Performance Evaluation of the Town of Monterey Wastewater Treatment Plant Utilizing Subsurface Flow Constructed Wetlands

by

Brandon D. Kiracofe

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Advisory Committee:

Dr. John T. Novak, Chair

Dr. Clifford W. Randall

Dr. Gregory D. Boardman

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Brandon Kiracofe
The Charles E. Via, Jr. Department of Civil and Environmental Engineering

(ABSTRACT)

Field tests were conducted and historical operating data were evaluated to assess the performance of the Monterey WWTP utilizing subsurface flow (SF) constructed wetlands. Previous work with SF wetlands has demonstrated adequate, but variable removal of organic matter, suspended solids, and nitrogen. Few research studies have observed the generation of compounds in the wetlands that affect other treatment processes, specifically reduced compounds that contribute to the chlorine demand. This study attempts not only to distinguish the factors leading to the inadequate performance of the SF wetlands in removing organic matter and nitrogen, but also to identify the cause of the frequent occurrences of a nondetectable chlorine residual in the chlorine contact tank at the Monterey WWTP. Collection and analysis of historical operating data from January 1998 to May 2000 revealed a constantly decreasing removal of carbonaceous biochemical oxygen demand ($CBOD_5$) by the SF wetlands and a poor removal of ammonia-N throughout the system. The decreasing removal of $CBOD_5$ appeared to be caused by clogging of the wetland bed media by accumulated solids. The inability to remove the accumulated solids by pumping was shown. Analysis of field data also showed that the SF wetlands removed 88% of the influent TSS and 71% of the influent $CBOD_5$, while experiencing a 18% increase in ammonia-N. Bisulfide produced in the anaerobic wetland beds accounted for 95% of the chlorine lost in contact tank. The variable production of sulfide is the cause of the frequent nondetectable chlorine concentrations observed. The results of this study suggest that chemical costs of chlorine and sulfur dioxide may be greatly reduced if bisulfide can be removed before chlorination. Also, the use of large rocks as media in SF wetland beds may significantly reduce the physical and biological removal of organic matter.

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This master's thesis is dedicated to my parents, Roger and Donna Kiracofe for their amazing support in everything I do.

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Literature Review

Introduction

Many “natural systems” are being given consideration for the purpose of wastewater treatment and water pollution control. The interest in natural systems is based on the conservation of resources associated with these systems as opposed to the conventional wastewater treatment processes which are intensive in regards to the use of both energy and chemicals. Wetlands are one of the many types of natural systems that can be used for treatment and pollution control. According to U.S. EPA (1993), a constructed wetland is defined as “a wetland specifically constructed for the purpose of pollution control and waste management, at a location other than existing natural wetlands.” Subsurface Flow (SF) wetlands are a commonly used type of constructed wetland. SF wetlands are characterized by the growth of emergent plants using soil, gravel, or rock as a growth substrate in a lined channel or bed. Within the bed, facultative microbes attach to the media and plant roots, thereby contacting the wastewater that flows horizontally through the bed while remaining below the surface of the media (Kadlec and Knight, 1996).

Constructed wetlands offer the treatment benefits of natural wetlands in a more controlled environment. In many regards however, constructed wetlands are difficult to maintain and control. Many factors, such as the availability of oxygen, the clogging of wetland media, and the inability to adequately remove solids from the wetland by pumping lead to this lack of control and make the prediction of long-term treatment efficiency of SF wetlands difficult. Because of this lack of control, SF wetlands may not always be capable of producing an effluent that meets all discharge permit requirements. This makes the wetlands an unattractive treatment option in certain situations (Fisher, 1990).

As with any treatment process there are several advantages and disadvantages that must be considered before choosing to utilize SF wetlands. Some of the advantages suggested by Brix (1987) are:

1. Minimal energy requirements
2. Minimal maintenance requirements
3. Less-complex operation than conventional treatment processes
4. Minimal working expenses

The advantages listed above are valid when the wetland system performs as designed. When working properly, a wetland allows more of a stand-by approach to operation. The treatment processes within the system are self-sustainable requiring little input of energy, chemicals, and operator maintenance. However, there are times when wetland systems do not achieve the designated treatment goals. The reasons for the inadequate performance are numerous with many being directly related to the disadvantages given by Hilton (1993) which are listed below:

1. Imprecise design and operating criteria
2. Plugging potential
3. Biological and hydrological complexity
4. Large land area requirements
5. Possible problems with pests

Before the disadvantages can be specifically discussed, the design theory of SF wetlands which leads to these disadvantages must be presented.

Root Zone Method

The root zone method (RZM) is generally accepted and used as the basis for the design of SF wetland systems. A typical horizontal flow SF wetland is shown in Figure 1, and a plan view of the system is shown in Figure 2. Wastewater is distributed uniformly across the wetland bed by the inlet zone which is composed of crushed rock. Wastewater then flows horizontally through the media filled channel where it is treated by physical, biological, and chemical processes. These processes are said to take place in the rhizosphere, which is composed of the media, the plant roots, the plant rhizomes, and the associated microbial communities (Conley et al., 1991). After treatment, the wastewater is collected in the outlet zone and directed to further treatment processes or to discharge into a waterway. Each component within the root zone bed plays a role in the system, however there are varying views as to the significance of those roles to the overall functioning of the treatment configuration. The following sections discuss the major components of the root zone bed.

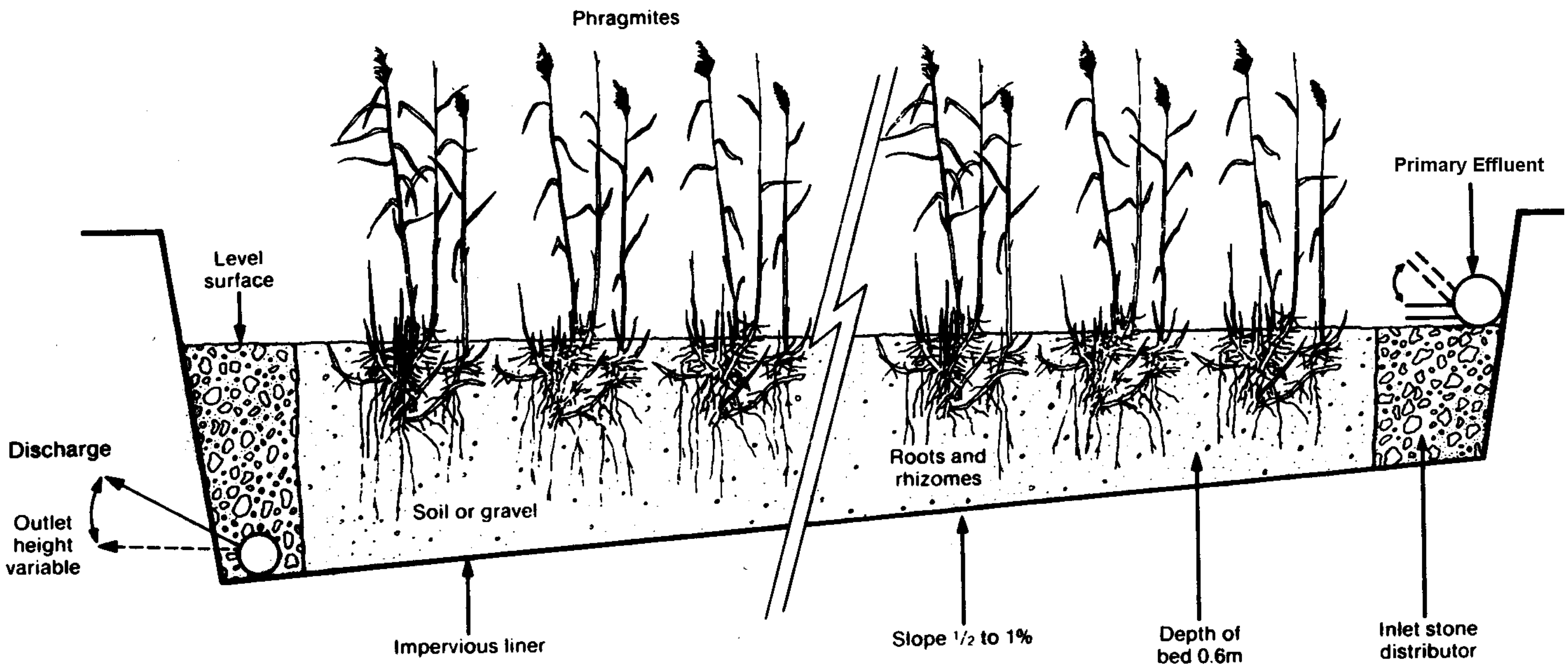

Figure 1 Typical Configuration of a Horizontal Flow Reed Bed System. Taken from Cooper, 1993.

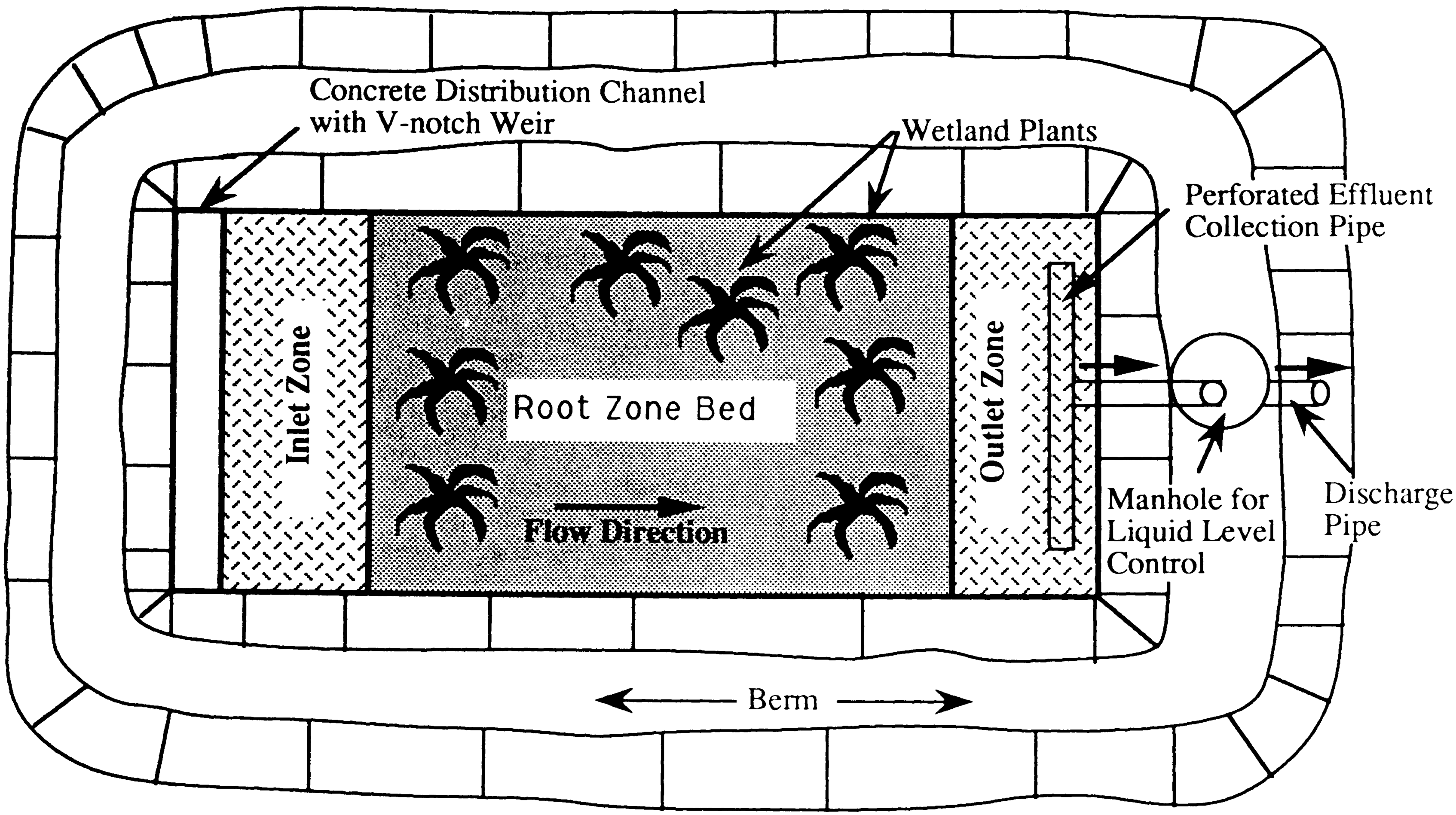

Figure 2. Plan view of typical horizontal flow reed bed system. Taken from Conley et al., 1991

Media

Media in the root zone bed serves many different purposes. The first role of the media is physical treatment of the wastewater (Wood, 1990). Filtration and sedimentation of suspended solids and pathogens occurs along with the sorption of phosphorus, and dissolved organics (Lance et al., 1976). Smaller media, such as sand, are more effective in sorption and filtration than gravel or rocks because the smaller media contain smaller pore sizes and larger surface areas (Conley et al., 1991). The media also provides a stable surface area for the attachment of microbial biofilms which perform biological treatment of the wastewater passing thorough the root zone bed. SF wetlands are commonly referred to as attached growth biofilters (Wood, 1995). The third function of the media is the solid support it gives for wetland plant growth (Wood, 1990). In most cases a layer of small diameter material is placed at the top of the bed to allow for better plant root establishment. It has been shown by Geller et al. (1990) that gravel, rocks, or soils containing sharp-edged material can inhibit prolific plant growth. Plants that usually grow in soils may not be able to develop their root networks properly in substratum that has large void spaces (Reed and Brown, 1992, Chambers and McComb, 1994).

Wetland beds have been constructed of many different types of materials. As stated earlier, when considering the appropriate media to use, the available surface area for the attachment of the microbial population must be considered. Also of importance is the selection of a media that has a hydraulic conductivity which allows the influent water to remain below the surface. The characteristics of typical media used in SF wetlands are shown in Table 1. Without optimizing both factors, there will be a reduction in the treatment efficiency of the system due to a lack of contact tank time between the wastewater and biofilm.

A decrease in the contact time between the wastewater and biofilm can be caused by surface flow conditions. Surface flow results from the clogging of bed media, which is a major cause of failure in SF wetland systems (Conley et al., 1991). Often times the media can not handle the organic loading that is introduced to the system. Table 2 shows organic loading rates and surface flow conditions for several facilities. It is observed that

Table 1 Characteristics of Typical Media for SF Wetlands

Type	Effective Size, D_{10}, (mm)	Porosity (%)	Hydraulic Conductivity ($m^3/m^2/d$)
Coarse Sand	2	32	1000
Gravelly Sand	8	35	5000
Fine Gravel	16	38	7500
Medium Gravel	32	40	10000
Coarse Rock	128	45	100000

From US EPA, 1993

Table 2 Subsurface Flow Wetlands: Organic Loading (BOD_5) on Entry Zone

Location	Cross-Section ($kg/d/m^2$)	Surface Flow?
Benton, KY	8	Yes
Great Britian (typ)	5	Yes
Denham Springs, LA	4	Yes
Haughton, LA	1.6	Yes
Mayo, MD	0.5	Yes
Pearlington, MS	0.2	No
Alexandria, LA	<0.2	No
Mesquite, NV	0.1	No
Bear Creek, AL	0.02	No

Data from Lekven et al., 1993

at an organic loading rate less than 0.2 $kg/d/m^2$, surface flow did not occur. Other factors such as solids loading and vegetative activity can change the characteristics of the media being used. In some instances larger media are used in order to allow a larger hydraulic loading to be applied to the wetland bed. By utilizing the larger media it is thought that more wastewater can be passed through the system without development of surface flow. However, as the size of the rock increases and the porosity decreases, wetland areas must be increased to provide the contact time between the wastewater and the microbial communities necessary for adequate biodegradation to occur (Freeman, Jr., 1993). In certain cases the wetland areas were not increased to allow for this reduced contact time and this resulted in poor treatment performance.

Plants

The plants used in wetland systems are known as emergent hydrophytes and macrophytes. The major portion of these plants (leaves and flowers) emerge above the media surface and are exposed to the air, while their roots and rhizomes are submerged beneath the water and media (Kadlec and Knight, 1996). Three of the commonly used plant species in SF wetlands are bulrush (Scirpus), reeds (Phragmites), and cattails (Typha). The extensive rooting structures of these species make viable options for wastewater treatment. Flowering plants, such as the Yellow Flag Iris, are also utilized to lesser extent (US EPA, 1993). It has been shown that Iris, sp. did not grow as vigorously as other plant types under simulated SF wetland conditions (Neralla et al., 1999).

Several of the plant functions are well established and several others are highly debated. One role that is beyond debate is the aesthetic appeal that wetland plants provide by covering the wetland bed and controlling odors. The wetland plants provide a habitat to many animals, including small mammals and birds. The plant cover also limits the amount of ponding water on the bed surface that serves as breeding environments for nuisance insects such as gnats and mosquitoes (Wood, 1995). Plant roots and rhizomes provide surfaces for microbial growth and also aid in the filtration of solids (Wood, 1990).

However, the other plant function in SF systems have led to many problems in both design and operation. A major premise of the root-zone method is that the wetland plants are able to provide oxygen to the heterotrophic bacteria in the rhizosphere thereby allowing aerobic degradation of organic matter and nitrification to occur (Brix, 1987). It is not debatable that oxygen is transferred from the aboveground parts of the plants through airways to the roots and rhizomes. Like other aerobic organisms, plants require oxygen for respiration, growth, and protection from phytotoxins in the root zone (Good and Patrick, 1987). Without oxygen the wetland plants would not be able to survive and grow. The question arises as to whether the amount of oxygen provided is in excess of the amount required by the plants. It was proposed that aerated microzones are developed around the roots and rhizomes by the leaking of oxygen through these structures (Brix, 1987). These oxidized areas in an otherwise anaerobic environment provide conditions in which the aerobic biological transformations occur. Many design approaches are based on the fact that significant aerobic biodegradation of organic wastes and nitrification take place in these microzones.

It was stated by Hiley et al., (1995) that the only situations in which plant roots and rhizomes are likely to leak any significant amount of oxygen into the rhizosphere are ones in which the oxygen demand is relatively low. It is very rare to encounter low oxygen demand conditions in SF wetlands treating wastewater, therefore from a quantitative view, the amount of oxygen provided to the surrounding media by roots and rhizomes is minimal. The respiration and growth of the wetland plants appear to require almost all of the oxygen transported to the root zone (Kadlec and Knight, 1996). Expectations that saturated organic-rich sediments can be sufficiently aerated by macrophytes are not realistic (Wetzel, 1993). Even if plants were able to leak a sufficient amount of oxygen, in most cases the roots and rhizomes do not extend throughout the entire depth of the wetland bed. Typical depths of SF wetland beds range from 0.3 m to 0.6 m, with some reaching depths of 0.84 m (US EPA, 1993). Regardless of the plant species, roots on operational systems seldom reach below 0.3 m (Reed et al., 1992). In a study performed by Parr (1990), rhizome growth was discovered to be limited to the top 0.2 m of the wetland bed.

The oxygen provided to the wetland is taken up through the surface of the water (Hiley et al, 1995), and is dominated by air-water-media interfacial transfer (Kadlec and Knight, 1996). The implication of this is that only the very top portions of the wetland bed experience aerobic conditions. The resulting anaerobic conditions throughout the rest of the root zone bed and their impact on treatment performance will be discussed in a later section.

The second questionable role of plants in SF wetlands is their ability to increase or stabilize the hydraulic conductivity of the media (Brix, 1987). This is of particular importance when utilizing a small sized media such as soil. By disturbing and loosening the soil, the growth of plant roots and rhizomes will increase the porosity of a soil thereby allowing less hindered flow through the rhizosphere (Reed et al, 1988). Kickuth proposed that regardless of the initial porosity of the soil, the root and rhizome growth would within two to five years increase the hydraulic conductivity to that of coarse sand (Brix, 1987). The design of SF wetlands based on an assumption of increased hydraulic conductivity can lead to severe problems of surface flow conditions. The failure of roots and rhizomes to increase hydraulic conductivity was demonstrated in the soil-based SF wetlands in Austria, Denmark, and the United Kingdom (Schierup et al., 1990).

Microorganisms

A large consortium of facultative microbes lives attached or associated with the media and plant roots. Although much of the treatment that is expected from SF wetlands is based on microbial degradation, a lack of knowledge is evident regarding the exact mechanisms that account for these processes. Based on the discussion earlier dealing with the limited availability of oxygen, aerobic respiration is limited to the upper portions of the wetland bed, and anaerobic metabolism dominates the remaining portion of the bed. Because nitrate is not readily oxidized in largely anaerobic systems, the presence of anoxic conditions and the microbes thriving under anoxic conditions is minimal.

Removal Processes

The removal processes for each constituent of concern that are commonly thought to occur in macrophyte-based wetlands are given in Table 3. In many systems utilizing SF wetlands, a single treatment cell is expected to accomplish many treatment goals. The

Table 3 Removal Mechanisms in Macrophyte-Based Wastewater Treatment Systems

Wastewater Constituent	Removal Mechanisms
Suspended Solids	Sedimentation/filtration
BOD	Microbial Degradation (aerobic and anaerobic) Sedimentation (accumulation of organic matter/sludge on the sediment surface)
Nitrogen	Ammonification followed by microbial nitrification and denitrification Plant Uptake Ammonia Volatilization
Phosphorus	Soil Sorption (adsorption-precipitation reactions) Plant Uptake
Pathogens	Sedimentation/filtration Natural die-off

From Brix, 1993. With modification.

removals of organic matter, solids, nitrogen, and pathogens are all considered when designing a particular system. A disadvantage of a single unit system which is anticipated to adequately remove all of these constituents is that the removal of each constituent is governed by different biological, physical, and chemical processes. Once a design, which is based on the knowledge available at that time, is set and a system is built, there is little that can be done to improve the performance of any one aspect without drastic changes in configuration (Brix, 1993).

Suspended Solids

The removal of suspended solids is accomplished through sedimentation and filtration by the media and plant roots. In order for adequate filtration to occur, the hydraulic conductivity of the bed must be large enough to allow the wastewater to contact the media (Findlater et al., 1990). Removal percentages for suspended solids in SF wetlands typically range from 71% to 98%. The removal percentages seldom drop below 70% regardless of the hydraulic or solids loading rate (Watson et al., 1990). A regression analysis based on 14 operating SF wetlands performed by Reed and Brown (1995) for TSS removal is shown below:

$$\mathbf{C_e = C_o[0.1058 + 0.0011(HLR)]} \qquad \mathbf{(1)}$$

where: C_e = effluent TSS concentration (mg/L)

C_o = influent TSS concentration (mg/L)

HLR = hydraulic loading rate (cm/d)

The hydraulic loading rate for SF wetland systems typically ranges from 0.4 cm/day to 20 cm/day (Crites, 1994). At these loading rates, a 30 mg/L TSS limit can be met even at influent TSS ranging from 235 mg/L to 272 mg/L. Most treatment systems perform primary treatment before the wetlands are encountered, therefore TSS at these levels are seldom experienced.

Biochemical Oxygen Demand

The removal of biochemical oxygen demand (BOD) is not as well understood as solids removal. Aerobic and anaerobic microbial degradation as well as sedimentation are known to contribute to BOD removal, however the extent to which each contributes is not well understood. According to Watson et al. (1990) and Findlater et al. (1990), coarse media beds act primarily as filters with most BOD being removed in association with the filtration and settling of suspended solids. With the availability of oxygen in the rhizosphere being questioned, aerobic activity may be limited to the very top portions of the wetland bed. This leaves the anaerobic microorganisms to perform the remaining treatment. Although the specific processes for BOD removal are not known, the treatment efficiencies have been quantified. A survey of 43 reed bed systems in the United Kingdom revealed an average BOD removal of 71.3 %. Of the 43 systems surveyed, 14 had effluent BOD concentrations in excess of 25 mg/L (Findlater, 1990). An evaluation of ten systems utilizing SF wetlands by Conley et al. (1991) showed BOD removal rates ranged from 64% to 96%.

A regression analysis based on 24 operating SF wetlands performed by Kadlec and Knight (1996) for BOD removal is shown below:

$$\mathbf{C_e = 0.33\ C_o + 1.4} \qquad \mathbf{(2)}$$

$$R^2 = 0.48$$

$$\text{Standard Error in } C_0 = 5.0$$

$$1 < C_i < 57 \text{ mg/L}$$

$$1 < C_0 < 36 \text{ mg/L}$$

$$1 < q_{avg} < 11.4 \text{ cm/d}$$

where: C_e = effluent BOD concentration (mg/L)

C_o = influent BOD concentration (mg/L)

q_{avg} = average hydraulic loading rate

The upper limit of acceptable influent BOD for the regression is limited to 57 mg/L. This restricts the use of the regression equation in many cases. The limited availability of useful BOD data reflects not only the variability found in many treatment systems, but also the inadequate understanding of the BOD removal mechanisms.

Nitrogen

Nitrogen removal within wetland systems is thought to take place mainly through nitrification, denitrification, plant uptake, and volatilization. Nitrification and denitrificaition are the predominant removal mechanisms. Nitrogen removal has not been successful at many SF wetland systems due to oxygen limitations. Treatment performance with respect to ammonia-N has especially been a problem. Ammonia removal rates have been reported to range from –1328 % to 94 % (US EPA, 1993). The wide range of efficiencies that can be expected from SF wetland systems make the design of SF wetlands for ammonia-N removal very difficult.

The limited availability of oxygen for nitrification, as discussed earlier, is contributed to the insufficient oxygen transfer of the wetland plants. Also, the high oxygen demand of the wastewater dictates that any oxygen transferred to the rhizosphere is preferentially used for carbon oxidation thereby leaving none for autotrophic nitrifier growth (White, 1995). With no oxygen present, nitrification ceases and subsequent denitrification does not occur. Wastewater passing through SF wetlands actually experienced an increase in ammonia-N concentration in 7 of the 14 systems evaluated by US EPA (1993). The increase in ammonia-N has been contributed to the anaerobic decomposition of organic nitrogen in the wetlands (White, 1995). The success of some SF wetlands in removing ammonia-N may be related to the transfer of oxygen across the surface of the bed. Effective removal of ammonia-N was accomplished in a gravel-based filter with no plants present. The only means by which oxygen could have been supplied was surface transfer (Sikora et al., 1995).

Anaerobic Treatment

After consideration of the limited availability of oxygen in SF wetlands, the SF wetland system can basically be viewed as an attached growth biofilter in an anaerobic contact chamber. The anaerobic microorganisms attach to the media and use it for support while degrading the influent organic matter into CH_3, H_2S, and CO_2 (Wood, 1995). Anaerobic conditions will dominate the SF wetland system if (1) the organic load to the wetland is high, and/or (2) the wetland is of a depth that surface transfer of oxygen from the atmosphere cannot provide aerobic conditions.

Kickuth proposed that anaerobic treatment is an important part of the treatment process occurring in SF wetlands (Hiley, 1995). In SF wetland microcosms Burgoon (1993) demonstrated the relative percentages of acetate that were oxidized by various pathways (Table 4). It is obvious from this data that there are many reactions, besides oxidation, that lead to degradation of organic matter in SF wetlands. In SF wetlands that operate under sulfate-rich environments the anaerobic processes can account for up to 82.7% of the organic matter degraded.

Anaerobic gravel beds without plants have been reported to accomplish high removal rates of BOD at short HRTs (Young and McCarty, 1969). Young and McCarty were the first to demonstrate the use of anaerobic filters for domestic wastewater treatment. An anaerobic rock filter without reeds achieved 88% removal of BOD and 71% removal of TSS, while a similar system with reeds achieved 97% removal of BOD and 91% removal of TSS (Wolverton, 1982). Anaerobic downflow stationary fixed-film reactors, although not exactly of the same configuration as SF wetlands, have also been shown to adequately treat wastewater at high organic loading rates and short HRTs (Kennedy and Droste, 1986). Even with adequate removals of organic matter having been observed at short HRTs, the opportunities for contact between the biofilm and wastewater play a major role in the treatment process. Because of this, the media is a controlling design parameter because it affects the surface area for biofilm attachment and the flow characteristics of the wastewater through the bed. The presence of plant roots also provides attachment surfaces for the anaerobic biofilm.

Disadvantages Caused by Lack of Understanding

The disadvantages of SF wetlands given earlier are in many ways related to the lack of complete understanding as to how the degradation processes take place in the wetland bed. With the true availability of oxygen to the bed unknown, design criteria based on the root zone method are imprecise and operating criteria which seek to maximize treatment processes which do not even occur lead to problems with treatment performance. The biological and hydrological complexity arise due to the same lack of knowledge. If the first issues can be solved, the disadvantages dealing with pest problems due to surface flow resulting from plugging of the media can be avoided.

One issue which has not been extensively reported in the literature is the production of compounds in the SF wetlands which may have adverse effects on other

Table 4 Percent Acetate Oxidized Via Various Pathways

	High Carbon Loading		Low Carbon Loading	
	Sulfate-Rich Environment			
Reaction	Plants	No Plants	Plants	No Plants
Oxidation	40.7	31.7	44.5	13.5
Nitrate Reduction	0	0	0	0
Sulfate Reduction	37.8	34.1	50.6	82.7
Ferric Iron Reduction	0.1	0.1	0.2	0.2
Methane Formation	19.6	32.1	0	0
Bacterial Biomass Formation	1.8	2	4.7	3.6
	Nitrate-Rich Environment			
Reaction	Plants	No Plants	Plants	No Plants
Oxidation	23.2	25.6	36.1	32.8
Nitrate Reduction	70.6	69.3	51.7	56.0
Sulfate Reduction	3.0	3.1	2.3	2.3
Ferric Iron Reduction	0.1	0.0	0.1	0.1
Methane Formation	0.0	0.0	0.0	0.0
Bacterial Biomass Formation	3.1	2.0	9.8	8.8

* Data from Burgoon, 1993, cited in Kadlec and Knight, 1996.

unit processes in the treatment system. By examining the possible reactions taking place in the anaerobic SF wetlands and identifying non-beneficial by-products of these reactions, the overall performance of a treatment system utilizing SF wetlands can be better understood.

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Chapter 2

Performance Evaluation of the Town of Monterey Wastewater Treatment Plant Utilizing Subsurface Flow Constructed Wetlands

BRANDON D. KIRACOFE AND JOHN T. NOVAK

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Abstract

Field tests were conducted and historical operating data were evaluated to assess the performance of the Monterey WWTP utilizing subsurface flow (SF) constructed wetlands. Previous work with SF wetlands has demonstrated adequate, but variable removal of organic matter, suspended solids, and nitrogen. Few research studies have observed the generation of compounds in the wetlands that affect other treatment processes, specifically reduced compounds that contribute to the chlorine demand. This study attempts not only to distinguish the factors leading to the inadequate performance of the SF wetlands in removing organic matter and nitrogen, but also to identify the cause of the frequent occurrences of a nondetectable chlorine residual in the chlorine contact tank at the Monterey WWTP. Collection and analysis of historical operating data from January 1998 to May 2000 revealed a constantly decreasing removal of carbonaceous biochemical oxygen demand ($CBOD_5$) by the SF wetlands and a poor removal of ammonia-N throughout the system. The decreasing removal of $CBOD_5$ appeared to be caused by clogging of the wetland bed media by accumulated solids. The inability to remove the accumulated solids by pumping was shown. Analysis of field data also showed that the SF wetlands removed 88% of the influent TSS and 71% of the influent $CBOD_5$, while experiencing a 18% increase in ammonia-N. Bisulfide produced in the anaerobic wetland beds accounted for 95% of the chlorine lost in contact tank. The variable production of sulfide is the cause of the frequent nondetectable chlorine concentrations observed. The results of this study suggest that chemical costs of chlorine

and sulfur dioxide may be greatly reduced if bisulfide can be removed before chlorination. Also, the use of large rocks as media in SF wetland beds may significantly reduce the physical and biological removal of organic matter.

Keywords: constructed wetlands, subsurface flow, anaerobic filter, bisulfide, and media clogging

Introduction

Constructed wetlands are extensively utilized in Europe and are increasingly being used in the United States to accomplish secondary and tertiary treatment of domestic, municipal, agricultural, and industrial wastewaters. When working properly, constructed wetlands offer an attractive option to communities, both small and large, who are looking for a process that is less energy intensive, requires less operator attention, and is more aesthetically pleasing than conventional wastewater treatment systems. Problems arise when the variability in the performance of a constructed wetland leads to inadequate treatment of organic material, nitrogen constituents, and pathogens. A less studied situation is the variability in the generation of compounds within the wetlands that adversely affect other processes in the treatment system. Fisher (1990) attributed the variability to an incomplete understanding of not only the biological, chemical, and physical processes within the system, but also to a lack of knowledge in the area of constructed wetland hydraulic characteristics.

There are two types of constructed wetlands used in water pollution control and wastewater treatment. Free water surface flow (FWS) wetlands are characterized by emergent vegetation that is rooted in a soil bed and a water surface that is open to the atmosphere (Reed and Brown, 1992). In subsurface flow (SF) wetlands the water flow is directed through a media filled bed, while remaining below the media surface. The SF wetland system serves basically as an attached growth biofilter in an anaerobic contact chamber. The anaerobic microorganisms present attach to the media and use it for support, while degrading the influent organic matter into CH_4, H_2S, and CO_2 (Wood, 1995). Anaerobic conditions will dominate the SF wetland system if (1) the organic load to the wetland is high, and/or (2) the wetland is of a depth that surface transfer of oxygen from the atmosphere cannot provide aerobic conditions.

The Town of Monterey, Virginia utilizes SF wetlands for secondary treatment of its municipal wastewater. The wastewater to be treated originates mainly from single-family homes, several small businesses, and two schools. Noticeably lacking is any industrial input in to the system. Infiltration and inflow into the sanitary sewer system, however, accounts for a major portion of the wastewater to be treated. The design flow of the system is 454 m^3/d (0.120 MGD); however, the daily influent flow has been observed to increase to over 3785 m^3/d (1.0 MGD). Over the past two years, numerous permit violations have occurred due to failure to meet the maximum effluent $CBOD_5$ concentration and loading, the minimum effluent pH level, and the minimum chlorine concentration in the chlorine contact tank.

Objectives

The objectives of this study were to evaluate and document the performance of the Monterey WWTP with respect to the removal efficiency of $CBOD_5$, TSS, and NH_3-N. In this evaluation, the effects of media clogging and inadequate solids removal from the SF wetlands were considered. Specific attention was given to design characteristics of the SF wetlands that could be responsible for the frequent permit violations. The identification of the chlorine-consuming constituents that cause the incidents of non-detectable chlorine residual concentrations in the contact tank was also a priority.

Background

On May 7, 1993, the Monterey Wastewater Treatment Plant (WWTP) utilizing SF wetlands was given a certificate to operate. The system layout is shown in Figure 1. Preliminary treatment is accomplished using a bar screen and two parallel gravity grit chambers. Primary treatment of wastewater before it enters SF wetlands is extremely important to decrease the loadings of TSS and BOD. Primary treatment occurs in a 26 ft deep Imhoff Tank, in which the upper compartment is used for primary sedimentation, and the lower compartment is used for anaerobic sludge digestion. After the Imhoff Tank, a splitter box directs the wastewater to six SF wetland cells operated in parallel. Wetland cell 1was originally a hyacinth pond that served as a pilot system to determine the effectiveness of a natural treatment system. Cell 1 was lined with a 12 in. layer of compacted silty clay. The flow leaving this cell was observed in this study to be much less than the flow entering, which suggests that wastewater is leaching through the clay liner. Wetland cells 2 – 6 are identical in design and are slightly larger than cell 1. The design characteristics of the Monterey SF wetlands are given in Table 1. Surface flow and standing water have been observed on all six wetland cells with the majority being at the influent end. The effluent of the six wetland cells is collected and fed into a single pipe and chlorinated using a solution feed chlorinator with a capacity of 50 lbs/day. At the design flow of 0.120 MGD, the chlorine contact tank provides a 30-minute detention time. Post aeration is accomplished at the effluent end of the contact tank using a diffused air blower. A sulfonator with a capacity of 50 lbs/day provides dechlorination ability. Both the chlorinator and sulfonator feed rates are controlled manually and are based on daily testing results for total chlorine residual in the contact tank and in the final effluent. The final plant effluent discharges into West Strait Creek.

Methodology

The first step taken in the Monterey study was the accumulation of performance data for the plant. This data was obtained from both the Valley Regional Office of the Department of Environmental Quality (VDEQ) and the town officials of Monterey. The time period reviewed was from January 1998 through May 2000. January 1998 was chosen as the starting date because this was the time that the present operator began employment at the facility. All information was taken from monthly Discharge Monitoring Reports (DMR) that were submitted to VDEQ and the daily logbook maintained at the Monterey WWTP. The parameters of importance in this study included the concentrations of TSS, $CBOD_5$, and ammonia-N in the final effluent and the subsequent TSS and $CBOD_5$ loadings imposed on West Strait Creek. By inspecting these parameters the long-term treatment performance of the system was evaluated. Of particular interest were the incidents of non-detectable chlorine residual that frequently occurred in the chlorine contact tank. By identifying any pattern of occurrence or correlation with flow variation, the problem with maintaining an adequate chlorine residual concentration could be better understood.

Evaluation of Overall Treatment Efficiency

In order to evaluate the treatment efficiency of the Imhoff Tank performing primary treatment and the SF wetlands performing secondary treatment, sampling was required before and after each process. Sampling points in the treatment configuration were selected at four locations (Figure 1). The selected sampling locations were as follows:

(1) Influent to Imhoff Tank

(2) Discharge from the Imhoff Tank

(3) The manhole which receives the combined effluent from the six wetland cells.

(4) The effleunt discharge into West Strait Creek.

Eight hour composite samples were taken at each sample site and analyzed for $CBOD_5$, TSS, VSS, and ammonia-N. All analyses performed in this project for the determination of $CBOD_5$, TSS, VSS, and ammonia-N were conducted according to Standard Methods for the Examination of Water and Wastewater.

Media Clogging

Hydraulic Detention Time

The mean residence time of the wastewater within a wetland cell was determined using bromide as a tracer. Bromide was chosen for this study because of its chemical stability and low sorptive properties (Netter, 1994). One hundred grams of sodium bromide was added as a single-pulse input in solution form to the intake pipe of wetland cell 6 in the splitter box. Samples were taken using an ISCO Autosampler at the outlet pipe of cell 6 every hour for the first eight hours, every one-half hour for the next fifteen hours, and every hour for the final seventeen hours. The samples were filtered on-site using 0.45 μm Whatman glass filters and stored on ice until the experiment was complete. The samples were then frozen until analysis was performed using a Dionex Anion Ion Chromatograph. The mean retention time and yield of bromide were calculated by integration of concentration-time data, after correction for the background level of bromide.

Wetland Solids Sampling

Units capable of sampling at depths of 0.69m, 0.59m, 0.30m, and 0.15m were constructed and installed in wetland cell 6 at distances from the inlet of 5.5m, 12.2m, 18.3m, and 22.9m. Sludge samples were drawn using a peristaltic pump and analyzed for TSS.

Fluctuations in Total Chlorine Residual Concentration in the Chlorine Contact Tank

When this study began the cause of the fluctuations in the residual chlorine in the contact tank was unknown. It was hypothesized that there was a slug of some constituent passing through the wetland filters that exerted a chlorine demand large enough to cause a depletion of the chlorine in the contact tank. In order to identify the chlorine-demanding material, sampling was required during a time in which the chlorine concentration dropped significantly. During a sampling event intended to obtain the chlorine demanding material, samples were taken from the manhole every hour for a 24-

hr period. These samples were analyzed for TSS, CBOD, ammonia-N, nitrate-N, nitrite-N, and sulfide. At the same time intervals, samples were taken from the contact tank and analyzed for chlorine concentration. By correlating a decrease in chlorine residual with an increase in the other parameter of concern, the chlorine-demanding constituent was identified.

Total residual chlorine was determined using a Hach pocket colorimeter provided by the Monterey WWTP staff. Samples to be analyzed for nitrate-N and nitrite-N were filtered on-site using 0.45 μm Whatman glass filters and stored on ice. The samples were analyzed within two days using a Dionex Anion Ion Chromatograph. Bisulfide was determined using the Hach Model DR/700 Portable Colorimeter. Method 8131 (Methylene Blue) was performed using module 61.01 and program # 61.12.1. A detailed protocol for each analytical method mentioned in this section is presented in Appendix B.

Wetland Sludge Characteristics

Various tests were performed on sludge taken from the wetland sumps and on sludge taken from the Blacksburg WWTP. The tests included Specific Oxygen Uptake Rate (SOUR), Sludge Volume Index (SVI), and Capillary Suction Time (CST). By determining the SOUR of the wetland sludge, its aerobic activity as compared to activated sludge was estimated. To measure the SOUR of the wetland sludge, fifty ml of sludge was placed at the bottom of a 300 mL BOD bottle and BOD dilution water was added. After oxygenation, bactopeptone was added to increase the COD concentration by 300 ml, and a DO probe with a mixer was used to measure the oxygen depletion. SOUR for the activated sludge was performed in the same fashion, with all determinations being conducted at room temperature.

The SVI allowed the determination of the volume occupied by one gram of settled suspended solids. This sludge was placed into a one liter graduated cylinder, and the settled volume was measured after thirty minutes of settling. Dividing this settled volume by the initial suspended solids concentration resulted in the SVI (Grady, et al., 1999).

A Triton Type P304M Capillary Suction Time (CST) device was used to measure the dewatering properties of the wetland sludge and the activated sludge. The samples were transferred from the holding containers to the CST device using 10 mL cutoff tip pipettes. Triplicates of each sludge were analyzed using the CST device and a simple average was taken in order to compare the two.

Results and Discussion

The performance evaluation of the Monterey WWTP included determining the removal efficiencies of $CBOD_5$, TSS, and NH_3-N. In this evaluation, the effects of media clogging and inadequate solids removal from the SF wetlands were considered. Specific attention was given to design characteristics of the SF wetlands that could be responsible for the frequent permit violations. Other areas of study included identifying the chlorine-consuming constituents that cause the incidents of non-detectable chlorine residual concentrations in the contact tank and determining the causes of the low effluent pH levels.

The importance and impact of infiltration and inflow (I&I) into the sanitary sewer system has been established by an engineering firm. Repairs to the sewer system to reduce (I&I) are planned, however the changes will be made over several years due to budget constraints. It was not the intent of this study to determine the effects of the reduced I&I on the treatment system. For this study, all calculations requiring an influent flow used an a flow of 84 m^3/day to each wetland cell. This flow was based on the average monthly flows measured from January 1998 to May 2000.

Historical Operating Data and Treatment Performance

<u>Total Suspended Solids</u>

The plant TSS data (Figure 3), shows that the VPDES average monthly concentration limit of 30 mg/L and loading limit of 13.6 kg/d were not violated at any point over the reviewed time period (January 1998 – May 2000). The spikes in effluent TSS coincided with pumping of sludge from the wetlands to the sludge drying beds. As the sludge was pumped, solids were agitated and carried out with the effluent. Even during these times, the treatment efficiency of the wetlands was such that the permit limits were met.

Samples collected as part of this study showed that the average TSS concentration was reduced from 168 mg/L to 51 mg/L by the Imhoff Tank with an average removal of 70% (Figure 4), which is typical for primary treatment processes. The average TSS through the wetlands was reduced from 51 mg/L to 6 mg/L for a removal of 88%. SF wetlands have been reported to achieve TSS removals of 71% to 98% (Conley et al., 1991). The Monterey treatment system is performing well with regard to TSS removal.

Carbonaceous Biochemical Oxygen Demand

The data for the treatment performance of the Imhoff Tank and the SF wetlands with respect to $CBOD_5$ can be found in Figure 4. The average $CBOD_5$ concentration was reduced from 162 mg/L to 94 mg/L in the Imhoff Tank for a removal of 42%. The average $CBOD_5$ concentration was reduced from 94 mg/L to 27 mg/L in the SF wetlands with an average removal of 71%. SF wetlands have been reported to achieve BOD removals of 64% to 96% (Conley et al., 1991). The Monterey wetlands performed within this range of removals; however, the treatment was not sufficient to consistently meet the permit limitations. Inspection of the historical operating data for $CBOD_5$ revealed a different trend than was observed for the TSS data. As can be seen in Figure 3, the effluent $CBOD_5$ has been steadily increasing since January 1998. From May 1999 to May 2000, the average monthly concentrations have been at or above the 25 mg/L limit. The loadings to West Strait Creek have shown variability with the trend being toward an increase in $CBOD_5$ input to the waterway.

Nitrogen

The removal of nitrogen, with specific attention being given to ammonia-N, is becoming a more important issue in wastewater treatment. Ammonia-N is of concern because of its toxicity to fish, the oxygen demand it exerts in the receiving water body, and the potential it presents for eutrophication in the receiving water body. Although there are permit limits listed for ammonia-N, the Monterey WWTP is not currently required to meet these limits by consent order of VDEQ. Discussions with VDEQ officials indicate that ammonia limits may be put in place in the next few years. The dashed lines shown in Figure 5 simply represent the tiered ammonia-N limits that may be put in place in the future.

A plot of the historical operating data (Figure 5) revealed that sufficient ammonia removal was occurring during the winter months to meet the proposed ammonia-N limit of 5.1 mg/L. However, from April to December the limit of 3.7 mg/L was being constantly exceeded. The data relating to the treatment performance of the Imhoff Tank and the SF wetlands with respect to ammonia-N is presented in Figure 6. The average ammonia-N concentration increased from 14.3 mg/L to 18.3 mg/L in the Imhoff Tank with an average ammonia-N production of 31%. The average ammonia-N concentration was increased from 18.3 mg/L to 21.7 mg/L in the SF wetlands with an average production of 18%.

The increase in ammonia-N concentration experienced across the system was also observed at seven of the fourteen SF wetland systems evaluated by the US EPA in 1993. The production of ammonia is due to the anaerobic decomposition of both the organic nitrogen in the influent and the organic nitrogen already present in the wetland beds from microbial death and plant decay (White, 1995). The nitrogen distribution between organic-N and ammonia-N through the treatment process is shown in Figure 7. As predicted, there was an increase in ammonia-N in both the Imhoff Tank and in the SF wetlands. The increase in ammonia-N in the SF wetlands corresponds to a decrease in organic-N. By examining total nitrogen concentrations at sampling locations 1 and 4, it was found that only a 4% removal of total nitrogen was accomplished in the treatment system.

Media Clogging

The inadequate treatment and steady decline in treatment performance associated with $CBOD_5$ can be attributed to several factors, including the clogging of the wetland media. The clogging seems to be caused by the overloading of organic matter and solids and the inability to adequately remove the accumulated solids through pumping. Tanner et al. (1998) reported significant accumulation of organic matter in gravel-bed constructed wetlands treating farm dairy wastewaters. The build-up of organic matter led to a 50% reduction in wastewater detention times in these systems. The following sections discuss the media clogging that is likely occurring at the Monterey WWTP.

Hydraulic Detention Time

Adequate wastewater detention time in a SF wetland system is necessary to ensure optimal performance (Brix, 1990). When a SF wetland is constructed, the theoretical detention time may be calculated by considering the effective volume and flow to be treated. However, the porosity of the substratum changes with time due to the accumulation of solids, microbial growth, and the penetration of plant roots into the gravel. As the porosity changes, the effective volume changes resulting in variable or deteriorating treatment performance. A bromide tracer study performed on Monterey wetland cell 6 was useful for determining the extent of porosity change and actual mean detention time of the wetland.

SF wetlands are modeled as either plug-flow or completely mixed reactors. As shown in Figure 8, a non-ideal flow pattern occurred in Monterey wetland cell 6 which caused the hydraulic regime to deviate from plug-flow. As expected for flow through an irregular rock substratum, the tracer-response curve indicated a large amount of dispersion and asymmetry with dead spaces that are not accessible to the wastewater flow. Because of this, a portion of the wastewater passed through the system faster than predicted by the theoretical detention time. Also, presence of a long-tail suggested that wetland cell 6 short-circuited and contained a large amount of dead space (DeBusk et al., 1990). The bromide added showed rapid passage through the system with a mean detention time of 0.51 days at an average flow of 69.8 m^3/d to each cell. The actual effective volume was calculated as 24.8 m^3. Using this effective volume, the relationship between influent flow and detention time was plotted (Figure 9). At the average influent flow of 500 m^3/d for the period from January 1998 to May 2000, the mean detention time was calculated as 0.43 d (10.3 hr). The minimum average monthly flow experienced during this period was 167 m^3/d in July 1999 which resulted in a mean detention time of 0.89 d (21.3 hr). All wetland cells were assumed to have similar hydraulic characteristics, and thus each cell had a mean detention time well below the recommended value of two to seven days (Table 2) proposed by Kadlec and Knight (1996) even at very low influent flows. The short detention time limited not only the $CBOD_5$ removal efficiency, but also the total nitrogen removal efficiency.

In May 1993, the effective volume of wetlands 2 – 6 was determined to be 81 m^3. After operating 9.7 years, the effective volume of wetland 6 was determined to be 24.8 m^3. This is a 69% reduction in effective volume which corresponds to the extensive clogging of the wetland media.

<u>$CBOD_5$ and TSS Loading on Entry Zone Cross Section</u>

Because the Monterey SF wetlands were designed for horizontal flow, an important factor dealing with $CBOD_5$ and TSS loading was the issue of entry zone cross section loading. Excessive loadings of this area over several years appear to have resulted in the pore spaces of the media being clogged. The clogging caused a decrease in detention time which resulted in less biological degradation of the organic material because of decreased exposure to the biofilm. This is a major reason for the failure of SF wetland systems (Conley et al., 1991). According to Crites (1994) in order to avoid clogging, the BOD and TSS loading rates on the entry zone cross section should not exceed 0.2 kg/m^2-day and 0.08 kg/m^2-day respectively. The calculated loadings for $CBOD_5$ and TSS onto the entry zone of Monterey wetland cells 2-6 were approximately five times this level at 1.03 kg/m^2-d and 0.56 kg/m^2-d. The entry zone loadings exceeded the recommended values and explained the reduction of the effective volume, the reduction of the actual detention time, the increase in short-circuiting (Figure 8), and the presence of surface-flow conditions. Of particular importance was the build-up of solids which seems to continually lessen the treatment efficiency of the SF wetlands with respect to the removal of organic matter.

<u>Removal of Solids from Wetland Systems</u>

Unlike many SF wetland treatment systems which do not remove sludge from the wetland beds, the Monterey WWTP pumps sludge on a bi-yearly basis. There are twelve sumps in each wetland cell from which the sludge is pumped. Using a yield of 0.2 gm biomass COD produced/gm COD removed, an accumulation period of 182.5 days, a COD removal of 115 mg/L, and an average wastewater flowrate of 83.4 m^3/d, it was calculated that 292 kg TSS is produced in the wetland over a 6-month period due to microbial activity. The removal of TSS by the wetlands contributed 685 kg of solids over the 6-month period. Assuming a 30% reduction of the accumulated solids due to washout from rain-events and decay, it was calculated that 684 kg of sludge had theoretically accumulated in wetland cell 6. Complete pumping of wetland 6 removed of 27.5 m^3 of sludge with had a TSS of 2518 mg/L. This resulted in the removal of 69 kg of solids which was only 10% of the theoretical solids production. The solids that remained in the wetland appear to be located in pore spaces that are inaccessible to pumping. It is likely that the incomplete removal of solids over the 9.7 years that the Monterey SF wetlands have been operating has led to the accumulation of solids which has reduced the treatment efficiency to an unacceptable level.

After six months of operation from January 2000 to June 2000, large increases in sludge TSS were found at depths of 0.69m, 0.59m, 0.30m, and 0.15m in wetland cell 6, with the most dramatic increases occurring near the inlet zone. TSS in the sample taken at a depth of 0.15m from the unit located 5.5m from the inlet of wetland cell 6 increased 350%. Of the sixteen depth samplers located in wetland cell 6, sludge was only able to be drawn from two of the samplers after complete pumping of the wetland. One sample was drawn from a depth of 0.69m and the other from 0.59m. The presence of sludge after pumping and the varying locations of the sludge likely indicates random pockets of sludge that are not removed by pumping. The six-month period between sampling events was based on the time between initial sampler installation and when it was feasible for the plant operator to pump wetland cell 6.

Other Factors Leading to Inadequate Treatment Performance

The following sections discuss variations in the Monterey SF wetland design that may be causing the problems in treatment performance.

<u>Hydraulic Loading</u>

The issue of hydraulic loading was of concern with the Monterey wetlands. Because of the infiltration and inflow problems and because of the limited surface area of the wetland cells, the hydraulic loading experienced is greatly in excess of the maximum recommended value for properly functioning SF wetland systems (Table 1). A further increase in influent flow will increase the hydraulic loading which will decrease the detention time of the wetland cells leading to a further reduction in BOD removal.

Media Size

The media within the wetland bed serves several purposes. Not only does it allow plant growth, but it also provides surfaces for biofilm growth and aids in the removal if suspended solids by settling and filtration (Tanner, 1995). The sizes of the rocks used to construct the beds in Monterey are reported to range from 0.0127 m to 0.35 m (Table 2). Many of the rocks are also sharply angled. In most cases, soil or at the most small gravel are the preferred media. There are many drawbacks to the media used in the Monterey wetlands. The main problem of the large media is that it decreases the specific surface area available for biofilm attachment. According to Hammer and Knight (1994), the limited availability of attachment substrates is a common limiting factor in SF wetland systems. With less biofilm present, the amount of microbial degradation is limited. Also the irregular surfaces of the rocks contribute to the clogging of the media. A rounded rock is recommended. A final drawback is that successful plant growth is greatly hindered by the presence of large, sharp-edged material especially near the bed surface (Geller et al., 1990). In most cases, a layer of pea size gravel is recommended on the bed surface to facilitate root establishment. Roots assist in the removal process by providing additional surfaces for microbial growth and providing additional filtration. Sizes of surface layer rocks from the Monterey SF wetlands ranged from 0.16m to 0.025m.

Bed Depth

The major concern with bed depth is the ability of wetlands to extend their roots throughout the bed depth in order to provide oxygen for aerobic degradation of organics and ammonia-N. There are varying views present in the literature as to the amount of oxygen that the plant roots provide in the bed, with recent studies concluding that plants do not provide oxygen in an amount that will significantly contribute to aerobic activity (Kadlec and Knight, 1996; Hiley, 1995; Brix, 1990). The plants which cover the Monterey wetlands consists mainly of Yellow Flag Iris and grasses. If these plants were able to transport a significant amount of oxygen through their roots, they still would not be able to extend their roots through the entire depth of 0.84 m which is found in the Monterey wetlands.

Availability of Oxygen

The critical factor in the nitrification process is the presence of oxygen. As stated earlier, plants are not able to introduce sufficient oxygen for nitrification. The oxygen that is available for aerobic activity can be approximately accounted for by the transfer of oxygen across the media-atmosphere interface (Hiley, 1995). Even if plants were able to provide oxygen into the bed, carbon oxidation would quickly deplete the available oxygen before nitrification would occur. Sampling performed at location 3 indicated nitrate-N and nitrite-N concentrations near non–detectable levels. As long as the concentration of organics remains at a high level in the wetlands, nitrification and subsequent denitrification will not be effective in total nitrogen removal from the system.

Total Chlorine Residual Concentration in Chlorine Contact Tank

The monthly occurrences of a non-detectable chlorine residual concentration were plotted with no discernable patterns being evident (Figure 10). There was also no relationship for fluctuations in chlorine concentration found for changes in flow or variations in TSS, $CBOD_5$, and ammonia-N entering the chlorine contact tank.

Sulfate Reduction

It has been established that SF wetlands are predominantly anaerobic environments. In the Monterey wetland cells, there are several factors contributing to the anaerobic conditions. The wetlands experience a very high organic loading. Wetland cells 2 – 6 have a $CBOD_5$ loading rate of 288 kg/ha-d. This is greatly in excess of the recommended maximum value of 75 kg/ha-d. At this high organic loading rate, any available oxygen is quickly utilized in the inlet zone. The wetland cells in Monterey have a sparse distribution of beneficial plant life. A portion of each wetland cell is covered in grass. The roots of the grass cannot extend throughout the depth of the beds. Even if the plants did provide substantial amounts of oxygen through their roots, it would be quickly utilized in the upper portions of the wetland beds. Anaerobic processes utilizing SO_4^{2-},

Mn^{4+}, Fe^{2+}, and CO_2 as electron acceptors occur if there is no oxygen or nitrate available (Wood, 1990). The third factor contributing the anaerobic conditions is the depth of the wetlands. Well-established plant roots and rhizomes would have difficulty reaching the 0.84 m depth of the Monterey wetlands.

The aerobic portions of the wetlands are limited to the inlet zone and to the portion of the wetland near the surface. The sharp gradients in oxygen concentration cause aerobic-anaerobic interfaces which support transformations leading to pollutant removal. The processes by which these transformations remove pollutants are difficult to measure (Reddy and D'Angelo, 1997). In an experiment studying acetate oxidation in controlled subsurface-flow wetland microcosms performed by Burgoon (1993), it was shown that sulfate reduction can contribute up to 82.7 % of the acetate oxidized (Table 3). The Monterey wetlands can be assumed to be under a high carbon loading with partial coverage by plants. From Table 3, the percentage of BOD removal that is attributed to sulfate reduction for the Monterey wetlands is estimated to be 35 %. By also considering the formation of methane, it is likely that anaerobic pathways contribute more to BOD removal than do aerobic pathways.

Of particular interest in this study was the reduction of sulfate to bisulfide. Heterotrophic bacteria, such as *Desulfovibrio desulphuricans*, mediate this reduction. The following reactions depict the transformations that take place in an anaerobic environment where sulfate is serving as the electron acceptor:

$$\mathbf{2\ \underset{lactate}{CH_3CHOHCOO^-} + SO_4^{2-} + H^+ \Rightarrow 2\ \underset{acetate}{CH_3COO^-} + 2\ CO_2 + 2H_2O + HS^-} \qquad \mathbf{(1)}$$

$$\mathbf{\underset{acetate}{CH_3COO^-} + SO_4^{2-} + H^+ \Rightarrow 2\ CO_2 + 2H_2O + HS^-} \qquad \mathbf{(2)}$$

Another source of bisulfide in the wetland bed is the release of reduced sulfate from heterotrophic bacteria when death and sedimentation occurs (Kadlec and Knight, 1996). The large amount of sludge accumulating in the wetland bed likely decays which generates reduced sulfur. Both processes result in the accumulation and subsequent transport of bisulfide out of the wetland beds by way of the wastewater flow.

The following relationship exists between the concentrations of bisulfide ion and hydrogen sulfide gas:

$$\mathbf{HS^- + H^+ \Leftrightarrow H_2S \qquad (3) \qquad\qquad pKa = 7.1}$$

A typical pH value of the wastewater leaving the Monterey wetlands is 6.8. At this pH approximately 43% of the reduced sulfate is in the form of bisulfide. A portion of the sulfide produced in the reduction of sulfate is lost to the atmosphere through volatilization of the hydrogen sulfide gas.

Several sampling events were performed in order to determine the relationship between the sulfide produced and the amount of chlorine lost in the contact tank. Sampling during Test 1 performed at sampling location 3 revealed an average sulfate concentration entering the wetland of 33 mg/L and an average sulfate concentration leaving the wetland of 16 mg/L. Through simple stoichiometry it can be seen that for every 1 mg/L of sulfate that enters the wetland, there is the potential for the formation of 0.5 mg/L of bisulfide. With a sulfate depletion of 17 mg/L, theoretically 8.5 mg/L of sulfide was produced within the wetland. Considering the pKa discussed earlier, approximately 3.7 mg/L of bisulfide should have been measured at sampling location 4. The bisulfide concentration measured at the manhole varied from 2.9 mg/L to 4.1 mg/L. Fluctuations in pH and volatilization of the hydrogen sulfide gas account for the variation.

Once the production of bisulfide within the wetland was quantified, the effects of this produced sulfide on the total chlorine residual concentration was studied. In order to calculate the mass of bisulfide and mass of chlorine in the contact tank several assumptions were made. The assumptions were as follows:

1. Flow into the contact tank was equal to the influent flow 12 hours earlier. This assumption was based on the detention time determined for the wetland cells using the bromide tracer.
2. At all flows the detention time of the contact tank was 30 minutes.
3. The mass of bisulfide measured at the manhole corresponded to a measured total chlorine residual concentration in the contact tank 30 minutes later.

Equation 4 was used to calculate the mass of bisulfide entering the contact tank.

$$\mathbf{MS_{CT} = (XS_{Man} * F * \tau)/1000} \qquad \textbf{(4)}$$

where: MS_{CT} = mass of bisulfide entering the contact tank (gm)

XS_{Man} = concentration of bisulfide measured at manhole (mg/L)

F = flow into contact tank (L/min)

τ = hydraulic detention time of contact tank (min)

Equation 5 was used to calculate the predicted mass of chlorine in the contact tank.

$$\mathbf{PMCl_{CT} = I_{Cl} / \tau} \qquad \textbf{(5)}$$

where: $PMCl_{CT}$ = predicted mass of chlorine in contact tank (gm)

I_{Cl} = feed rate of chlorine (gm/min)

Equation 6 was used to calculate the actual mass of chlorine in the contact tank.

$$\mathbf{MCl_{CT} = (XCl_{CT} * F * \tau)/1000} \qquad \textbf{(6)}$$

where: MCl_{CT} = mass of c in chlorine in contact tank (gm)

XCl_{CT} = concentration of chlorine measured in contact tank (mg/L)

F = flow into contact tank (L/min)

τ = hydraulic detention time of contact tank (min)

Bisulfide is readily oxidized by chlorine or by any other oxidizing agent. The following balanced oxidation-reduction reaction shows that 8.6 gm of Cl_2 are required to oxidize every 1 gm of HS^- produced:

$$\mathbf{4Cl_2 + HS^- + 4H_20 \Rightarrow SO_4^{2-} + 9H^+ + 8Cl^-} \qquad \textbf{(7)}$$

The results of Test 1 are shown in Figure 11. As the bisulfide increased, the total chlorine residual concentration decreased to the point that the concentration was at a non-detectable level. For the time in which the total chlorine residual concentration was at a detectable level it was possible to determine the amount of chlorine loss that could be contributed to the presence of bisulfide for a set of samples taken. The measured mass of chlorine lost equaled the predicted mass of chlorine in the contact tank minus the measured mass of chlorine in the contact tank. The mass of chlorine lost due to the presence of bisulfide equaled the mass of bisulfide entering the contact tank multiplied by

8.6. A plot of these values revealed that 94 % of the observed mass of chlorine lost could be accounted for by the presence of bisulfide in the wastewater (Figure 12). In Test 2 the same trend was found (Figure 13), with 96 % of the observed mass of chlorine lost being due to the presence of bisulfide (Figure 14).

Violation of Minimum pH Permit Limit

Chlorination

The minimum permitted pH of the effluent wastewater being discharged into West Strait Creek is 6.5 SU. The effluent is usually below this minimum, meeting compliance standards only about 1 in 10 days. The feed rate of chlorine during Test 1 was 19.1 kg/d. At this feed rate the total chlorine residual concentration should have been approximately 40 mg/L. For at least 20 hours during that day the total chlorine residual concentration was at a non-detectable level. Examination of Equation 7 showed that for every four moles of Cl_2 that were consumed, nine moles of H^+ were formed. The reduction of 40 mg/L Cl_2 led to a hydrogen ion concentration of 0.00025 mol/L. This corresponds to a pH of 3.6. The actual pH will depend on the buffering capacity of the wastewater, but a value below 6.5 is expected. Therefore, it is likely that excessive chlorine use leads to the reduction in pH.

Dechlorination

The dechlorination process also played a role in the decrease in pH. Dechlorination is accomplished using sulfur dioxide. During periods in which the mass of bisulfide entering the contact tank was not of sufficient quantity to completely reduce the chlorine, there was a large amount of chlorine leaving the contact tank. The chlorine leaving the contact tank reacted with the sulfite ion as shown in Equation 8 producing hydrogen ions.

$$\mathbf{SO_2^- + Cl_2 + 2H_2O \Rightarrow SO_4^{2-} + 4H^+ + 2Cl^-} \quad \mathbf{(8)}$$

At times the chlorine concentration leaving the contact tank was as high as 42 mg/L. At this concentration, dechlorination theoretically produced a hydrogen ion concentration of

0.0024 mol/L. This corresponded to a pH of 2.6. Again the actual pH will depend on the buffering capacity of the wastewater, but a value below 6.5 is expected.

Wetland Sludge Characteristics

In order to better understand the processes that take place within the Monterey wetlands, several comparisons were made between the characteristics of the sludge that accumulates in the wetlands and the characteristics of activated sludge. By examining the average SOUR for both the wetland sludge and activated sludge (Table 4) it was observed that the SOUR for the wetland sludge was 24 % of that for activated sludge. The study performed by Burgoon (1993), showed that in SF wetlands under a high carbon loading and with a sulfate-rich environment, oxidation accounted for 23.2 % of the acetate oxidized in a system with plants, and 25.6 % in a system with no plants. If the degradation in the activated sludge was assumed to be nearly 100 % due to oxidation, then the aerobic fraction found in the Monterey wetlands corresponded directly to the oxidation activity found by Burgoon. The SVI of the wetland sludge was 73.1 mL/g, which was slightly lower than the activated sludge SVI of 90.6 mL/g. The wetland sludge, however, did not result in a well-settled sample. There were many small particles remaining in suspension after the thirty-minute settling period. The CST value for the wetland sludge was found to be 82.7 sec/g, as compared to a CST of 8 sec/g for activated sludge. The colloidal nature of the sludge and the absence of any noticeable flocculation led seemed to lead directly to the poor settling characteristics of the anaerobic sludge.

Conclusions

The causes of the frequent permit violations at the Monterey WWTP were determined by evaluating the treatment efficiency of each step in the treatment process, by examining the feasibility of removing accumulated wetland solids by pumping, and by identifying substances generated within the SF wetlands that exerted a chlorine demand in the contact tank. The following conclusions were drawn from the research:

1. The wetland beds were experiencing significant clogging due to the overloading of TSS and $CBOD_5$ in the inlet zone and due to the inability of pumping to remove a significant portion of the accumulated solids. Clogging has continually increased over time, and the removal efficiency of organic matter has continually declined due to a reduction in the mean detention time of the wetland cells.

2. The major causes of the inadequate treatment performance of the SF wetlands were the short wastewater detention time due to clogging and the large size of the rocks used in the wetland beds. The large media led to less surface area for biofilm growth which reduced the amount of biodegradation of organic matter that occurred in the system.

3. The variable production of bisulfide from the reduction of sulfate in the anaerobic Monterey wetlands caused the frequent occurrences of a non-detectable total chlorine residual concentration in the chlorine contact tank.

4. The overfeeding of chlorine and sulfur dioxide led to the decrease in pH to a level below permit limits and to a level toxic to the receiving stream.

5. The inadequate removal of organic matter and the limited supply of oxygen to the wetland bed resulted in little or no removal of nitrogen in the system. In its current configuration, the Monterey WWTP will not be able to meet future ammonia-N limits set by VDEQ.

6. The Imhoff Tank was functioning properly as a primary treatment step.

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Table 1 Design Characteristics of Monterey SF Wetlands

PARAMETER	Monterey SF Wetland Cell 1	Monterey SF Wetland Cells 2 - 6
BOTTOM LENGTH	19.5 m	22.9 m
BOTTOM WIDTH	7.0 m	7.6 m
INSIDE SLOPE	2:1	2:1
SURFACE AREA	237 m^2	288 m^2
DEPTH	0.84 m	0.84 m
VOLUME	156.6 m^3	184.1 m^3
POROSITY	0.44	0.44
EFFECTIVE VOLUME	69 m^3	81 m^3

Table 2 Design Parameters of Importance to Treatment Performance and Typical Values Reported in the Literature

PARAMETER	Monterey SF Wetland Cell 1	Monterey SF Wetland Cells 2 - 6	Typical Design Values Reported in Literature
DETENTION TIME	0.37 day	0.43 day	2 to 7 days (1)
$CBOD_5$ LOADING ON ENTIRE WETLAND[a]	333 kg/ha-day	274 kg/ha-day	< 75 kg/ha-day (2)
$CBOD_5$ LOADING ON ENTRY ZONE CROSS SECTION	1.08 kg/m^2-day	1.03 kg/m^2-day	< 0.2 kg/m^2-day (2)
TSS LOADING ON ENTRY ZONE CROSS SECTION	0.59 kg/m^2-day	0.56 kg/m^2-day	< 0.08 kg/m^2-day (2)
HYDRAULIC LOADING[b]	0.35 m^3/m^2-day	0.29 m^3/m^2-day	0.02 - 0.20 m^3/m^2-day (1)
MEDIA SIZE	0.0127 m – 0.305 m	0.0127 m – 0.305 m	0.013 m – 0.076 m (2)
BED DEPTH	0.84 m	0.84 m	0.3 m – 0.6 m (3)
VEGETATION	Yellow Flag Iris, Cattails, Bulrushes, Grass	Grass, Yellow Flag Iris	Reed (Phragmites), Cattails, Bulrush (3)

[a] All loadings based on average CBOD5 concentration = 94.2 mg/L and average TSS concentration = 51.1 mg/L entering wetlands as determined during project.
[b] The flow used in all calculations is 617 m^3/day which is the average of the monthly average flows as reported to the Virginia Department of Environmental Quality for the period of January 1998 – March 2000. The flow to each wetland cell was assumed to be equal.

(1) Kadlec and Knight, 1996
(2) Crites, 1994
(3) U.S. EPA, 1993

Table 3 Percent Acetate Oxidized Via Various Pathways in a Sulfate-Rich Environment

	High Carbon Loading		**Low Carbon Loading**	
Reaction	Plants	No Plants	Plants	No Plants
Oxidation	40.7	31.7	44.5	13.5
Nitrate Reduction	0	0	0	0
Sulfate Reduction	37.8	34.1	50.6	82.7
Ferric Iron Reduction	0.1	0.1	0.2	0.2
Methane Formation	19.6	32.1	0	0
Bacterial Biomass Formation	1.8	2	4.7	3.6

* Data from Burgoon, 1993, cited in Kadlec and Knight, 1996.

Table 4. Characteristics of Wetland Sludge and Activated Sludge

	Average SOUR (mg/g)/h	Average OUR (mg/L)/h	SVI (mL/g)	CST (sec)/gm
Wetland Sludge	4.37	16.3	73.1	82.7
Activated Sludge	18.17	25.0	90.6	8.0

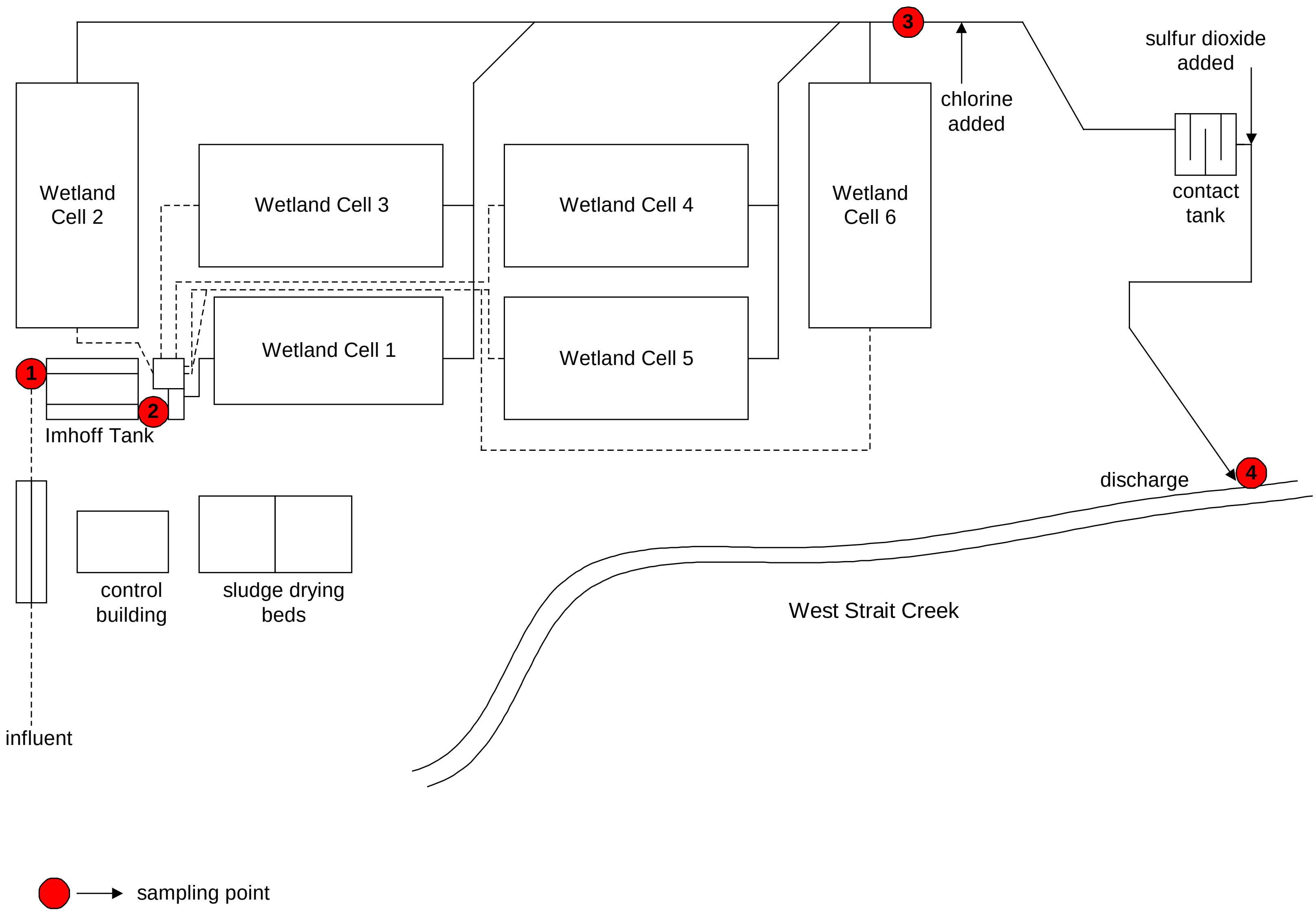

Figure 1 Treatment Configuration and Sampling Locations

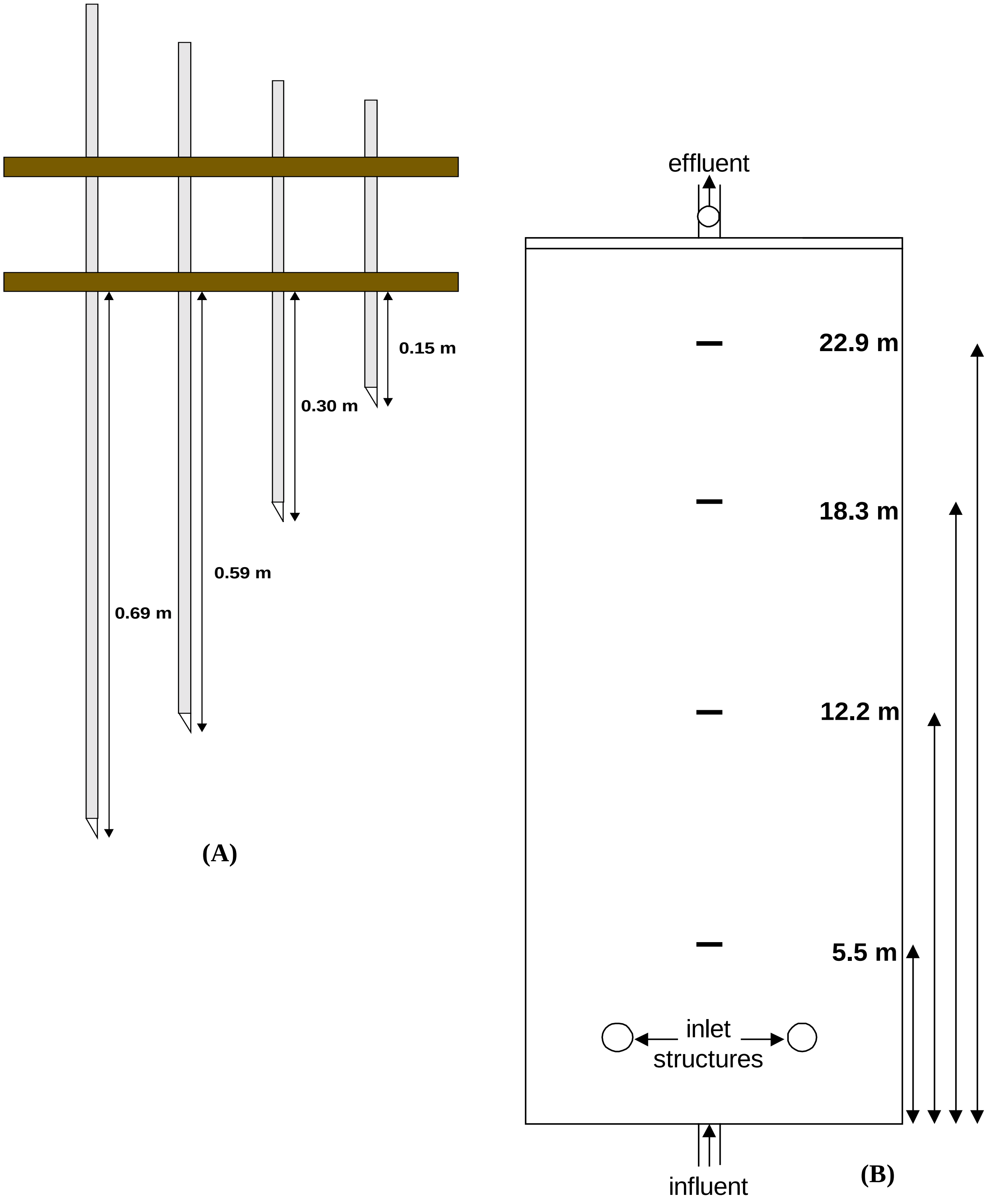

Figure 2 (A) Configuration of Depth Sampler and (B) Layout of Depth Samplers in Wetland Cell 6.

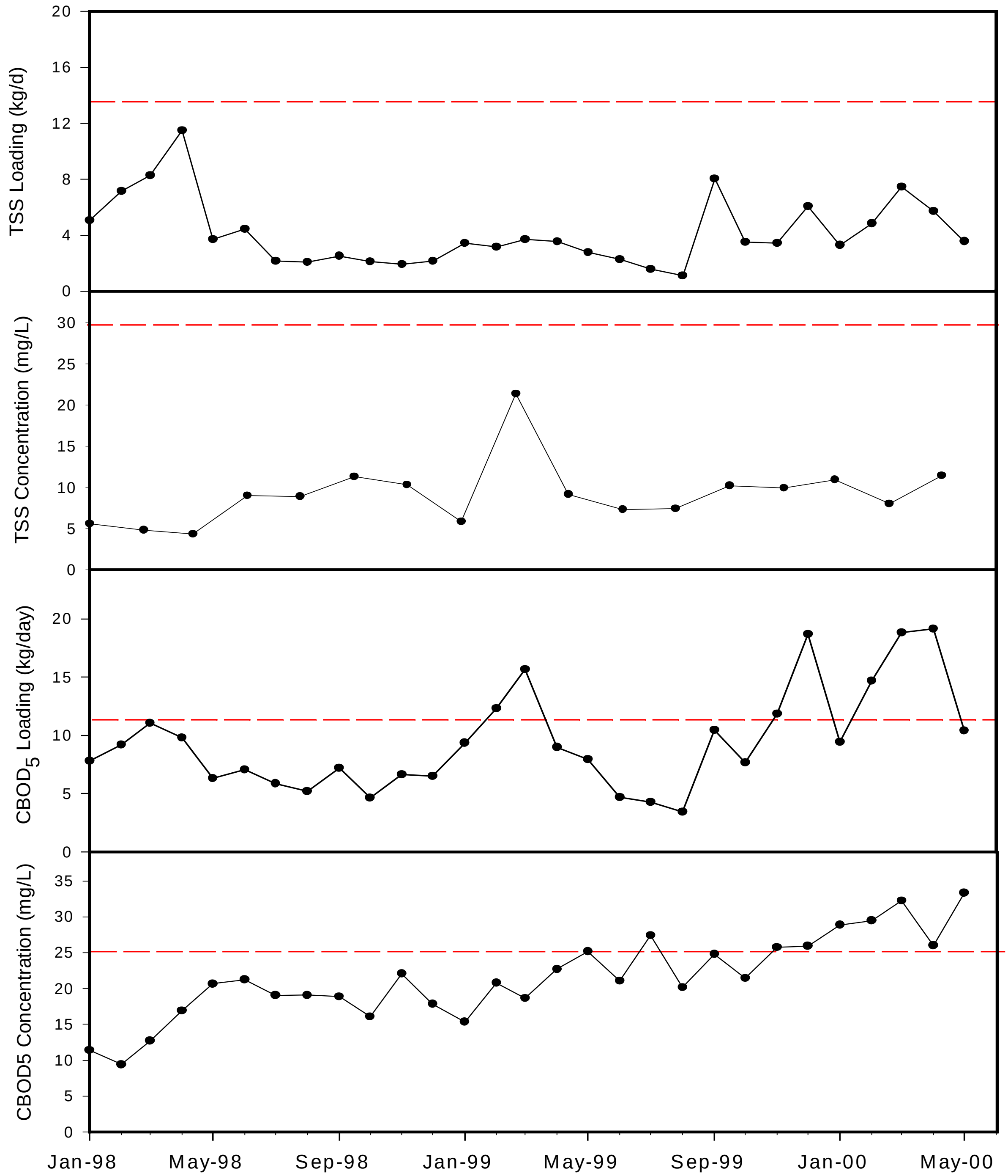

Figure 3 Average Monthly COD_5 and TSS Concentrations and Loadings as Reported on Discharge Monitoring Reports by Monterey Official for the Period of January 1998 - May 2000.

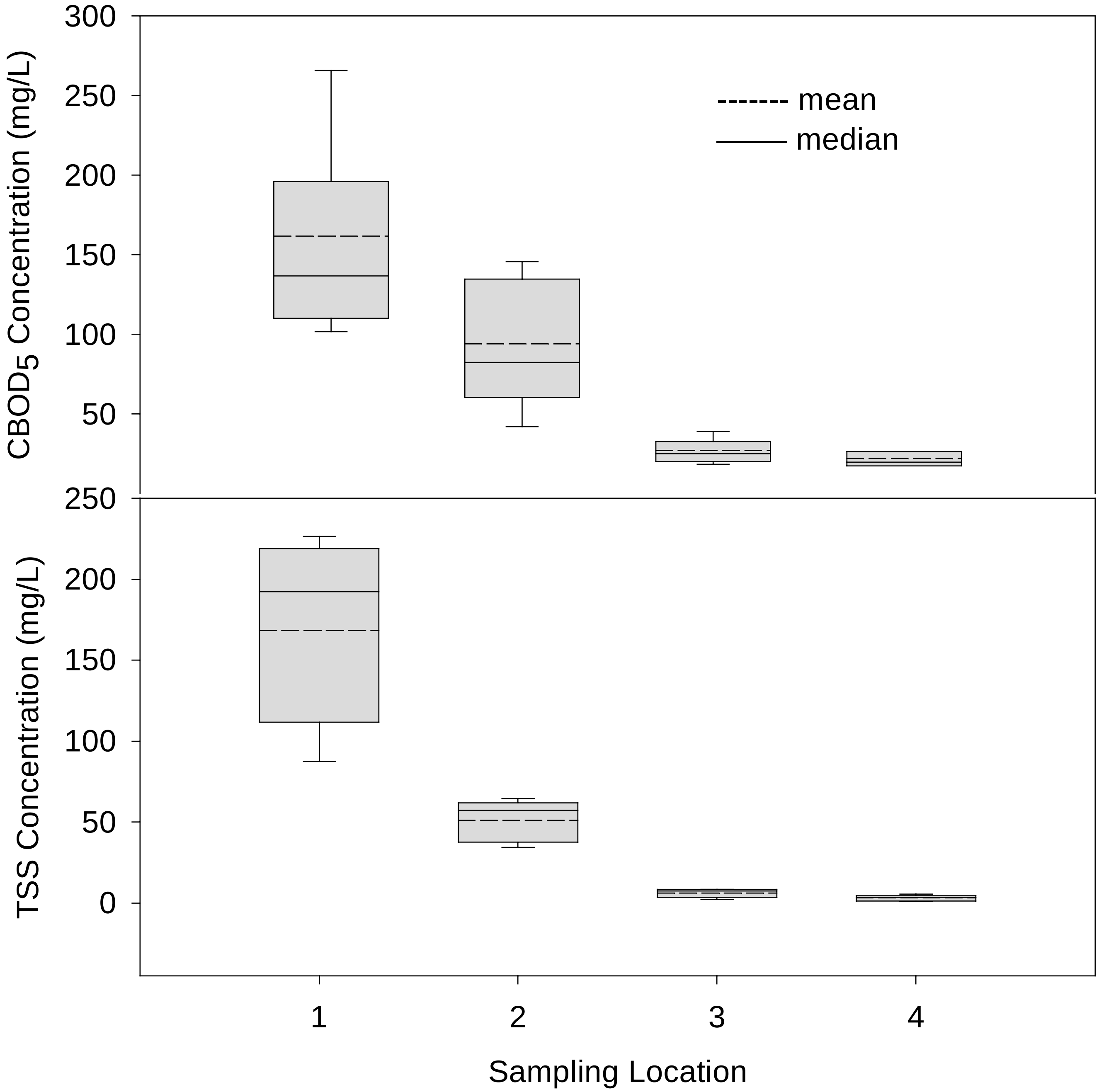

Figure 4 CBOD$_5$ and TSS Concentrations throughout the Treatment Process. The boxed area represents the 75th percentile to the 25th percentile. The upper bar represents the 95th percentile and the lower bar represents the 5th percentile.

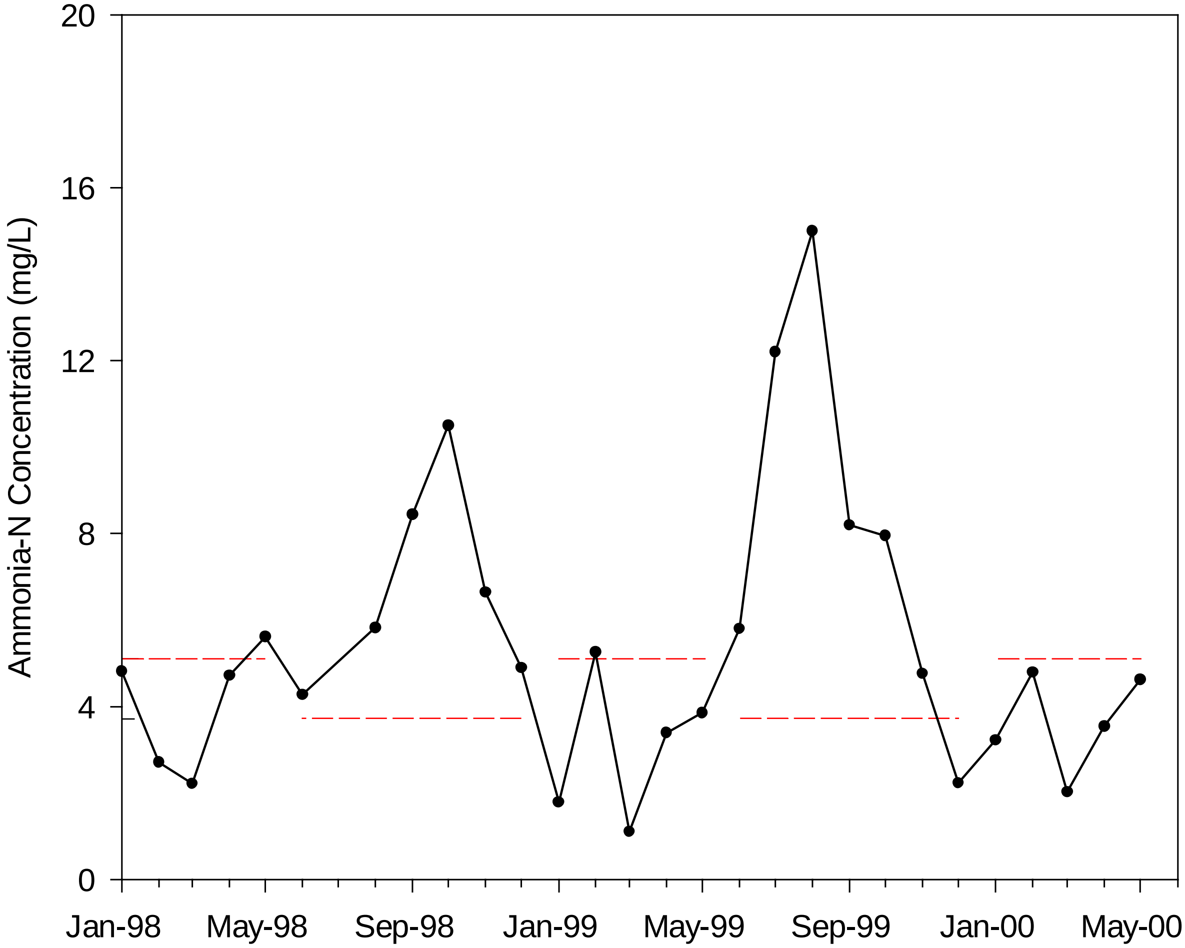

Figure 5 Effluent Ammonia-N Concentrations as Taken by Monterey Officials for the Period of January 1998 – April 2000.

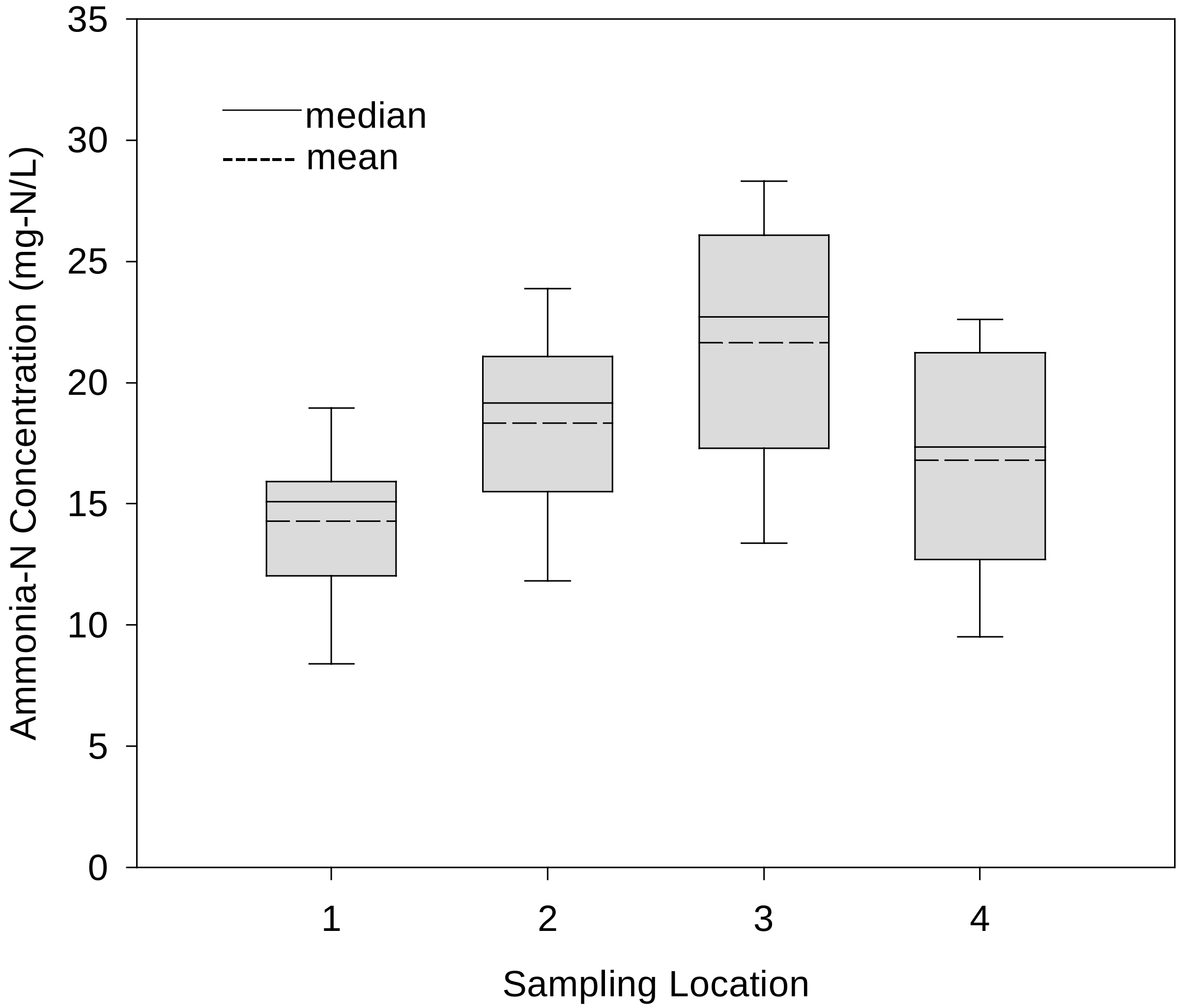

Figure 6 Ammonia-N Concentrations throughout the Treatment Process. The boxed area represents the 75th percentile to the 25th percentile. The upper bar represents the 95th percentile and the lower bar represents the 5th percentile.

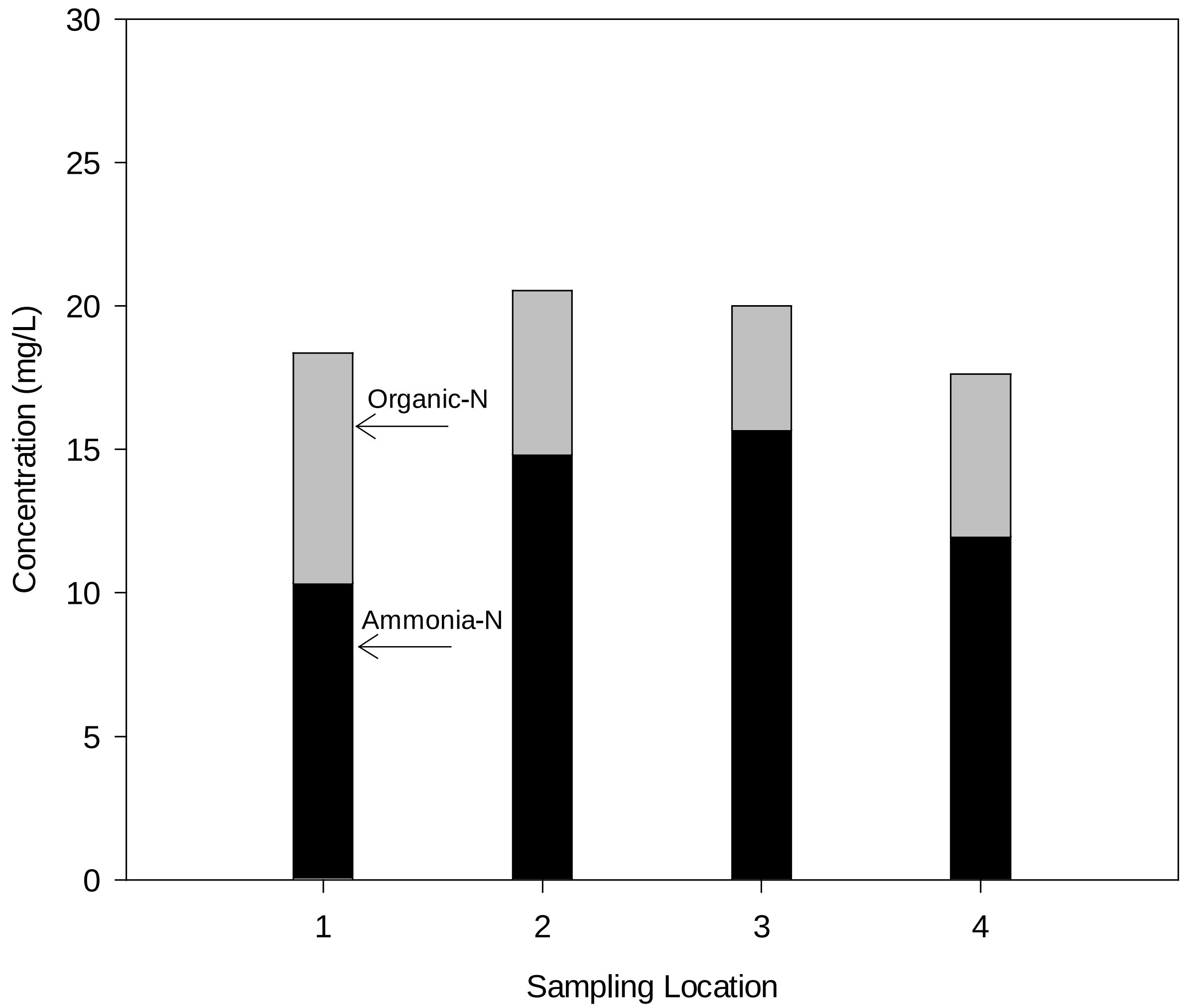

Figure 7 Nitrogen Distribution throughout the Treatment Process. Ammonia-N and Organic-N concentrations are averages of three sampling events.

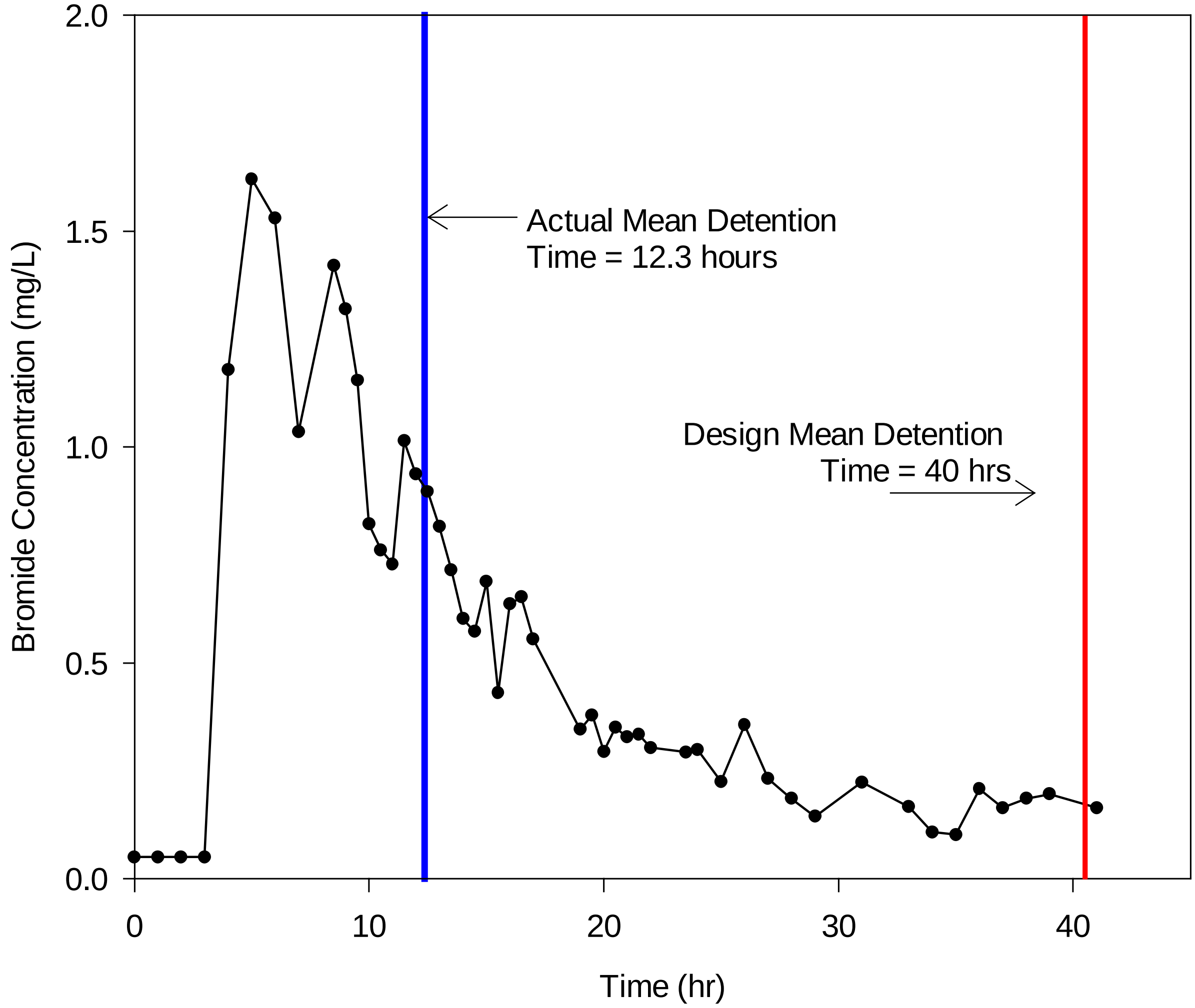

Figure 8 Concentration-time curve for the passage of a single pulse of bromide through Wetland Cell #6 at an average influent flow of 291 m^3/day.

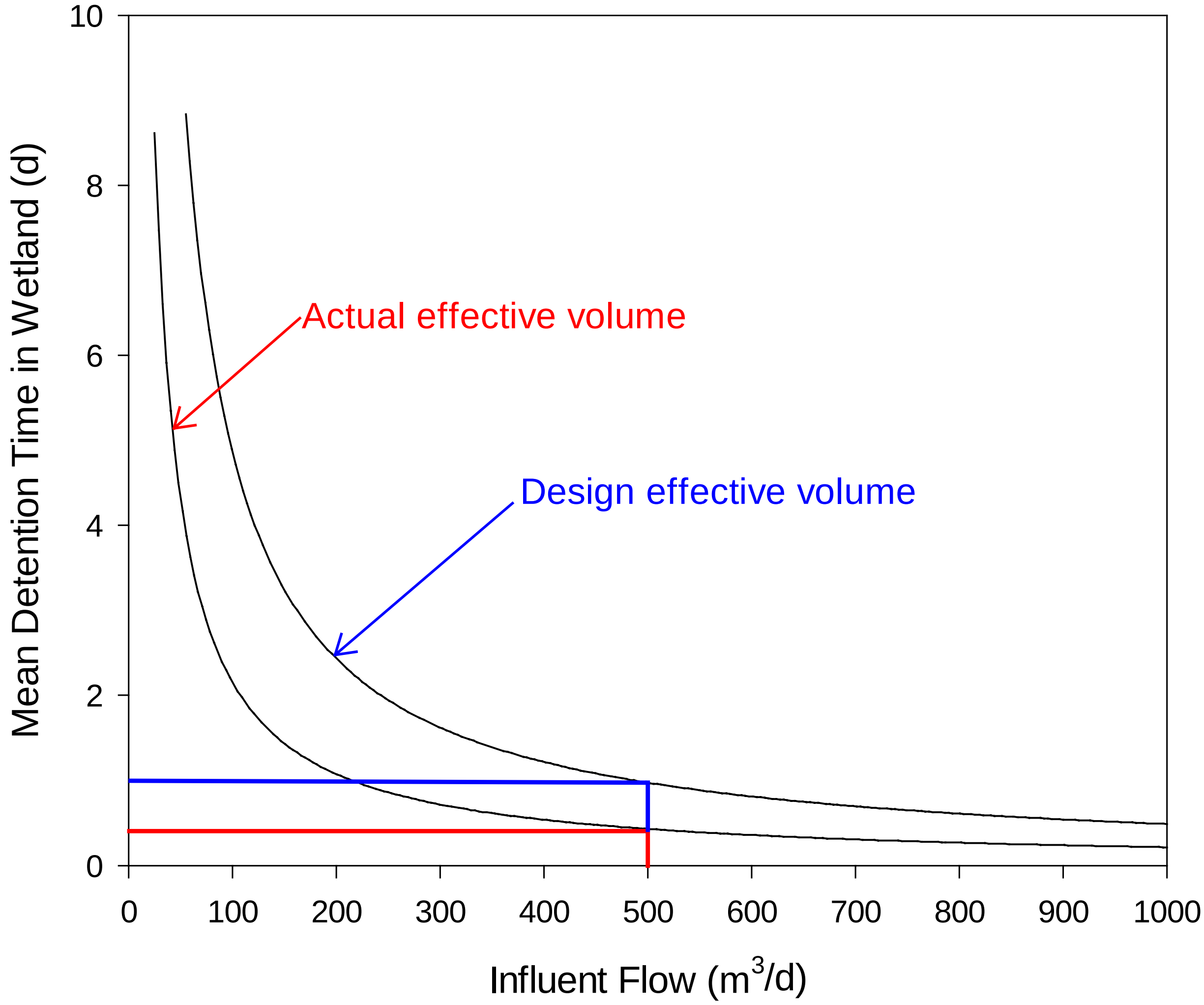

Figure 9 Relationship between Influent Flow and Mean Detention Time in Wetland Using Actual Effective Volume and Design Effective Volume.

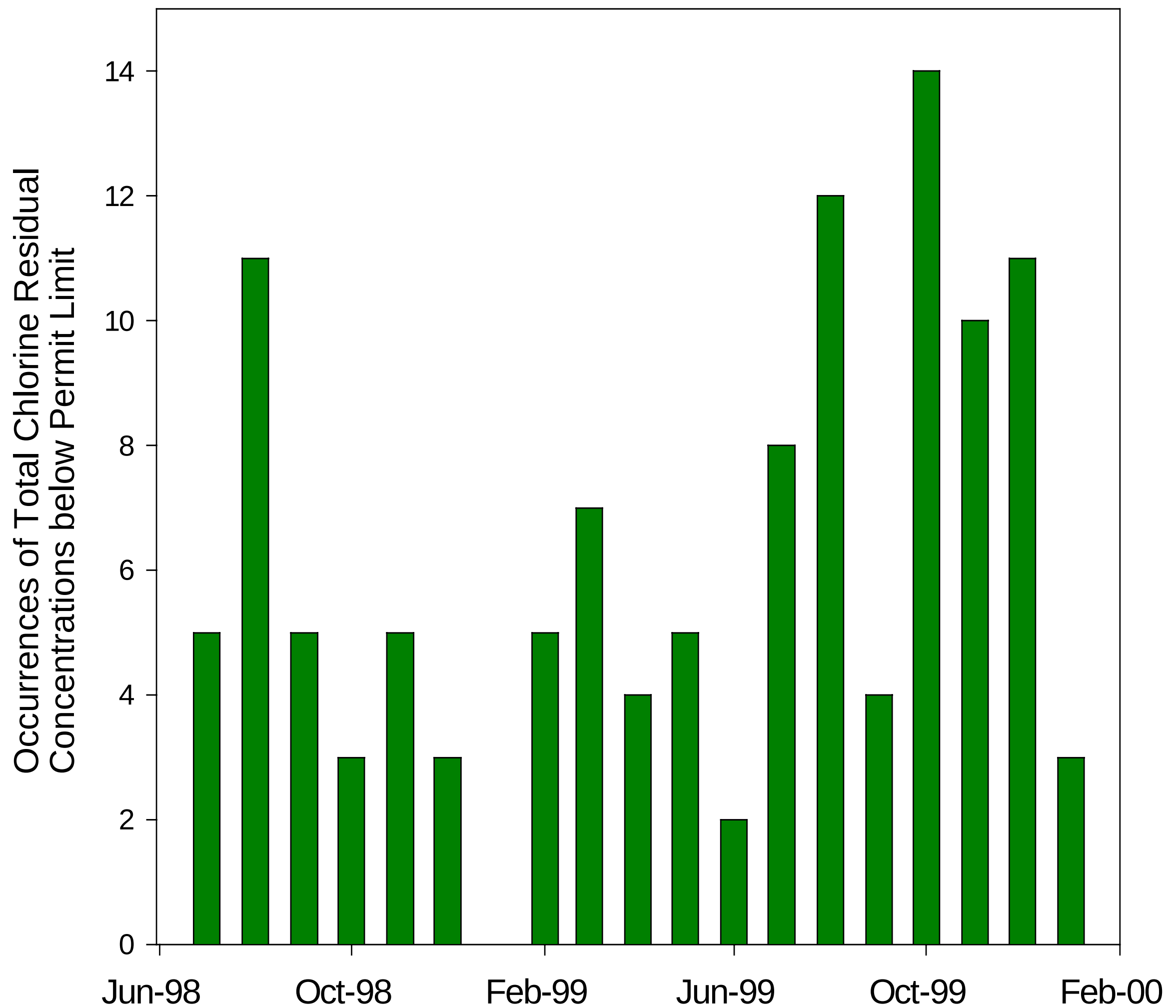

Figure 10 Monthly Occurrences of Total Chlorine Residual Concentrations below Permit Limit for Period of July 1998 - January 2000.

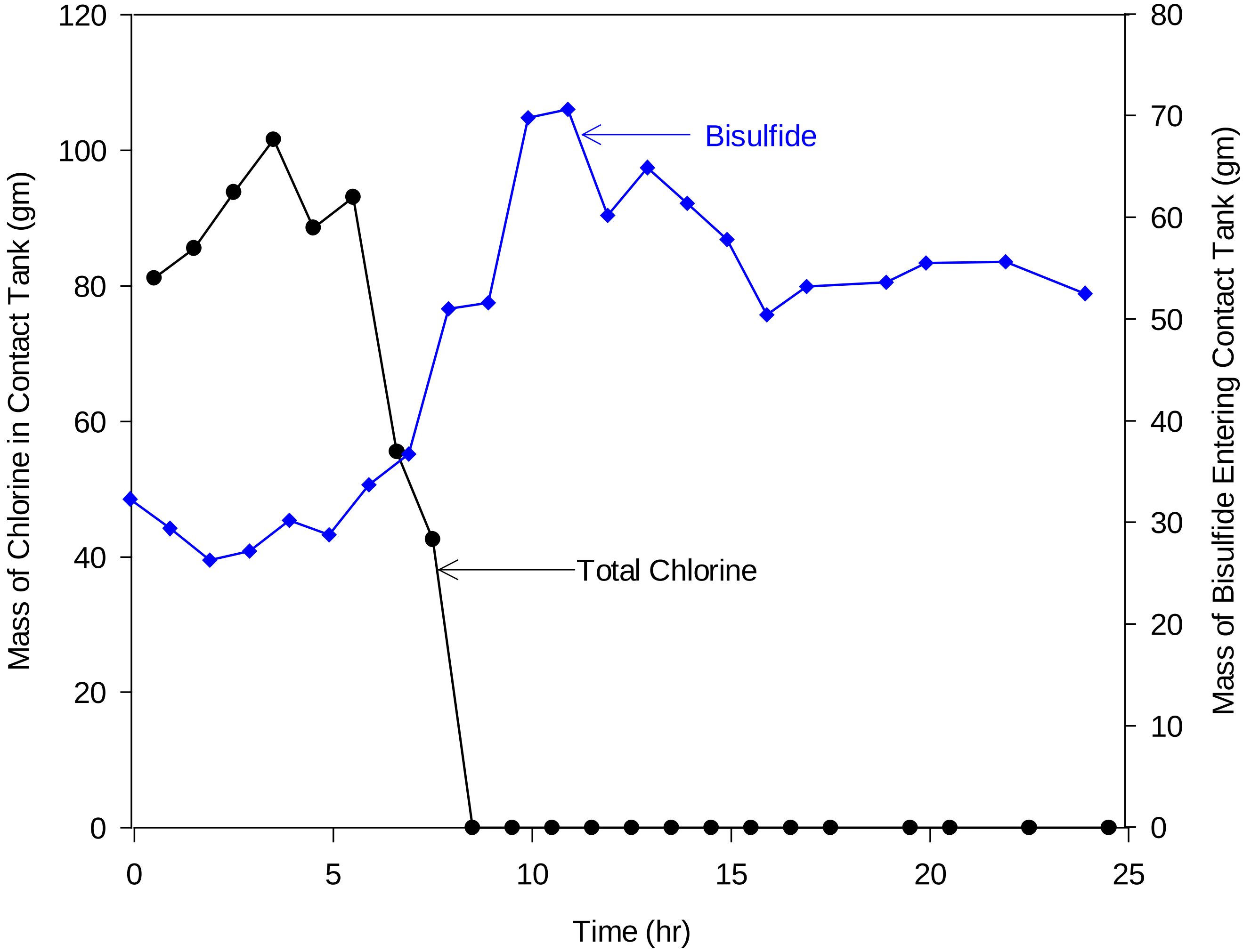

Figure 11 Relationship between Mass of Bisulfide Entering Contact Tank and Mass of Chlorine Measured in Contact Tank in Test 1.

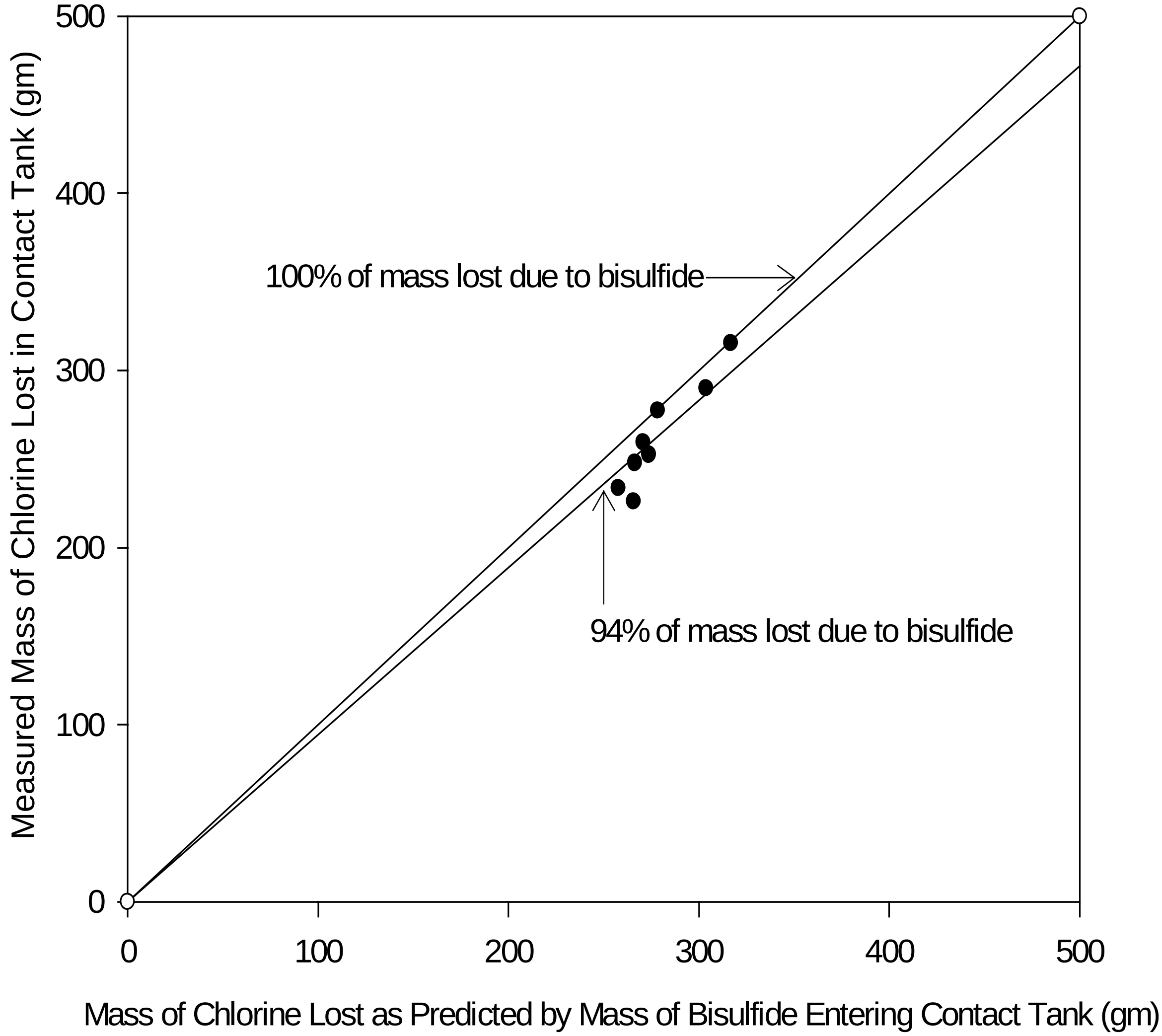

Figure 12 Extent to which the Presence of Bisulfide in the Wastewater Accounted for the Observed Mass of Chlorine Lost in Test 1.

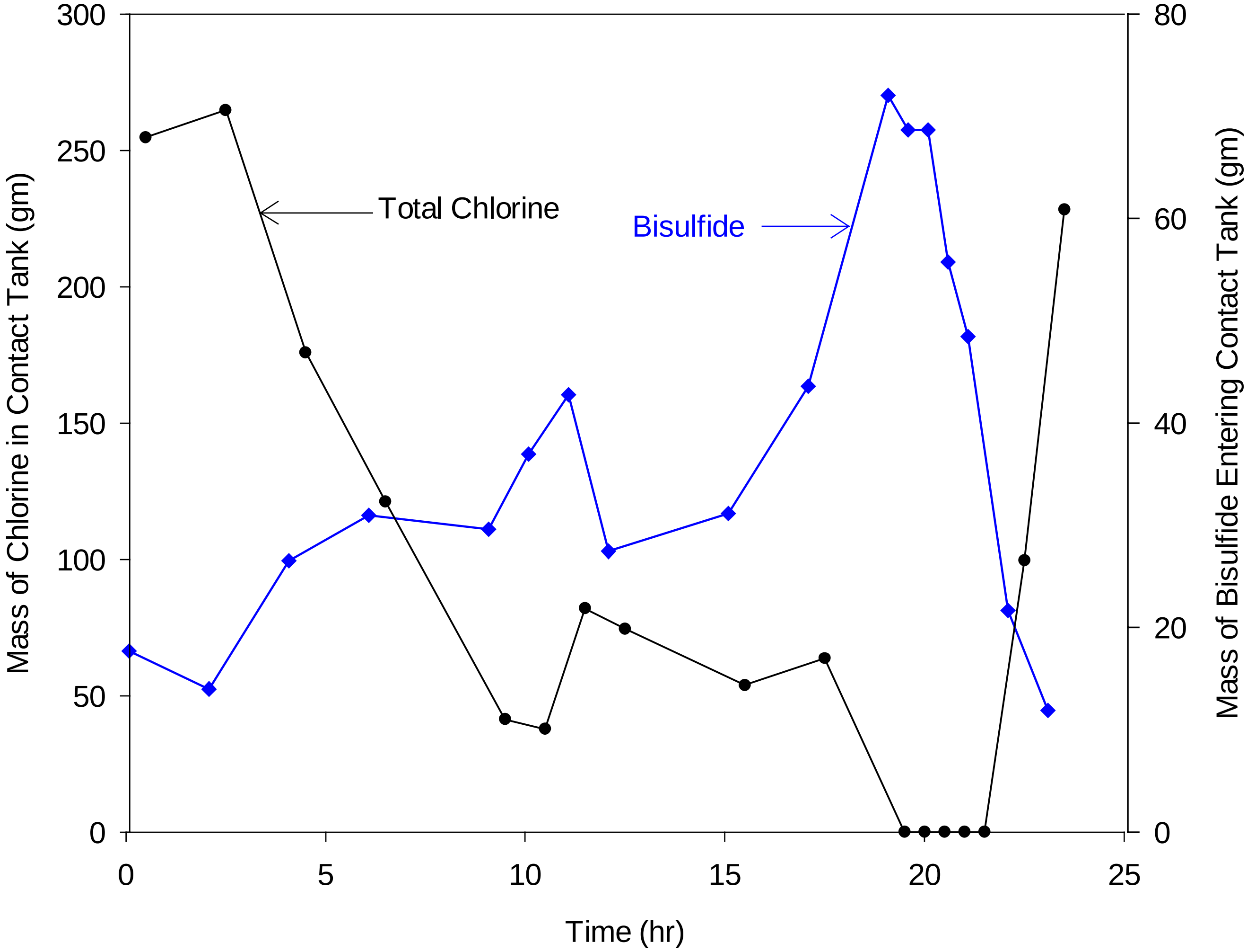

Figure 3 Relationship between Mass of Bisulfide Entering Contact Tank and Mass of Chlorine Measured in Contact Tank in Test 2.

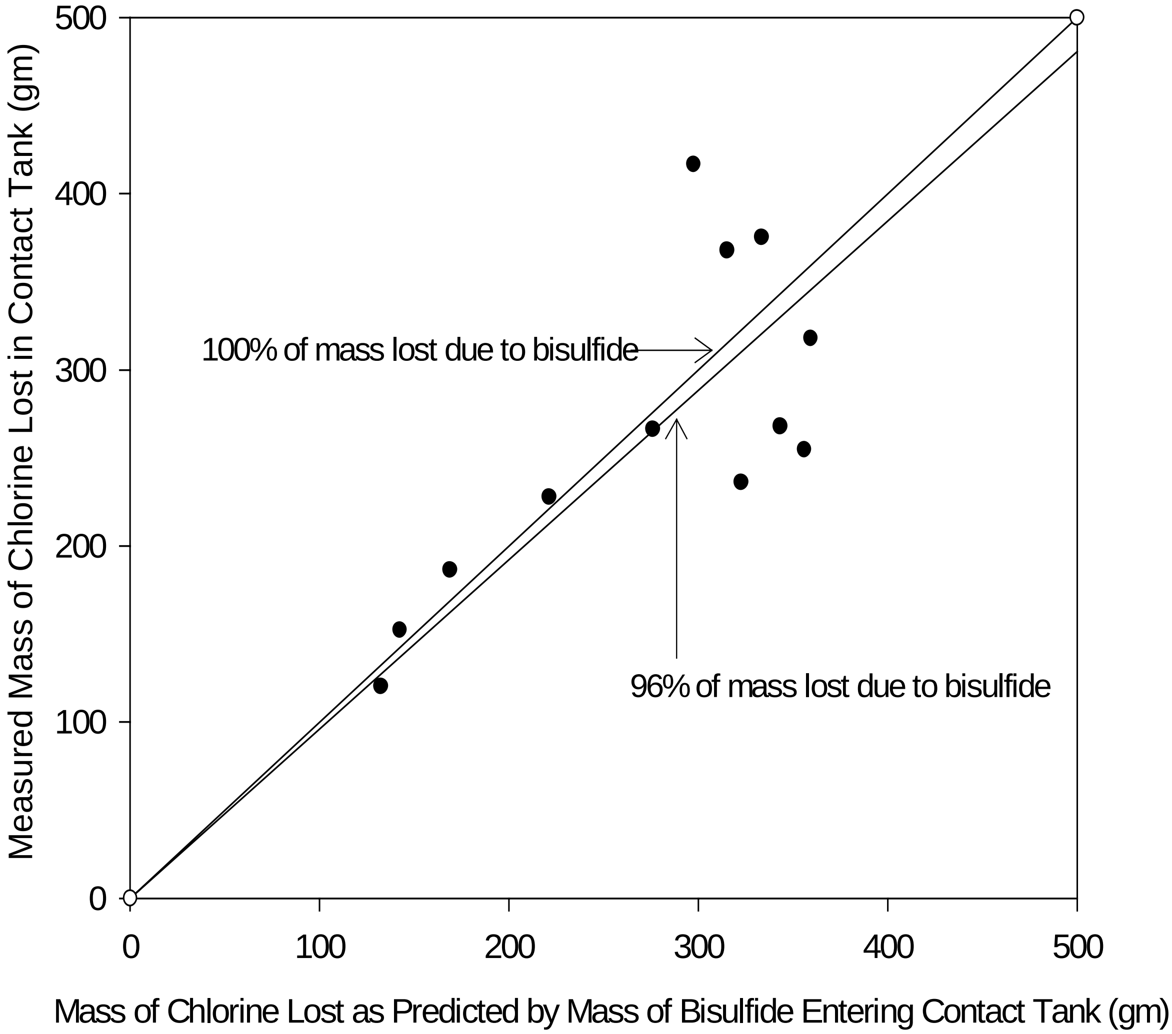

Figure 14 Extent to which the Presence of Bisulfide in the Wastewater Accounted for the Observed Mass of Chlorine Lost in Test 2.

CHAPTER 3

Permit Limitations and Monitoring Requirements

The Monterey WWTP is regulated by the Virginia Department of Environmental Quality in accordance with VPDES permit (#VA0023281). Whenever the average monthly influent flow is greater than 360 m^3/day (0.095 MGD) for three consecutive months, the permit limits and monitoring requirements found in Table 1 apply. Whenever the average monthly influent flow is less than 360 m^3/day (0.095 MGD), the permit limits and monitoring requirements found in Table 2 apply. Although there are permit limits listed for ammonia-N, the Monterey WWTP is not currently required to meet these limits by consent order of VDEQ.

The VPDES permit limits have been violated frequently at the Monterey WWTP. Table 3 contains the permit violations that occurred from July 1998 to January 2000. In this time period there were 84 monthly permit violations.

Table 1 Effluent Limitations and Monitoring Requirements for whenever the average monthly influent flow is greater than 360 m^3/day (0.095 MGD) for three consecutive months.

Parameter	Monthly Average		Maximum Weekly Average		Minimum	Maximum		Exceptions	Sample Type	Sample Frequency
Flow, m^3/day	NL		----		----	NL		0	T/I/R	Continuous
pH, S.U.	----		----		6.5	9.5		0	Grab	1/day
$CBOD_5$	25 mg/L	11.4 kg/d	40 mg/L	18.2 kg/d	----	----		0	8 hr composite	3 days/week
TSS	30 mg/L	13.6 kg/d	45 mg/L	20.4 kg/d	----	----		0	8 hr composite	3 days/week
D.O., mg/L	----		----		5.0	----		0	Grab	1/day
NH_3-N (Jan.1 – May 31)	5.1 mg/L	2.3 kg/d	----		----	5.1 mg/L	2.3 kg/d	0	8 hr composite	1/month
NH_3-N (June 1 – Dec. 31)	3.7 mg/L	1.7 kg/d	----		----	3.7 mg/L	1.7 kg/d	0	8 hr composite	1/month
Total Residual Chlorine – Contact, mg/L	----		----		1.0	----		9	Grab	3/day
Total Residual Chlorine Inst. Max., mg/L	----		----		----	ND		0	Grab	3/day
Total Residual Chlorine Tech. Max., mg/L	----		----		----	1.0		0	----	----
Total Residual Chlorine Tech. Min., mg/L	----		----		0.6	----		0	----	----

T/I/R = Totalizing, Indicating, and Recording Equipment
NL = No Limitation, but monitoring is required
ND = Non-detectable
A block containing ---- indicates not applicable.

Table 2 Effluent Limitations and Monitoring Requirements for whenever the average monthly influent flow is less than 360 m^3/day (0.095 MGD) for six consecutive months.

Parameter	Monthly Average		Maximum Weekly Average		Minimum	Maximum		Exceptions	Sample Type	Sample Frequency
Flow, m^3/day	NL		----		----	NL		0	T/I/R	Continuous
pH, S.U.	----		----		6.5	9.5		0	Grab	1/day
$CBOD_5$	25 mg/L	9.5 kg/d	40 mg/L	15.1 kg/d	----	----		0	8 hr composite	3 days/week
TSS	30 mg/L	11.4 kg/d	45 mg/L	17.0 kg/d	----	----		0	8 hr composite	3 days/week
D.O., mg/L	----		----		5.0	----		0	Grab	1/day
NH_3-N (Jan.1 – May 31)	5.1 mg/L	1.9 kg/d	----		----	5.1 mg/L	1.9 kg/d	0	8 hr composite	1/month
NH_3-N (June 1 – Dec. 31)	3.7 mg/L	1.4 kg/d	----		----	3.7 mg/L	1.4 kg/d	0	8 hr composite	1/month
Total Residual Chlorine – Contact, mg/L	----		----		1.0	----		9	Grab	3/day
Total Residual Chlorine Inst. Max., mg/L	----		----		----	ND		0	Grab	3/day
Total Residual Chlorine Tech. Max., mg/L	----		----		----	1.0		0	----	----
Total Residual Chlorine Tech. Min., mg/L	----		----		0.6	----		0	----	----

T/I/R = Totalizing, Indicating, and Recording Equipment
NL = No Limitation, but monitoring is required
ND = Non-detectable
A block containing ---- indicates not applicable.

Table 3. Violations of VPDES permit limts from July 1998 to January 2000

Month	Parameter	Concentration/Loading	Reported	Limit	Exceptions
July 1998	pH	Conc. Minimum	5.8 SU	6.5 SU	28
July 1998	Cl_2 Inst Res (Max)	Conc. Maximum	1.88 mg/L	Non-Detect	1
July 1998	Cl_2 Inst Res (Max) Tech	Conc. Maximum	1.88 mg/L	1.0 mg/L	1
July 1998	Inst. Cl_2 Tech Min Lim	Conc. Minimum	Non-Detect	0.6 mg/L	5
Aug. 1998	pH	Conc. Minimum	4.1 SU	6.5 SU	31
Aug. 1998	Total Cl_2 - Contact	Conc. Minimum	Non-Detect	1.0 mg/L	11
Aug. 1998	Total Cl_2 - Contact	Exceptions	11	9	
Aug. 1998	Cl_2 Inst Res (Max)	Conc. Maximum	2.2 mg/L	Non-Detect	2
Aug. 1998	Inst. Cl_2 Tech Min Lim	Conc. Minimum	Non-Detect	0.6 mg/L	11
Aug. 1998	Cl_2 Inst Res (Max) Tech	Conc. Maximum	2.2 mg/L	1.0 mg/L	2
Sept. 1998	pH	Conc. Minimum	5.5 SU	6.5 SU	24
Sept. 1998	Cl_2 Inst Res (Max)	Conc. Maximum	2.2 mg/L	Non-Detect	4
Sept. 1998	Cl_2 Inst Res (Max) Tech	Conc. Maximum	2.2 mg/L	1.0 mg/L	4
Sept. 1998	Inst. Cl_2 Tech Min Lim	Conc. Minimum	Non-Detect	0.6 mg/L	5
Oct. 1998	pH	Conc. Minimum	4.8 SU	6.5 SU	28
Oct. 1998	Cl_2 Inst Res (Max)	Conc. Maximum	1.82 mg/L	Non-Detect	1
Oct. 1998	Cl_2 Inst Res (Max) Tech	Conc. Maximum	1.82 mg/L	1.0 mg/L	1
Oct. 1998	Inst. Cl_2 Tech Min Lim	Conc. Minimum	Non-Detect	0.6 mg/L	3
Nov. 1998	pH	Conc. Minimum	4.5 SU	6.5 SU	29
Nov. 1998	Total Sus. Solids	Sampling Frequency	11/13	3D/W	2
Nov. 1998	$CBOD_5$	Sampling Frequency	11/13	3D/W	2
Nov. 1998	Inst. Cl_2 Tech Min Lim	Conc. Minimum	Non-Detect	0.6 mg/L	5
Dec. 1998	pH	Conc. Minimum	5.2 SU	6.5 SU	30

Month	Parameter	Concentration/Loading	Reported	Limit	Exceptions
Dec. 1998	Total Sus. Solids	Sampling Frequency	12/14	3D/W	2
Dec. 1998	$CBOD_5$	Sampling Frequency	12/14	3D/W	2
Dec. 1998	Inst. Cl_2 Tech Min Lim	Conc. Minimum	Non-Detect	0.6 mg/L	3
Dec. 1998	Cl_2 Inst Res (Max)	Conc. Maximum	2.2 mg/L	Non-Detect	1
Dec. 1998	Cl_2 Inst Res (Max) Tech	Conc. Maximum	2.2 mg/L	1.0 mg/L	1
Jan. 1999	pH	Conc. Minimum	5.2 SU	6.5 SU	30
Feb. 1999	pH	Conc. Minimum	5.28 SU	6.5 SU	28
Feb. 1999	Inst. Cl_2 Tech Min Lim	Conc. Minimum	Non-Detect	0.6 mg/L	5
March 1999	pH	Conc. Minimum	5.8 SU	6.5 SU	30
March 1999	Total Sus. Solids	Sampling Frequency	12/15	3D/W	3
March 1999	$CBOD_5$	Sampling Frequency	12/15	3D/W	3
March 1999	$CBOD_5$	Loading Average	15.66 kg/d	11.4 kg/d	3
March 1999	$CBOD_5$	Loading Maximum	22.00 kg/d	18.2 kg/d	3
March 1999	Inst. Cl_2 Tech Min Lim	Conc. Minimum	Non-Detect	0.6 mg/L	7
April 1999	pH	Conc. Minimum	5.5 SU	6.5 SU	30
April 1999	Inst. Cl_2 Tech Min Lim	Conc. Minimum	Non-Detect	0.6 mg/L	4
May 1999	pH	Conc. Minimum	5.6 SU	6.5 SU	31
May 1999	Inst. Cl_2 Tech Min Lim	Conc. Minimum	Non-Detect	0.6 mg/L	5
May 1999	$CBOD_5$	Conc. Average	25.17 mg/L	25 mg/L	1
June 1999	pH	Conc. Minimum	5.2 SU	6.5 SU	27
June 1999	Inst. Cl_2 Tech Min Lim	Conc. Minimum	Non-Detect	0.6 mg/L	2
June 1999	Cl_2 Inst Res (Max)	Conc. Maximum	2.2 mg/L	Non-Detect	3
June 1999	Cl_2 Inst Res (Max) Tech	Conc. Maximum	2.2 mg/L	1.0 mg/L	3
July 1999	pH	Conc. Minimum	3.8 SU	6.5 SU	29

Month	Parameter	Concentration/Loading	Reported	Limit	Exceptions
July 1999	Inst. Cl_2 Tech Min Lim	Conc. Minimum	Non-Detect	0.6 mg/L	8
July 1999	$CBOD_5$	Conc. Average	27.41 mg/L	25 mg/L	1
July 1999	Cl_2 Inst Res (Max)	Conc. Maximum	2.2 mg/L	Non-Detect	7
July 1999	Cl_2 Inst Res (Max) Tech	Conc. Maximum	2.2 mg/L	1.0 mg/L	7
Aug. 1999	pH	Conc. Minimum	4.5 SU	6.5 SU	21
Aug. 1999	Inst. Cl_2 Tech Min Lim	Conc. Minimum	Non-Detect	0.6 mg/L	12
Aug. 1999	Cl_2 Inst Res (Max)	Conc. Maximum	2.2 mg/L	Non-Detect	3
Aug. 1999	Cl_2 Inst Res (Max) Tech	Conc. Maximum	2.2 mg/L	1.0 mg/L	3
Aug. 1999	Total Cl_2 - Contact	Conc. Minimum	Non-Detect	1.0 mg/L	12
Sept. 1999	pH	Conc. Minimum	5.3 SU	6.5 SU	3
Sept. 1999	Inst. Cl_2 Tech Min Lim	Conc. Minimum	Non-Detect	0.6 mg/L	4
Sept. 1999	Cl_2 Inst Res (Max)	Conc. Maximum	2.2 mg/L	Non-Detect	4
Sept. 1999	Cl_2 Inst Res (Max) Tech	Conc. Maximum	2.2 mg/L	1.0 mg/L	4
Sept. 1999	$CBOD_5$	Loading Maximum	23.76 kg/d	18.2 kg/d	1
Oct. 1999	pH	Conc. Minimum	5.36 SU	6.5 SU	14
Oct. 1999	Inst. Cl_2 Tech Min Lim	Conc. Minimum	Non-Detect	0.6 mg/L	14
Oct. 1999	Cl_2 Inst Res (Max)	Conc. Maximum	2.2 mg/L	Non-Detect	1
Oct. 1999	Cl_2 Inst Res (Max) Tech	Conc. Maximum	2.2 mg/L	1.0 mg/L	1
Oct. 1999	Total Cl_2 - Contact	Conc. Minimum	Non-Detect	1.0 mg/L	14
Oct. 1999	Total Cl_2 - Contact	Exceptions	14	9	
Nov. 1999	pH	Conc. Minimum	4.49 SU	6.5 SU	26
Nov. 1999	Inst. Cl_2 Tech Min Lim	Conc. Minimum	Non-Detect	0.6 mg/L	14
Nov. 1999	Cl_2 Inst Res (Max)	Conc. Maximum	2.2 mg/L	Non-Detect	2
Nov. 1999	Cl_2 Inst Res (Max) Tech	Conc. Maximum	2.2 mg/L	1.0 mg/L	2

Month	Parameter	Concentration/Loading	Reported	Limit	Exceptions
Nov. 1999	Total Cl_2 - Contact	Conc. Minimum	Non-Detect	1.0 mg/L	10
Nov. 1999	Total Cl_2 - Contact	Exceptions	10	9	
Nov. 1999	$CBOD_5$	Loading Average	11.85 kg/d	11.4 kg/d	2
Nov. 1999	$CBOD_5$	Conc. Average	25.75 mg/L	25 mg/L	2
Dec. 1999	pH	Conc. Minimum	3.19 SU	6.5 SU	30
Dec. 1999	Total Sus. Solids	Loading Maximum	26.44 kg/d	20.4 kg/d	1
Dec. 1999	Total Cl_2 - Contact	Conc. Minimum	Non-Detect	1.0 mg/L	11
Dec. 1999	Total Cl_2 - Contact	Exceptions	11	9	
Dec. 1999	$CBOD_5$	Loading Average	18.68 kg/d	11.4 kg/d	3
Dec. 1999	$CBOD_5$	Loading Maximum	43.95 kg/d	18.2 kg/d	3
Dec. 1999	$CBOD_5$	Conc. Average	25.90 mg/L	25 mg/L	3
Dec. 1999	Inst. Cl_2 Tech Min Lim	Conc. Minimum	Non-Detect	0.6 mg/L	11
Jan. 2000	pH	Conc. Minimum	2.98 SU	6.5 SU	31
Jan. 2000	$CBOD_5$	Conc. Average	28.85 mg/L	25 mg/L	1
Jan. 2000	Inst. Cl_2 Tech Min Lim	Conc. Minimum	Non-Detect	0.6 mg/L	3
Jan. 2000	Cl_2 Inst Res (Max)	Conc. Maximum	2.2 mg/L	Non-Detect	1
Jan. 2000	Cl_2 Inst Res (Max) Tech	Conc. Maximum	2.2 mg/L	1.0 mg/L	1

CHAPTER 4

Recommendations for the Town of Monterey, VA

The wetland system at Monterey is not performing satisfactorily. The system needs to be improved and this can be accomplished using several approaches. The approach taken will depend on the likelihood that the VDEQ will set an ammonia limit for the effluent.

If ammonia removal (nitrification) is required, it may be appropriate to abandon the existing wetland system and add an activated sludge system, most likely a package plant, to achieve BOD and TSS removal and also nitrification. The wetlands may remain in operation as an overflow unit to treat the wastewater during high flow conditions.

If ammonia removal is not required, it is possible that the wetlands can be renovated and successfully operated for several years before renovation is again needed. The wetland beds appear to have become clogged with solids. Excavating and cleaning the media may remove the accumulated solids from the system. Without remedying the overloading of the wetlands, clogging is likely to occur again, which would require additional renovation in the future.

If the wetlands remain in operation, it is necessary to address the removal of bisulfide. The bisulfide produced in the anaerobic wetlands should be removed prior to chlorination. Removing the bisulfide will not only reduce the chlorine and sulfur dioxide requirements dramatically, but should also alleviate the frequent violations of nondetectable total chlorine residual concentration in the chlorine contact tank and eliminate the pH decrease. The bisulfide can be removed through aeration of the wastewater. Two possible methods are (1) an aeration basin between the manhole and chlorination point in which utilized diffused air blowers, (2) a cascade, which is a flight of three or four concrete steps over which the wastewater flows as a thin sheet.

There is a need for future work at the Monterey WWTP to evaluate possible solutions to the problems currently being encountered. A possible study may include renovation of two wetland cells to determine the effectiveness of excavation and cleaning of the media. After this study, a cost comparison could be made between renovating the entire wetland system and installing a package plant.

The above study may include operation of the wetland cells in series. Significant removal of solids and BOD due to sedimentation and filtration may occur in the first two wetland cells, thereby leaving the remaining cells unclogged and more effective at BOD removal. Renovation of only the first two wetland cells may then be needed on a periodic basis.

If a package plant is installed at Monterey, the one or more of the wetland cells may be used for denitrification. Research studying the denitrification efficiency of the wetlands and also the production of bisulfide under the denitrifying conditions may need to be performed.

APPENDIX A

ANALYTICAL DATA

Table A-1 Effluent Characteristics as Reported on Monthly Discharge Monitoring Reports Submitted to VDEQ

Month	Flow (MGD)	$CBOD_5$ Concentration (mg/L)	$CBOD_5$ Loading to West Strait Creek (kg/d)	TSS Concentration (mg/L)	TSS Loading to West Strait Creek (kg/d)	Ammonia Concentration (mg/L)
Jan-98	0.262	11.40	7.80	7.75	5.05	4.81
Feb-98	0.295	9.40	9.20	7.00	7.15	2.71
Mar-98	0.260	12.71	11.03	7.14	8.29	2.21
Apr-98	0.175	16.91	9.79	14.33	11.48	4.72
May-98	0.102	20.66	6.33	12.75	3.71	5.61
Jun-98	0.110	21.22	7.06	12.33	4.45	4.27
Jul-98	0.074	19.07	5.87	8.86	2.15	----
Aug-98	0.075	19.08	5.21	7.58	2.09	5.82
Sep-98	0.085	18.86	7.21	6.79	2.53	8.44
Oct-98	0.083	16.07	4.65	7.15	2.11	10.50
Nov-98	0.081	22.09	6.63	6.55	1.92	6.64
Dec-98	0.088	17.83	6.51	5.83	2.15	4.89
Jan-99	0.217	15.33	9.35	5.58	3.45	1.78
Feb-99	0.156	20.79	12.29	4.83	3.16	5.25
Mar-99	0.234	18.66	15.66	4.33	3.70	1.10
Apr-99	0.116	22.71	8.98	9.00	3.54	3.39
May-99	0.089	25.17	7.93	8.92	2.78	3.85
Jun-99	0.062	21.07	4.70	11.33	2.29	5.80
Jul-99	0.044	27.41	4.27	10.33	1.59	12.20
Aug-99	0.062	20.15	3.45	5.85	1.11	15.00
Sep-99	0.128	24.77	10.44	21.38	8.05	8.20
Oct-99	0.104	21.45	7.67	9.18	3.50	7.95
Nov-99	0.120	25.75	11.85	7.33	3.44	4.76
Dec-99	0.134	25.90	18.68	7.45	6.06	2.23
Jan-00	0.090	28.85	9.41	10.23	3.30	3.22
Feb-00	0.172	29.46	14.68	9.92	4.85	4.79
Mar-00	0.168	32.23	18.81	10.95	7.45	2.02
April-00	0.150	26.12	19.12	8.00	5.73	3.54
May-00	0.089	33.31	10.39	11.46	3.57	4.62

- Flow, $CBOD_5$ and TSS values are monthly averages

Table A-2 $CBOD_5$ Concentrations (mg/L)

Sampling Location	Sampling Date						
	Jun-12	Jun-21	Jun-30	Aug-9	Nov-27	Jan-8	Feb-3
1	200.0	184.8	281.9	136.7	106.3	100.8	121.1
2	147.0	140.0	119.6	82.5	38.9	75.9	55.2
3	40.6	22.1	32.4	25.0	18.0	32.8	19.6
4	---	---	---	21.9	17.1	31.0	18.0

Table A-3 TSS Concentrations (mg/L)

Sampling Location	Sampling Date				
	Jun-12	Jun-21	Nov-27	Dec-8	Jan-8
1	192.21	216.34	226.47	119.7	87.50
2	60.78	57.24	64.50	34.3	38.78
3	2.17	4.03	8.43	7.40	8.58
4	1.35	0.94	4.25	5.60	3.53

Table A-4 Ammonia-N Concentrations (mg/L)

Sampling Location	Sampling Date							
	Jun-12	Jun-21	Jun-30	Jul-8	Jul-23	Dec-8	Feb-3	Jun-5
1	15.12	15.60	20.11	15.05	16.24	9.21	8.06	14.84
2	19.88	20.23	24.73	18.46	21.92	10.46	14.95	16.07
3	29.15	21.43	23.98	25.81	26.32	11.89	17.81	16.80
4	20.97	16.60	23.08	18.06	21.48	8.83	14.28	11.14

Table A-5 Nitrogen Distribution Data

	Sampling Date					
	Feb-3		Jun-5		Jun-27	
Sampling Location	NH_3-N (mg/L)	Organic-N (mg/L)	NH_3-N (mg/L)	Organic-N (mg/L)	NH_3-N (mg/L)	Organic-N (mg/L)
1	8.064	12.32	14.84	1.74	8.12	10.02
2	14.95	10.36	16.07	3.19	13.496	3.53
3	17.81	3.47	16.80	4.48	12.376	5.10
4	14.28	5.21	11.14	7.45	10.472	4.31

Table A-6 Bromide Tracer Study Data

Time (hr)	Bromide Concentration (mg/L)	Flow (L/min)
0	0.05	55.4
1	0.05	55.8
2	0.05	55.8
3	0.05	54.4
4	1.18	59.8
5	1.62	56.3
6	1.53	53.1
7	1.03	57.5
8.5	1.42	57.5
9	1.32	50.3
9.5	1.15	50.3
10	0.82	50.3
10.5	0.76	50.3
11	0.73	45.7
11.5	1.01	50
12	0.94	50.4
12.5	0.90	46.7
13	0.82	46.7
13.5	0.71	46.7
14	0.60	46.7
14.5	0.57	46.7
15	0.69	46.7
15.5	0.43	46.7
16	0.64	46.7
16.5	0.65	46.7
17	0.55	46.7
19	0.35	46.7
19.5	0.38	56.4
20	0.29	57.6
20.5	0.35	57.6
21	0.33	49.5
21.5	0.33	55.3
22	0.30	53.1
23.5	0.29	50.4
24	0.30	50.4
25	0.23	46.7
26	0.36	45.4
27	0.23	45.4
28	0.19	45.4
29	0.14	45.5
31	0.22	42.7
33	0.17	41.7
34	0.11	38.1
35	0.10	38.1
36	0.21	38.1
37	0.16	38.1
38	0.19	38.1
39	0.20	38.1
41	0.16	38.1

Table A-7 Sulfide Test 1 Data

Time (hr)	Bisulfide Conc. (mg/L)	Total Chlorine Conc. (mg/L)	Influent Flow (MGD)
0	2.06	6.00	0.181
1	1.85	6.58	0.181
2	1.83	6.84	0.188
3	2.16	6.34	0.177
4	2.09	6.75	0.175
5	2.45	4.03	0.175
6	2.76	3.20	0.169
7	3.80	ND	0.170
8	3.84	ND	0.170
9	4.81	ND	0.184
10	5.52	ND	0.162
11	4.47	ND	0.171
12	4.76	ND	0.173
13	4.80	ND	0.162
14	4.52	ND	0.162
15	4.32	ND	0.148
16	4.55	ND	0.148
18	4.96	ND	0.137
19	5.50	ND	0.128
21	5.18	ND	0.136
23	4.21	ND	0.158

Time (hr)	Mass of Bisulfide Entering CT (gm)	Mass of Chlorine Expected in CT (gm)	Mass of Chlorine Measured in CT (gm)	Mass of Chlorine Lost (gm)	Mass of Chlorine Lost Due to Bisulfide (gm)
0	29.4	359.4	85.5	273.9	252.5
1	26.3	359.4	93.8	265.6	226.1
2	27.2	359.4	101.6	257.8	233.7
3	30.2	359.4	88.5	270.9	259.3
4	28.8	359.4	93.1	266.4	247.8
5	33.7	359.4	55.5	303.9	290.1
6	36.7	359.4	42.5	316.9	315.5
7	51.0	359.4	ND	----	----
8	51.6	359.4	ND	----	----
9	69.8	359.4	ND	----	----
10	70.6	359.4	ND	----	----
11	60.2	359.4	ND	----	----
12	64.9	359.4	ND	----	----
13	61.4	359.4	ND	----	----
14	57.8	359.4	ND	----	----
15	50.4	359.4	ND	----	----
16	53.2	359.4	ND	----	----
18	53.6	359.4	ND	----	----
19	55.5	359.4	ND	----	----
21	55.6	359.4	ND	----	----
23	52.5	359.4	ND	----	----

Table A-8 Sulfide Test 2 Data

Time (hr)	Bisulfide Conc. (mg/L)	Total Chlorine Conc. (mg/L)	Influent Flow (MGD)
0	1.43	20.50	0.158
1	1.23	23.25	0.144
3	2.26	15.00	0.149
4	2.64	10.34	0.149
8	2.87	4.00	0.131
9	3.58	3.67	0.131
10	3.76	7.20	0.144
11	2.41	6.55	0.144
14	3.41	5.90	0.116
16	3.83	5.60	0.144
18	3.51	ND	0.261
18.5	3.34	ND	0.261
19	2.79	ND	0.313
19.5	2.05	ND	0.346
20	1.62	ND	0.379
21	0.67	3.08	0.410
22	0.33	6.40	0.452

Time (hr)	Mass of Bisulfide Entering (gm)	Mass of Chlorine Expected in CT (gm)	Mass of Chlorine Measured in CT (gm)	Mass of Chlorine Lost (gm)	Mass of Chlorine Lost Due to Bisulfide (gm)
0	17.7	397.3	254.8	142.4	152.3
1	14.0	397.3	264.8	132.4	120.4
3	26.5	397.3	176.0	221.3	228.0
4	31.0	397.3	121.2	276.0	266.3
8	29.6	397.3	41.4	355.9	254.8
9	37.0	397.3	37.9	359.3	318.0
10	42.8	397.3	82.0	315.2	367.8
11	27.5	397.3	74.6	322.6	236.1
14	31.2	397.3	54.0	343.3	268.0
16	43.6	397.3	63.8	333.5	375.2
18	72.0	397.3	ND	----	----
18.5	68.7	397.3	ND	----	----
19	68.6	397.3	ND	----	----
19.5	55.7	397.3	ND	----	----
20	48.5	397.3	ND	----	----
21	21.7	397.3	99.6	297.6	416.8
22	11.9	397.3	228.3	169.0	186.4

APPENDIX B

ANALYTICAL METHODS

Total Residual Chlorine Concentration

Total Residual Chlorine Concentration was determined using a Hach pocket colorimeter provided by the Monterey WWTP staff.

1. A ten mL vial was filled to the 10 mL mark with the sample to be analyzed. This vial served as the blank.
2. Another ten mL vial was filled to the ten mL mark with the sample to be analyzed.
3. The contents of one DPD Total Chlorine reagent pillow was added to the sample vial.
4. The vial was capped and shook for 20 seconds.
5. The vial was allowed to sit undisturbed for at least three minutes, but no longer than five minutes (if chlorine was present a pink-red color developed).
6. The vial serving as the blank was wiped off using a Kim-wipe and inserted into the Hach colorimeter.
7. The cover was placed over the vial, and the colorimeter was zeroed by pushing the zero button.
8. After zeroing, the blank was removed and the sample vial was inserted after being wiped off.
9. The total chlorine concentration was determined by pushing the read button, which gave the concentration in units of mg/L.
10. Duplicates were performed for each total chlorine concentration analysis.
11. The limit of detection for this analytical method was 0.10 mg/L.

The Hach pocket colorimeter has an upper concentration limit of 2.2 mg/L. At sample concentrations above this limit, the colorimeter blinked the number 2.2 mg/L. In the case of high concentrations the following procedure was used:

1. An estimation of the total chorine concentration was made by visually inspecting the color that had developed in the sample vial.
2. In most cases a 1:50 dilution was made by pipetting one mL of sample into a 50 mL volumetric flask and then filling with nanopure water.
3. Duplicate dilutions were performed in each case.
4. After dilution, the procedure explained above was performed.
5. The total chlorine concentration from the colorimeter was divided by the dilution factor to obtain the actual total chlorine concentration.

Standards provided by the Monterey staff were used to ensure the accuracy of the pocket colorimeter. In all cases, the chlorine concentrations obtained for the standards were within the acceptable limits of accuracy.

Sulfide

Sulfide was determined using the Hach Model DR/700 Portable Colorimeter. Method 8131 (Methylene Blue) was performed using module 61.01 and program # 61.12.1. This method was applicable for the sulfide concentration range of 0 to 0.600 mg S^{2-}/L.

Method 8131 (Methylene Blue) for Sulfide Determination

1. A 25 mL vial was filled to the 25 mL mark with distilled water, and this vial served as the blank.
2. Another 25 mL vial was filled to the 25 mL mark with the sample to be analyzed.
3. One mL of Sulfide 1 reagent was added to each the blank and the sample vial.
4. Both vials were capped and swirled for ten seconds.
5. One mL of Sulfide 2 Reagent was added to each the blank and the sample vial.
6. Both vials were capped immediately and swirled for ten seconds.
7. The vials were allowed to sit undisturbed for five minutes.
8. If sulfide was present a blue color would develop.
1. The blank was used to zero the colorimeter by placing the vial into the colorimeter and pushing the zero button.
2. The vial containing the prepared sample was placed into colorimeter and the read button was pushed to obtain the sulfide concentration.
3. The limit of detection for this analytical method was 0.010 mg S^{2-}/L.

Hach Method 8131 has an upper concentration limit of 0.600 mg S^{2-}/L. At sample concentrations above this limit, the colorimeter blinked the number 0.766 mg S^{2-}/L. In the case of high concentrations the following procedure was used:

1. An estimation of the sulfide concentration was made by visually inspecting the color that had developed in the sample vial.
2. In most cases a 5:50 dilution was made by pipetting five mL of sample into a 50 mL volumetric flask and then filling with nanopure water.
3. Duplicate dilutions were performed in each case.
4. After dilution, the procedure explained above was performed.
5. The sulfide concentration from the colorimeter was divided by the dilution factor to obtain the actual sulfide concentration.

Standard sulfide solutions at concentrations of 0.5, 0.35, 0.1, 0.05, and 0.01 mg S^{2-}/L were prepared using Sodium Sulfide and nanopure water.

VITA

Brandon Dean Kiracofe was born August, 19 1976, in Harrisonburg, VA. He graduated in June 1994, from Buffalo Gap High School. His undergraduate degree was completed in May 1998 from Bridgewater College where he double majored in Chemistry and Biology. Brandon completed his Master's degree in Environmental Engineering from Virginia Tech in July 2000. He will begin work with the Department of Environmental Quality, Valley Regional Office in Harrisonburg, VA in July 2000.