FIELD DEMONSTRATION OF BIOLOGICAL DENITRIFICATION OF POLLUTED GROUNDWATER
and
PILOT SCALE FIELD TESTING OF BIOLOGICAL DENITRIFICATION WITH WIDELY VARIED HYDRAULIC LOADING RATES

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ABSTRACT

Field Demonstration of Biological Denitrification of Polluted Groundwater

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Pilot Scale Field Testing of Biological Denitrification with Widely Varied Hydraulic Loading Rates

From 1989 through 1991 a pilot-scale field study of biological denitrification of shallow groundwater was conducted by faculty and students at the University of Colorado at a site in Brighton, Colorado. One purpose of this study was to demonstate the reliability of the biological process under field conditions, including natural variations in water quality, especially nitrate concentration. A second more specific goal was to study the effect of flow variations on the denitrification process. Especially of interest were large changes in flow resulting from seasonal fluctuations in water use. We have found that the fixed biomass denitrification reactor used in the Brighton study readily adapted to fourfold variation of hydraulic loading rate (1.8 to 7.2 $m^3/m^2/day$). At all loading rates, nitrate was reduced from over 13 mg/l to less than 5 mg/l, as N. These findings were significant for two reasons: 1) they have demonstrated that it is not necessary to maintain unused biological capacity to insure consistent denitrification performance during seasonal water demand changes, and 2) the adaptation was not predicted by previous investigators, whose observations were based on short duration experiments.

Final polishing of biologically denitrified water is necessary to remove biomass-generated by-products. We have studied the removal of these contaminants in two downstream processes, a biofilm prefilter and a slow sand filter. The prefilter/slow sand filter combination consistently produced water with turbidity of 0.5 NTU. After maturation, total coliform bacteria were reduced by over 95%; no *E. coli* were ever found. Furthermore, the effluent water had the same organic "fingerprint" as the untreated Brighton water supply, indicating that a polishing process had been developed to reliably produce denitrified water to meet potable standards.

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1. MOTIVATION FOR RESEARCH

Nitrate contamination of much of the world's groundwater resources has reached serious levels. As researchers document the acute and chronic health effects of nitrate contamination of drinking water, the need for effective treatment strategies becomes increasingly important. Although current technologies such as reverse osmosis and ion exchange can remove nitrates from drinking water, these processes are not feasible for many communities which are most affected by nitrate pollution, namely small towns in rural and semi-rural areas. For this reason, new nitrate removal methods must be developed which can be successfully implemented by these small communities.

The major acute health concern posed by nitrate contaminated water is the blood disorder methemoglobinemia which is potentially fatal to infants. In infant methemoglobinemia or "blue baby" disease, nitrate is converted to nitrite, which is taken up by hemoglobin to form methemoglobin. Methemoglobin renders red blood cells unable to transport oxygen from the lungs. In infants, nitrate is converted especially efficiently to nitrite and methemoglobin is not destroyed fast enough, leading to oxygen deficiency and possible suffocation (Shuval et al., 1980).

Other chronic health effects of nitrate have been noted, but not as clearly. In adults, nitrates can be converted to N-nitroso compounds by bacteria in the saliva, stomach, or infected urinary bladder. These compounds have been associated with cancers of the stomach, esophagus, nasopharynx, and urinary bladder (Forman, 1989). It has also been suggested that leukemia and non-Hodgkin's lymphoma may be induced by N-nitroso compounds (Weisenburger, 1990).

Because nitrate is very soluble and therefore highly mobile in water, it has become a common groundwater contaminant. The major sources of nitrates in groundwater are agricultural fertilizers (both chemical and natural), certain industrial wastewaters, and municipal waste waters. Nitrate contamination of groundwater is especially a problem for small rural communities where over-fertilization and minimal wastewater treatment have pushed nitrate concentrations to critical levels in many regions.

In Europe, a survey of the nitrate contamination situation in the twelve European Economic Community Member States was presented at a Seminar of the European Institute for Water in 1985. The data indicated a trend of increasing contamination of European waters. An alarming percentage of these waters exceeded the EEC standard of 11.3 mg/l NO_3-N. The following is a summary of some of the findings (Fried, 1990). (Concentrations of nitrate have most likely increased in the six years since this survey was conducted.)

> Belgium - Groundwater concentrations of nitrate ranged from 2.3 mg/l NO_3-N in the Ardennes to 22.6 mg/l NO_3-N in the Bruxellian sand, with an average concentration of 4.5 to 11.3 mg/l NO_3-N in the agricultural zones south of Brussels.

Denmark - Waterworks supplying more than 99% of the drinking water to the population, have shown nitrate concentrations greater than 11.3 mg/l NO_3-N in 8% of the plants.

Germany - Surveys show that 5% of the waterworks supply drinking water with nitrate concentrations in excess of 11.3 mg/l NO_3-N.

France - In 1981, the Ministry of Health found that more than one million people were supplied with water containing nitrate concentrations greater than 11.3 mg/l NO_3-N (approximately 2% of the population). If the present trend were allowed to continue, it was estimated that 20% of the population would be supplied with nitrate contaminated water exceeding 11.3 mg/l NO_3-N by 1995.

In the United States, surveys have found that approximately 5 percent of the private wells sampled had nitrate levels above the U.S. Primary Drinking Water Standard of 10 mg/l NO_3-N. In areas of the country that are heavily farmed, up to 20 percent of the wells exceed the standard (Amsden, U.S. E.P.A.). In Colorado, a recent report of the Colorado Water/Sewer Needs Committee (Colorado Division of Local Government, 1989) categorized the drinking water supplies of Baxter, Brighton, Chambers Subdivision, Fort Lupton, Gilcrest, Hudson, Kim, LaSalle, Milner, Peyton, Platteville, and Southgate as demonstrated health hazards or producing immediate health effects due to high nitrate concentrations.

By law water sources exceeding the nitrate standard of 10 mg/l NO_3-N must be treated. Most nitrate polluted groundwaters which are potential sources of potable water contain between 10 and 40 mg/l NO_3-N. Within this range biological denitrification, using a fixed biofilm reactor with acetic acid as the supplemental carbon source, is economically competitive with ion exchange and reverse osmosis. Furthermore, it avoids the problems of by-product (brine) disposal associated with ion exchange and reverse osmosis, and does not require specially trained operators. The simplicity of operation of fixed biofilm processes makes biological denitrification a suitable choice for water treatment in small communities which do not have the resources to run an ion exchange or reverse osmosis plant.

There are some aspects of potable water denitrification that are significantly different from wastewater processes. One is the requirement for reliability. Another is the concern about bacteria contacting potable supplies (even nonpathogenic bacteria). For these reasons, the following study was undertaken.

2. BIOLOGICAL DENITRIFICATION

Nitrate is removed from water readily by denitrification, a bacterial respiration process which has end products of dinitrogen gas and nitrous oxide (Payne, 1973). Denitrification is carried out by numerous bacterial species found in soil and aquatic environments. These bacteria are primarily facultative heterotrophs, which can respire

using either oxidized nitrogen or oxygen as a terminal electron acceptor. Denitrification is inhibited by the presence of oxygen which is a more energetically favorable electron acceptor for the bacterial cell (Painter, 1970).

Heterotrophs require an organic carbon energy source (electron donor) for respiration. Since most drinking water supplies contain very little organic carbon, supplemental organic carbon must be added to the reactor influent for significant denitrification to occur. The carbon source chosen for the research described here was acetic acid, because of its availability, non-toxicity to humans, solubility in water and the ease with which it is degraded by a broad range of bacterial species. Furthermore, it can be stored without some of the special precautions required for the more flammable organic compounds, methanol and ethanol.

3. PREVIOUS RESEARCH AT C.U., BOULDER

Stoichiometry

Research at the University of Colorado was begun by Nevis Cook, J. Silverstein and Victor Ketellapper (1988) who created a rational theoretical framework for determining the quantity of acetic acid required for complete reaction of all influent nitrate and dissolved oxygen. The stoichiometric equations given below were developed by assuming that 65 percent of the carbon source (acetate) is used for cell production when molecular oxygen is the terminal electron acceptor and that 35 percent of the acetate is converted to cell mass when nitrate is the terminal electron acceptor:

$$1.00\ O_2 + 1.43\ CH_3COO^- + 0.263\ NO_3^- + 0.263\ H^+ \rightarrow$$
$$0.265\ C_5H_7NO_2 + 0.0523\ CO_2 + 1.43\ HCO_3^- + 0.63\ H_2O \quad \text{Equation (1)}$$

$$1.00\ NO_3^- + 0.877\ CH_3COO^- + H^+ \rightarrow$$
$$0.0877\ C_5H_7NO_2 + 0.456\ N_2 + 0.422\ CO_2 + 0.877\ HCO_3^- + 1.07\ H_2O \quad \text{Equation (2)}$$

These equations were then verified experimentally using a bench scale packed tower denitrification reactor (Ketellapper, 1988).

Kinetics

A half-order model has been used to model fixed film denitrification kinetics (Harremoes, 1976). The half-order rate expression is:

$$r = -kC^{1/2} \quad \text{Equation (3)}$$

where: C = Substrate concentration (mg/l).
k = Half-order reaction coefficient $((mg/l)^{1/2}/min)$.
r = Rate of substrate removal (mg/l-min).

From Equation (3), assuming an ideal plug-flow reactor:

$$C^{1/2} = C_0^{1/2} - K\theta_H \quad \text{Equation (4)}$$

where: C = Nitrate nitrogen concentration after detention time θ in the reactor.
C_0 = Initial (reactor influent) nitrate nitrogen concentration.
K = Combined half-order reaction coefficient (includes integration constant).
θ_H = Hydraulic detention time (empty bed).

Reactor

From November 1988 through October 1989, pilot scale tests of a fixed bed denitrification process were conducted by Bill Hogrewe and faculty investigators (1990) in the University of Colorado Environmental Engineering Laboratories. Table 1 shows the standard operating conditions for these tests. Hogrewe et al.. developed a procedure for removing excess biomass which allowed continuous reactor operation as well as completing an extensive series of tests to determine the steady state performance of the reactor system with respect to nitrate concentration reduction. Hogrewe verified the half-order kinetic model, with the fitted constant K ranging from 0.025 to 0.049 $(mg/l)^{1/2}/min$ (Hogrewe, 1990).

Table 1: Standard Reactor Operating Conditions for Laboratory Study at C.U.

Influent Nitrate:	19.5 mg/l NO_3-N
Influent Dissolved Oxygen:	2.8 mg/l
Influent Acetate:	90% of stoichiometric
Influent Alkalinity:	200 mg/l as $CaCO_3$ added
Temperature:	21 C
pH:	6.5
Flow:	1.1 l/min.
Hydraulic Loading:	3.63 $m^3/hr./m^2$

Filtration

To demonstrate a filtration technology appropriate for small communities capable of producing water meeting the turbidity and bacteriological standards for potable water, the denitrification reactor effluent was applied to two slow sand filters operated in parallel for a period of three months. During the first month of operation, two different filter configurations were investigated, during which odor and head loss problems arose. For the last two months, the operating conditions summarized in Table 2 were used for both filters (Bram, 1990). The denitrification effluent was also aerated within both of the slow sand filters by using an air stone suspended in the filter influent basin above the filter sand surface.

Table 2: Standard Filter Operating Conditions for Tests Conducted at C.U.

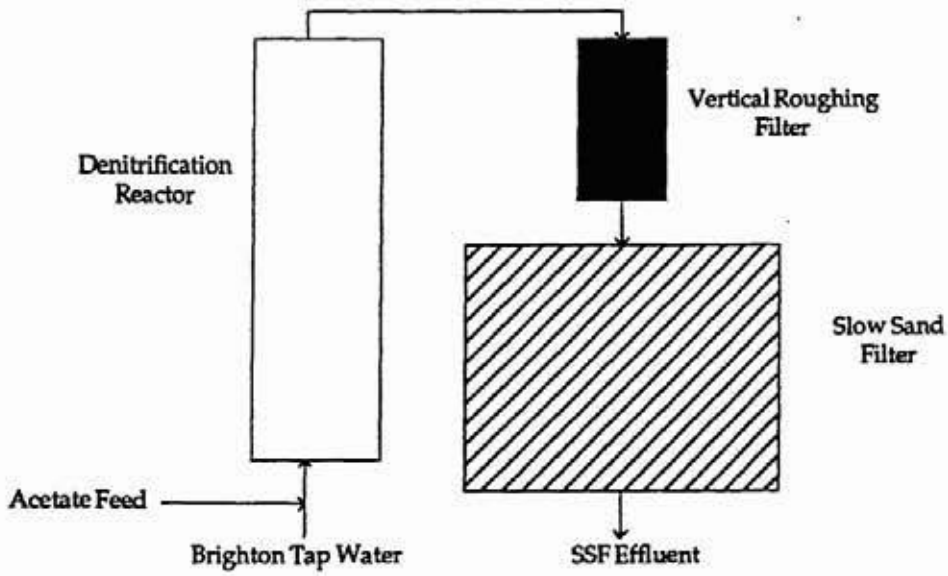
Sand Depth:	1.0 meter (3.25 ft.)
Sand Size (D_{10}):	0.92 mm (0.036 in.)
Uniformity Coefficient (D_{60}/D_{10}):	2.3
Hydraulic Loading:	0.08 gpm/sq. ft.
Terminal Head:	1.0 meter (3.25 ft.)

Both filters achieved effluent turbidity levels consistently below 1.0 NTU after the fifth day of operation. The filters operated for 51 and 59 days respectively before one meter of headloss developed, and the run was stopped (Bram, 1990). This was considered an acceptable run length, demonstrating that slow sand filtration was a viable filtration choice following denitrification and preceding disinfection.

4. DESCRIPTION OF CURRENT PROJECT

Based on the previous research, a field demonstration of the biological denitrification process followed by slow sand filtration was initiated. A simplified schematic of this facility is shown in Figure 1. It was the objective of the project to demonstrate the operation of the plant under conditions which are typical of a small community water supply operation, i.e., significant periods of unattended operation, natural variations of water temperatures and influent nitrate concentrations, and seasonal variations in water demand producing variations in hydraulic loading to the denitrification tower. After slow sand filtration the performance of the plant was expected to produce a high-quality water meeting the Safe Drinking Water Act standards for nitrate, turbidity, and coliform bacterial concentrations.

Figure 1: Schematic of Pilot Plant



Out of several Weld County communities, Brighton was selected as the final location for the demonstration facility. Brighton's water supply is drawn exclusively from a shallow aquifer with a nitrate-contaminated groundwater which contains between 13 and 17 mg/l nitrate nitrogen (1.3 to 1.7 times the standard). Using Brighton water as an influent to the denitrification reactor provided an excellent opportunity to test the reactor's performance on a naturally occurring contaminated groundwater. A summary of Brighton water characteristics is shown in Table 3.

Table 3: Summary of Brighton Water Characteristics

Nitrate-Nitrogen:	13 - 17 mg/l NO_3-N
Dissolved Oxygen:	5 - 6 mg/l
Temperature:	15 - 16 C
pH:	7.8
Alkalinity:	320 mg/l
Calcium:	440 mg/l
Sulfate:	330 mg/l

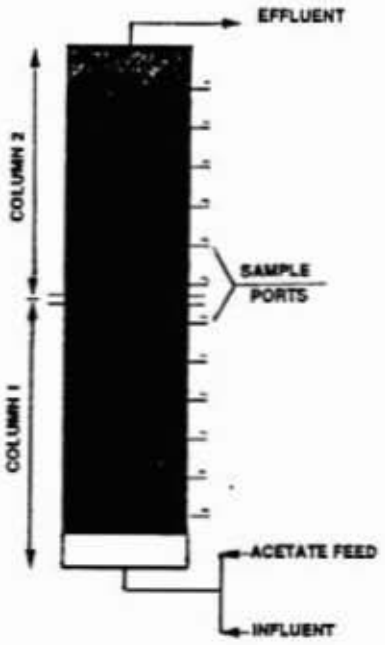
Denitrification Reactor (DNR)

The effects of varying the hydraulic loading on the DNR were investigated by Nevis Cook, J. Silverstein and Marcia Mendonca (1991). The long-term (21 day average) performance of the reactor and the short-term (48 hour) response to regular air scour were of primary interest. An investigation into the effects of varying the hydraulic loading rates on a biological denitrification reactor has been previously done by Harremoes (1976). He reported no significant changes in the half-order rate coefficient, K, when hydraulic loading rates ranging from 11.4 to 27.3 $m^3/hr/m^2$ were applied to a down-flow denitrification packed tower. Based on these results, an increase of the hydraulic loading should result in a decrease of the reactor's performance, i.e., an increase in nitrate concentration in the reactor effluent.

However, an increase in hydraulic loading rate at a constant concentration of nitrate and acetic acid results in an increase in mass loading of both electron donor and electron acceptor to the denitrification reactor. From previous studies (Cook et al., 1990), it was suspected that the reactor could accommodate biofilm growth resulting from long-term changes in hydraulic loading. If this were the case, K would increase and long-term denitrification performance of the reactor would not decrease. Therefore, the primary purpose of these experiments was to test the constancy of K at various hydraulic loadings, as well as to investigate the recovery of the reactor after air scouring procedures.

Operation of the reactor began in January 1990 continuing through May 1991, for a total of 17 months of continuous operation. Figure 2 shows the denitrification reactor configuration. The reactor consisted of a single upflow packed column 15.2 cm in diameter and 5.2 m high. Twelve sample ports were evenly distributed along the length of the reactor (41cm apart), with an additional sample port on the influent line as well. 4.7 meters of the reactor height was packed with a high porosity plastic media (Jaegar Tripack #2). The remaining volume was reserved for bed expansion during the air scour procedure which was periodically carried out to remove excess biomass.

Figure 2: Denitrification Reactor Configuration



During the period of pilot plant operation, two separate hydraulic loading sequences were performed. In the first sequence, there were three loading periods: from January 22 to May 4, 1990 the hydraulic loading was 3.6 $m^3/hr/m^2$, from May 5 to July 7, 1990 the loading was increased to 7.2 $m^3/hr/m^2$, after July 7, 1990 the loading was reduced back to 3.6 $m^3/hr/m^2$. In the second sequence, performed by graduate students at the University of Colorado (Dunning et al., 1991), there were two periods of constant loading: from January 24 to February 26, 1991 the hydraulic loading was 1.8 $m^3/hr/m^2$, after February 26, 1991 the loading was increased to 11.6 $m^3/hr/m^2$.

Because of the possibility of producing anaerobic conditions in the reactor effluent which could lead to odor problems, and the fact that the nitrate concentration need only be reduced to less than 10 mg/l NO_3-N, it was decided to limit the acetic acid feed to 75 percent of the stoichiometric requirement for complete removal of influent dissolved oxygen and nitrate. In addition to avoiding anaerobic conditions in the reactor effluent, this "standard" mode of operation has the advantage of minimizing acetic acid requirements.

For continuous operation of the reactor, it is critical to remove excess biomass which accumulates in the reactor due to bacterial growth. Once every three weeks, a five minute air scour of 0.5 m^3/min per m^2 of reactor cross-section was applied. The reactor was then drained, removing any biomass that was loosened during the air scour. The period of time between air scours constituted a cycle, making one cycle equivalent to 21 days of reactor operation.

Two or three times per week, samples of the DNR influent and effluent were analyzed for nitrate using the uV spectrophotometric method (Standard Methods, 1990), as well as for turbidity (Hach 2100A). Once per cycle, a reactor profile was also

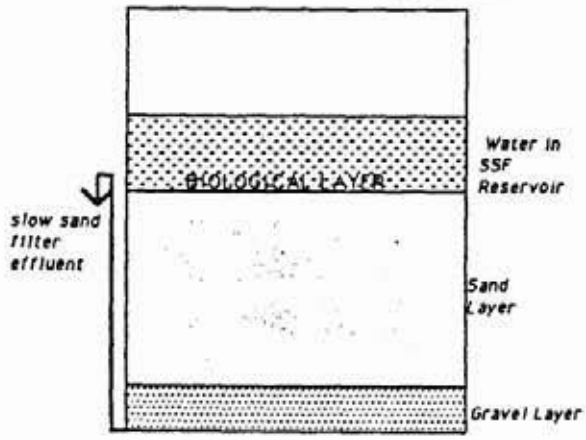
analyzed for nitrate. Pressure in the DNR was measured with a pressure gauge connected to sample port #1 at the bottom of the reactor. An ISCO (model 1390) automatic sampler was used to take samples from the DNR effluent every 2 hours for 48 hours after a backwash to form a reactor recovery profile. These results are presented and discussed in Section 5.

Slow Sand Filter (SSF)

The ability of a slow sand filter and a roughing filter + slow sand filter combination to act as a polishing process for DNR effluent was investigated by Nevis Cook, J. Silverstein and Roger Sydney (1991). In particular, the fate of particulates, pathogenic and non-pathogenic bacteria, dissolved and colloidal organic compounds, and chlorine demand in the downstream processes were examined. Run time was also investigated as another important parameter of filter system performance.

Operation of the slow sand filter ran concurrently with the denitrification reactor from January 1990 to May 1991. Figure 3 shows a schematic of the slow sand filter. The slow sand filter stood 2.4 m high, with a cross sectional area of .348 m^2 (.6 m by .58 m). The filter medium consisted of 90 cm of sand (d = .85mm, uniformity coef. = 1.53), placed over 20 cm of gravel which acted as an underdrain. The effluent outlet was placed 1.4 m from the bottom of the filter providing a minimum water level of 40 cm above the surface of the sand.

Figure 3: Filter Configuration



The hydraulic loading on the SSF was maintained at 3.16 l/min.-m^2 (approximately .08 gal/min-ft^2), corresponding to a flow rate of 1.1 l/min. When a headloss of 90 cm above the effluent outlet was achieved, the filter run was terminated and the top layer of sand was removed.

The filter runs can be divided into three categories, runs 1-6 without a roughing filter (RF), runs 7 and 8 with a roughing filter preceding the SSF and run 9 with a roughing filter and with the DNR covered to control algae growth. The roughing filter consisted of a 2.6 m high acrylic cylinder, 15.2 cm in diameter, packed with high rate plastic roughing filter media (Koch Flexirings). The cylinder was covered to prevent

the growth of algae.

Two or three times per week dissolved oxygen, flow rate, nitrate concentration, pH, total organic carbon, total suspended solids, turbidity, temperature, and headloss were measured. The presence or absence of coliform bacteria and E. coli were tested for runs 7, 8, and 9. In addition, chlorine demand and a heterotrophic plate count were conducted for run 8, and an apparent molecular weight distribution of nonpurgable organic carbon was determined for run 9.

5. DENITRIFICATION REACTOR RESULTS

The following is a summary of Mendonca's work on biological denitrification of a nitrate contaminated groundwater in Brighton, Colorado (Mendonca, 1991).

Long Term Results

Figure 4 shows the long term influent and effluent nitrate data. It should be noted that sporadic failures of the feed system occurred during the first 11 days of operation at 7.2 $m^3/hr/m^2$. Table 4 shows the average long term nitrate removal and standard deviations for flow rates of 1.8, 3.6, 7.2, and 11.6 $m^3/hr/m^2$.

Figure 4: Long Term Influent and Effluent Nitrate-Nitrogen Data

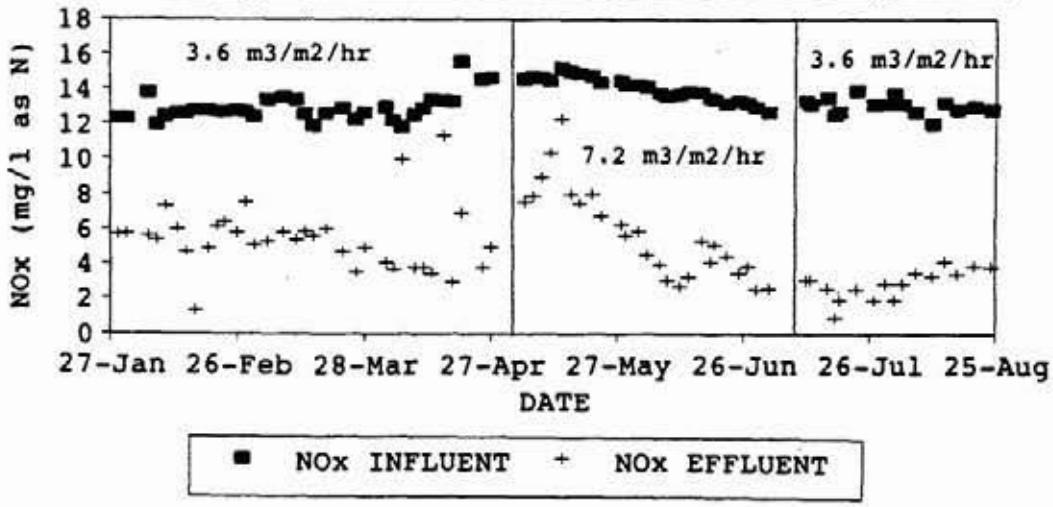


Table 4: Average Long Term Nitrate-Nitrogen Removals

Hydraulic Loading	1.8 $m^3/hr/m^2$		3.6 $m^3/hr/m^2$		7.2 $m^3/hr/m^2$		11.6 $m^3/hr/m^2$	
Statistic	Mean	Std. Dev.	Mean	Std. Dev.	Mean	Std. Dev.	Mean	Std. Dev.
Influent NO_3 (mg/l - N)	12.5	0.3	13.0	0.7	14.0	0.7	16.8	0.4
Effluent NO_3 (mg/l - N)	4.4	1.0	4.6	2.0	5.7	2.5	12.0	1.2
Removal NO_3 (mg/l - N)	8.1	1.3	8.4	2.1	8.3	2.0	4.8	1.6

The system was obviously overloaded at a hydraulic loading of 11.6 $m^3/hr/m^2$, and no other useful results were collected. A t-test was applied to the nitrate removal data (excluding data at 11.6 $m^3/hr/m^2$) and it was concluded with 99% probability that the nitrate removals were the same. These results indicate that the DNR was able to accommodate long term changes in hydraulic loading by increasing the rate of nitrate removal (K) at higher flow rates, thus maintaining nitrate removal performance. It is interesting to note that although the DNR was able to adjust to the higher flow rate, it did take some time for it to reach the same denitrification level achieved at 3.6 $m^3/hr/m^2$. This can be seen in Figure 4 by the steady improvement of the DNR's nitrate removal performance during the 7.2 $m^3/hr/m^2$ period. This seems to indicate that an adjustment period is necessary for the biomass to assimilate to the new loading conditions.

Kinetics

Figure 5 shows typical nitrate profiles for hydraulic loadings of 1.8, 3.6, and 7.2 $m^3/hr/m^2$. Average values of K were calculated for each of the flow rates and are shown in Table 5. The average rate coefficients K_1, K_2, and K_3 were compared using a t-test. It was found that K_2 was significantly greater than K_1, and K_3 was significantly greater than K_2 ($K_1 < K_2 < K_3$), i.e., the half-order reaction rate coefficient appears to be a function of hydraulic loading.

Figure 5: Typical DNR Nitrate-Nitrogen Profiles

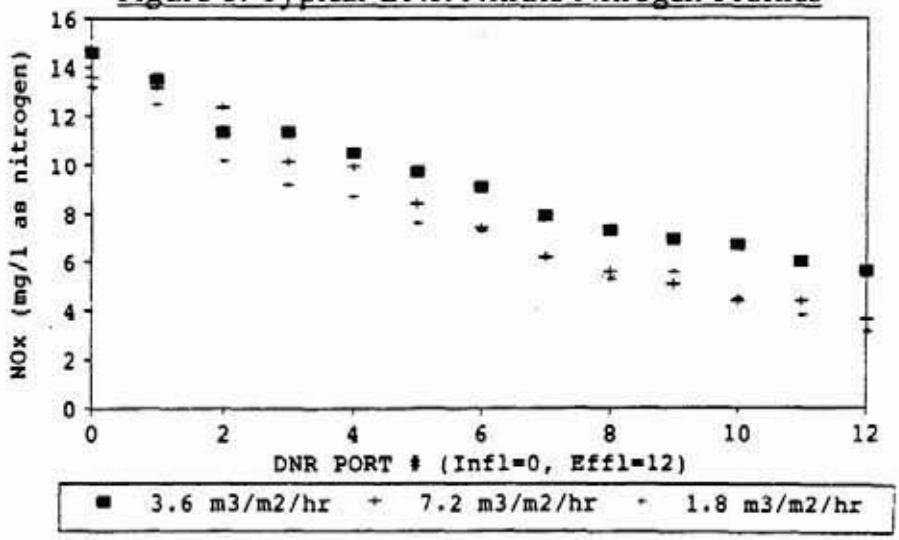


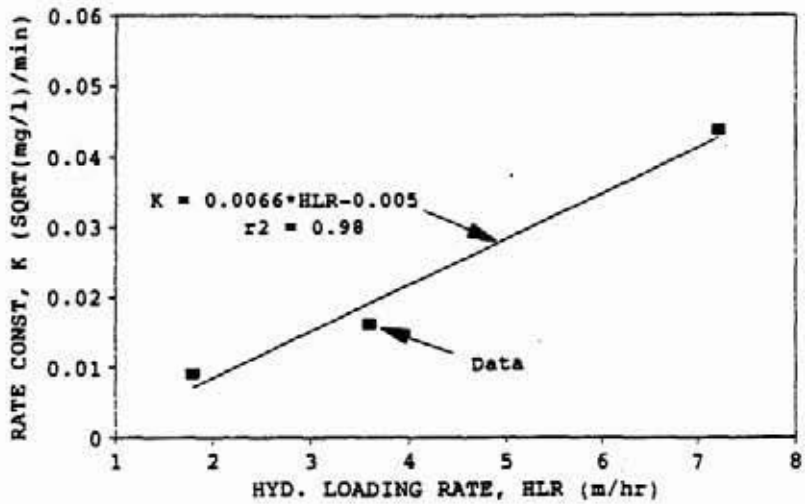
Table 5: Average Half-order Denitrification Coefficients

Hydraulic Loading ($m^3/hr/m^2$)	K ($(mg/l)^{1/2}/min$)	Std. Dev. ($(mg/l)^{1/2}/min$)
1.8	.0090	.0022
3.6	.0161	.0056
7.2	.0437	.0078

Figure 6 shows the half-order reaction coefficient as a function of hydraulic loading. This result contradicts Harremoes' (1976) finding that the half-order

denitrification coefficient was unaffected by changes in hydraulic loading. It was speculated that the period of loading in Harremoes' experiments was shorter than the three week minimum periods used for this project, not allowing time for the biomass to adapt to the new loading conditions. While Harremoes did not specify the durations he used, there is some evidence in his 1976 paper that each loading period was very short.

Figure 6: Half-Order Reaction Coefficients vs. Hydraulic Loading

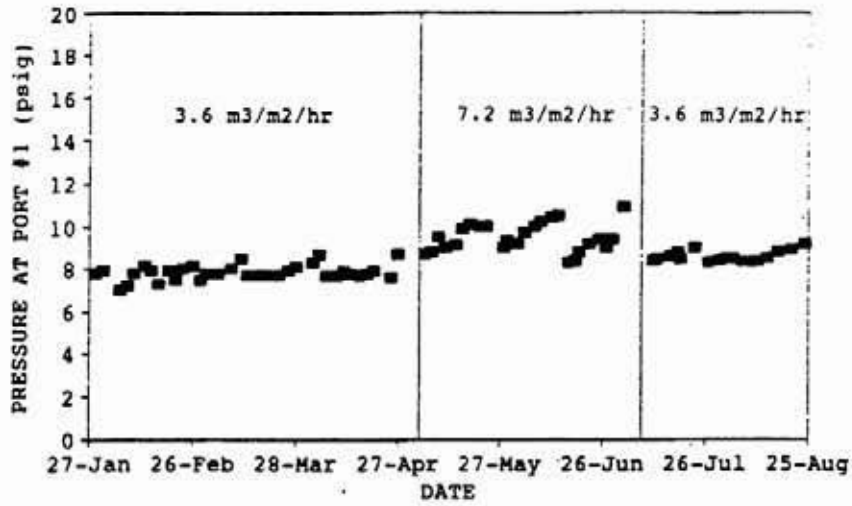


Biomass, Pressure, Effluent Solids and Turbidity

One explanation for the proportional increase in K with flow rate (Figure 6) is that the increase in substrate loading associated with an increase in hydraulic loading resulted in increased DNR biomass growth. In turn, the biological capacity of the DNR was increased, resulting in greater denitrification. This hypothesis is supported directly by scoured solids and DNR pressure results from hydraulic loading periods of 3.6 and 7.2 $m^3/hr/m^2$. Effluent suspended solids and turbidity data from these hydraulic loadings reinforce the conclusion that the DNR biomass is greater at higher hydraulic loading rates.

Biofilm growth is confirmed by the long term DNR pressure profile shown in Figure 7. The general pattern is an increase in pressure from a low value just after air scour to a high value at the end of the 21 day cycle, with the pattern being more pronounced at the higher hydraulic loading (7.2 $m^3/hr/m^2$). Maximum DNR pressure at 3.6 $m^3/hr/m^2$ was approximately 9 psig with a cycle increase of about 1.5 psig. At 7.2 $m^3/hr/m^2$ the maximum DNR pressure was approximately 11 psig with a cycle increase of about 2 psig.

Figure 7: Reactor Influent Pressure, January - September 1990



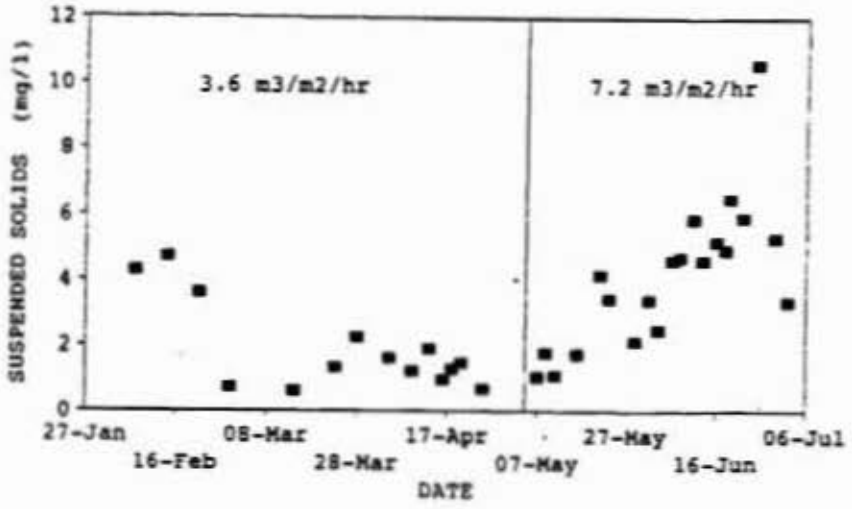
Increased biofilm growth resulting from an increase in hydraulic loading is indicated by an increase in the dry weight of scoured solids drained from the reactor at the higher loading rate. Table 6 shows scoured biomass data for hydraulic loadings of 3.6 and 7.2 $m^3/hr/m^2$. Scoured biomass is approximately 67% greater at 7.2 $m^3/hr/m^2$ than at 3.6 $m^3/hr/m^2$.

Table 6: Scoured Biomass

Date	Hyd. Loading ($m^3/hr/m^2$)	Scoured Biomass (g)	Mean (g)	Std. Dev. (g)
June 14	7.2	100	100	1
July 5	7.2	99	-	-
July 26	3.6	51	59	8
Aug. 6	3.6	67	-	-
Aug. 27	3.6	59	-	-

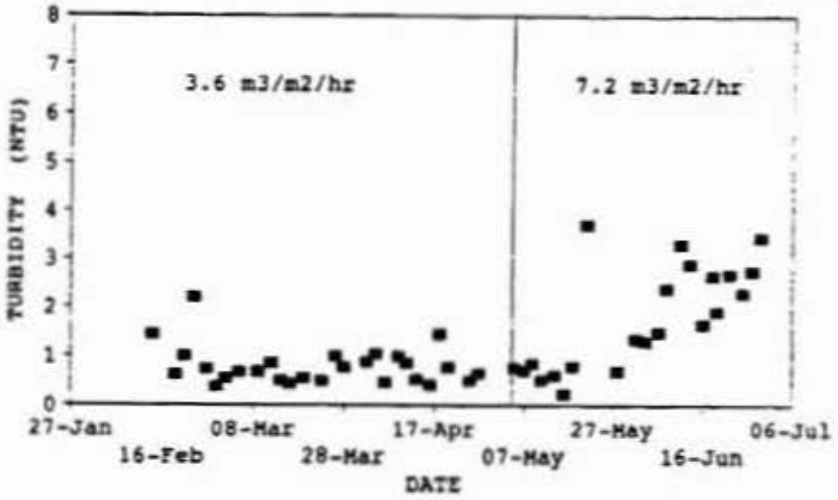
The biomass increase at 7.2 $m^3/hr/m^2$ had a marked effect on the effluent suspended solids. Figure 8 shows that at a hydraulic loading of 7.2 $m^3/hr/m^2$ the effluent suspended solids increased to more than three times the average at 3.6 $m^3/hr/m^2$. It is interesting to note that the effluent suspended solids at 2.2 l/min did not start to increase until after the first cycle, reinforcing the theory that it takes time for the biomass to adjust to the new loading conditions.

Figure 8: Reactor Effluent Suspended Solids, January - July 1990



The biomass increase also had an effect on the effluent turbidity. Figure 9 shows effluent turbidities for hydraulic loadings of 3.6 and 7.2 $m^3/hr/m^2$. It can be seen that the effluent turbidity during the 7.2 $m^3/hr/m^2$ loading period gradually increased until it was double the effluent turbidity at 3.6 $m^3/hr/m^2$. Once again this increase did not begin until after the first cycle at 7.2 $m^3/hr/m^2$, further reinforcing the theory that it takes time for the biomass to adjust to the new loading conditions.

Figure 9: Reactor Effluent Turbidities, January - July 1990



Short Term Response

It was expected that immediately after air scour the denitrification performance of the DNR would decrease for a short period while the biomass recovered before returning to the performance previous to the air scour. It was discovered that the recovery behavior of the DNR operating at a hydraulic loading of 3.6 $m^3/hr/m^2$ differed from the recovery of the DNR operating at 7.2 $m^3/hr/m^2$.

3.6 $m^3/hr/m^2$

The recovery results for the 3.6 $m^3/hr/m^2$ flow rate presented and discussed here are from the period of operation from July 8 to August 29. These results were selected because the water temperature during this period was comparable to that of the June - July experiments at 7.2 $m^3/hr/m^2$. Surprisingly, it was found that the denitrification performance of the DNR operating at 3.6 $m^3/hr/m^2$ showed no decrease after air scour. Figure 10 shows two DNR recovery profiles. Although some variations in nitrate concentration occur in the profiles, the concentration in the first sample of a profile does not differ significantly from the concentration in the last sample of the profile taken over 40 hours later. This is indicated by the relatively flat appearance of the profiles in Figure 10. Statistical analysis of the profile data shown in Table 7 verified that the average of the recovery data was identical (99% probability) to the long term average denitrification during that same period (July 27 - Aug. 28). Thus, it appears that the denitrification capacity of the DNR operating at 3.6 $m^3/hr/m^2$ was not exceeded even though the biomass had been vigorously stripped by the air scour procedure.

Figure 10: DNR Recovery Profiles after Air Scour (3.6 $m^3/hr/m^2$)

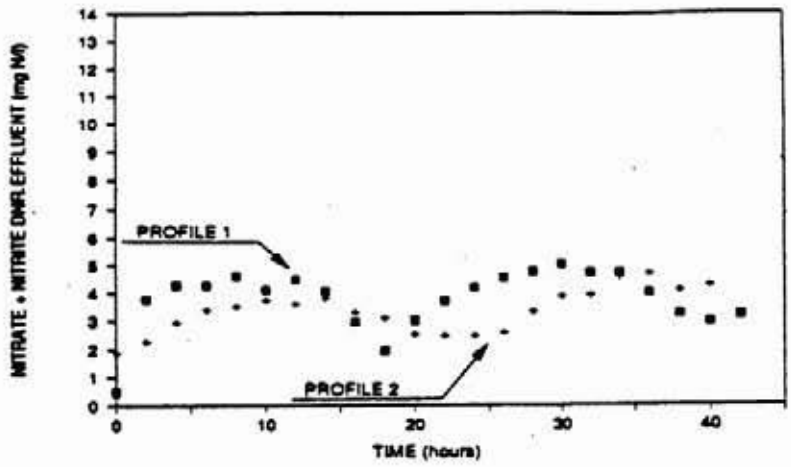


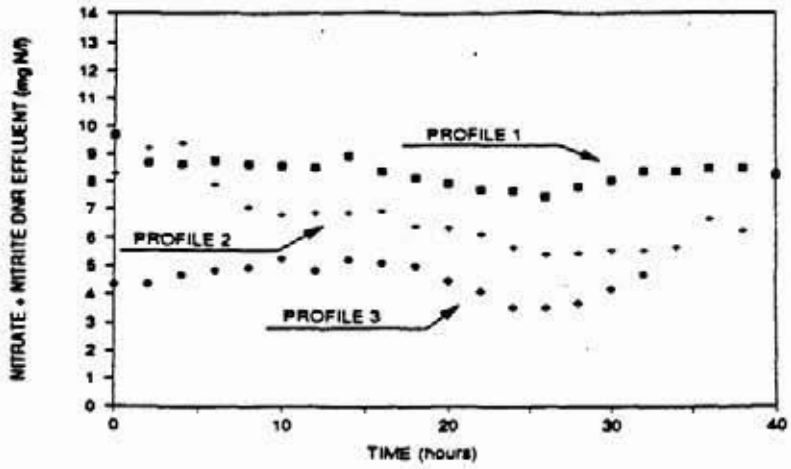
Table 7: Short Term Influent and Effluent Nitrate Response (3.6 $m^3/hr/m^2$)

Hyd. Loading 3.6$m^3/hr/m^2$	Profile 1 Aug. 6 - 8		Profile 2 Aug. 27 - 29		Long Term Denit.	
Statistic:	Mean x_1	Std. Dev.	Mean x_2	Std. Dev.	Mean	Std. Dev.
Influent NO_3 (mg/l - N)	13.0	0.4	13.0	0.4	13.0	0.4
Effluent NO_3 (mg/l - N)	3.4	0.8	3.8	1.0	3.0	1.1
Removal NO_3 (mg/l - N)	9.7	0.8	9.2	1.0	10.0	1.1

7.2 $m^3/hr/m^2$

The recovery response of the DNR operating at a hydraulic loading of 7.2 $m^3/hr/m^2$ was significantly different than the response at 3.6 $m^3/hr/m^2$. Figure 11 shows three recovery profiles. In the first two profiles, taken on May 24 - 26 and June 14 -16, the nitrate concentration in the first sample taken immediately after air scour is significantly greater than the nitrate concentration in the last sample taken 40 hours later. But, by the third cycle, after six weeks of operation, the profile (taken July 5 - 7) was much flatter (as in the profiles shown for 3.6 $m^3/hr/m^2$).

Figure 11: DNR Recovery Profiles after Air Scour (7.2 $m^3/hr/m^2$)



To verify the slope observations made above, linear regressions were performed on each profile. The results of these regressions are shown in Table 8. Using a t-test, it was concluded that $\beta 1 < 0$ and $\beta 2 < 0$, indicating steady improvement in denitrification over the 48 hour recovery period. But, it was determined with 99% probability that $\beta 3 = 0$, indicating no improvement in denitrification during the recovery period and suggesting no decline in the denitrification performance of the DNR after air scour.

Table 8: Profile Linear Regression Slopes (7.2 $m^3/hr/m^2$)

	Profile 1 May 24 - 26	Profile 2 * June 14 - 16	Profile 3 July 5 - 7
Slope (B)	- 0.6	- 0.8	- 0.1
y - intercept	8.8	7.1	4.7

* Note: Samples 1, 2, and 3 were not considered.

It appears from Figure 11 that the average denitrification performance of the reactor after air scour improved after each cycle. This trend was verified by making a statistical comparison of the average nitrate removals, shown in Table 9, for each profile. It was determined with 99% probability that the average denitrification (nitrate removal) in the profiles increased from profile 1 to profile 2 to profile 3, i.e., $x_1 < x_2 < x_3$. In addition, the average denitrification data from profile 3 is similar to the long term average denitrification for the 7.2 $m^3/hr/m^2$ period. It appears that the DNR

adapted to the higher loading rate after the third cycle, and is performing much as it did at a flow rate of 3.6 $m^3/hr/m^2$.

Table 9: Short Term Influent and Effluent Nitrate Response (7.2 $m^3/hr/m^2$)

Hyd. Loading 7.2 $m^3/hr/m^2$	Profile 1 May 24 - 26		Profile 2 June 14 - 16		Profile 3 July 5 - 7	
Statistic:	Mean x_1	Std. Dev.	Mean x_2	Std. Dev.	Mean x_3	Std. Dev.
Influent NO_3 (mg/l - N)	14.0	0.7	14.0	0.7	14.0	0.7
Effluent NO_3 (mg/l - N)	8.3	0.5	6.3	0.6	4.6	0.5
Removal NO_3 (mg/l - N)	5.7	0.5	7.8	0.6	9.4	0.5

6. SLOW SAND FILTER RESULTS

The following is a summary of Sydney's work on slow sand filtration of a biologically denitrified water in Brighton, Colorado (Sydney, 1991).

Run Time

Table 10 shows the run durations for slow sand filter runs 1 through 9. The durations of runs 1 - 6 were unacceptably short (average of 11 days). It was theorized that the high organic carbon loading on the SSF led to intense microbiological activity, clogging the SSF with a thick biofilm growth and that a reduction of the organic carbon loading would lead to longer run times. An aerobic biofilm reactor (trickling filter) was placed upstream of the SSF to act as a roughing filter to reduce the organic carbon loading on the SSF. During run 8, algae growth was apparent in the DNR, so for run 9 the DNR was covered to investigate the effect of algae growth in the DNR on the SSF run times.

Table 10: Run Times for SSF Runs 1 - 9.

SSF Run	Start Date	End Date	Number of Days
1	1/22	1/29	7
2	1/31	2/12	12
3	2/13	2/26	13
4	2/26	3/05	7
5	3/08	3/19	11
6	3/23	4/09	17
Average for Runs 1 - 6:			11
7	5/29	7/07	39
8	7/09	8/24	46
9	8/25	12/04	101 +

The addition of the roughing filter led to a dramatic increase in the SSF run times confirming the theory that high organic carbon loading played a role in the short durations of runs 1 - 6. Runs 7 and 8 had durations of 39 and 46 days respectively. Another dramatic increase in run time occurred during run 9 (with the DNR covered) which ran for 101 days before being terminated with a headloss of only 10 cm, leaving 80 cm still available. This result demonstrated the importance of algae control if maximum SSF run times are to be achieved.

Turbidity

Figures 12, 13, 14, and 15 show turbidity profiles for SSF runs 1 - 6, 7, 8, and 9 respectively. The SSF performed very well with respect to turbidity removal. It can be seen from the turbidity profiles that under widely varying turbidity conditions in the SSF reservoir, the SSF consistently produced an effluent with a turbidity less than the MCL of 1.0 NTU and often produced an effluent with a turbidity less than 0.5 NTU.

Figure 12: Turbidity Profile for Runs 1 - 6

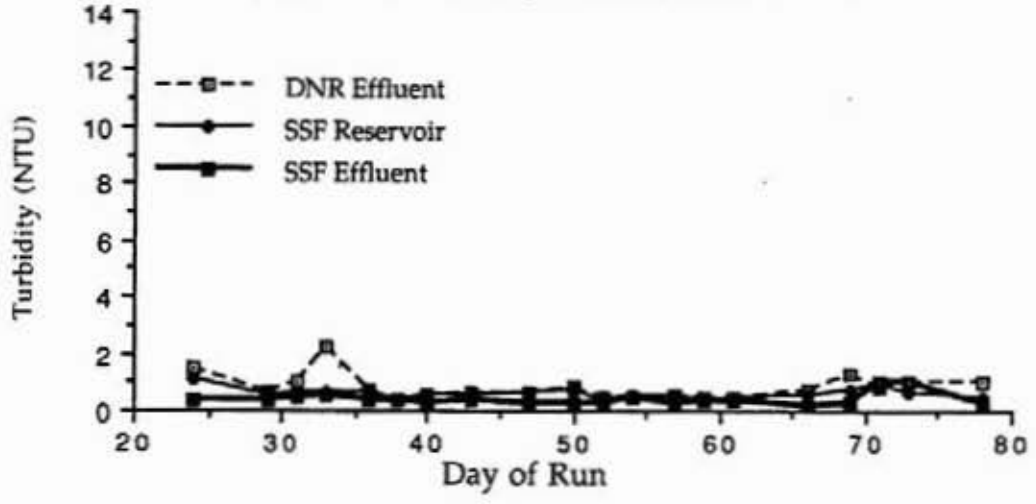


Figure 13: Turbidity Profile for Run 7

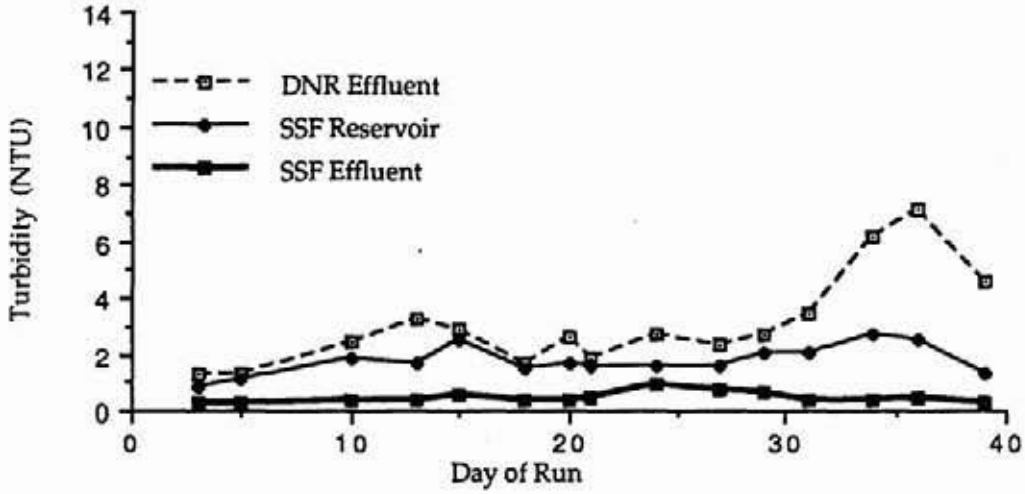


Figure 14: Turbidity Profile for Run 8

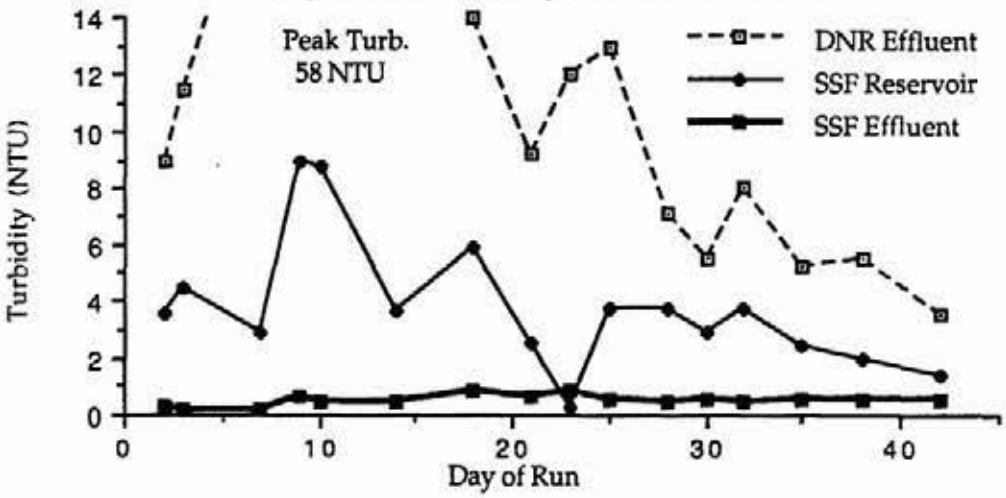
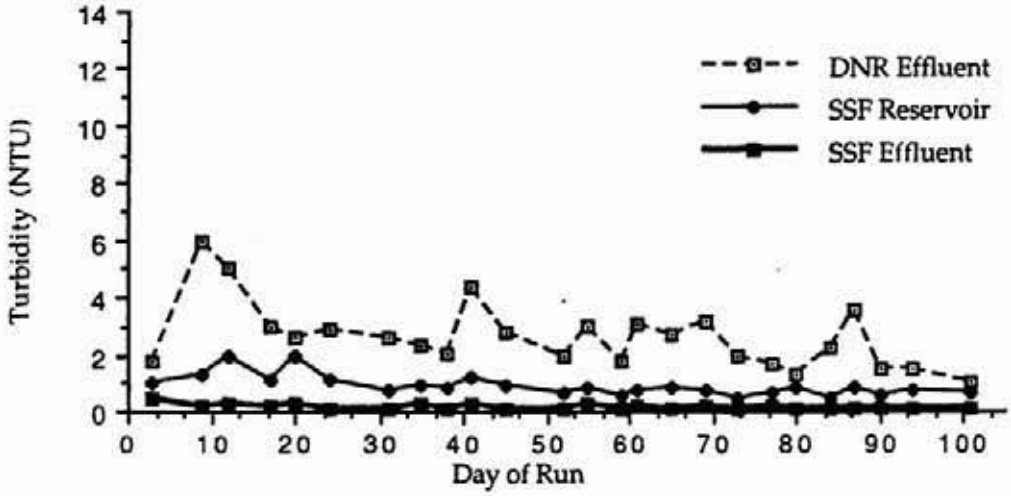


Figure 15: Turbidity Profile for Run 9



Total Suspended Solids (TSS)

Figures 16, 17, 18, and 19 show total suspended solids profiles for runs 1 - 6, 7, 8, and 9 respectively. During runs 1 - 6, the SSF did not remove TSS very efficiently, but after the installation of the roughing filter (runs 7 - 9), the SSF consistently produced an effluent with a TSS concentration less than 1.0 mg/l.

Figure 16: TSS Profile for Runs 1 - 6

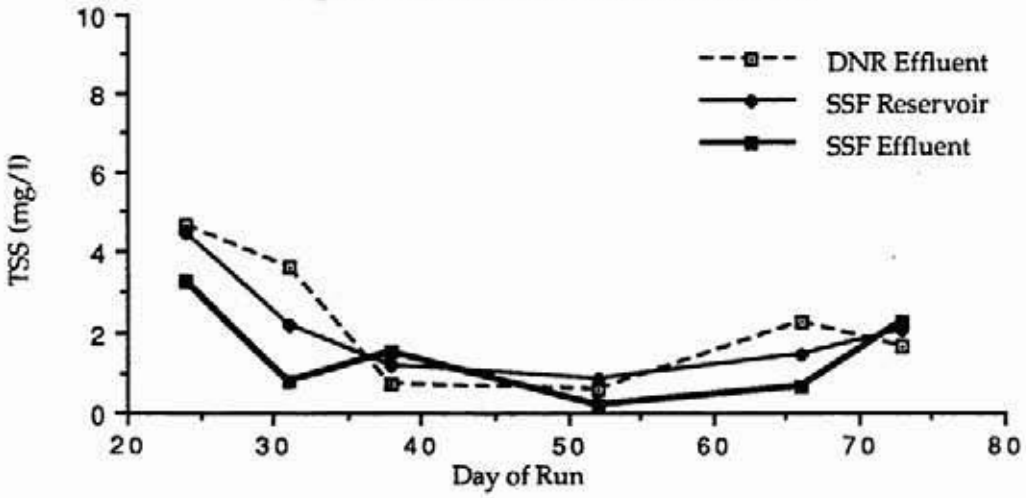


Figure 17: TSS Profile for Run 7

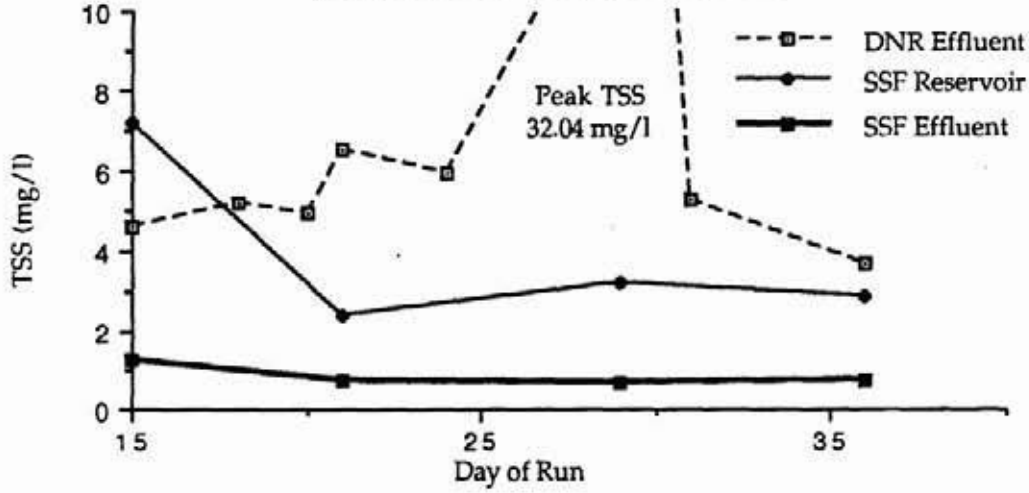


Figure 18: TSS profile for Run 8

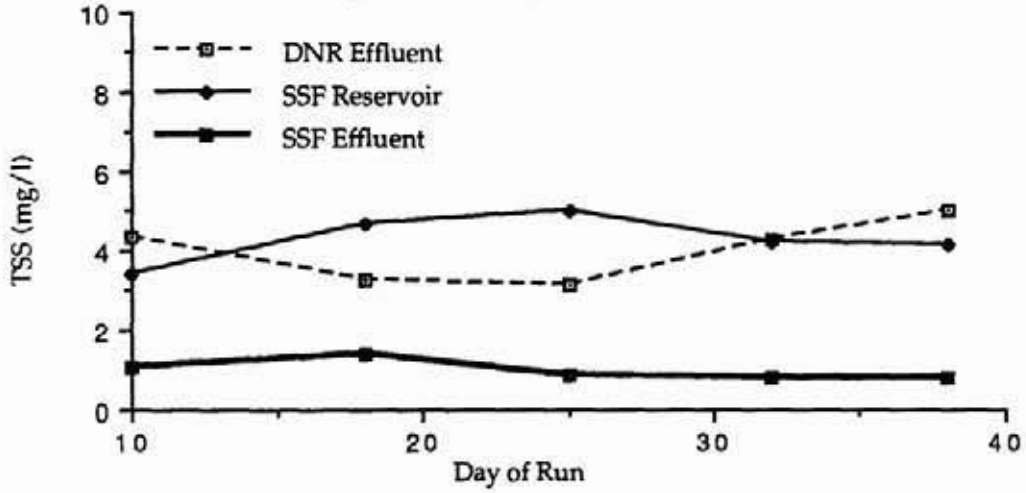
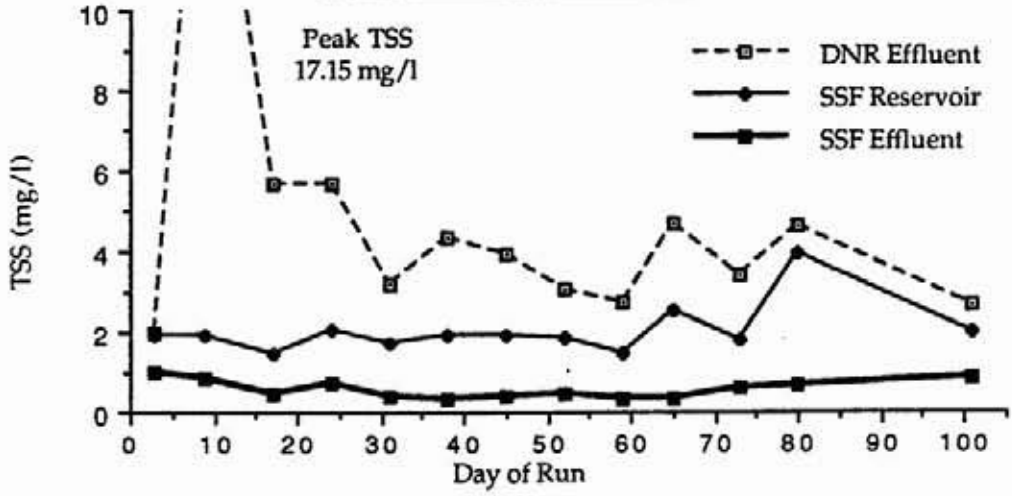


Figure 19: TSS Profile for Run 9



Coliform Bacteria

The presence or absence of coliform bacteria and *E. coli* were investigated for runs 7, 8, and 9. In addition to determining the presence or absence of coliforms, total coliforms were found for run 9. Tables 11, 12, and 13 show the results of the presence/absence tests performed for run 7, 8, and 9 respectively, and Table 14 shows the results of the total coliform investigations performed during run 9.

Table 11: Presence/Absence of coliforms and *E. coli* for Run 7

Date	Dilution	Coliforms		*E. coli*	
		DNR Eff.	SSF Eff.	DNR Eff.	SSF Eff.
6/13	0	+	-	-	-
6/19	0	-	-	-	-
6/27	0	+	+	-	-
7/04	0	+	+	-	-

Table 12: Presence/Absence of coliforms and *E. coli* for Run 8

Date	Dilution	Coliforms		*E. coli*	
		DNR Eff.	SSF Eff.	DNR Eff.	SSF Eff.
7/16	0	+	+	-	-
7/23	0	+	+	-	-
7/23	1/10	+	+	-	-
7/23	1/100	+	+	-	-

Table 13: Presence/Absence of coliforms and *E. coli* for Run 9

Date	Dilution	Coliforms		*E. coli*	
		DNR Eff.	SSF Eff.	DNR Eff.	SSF Eff.
9/03	0	+	+	-	-
9/11	0	+	+	-	-
9/18	0	+	+	-	-
9/25	0	+	+	-	-
10/02	0	+	+	-	-
10/09	0	+	+	-	-
10/09	1/10	NA	+	NA	-
10/09	1/100	NA	-	NA	-
10/16	0	+	+	-	-
10/16	1/10	NA	+	NA	-
10/16	1/100	NA	+	NA	-
10/23	0	+	+	-	-
10/23	1/10	NA	+	NA	-
10/23	1/100	NA	-	NA	-
10/29	0	+	+	-	-
10/29	1/10	NA	+	NA	-
10/29	1/100	NA	-	NA	-

Table 14: Total Coliforms for Run 9

Date	DNR Eff.	SSF Eff.	% Reduction
8/28	930	450	51.6
9/03	330	160	51.5
9/11	570	23	96.0
9/18	1000	10	99.0
9/25	240	5	97.9
10/02	200	10	95.0

Coliform bacteria were present in the SSF effluent on all but two occasions when the sample was undiluted, and at no time were *E. coli* ever discovered. When the samples were diluted to 1/100 for tests run during run 9, only 1 out of 4 results indicated the presence of coliforms, suggesting a coliform concentration of less than 2 logs/ml in the SSF effluent.

The results shown in Table 14 indicate an increase in coliform removal efficiency from approximately 51% to 95% as run time increased. This increase in efficiency is probably due to the maturation of the SSF's biofilm layer or schmutzdecke, leading to an increase in the SSF's pathogenic removal capacity. These results also confirm that the SSF effluent contains an average coliform concentration of less than 2 logs/ml once the filter matures.

Chlorine Demand

The 10 minute chlorine demand was measured four times during run 8. The results of these measurements are shown in Table 15.

Table 15: 10 Minute Chlorine Demand for Run 8

Date	Day of Run	SSF Reservoir (mg/l)	SSF Effluent (mg/l)
7/16	7	3.4	2.4
7/24	15	6.5	4.3
8/06	28	2.1	2.7
8/14	36	13.9	3.5

The SSF effluent had an average chlorine demand of 3.2 mg/l. The wide variations in the chlorine demand of the SSF reservoir may have been caused by the presence of algae. Even with these variations, the SSF produced an effluent with a fairly consistent 10 minute chlorine demand.

Heterotrophic Plate Count

Heterotrophic plate counts were conducted four times during run 8. The results of these counts is shown in Table 16. The results indicate a fairly consistent SSF reservoir and effluent count.

Table 16: Heterotrophic Plate Count for Run 8

Date	Day of Run	SSF Res. (cfu/ml)	SSF Eff. (cfu/ml)	SSF Res. (log/ml)	SSF Eff. (log/ml)
7/23	14	6.0E+5	9.3E+3	5.78	3.97
7/30	21	4.9E+5	2.2E+5	5.69	5.34
8/06	28	1.2E+4	1.6E+4	4.08	4.20
8/14	36	7.0E+5	7.1E+3	5.85	3.85
	Average:	4.5E+5	6.3E+4		

Total Organic Carbon (TOC)

Figures 20, 21, and 22 show TOC profiles for runs 1 - 6, 8, and 9 respectively. A TOC profile was not produced for SSF run 7. Several interesting observations can be made about these figures.

Figure 20: TOC Profile for Runs 1 - 6

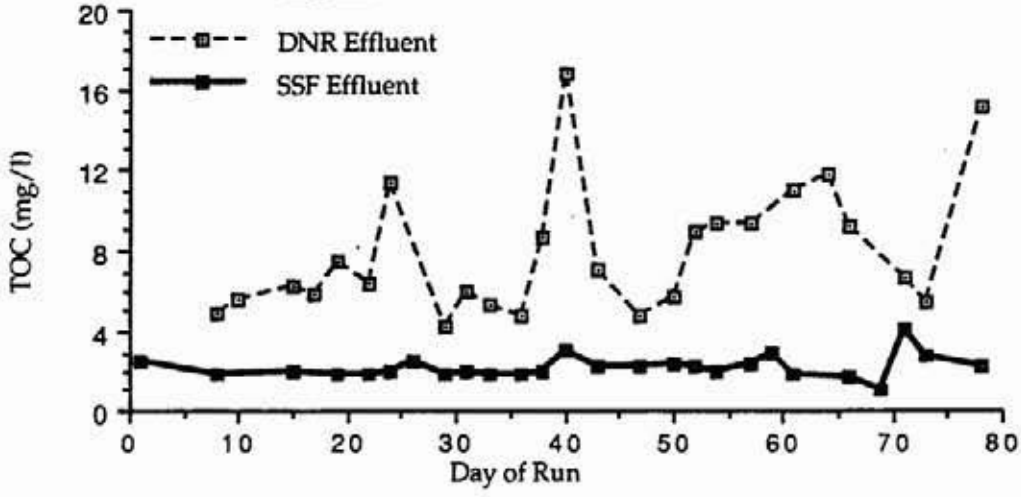


Figure 21: TOC Profile for Run 8

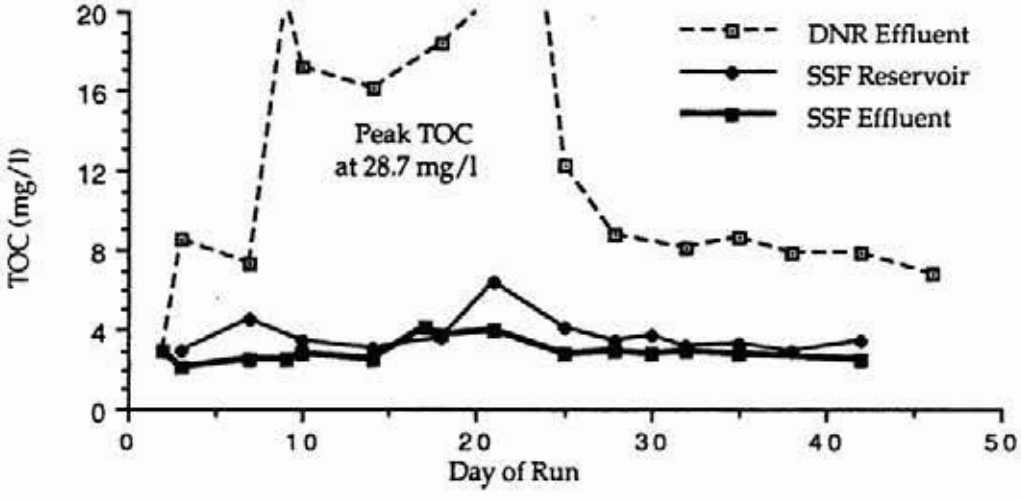
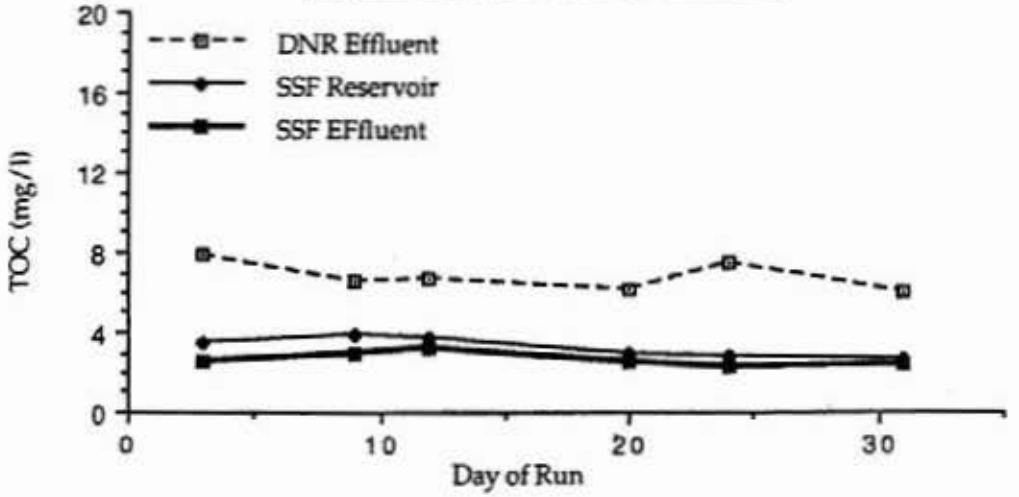


Figure 22: TOC Profile for Run 9



The roughing filter was very effective at removing organic carbon. Figures 21 and 22 show that most of the TOC removed by the polishing process (RF + SSF) during runs 8 and 9 was removed by the roughing filter, with only a small amount subsequently removed by the SSF. It can be seen in Figure 20 that without the roughing filter, the SSF was capable of reducing the TOC to approximately the same levels as in runs 8 and 9, but at the expense of run time.

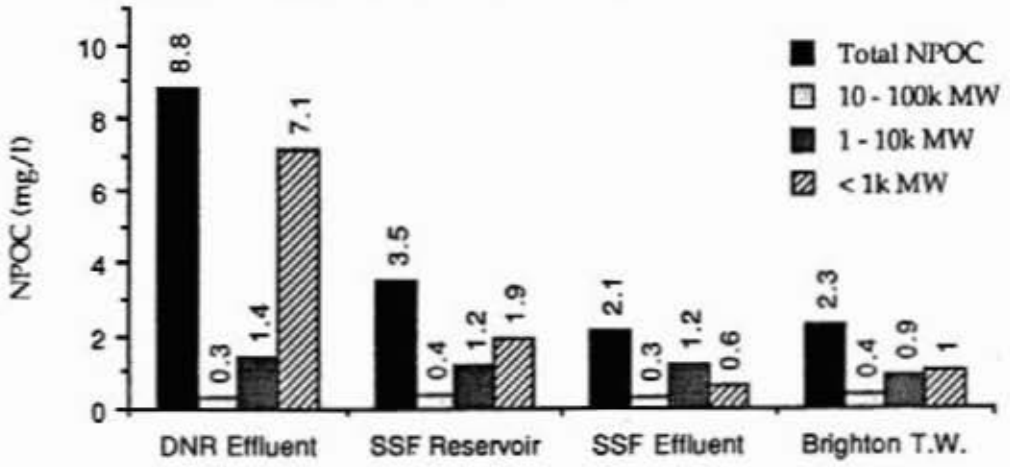
The roughing filter also proved to be very effective at protecting the SSF from organic carbon spikes. Figure 21 shows a region of high TOC in the DNR effluent due to an accidental overfeeding of acetic acid to the DNR from days 9 to 28. The roughing filter almost completely reduced this TOC spike, protecting the SSF from a high organic carbon loading and a possible decrease in run time.

It was interesting that the TOC concentration in the SSF effluent was very consistent for all of the runs, averaging 2.5 mg/l (std. dev. = 0.6). Because of this consistency and the low TOC removal by the SSF in runs 8 and 9, it was theorized that the TOC remaining in the SSF effluent was not very biodegradable and was possibly present in the DNR influent (Brighton tap water).

Apparent Molecular Weight Distribution of Total Nonpurgable Organic Carbon

To better understand the addition and removal of nonpurgable organic carbon (NPOC) throughout the system, an investigation of the apparent molecular weight (AMW) distribution of total NPOC was conducted during run 9. The distribution was determined for samples taken from Brighton tap water, the DNR effluent, the SSF reservoir, and the SSF effluent. The results are shown in Figure 23. It is important to note that the NPOC for the SSF effluent and Brighton tap water were measured on different days than the DNR effluent and SSF reservoir (Brighton tap water (12/04/90), SSF Effluent (11/27/90), DNR Effluent (11/6/90), SSF Res. (11/6/90)).

Figure 23: Apparent Molecular Weight Distribution for Run 9



Examining the figure, it appears that the concentration (mg/l) of NPOC with an AMW of 100k - 10k and 10k - 1k remain constant throughout the system, while the concentration (mg/l) of NPOC with an AMW distribution less than 1k undergoes an increase from Brighton tap water to the DNR effluent and a decrease from the DNR effluent to the SSF effluent. It is important to note the similarity between the AMW distribution of Brighton tap water and the AMW distribution of the SSF effluent. It appears that any readily biodegradable organic carbon is completely removed by the system (DNR + RF + SSF), leaving the less biodegradable organic carbon and producing an effluent very similar to the system influent (Brighton tap water).

7. CONCLUSIONS

The purpose of operating a pilot scale denitrification plant in Brighton was to demonstrate the operation of the plant under conditions which are typical of a small community. A more specific goal was to investigate the response of the denitrification reactor to variations in hydraulic loading and periodic air scourings. The performance of the polishing process was also investigated to determine if a potable quality water was produced by the system.

The denitrification reactor performed well under field conditions. It was discovered that the reactor could accommodate changes in hydraulic loading without a decrease in denitrification performance, contradicting other researcher's findings. It is believed that this was due to increased biomass growth in the reactor with a corresponding increase of the combined half-order rate coefficient. This was verified by scoured biomass and reactor pressure measurements. Short term denitrification performance recovery after air scour, which was normally immediate, exhibited a lag period at the beginning of a higher hydraulic loading stage. However, after six weeks the biomass adapted to the new loading conditions and short term recovery once again became immediate.

The roughing filter/slow sand filter polishing system also performed well under field conditions. The polishing system consistently produced water with a turbidity of 0.5 NTU after maturation of the slow sand filter's schmutzdecke. In addition, total coliform bacteria were reduced by over 95% and no *E. coli* were ever found. It was also found that the effluent water exhibited the same TOC "fingerprint" as the normal Brighton water supply, indicating that the polishing process was capable of producing a denitrified water of drinking water quality.

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