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16. Abstract

In November of 1996 an interstate rest area was opened on I- 49, approximately 20 miles north of Opelousas, Louisiana. Wastewater generated in the main building as well as that from an RV dump station is treated using subsurface flow, rock plant filters preceded by two septic tanks in series. Treated effluent is disinfected and discharged to Lake Dubuisson. Discharge permit limits are 45 mg/liter biochemical oxygen demand (BOD) and 45 mg/liter total suspended solids (TSS). The purposes of this study were to (1) compare the hydraulic regime in"long narrow" filters with that in "short wide" filters, (2) compare the treatment effectiveness of this process so far as meeting existing and expected permit limitations with other waste treatment processes in use at Louisiana rest areas, and (3) assess the nature and amount of operation and maintenance required at this facility compared to that required at package mechanical plants. Results to date indicate that the system is insensitive to variations in flow and will meet or exceed it's permit requirements approximately 90 percent of the time. The system requires no proactive operation and minimal maintenance. Skill levels required are with the capabilities of current DOTD personnel. It is recommended that processes for the removal or conversion of ammonia be investigated in anticipation of future discharge limitations.

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Feasibility Study of a Rock Plant Filter Wastewater Treatment System

by
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LTRC Project No. 736-99-0217
State Project No. 91-8-GT

conducted for

Louisiana Department of Transportation and Development
Louisiana Transportation Research Center

July 2000

ABSTRACT

In November of 1996 an interstate rest area was opened on I-49, approximately 20 miles north of Opelousas, Louisiana. Wastewater generated in the main building that from an RV dump station is treated using subsurface flow, rock plant filters preceded by two septic tanks in series. Treated effluent is disinfected and discharged to Lake Dubuisson. Discharge permit limitations are 45 mg/liter biochemical oxygen demands (BOD) and 45 mg/liter total suspended solids (TSS).

This purposes of this study were to (1) compare the treatment efficiency of "long narrow" cells to "short wide" cells, (2) examine the effectiveness of this treatment process so far as meeting existing and expected permit limitations compared to other biological processes, and (3) assess the nature and amount of operation and maintenance required when compared to mechanical treatment systems used at other rest areas in Louisiana.

Results to date indicate that the facility can reliably meet its existing discharge permit over 90 percent of the time. The system requires no proactive operation and minimal maintenance. Skill levels required for maintenance are within the capabilities of current DOTD rest area personnel. It is recommended that processes for the removal or conversion of ammonia be investigated in anticipation of future discharge limitations.

ACKNOWLEDGMENTS

The authors wish to thank DOTD and LTRC for sponsoring this research; special thanks to Curtis Fletcher and to the rest area personnel for their assistance and patience during the conduct of this project. The authors would also like to thank the Project Review Committee for their counsel, ideas, and assistance.

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IMPLEMENTATION STATEMENT

Based on our search of the literature, the Grand Prairie rock plant filter is treating the strongest wastewater of any continuous flow system in the country. While we are, at present, using two cells, each 150' long by 30' wide (rock media 18" deep) we have been able to meet BOD and TSS permit limits reliably for 18 consecutive months using only the first cell.

The nature of the treatment facility (an interstate rest area) imposes wide swings in flow rate. Transit time flow meters at the filters routinely measure flow rates ranging from near zero at night to as much as 50 gpm during rainfall events. The median, long term flow rate for 1999 was approximately five gpm. Because of the septic tanks preceding the filters as well as the nature of the filters, wide swings in flow rate are effectively dampened and have not effected treatment efficiency as far as we can tell. If additional detention time should be required in the future, the depth of rock in the cell can be increased to provide it.

Contrary to what was initially expected, we have had no adverse effects from toxic substances being dumped into the system. However, the BOD and TSS concentrations measured at the pump station range from 200 to 1500 mg/liter on a routine basis. Such variation has not effected process efficiency.

Using only two sump pumps ($189 each) and $10 worth of PVC pipe, we have constructed a simple aeration device and, as a result, have been able to reduce ammonia in the effluent from around 50 mg/liter to between five and 15 mg/liter.

However, these results were obtained before the flow direction in cell 2 was changed and the orientation of the aeration device was modified. With the new configuration, we believe we can meet the expected limit of 5 mg/liter of ammonia in the effluent. Thus far, the addition of this process has resulted in no additional maintenance. However, power usage at the facility has increased. Electricity bills have increased from $10 per month to approximately $80 per month.

Based on these results we have no reason to believe we cannot reliably meet similar BOD and TSS permit limitations at any rest area in the state with a single cell, 150' long by 30' wide. This assumes there are properly sized septic tanks(s) preceding the cells and that flow rates, are not outrageous, compared to what we receive now.

The facility at Grand Prairie reliably produces an effluent containing less than 30 mg/liter BOD_5 and TSS while providing a low maintenance alternative to existing mechanical waste treatment systems. It contains a minimum of mechanical/electrical equipment and requires no operational expertise on the part of rest area personnel. The system has required only minimal maintenance, averaging no more than 30 minutes per day.

In order to minimize the potential for clogging, the filters should be preceded by a septic tank to serve as a settling basin. Media with a nominal dimension of 4.0 inches is recommended. Grand Prairie results demonstrate that a zero percent slope on the filter bottom results in maximum filter utilization without producing surface flow. A liner is required to prevent seepage into the groundwater. The liner should be extended up the sides of the filter, as necessary, to prevent migration of soil into the filter. The water surface elevation in the filter should be maintained far enough below the rock surface to avoid contact by sunlight, thereby minimizing algal production. Our experience suggests 4.0" (1 nominal dimension) or more below the rock surface. During the majority of the study period there were no plants in any of the cells, thus the effects of plants has not been fully investigated.

Based on discussions with DOTD personnel, the following problems, experienced at mechanical plants, are non-existent at the Grand Prairie facility: (1) activated sludge aeration control associated with blowers and timers, (2) secondary clarifier upsets spilling biological solids into the effluent and (3) algae production in the lagoons which follow some mechanical systems.

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INTRODUCTION

The first subsurface flow rock plant filter (RPF) in Louisiana was constructed in Haughton, LA in 1987. A rock plant filter is one example of a class of systems collectively referred to as “natural treatment systems”. It is simply a bed of rock media through which wastewater flows horizontally. It is usually preceded by a preliminary treatment process of some type, such as a lagoon. The term filter in the name is somewhat of a misnomer since little, if any physical filtration occurs in these systems. However, solids removal may occur as a result of settling. RPFs are essentially biological processes akin to a trickling filter. In both types of systems bacterial slimes, which accumulate on the surface of the media, utilize soluble organics in the waste flow as food, breaking them down to release energy for maintenance and cell production. Semi-aquatic plants are sometimes rooted on the surface of the bed. Plants produce oxygen as a waste product, which is released through the root systems. Initially, it was believed that this would maintain aerobic conditions in the bed; however, this has subsequently been shown not to occur. The impetus for the current interest in natural treatment systems in general, and rock plant filters in particular, apparently originated as a result of work by NASA scientists looking for a means of treating waste during space flights *[1]*.

Initially, RPF systems were hailed as a low maintenance alternative to the more expensive and labor intensive mechanical systems. They were touted as being able to produce effluents with low concentrations of biochemical oxygen demand (BOD) and total suspended solids (TSS). In Louisiana, lagoon/RPF systems were often used in situations where the waste discharge permit specified effluent BOD_5 and TSS concentrations of no more than 10 mg/liter. From 1985 until 1994, Region 6, of the EPA designated rock plant filters as “innovative treatment.” This meant that a municipality could apply for grants to cover up to 85 percent of the cost of designing and building such a system to treat its wastewater. If the system failed to meet its discharge permit, EPA would fund a replacement. As a result, a substantial number of these systems were built in Louisiana. They were used mainly in small towns, located on upland streams where pollutant discharge limits were quite strict. Many of these towns could not afford mechanical plants or the trained personnel to operate them.

Little research or data collection was done early-on in Louisiana to confirm the claims regarding the treatment effectiveness of rock plant filters. In the years since RPFs were introduced, analysis of regulatory data by Griffin, D. M., B.E. Price, and G. Pennison, concluded that there was no generally accepted design procedure for RPFs *[2]*. The average RPF in Louisiana violated its permit one out of every four months. Some had never met their discharge permit. For many, the media appeared to be clogged, as evidenced by surface flow.

Most plant operators were, and still are, only required to run tests on the plant effluent in order to demonstrate permit compliance. They are not required to run tests or report test results on the influent or internal process streams. Without such data, it was impossible to determine how much of a pollutant was actually being removed by a specific process *[2]*. In many cases, if a lagoon/RPF system produced an effluent low in BOD_5 and TSS, it was assumed that substantial removals had occurred (remember, no influent data). Therefore, such removals were often incorrectly attributed entirely to the RPF.

In the late 80s, DOTD began investigating the use of RPF technology as an alternative for mechanical treatment systems at Interstate highway rest areas. Existing mechanical systems were often in violation of their permit and were difficult for untrained rest area personnel to understand and operate. If the difficulties described above could be addressed, RFPs appeared to offer a low maintenance alternative. It required little, if any, proactive operation and could be maintained by DOTD personnel while reliably meeting discharge requirements.

OBJECTIVES

This project originally had two primary objectives:

1. To obtain operational and research data in order to evaluate and validate various aspects of the operation and maintenance of RPFs such as removal efficiencies for various waste constituents, the environment within a RPF as it affects waste treatment (aerobic or anaerobic), the potential for clogging, and the potential for use at interstate rest areas. It was also considered necessary to determine what differences, if any, existed in the design, operation and performance of a RPF with a high length to width ratio (aspect ratio) compared to one with a low length to width ratio.

2. To develop operation, maintenance, and training guidelines pertinent to the use of a RPF at an Interstate rest area. To compare the labor and skill necessary to operate and maintain a RPF as compared to the mechanical waste treatment systems currently used at interstate rest areas.

SCOPE AND METHODOLOGY

The Grand Prairie rest area was opened to the public with the RPF system placed in operation November 1, 1996. The treatment facility receives waste from the main building as well as an RV dump station. Waste drains to a pump station where it is pumped into two 8,000 gallon septic tanks piped in series. Waste can then flow to any of four RPF cells: cells 1 and 2, connected in series and having a L:W ratio of 1:5 and cells 3 and 4, in series haviing a L:W ratio of 5:1. All four cells are lined on the bottom and sides. Treated effluent is disinfected and discharged to Lake Dubuisson. Routine sample collection from the plant influent, plant effluent, and internal points in the treatment process began immediately on a biweekly basis and continues at this writing. The purpose of this program is to develop a data base for assessing the pollutant removal characteristics of each of the unit processes. Samples are returned to the Folk Laboratory at Louisiana Tech and analyzed for BOD_5, TSS, TKN, NH_3, NO_3^-, and org-N (by calculation). Tests for CBOD, VSS, TP, metals and organics are carried out as needed. Routine field analyses (pH, conductivity, dissolved oxygen, temperature, chlorine residual, fecal coliforms) at several points in the treatment system began about the same time. A schematic of the treatment facility with the sampling locations marked is shown in figure 1. All sampling, analysis and quality control procedures are carried out in strict conformance with standard methods *[3]*.

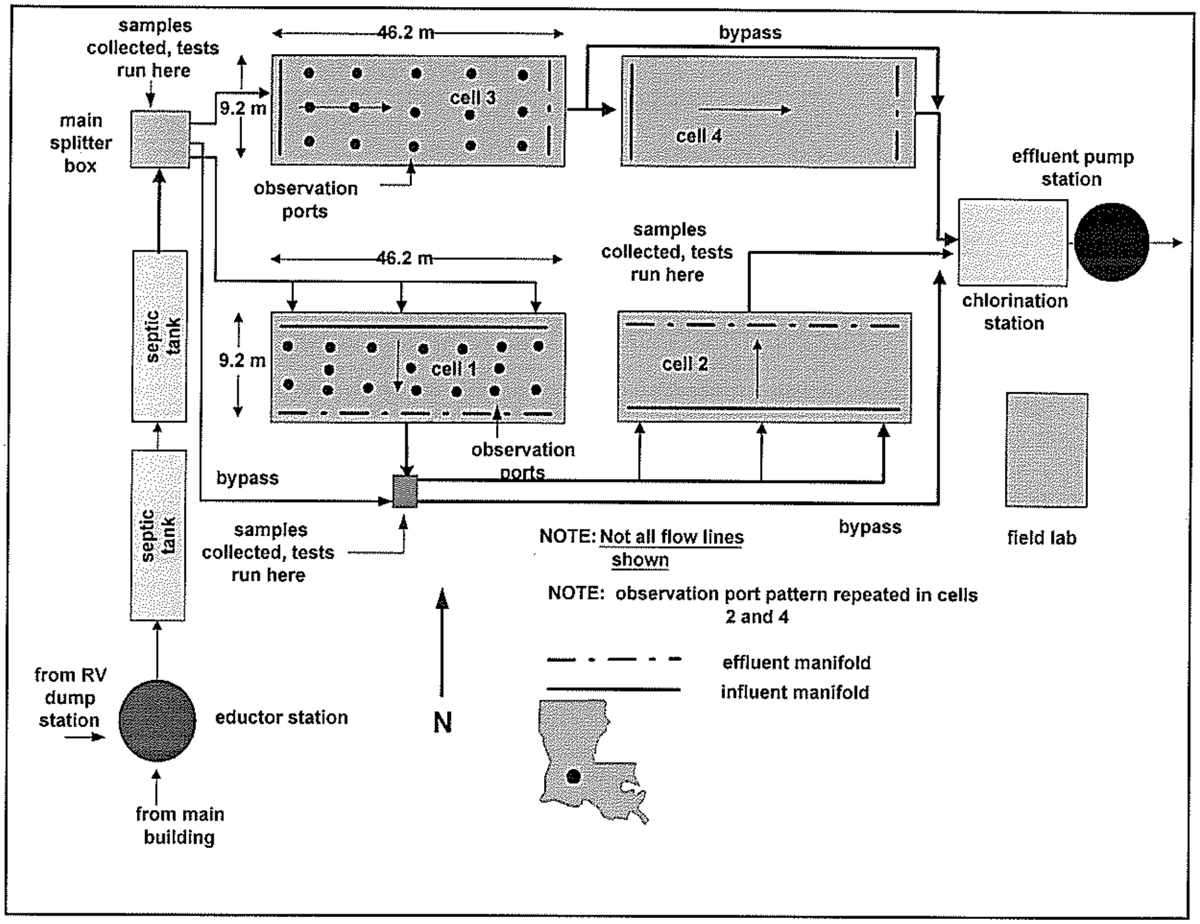

Figure 1
Layout of Grand Prairie treatment facility

To date, only cells 1 and 2 have been placed in operation because the flow has not increased to the point where all 4 cells are required. As of this writing, no data exists regarding the effectiveness of the plants in cell 2. Continuous monitoring of the pH at the plant influent was initially discussed but has not been implemented because the high alkalinity of the water (600-1000 mg/liter as $CaCO_3$) makes any significant variation in pH unlikely. This has been confirmed by ongoing pH measurements. Recently, a temperature probe was placed below the surface of the media in cell 1. It logs the waste temperature at 15 minute intervals and is downloaded every two weeks.

In order to investigate the possibility that daily water usage might be a reasonable surrogate measurement for waste flow, water meter readings are recorded daily at 5 A.M. from a totalizing water meter. This began when the facility opened to the public. However, in early 1997, the original meter was found to be both oversized and located in the wrong place which resulted in inaccurate readings during low flow periods. A new water meter was installed and accurate water use data has been collected since July 1, 1997.

Transit time flow meters were installed at the main splitter box and the splitter box receiving effluent from cell 1. These meters log the waste flow rate at one minute intervals. This allows water and waste volumes to be compared. A recording rain gauge is used to determine the time of occurrence, duration, intensity, and amount of rainfall. Rainfall data is used when necessary to help explain otherwise anomalous behavior in the waste flow data. Data from these devices is downloaded approximately every two weeks.

Field measurements of the porosity of the media in all cells and the hydraulic conductivity in cells 3 and 4 have been conducted. Full scale dye tests have been carried out on all cells to assess the flow pattern and quantify the amount of dispersion/short circuiting occurring.

In June of 1998, a subcontract was established with Dr. Harold Bounds of Northeast Louisiana University to analyze samples collected at several points in the treatment process for a variety of parameters related to the quantity, type, and activity of bacteria in the various treatment processes. In addition, during this period Dr. Bounds evaluated the effectiveness of a bacterial additive (E-bac 2000). The addition had no effect on the treatment efficiency of the filter.

Analytical results as well as data downloaded from the various pieces of equipment described above are maintained on computers in the Folk Laboratory. Data bases are maintained and additional statistical and mathematical analysis carried out using the following software applications: Axum (v5,v6), MathCad (v7, v8), Splus (4.5,2000), Visio 5 and Excel.

A traffic counter/logger is located in the exit lane of the rest area. Presumably it is downloaded on a regular basis. Since January of 2000 we have been downloading traffic data.

ANALYSIS OF DATA

A great deal of data, of numerous types, has been collected since the Grand Prairie rest area was placed in operation. In an attempt to simplify the presentation and minimize confusion, much of the data will be presented in graphics of various types rather than as tables of numbers or numerical summaries. The data summarized in this report includes daily water and wastewater flow, BOD, total suspended solids, total Kjeldahl nitrogen, ammonia, and nitrate. Because of the uncertainty regarding which water quality problems might show up at Grand Prairie and consequently affect the operation and maintenance of the system, a variety of field data was collected. Since the infrastructure already existed at the site, the cost was minimal. The parameters measured and potential concerns were:

1. pH - potential toxic effects from dumping at the RV station
2. Temperature - effect on kenitics which could require increased detention time or other operational changes during the colder months
3. Alkalinity and conductivity - used to assess the ability of the water to buffer pH changes caused by dumping
4. Heavy metal concentrations - potential toxicity, potential for removal by the rock filter or plants
5. Fecal coliforms and chlorine residual - adequate disinfection of plant effluent prior to discharge and proper operation of tablet chlorinators

Based on the data collected, we found no significant variation in pH. This was interesting because other rest areas in the state have closed their RV dump stations because of this concern. Temperature measurements coupled with BOD measurements clearly demonstrated that the system exhibits a drop off in treatment efficiency at wastewater temperatures below 20° C. This had not been documented in the literature previously and may require operational changes during cold weather. The alkalinity and conductivity of the well water at Grand Prairie was high; thus, the water has substancial buffering capacity and required no additional chemical addition to maintain an acceptable pH level. Other rest areas in the state periodically add lime to raise pH. Heavy metal concentrations are low, measure in the parts per billion range, and present no toxicity hazard. However, claims in the literature of heavy metal removal by rock-plant filters was not observed. Concurrent measurement of fecal coliforms and chlorine residual clearly demonstrated that the existence of any measurable chlorine in the effluent virtually guarantees a coliform free effluent. This is significant because rest area personnel can and do run chlorine residual tests on a daily basis. This measurement serves as a check on the proper operation of the tablet chlorinators which sometimes malfunction when the tablets hang up in the tube.

Water Usage

Each morning at 5 o'clock, rest area personnel record the cumulative water volume from a totalizing flow meter in the well discharge. By subtracting sequential values, the volume of water used in the previous 24 hour period is obtained. During holiday periods, readings are taken every hour. An accurate data base has existed since July 1, 1997. Figure 2 is simply a plot of daily water use expressed in gallons per minute (gpm) vs time. The solid curve is a local regression plot meant to show trends in the data. It shows that the daily flow has increased since 1998. Day-to-day variations in water use can be caused by a large number of factors in addition to variation in the number of cars using the facility. Some of these are: watering of greenery during the summer, water use at the picnic facilities, general cleaning and maintenance, as well as leaks in the water distribution system. Because of this "noise," little can be gained in the way of useful design information from figure 2.

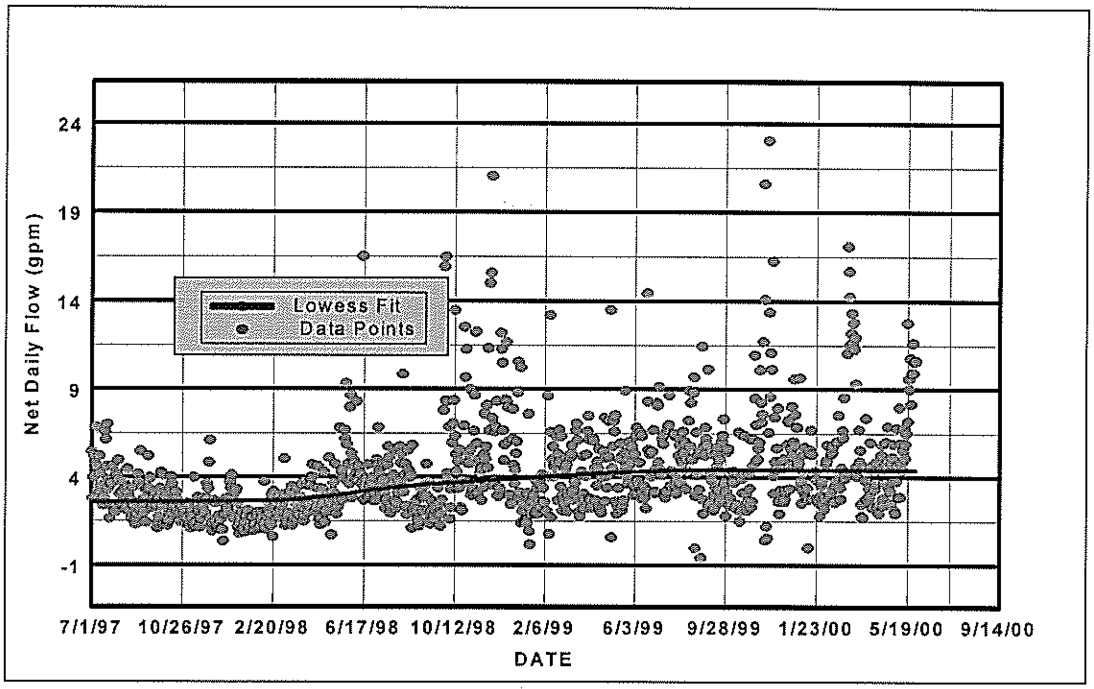

Figure 2
Daily water usage

Most recently, a water meter with the capability to transmit a signal to a data logger has been installed on the well. The average flow rate is logged every two minutes. Data is downloaded every two weeks. We have compared the daily flow volumes obtained to those from the manual procedure described previously and found the agreement to be within five percent. The equipment is easy to use, provides an abundance of data which can be downloaded to other software applications for manipulation, and does not require an individual to read the water meter. In figure 3, the same data is presented as a log-probability plot. The details of developing probability plots can be found in a variety of sources *[4], [5]*. Simply stated, figure 3 gives the probability (on the x axis) of daily water use being less than or equal to the corresponding value on the y axis. Using figure 3, we see that the water use rate is expected to be less than or equal to 3.3 gpm fifty percent of the time. Thus, the usage rate will be greater than 3.3 gpm fifty percent of the time. This is the median flow at the facility. It was somewhat surprising to find that the data plots as nearly a straight line. This indicates that it is possible to develop a statistical model of daily water use at this facility based on a log normal distribution.

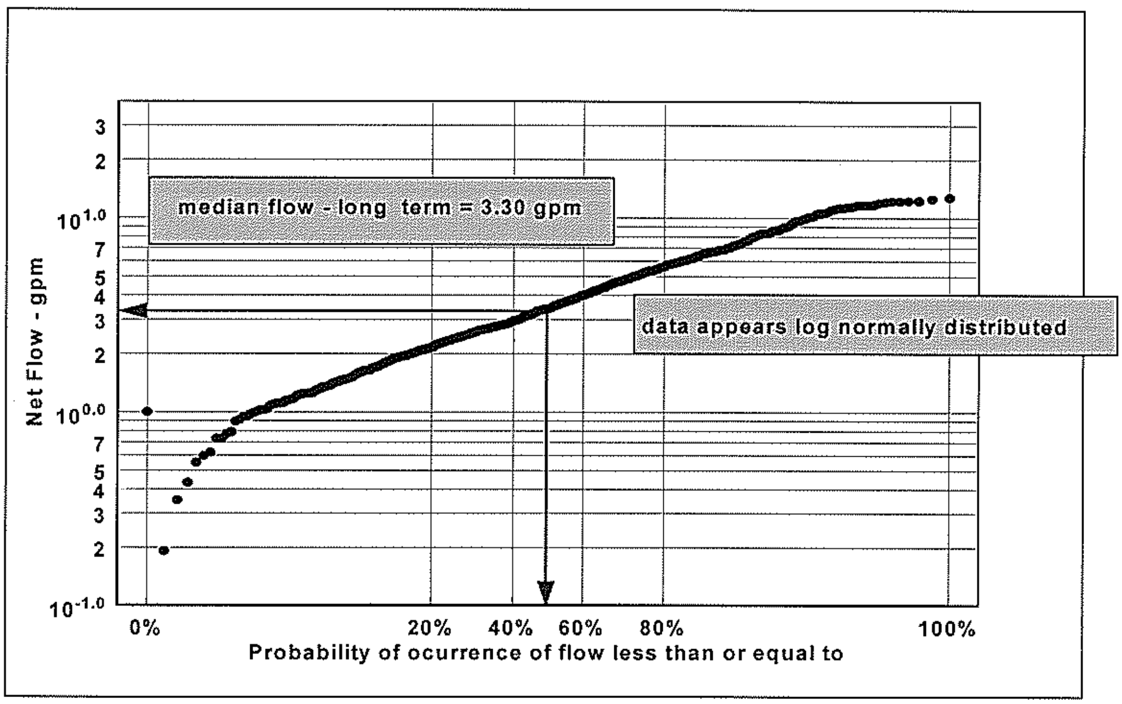

Figure 3
Probability plot daily water usage

Figure 4 is often referred to as a correlogram. To obtain it, the correlation coefficients for water use values, separated by varying times, referred to as "lags" are computed and plotted vs the lag period. The 95 percent confidence limits on these coefficients indicate that all of them are statistically significant. This correlogram exhibits a regular cyclic pattern within a seven day period. This indicates that water usage is strongly cyclic, with a seven day period. The highest water use occurs on Sunday, 5.2 gpm, while the lowest occurs on Tuesday, 3.29 gpm.

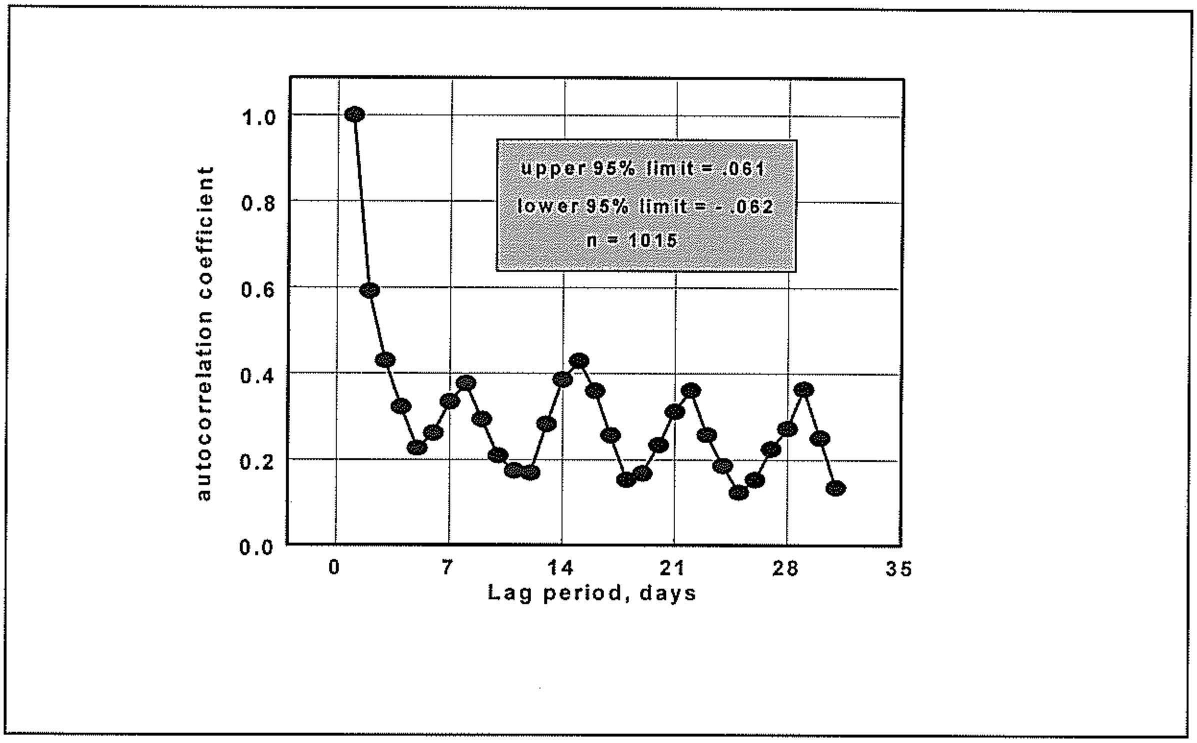

Figure 4
Autocorrelation analysis

Water Use vs Waste Generation

If we desire to predict the waste flow rate from a rest area, one very simple way would be to simply use water usage as a surrogate measurement. This however assumes that the two flows are nearly the same over time. For short time periods, this would not be the case, since a variety of factors can affect water use and/or waste flow, independently. However, it may be a reasonable assumption for longer averaging periods. Waste flow at Grand Prairie is measured independently of water use by transit time flow meters at the treatment facility. One of these flow meters is located at the "main splitter box" as shown in figure 1. All of the waste generated passes through this box. Figure 5 is a histogram of the ratio of daily water use to daily waste flow, measured at the splitter box. The histogram peaks around one and the median ratio is 1.03. Additional analysis (not shown) confirms that monthly waste flow and water use are very nearly equal, within 10 percent. This results in a hydraulic loading of 2.4 cm/day. The mean hydraulic loading for 102 aquatic treatment systems is 3.4 cm/day *[6]*.

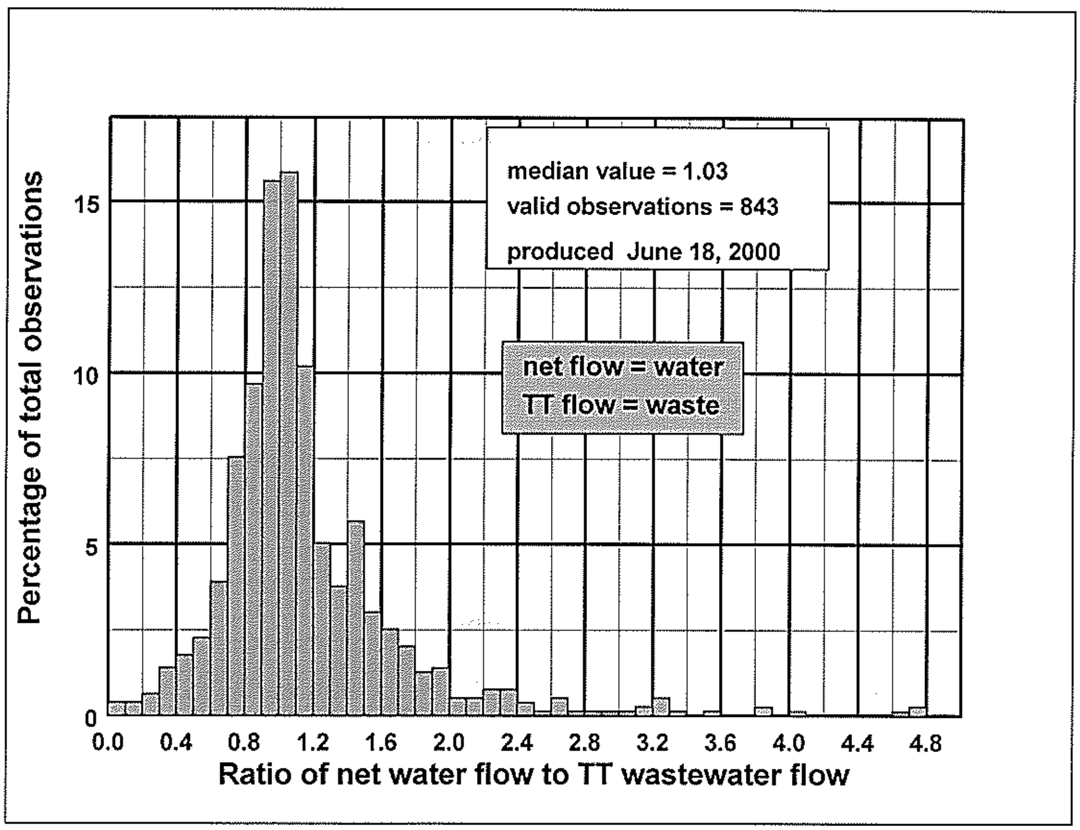

Figure 5
Comparison of water and wastewater flow rates

Influent waste strength

The wastewater generated at this facility is significantly stronger than from domestic sources or from other rest areas in Louisiana. This is primarily due to the steps taken to minimize water use at the facility such as low flow fixtures and sensor controlled water use. If the Grand Prairie facility is considered typical of new or renovated rest areas in the future, then wastewater of a similar quality should be expected. The following paragraphs describe several of the more important characteristics of the raw wastewater.

pH and Alkalinity

The total alkalinity of the well water was measured and found to be between 600 and 1000 mg/liter as $CaCO_3$. pH is routinely measured at the splitter box, the effluent from cell 1, the effluent from cell 2, and the chlorination chamber. Values obtained are consistently near 8.0. This lack of variation is directly attributable to the high alkalinity which imparts buffering capacity to the water. The measurements are collected because a common complaint at DOTD rest areas is that RVs and other vehicles routinely "dump" materials into the RV dump stations, which drastically raise or lower the pH in the treatment facility, affecting treatment. This has not yet occurred at Grand Prairie.

Heavy Metals

During the last six months of this project, heavy metals (Cu, Pb, As, Se, Cd, Cr, Zn, Mn, and Hg) were measured at the splitter box and the effluent from cell 2. All concentrations were found to be in the parts per billion range and did not change appreciably across the plant.

BOD_5

Figure 6 shows the BOD of the pump station discharge and the BOD at the main splitter box.

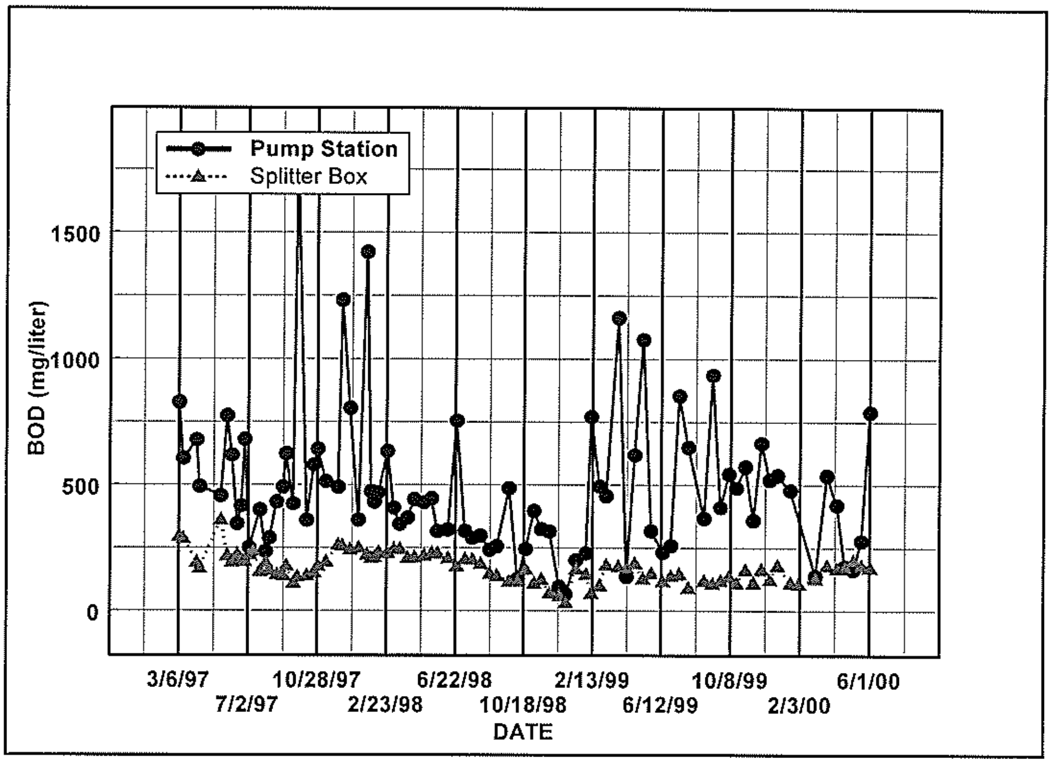

Figure 6
BOD at pump station and splitter box

Figure 7 is a log-probability plot of the BOD of the waste at the pump discharge.

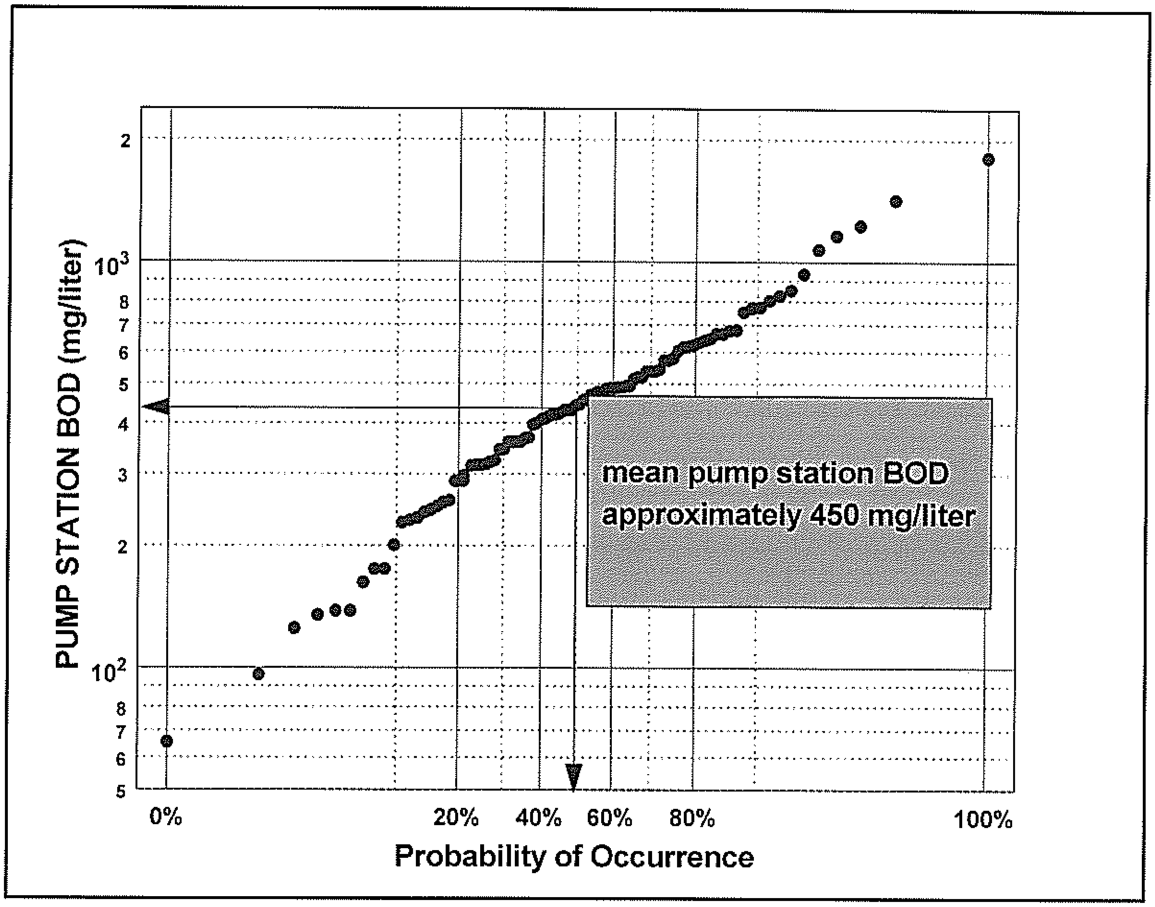

Figure 7
Probability Plot - Pump Station BOD

As shown in figure 6, the BOD of the waste is highly variable (mean = 491 mg/liter, s.d.= 309 mg/liter) and strong, on occasion exceeding 1500 mg/liter. The probability plot in figure 7 indicates a median BOD at the pump station discharge of approximately 450 mg/liter. However, the BOD at the splitter box, after the waste has passed through the septic tanks, exhibits much less variation (mean = 172 mg/liter, s.d. = 56 mg/liter) and the strength has decreased substantially.

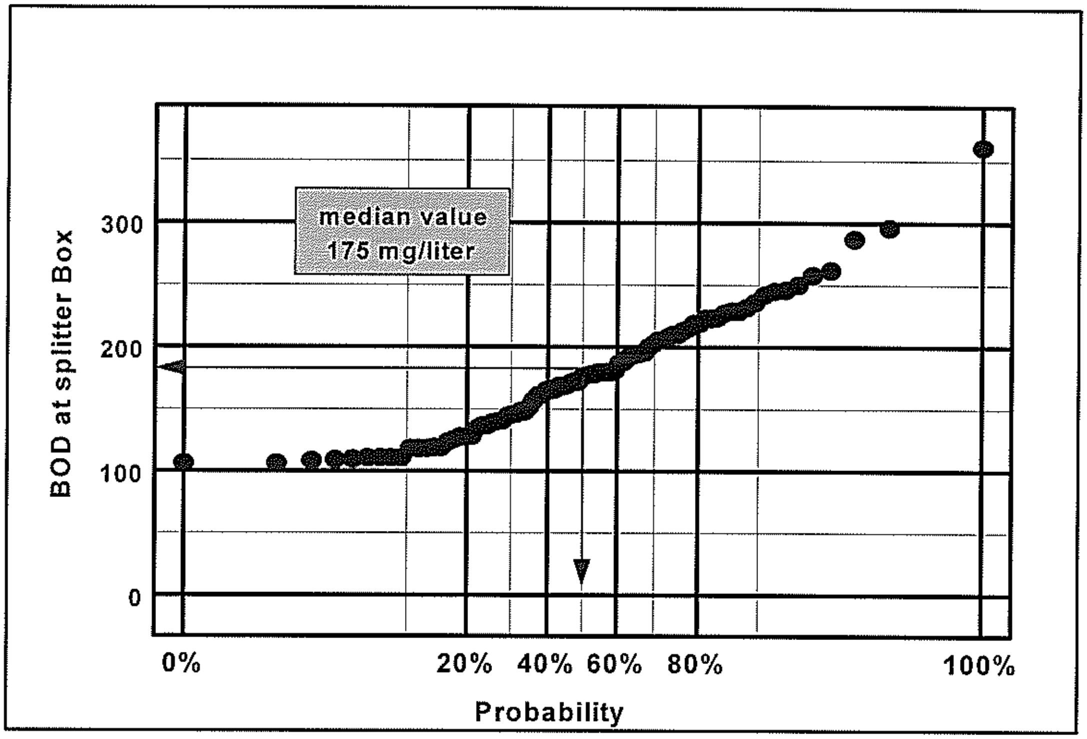

Figure 8
Probability plot - splitter box

Figure 8 shows a median BOD entering cell 1 of 175 mg/liter. The resulting median BOD loading for cell 1 is 67.2 lb/acre*day. Several points are of interest here. First, the waste generated at this facility is significantly stronger than domestic wastewater, which has a BOD of 200-300 mg/liter *[7]*. Second, the septic tanks act to dramatically reduce the BOD_5 and BOD_5 variation. Finally, the BOD_5 of the waste entering cell 1 is much higher than at any other facility in Louisiana of which we are aware. Kadlec and Knight, 1996, report an average BOD loading of 26 lb/acre*day (29.2 kg/ha*day) for North American subsurface flow treatment systems.

Total Suspended Solids

Figures 9 and 10 are analogous plots for TSS at the pump station discharge and the main splitter box.

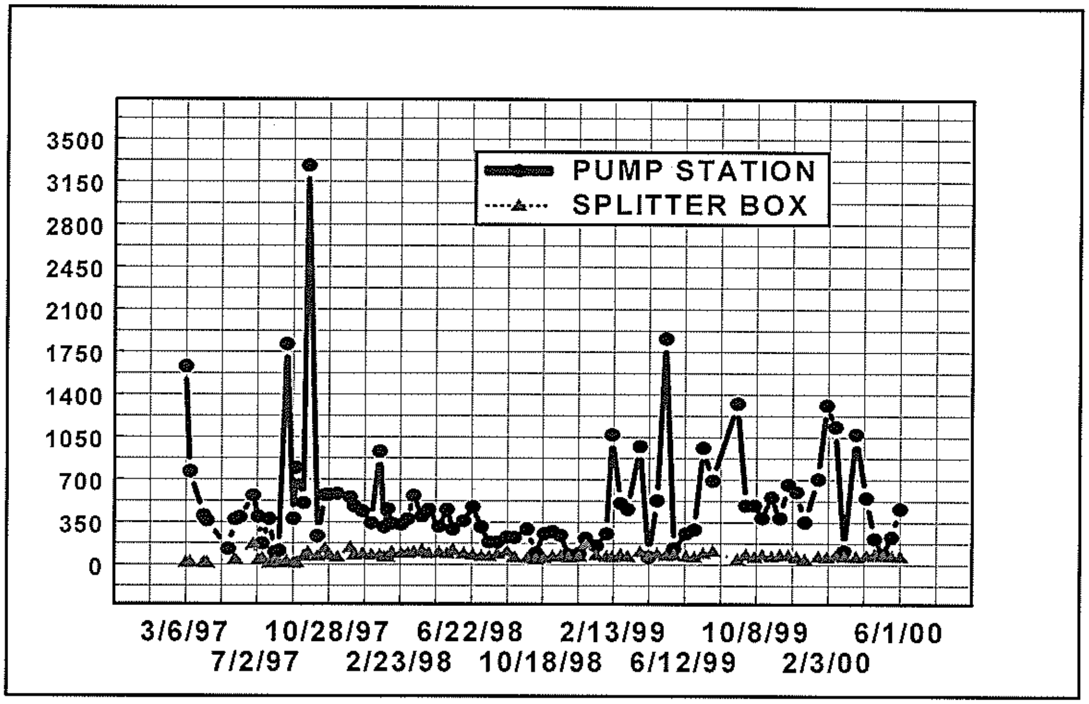

Figure 9
Suspended solids pump station and splitter box

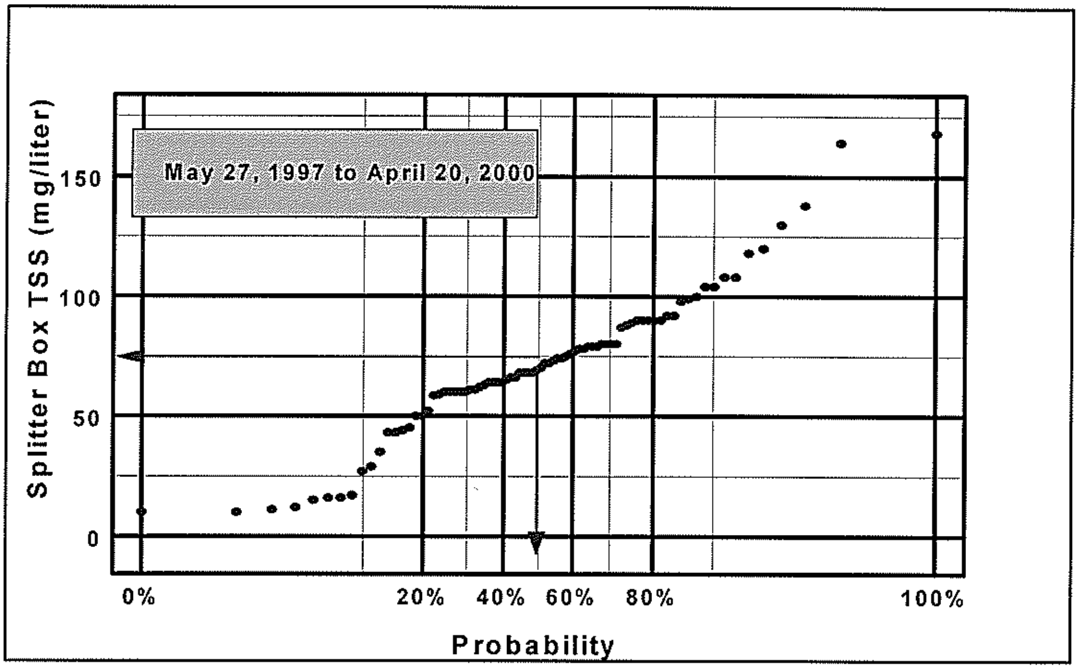

Figure 10
Probability plot TSS - splitter box

The mean and standard deviation at the pump station discharge are 523 and 480 mg/liter; corresponding values at the splitter box are 71 and 32 mg/liter. Thus, a similar phenomenon to that occurring for BOD occurs with TSS. Since much of the BOD in this wastewater is present as TSS, it appears that the primary mechanism for waste strength reduction prior to the RPF is the settling of solids in the septic tanks. The TSS concentration going to cell 1 is approximately 71 mg/liter (27.2 lb/acre*day, 30.5 kg/ha*day). RPFs treating domestic wastewater in Louisiana usually receive the effluent from a pond or lagoon. Waste strength is rarely greater than 30 mg/liter for BOD and TSS and is substantially less in many cases. The mean TSS loading for

North American treatment systems is 48.1 kg/hectare*day *[6]*. This value is higher than that at Grand Prairie, probably because of algae growth in the processes preceding other RPFs.

Ammonia

Depending on the pH of the waste, ammonia in wastewater exists in two forms, NH_3 and NH_4^+. Unionized ammonia (NH_3) is toxic to higher aquatic life forms at concentrations far below 1 mg/liter *[8]*. Figure 11 shows the ammonia concentration at various points in the Grand Prairie treatment system. In order to minimize clutter caused by the data, local regression (lowess) curves are presented for each location, rather than individual data points. The ammonia concentration in the raw waste at this facility is not excessively high. The mean is 40.1 mg/liter. Ammonia concentrations in strong, untreated domestic wastewater may reach as high as 50 mg/liter-N *[7]*. However, at this point, only about ½ of the incoming nitrogen is in the ammonia-N form. The mean TKN concentration is high, 111 mg/liter, as compared to 85 mg/liter for strong domestic waste *[7]*.

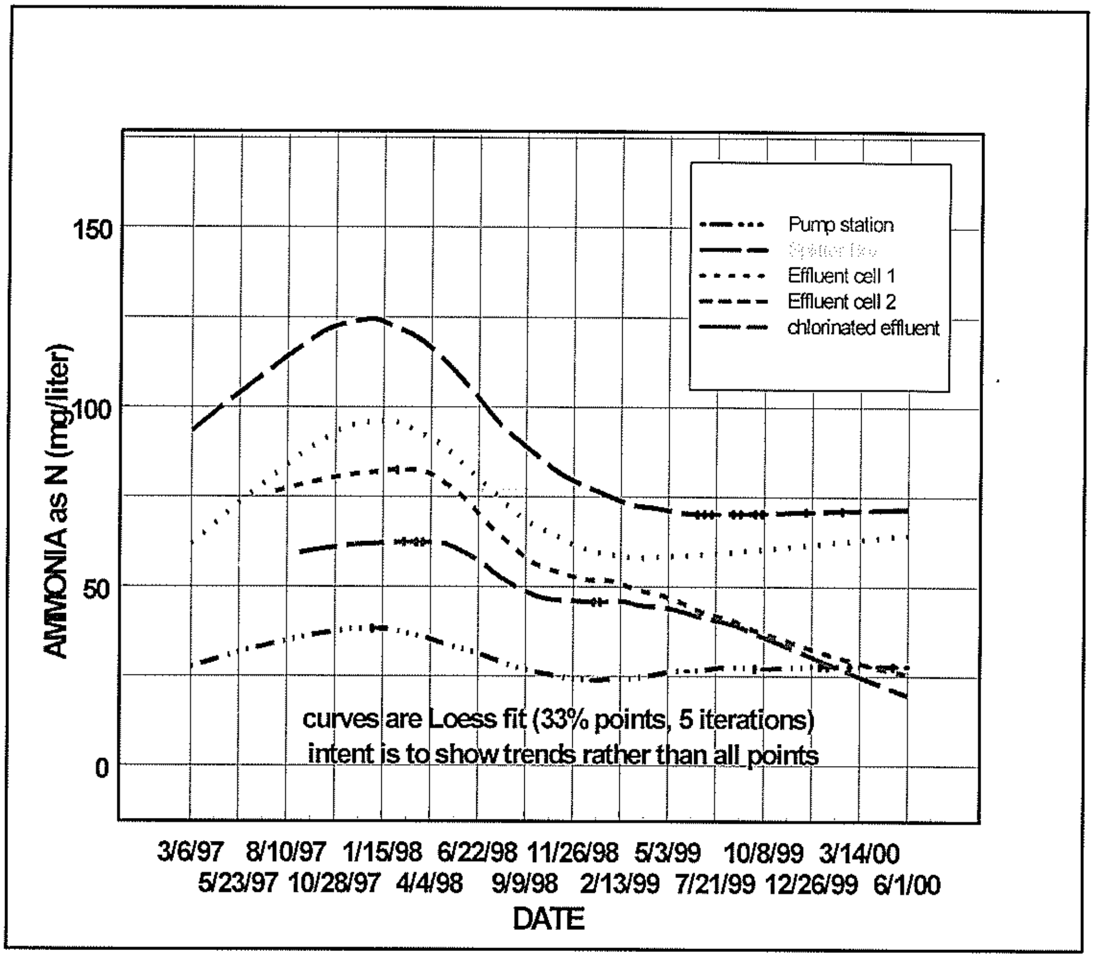

Figure 11
Ammonia-N at various locations

However, by the time the waste leaves the septic tanks, essentially all of the nitrogen has been converted to ammonia-N. The range of values in the waste at the splitter box is between 37 and 161 mg/liter. The mean and standard deviation are 94 and 30.3 mg/liter, respectively, resulting an ammonia loading to cell 1 of 40.3 kg/ha*day. This compares to an average ammonia loading for subsurface flow treatment systems of 7.02 kg/ha*day, reported by Kadlec and Knight, 1996 *[6]*. The anaerobic conditions in the septic tanks allow the organic nitrogen to decay to ammonia while preventing nitrification from occurring.

Cell Hydraulics

A primary objective of this research was to compare the flow patterns in long narrow cells to those in short wide cells. At Grand Prairie, cells 1 and 2 initially had a 1:5 L:W ratio while cells 3 and 4 had a 5:1 L:W ratio. Comparison of flow patterns using dye tests was carried out on all four cells. In a standard dye test, a slug of Rhodamine WT dye is injected at the influent end of a cell. Samples are collected, over time, at the effluent end of the cell using an automatic sampler. The dye concentration in the effluent is plotted over time. An example of the results of a dye test for a narrow cell and a wide cell are shown in figures 12 and 13.

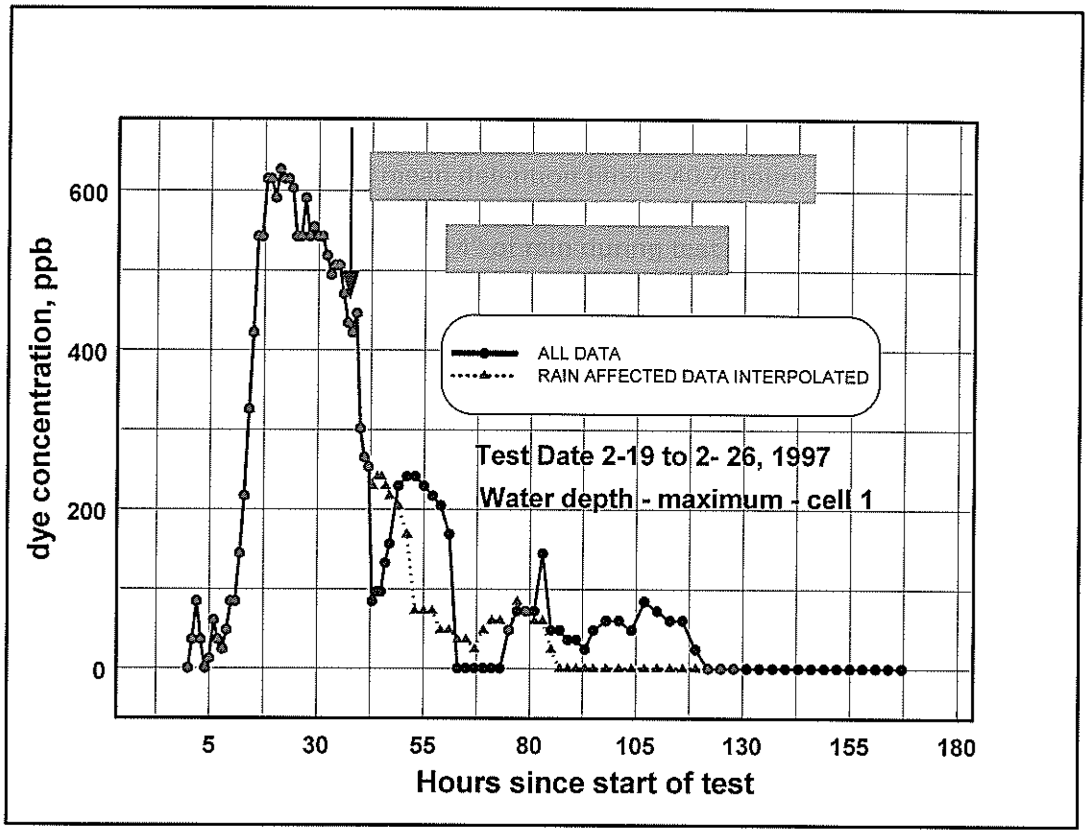

Figure 12
Tracer test - cell 3 - maximum water depth

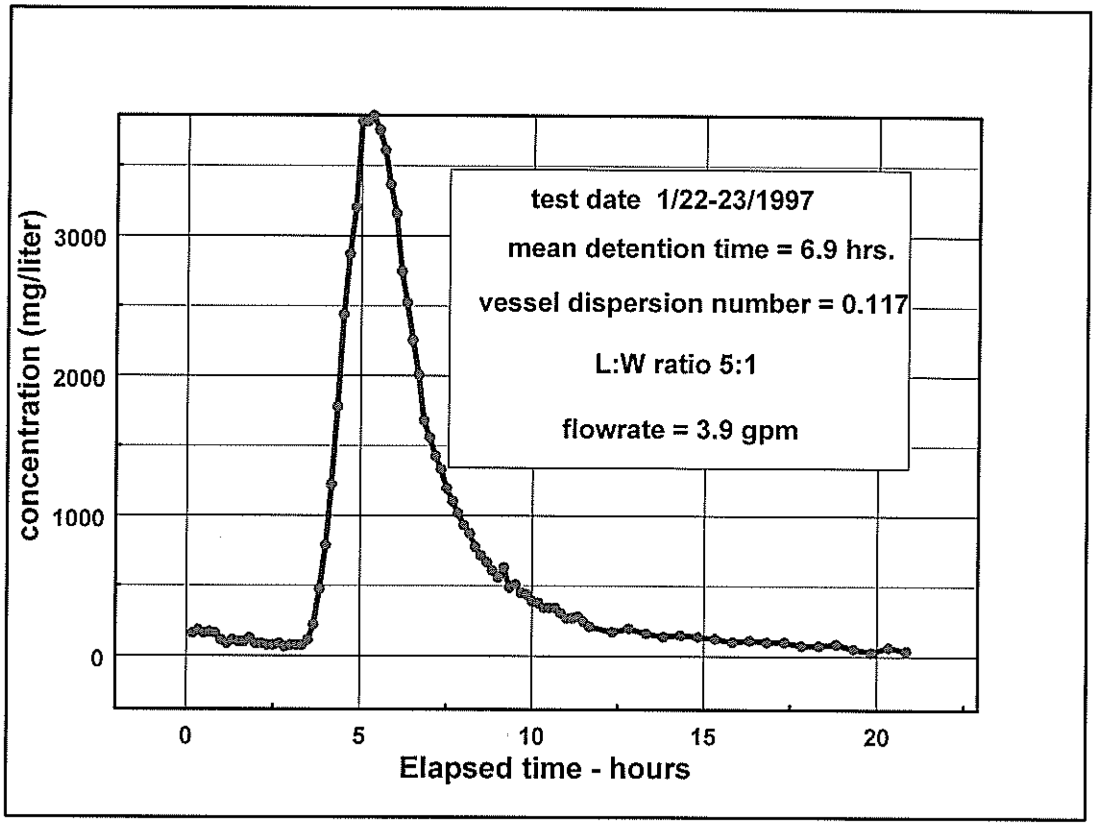

Figure 13
Tracer test - cell 3 - minimum water depth

Some of the results obtained from these tests are listed in table 1. Cells 1 and 2 have substantially longer detention times than 3 and 4. The primary reason is that the bottoms of all cells have a one percent slope. In order to keep the wastewater below the rock surface and avoid sunlight, approximately ½ the volume of cells 3 and 4 could not be used. In the wide cells, a much greater fraction of the volume could be used. However, the dispersion numbers for cells 3 and 4 are smaller than those for cells 1 and 2 indicating less dispersion and a better approximation of plug flow. Analysis of dye test data can be used to determine whether or not the flow regime in the cell can be called plug flow.

Table 1
Mean residence time

Cell number	Aspect ratio	Mean hydraulic detention time (hrs.)***	Flowrate gpm (l/sec)	Remarks
maximum depth conditions				
1	1:5	39.4* 40.7**	4.1 (0.25)	3.8"(9.7 cm) intermittent rain
1	5:1	44*	4.1 (0.25)	
2	1:5	80.5* 80.1**	4.1 (0.25)	approx. .04" (1 mm) rain
2	5:1	38.4*	4.1 (0.25)	
3	5:1	15.3	3.9 (.25)	
4	5:1	14.8	4.0 (0.25)	
minimum depth conditions				
1	1:5	12.0	4.5 (0.28)	falling leg extrapolated
3	5:1	6.9	3.9 (0.25)	

* all data included
** rain induced scatter removed
** splined curves were plotted and examined for fit prior to computing detention time

Details of these analyses and calculations are presented in a number of chemical engineering and environmental engineering texts *[9], [10]*. This determination is important because current design procedures assume plug flow *[11]*. Based on our results, we concluded that the wide cells could not reasonably be modeled hydraulically as plug flow reactors while the long, narrow cells could. In addition, dye test results from the narrow cells were more reproducible than those from the wide cells *[12]*. Because of these results, the flow direction in cells 1 and 2 was changed to give a 5:1 L:W ratio. Once the flow direction was changed, the dispersion number in cells 1 and 2 dropped considerably, as shown in table 2, below.

Table 2
Dispersion number

Cell No	Aspect Ratio	Dispersion Number	Remarks
maximum depth conditions			
1	1:5	0.250** 0.285*	3.8"(9.7 cm) rain during test period
2	1:5	0.183* 0.178**	0.4"(1 mm) rain during test period
1	5:1	.041***	
2	5:1	.044***	
3	5:1	0.063	
4	5:1	0.086	
minimum depth conditions			
1	1:5	0.313	Latter portion (>8 hr.) of data interpolated
3	5:1	0.117	

* all data included
** rain data interpolated or removed
*** flow direction changed in 1999

Effluent Waste Strength

BOD

Figure 14 shows the variation over time in BOD_5 from cells 1 and 2. In general, the system seems to perform reliably. The last violation of the facility's discharge permit occurred near the beginning of 1998.

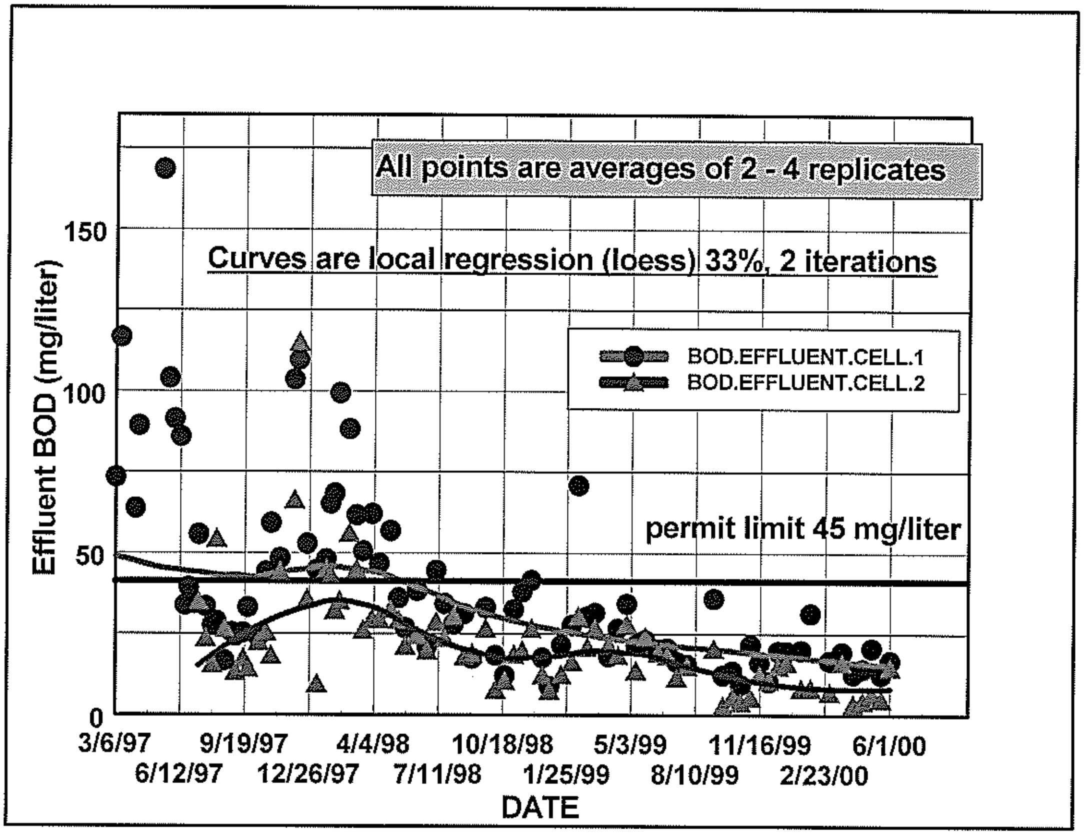

Figure 14
Effluent BOD cells 1 and 2

Figure 15 is a log probability plot of the BOD in the effluent from cell 1. As shown, the median value is 30 mg/liter.

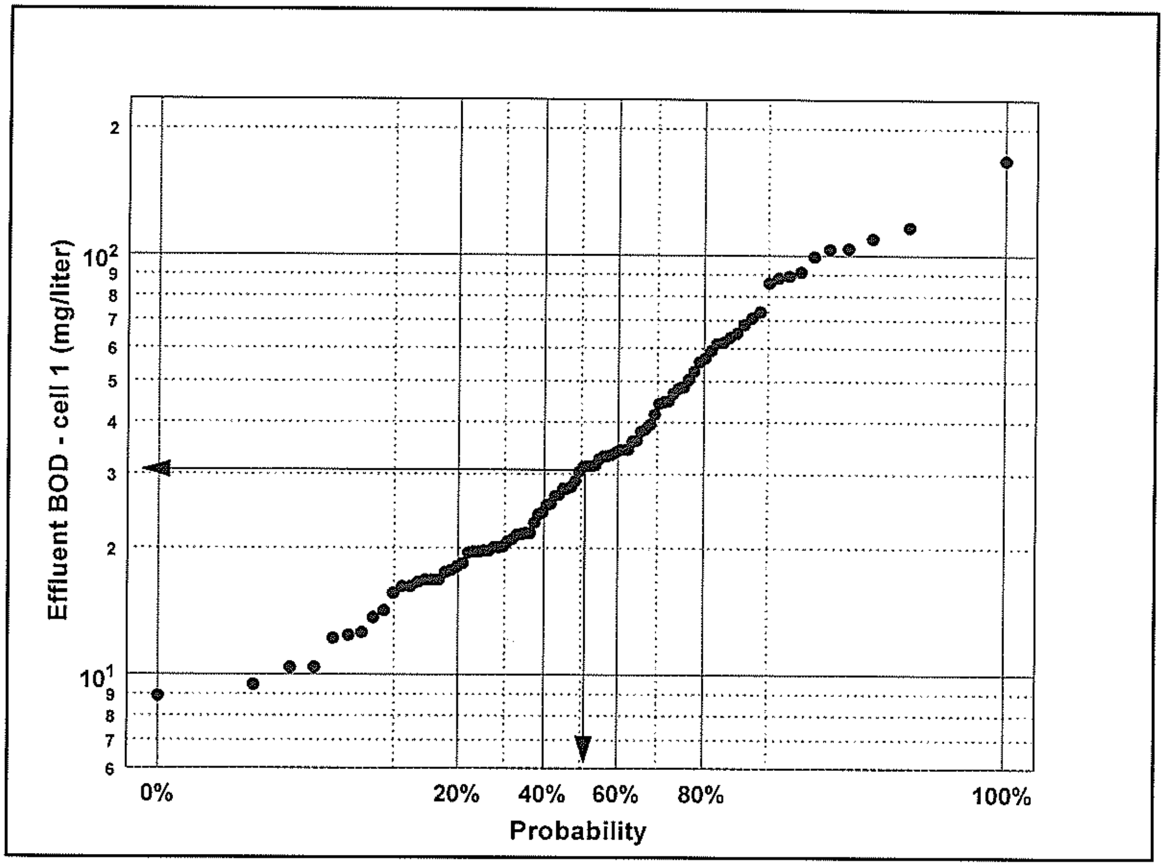

Figure 15
Probability plot - effluent BOD cell 1

The plot suggests that cell 1 alone could produce an effluent meeting the current permit requirement (45 mg/liter) nearly seventy percent of the time. Griffin, Bhattarai and Xiang have shown that the BOD from cell 1 varied with the temperature of the waste in the splitter box, this variation could be described reasonably well using an exponential model *[12]*. Figure 16 is a similar plot for cell 2, showing a median effluent BOD of 20 mg/liter. Based on this figure, the treatment facility, using cells 1 and 2 in series, has met its permit limit over ninety percent of the time. This compares to roughly seventy five percent for other RPF systems in Louisiana *[2]*.

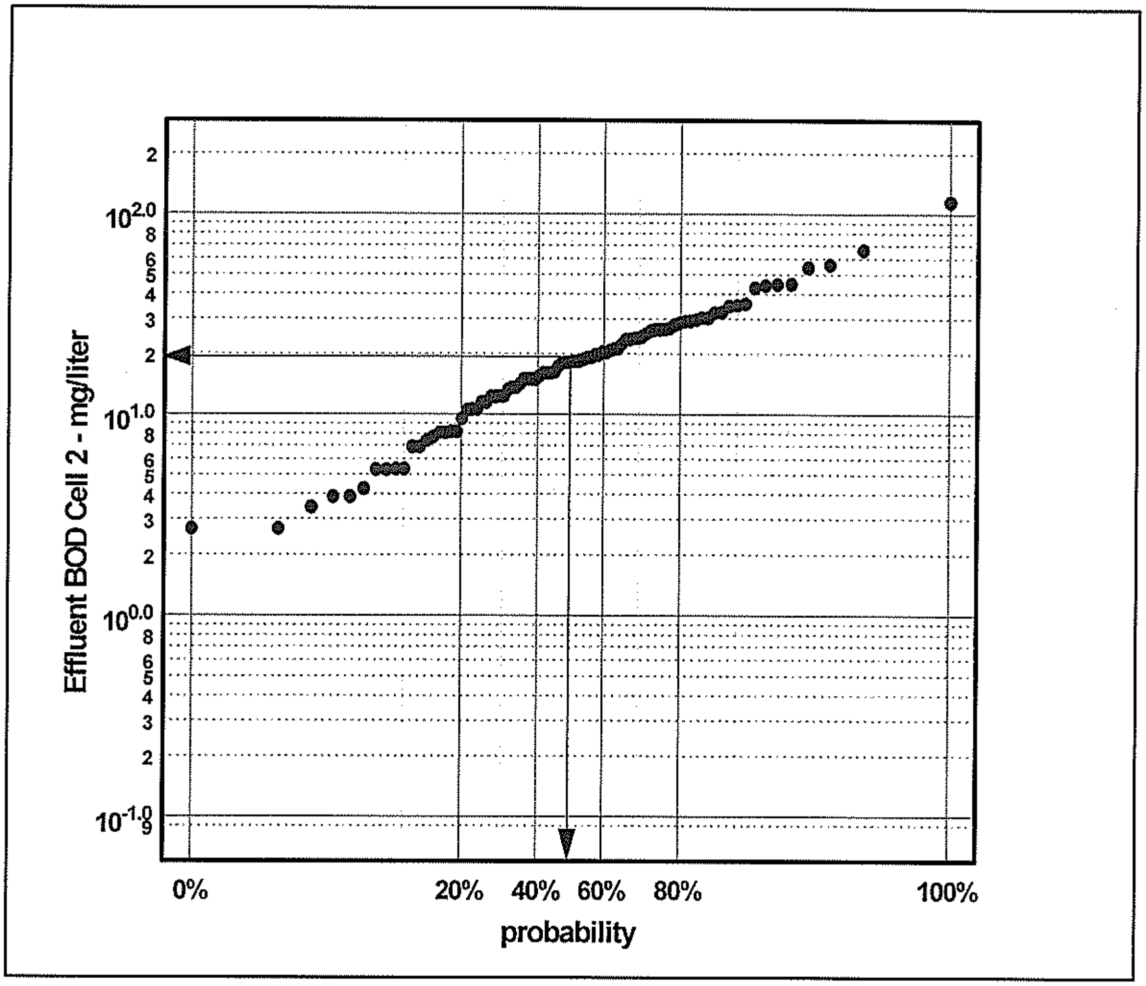

Figure 16
Probability plot - effluent BOD cell 2

During a six month period beginning in September 1998, a subcontract was issued to Dr. Harold Bounds at the University of Louisiana at Monroe to examine the bacterial growth occurring in the system. Also during this time, a solution containing bacteria (E-bact 2000) was added on a regular basis to the splitter box. Dr. Bounds found that the majority of the bacteria in the system were facultative and the additive increased the number of bacteria in the system. However, no effect on effluent BOD in either cell 1 or cell 2 could be established and the addition was discontinued.

Total Suspended Solids

Figure 17 is a plot of the effluent TSS concentrations from cells 1 and 2 with time.

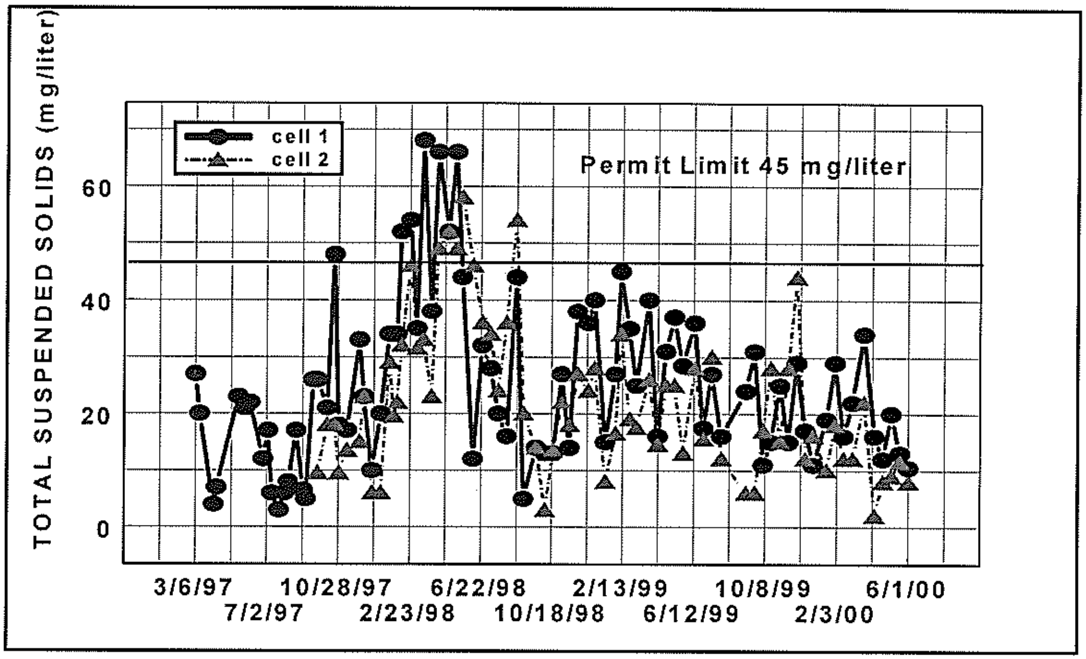

Figure 17
Effluent TSS cells 1 and 2

The system seems to be functioning reliably with respect to TSS. The last permit violation occurred in August of 1998. This probably occurred as a result of disturbance in cell 2 because plants were being placed during that time. The probability plot in figure 18 indicates a median effluent concentration of 24 mg/liter and that cell 1 produced an effluent acceptable for discharge approximately eighty five percent of the time. Figure 19 shows a median TSS concentration of 15 mg/liter.

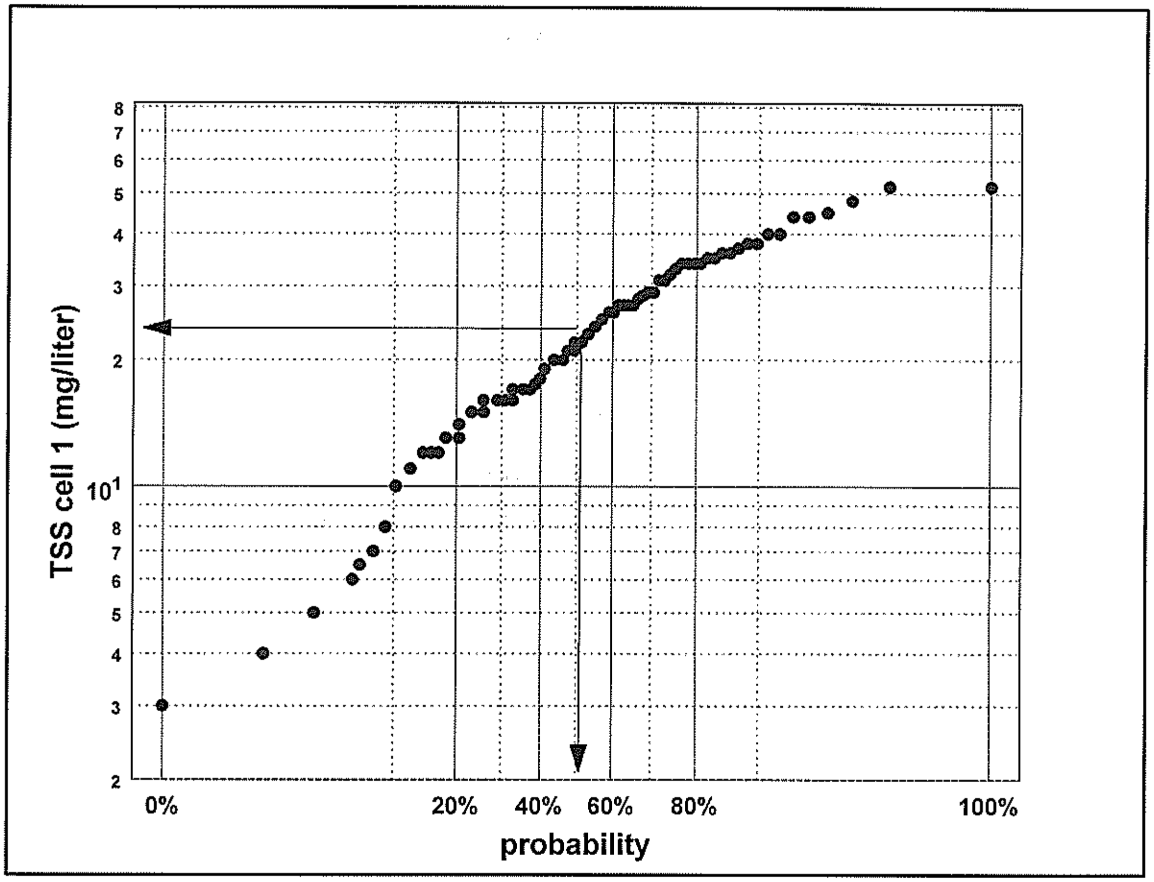

Figure 18
Probability plot TSS effluent cell 1

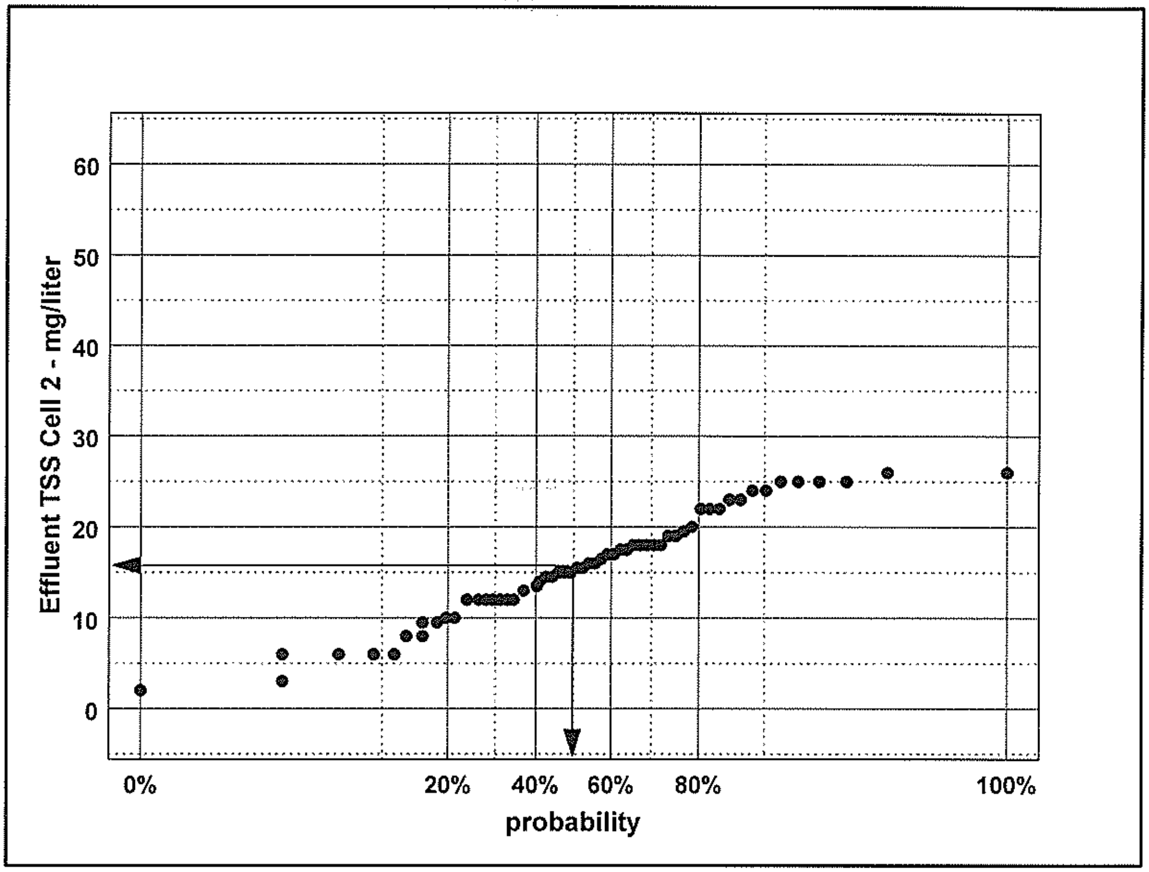

Figure 19
Probability plot TSS effluent cell 2

Ammonia

The regression curve for the chlorinated effluent (figure 10) suggests that the ammonia concentration <u>leaving</u> the treatment facility is rarely less than 50 mg/liter (mean = 56 mg/liter, s.d. = 26 mg/liter). The only biological means of getting rid of ammonia is microbial conversion to nitrate, NO_3^- or incorporation into plant biomass. Unfortunately, the first method requires dissolved oxygen levels in excess of 4 mg/liter, which were not originally present in the RPF at this facility. The mean dissolved oxygen concentration in cell 1 is 1.31 mg/liter while that in cell 2 is 1.52 mg/liter. There are plants in cell 2; however, a review of the literature suggests that plant uptake can remove only small amounts of amounts of ammonia-N, on the order of 5 mg/liter. Kadlec and Knight, 1996, report the mean percentage removal in North American subsurface flow systems to be nine percent. This suggests that plants will do little good in reducing ammonia concentrations to acceptable levels at this facility. Fortunately, the discharge permit for this facility does not currently have a limit for nitrogen. The limit in this drainage basin is 5 mg/liter of unionized ammonia (NH_3). However, we believe that such a discharge limitation will happen in time, and when it does, this facility with its present configuration, will have a difficult time meeting it. In all probability an additional process, involving some form of biological nitrification, will be needed. In an attempt to meet this challenge a simple, inexpensive aeration device was constructed approximately 20' downstream of the inlet manifold in cell 2. It consists of two standard sump pumps, purchased at a local plumbing supply house. They are located on opposite sides of the filter. The discharge from each is into a common 2" diameter PVC manifold with holes drilled into it. The holes are pointed upstream and oriented at approximately 45° above the horizontal. It is estimated that each pump, installed as described, delivers approximately 35 gpm. The liquid streams execute a parabolic path in the air. The water is in the air for approximately 1 second. Assuming an average flow of 4 gpm each parcel of liquid would be recycled nearly 20 times. The limited amount of data plotted in figures 20 and 21 suggests the following:

1. The ammonia concentration has been reduced from 50-80 mg/liter to 5-15 mg/liter.
2. The ammonia concentration increases as the liquid passes through the planted section of cell 2, presumably due to plant decay.

3. The primary ammonia removal mechanism appears to be nitrification. Some of the nitrate is then taken up by the plants.

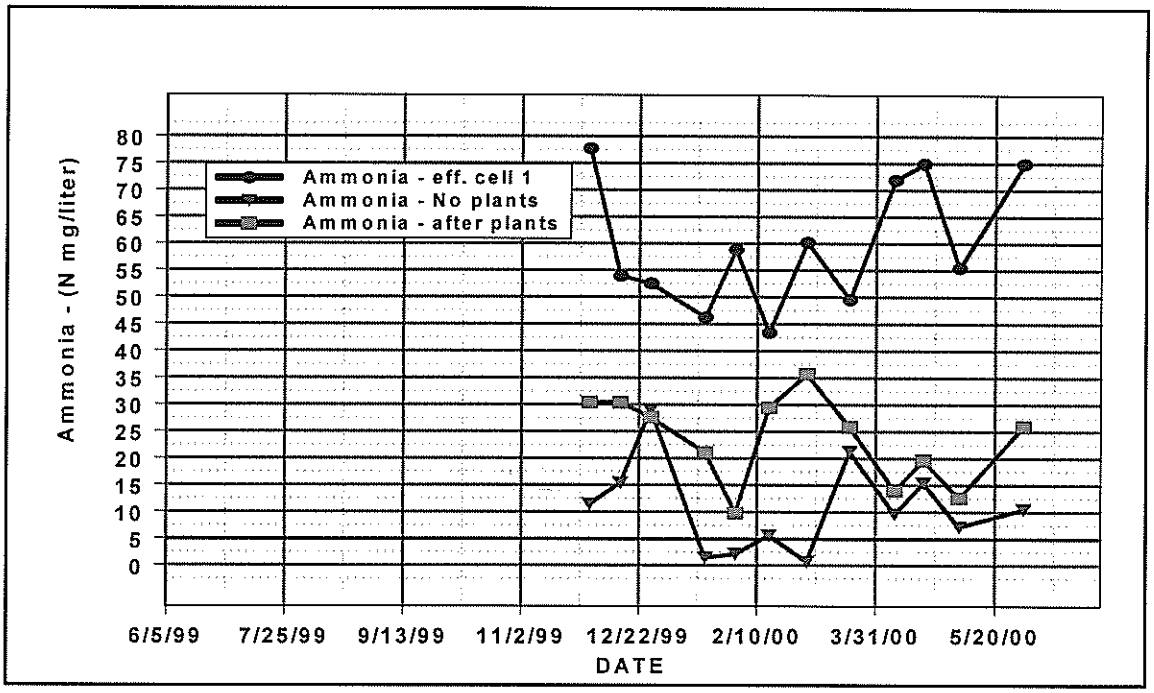

Figure 20
Ammonia variation - cell 2

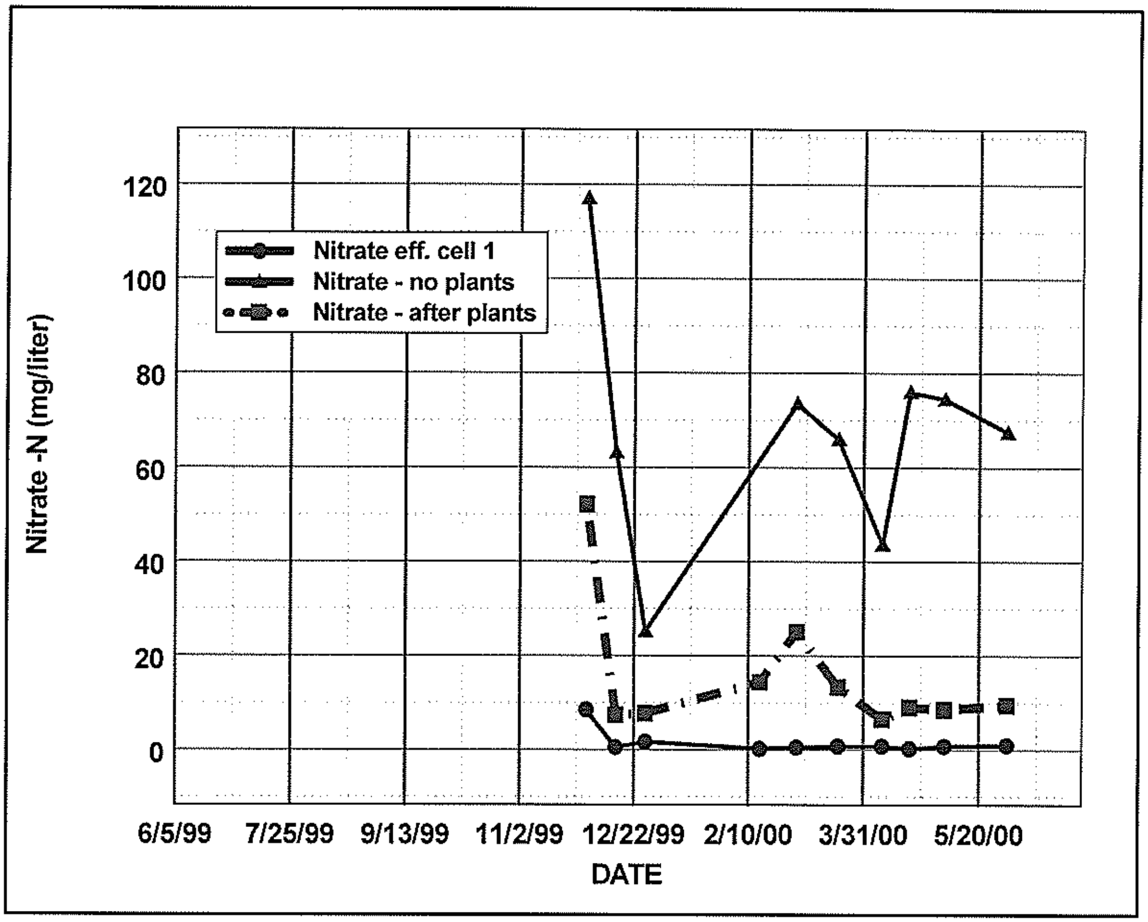

Figure 21
Nitrate variation - cell 2

Fecal Coliforms and Chlorine Residual

Fecal coliforms were measured at the splitter box, the effluent from cell 2 and the chlorination chamber. Measurements at the splitter box and effluent cell 2 were made to examine the degree of natural coliform die off, which occurs as waste passes through the plant. In most cases, the bacterial concentration declined between seventy five and ninety percent; however, the remaining coliform concentration still exceeded the permit limit (400 colonies/100 ml). Coliform measurements made at the chlorination chamber were used to verify that the plant was meeting its coliform permit limitation.

Chlorine Residual

Chlorine residual measurements were made at the chlorination chamber. These were correlated to fecal coliform concentrations there. Results showed that, as long as a measurable chlorine residual existed, the fecal coliform concentration was zero. As a result, DOTD maintenance personnel were instructed how to measure chlorine residual and did so on a daily basis. The existence of a measurable residual virtually guarantees the plant will meet its coliform permit limit. A chlorine residual test is quite easy and relatively inexpensive to run.

Table 3 summarizes the performance of the Grand Prairie Treatment system for the period of record.

Table 3 Summary

	BOD_5(mg/l)	TSS(mg/l)	TKN-N(mg/l)	NH_3-N(mg/l)	NO_3-N(mg/l)
Pump Station	495* 442** n = 83***	523 390 n = 83	111 105 n = 64	40.1 32.3 n = 80	5.3 3.7 n = 57
Splitter Box	172 171 n = 85	70.9 69.0 n = 81	105.5 105.4 n = 64	93.8 92.4 n = 87	3.28 2.71 n = 56
Effluent cell 1	40.9 32.0 n = 84	25.3 23.0 n = 83	81.4 80.6 n = 64	73.3 71.8 n = 81	2.5 2.3 n = 57
Effluent cell 2	23.1 19.8 n = 72	22.7 19.7 n = 66	66.7 63.9 n = 63	58.1 58.8 n = 72	12.5 3.52 n = 57
Chlorination chamber	N.A.	N.A.	58.3 56.7 n = 63	48.8 41.8 n = 69	11.6 3.9 n = 54

* mean
** median
*** number of valid observations

Design Procedures

Based on dye test results we believe that the flow in "long narrow" filters (high L:W ratio) can be assumed to be plug flow. The use of relatively large media results in such a high hydraulic conductivity that the design can be based on the removal kinetics in the RPF. Samples collected along the length of both cell 1 and cell 2 indicate that BOD removal can be described as a first order process. Therefore, RPFs can be designed using a design equation which assumes plug flow and a lumped first order BOD removal mechanism. BOD removal thus becomes a function of "contact time in the filter" provided the plug flow assumption is met.

$$\frac{s_o}{s_i} = \exp(-k * t)$$

where: $\mathbf{s_o}$ is the influent BOD, mg/liter

$\mathbf{s_i}$ is the effluent BOD, mg/liter

k is the reaction rate constant, 1/day

t is the hydraulic detention time, day

At this point, using average values for the influent BOD (200 mg/liter) and reaction rate constant, k (1.2 day^{-1}), determined from measurements at this facility, we believe one 175' by 75' filter with a depth of 3' and zero percent bottom slope could produce an effluent containing not more than 20 mg/l of BOD and suspended solids assuming a flow not exceeding 25,000 gpd.

Disinfection

Disinfection at this facility is accomplished using hypochlorite tablets stacked in PVC tubes with one end submerged in the treated effluent from cell 2. As the waste flows past, the tablets dissolve producing hypochlorous acid and hypochlorite ions. Two types of problems have occurred with this system. The first involves the hypochlorite tablets breaking up and/or absorbing moisture. This results in the tablets sticking to the walls of the tube, with the result that they do not "drop down" as the tablets below dissolve. As a result disinfection ceases. The problem has minimized but has not been eliminated in two ways: 1) An epoxy coated weight placed on top of the tablets in the tube, and 2) The system is checked more frequently (twice a day) and fewer tablets added each time to minimize the time during which absorption occurs. The second problem involves the level of control possible with such a system. There are four tubes which can be filled with tablets. However experience has shown that only one tube with tablets is needed to produce an acceptable chlorine residual under normal conditions. However, if abnormal conditions occur, such as a rainfall event, which increase the flow quickly and dramatically, the tablets dissolve faster than expected and disinfection ceases until the next time the tube is filled. If more than one tube is used for redundancy an excessive chlorine residual results. The only solution proposed for this problem at this time is more frequent checking of the system.

Maintenance Requirements

Since the facility opened, Griffin and Bhattarai have been primarily responsible for its operation and maintenance. We are on site one to two days every two weeks. Until recently there were only a few plants in cell 2. The only mechanical components in the facility are the pump stations at the influent and effluent ends of the plant. Malfunctions of these are handled by DOTD plumbing and electrical personnel, and in a few instances, by private contractors. As far as plant operation itself is concerned, in the absence of plant care, there is essentially none required. There are several maintenance chores, the most significant one being cleaning the influent manifolds in each cell to insure good influent distribution across the cells. This takes about one hour for the wide cells and about 15 minutes for the narrow cells. It is done every two weeks in the summer and every month in the winter. The procedure consists of removing each 90° elbow on the manifold clearing the nipple with a hose or straight object and replacing the elbow. Each elbow and nipple are marked. These marks are lined up so that flow from the manifold is balanced with equal flow from each elbow. Rest area personnel check the tablet chlorinators on a daily basis and add tablets as needed. This takes about 15 minutes per day. The remainder of our time on site is spent carrying research activities. However, if plants become a permanent part of the system, they may require substantial maintenance because dead and decaying plant material should be removed from the cell. If not, nutrients taken up from the waste by the plants will be re-released into the system resulting in net nutrient removals of zero.

Clogging

In Louisiana as well as other locations in the country, clogging of RPF media appears to be a common problem. Once the media is clogged, the waste flow is forced to the surface. At this point, little treatment occurs and both algal and bacteriological growth is stimulated. Observations at Grand Prairie and other locations in Louisiana suggest that during periods of intense sunlight, algae growth in standing or slow moving waste can effectively blind the surface of an RPF within 12- 24 hours. Several causes of this clogging phenomenon have been proposed. Some suggest it is not really clogging at all that initiates surface flow but incorrectly placed influent and effluent structures which force the waste level to rise above the surface of the media for flow to occur. Others point to clogging as a result of biological growth in the media. Thus far at Grand Prairie there is no evidence whatsoever of clogging even though the filters are receiving comparatively high strength wastewater. Initial measurements (1996) of media porosity in all four cells resulted in values of around thirty nine percent. More recent measurements (1998) in cell 1 indicate that this value has changed very little. Perhaps the best evidence we have to present describing a mechanism for media clogging comes from a RPF serving the town of Delcambre, Louisiana. Delacmbre has two RPF cells each 400 feet by 150 feet . They are preceded by a lagoon and surface flow wetland. As a result, the BOD_5 at the inlet is low. These cells had been in operation approximately seven years when one of them began to exhibit surface flow near the inlet manifold. In order to investigate this problem, we placed observation ports at 20 foot intervals down the center of the cell. We then determined and plotted the water surface elevation along the centerline of the cell. In addition, we dug a number of holes in the rock. We visually observed that the media did appear clogged in the vicinity of the inlet manifold as well as along the sides of the filter. There was little or no apparent clogging elsewhere. Samples of the interstitial material were collected and analyzed. It was found that the material was largely inorganic in nature (VSS <20%); it was basically sediment. A plot of the filter bottom and water surface elevation in the filter are shown in figure 22.

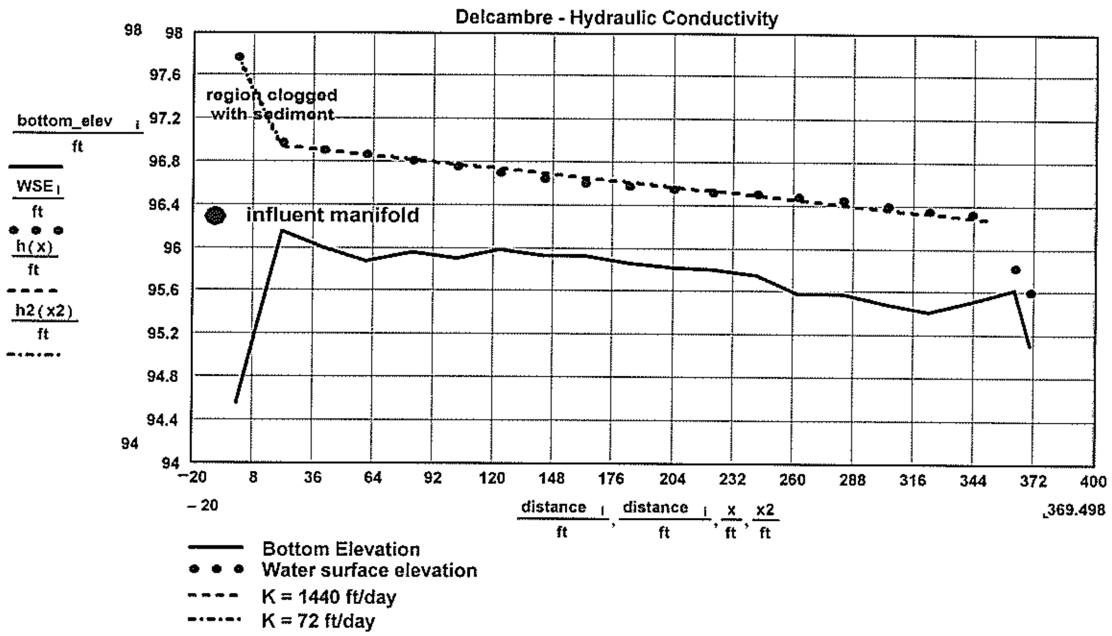

Figure 22
Water surface profile - Delcambre, LA

Based on the water surface elevations collected, we were able to determine the hydraulic conductivity from Darcy's Law. In the region of the cell near the manifold, it was found to be 72 ft/day, a very low value. However, as flow moved downstream from the inlet manifold (away from the clogged section), the hydraulic conductivity achieved a constant value of 1440 ft/day, a textbook value for "gravel" or "well sorted gravel" such as that used in RPFs. A second test conducted in January of 2000 resulted in a value of 576 ft/day, suggesting that the clogging is becoming worse. Based on these results we postulated the failure mechanism as being due to the migration, over time, of sediment from the berms around the sides of the cell into the cell, producing progressive clogging. The influent manifold in this system lies parallel to one of the berms, about 10 feet away. This 10 foot strip was not a part of the operating filter, per se, and clogging which occurred behind the influent manifold had no effect on the operation of the system. However, once the clogging reached the influent manifold, the flow was forced to the surface. At this point, substantial algal growth began to occur on the surface of the media, exacerbating the problem. We concluded that the problem was caused by the lack of a lining along the sides of the cell to prevent soil migration into the filter. We also concluded that while the growth of biological slime on the media was the first visible sign of clogged media, it was NOT the cause of the problem.

CONCLUSIONS

1. Cells 1 and 2 appear to reliably reduce influent BOD_5 and TSS to acceptable levels. As of this writing, the system has not had a violation in nearly two years. Based on previous data, the system can be expected to meet its permit limit for these constituents over ninety percent of the time. However, because of the low dissolved oxygen levels in both cells, little nitrification can be expected without additional aeration in some form.

2. In terms of BOD_5 and TSS, the wastewater from this facility is somewhat stronger than that from domestic sources or from other rest areas in Louisiana. This is due to the nature of the facility as well as the restricted water use. Assuming Grand Prairie is a prototype, waste of similar quality should be expected from new or newly renovated rest areas in the future.

3. While the ammonia concentration at the pump station is not great when compared to domestic wastewater, the total Kjeldahl nitrogen concentration is excessive.

4. The septic tanks preceding the RPF at Grand Prairie function to reduce the BOD_5 and TSS of the waste as well as the degree of variation of these constituents over time. They also avoid exposing the waste to sunlight as would occur in a pond with subsequent algal growth.

5. Unfortunately, the anaerobic conditions in the septic tanks enhance the conversion of organic-N to ammonia while preventing nitrification. We believe the installation of a simple aeration device in cell 2 will largely eliminate this problem.

6. Water usage at this facility appears to be an acceptable substitute for measuring waste flow directly. This is significant because the technology required for measuring waste flow at this facility is substantially more complex and costly than that for measuring/logging water use.

7. At present, various bacterial additives are being marketed claiming to improve the performance of treatment systems in various ways. One of these, E-bac 2000, was added to the waste stream for six months in 1998 -1999. We could detect no beneficial effects associated with it and addition was discontinued.

8. After four years of service, cells 1 and 2 exhibit no signs of clogging. Our research at an RPF in Delcambre, Louisiana has demonstrated that one clogging mechanism involves migration of soil from berms into the filter to the point where the influent manifold is blocked, forcing flow to the surface. This can be prevented by lining RPF cells to prevent soil from being washed into the filter.

9. The tablet type disinfection system, while simple to operate, is difficult to control and requires frequent inspections for reliable operation.

10. A number changes in design and construction of rock plant filters are recommended as a result of this project.

a. The large, uniformly sized granite media has functioned well. No evidence of clogging exists. Field measurements of the hydraulic conductivity indicate no change since the plant was placed into operation. In future projects care should be taken to minimize the amount of fines and broken pieces for rock in the placed media. Small concentrations of these can dramatically reduce hydraulic conductivity.

b. When using large media, a bottom slope on the filter is not needed and may well limit the usable filter volume.

c. Work at Grand Prairie and other towns in Louisiana strongly suggests that clogging of filters, resulting in surface flows, occurs as a result of drainage and erosion of soil into the cells, rather than bacteriological growth. Future systems must take great care to see that the site is graded to eliminate drainage into the filter cell.

RECOMMENDATIONS

The only significant problem remaining is that substantial ammonia remains in the effluent. It is therefore recommended that methods of removing ammonia either by plant uptake or nitrification be investigated.

1. Consider the use of rock-filters at other rest areas, particularly those with the topography to allow gravity flow throughout. A 150 foot by 30 foot cell with a 24 hour settling tank and three feet of effective rock filter can treat the wastewater at any rest area in the state to current permit limits.

2. Install 24 hour capacity holding tanks (septic tanks) upstream of treatment plants, rock-filter and mechanical aeration, to improve overall efficiency and to buffer potential toxic dumps.

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