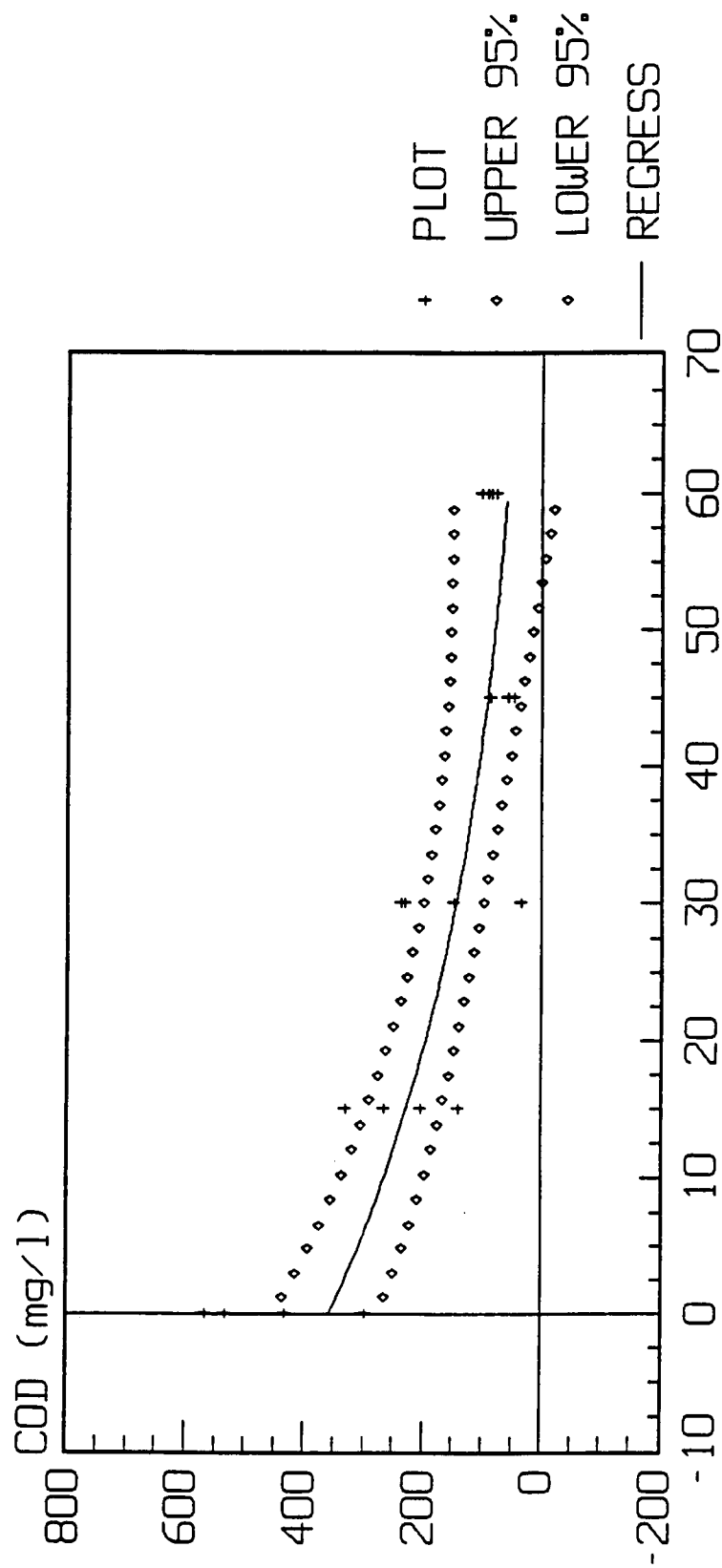


Figure 5.2
 Haynes House 6" Soil Cover COD Prediction Equation

95% Prediction Confidence Band
 $Y = 174.545 * \text{Exp}(-0.0298 * X)$

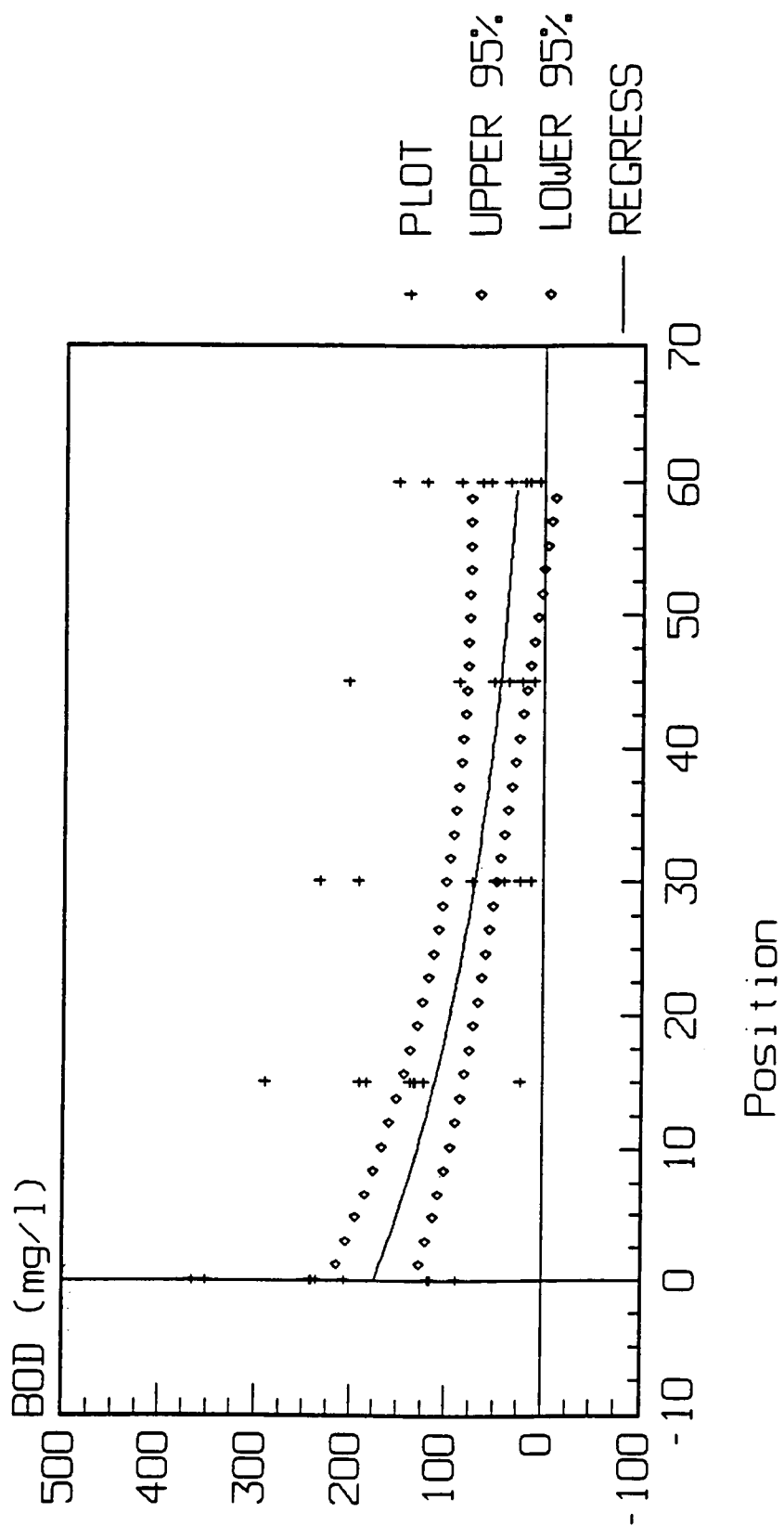


Figure 5.3
 Haynes House 12" Soil Cover BOD Prediction Equation

95% Prediction Confidence Band
 $Y = 322.9 * \text{Exp}(-0.0315 * X)$

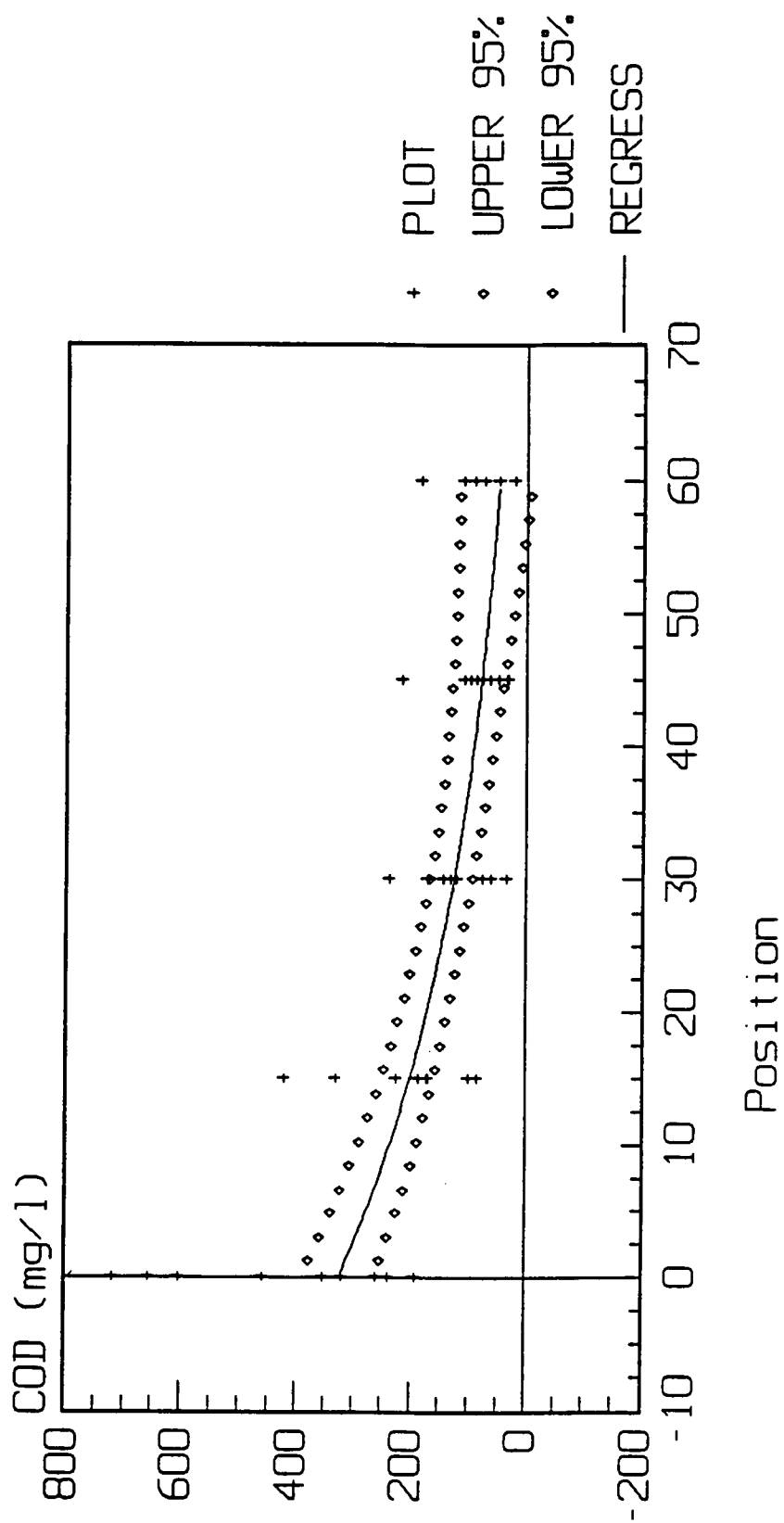


Figure 5.4
 Haynes House 12" Soil Cover COD Prediction Equation

equation and 47 ft by 4 ft for the 12-in. BOD equation, with corresponding bed depths of 2.5 and 3 ft. These area requirements are based on the influent BOD concentrations to the SSHF bed, which were 281 mg/l for the 6-in. period and 232 mg/l for the 12-in. period and the effluent BOD concentrations stated above. Thus, using this type of analysis, shows that the land requirements for the septic tank/ANFLOW unit and SSHF bed tend to be measurably less than the land requirements for an artificial wetland root zone system.

In formulating the above equations (equations 2 through 5), the kinetic constants for the BOD equations for the 6- and 12-in. periods were similar in value. The same was noted for the kinetic constants of the COD equations (see Table 5.2). It was also observed that the BOD and COD removal efficiencies for the 6- and 12-in. periods were almost identical (see Table 5.1). The 6-in. of additional soil cover did not seem to have any effect on the organic removal efficiency of the SSHF bed. If the oxygen transfer into the bed was by surface diffusion, and if oxygen diffusion decreases with depth, then the organic removal efficiency of the bed should have decreased with the additional soil cover if this was an aerobic system. If in fact no real difference between the kinetic constants and the organic removal efficiencies with the different soil cover depths resulted, this would lead one to believe that the oxygen transfer into the bed may not have been a limiting factor for BOD and COD removal in the system. The system may have been anaerobically controlled.

Since this was a performance study, no oxygen measurements were taken during this research to determine if the SSHF beds stayed anaerobic throughout their length or if, in fact, at some point in the bed the bed became facultative or even aerobic. As stated previously, the organic removal efficiency for the Haynes House SSHF bed was good. The influent waste stream

to the SSHF bed was anaerobic and came from the ANFLOW unit. Thus, initially at least, the SSHF bed should have been an anaerobic system. The literature states that organic removal rates in anaerobic systems proceed very well. Nitrogen removal from the treatment systems was not required. Ammonia concentrations were measured for informational purposes. Ammonia was also being removed from the Haynes Home SSHF bed at efficiencies of 47% and 57%. The method of ammonia removal from the system is unknown.

In an effort to better understand what the oxygen transfer rate into the SSHF bed might be, some theoretical calculations were performed. Using equations from *ChemoDynamics: Environmental Movement of Chemicals in Air, Water, and Soil* (45), the theoretical effective diffusivity of oxygen in air was found to be $3.1\text{E-}2\text{ cm}^2/\text{s}$, and the theoretical effective diffusivity of oxygen in water was found to be $3.1\text{E-}6\text{ cm}^2/\text{s}$. Thus, the diffusion rate of oxygen into the saturated bed layer (1.5 ft of gravel) should be measurably slower -- orders of magnitude slower -- than the diffusion rate of the oxygen into unsaturated layers (6 in. each of soil and gravel). The maximum theoretical flux rate of oxygen into the unsaturated layer was calculated to be approximately $2.52\text{E-}7\text{ gms of O}_2/\text{cm}^2\text{-s}$. This was calculated using a soil/gravel depth of 1 ft and the assumption that the oxygen concentration at the bottom of the layer was zero. The average theoretical flux rate of oxygen down into the water-saturated gravel layer was calculated to be $1.22\text{E-}12\text{ gms of O}_2/\text{cm}^2\text{-s}$. This was calculated using a depth of 9 in. and the assumption that the oxygen concentration at the bottom of the bed was zero. The theoretical flux rate at the surface of the water-saturated zone was found to be $1.62\text{E-}10\text{ gms of O}_2/\text{cm}^2\text{-s}$, which was still orders of magnitude lower than the flux rate into the unsaturated layer. These last two flux rates could theoretically support a BOD loading of 70 mg/l at the surface of the water-saturated zone and a BOD loading of $1.2\text{E-}2\text{ mg/l}$ within the water-saturated zone. Theoretically, the

concentration of oxygen in the water-saturated zone decreases rapidly within the first inch of depth. These calculations are shown in Appendix C.

These theoretical calculations would seem to support the argument that the SSHF bed tends to be an anaerobic system since the bed may only rely on surface diffusion of oxygen into the SSHF bed. These theoretical oxygen diffusion rates as calculated above are low, too low to support the organic loading to the system as an aerobic system. These calculations would also suggest that a thin surface layer where the oxygen concentrations were high enough to support the nitrification process might have existed. The mechanism or mechanisms by which the ammonia was removed from the systems studied for this research are undetermined, since there is no evidence to support any of the assumptions.

CHAPTER VI

CONCLUSIONS

The septic tank/ANFLOW unit in conjunction with a SSHF bed proved to be a very effective wastewater treatment system for the two projects studied. The overall system removal efficiencies were 78 to 86% for BOD, 67 to 88% for COD, 77 to 87% for TSS, and 47 to 57% for ammonia. The performance data showed that, even under adverse loading and operating conditions, the systems performed very well. The range in the system removal efficiencies between the two systems was due to the SSHF bed and not the ANFLOW unit. The ANFLOW unit for both systems had similar removal efficiencies; however, the SSHF bed exhibited higher removal efficiencies when it had a high organic loading than when it was not uniformly loaded hydraulically. The SSHF beds tended to follow first-order reaction kinetics as described in Chapter V, where the organic removal efficiency in the SSHF bed tended to be independent of the influent concentrations and dependent of the length (directly) and flow (inversely).

In both treatment systems, the SSHF beds had little or no vegetative cover. The oxygen translocation down into the SSHF bed was most likely by surface diffusion and not by the root system of vegetation, as in an artificial wetland root zone system. Because of this difference, the SSHF beds functioned similarly to a horizontal leaching field or a horizontal fixed-media filter. The performance data suggested that the SSHF bed seemed to be insensitive to or independent of the amount of oxygen diffused into the bed for BOD or COD removal. This appearance was made evident by the Haynes House data, which showed that even with an additional 6 in. of soil

cover added to the SSHF bed, the removal efficiencies for BOD and COD were comparable, within a few percentage points.

Since it is an anaerobic system, the ANFLOW unit tended to function as a pretreatment system for the SSHF bed. The wastewater flowed from one anaerobic environment to another, greatly reducing the startup time required for the SSHF bed as compared to a system in which the initial treatment was an aerobic system. Thus, no time requirement for the microorganisms to get acclimated to an anaerobic environment was assigned, because they came from an anaerobic, not an aerobic, environment.

No oxygen measurements (dissolved oxygen concentrations) were taken during this research to determine if the SSHF bed stayed anaerobic down its length or, if at some point in the bed, it became facultative or even aerobic. These types of measurements would help to explain the process by which the ammonia was being removed from the SSHF bed. Ammonia removal efficiencies were from 47 to 57% for the Haynes Home SSHF bed. Since no oxygen measurements were taken, theoretical calculations were performed to help determine what the flux rate of oxygen down into the SSHF bed should be. The results of these calculations suggested that the SSHF beds studied should remain anaerobic and that a thin oxygenated layer may exist where nitrification could have occurred.

Additional research is needed to better quantify the kinetic constants needed for the derivation of design equations for SSHF beds. It would be desirable to have not only additional long-term SSHF bed profile data but flow data corresponding to the profile data

so that the relationship between position in the SSHF bed and time can be solidified. Further research would aid in identifying the strengths and limitations of design equations.

Research is needed to verify the assumption that the SSHF bed is an anaerobically controlled system and to determine if at any time the bed transforms into a facultative zone or even an aerobic zone. Oxygen profile data down the length of the bed are needed to help determine the actual oxygen diffusion rates down into the bed and along its length. This type of information would also help to determine which mechanism removes the ammonia from the system. With these determinations, a complex design equation could be formulated to model both the BOD and COD removal in the SSHF bed, as well as the ammonia removal.

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APPENDICES

**APPENDIX A
RAW DATA**

SHAGBARK DATA

DATE	WEEK	COD (mg/l)		BOD (mg/l)	
		SAMP#1	SAMP#2	SAMP#1	SAMP#2
5-28-87	1	316	138	74	75
602	2	324	214	39	180
611	3	264	208	84	204
618	4	319	250	146	130
625	5	234	215	143	165
702	6	244	203	137	175
709	7	220	169	105	138
716	8	322	262	80	88
723	9	225	175	67	90
730	10	375	169	64	103
807	11	273	213	105	87
813	12	246	133	108	165
910	16	340	166	92	145
917	17	366	190	103	80
1001	19	311	217	85	157
1008	20	292	149	58	70
1015	21	254	186	87	204
1022	22	263	175	81	81
1029	23	265	159	74	223
1105	24	278	157	71	61
1112	25	238	104	77	24
1119	26	187	181	99	86
1204	28	250	163	96	39
1210	29	220	113	87	112
1217	30	342	113	113	107
1230	32	251	233	108	59
1-14-88	34	189	122	75	72
121	35	312	261	152	82
128	36	346	249	161	34
204	37	250	200	122	86
211	38	255	194	132	216
218	39	438	131	92	189
225	40	255	132	65	138
303	41	222	160	123	95
					132
					90
					76
					61
					44
					66
					141
					158
					40
					57
					110
					164
					65
					123
					57

SHAGBARK DATA									
DATE	WEEK	TEMP (C)		PH					
		SAMP#1	SAMP#2	SAMP#3	SAMP#1	SAMP#2	SAMP#3	SAMP#1	SAMP#2
5-28-87	1.0								
6-02-87	2.0	19.0	21.5	20.0	7.1	8.4	8.8		
6-11-87	3.0	19.6	19.9	19.8	6.9	7.0	8.3		
6-18-87	4.0	20.0	20.4	19.8	6.8	6.9			
6-25-87	5.0	20.9	21.7	20.6	7.0	7.0	7.3		
7-02-87	6.0	20.9	21.6	20.6	7.1	7.6	7.7		
7-09-87	7.0	22.3	26.5	22.8	7.0	7.9	7.8		
7-16-87	8.0	21.0	22.0	20.5	6.8	8.3	8.2		
7-23-87	9.0	21.6	24.4	22.2	6.8	8.2	7.9		
7-30-87	10.0	21.6	22.2	22.4	6.8	6.8	8.1		
8-07-87	11.0	22.7	23.1	23.0	6.9	6.8	7.4		
8-13-87	12.0	22.6	23.1	23.0	6.9	7.0	7.6		
9-10-87	16.0	20.9	21.4	21.0	6.2	6.4	6.9		
9-17-87	17.0	21.0	21.4	21.1	6.5	6.6	7.6		
10-01-87	19.0	18.5	18.6	18.2	6.7	6.9	7.7		
10-08-87	20.0	17.0	17.6	15.8		7.0	7.6		
10-15-87	21.0	16.9	16.3	16.2	7.2	7.0	7.5		
10-22-87	22.0	15.3	15.4	14.3	7.3	7.0	7.3		
10-29-87	23.0	14.6	14.8	12.8	7.0	7.2	7.8		
11-05-87	24.0	16.4	16.2	15.9	7.0	6.8	7.5		
11-12-87	25.0	14.3	14.2	12.0	4.1	4.4	5.0		
11-19-87	26.0	14.0	13.9	13.5	3.8	4.0	4.2		
12-04-87	28.0	11.5	11.4	9.1	7.3	7.0	7.5		
12-10-87	29.0	12.7	12.4	11.7	6.7	6.5	6.7		
12-17-87	30.0	9.5	11.2	7.5	7.6	7.5	7.5		
12-30-87	32.0	9.5	10.1	7.9	7.1	7.3	7.7		
1-14-88	34.0	7.1	7.2	8.2	6.8	6.7	6.7		
1-21-87	35.0	8.5	8.7	8.4	6.7	6.9	7.1		
1-28-87	36.0	8.0	8.0	7.5	6.9	7.2	7.2		
2-04-87	37.0	9.5	9.0	8.8	6.7	6.8	6.9		
2-11-87	38.0	9.6	9.4	9.4	6.6	6.8	6.8		
2-18-87	39.0	9.5	9.0	8.5	7.0	6.9	6.8		
2-25-87	40.0	8.0	7.7	7.8	6.3	6.5	6.6		
3-03-87	41.0	11.0	11.7	9.8	6.7	6.5	6.9		

SHAGBARK DATA									
DATE	WEEK	NO3/NO2		NH3			NH3-N		
		SAMP#1	SAMP#2	SAMP#3	SAMP#1	SAMP#2	SAMP#3	SAMP#1	SAMP#2
5-28-87	1.0								
6-02-87	2.0	0.0	0.0	0.0	26.8	20.7	11.5	22.1	17.0
6-11-87	3.0	0.0	0.0	0.0	29.4	16.1	13.3	24.2	13.3
6-18-87	4.0	0.0	0.0	0.0	33.6	31.6	18.4	27.7	26.0
6-25-87	5.0	0.0	0.0	0.0	34.0	34.5	24.3	28.0	28.4
7-02-87	6.0	0.0	0.0	0.0	32.9	24.1	26.7	27.1	19.9
7-09-87	7.0	0.0	0.0	0.0	24.7	28.9	24.7	20.3	23.8
7-16-87	8.0	0.0	0.0	0.0	29.9	20.5	15.7	24.6	16.8
7-23-87	9.0	0.0	0.0	0.0	26.7	21.5	16.2	22.0	17.7
7-30-87	10.0	0.0	0.0	0.0	27.6	30.6	20.1	22.8	25.2
8-07-87	11.0	0.0	0.0	0.0	25.6	21.1	24.6	21.1	17.4
8-13-87	12.0	0.0	0.0	0.0	27.1	30.1	25.1	22.3	24.8
9-10-87	16.0	0.0	0.0	0.0	19.7	15.6	16.7	16.3	12.9
9-17-87	17.0	0.0	0.0	0.0	25.9	26.9	21.8	21.3	22.2
10-01-87	19.0	0.0	0.0	0.0	24.0	28.0	22.0	19.8	23.1
10-08-87	20.0	0.0	0.0	0.0	23.4	25.5	26.0	19.3	21.0
10-15-87	21.0	0.0	0.0	0.0	20.7	15.9	21.6	17.0	13.1
10-22-87	22.0	0.0	0.0	0.0	28.9	28.4	23.1	23.8	23.4
10-29-87	23.0	0.0	0.0	0.0	24.0	29.4	21.6	19.8	24.2
11-05-87	24.0	0.0	0.0	0.0	24.8	26.2	22.8	20.4	21.6
11-12-87	25.0	0.9	0.1	0.5	20.6	21.1	20.1	17.0	17.4
11-19-87	26.0	0.1	0.2	0.2	21.4	21.4	16.3	17.7	17.7
12-04-87	28.0	0.2	0.2	0.2	23.5	18.4	18.4	19.3	15.1
12-10-87	29.0	0.2	0.2	0.2	27.6	25.0	27.0	22.7	20.6
12-17-87	30.0	0.3	0.3	0.3	26.0	29.0	24.0	21.4	23.9
12-30-87	32.0	0.3	0.3	0.3	25.0	29.0	21.5	20.6	23.9
1-14-88	34.0	0.3	0.3	0.3	18.0	19.5	18.5	14.8	16.0
1-21-87	35.0	0.3	0.3	0.3	23.0	21.0	21.0	18.9	17.3
1-28-87	36.0	0.3	0.3	0.3	20.0	21.0	20.5	16.5	17.3
2-04-87	37.0	0.3	0.3	0.3	17.0	18.0	15.5	14.0	14.8
2-11-87	38.0	0.3	0.3	0.3	13.0	18.5	15.5	10.7	15.2
2-18-87	39.0	0.1	0.1	0.1	11.6	12.1	13.4	9.5	10.0
2-25-87	40.0	0.0	0.0	0.0	16.4	10.9	13.4	13.5	9.0
3-03-87	41.0	0.2	0.4	0.7	13.6	16.7	13.6	11.2	13.4

DATE	WEEK	TSS (mg/l)		FECAL COL		SAMP#3	SAMP#2	SAMP#1	SAMP#2	SAMP#3
		SAMP#1	SAMP#2	SAMP#1	SAMP#2					
5-28-87		1	89	69	111					
602		2	27	26						
611		3	25	17	3					
618		4	51	21	20					
625		5	30	23	16			500000	7900	1000
702		6	47	47	25			600	1300	16
709		7	49	81	12			17200	3100	700
716		8	61	193	24			5000	3800	46
723		9	63	129	11			2000	200	2
730		10	51	10	23			1400	300	15
807		11	39	14	5			90000	2600	1100
813		12	41	13	10			7200	1300	106
910		16	73	10	8			22000	2700	37
917		17	105	18	13			2200	600	300
1001		19	30	13	4					
1008		20	56	13	7			570000	190000	8000
1015		21	24	11	4			400	63	100
1022		22	32	12	5			200000	5300	4900
1029		23	25	8	4			1630000	160000	500
1105		24	26	11	3			15000000	60000	1600
1112		25	25	12	3			2000000	20000	1200
1119		26	29	17	12			50000	5000	1900
1204		28	33	6	1			50000	2100	100
1210		29	47	12	11			2400	500	300
1217		30	17	26	3			1	0	0
1230		32	27	19	10			16	0	0
1-14-88		34	15	6	1			0	16	6
121		35	14	9	8			1200000	20000	11
128		36	83	17	14			4200	3400	1500000
204		37	45	23	10			71	53	9
211		38	37	6	8			8600	7500	6600
218		39	263	13	5			4500	2000	1100

HAYNES HOUSE WETLANDS DATA
FILE NAME:HAYNES.WK1

DATE	SAMPLE	TEMP (C)	PH	TSS (mg/l)	BOD (mg/l)	COD (mg/l)	NH3 (mg/l)	NH3-N (mg/l)
05-10-88	INFL	21.6	6.2	111.5	701.1	956.6	8.52	7.01
	I/E	19.7	6.8	56	297.8	562.4	40.84	33.64
	15	25.1	6.8	2415	178.9	136.2	4.76	3.92
	30	25.6	6.9	75.3	128.8	144.9	4.28	3.52
	45	27	6.7	67.6	115.9	52.2	5.24	4.32
	60/OUT	18.6	6.7	25.3	118.1	81.2	3.7	3.05
05-20-88	INFL	21.3	5.9	51	502.9	707.7	31.03	25.55
	I/E	20.3	6.7	41.5	189.7	292.3	13.74	11.31
	15	15.9	6.7	38	199.2	200	20.16	16.6
	30	16.6	6.8	31.5	180.7	226.9	24.61	20.26
	45	NO SAMPLE						
	60/OUT	19.9	6.7	36	51.3	73.1	6.86	5.65
06-10-88	INFL	21.3	6.9	187.8	273.7	523.6	33.5	27.5
	I/E	20.4	6.6	39.9	369.1	429.1	28.8	23.7
	15	17.4	6.8	56.4	147.8	261.8	17.5	14.4
	30	15.7	6.7	23.5	90.9	29.1	12.8	10.6
	45	17.5	6.7	1.66	38.7	43.6	12.8	10.6
	60/OUT	17.8	6.8	4.4	43.5	87.27	19.1	15.7
06-17-88	INFL	23.7	6.4	266.4	713.7	645.4	44.7	38.8
	I/E	24.3	6.5	63.5	268.1	528.4	43.7	36
	15	20	6.55	73.6	329.9	326.2	44.7	36.8
	30	19.6	6.7	54.3	310.2	234	40.2	33.1
	45	20.5	6.6	37.2	107.9	83.3	64.1	52.7
	60/OUT	22	6.9	14.4	86.1	97.5	20.4	16.8
06-24-88	INFL	19.5	6.7	153.6	BAD DATA	379.3	22.07	18.17
	I/E	20	6.6	70.2	BAD DATA	231.1	22.07	18.17
	15	20.4	6.5	51.4	BAD DATA	201.5	34.84	28.69
	30	20.2	6.5	39.2	130.4	278.5	35.84	29.52
	45	22	6.6	92.2	113.8	240	34.82	28.68
	60/OUT	22.9	6.6	36.1	141.8	234	37.88	31.2
07-01-88	INFL	19.3	6.2	265	321.3	353.3	22	18.2
	I/E	19	6.3	66	192.1	293.3	22	18.2
	15	18.2	6.4	55	78.1	153	35.8	29.5
	30	19.3	6.5	18	154	213.3	34.8	28.7
	45	20.1	6.6	42.2	BAD DATA	235	34.8	28.7
	60/OUT		6.7	17.9	54.6	230	37.9	31.2

HAYNES HOUSE WETLANDS DATA
FILE NAME:HAYNES.WK1

DATE	SAMPLE	TEMP (C)	PH	TSS (mg/l)	BOD (mg/l)	COD (mg/l)	NH3 (mg/l)	NH3-N (mg/l)
07-08-88	INFL	22.4	6.7			316.6	256	18.62
	I/E	22.6	6.7	58		97	144	15.33
	15	19	6.6	20				12.27
	30	19	6.6	42		128.1	216	17.02
	45	19.9	6.7	60		113.7	208	14.89
07-15-88	60/OUT	15.8	6.6	54.6		154.5	284	18.4
			6.6	31.5		178.6	240	24.09
	INFL	20.9	6.4			617.5	654.6	18.97
	I/E	21.9	6.4	247		347.9	454.6	15.62
	15	19.9	6.5	53		287	181.8	15.03
07-22-88	30	21.7	6.6	17		230.9	73.2	7.91
	45	22.1	6.7	6		200.5	60	5.42
	60/OUT	22.9	6.7	12.1		119.7	69.1	6.11
	INFL	19.9	6.4	168		515.3	626.7	31.03
	I/E	20.5	6.4	68		362.6	649.9	31.03
07-29-88	15	19	6.5	65		180.6	219.98	20.89
	30	20.7	6.5	46		189.1	166.5	18.86
	45	21.2	6.6	29.3		85.5	73.3	9.53
	60/OUT	22.7	6.6	20.7		148.3	106.7	13.18
	INFL	23.2	6.2	115		482.8	890.5	55.55
08-05-88	I/E	23.5	6.3	73		233.3	598.3	51.82
	15	22	6.35	129		20.3	166.95	39.79
	30	21.7	6.45	110		37.97	139.13	28.19
	45	23	6.8	35.7		48.85	104.4	18.65
	60/OUT	24.1	6.79	17.13		32.4	257.4	18.24
08-12-88	INFL	24.4	5.5	76		291.2	728.6	
	I/E	24.2	5.6	15		240.6	714.3	
	15	22.7	5.65	122		188.2	417.85	
	30	22.5	5.7	28		71.12	167.86	
	45	23.4	5.8	44.3		19.8	44.6	
08-12-88	60/OUT	24.4	5.95	14.9		52.6	239.3	
	INFL	24.5	5.9	285		74.66	560	52.3
	I/E	24.4	6	45		115	313.3	43.1
	15	23.2	6	87		120.2	326.7	50.35
	30	22.4	6.1	79		46.2	233.4	51.2
08-12-88	45	23.2	6.2	173.6		--	93.3	35.4
	60/OUT	24.45	6.25	12.86		2.4	32.64	26.9
							34.2	28.2

HAYNES HOUSE WETLANDS DATA
FILE NAME:HAYNES.WK1

DATE	SAMPLE	TEMP (C)	PH	TSS (mg/l)	BOD (mg/l)	COD (mg/l)	NH3 (mg/l)	NH3-N (mg/l)
08-19-88	INFL	21	6	186		448.3	649.1	45.12
	I/E	23.5	6.2	193		112.9	347.2	53.2
	15	21.3	6.3	132		134.67	166.04	57.74
	30	22	6.4	157		9.92	128.3	59.76
	45	23.3	6.5	92.85		222.6	83.02	36.53
	60/OUT	24.8	6.6	104.3		17.68	45.28	--
08-26-88	INFL	21.8	6.6	539.5		689.5	407.3	48.65
	I/E	21.2	6.5	40		203.8	254.5	57.24
	15	19.4	6.5	55		131.2	327.3	38.55
	30	19.3	6.6	10		37.66	29.09	16.84
	45	19.6	6.6	32.14		7.3	29.09	10.77
	60/OUT	21.2	6.7	14.99		12.78	87.3	27.44
09-23-88	INFL	21.7	6.6	94		254.5	326.5	53.2
	I/E	20.5	6.5	55		238.2	232.65	57.74
	15	19.5	6.55	22		130	81.63	53.2
	30	19	6.5	7		21.1	57.14	48.35
	45	19.4	6.6	5.7		34.13	28.6	16.84
	60/OUT	20.1	6.65	7.14		83.4	--	26.43
09-30-88	INFL	15.8	7.2	120		--	272	56.04
	I/E	16.3	7	70		86.7	186	62.08
	15	19.1	7.1	29		--	95.99	60.07
	30	16	7.2	42		--	116.13	38.93
	45	17.2	7.3	20.7		--	212.9	17.79
	60/OUT	20.4	7.4	5.7		62.7	17.3	43.96

APPENDIX B
STATISTICAL DATA

Haynes House BOD Data
12" Soil Cover
Statistics

EXPONENTIAL $Y = A * EXP(B*X)$

$Y = 174.545 * EXP(-.0298 * X)$

UNADJUSTED R² = .3245
CORRELATION COEFFICIENT = -0.5697

STD ERROR OF ESTIMATE = 80.3618
VARIANCE OF ESTIMATE = 6458.017

ANOVA FOR PREDICTION F = 20.3617
CHANCE PROBABILITY P = .0002

DEGREES OF FREEDOM = 1 AND 38

WITH X = POS AND Y = BOD

6 PAIRS WITH MISSING VALUES REMOVED

Haynes House BOD Data
6" Soil Cover
Statistics

EXPONENTIAL $Y = A * EXP(B*X)$

$Y = 287.8535 * EXP(-.0244 * X)$

UNADJUSTED R² = .6333
CORRELATION COEFFICIENT = -0.7958

STD ERROR OF ESTIMATE = 63.0911
VARIANCE OF ESTIMATE = 3980.488

ANOVA FOR PREDICTION F = 26.9626
CHANCE PROBABILITY P = .0002

DEGREES OF FREEDOM = 1 AND 17

WITH X = POS AND Y = BOD

1 PAIRS WITH MISSING VALUES REMOVED

Haynes House COD Data
12" Soil Cover
Statistics

EXPONENTIAL $Y = A * \text{EXP}(B * X)$

$Y = 322.9001 * \text{EXP}(-.0315 * X)$

UNADJUSTED R^2 = .5255
CORRELATION COEFFICIENT = -.7249

STD ERROR OF ESTIMATE = 117.7183
VARIANCE OF ESTIMATE = 13857.6

ANOVA FOR PREDICTION F = 29.6696
CHANCE PROBABILITY P = <.0001

DEGREES OF FREEDOM = 1 AND 40

WITH X = POS AND Y = COD

4 PAIRS WITH MISSING VALUES REMOVED

Haynes House COD Data
6" Soil Cover
Statistics

EXPONENTIAL $Y = A * \text{EXP}(B * X)$

$Y = 358.6752 * \text{EXP}(-.0302 * X)$

UNADJUSTED R^2 = .5992
CORRELATION COEFFICIENT = -.7741

STD ERROR OF ESTIMATE = 105.3831
VARIANCE OF ESTIMATE = 11105.6

ANOVA FOR PREDICTION F = 30.633
CHANCE PROBABILITY P = .0001

DEGREES OF FREEDOM = 1 AND 17

WITH X = POS AND Y = COD

1 PAIRS WITH MISSING VALUES REMOVED

APPENDIX C
CALCULATIONS

OXYGEN CALCULATIONS

REF: ChemoDynamics (45) - CHAPTER 5

$$D_{A3} = \frac{\alpha D_{A2} \epsilon}{T_h}$$

D_{A3} - effective Diffusion Coeff. (EQU. 5.1-41)

D_{A2} - molecular Diffusivity

T_h - tortuosity factor

ϵ - Porosity

A - Oxygen

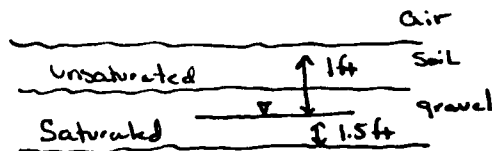
$$\alpha_{A2} \text{ in Air} = .178 \text{ cm}^2/\text{s}, \quad \alpha_{A2} \text{ in Water} = 1.8 \times 10^{-5} \text{ cm}^2/\text{s} \quad (\text{TABLES C.8 + C.9})$$

Assume a porosity of 30%, $T_h = \sqrt{3}$ if the flow path is 3 dimensional (pg. 246)

$$D_{A3} \text{ in Air} = \frac{0.178 (.3)}{\sqrt{3}} = 3.1 \times 10^{-2} \text{ cm}^2/\text{s}$$

$$D_{A3} \text{ in Water} = \frac{1.8 \times 10^{-5} (.3)}{\sqrt{3}} = 3.1 \times 10^{-6} \text{ cm}^2/\text{s}$$

SSHFBED



$$N_A = \frac{D_{A3} (P_{A1}^* - P_{A1})}{h} \quad (\text{from EQU. 5.1-40})$$

N_A - Flux

h - distance

P_{A1}^* - mass concentration in pore spaces, at the Surface

P_{A1} - at the Bottom $\cong \theta$

I. $N_{A \text{ soil}}$ = Flux of O_2 in unsaturated zone

$$N_{A \text{ soil}} = \frac{3.1 \times 10^{-2} \text{ cm}^2/\text{s}}{1 \text{ ft} \left(\frac{30.48 \text{ cm}}{\text{ft}} \right)} \left(\frac{1.18 \text{ g}}{1000 \text{ cm}^3} (.21) - \theta \right)$$

$$P_{A1}^* = 1.18 (.21) \text{ g/l} - \text{Table D.1}$$

$$P_{A1}^* = 9 \text{ mg/l} - \text{Table C.2}$$

$$N_{A \text{ soil}} = 2.52 \times 10^{-7} \text{ g } O_2 / \text{cm}^2 \cdot \text{s}$$

II. η_{ws} = Flux of O_2 in Saturated Zone

$$\eta_{ws} = \frac{3.1 \times 10^{-6} \text{ cm}^2 \cdot \text{s}}{(1.5/2)(30.48 \text{ cm}_{ft})} \left(\frac{9 \text{ mg}}{1000 \text{ cm}^3} - 0 \right) \frac{9}{1000 \text{ mg}}$$

$$\eta_{ws} = 1.22 \times 10^{-12} \text{ g } O_2 / \text{cm}^2 \cdot \text{s} \Rightarrow \text{Flux of } O_2 \text{ in Saturated Zone at average Depth}$$

$$\text{BOD} = 1.22 \times 10^{-12} \frac{\text{g } O_2}{\text{cm}^2 \cdot \text{s}} (1 \text{ cm}^2)(5 \text{ d})(3600 \text{ s/hr})(24 \text{ hr/d})$$

$$\text{BOD} = 5.27 \times 10^{-7} \text{ g of } O_2$$

$$\text{BOD mg/L} = 5.27 \times 10^{-7} \left(\frac{1000 \text{ mg}}{\text{g}} \right) \left(\frac{1}{(1 \text{ cm}^2)(1.5/2)(30.48)} \right) \frac{1000 \text{ cm}^3}{\text{L}}$$

$$\text{BOD mg/L} = 1.15 \times 10^{-2} \text{ mg/L} \Rightarrow \text{BOD that could be supported at the average Flux rate.}$$

III $\eta_{ws} \Rightarrow$ Max theoretical Flux at Water Surface

Assume: $\epsilon = 1$ $\tau_h = \sqrt{1}$ Depth = 1 cm

$$D_{ws} = \frac{1.8 \times 10^{-5} (1)}{\sqrt{1}} = 1.8 \times 10^{-5} \text{ cm}^2/\text{s}$$

$$\eta_{ws} = \frac{1.8 \times 10^{-5}}{1 \text{ cm}} \left(\frac{9}{1000 \text{ mg}} - 0 \right) \left(\frac{9}{1000 \text{ mg}} \right)$$

$$\eta_{ws} = 1.62 \times 10^{-10} \text{ g } O_2 / \text{cm}^2 \cdot \text{s}$$

$$\text{BOD} = 1.62 \times 10^{-10} (1 \text{ cm}^2)(5 \text{ d})(3600 \text{ s/hr})(24 \text{ hr/d})$$

$$\text{BOD} = 7.0 \times 10^{-5} \text{ g } O_2$$

$$\text{BOD} = 7.0 \times 10^{-5} \text{ g } O_2 \left(\frac{1000 \text{ mg}}{\text{g}} \right) \left(\frac{1}{1 \text{ cm}^2} \right) \left(\frac{1000 \text{ cm}^3}{\text{L}} \right)$$

$$\text{BOD} = 69.98 \text{ mg/L} \Rightarrow \text{BOD that could be supported at a 1 cm depth in the Saturated Zone with a Porosity} = \text{to } 1 \text{ and a tortuosity} = \text{to } 1.$$

IV C $\epsilon = .6$ $\tau_h = \sqrt{3}$ D = 1 cm

$$D_A = 6.24 \times 10^{-6} \text{ cm}^2/\text{s}$$

$$\eta_{ws} = 5.61 \times 10^{-11} \text{ g } O_2 / \text{cm}^2 \cdot \text{s}$$

$$\text{BOD} = 24.24 \text{ mg/L} \quad 125$$

WETLAND CALCULATIONS

HAYNES DATA	BOD (mg/L)		
	<u>Influent</u>	<u>Influent to SSHF Bed</u>	<u>Effluent</u>
6" period	548	281	75
12" period	422	232	59

$$Q = 4 \text{ people} \times 60 \text{ gpd} = 240 \text{ gpd} \rightarrow 32 \text{ ft}^3/\text{d} \rightarrow 0.9087 \text{ m}^3/\text{d} \\ \rightarrow 1.0518 \text{ E}^{-5} \text{ m}^3/\text{s}$$

$$A = 5.2 Q_d [Lw C_o - Lw C_e] \quad \text{equ. 2 Chapter II}$$

I. 6' DATA

$$\text{w/o ANFLOW} \quad A = 5.2 (.9087) [Lw 548 - Lw 75] \\ A = 9.3975 \text{ m}^2$$

$$\text{If Depth} = .6 \text{ m} \quad \text{then} \quad L = 15.66 \text{ m or } 52 \text{ ft.}$$

$$\text{w/ANFLOW} \quad A = 5.2 (.9087) [Lw 281 - Lw 75] \\ A = 6.24 \text{ m}^2$$

$$\text{If Depth} = .6 \text{ m} \quad \text{then} \quad L = 10.4 \text{ m or } 34 \text{ ft.}$$

$$A_c - \text{Cross sectional Area} = Q_s / K_f dH/ds \quad \text{where } K_f = 3 \text{ E}^{-5}$$

(Equ. 3 Chapter 2)

$$dH/ds = 2\% \text{ to } 8\% \\ \text{- use } 2\%$$

$$A_c = 1.0518 \text{ E}^{-5} / 3 \times 10^{-5} \cdot .02$$

$$A_c = 17.53 \text{ m}^2 \quad \text{If } D = .6 \text{ m} \quad \text{then} \quad L = 29.2 \text{ m or } 96 \text{ ft.}$$

II. 12" DATA

$$\text{w/o ANFlow} \quad A = 5.2 (.9087) (Lw 422 - Lw 59)$$

$$A = 9.29 \text{ m}^2$$

$$\text{If } D = .6 \text{ m then } L = 15.49 \text{ m or } 51 \text{ ft}$$

$$\text{w/ANFlow} \quad A = 5.2 (.9087) (Lw 232 - Lw 59)$$

$$A = 6.469 \text{ m}^2$$

$$\text{If } D = .6 \text{ m then } L = 10.78 \text{ m or } 35 \text{ ft}$$

$$A_c = 1.0518 \text{ E}^{-5} / 3 \text{ E}^{-5} (.02)$$

$$A_c = 17.53 \text{ m} \quad \text{if } D = .6 \text{ m then } L = 29.2 \text{ m or } 96 \text{ ft.}$$

∴ Wetland Size For 6" Data w/o ANFlow is:

$$52 \text{ ft} \times 96 \text{ ft} \times 2 \text{ ft deep}$$

Wetland Size for 12" Data w/o ANFlow is:

$$51 \text{ ft} \times 96 \text{ ft} \times 2 \text{ ft deep}$$

VITA

Christine M. Rudolph was born in Nice, France, on October 24, 1961. Her father and family were living there while he served in the United States Navy on the *USS Springfield*. She attended elementary schools in Virginia and California. She began high school in Guam and graduated from Marlboro High School, Marlboro, New Jersey, in June 1979. The following year she entered the University of Tennessee and in June 1984 received the degree of Bachelor of Science in Agriculture in Animal Science. She continued her education in agriculture and received a degree of Bachelor of Science in Agricultural Engineering in 1986. In 1987, she entered graduate school in the College of Engineering in Environmental Engineering. She received a Master of Science degree in Environmental Engineering in December 1992. She was married on June 6, 1992 to Michael W. Goddard. Her married legal name is Christine Marie Rudolph Goddard.

She was employed by James M. Montgomery Engineers in Pasadena, California, while working on her degree, from January 1989 to June 1991. She is currently employed by Martin Marietta Energy Systems at Oak Ridge National Laboratories as a process engineer.