Design and optimization of two recirculating sand filters for the removal of nitrogen and organic chemicals from domestic wastewater. [DNR-095] 1998

Graham, Joseph; Shaw, Byron H.
Madison, Wisconsin: Wisconsin Department of Natural Resources, 1998

DESIGN AND OPTIMIZATION OF TWO RECIRCULATING SAND FILTERS FOR THE REMOVAL OF NITROGEN AND ORGANIC CHEMICALS FROM DOMESTIC WASTEWATER

Joseph Graham
Byron Shaw

January 1998

Project Final Report to Wisconsin DNR

DISCLAIMER

This report does not necessarily reflect the views of the Wisconsin Department of Natural Resources, and no official endorsement should be inferred. Mention of trade names of commercial products is not an endorsement or recommendation for use.

ACKNOWLEDGMENTS

The authors of this report would like to recognize and thank the owners of the private sewage systems for their continued cooperation and patience in allowing these experimental systems to be installed on their properties.

We appreciate the grants received from the Wisconsin Department of Natural Resources, which allowed this study to take place.

Water and wastewater analysis was done by the University of Wisconsin-Stevens Point Environmental Task Force. We express our gratitude to the Task Force staff who assisted in the laboratory and field.

We would also like to thank Ralph Loeffler and Raymond Schmidt of the Portage County Planning and Zoning Department for their assistance in designing and drawing up the plans for these systems.

We also appreciate the efforts of Jim Tuszka, an electrician at UW-Stevens Point, for his assistance in the design and troubleshooting of the electrical timing systems.

ABSTRACT

Removal of nitrogen from domestic wastewater prior to discharge to a soil absorption system can help to prevent nitrate concentrations in groundwater from exceeding the drinking water standard. This is of increasing importance in Wisconsin and elsewhere, as rural development increases and people demand high quality drinking water.

This study involved the design, installation, and optimization of two sand filter denitrification systems on private residences in Central Wisconsin. The recirculating sand filter design involved the use of 7,571 l (2,000 gal) septic tanks with wooden covers. The systems had foam insulation to conserve heat during Wisconsin's cold winters. The sand filter contained a rock storage layer consisting of 41 cm (16 in) of 2.5 to 3.8 cm (1 to 1.5 in) limestone. The rock storage layer was fully saturated to promote denitrification in the sand filter. The filter media overlaying the rock storage layer consisted of 7.6 cm (3 in) of pea gravel and 61 cm (24 in) of coarse sand. The sand filter also contained a pump chamber with a pump and flow regulation equipment to recirculate various amounts of effluent to the top of the sand filter, back to the septic tank for denitirification, and to the soil absorption system for disposal.

One of the sand filters was installed on a mound system, the other on a conventional soil absorption system. At the mound system site, effluent was pumped to the top of the sand filter from a pump located in a sewage ejector pit. At the conventional soil absorption system site effluent flowed by gravity from the septic tank to the bottom of the sand filter. The method of applying effluent to the top of the sand filter from a pump was much more effective than the gravity distribution system at the conventional site, where occasional clogging of the gravity distribution system resulted in effluent bypassing the sand filter.

Temperature was a minor problem at one site during a severe winter with low water and heat inputs, but was not a problem at the other site which had more normal water usage.

A series of studies were conducted varying the recirculation rates to the sand filter, septic tank, and the rates discharged to the soil absorption systems. Nitrogen removal achieved by these denitrification systems averaged between 70 and 76%. More nitrogen was removed in the rock storage unit of the sand filters (48%) than in the septic tanks (28%). BOD_5 and COD removal averaged 95 and 88%, respectively. Total phosphorous removal was variable, ranging between 18 and 74%. These systems also showed good potential for attenuating the volatile organic compounds (VOCs) typically found in private septic systems.

A minimum recirculation rate to the sand filter of three to six times the daily water usage was required to achieve at least 90% conversion of NH_4^+-N to nitrate-N. The most practical total nitrogen removal by the RSF systems was 85%, achieved with sand filter recirculation rates approximately eight times the daily

water usage. The circulation rate to the septic tank should be limited to approximately three times the daily water usage to ensure complete denitrification by maintaining anaerobic conditions and a sufficient carbon supply.

Nitrate-N reductions in down-gradient monitoring wells showed average reductions between 53 and 73%. These reductions reflected bypass of effluent at one site and the nitrogen removal capabilities of the RSF system at the other site. Due to dilution by groundwater, concentrations in these downgradient wells were generally half of the concentrations discharged. Denitrification was not found to occur in the soil absorption systems or in the groundwater.

RSF systems that incorporate denitrification should be adopted into the current on-site sewage treatment code because they produce a high quality effluent and protect groundwater quality. It is recommended that RSF systems be monitored once every six months to ensure that they are maintained and functioning properly. It is also recommended that some type of warning mechanism be incorporated into these systems to make homeowners aware of any system failures.

Table of Contents

ACKNOWLEDGMENTS ii

ABSTRACT iii

TABLE OF CONTENTS v

LIST OF TABLES vii

LIST OF FIGURES ix

LIST OF APPENDICES xi

1.0 INTRODUCTION AND JUSTIFICATION 1
- 1.1 Introduction 1
- 1.2 Justification 2
- 1.3 Objectives 3

2.0 LITERATURE REVIEW 4
- 2.1 On-Site Waste Disposal Systems 4
- 2.2 Nitrification and Denitrification Processes 9
- 2.3 Nitrogen Removal Systems for On-Site Waste Disposal 13
- 2.4 Volatile Organic Compounds 18

3.0 METHODS 19
- 3.1 Study Sites 19
  - 3.11 Study Site #1 20
  - 3.12 Study Site #2 20
- 3.2 Denitrification System Design 22
  - 3.21 Major Components 22
  - 3.22 Temperature Probes 30
- 3.3 Denitrification System Operation 30
  - 3.31 Theory of Operation 30
  - 3.32 Flow Control 32
    - 3.321 Flow Control 'Valves' 32
    - 3.322 Electrical Controls 36
- 3.4 Denitrification System Monitoring 38
  - 3.41 Wastewater Sampling 38
  - 3.42 Temperature Recordings 39
  - 3.43 Flow Rates 39
- 3.5 Monitoring Well installation and Design 40
  - 3.51 Study Site #1 40
  - 3.52 Study Site #2 45
  - 3.53 Groundwater Sampling Techniques 47

TABLE OF CONTENTS (Continued)

4.0 RESULTS 49
    4.1 Study Site #1 49
        4.11 Temperature Variations 50
        4.12 Flow Conditions 54
            4.121 Study Period #1 54
            4.122 Design Flow Conditions 60
            4.123 Actual Flow Conditions 69
        4.13 Mass Balance Calculations 73
        4.14 BOD_5, COD, and Total Phosphorous 77
        4.15 Nitrogen 86
            4.151 Nitrification 86
            4.152 Denitrification 95
            4.153 Total Nitrogen 101
        4.16 Volatile Organic Compound Attenuation 114
        4.17 Study Periods Omitted 116
        4.18 Groundwater Impacts 122
            4.181 Installation of Monitoring Wells 122
            4.182 Water Table Fluctuations 127
            4.183 Nitrate-N Reductions 129
            4.184 Phosphate 139
            4.185 Volatile Organic Compounds 141

    4.2 Study Site #2 143
        4.21 Temperature Variations 144
        4.22 Flow Conditions 147
            4.221 Design Flow Conditions 150
            4.222 Actual Flow Conditions 154
        4.23 Mass Balance Calculations 157
        4.24 BOD_5, COD, and Total Phosphorous 162
        4.25 Nitrogen 170
            4.251 Nitrification 170
            4.252 Denitrification 179
            4.253 Total Nitrogen 186
        4.26 Volatile Organic Compound Attenuation 196
        4.27 Groundwater Impacts 200
            4.271 Water Table Fluctuations 200
            4.272 Nitrate-N Reductions 203
            4.273 Phosphate 214
            4.274 Volatile Organic Compounds 215

5.0 SUMMARY OF EFFLUENT TREATMENT BY THE RSF SYSTEMS 216
        5.1 Study Site #1 216
        5.2 Study Site #2 218
        5.3 Optimal Flow Conditions 221

TABLE OF CONTENTS (Continued)

6.0 CONCLUSIONS AND RECOMMENDATIONS 223

7.0 LITERATURE CITED 225

LIST OF TABLES

Table 4.1 Settings of the programmable timers, design flow rates, and actual duration of pump operation at Site #1 for all study periods after the installation of the "dam". 61

Table 4.2 Comparison of design and actual flow rates to the drainfield with water usage for all study periods after the installation of the "dam" at site #1. 64

Table 4.3 Dates omitted from the flow rate and concentration averages used in the mass balance calculations at site #1 70

Table 4.4 Average hydraulic conditions for all study periods after the installation of the "dam" at site #1. 71

Table 4.5 Flow ratios for the average hydraulic conditions for all study periods after the installation of the "dam" at site #1. 72

Table 4.6 BOD_5 removal by the RSF system at site #1. 81

Table 4.7 COD removal by the RSF system at site #1. 83

Table 4.8 Total Phosphorous removal by the RSF system at site #1. 84

Table 4.9 Concentrations of water quality characteristics in the sand filter monitoring well, lysimeter, pump chamber and septic tank at site #1. 86

Table 4.10 Changes in NH_3-N and nitrate-N concentrations from single pass sand filter systems. 87

Table 4.11 Theoretical changes in NH_4-N and nitrate-N concentrations in the RSF for all study periods used in the evaluation of effluent treatment at site #1. 89

Table 4.12 Average removal of nitrate-N in the septic tank by the denitrification process for all the study periods used in the evaluation of effluent treatment at site #1. 96

Table 4.13 Average nitrate-N, temperature, septic tank ratio (STR), BOD_5, COD, and pH in the septic tank at site #1. 100

Table 4.14 Variables, coefficients, and probability values for the multiple regression model of denitirification in the septic tank at site #1. 101

Table 4.15 Total Nitrogen removal for the RSF system at site #1 based on mass applied to the RSF. 103

Table 4.16 Total Nitrogen removal for the RSF system at site #1 based on mass applied to the drainfield. 104

Table 4.17 Percent daily loading and removal of Total N mass for the RSF system at site #1. 109

Table 4.18 Volatile Organic Compounds detected in the septic tank and pump chamber of the RSF at site #1. 115

Table 4.19 Average, ranges and percent removals of several water quality characteristics for the septic tank and pump chamber of the RSF at site #1, study period #1. 117

Table 4.20 Concentrations of several water quality characteristics in the septic tank and pump chamber of the RSF from before, and after the installation of the system during study period #1, at site #1. 119

LIST OF TABLES (continued)

Table 4.21 Concentrations of chemical species in the septic tank and sand filter before, during, and after the pump failure of study period R. 121
Table 4.22 Elevations of the monitoring wells and screen lengths at site #1. 123
Table 4.23 Average nitrogen concentration (Total Nitrogen or Nitrate-N) in the septic tank, RSF, well nest REC, and well REE UG at site #1. 131
Table 4.24 Average chloride concentrations in the septic tank, RSF, well nest REC and well REE UG at site #1. 131
Table 4.25 Average Nitrogen:Chloride ratios for the septic tank, RSF, well nest REC, and well REE UG at site #1. 134
Table 4.26 Settings of the programmable timers, design flow rates, and actual duration of pump operation for all study periods at site #2. 151
Table 4.27 Average hydraulic conditions for all study periods at site #2. 155
Table 4.28 Flow ratios for the average hydraulic conditions for all study periods at site #2. 156
Table 4.29 Dates omitted from the flow rate and mass balance calculations at site #2. 157
Table 4.30 BOD$_5$ removal by the RSF system for all study periods at site #2. 166
Table 4.31 COD removal by the RSF system for all study periods at site #2. 168
Table 4.32 Total Phosphorous removal by the RSF system for all study periods at site #2. 169
Table 4.33 Concentrations of water quality characteristics in the sand filter lysimeter, monitoring well, pump chamber, in Dosing Chamber #1 and in Dosing Chamber #2 at site #2. 171
Table 4.34 Theoretical changes in NH$_4$-N and nitrate-N concentrations in the RSF for all study periods used to evaluate effluent treatment by the RSF at site #2. 172
Table 4.35 Average removal of nitrate-N in the septic tank by the denitrification process at site #2. 180
Table 4.36 Average percent denitrification, temperature, septic tank ratio (STR), BOD$_5$, COD, and pH in the septic tank at site #2 for all study periods. 184
Table 4.37 Variables, coefficients, and probability values for the multiple regression model of denitrification in the septic tank at site #2. 185
Table 4.38 Total Nitrogen removal by the RSF system for all study periods at site #2. 191
Table 4.39 Volatile Organic Compounds detected in Dosing Chamber #1, the pump chamber of the RSF, and Dosing Chamber #2 at site #2. 198
Table 4.40 Nitrate-N and chloride concentrations in groundwater monitoring well DGE for all sample dates after the installation of the RSF at site #2. 206
Table 4.41 Nitrate-N and chloride concentrations in groundwater monitoring well DGW for all sample dates after the installation of the RSF at site #2. 207
Table 4.42 Average nitrate-N and chloride concentrations, and nitrate-chloride ratios before and after the installation of the RSF system at site #2. 208
Table 4.43 Total Nitrogen:Nitrate-N and Chloride to Chloride ratios for before and after the installation of the RSF at site #2. 212
Table 4.44 Ranges and average phosphate concentrations for all depths and sample dates of wells DGE, DGW, and upgradient well SNINEW. 214
Table 5.1 Average, minimum, and maximum percent removal of BOD5, COD, Total Phosphorous, and Total Nitrogen achieved by the various flow conditions at site #1. 216

LIST OF TABLES (continued)

Table 5.2 Average, minimum, and maximum percent removal of BOD5, COD, Total Phosphorous, and Total Nitrogen achieved by the various flow conditions at site #2. 219
Table 5.3 Comparison of study sites with flow conditions that will achieve approximately 85% nitrogen removal. 222

LIST OF FIGURES

Figure 2.1 Typical recirculating sand filter. 15
Figure 3.1 Location of study sites in the Central Sands region of Portage County, WI. 19
Figure 3.2 Top view of system layout at site #1 before the installation of the denitrification system. 20
Figure 3.3 Top view of system layout at site #2 before the installation of the denitrification system. 20
Figure 3.4 Cross sectional view of the RSF at site #1. 24
Figure 3.5 Cross sectional view of the RSF at site #2. 25
Figure 3.6 Top view of the collection system located on the bottom of the RSF. 27
Figure 3.7 Cross section of the pump chamber in the RSF. 27
Figure 3.8 Top view of the distribution system located on the top of the RSF at site #1. 28
Figure 3.9 Cross section of the gravity distribution system for the septic tank effluent into the RSF at site #1. 28
Figure 3.10 Top view of the distribution system on the top of the RSF at site #2. 29
Figure 3.11 Top view of the four thermocouple groups in the RSF at site #1. 31
Figure 3.12 Cross sectional view of individual thermocouple depth at site #1. 31
Figure 3.13 Top view of the system layout at site #1 after the installation of the RSF. 33
Figure 3.14 Top view of system layout at site #2 after the installation of the RSF. 33
Figure 3.15 Side and cross sectional view of the flow control 'valve'. 35
Figure 3.16 Electrical schematic of the timer control system. 37
Figure 3.17 Monitoring well locations at site #1. 42
Figure 3.18 REC and REW well design at site #1. 42
Figure 3.19 Multiport well design at site #1. 44
Figure 3.20 Cross section of multiport wells at site #1. 44
Figure 3.21 Location of down-gradient wells at site #2. 46
Figure 3.22 DGW and DGE well design at site #2. 46
Figure 4.1 Temperature readings in the septic tank and pump chamber of the sand filter at site #1. 51
Figure 4.2 Average temperature readings for all depths in the RSF at site #1. 52
Figure 4.3 Temperature readings at various depths in the RSF at site #1. 53
Figure 4.4 Side profile of the upward pitch in the existing line to the drainfield at site #1. 54
Figure 4.5 Flow schematic for study period #1. 57
Figure 4.6 Cross section and side view of the "dam" installed in the drainfield line. 59
Figure 4.7 Side profile of the drainfield line with the "dam" installed. 59
Figure 4.8 Water use at site #1 from 1991 to 1995. 63

LIST OF FIGURES (continued)

Figure 4.9 Cross section of RSF at site #1 showing bypass and delay interval levels and also problem with partial clogging of the gravity distribution system. 68
Figure 4.10 Schematic for the mass balance calculations at site #1. 74
Figure 4.11 BOD_5, concentrations in the septic tank and pump chamber of the RSF at site #1 from 1991-1995. 78
Figure 4.12 COD concentrations in the septic tank and pump chamber of the RSF at site #1 from 1991 to 1995. 79
Figure 4.13 Total P concentrations in the septic tank and pump chamber of the RSF at site #1 from 1991-1995. 80
Figure 4.14 Percent conversion of ammonium to nitrate-N in the sand filter vs hydraulic loading rate at site #1. 91
Figure 4.15 Percent conversion of ammonium to nitrate-N in the sand filter vs sand filter ratio (SFR) at site #1. 91
Figure 4.16 Nitrate-N concentrations and temperature in the pump chamber of the RSF at site #1 from 1991-1995. 92
Figure 4.17 Alkalinity and total hardness concentrations in the septic tank and pump chamber of the RSF at site #1 from 1991-1995. 94
Figure 4.18 Nitrate-N concentrations and temperature in the septic tank at site #1 from 1991-1995. 98
Figure 4.19 pH in the septic tank at site #1 from 1991-1995. 99
Figure 4.20 Total Nitrogen and chloride concentrations in the septic tank at site #1 from 1991-1995. 105
Figure 4.21 Total Nitrogen and chloride concentrations in the sand filter at site #1 from 1991-1995. 106
Figure 4.22 Flow schematic for study period C at site #1. 108
Figure 4.23 Flow schematic for study period J at site #1. 111
Figure 4.24 Percent Total Nitrogen removal in the septic tank and sand filter at site #1. 113
Figure 4.25 Watertable fluctuations: wells REE DG and REE UG, 1988-1995. 124
Figure 4.26 Chloride concentrations in wells REE DG and REE UG, 1988-1995. 124
Figure 4.27 Preferential discharge of the drainfield at site #1. 126
Figure 4.28 Watertable fluctuations and screened interval of each well at well nest REC, 1988-1995, site #1. 128
Figure 4.29 Watertable fluctuations and screened interval of each port for multiport well RSDS C, 1988-1995, site #1. 128
Figure 4.30 Nitrate-N concentrations in well nest REC, 1988-1995, site #1. 130
Figure 4.31 Chloride concentrations in well nest REC, 1988-1995, site #1. 130
Figure 4.32 Average nitrate-N and chloride concentrations in RSDS multiport wells before the installation of the RSF system at site #1. 138
Figure 4.33 Average nitrate-N and chloride concentrations in RSDS multiport wells after the installation of the RSF system at site #1. 138
Figure 4.34 Phosphate concentrations in well nest REC, 1988-1995. 140
Figure 4.35 Phosphate concentrations in wells REE DG and REE UG, 1988-1995. 140
Figure 4.36 Phosphate concentrations in multiport wells RSDS C, 1988-1995. 142
Figure 4.37 Temperature readings in Dosing Chamber #1, the pump chamber of the sand filter, and Dosing Chamber #2, at site #2, 1992-1995. 145
Figure 4.38 Temperature readings for all depths in the RSF at site #2. 146
Figure 4.39 Flow schematic for study period #1 at site #2. 148

LIST OF FIGURES (continued)

Figure 4.40	Flow schematic for study period #2 at site #2.	149
Figure 4.41	Water use at site #2 from 1991 to 1995.	153
Figure 4.42	Schematic for the mass balance calculations at site #2.	159
Figure 4.43	BOD in Dosing Chamber #1, Dosing Chamber #2, and the pump chamber of the RSF at site #2 from 1991 to 1995.	163
Figure 4.44	COD in Dosing Chamber #1, Dosing Chamber #2, and the pump chamber of the RSF at site #2 from 1991 to 1995.	164
Figure 4.45	Total P in Dosing Chamber #1, Dosing Chamber #2, and the pump chamber of the RSF at site #2 from 1991 to 1995.	165
Figure 4.46	Percent conversion of ammonium to nitrate-N in the sand filter vs hydraulic loading rate at site #2.	174
Figure 4.47	Percent conversion of ammonium to nitrate-N in the sand filter vs sand filter ratio (SFR) at site #2.	174
Figure 4.48	Nitrate-N concentrations and temperature in the pump chamber of the RSF at site #2.	175
Figure 4.49	Alkalinity in Dosing Chamber #1, Dosing Chamber #2, and the pump chamber of the RSF at site #2 from 1991-1995.	176
Figure 4.50	Total Hardness in Dosing Chamber #1, Dosing Chamber #2, the pump chamber of the RSF at site #2, from 1991-1995.	177
Figure 4.51	Nitrate-N concentrations and temperature in Dosing Chamber #1 at site #2.	181
Figure 4.52	pH in Dosing Chamber #1 at site #2 from 1991-1995.	183
Figure 4.53	Total Nitrogen and chloride concentrations in Dosing Chamber #1 at site #2 from 1991 to 1995.	187
Figure 4.54	Total Nitrogen and chloride concentrations in the pump chamber of the RSF at site #2 from 1991 to 1995.	188
Figure 4.55	Total Nitrogen and chloride concentrations in Dosing Chamber #2 at site #2 from 1991 to 1995.	189
Figure 4.56	Flow Schematics for study period H at site #2.	193
Figure 4.57	Percent Total N removal in the septic tank and sand filter at site #2.	195
Figure 4.58	Location of monitoring wells at site #2.	201
Figure 4.59	Watertable fluctuations in wells DGE and DGW at site #2.	202
Figure 4.60	Nitrate-N concentrations in well nest DGE 1990- 1995 site #2.	204
Figure 4.61	Chloride concentrations in well nest DGE 1990- 1995 site #2.	204
Figure 4.62	Nitrate-N concentrations in well nest DGW 1990 -1995 site #2.	205
Figure 4.63	Chloride concentrations in well nest DGW 1990- 1995 site #2.	205
Figure 4.64	Nitrate-N concentrations in up-gradient well SNINEW 1990-1995 site #2	210
Figure 4.65	Chloride concentrations in up-gradient well SNINEW 1990-1995 site #2	210

LIST OF APPENDICES

(NOTE: APPENDICES NOT INCLUDED WITH ALL COPIES)

Appendix A	Mass Balance Calculations at Site #1.	A-1
Appendix B	Mass Balance Calculations at Site #2.	B-1

LIST OF APPENDICES (continued)

Appendix C RSF System Chemistry Data, Study Site #1 C-1

Appendix D RSF System Chemistry Data, Study Site #2 D-1

Appendix E Groundwater Chemistry Data, Study Site #1 E-1

Appendix F Groundwater Chemistry Data, Study Site #2 F-1

1. INTRODUCTION AND JUSTIFICATION

1.1 Introduction

The central Wisconsin sand plain region contains some of the largest and most productive aquifers in the state of Wisconsin. The sandy glacial outwash soils of the area are highly permeable and the terrain is relatively flat. These factors combined with a relatively shallow unconfined aquifer, make the groundwater of this region particularly susceptible to contamination from various land use practices.

One of the land use practices which is increasingly a concern, in Wisconsin and elsewhere, involves the residential development of unsewered areas. Trends in the 1970's and 1980's saw population increases in suburban areas. As a result, suburban development quickly expanded beyond the reaches of municipal water and sewage, and thus private sewage systems became common.

The primary purposes of private sewage systems are the disposal of wastewater and the removal of bacteria. Soil absorption systems are designed to receive wastewater from a septic tank and dispose of it below ground, where it is subject to purification processes before entering the groundwater. It is generally recognized that at least 0.9 m (3.0 ft) of unsaturated soil is required to allow for the adequate removal of disease-causing bacteria, viruses, suspended solids, and some organic materials from sewage effluent (WDILHR, 1992). Only recently has consideration been given to the level of chemical treatment which can be expected from private sewage systems and their potential for groundwater pollution.

Private sewage systems do not effectively remove nitrogen from wastewater. Nitrate, a breakdown of organic nitrogen compounds, is very soluble and does not adsorb to soil, it often reaches groundwater from what are considered well functioning septic systems. When used on sandy soils, many properly functioning septic systems have been shown to result in significant nitrate concentrations in groundwater (Shaw et al., 1993, Shaw and Turyk, 1992).

1.2 Justification

This project was initiated due to increasing concern regarding the impact that private on-site waste disposal systems have on groundwater quality. This concern has led to a number of ordinances requiring lot sizes of two or more acres to allow for adequate dilution of septic system waste. Unfortunately, increased lot size results in additional urban sprawl. Interest in alternative or improved waste disposal systems has increased with a number of innovative systems being used in many areas.

The major quantifiable contaminant that has been shown to impact groundwater from private sewage systems is nitrate-N. This project was designed to evaluate and optimize a denitrification system's ability to reduce nitrogen loading to groundwater from on-site sewage disposal systems. This was done over the course of four years in two M.S. graduate studies directed by Dr. Byron Shaw at UW-Stevens Point. The first study involving the design, installation, and monitoring of two systems was conducted by Steve Osesek from September 1991 through August of 1993. The second study involving the optimization of these systems for nitrogen removal was done by this author from September 1993 through June 1995.

1.3 Objectives

Primary objectives of these projects were as follows:

1) Design and install two low-cost denitrification systems.
2) Evaluate wastewater treatment by the systems.
3) Determine the optimal nitrogen removal potential of these systems.
4) Determine any seasonal variation in nitrogen removal.
5) Determine the extent of improvement that occurs in the groundwater from the use of these systems.
6) Determine the removal capabilities of these systems for volatile organic chemicals (VOCs).

2. LITERATURE REVIEW

2.1 On-Site Waste Disposal Systems

Most on-site waste disposal systems consist of a septic tank followed by a subsurface soil absorption system. Results vary relative to the treatment efficiency of wastewater by septic tanks. Lawrence (1973) reported suspended solid removals of 35 to 45%, and biochemical oxygen demand (BOD_5) removals of 15% or less. However, Viraraghaven (1976) reported total suspended solids (TSS) removal of 25% with BOD_5 and chemical oxygen demand (COD) removals on the order of 50%. Typical effluent concentrations from septic tanks for suspended solids, BOD_5, COD, total nitrogen (TN) and total phosphorous (TP) are 75, 140, 300, 40, and 15 mg/l, respectively (Canter and Knox, 1986).

The quantities of indicator bacteria such as fecal coliform, whose presence suggests that other enteric organisms are also possibly present, are usually high in septic tank effluent, with pathogenic bacteria such as Pseudomonas aeruginosa commonly being isolated. When infections have occurred, viruses are also found in septic tank effluent in high concentrations (Canter and Knox, 1986). Because of the limited wastewater treatment provided by septic tanks, their effluent must be purified further prior to release to either surface or groundwater. The primary mechanism for providing this treatment is through on-site soil absorption systems.

Soil absorption systems are essential components of septic systems. Soil absorption systems (which may be trenches, beds, pits or mounds), receive septic tank effluent and discharge it below ground, where it is absorbed and treated by the soil as it percolates to groundwater.

Soil absorption systems are capable of treating organic materials, some inorganic substances, and pathogens present in the wastewater through physical, chemical, and biological processes. By acting as a filter, exchanger, adsorber, and by providing a surface on which many chemical and biological processes may occur, soil adsorption systems are capable of enhancing treatment of wastewater from septic tanks (U.S. EPA, 1980).

Wastewater microbes can be effectively removed through 1.2 m of soil if the soil is unsaturated. Unsaturated conditions enhance the removal of pathogenic organisms and other pollutants from the wastewater by increasing their chances to react with soil particles. Furthermore, under unsaturated conditions the larger pore spaces generally exchange gases with the above atmosphere. This allows for efficient aerobic decomposition of many suspended and dissolved organic substances present in the wastewater. These processes tend to work much better under unsaturated conditions because the wastewater movement is primarily through the smaller pores of the soil, which increases both the retention and liquid-solid contact time. USEPA (1980) reports that under unsaturated flow conditions, bacteria can be removed within 0.9 to 1.2 m of effluent flow through the soil. In contrast when saturated conditions exist, the water flows through the larger pores and receives minimal treatment. Romero (1970) cited a number of studies in which the effluent intersected or was close to the water table. Elevated bacteria levels were temporarily detected up to 24.4 m horizontally away from the source.

Present site criteria that must be met for the approval of a soil absorption system include a specified percolation test, and a minimum of 1.2 m separation between the bottom of the seepage system and the maximum seasonal elevation of groundwater (Canter and Knox, 1984). In Wisconsin, soil profile descriptions are used to determine this separation distance. This is required so the unsaturated soil has the ability to remove TSS, BOD_5, COD, and soluble organic carbon; a 75-95% reduction in these concentrations typically occurs within the first 1.5 m of soil.

Unfortunately, the unsaturated flow of septic tank effluent increases the chance of groundwater contamination by nitrate and other soluble, weakly absorbed chemicals. The principal sources of nitrogen in wastewater are feces and urine, which contain urea, uric acid, ammonia, undigested proteinaceous materials and bacterial cells. Typically, 75% of the nitrogen in septic tank effluent exists in the ammonium (NH_4^+-N) form and 25% exists in the organic (organic-N) form (Canter and Knox, 1986). Most of the ammonium is biologically converted to nitrate as the wastewater moves through the unsaturated soil beneath the crust of the soil absorption system. Walker et al. (1973) studied five subsurface seepage beds in which the subcrust contained 19.6% oxygen. They concluded that nitrification of NH_4^+-N to nitrate was essentially complete and commenced in the unsaturated subcrust soil within about 0.02 m of the crust. The NH_4^+-N levels were relatively high beneath the seepage beds but decreased to low levels within a few centimeters. The general increase in nitrate with depth concurrent with the decrease in NH_4^+-N suggests that nitrification was the major mechanism of NH_4^+-N removal.

The high solubility of the nitrate anion allows it to move freely with percolating water in the unsaturated zone and within the groundwater. If the nitrates enter an anaerobic environment in which organic material is available, denitrification, the reduction of nitrate to nitrogen gas, may occur. However, significant denitrification is unlikely to occur in a well-aerated subsoil or in a carbon-deficient groundwater (Walker et al., 1973).

Nitrate leaching from on-site sewage disposal systems has been shown to threaten both surface and groundwater quality in unsewered areas of the United States (Lamb et al., 1989). Nitrates may contribute to the eutrophication of surface waters and they have also been linked to cases of methemoglobinemia in infants (U.S. EPA, 1975). Consequently, the U.S. Environmental Protection Agency and World Health Organization drinking water standard for nitrate is 10 mg/l as nitrate and 45 mg/l as nitrate (Kaplan, 1987). While this standard is based on human health, a 0.3 mg/l concentration of nitrate in surface water is considered sufficient to support algae blooms.

Nitrate contamination of groundwater from septic tanks has been documented by many authors. For example, Walker et al. (1973) found that soil disposal systems of septic tank effluent in sands added significant quantities of nitrate to the underlying groundwater. Concentrations as high as 40 mg/l of nitrate-N ($NO_3^- + NO_2^-$-N) were found in the upper 0.3 m of aquifer adjacent to the systems. Relatively large down-gradient areas of 0.2 ha, or 0.5 acre were needed before concentrations attenuated to less than the 10 mg/l standard.

In studying 15 alternative septic systems, Shaw and Turyk (1992) found average nitrate concentrations of 33 mg/l occurring in the upper 2.4 m of the aquifer down-gradient of the systems in the central Wisconsin's sand plain region. Nitrate-N concentrations as high as 108 mg/l were found in the upper 0.5 m of the aquifer. Furthermore, Shaw et al. (1993) found maximum nitrate-N concentrations of 70 mg/l in the upper 0.3 m of the aquifer 5 m down-gradient of a drainfield. Average nitrate-N concentrations of 25 mg/l were found in a monitoring well located 38 meters down-gradient of the same drainfield.

In conditions of high groundwater or soils with low permeability, anaerobic soil conditions may exist. Under these conditions, nitrification will not occur and the nitrogen will remain in the form of NH_4^+-N. NH_4^+-N is readily adsorbed by soil materials of high clay content and hence migrates much more slowly (U.S. EPA, 1978). As adsorption sites for NH_4^+-N are exhausted on the soil particles, the NH_4^+-N will migrate farther and farther from the septic system. Most of the NH_4^+-N is later subjected to nitrification and leaching if aerobic conditions become reestablished (Lance, 1972).

Shaw et al. (1993) used the BURBS model, which was developed at Cornell University by Hughes et al. (1985), to simulate a subdivision's impact on nitrate-N concentrations in groundwater. They concluded that densities of less than 1.1 dwellings per ha would be needed to maintain nitrate-N concentrations below the 10 mg/l standard. This density was determined for a subdivision located in the sandy soil area of Central Wisconsin utilizing an average groundwater recharge rate of 24.6 cm per year and an average household population of three people. For heavily textured soils, utilizing a groundwater recharge rate of 10.2 cm per year, the lot size needed to achieve nitrate-N concentrations of 10 mg/l was 2.0 ha per dwelling.

Because denitrification is unlikely to occur beneath soil absorption systems and the adsorption of NH_4^+-N to soil particles is limited, dilution by groundwater has been the primary mechanism for reducing the concentration of nitrate. Dilution should not be the principle mechanism for nitrate reduction from wastewater treatment systems because groundwater flow patterns are difficult to predict and many residential areas have high densities of homes. Wastewater treatment systems that incorporate denitrification can provide a feasible means to reduce the nitrogen concentration of the effluent (Walker et al., 1973).

2.2 Nitrification and Denitrification Processes

Nitrogen entering a conventional septic system is in the organic-N and NH_4^+-N forms. A properly functioning septic tank will remove approximately 10% of the influent organic-N which is stored in the sludge (Laak et al., 1981). In the septic tank, settlement and ammonification occur, resulting in effluent containing primarily NH_4^+-N (USEPA, 1980; Canter and Knox, 1985). One of the most effective means of NH_4^+-N removal is through biological nitrification and denitrification. NH_4^+-N is converted to nitrate (nitrification, Eq 3) and the nitrate is then converted to nitrogen gas (denitrification, Eq. 6) which is released to the atmosphere.

The nitrification and denitrification processes require a variety of bacteria and environmental conditions. For these processes to be successful, an understanding of the conditions necessary for each process is essential.

Nitrification is commonly defined as *the biological oxidation of NH_4^+-N to* *nitrate with nitrite as an intermediate*. Autotrophic microorganisms are largely, if not entirely, responsible for nitrification in natural systems. These nitrifying autotrophs require oxygen and derive the carbon for cell synthesis largely from CO_2, carbonates, or bicarbonates (Delwiche,1981).

Oxidation of NH_4^+-N to nitrite by Nitrosomonas and the subsequent oxidation of nitrite to nitrate by Nitrobacter is usually represented by the following equations:

$$NH_4^+ + 1.5\ O_2 + 2\ HCO_3^- \dashrightarrow NO_2^- + 2\ H_2CO_3 + H_2O \qquad (1)$$

$$NO_2^- + 0.5\ O_2 \dashrightarrow NO_3^- \qquad (2)$$

Nitrosomonas obtain energy from the oxidation of NH_4^+-N to nitrite while Nitrobacter obtain energy from the further oxidation of nitrite to nitrate.

The overall oxidation of ammonium to nitrate is shown in Equation 3 (EPA , 1975).

$$NH_4^+ + 2\ O_2 + 2\ HCO_3^- \dashrightarrow NO_3^- + 2\ H_2CO_3 + H_2O \qquad (3)$$

Equation 3 shows that alkalinity is destroyed by the oxidation of ammonia and that carbon dioxide (H_2CO_3 in the aqueous phase) is produced. Past studies have shown that 6.3 to 7.4 mg of alkalinity are destroyed for every mg of NH_4^+-N oxidized in attached growth systems (EPA, 1975). Thus, the process of nitrification tends to lower the pH. The significance of this pH depression is that nitrification rates can be rapidly depressed. Almost all nitrifying bacteria have an optimum pH in the alkaline range, usually near 8.0, and grow slowly at pH values much below neutral (Gaudy & Gaudy, 1980). Lamb et al. (1990) reported that low alkalinity levels in the septic tank effluent appeared to limit the nitrification process in a sand filter during warm weather. If sufficient alkalinity is not available, the pH of the system can drop below 5.5, at which point nitrification could be inhibited (Loudon et al., 1989). Thus, it is recommended that the level of alkalinity as $CaCO_3$ be maintained above 40 mg/l (Sandy, 1987).

Equation 3 also shows that dissolved oxygen (DO) is required for the nitrification process. An oxygen requirement of 4.6 mg of O_2 for every mg of NH_4^+-N oxidized has been theorized to be sufficient for aeration requirements (EPA, 1975). Several investigations have provided indirect evidence of the importance of the effect of DO on nitrification rates (EPA, 1975). Low DO levels can inhibit nitrification; therefore, it is recommended that DO levels should be maintained above 2 mg/l for nitrification systems (Grady and Lim, 1980).

Temperature also plays an important role in the nitrification process. The optimum temperature range for nitrification has been reported as 18 to 35°C with nitrification ceasing at 5°C and below (Shammas, 1986). Lamb et al. (1989) reported nitrification rates decreased to 25% at temperatures lower than 10°C and a number of other studies also suggests similar reductions below 15°C.

The biological process of denitrification involves the conversion of nitrate-N to a gaseous nitrogen species. The gaseous product is primarily nitrogen gas but also may be nitrous oxide or nitric oxide. Denitrification can be accomplished by a relatively broad range of facultative heterotrophic bacteria including Psuedomonas, Micrococcus, Archromobacter and Bacillus (EPA, 1975).

Because denitrifying bacteria are facultative anaerobes, a sufficiently high concentration of DO can prevent the use of NO_3^- as the terminal electron acceptor. In general, cells exposed to more than 0.1 to 0.2 mg/l of O_2 do not denitrify

(Rittman and Langeland, 1985).

Denitrification is also a two-step process in which the first step is a conversion of nitrate to nitrite. The second step converts the nitrite to nitrogen gas:

$$NO_3^- + 0.33\ CH_3OH \dashrightarrow NO_2^- + 0.33\ H_2O + 0.33\ H_2CO_3 \qquad (4)$$

$$NO_2^- + 0.5\ CH_3OH + 0.5\ H_2CO_3 \dashrightarrow 0.5\ N_2 + HCO_3^- + H_2O \qquad (5)$$

The overall denitrification reaction is shown in Equation 6 (EPA, 1975).

$$NO_3^- + 0.833\ CH_3OH + 0.167\ H_2CO_3$$

$$\dashrightarrow 0.5\ N_2 + 1.33\ H_2O + HCO_3^- \qquad (6)$$

Equation 6 shows that bicarbonate is produced and the carbonic acid concentration is reduced whenever nitrate or nitrite is denitrified to nitrogen gas. Experiments have shown that approximately 3.0 mg alkalinity as $CaCO_3$ are produced for every mg of nitrogen reduced. Thus, the tendency of denitrification is to at least partially reverse the effects of nitrification and raise the pH of the wastewater.

CH_3OH (methanol) is shown in the above equation to indicate that an adequate carbon source is needed for the denitrifying bacteria as a source of energy and carbon. In general, 3 mg of methanol for every mg of nitrate will enable "complete" denitrification (95% removal of nitrate) (EPA, 1975).

Denitrification is also influenced by temperature. Bremner and Shaw (1958) reported that denitrification rates increased with temperature between 2 and 25°C. Crites et al. (1981) reported the minimum temperature for denitrification in land treatment systems is between 2 and 5°C.

Denitrification rates are also affected by pH. Denitrification rates are depressed below pH 6.0 and above pH 8.0. The highest rates of denitrification occur within the range of pH 7.0 to 7.5 (EPA, 1975).

2.3 Nitrogen Removal Systems for On-Site Waste Disposal

Intermittent sand filters have been shown to produce effluents of very high quality and are presently used throughout the United States. They are well suited to on-site wastewater treatment and disposal because the process is highly efficient, yet requires a minimum of operation and maintenance (Ronayne et al., 1982).

The quality of effluent from intermittent sand filters was documented in Oregon by the Department of Environmental Quality (Ronayne et al. 1982). BOD_5, and TSS were consistently less than 5 mg/l, ammonia less than 1 mg/l, nitrate-N between 20 and 40 mg/l and fecal coliform bacterial averaged a little more than 400 organisms/100 ml.

Sand filtration of septic tank effluent was also studied by Sauer and Boyle (1977). They found that while the system was efficient for nitrification of the septic tank effluent, there was no change in nitrogen concentration. Only after the filters remained continuously ponded for over three weeks did ammonia appear in the effluent. The BOD_5 concentrations for the sand filter effluent were less than 10 mg/l. The same conclusion was reached by Kristiansen (1981a, 1981b), who reported on the operation of sand filter trenches. Due to aerobic conditions and lack of an available energy source, denitrification was not found to occur.

Sikoro and Keeney (1974) stated that in a septic tank adsorption field, the energy source is the most difficult problem in promoting denitrification. Because sand filters accomplish excellent BOD_5 and TSS removal, the addition of a suitable energy source is needed for denitrification. A nitrogen reducing on-site wastewater disposal system for individual homes may be practical if the organic matter in wastewater could be used as the carbon source for denitrification. Laak et al. (1981) and Laak (1981) reported on a different modification of a conventional septic tank system. Laak developed the RUCK system in which the organic matter in greywater (kitchen and laundry wastewater) is used as the carbon source for denitrification of nitrified blackwater (bathroom wastewater). They concluded that organic carbon in the greywater was as efficient as methanol in supporting denitrification and that an overall nitrogen removal level of 70% could be achieved using the passive RUCK system.

Warnock and Biswas (1981) used effluent from a kitchen garbage grinder as an energy source for denitrification in columns. A C:N ratio of 4:1 was found to be optimal to produce satisfactory denitrification. However, the use of garbage grinders with on-site systems causes increased solids accumulation and requires more frequent septic tank pumping (Converse, 1992).

One of the most recent on-site wastewater disposal systems with nitrogen reducing potential is a recirculating sand filter (RSF) that utilizes the organic matter in septic tank effluent as the carbon source for denitrification.

The recirculating sand filter is a simple, compact method of providing improved treatment of wastewater with a low level of maintenance. Recirculating sand filters provide secondary treatment beyond a septic tank prior to surface or subsurface disposal (Loudon et al., 1985).

A typical recirculating sand filter consists of a septic tank, a free access sand filter, and a recirculation tank as shown in Figure 2.1. The recirculation tank is typically 1/4 to 1/2 the size of the septic tank (or a volume equivalent to at least one day's volume of raw wastewater flow) and receives both the effluent from the septic tank and a portion of the sand filter effluent. When the recirculation tank is full, the sand filter effluent by-passes the tank and is discharged (USEPA, 1980). Since nitrified sand filter effluent mixes with septic tank effluent in the recirculation tank, it is possible that denitrification of the nitrified sand filter effluent can take place through utilization of the carbon source provided by the septic tank effluent.

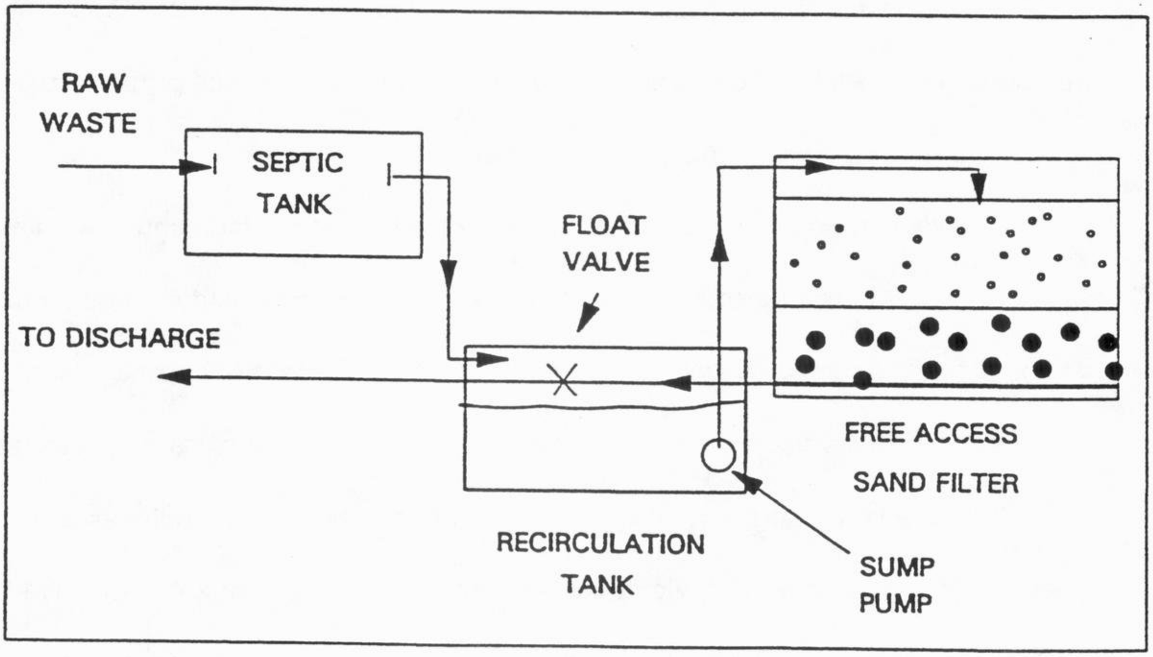


Figure 2.1 Typical recirculating intermittent filter system (EPA, 1980).

Recirculating sand filters were not originally designed for nitrogen removal and thus little data on the effectiveness of the nitrogen removal capabilities of recirculating sand filters are available from the literature. Two recirculating sand filters which were installed in Michigan generally showed nitrogen removal rates of 40 to 60% (Loudon et al., 1989).

Another study of a recirculating sand filter was conducted in which the recirculation tank was replaced with a tank filled with rock. The carbon source was added directly to the rock tank. When septic tank effluent was used as a carbon source, an average of only 25% denitrification was observed in the rock tank with the whole system achieving a total nitrogen removal of 36%. The low amount of denitrification was assumed to be the result of the low C:N ratio found in the rock tank. A C:N ratio of 0.7:1 was maintained, while past studies generally suggest an optimal C:N ratio ranging between 1:1 and 3:1 (Lamb et al., 1990). Thus, the amount and availability of carbon in the septic tank effluent was probably the factor limiting denitrification (Lamb et al., 1990).

Swanson and Dix (1986) modified the traditional recirculating sand filter, putting gravel in the bottom of the sand filter, which also served as the recirculation tank. They also used bottom ash, a waste product of coal-fired power plants, instead of sand as the filter media. The filter plus the gravel storage is referred to as a batch recirculating bottom ash filter (BRBAF). The system includes a septic tank, a BRBAF, and an ultraviolet disinfection unit. The 2.4 m x 4.5 m x 1.4 m BRBAF is filled with 80 cm of screened bottom ash overlying 15 cm of peagravel atop 40 cm of washed gravel. These layers are enclosed within treated plywood walls and posts and sealed with three layers of 6-mil plastic.

Swanson and Dix (1986) concluded that the bottom ash recirculating sand filter system produced a good quality effluent consistent with effluent from other RSF's in terms of pH, BOD_5, TSS, and TKN. However, nitrates and nitrites were not monitored, so the nitrogen removal of the system could not be determined.

Sandy (1987) modified the BRBAF system and monitored its potential for nitrogen reduction. The primary modification was a provision to recycle the highly nitrified filter effluent back to the septic tank. Since the amount of organic carbon is often a limiting factor in achieving nitrogen removal, it was felt that the "sink of carbon" in the septic tank could be tapped for this purpose.

Sandy's study was divided into eight runs starting in August, 1986 and ending in February, 1987. During the first three runs, no effluent from the filter was recycled to the septic tank. Nitrification was sought in the bottom ash filter and denitrification in the rock filter. Nitrification worked reasonably well in the bottom ash filter as could be seen by the reduced ammonia levels ranging from 3.4 to 14.9 mg/l (average 9.6 mg/l). Ammonia levels in the septic tank were 33.3 to 51.5 mg/l (average 43.6 mg/l). However, denitrification in the rock filter was incomplete, allowing effluent nitrate-N concentrations from 9.4 to 14.6 mg/l (average 11.9 mg/l).

In runs 4 to 8, a portion of the BRBAF effluent was recirculated to the septic tank. Total nitrogen removal of the system was much better during these runs. For runs 4 to 8 the average TN values for the system effluent was 7.2 to 9.6 mg/l (average 8.4 mg/l) as compared to runs 1 to 3 in which the values ranged from

15.8 to 25.7 mg/l (average 22.1 mg/l). Apparently the anoxic conditions and organic carbon in the septic tank provided suitable conditions for denitrification to occur. A modification of this design has been utilized for this research.

2.4 Volatile Organic Compounds

Volatile organic compounds (VOCs) are characterized by high vapor pressures and low solubilities, and many are listed by the USEPA as priority pollutants. VOCs are found in many household products such as degreasers, solvents, detergents, and cosmetics (Hathaway, 1980). VOCs which are commonly found in household products that can degrade groundwater quality include; toluene, benzene, 1,1,1-trichloroethane, dichlorobenzenes, nitrobenzene, napthalene, and many others (Hathaway, 1980). VOCs are not always removed by on-site wastewater treatment systems and may contaminate groundwater (Davis and Cornwell, 1985).

A laboratory study conducted by Sauer and Tyler (1991) showed that volatilization of VOCs occurred as septic tank effluent percolated through the unsaturated soil beneath a soil absorption field. The unsaturated media of a RSF may remove VOCs before discharge to the soil absorption field, thus enhancing the overall removal and reducing the VOC impacts on groundwater quality.

3. METHODS

The following sections briefly describe the methods, techniques, and procedures employed in the construction, installation, and sampling of the denitrification systems and groundwater monitoring wells.

3.1 Study Sites

This project was designed to evaluate a denitrification system's ability to reduce nitrogen loading to groundwater from on-site sewage disposal systems. Two single-family homes north of Stevens Point in Portage County, Wisconsin (Figure 3.1) were selected for the installation of experimental denitrification systems in the summer of 1992.

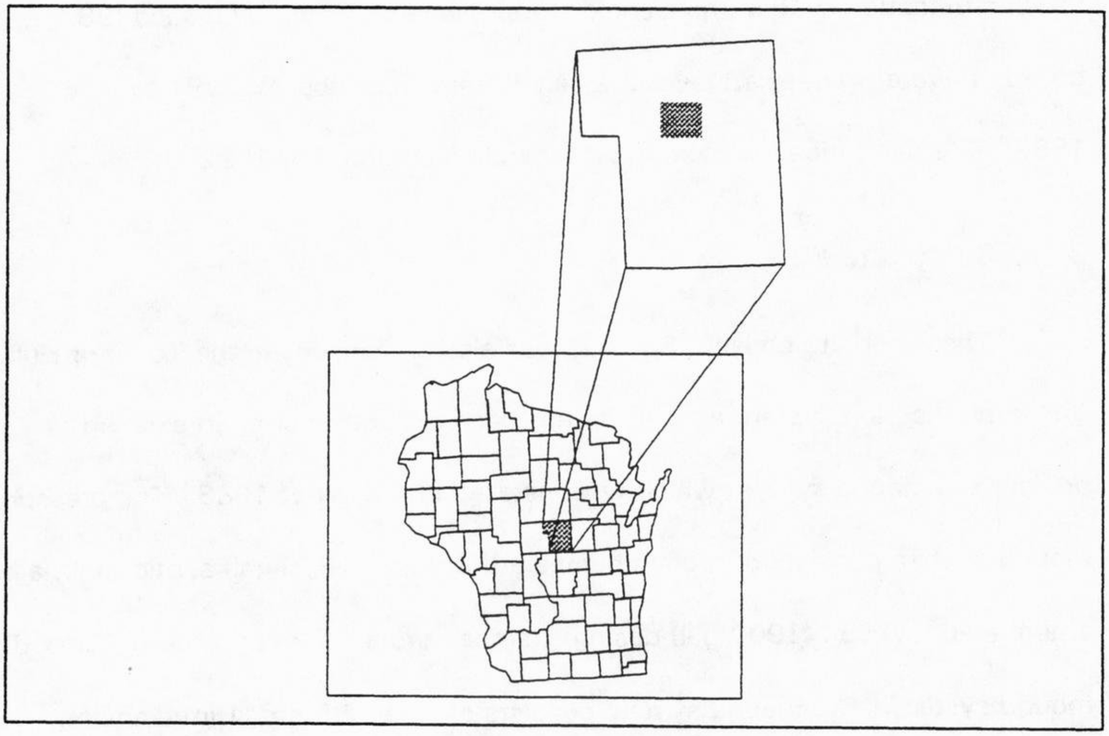


Figure 3.1 Location of study sites in The Central Sands region of Portage County, WI.

3.11 Study Site #1

One of the sites chosen, Site #1, was located in the Jordan Acres Subdivision in the Town of Hull. The denitrification system at this location has been retrofitted onto an existing conventional septic system which was installed in the summer of 1982. The conventional system consists of a one chambered, 3,785 l (1000 gal) concrete septic tank which feeds a 3.7 m by 15.9 m (12 x 52 ft) conventional drainfield. The drainfield consists of two 10.2 cm (4 in) diameter perforated PVC (polyvinyl chloride) distribution pipes 1.8 m (6 ft) apart and 0.9 m (3 ft) from the edge of the drainfield. A view of the system layout at Site #1 can be seen in Figure 3.2.

This septic system serves a three bedroom home. Before the installation of the denitrification system, household water usage averaged 371.3 lpd (98.1 gpd) based on water meter data over the period from December 9, 1991 to July 15, 1992. The denitrification system was installed on July 16, 1992.

3.12 Study Site #2

The other site chosen, Site #2, was also a residence in the Town of Hull. The denitrification system at this site has been retrofitted onto an existing pressurized mound system which was installed in the fall of 1983. The pressurized mound system consists of a one chambered, 3,785 l (1000 gal) septic tank, a one chambered, 3,785 l (1000 gal) dosing chamber and a 8.5 m by 20.4 m (28 x 67 ft) mound system. The mound system consists of three 2.5 cm (1 in) diameter perforated PVC distribution pipes 14.0 m (46 ft) long and 0.6 m (2 ft) apart. A

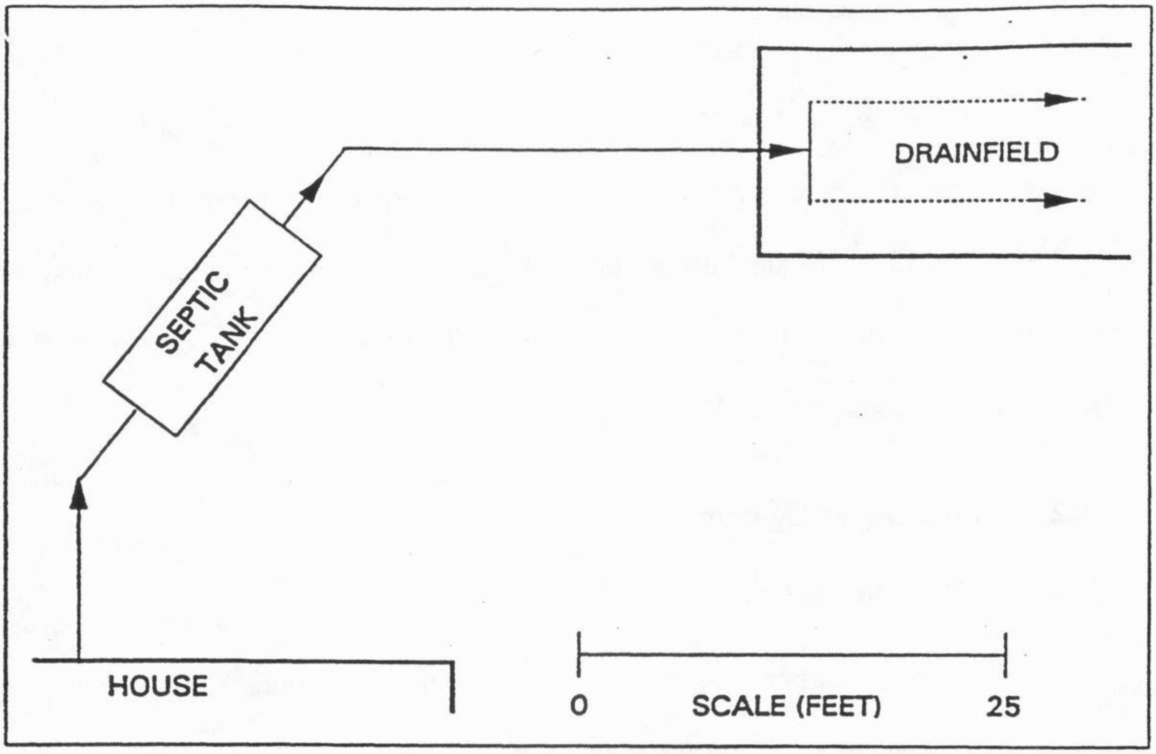


Figure 3.2 Top view of system layout at Site #1 before the installation of the denitrification system.

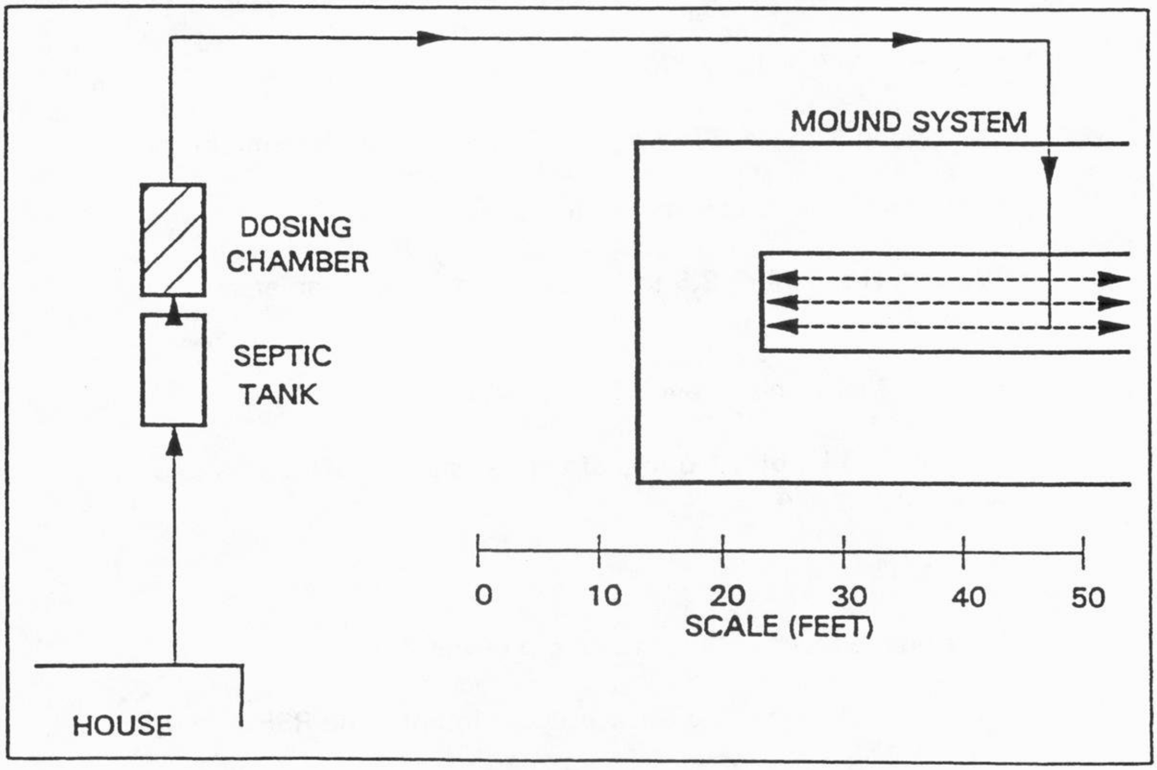


Figure 3.3 Top view of system layout at Site #2 before the installation of the denitrification system.

view of the system layout at Site #2 can be seen in Figure 3.3.

The system at Site #2 also serves a three bedroom home. Before the installation of the denitrification system, the household water usage averaged 1,081 lpd (285 gpd) based on dosing chamber pumping rates over the period from December 21, 1991 to August 6, 1992. The denitrification system at Site #2 was installed on August 13, 1992.

3.2 Denitrification System Design

3.21 Major Components

The denitrification systems retrofitted onto the two existing septic systems were quite similar. The denitrification systems involved using a recirculating sand filter (RSF) with a built in rock storage unit similar to that described by Swanson and Dix (1986). A 7,571 l (2000 gal) septic tank has been used to house the various components of the RSF system. The major components included:

1) A collection system located at the bottom of the RSF.
2) 39.4 cm (15.5 in) of 2.5 to 3.8 cm (1 to 1.5 in) diameter limestone.
3) 7.6 cm (3 in) of pea gravel.
4) 58.4 cm (23 in) of a 1.8 mm effective size sand with a uniformity coefficient of 1.4.
5) A pump chamber.
6) A distribution system located on top of the RSF.
7) A distribution system to allow effluent to enter the RSF from the septic tank.
8) A timing system

In order to allow the septic tank enclosing the RSF to be placed deeper in the ground, 2 cm (3/4 in) of treated plywood and 5.1 cm (2 in) of polystyrene foam were placed around the septic tank enclosing the RSF. This was done to allow septic tank effluent to flow by gravity into the RSF at Site #1 and to help maintain heat throughout the winter at both sites. To allow for easy access, the top of the RSF is at land surface and was covered with a three piece insulated plywood cover.

Despite the two RSF's similarities, one main difference should be noted. At Site #1, septic tank effluent flowed by gravity to the limestone storage zone at the bottom of the RSF. However, due to the depth of the existing system at Site #2, gravity flow of septic tank effluent to the RSF was impossible. Thus, a sewage ejector pit was installed in the dosing chamber and, through the use of floats, the effluent from this pit was pumped up to the top, rather than the bottom, of the RSF.

Throughout the remainder of this paper, the sewage ejector pit located at Site #2 will be referred to as Dosing Chamber #1 and the rest of the dosing chamber, which applies effluent to the mound system, will be referred to as Dosing Chamber #2. Cross sectional views of the RSFs located at Site #1 and Site #2 can be seen in Figures 3.4 and 3.5, respectively.

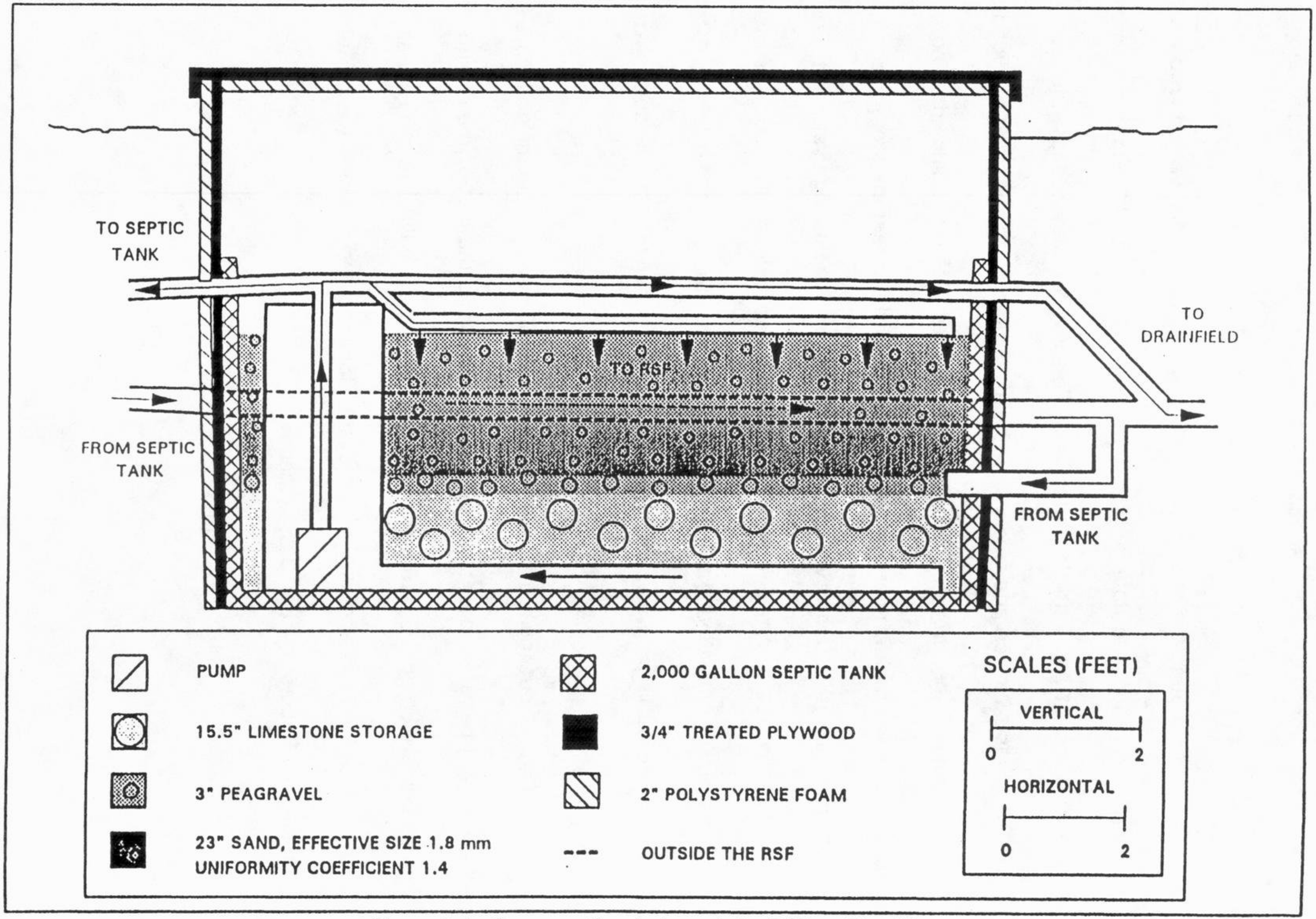


Figure 3.4 Cross sectional view of the RSF at Site #1.

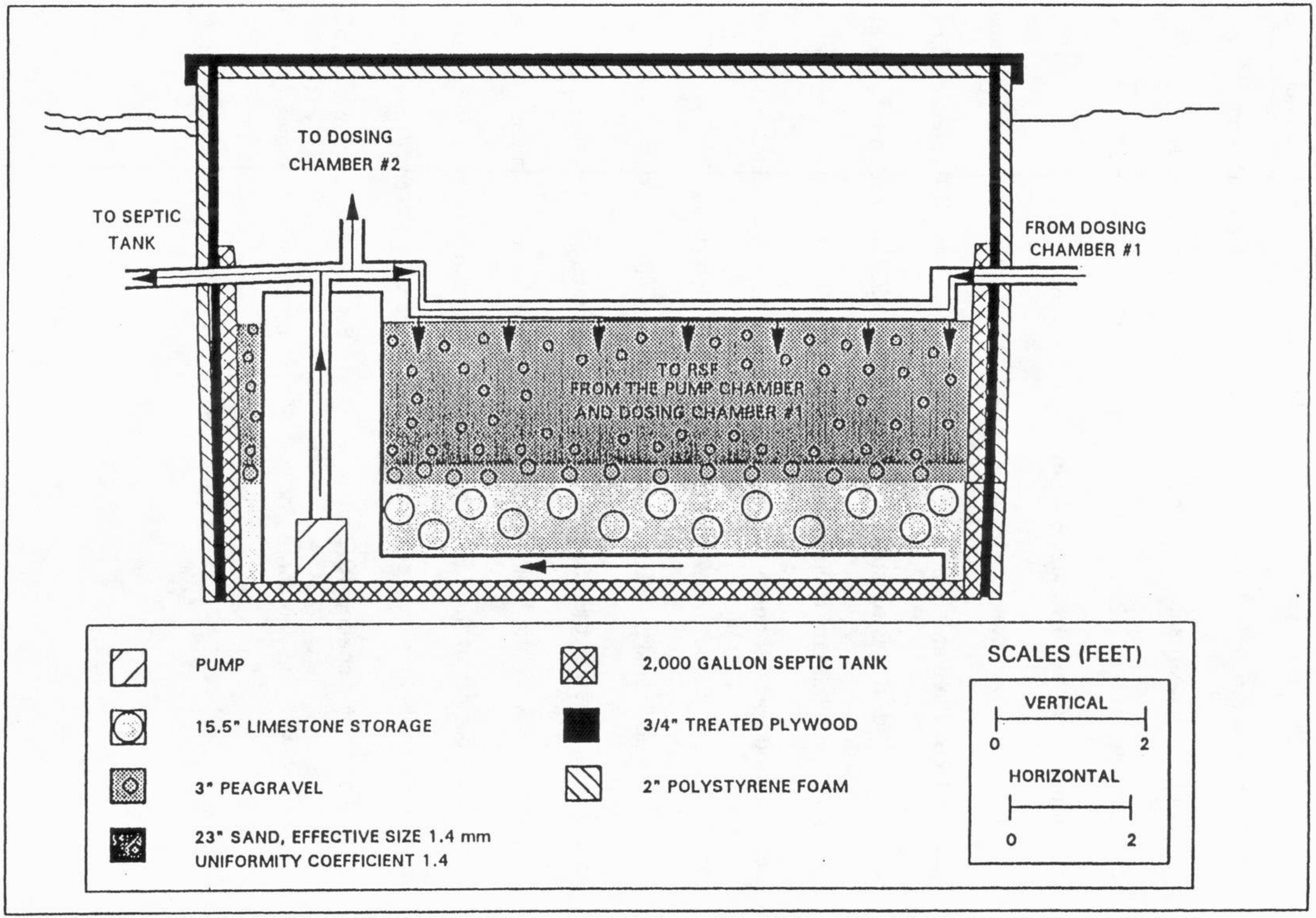


Figure 3.5 Cross sectional view of the RSF at Site #2.

The collection system, located on the bottom of the RSFs, consisted of three 10.2 cm (4 in) diameter schedule 40 PVC pipes with cut slots 0.6 cm ($^1/_4$ in) wide, 6.4 cm (2.5 in) deep and 10.2 cm (4 in) on center (Figure 3.6). The pump chamber consists of a 60.9 cm (24 in) diameter schedule 40 or Schedule 80 PVC pipe with cut slots 0.6 cm($^1/_4$ in) wide, 25.4 cm (10 in) long and 7.6 cm (3 in) on center. The cut slots were not installed where the collection system enters the pump chamber (Figure 3.7). The distribution system located on top of the RSFs consisted of three 2.5 cm (1 in) diameter schedule 40 PVC pipes each 2.7 m (9 ft) in length. Each pipe had seven 0.3 cm ($^1/_8$ in) diameter holes with holes 1, 3, 4, 5, and 7 pointed upwards and holes 2, and 6 pointed downwards (Figure 3.8). Each hole pointing upwards was covered by a 7.6 cm (3 in) Orenco orifice shield.

One additional distribution system was located in each RSF to allow septic tank effluent to enter the RSF. At Site #1, a second distribution system was installed to allow septic tank effluent to flow by gravity to the limestone storage zone at the bottom of the RSF. This distribution system consisted of a 1.5 m (5 ft) long, 10.2 cm (4 in) diameter schedule 40 PVC pipe with 1.3 cm (0.5 in) holes pointed down and spaced 10.2 cm (4 in) apart. Each end of the pipe was raised to the surface with one end containing two 45° bends to allow for cleaning (Figure 3.9).

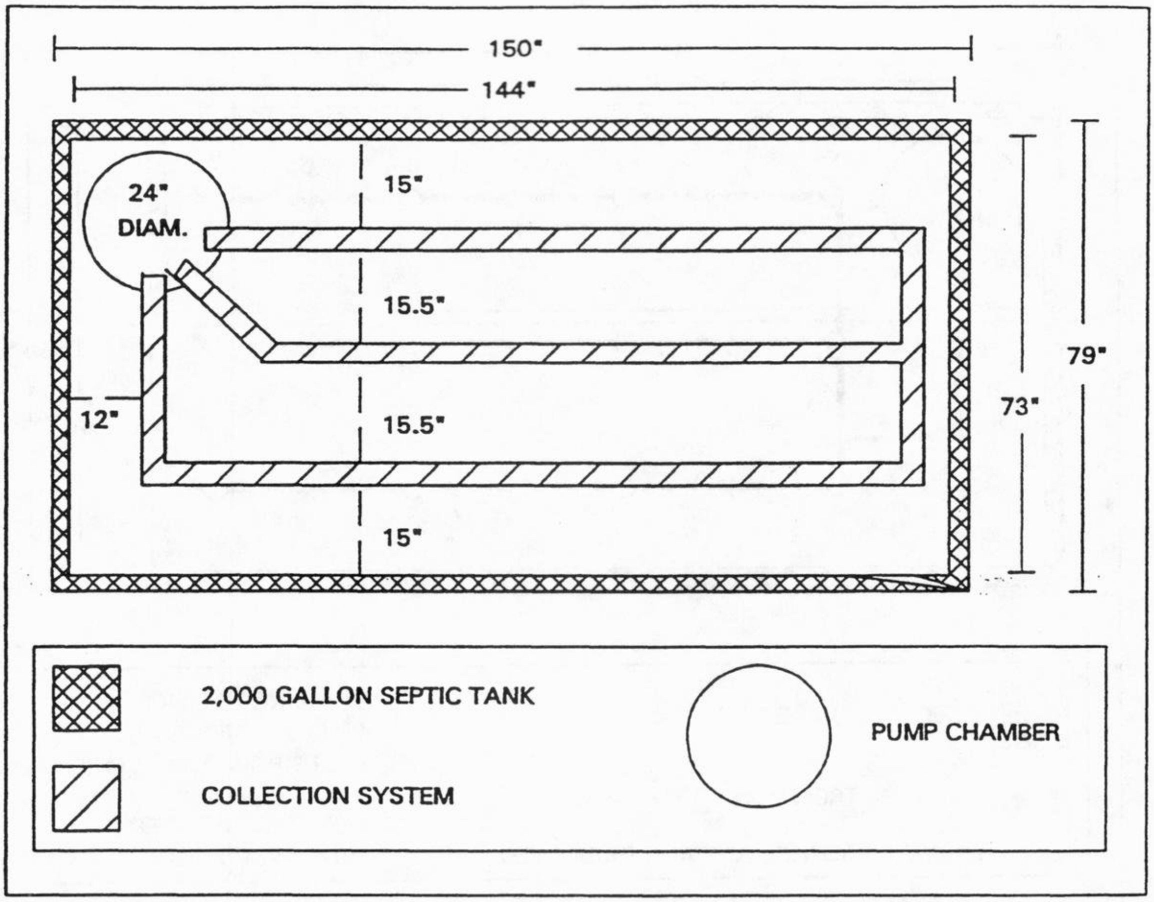


Figure 3.6 Top view of the collection system located on the bottom of the RSF.

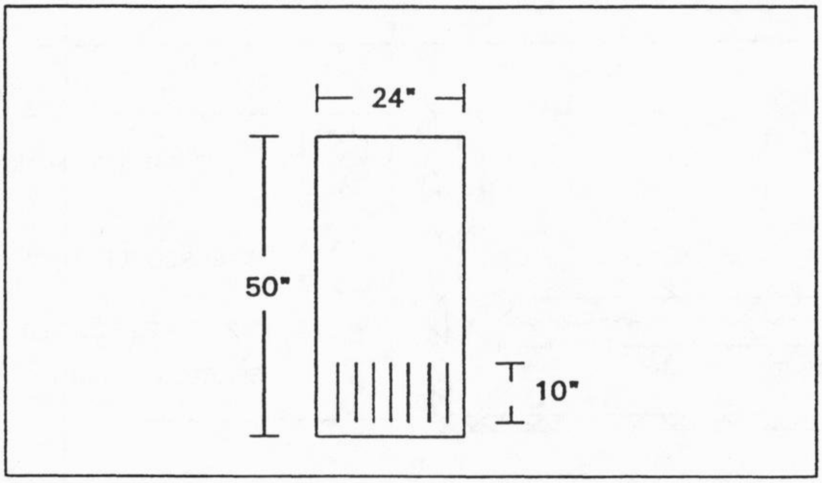


Figure 3.7 Cross section of the pump chamber in the RSF.

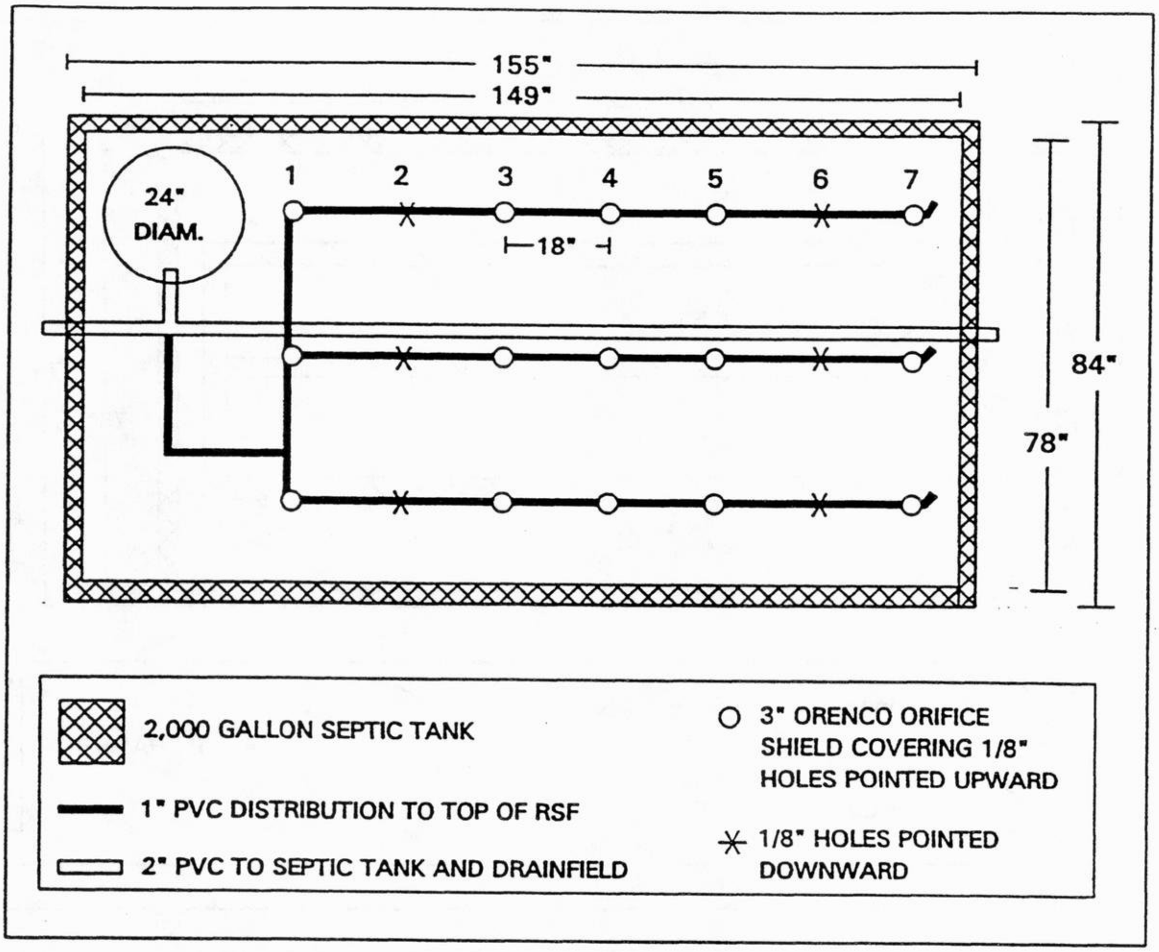


Figure 3.8 Top view of the distribution system located on the top of the RSF at Site #1.

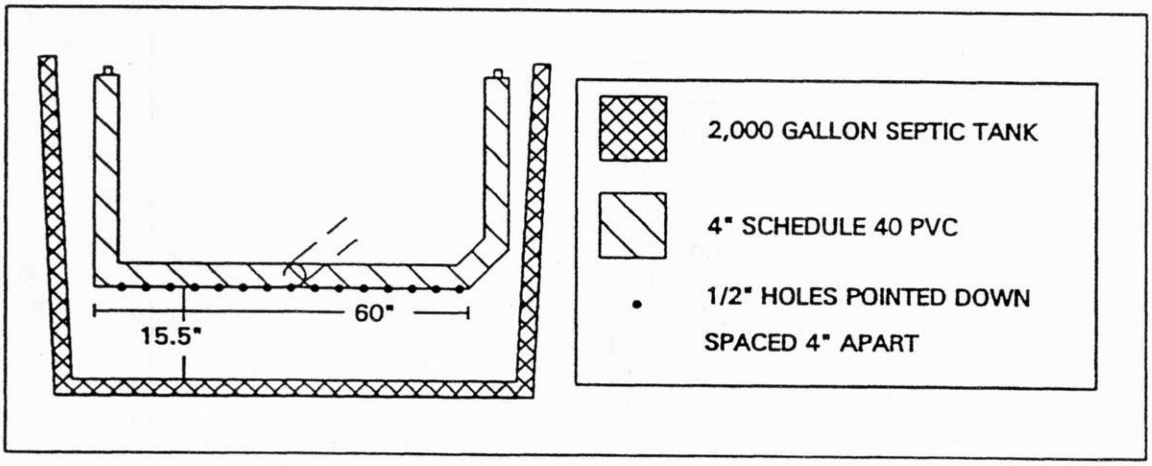


Figure 3.9 Cross section of the gravity distribution system for the septic tank effluent into the RSF at Site #1.

As noted previously, septic tank effluent at Site #2 was applied to the top of the RSF. To accomplish this, a second distribution system was located on the top of the RSF at this site. This second distribution system consisted of two 2.5 cm (1 in) diameter schedule 40 PVC pipes with each pipe having the same configuration of holes as the original distribution system (Figure 3.10).

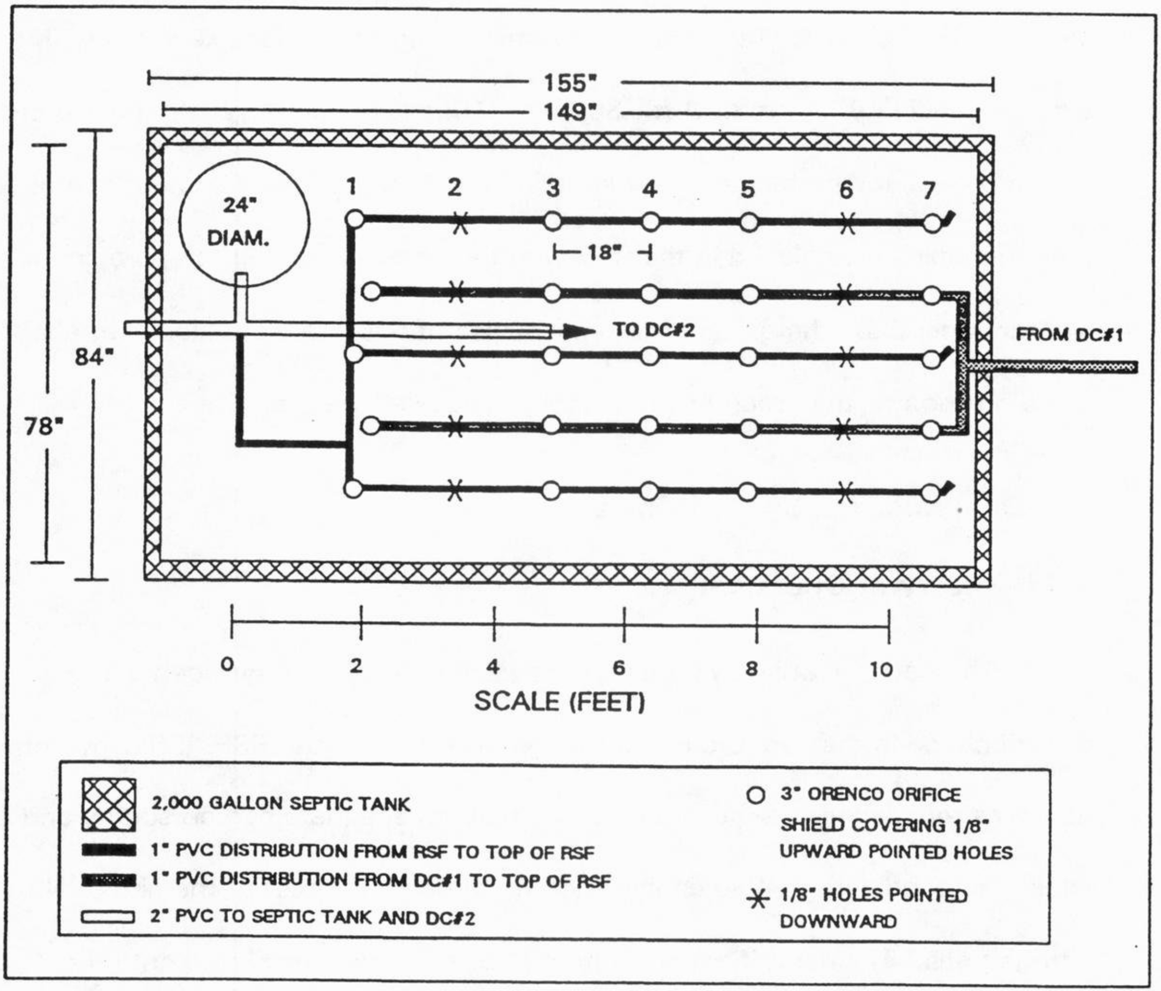


Figure 3.10 Top view of distribution systems located on the top of the RSF at Site #2.

3.22 Temperature Probes

Temperature probes were also been placed in the two RSFs. The temperature probes were made out of type T thermocouple wire which were soldered together at the bottom end and sealed with a silicone rubber adhesive sealant. At Site #1, four groups of these thermocouples were placed in the RSF as shown in Figure 3.11. Each group consisted of four thermocouple wires with the wires extending 67.1 cm (2.2 ft), 36.6 cm (1.2 ft), 6.1 cm (0.2 ft), and 0.0 cm (0.0 ft) below the surface of the sand (Figure 3.12). At Site #2, one group of four thermocouples was placed in the RSF with the wires extending to approximately the same depths. The group of thermocouples at this site were placed at about the same position as the group of thermocouples labeled west at Site #1.

3.3 Denitrification System Operation

3.31 Theory of Operation

The denitrification systems were designed to remove nitrogen via denitrification in the septic tank following nitrification in the RSF. Effluent from the RSF was recirculated to the septic tank where an adequate carbon source and anaerobic conditions should enable bacteria to denitrify most of the nitrate-N to nitrogen gas. The denitrification system at Site #1 was retrofitted onto the conventional septic system on July 16, 1992. Septic tank effluent flowed from the septic tank by gravity to the bottom of the RSF. Once there, with the help of floats and timers, it was pumped at different rates to the top of the RSF for nitrification,

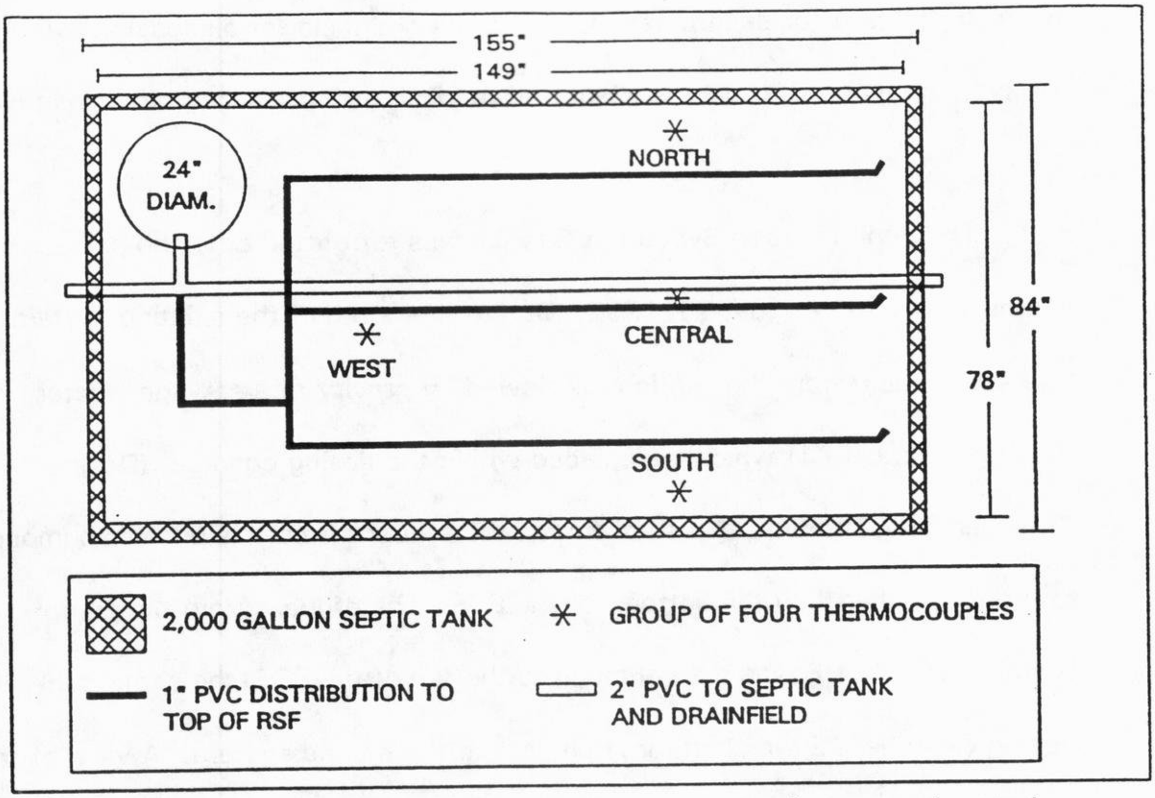


Figure 3.11 Top view of the four thermocouple groups located in the RSF at Site #1.

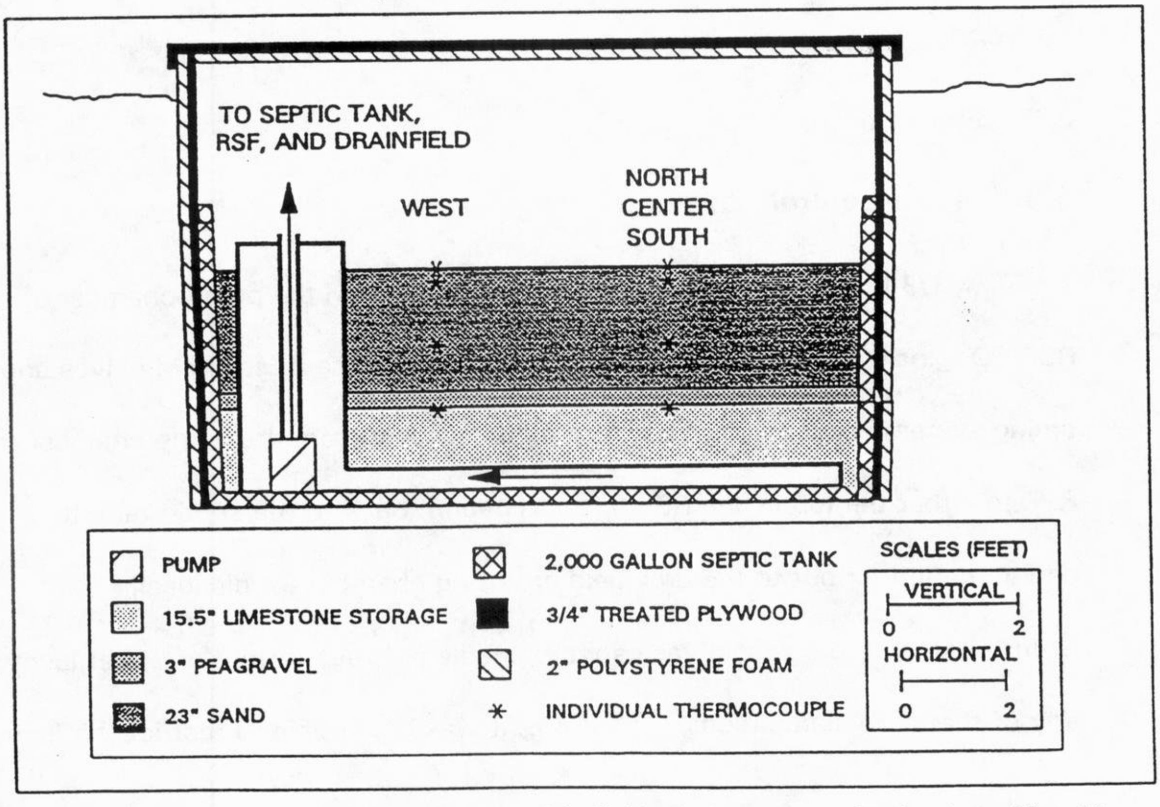


Figure 3.12 Cross sectional view of individual thermocouple depth at Site #1.

to the septic tank for denitrification, and to the drainfield for disposal. A view of the system layout after the installation of the RSF at Site #1 can be seen in Figure 3.13.

The denitrification System at Site #2 was retrofitted onto the pressurized mound system on August 13, 1992. Due to the depth of the existing system at Site #2, effluent from the septic tank flowed by gravity to a sewage ejector pit (Dosing Chamber #1) which was placed within the dosing chamber (Dosing Chamber #2). Through the use of floats, the effluent from this pit was pumped up to the top, rather than the bottom, of the RSF. The effluent from the pump chamber of the RSF was then pumped to the top of the RSF, the septic tank, and Dosing Chamber #2 which in turn pumps it to the mound system. A view of the system layout after the installation of the RSF at Site #2 can be seen in Figure 3.14.

3.32 Flow Control

3.321 Flow Control 'Valves'

A 1/3 hp Zoeller 57 sump pump was situated in the pump chamber of the RSF. Originally, it was anticipated that through the use of solenoid valves and a timing system it would be possible to pump effluent from the pump chamber of the RSF to either the top of the RSF for nitrification, back to the septic tank for denitrification, or out to the drainfield or dosing chamber for disposal. Unfortunately, the solenoid valves were unable to prevent the flow of effluent to any of the three destinations. Therefore, it was impossible to restrict the flow to

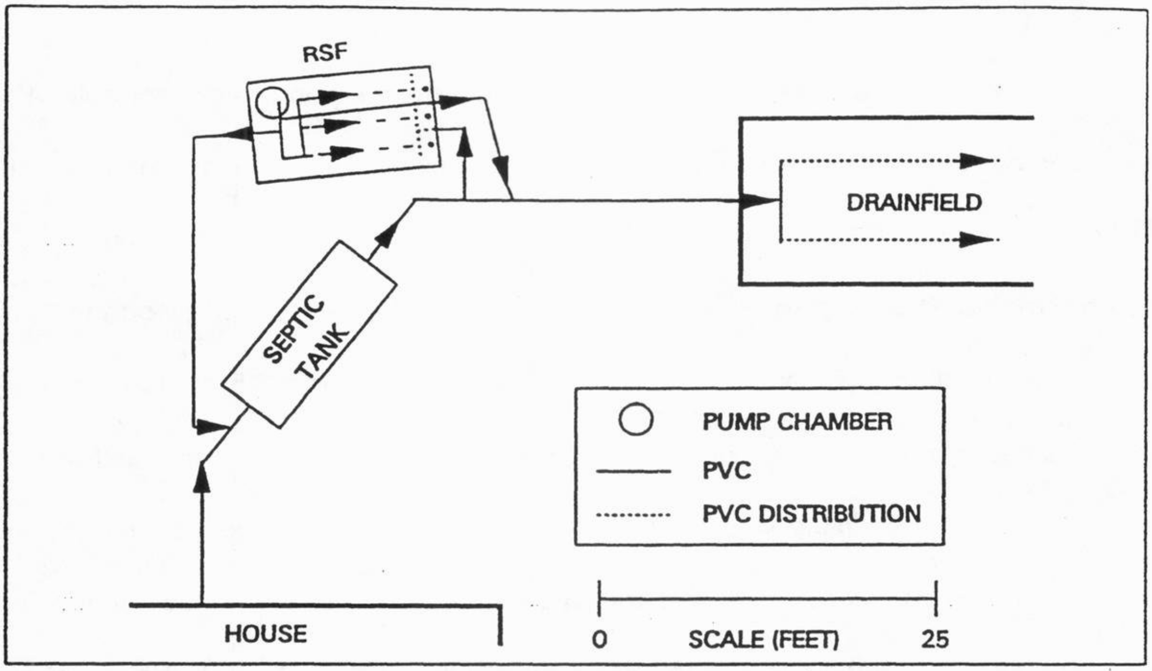


Figure 3.13 Top view of system layout at Site #1 after the installation of the RSF.

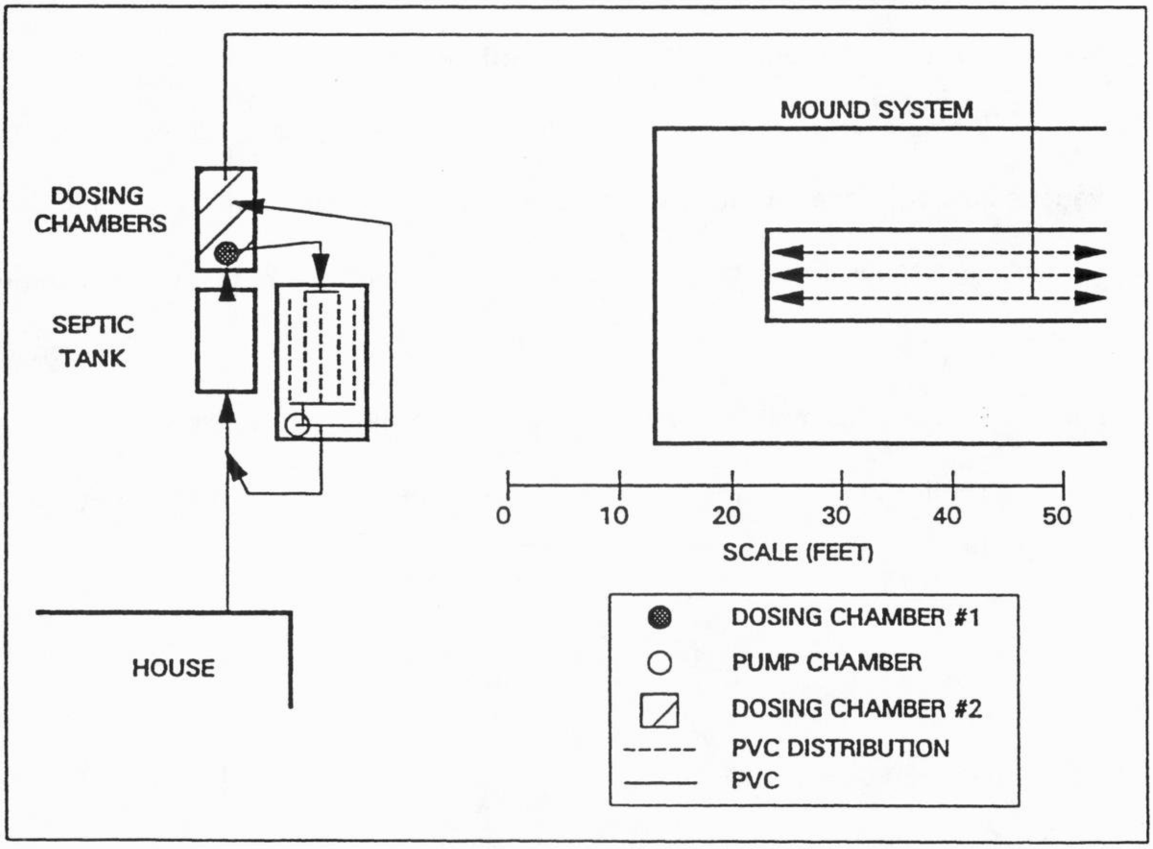


Figure 3.14 Top view of system layout at Site #2 after the installation of the RSF.

only one location at a time.

This problem was alleviated by changing the method of flow control. While gate valves were originally located along the three destination lines, the flow rates obtained through the use of these valves were not consistent enough. In the summer of 1993, a new type of flow control 'valve' was installed along the three destination lines. These flow control 'valves' consisted of a 2.5 cm (1 in) diameter PVC cap installed on the existing line with a hole drilled into the cap. Half of a 2.5 cm (1 in) PVC coupling was placed on the PVC pipe in front of and behind the PVC cap in order to maintain a constant diameter. A 2.5 cm (1 in) Fernco coupling was then placed over the entire area as shown in Figure 3.15. Through the use of a timing system, effluent from the pump chamber of the RSF was pumped to all three locations at the same time while varying the flow rate to each location.

By changing the size of the hole drilled into the 2.5 cm (1 in) PVC cap, various flow rates were obtained with at least a 10% accuracy. The original goal of pumping various amounts to each destination was accomplished by pumping to all three locations at once and varying the flow rate to each location. This concept proved to be much simpler and less expensive than the original plan yet still allowed the opportunity to change the amount pumped to each location at one time.

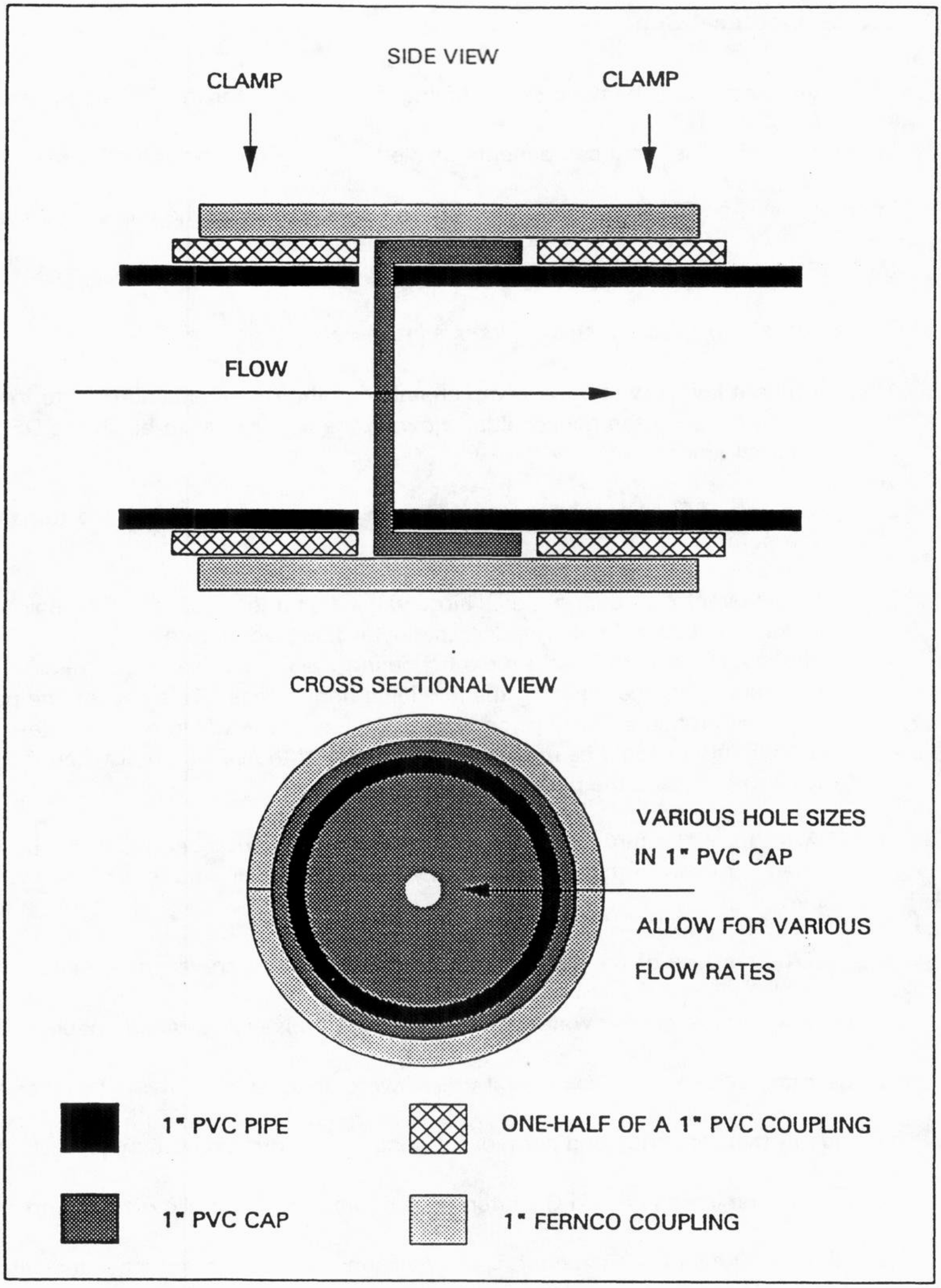


Figure 3.15 Side and cross sectional view of flow control 'valve'.

3.322 Electrical Controls

An electrical schematic of the timing system used for the RSFs can be seen in Figure 3.16. The main components of the timing system include a DELAY interval timer, an ON interval timer, and two floats located in the pump chamber of the RSF.

The theory of operation of the timers is as follows:

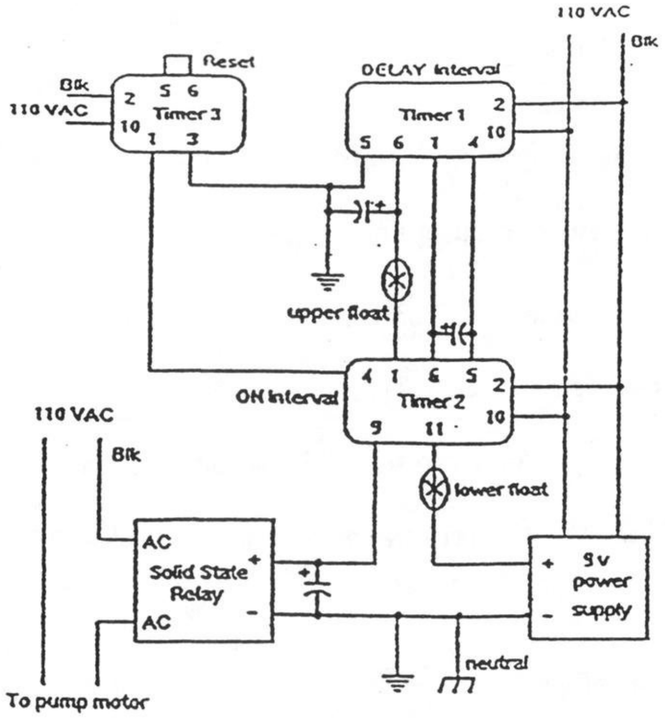
1) Effluent levels within the pump chamber of the RSF rise according to the water usage in the households, allowing the upper float to begin the DELAY interval timer.

2) After the DELAY interval is over, the ON interval timer begins and turns on the pump.

3) In the event that during the ON interval, effluent levels drop to the point where the pump would run dry, the lower float would turn the pump off. However, when the pump turns off during the ON interval water re-enters the pump chamber causing the low level float to rise. This causes the pump to surge on and off during the time remaining in the ON interval. A design modification should be made which would end the ON interval when the lower float turns the pump off.

4) After the ON interval is over, as soon as the effluent level within the pump chamber rises to the upper float, the DELAY interval timer starts the cycle over.

The purpose of the DELAY interval was merely to prevent the pump from running continuously if the water usage within the households would enable it do so. Both the DELAY and ON interval timers were adjustable, allowing changes to be made in the frequency and duration of pumping events. For example, a DELAY interval of 25 minutes and an ON interval of 5 minutes would allow the pump to run a maximum 10 minutes each hour. If the positions of the float switches indicated



COMPONENTS:

All capacitors are 3.3 ufd 16 v tantalum

Timer 1,2, and 3 are Potter & Brumfield CNT-35-76
Timer 1 is operated in mode "A"
Timers 2 and 3 are operated in mode "C"

The float contacts are closed when they are floating (fluid level is high)

The solid state relay is a Grayhill 70S2-04-C-12-S rated 3-30 VDC input 240 VAC 12 Amp load
Relay should be mounted on a heat sink.

The 9 volt power supply can be any generic plug-mounted style...It needs only to supply the few milliamps needed to turn on the solid state relay. A Radio Shack 273-1651 works fine.

THEORY OF OPERATION:

Timer 1 and 2 work together triggering each other on and off (astable multivibrator), with the time setting on each determining the duty cycle. If the fluid level is below the upper float, the open contacts on that float prevent the DELAY timer from starting. As soon as the fluid level rises to the upper float, the delay timer starts the cycle over. The lower float turns the pump off in the event the fluid level drops so low that the pump would run dry.

Concept, Design, and Installation:
Jim Tuszka, Electrician
UW-Stevens Point (1993)

Figure 3.16 Electrical schematic of the timing system.

sufficient effluent levels in the RSF, the pump could run a maximum of 48 times a day for a total duration of 240 minutes/day.

3.4 Denitrification System Monitoring

3.41 Wastewater Sampling

Wastewater samples were taken from the denitrification systems on at least a biweekly basis. At Site #1, samples were collected from the septic tank and from the pump chamber in the RSF. At Site #2, samples were collected from Dosing Chamber #1, from the pump chamber in the RSF, and from Dosing Chamber #2.

The samples were collected by lowering a polypropylene bottle into the wastewater with a string. The sample bottle was rinsed four times with the wastewater before a sample was collected into a 125 ml polypropylene bottle with a polyethylene cap. One ml of concentrated H_2SO_4 was placed within the bottles before the sample was collected to preserve the various nitrogen forms. The samples were then placed within a cooler with ice packs and transported to the Environmental Task Force Laboratory (Lab State ID No. 750040280) at the University of Wisconsin-Stevens Point for analysis and storage at 4°C. These weekly samples were analyzed for NO_3^--NO_2^--N (nitrate-N), NH_4^+-N, total Keldjhal nitrogen (TKN), and chlorides.

On a monthly basis, samples were taken from the same places by the same method and collected in 500 ml polypropylene bottles with polyethylene caps. These samples had no preservatives in them and were analyzed for pH, electrical conductance, alkalinity, total hardness, sodium, total phosphorous (TP), BOD_5, COD, and fluorescence.

On a seasonal basis, samples for VOCs analyses were taken from the same locations using the same method and collected in 40 ml glass vials with TEFLON caps. These samples had a 0.5 ml hydrochloric acid preservative and were analyzed according to EPA Method 8021. Although the samples were collected by the same method as the inorganic analytes, additional care was taken to create minimal disturbance when lowering the sample vessel into the sampling points.

3.42 Temperature Recordings

Temperature readings were taken with an Omega HH21 hand held microprocessor digital thermometer from all of the thermocouples within the RSFs on at least a biweekly basis. At Site #1, temperature readings were also taken at least biweekly from the septic tank and from the pump chamber in the RSF by lowering a thermocouple into the wastewater within them. At Site #2, additional temperature readings were taken from Dosing Chamber #1, from the pump chamber within the RSF, and from Dosing Chamber #2 using the same method.

3.43 Flow Rates

Flow rates to the various locations were measured through the use of a quick disconnect value located on each of the three destination lines. The lines were disconnected beyond the flow control 'valves' and a hose was connected to it. By measuring the amount of effluent obtained in a specific time period, a flow rate in liters/minute was obtained for each of the three destination lines.

It should be noted that a potential problem exists for measuring various flow rates through the use of this method. Effluent is under pressure when it is applied to the top of the RSF through the distribution system and is not under pressure when its flow rate is measured. Therefore, it is possible that the amount of effluent measured and the amount applied to the top of the RSF during a specific time period are slightly different. To ensure that no differences were occurring, it was necessary to install a pressure gauge within the line. Since pressure readings taken when the distribution system to the top of the RSF was both connected and disconnected revealed no difference, it was concluded that the measured and applied flow rates were the same. However, differences between the two may have been significant if the effluent applied to the top of the RSF was under enough pressure.

3.5 Monitoring Well Installation and Design

Although the author was not directly involved with the installation for most of the monitoring wells, the following is a description of the methods, techniques, and procedures employed in the construction, installation, and sampling of the groundwater monitoring wells. The information for these descriptions is based on documentation provided in Shaw et al. (1983), and Shaw and Turyk (1992). Additional wells were installed by the author at each site for VOC monitoring.

3.51 Study Site #1

At Site #1, originally one up-gradient and one down-gradient well were installed in the summer of 1988. These monitoring wells were constructed of 3.2

cm (1 $^1/_4$ in) PVC and were fitted with 91 cm (36 in) slotted, 0.02 cm (.01 in) slot size, PVC screens.

The original down-gradient well (REE DG) showed no appreciable difference in water quality from the up-gradient well (REE UG). Thus, in the summer of 1989, two nested wells (REC and REW) were also installed down-gradient of the drainfield. These two wells were installed in an east-west transect with the existing down-gradient well, 4.9 m (16 ft) away from and parallel to the down-gradient edge of the drainfield as shown in Figure 3.17. It was believed that these wells would show whether or not preferential percolation was occurring out of this system or if strong vertical flow components were transporting contamination deeper into the aquifer and below the existing monitoring well.

These two well nests consisted of three 1.9 cm ($^3/_4$ in) PVC pipes taped together with nylon reinforced tape. The threaded joint pipes were screened with 30.5 cm (1 ft), 0.02 cm (0.10 in) slotted, PVC points. The screens were positioned at 15.2 cm (6 in) intervals with the lower portion of the uppermost screen being placed at the water table, as shown in Figure 3.18.

During the summer of 1990 five more multilevel monitoring wells were installed at this location. Wells RSDS A-E were installed in a transect perpendicular to groundwater flow with well "B" being positioned 33.5 meters (110 ft) down-gradient of well REC, with 3.0 m (10 ft) of separation between each of the five wells as shown in Figure 3.17.

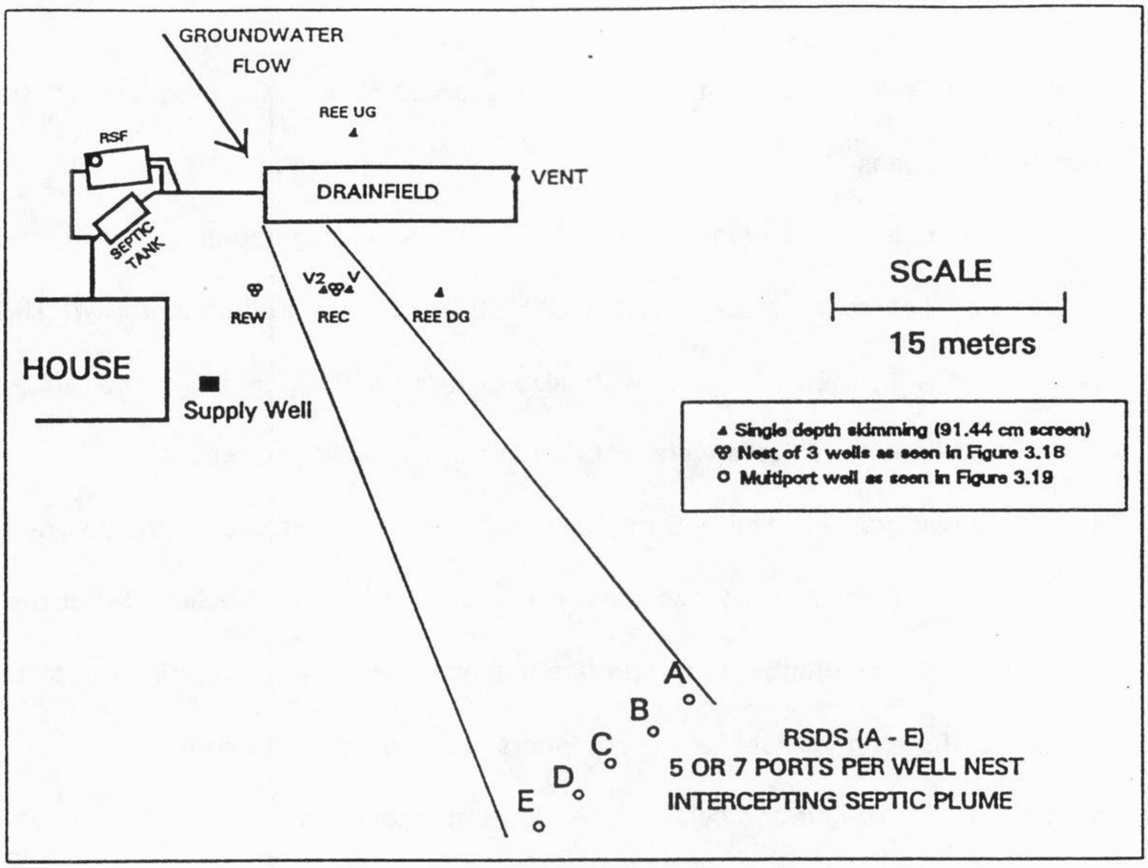


Figure 3.17 Monitoring well locations at Site #1.

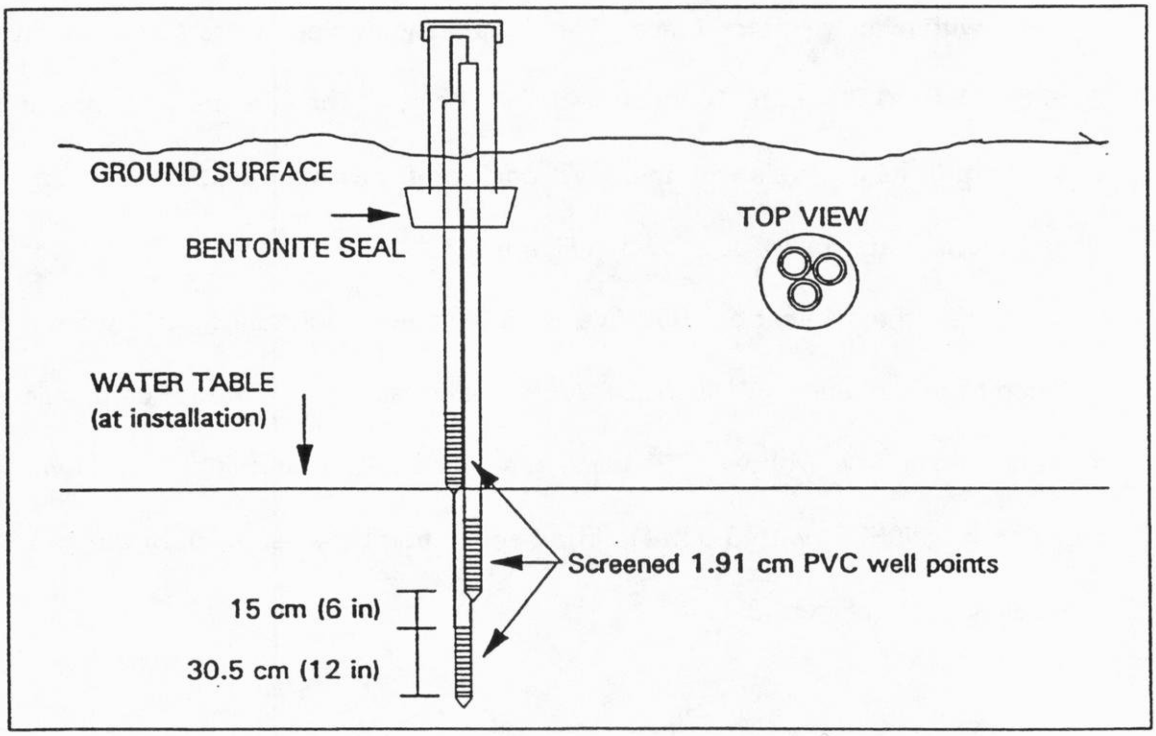


Figure 3.18 REC and REW well design located 4.6 m down-gradient of drainfield.

These wells were constructed based on a design by Bradbury and Bahr (1987). The wells consisted of a 1.9 cm ($^3/_4$ in) PVC spine which was screened over its last foot interval with a 30.5 cm (1 ft) slotted point with 0.02 cm (0.01 in) openings. Surrounding the spine are up to six, 0.6 cm inside diameter polypropylene tubes which were attached to the PVC center spine with nylon reinforced tape. The polypropylene tubes were perforated with 0.3 cm ($^1/_8$ in) holes and screened with TYPAR over the last 25 cm (10 in) section at the bottom of each tube. Each tube extends to a different depth in the aquifer to allow discrete samples to be taken from various depths as shown in Figure 3.19.

Four of the wells (A,C,D,E) have five sampling ports, including the spine, at 30.5 cm (1 ft) intervals with the upper most screened interval at or just below the watertable. Thus, the five wells were capable of sampling the upper 1.5 m (5 ft) of the aquifer at 30.5 cm (1 ft) intervals over a 12.2 meter wide transect as shown in Figures 3.17 and 3.20. Well "B" had two additional sampling ports as shown in Figure 3.20.

Because the diameters of the existing wells located in the plume were too small to accommodate a bailer, two new wells were installed to monitor the loading of VOCs to groundwater. These wells have the same construction as wells REE DG and REE UG. The first well (REC V) was installed on October 26, 1993. This well was placed to skim the water table and is located approximately 20 cm (8 in) to the east of well nest REC (Figure 3.17). Unfortunately, at the time well REC V was installed the water table was unusually high due to the above average precipitation of 1993. The water table dropped below the well screen during 1994, and a

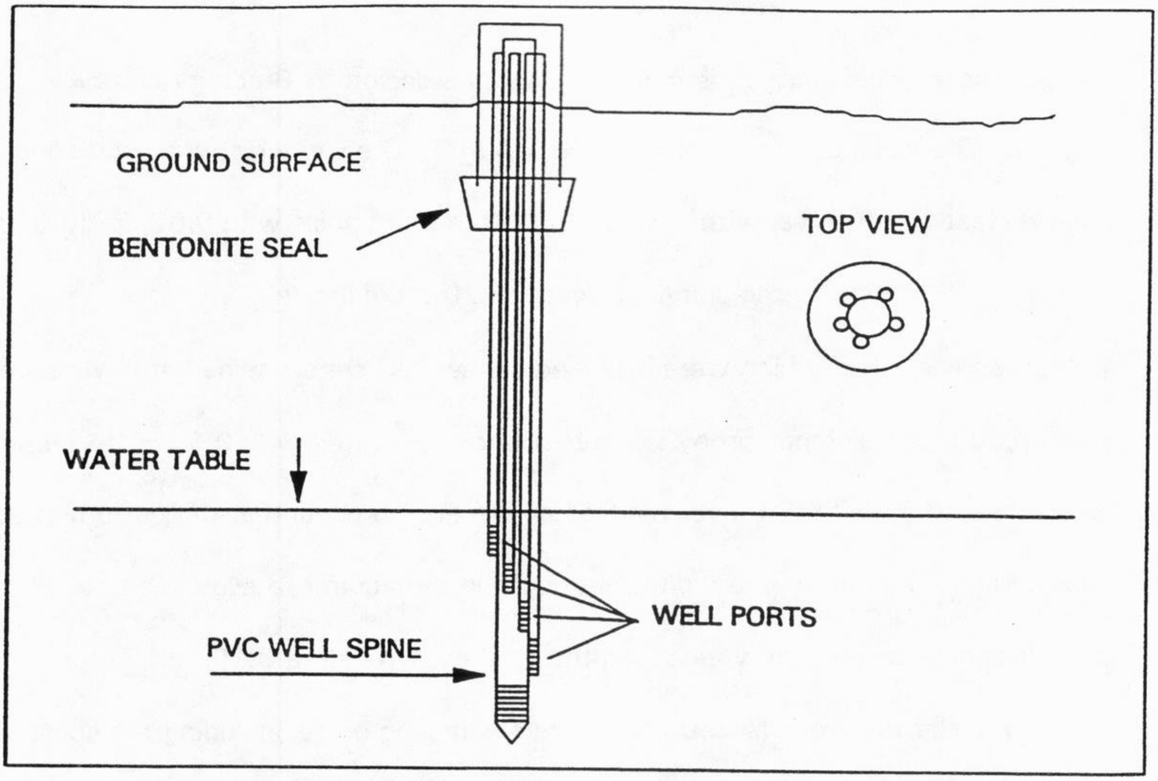


Figure 3.19 Multiport well design at Site #1.

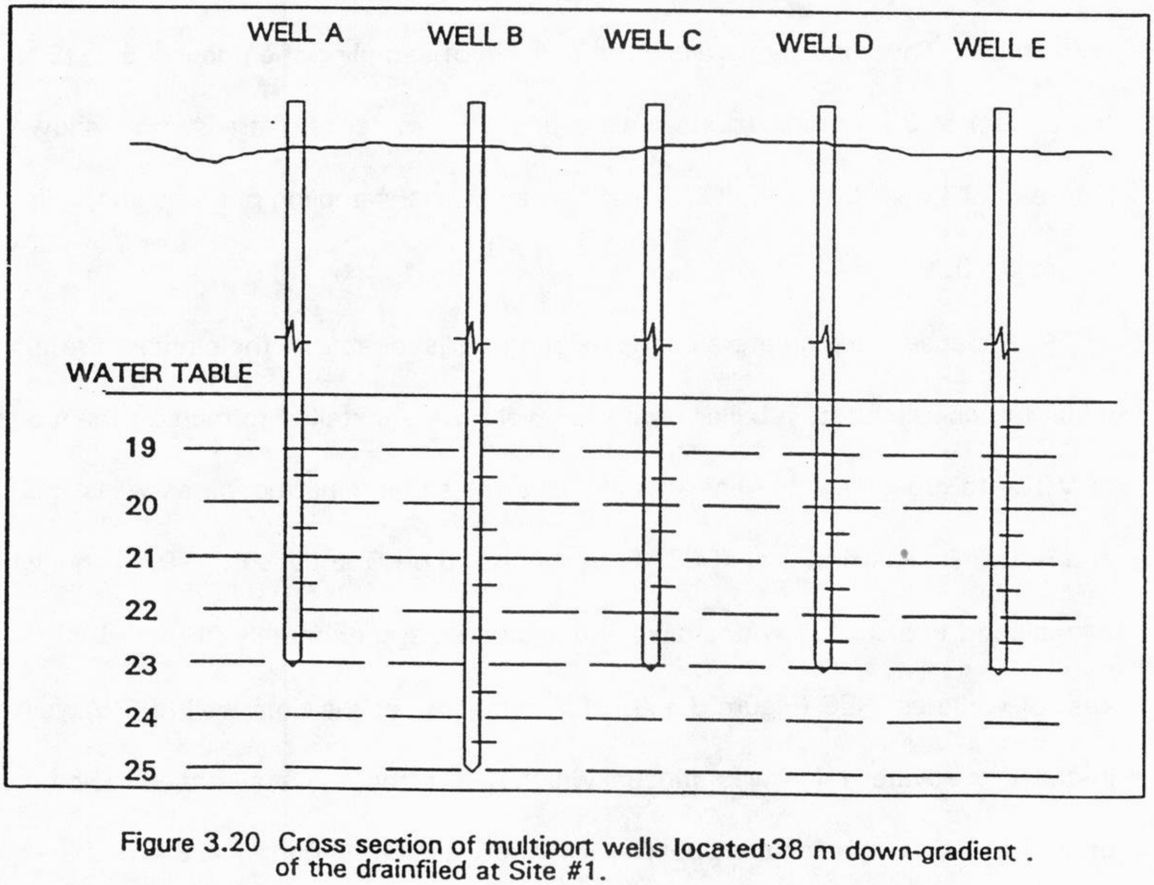


Figure 3.20 Cross section of multiport wells located 38 m down-gradient of the drainfiled at Site #1.

second VOC well was installed (REC V2) on December 1, 1994. This well is located approximately 25 cm (10 in) to the west of well nest REC. It should be noted that this well could not be located on the east side of well REC like REC V because underground utility lines are directly to the east of well REC V.

3.52 Study Site #2

At Site #2, monitoring wells were installed down-gradient of the mound system in the fall of 1990 as shown in Figure 3.21. Wells DGE and DGW were installed approximately 10.5 and 13.5 m (34 and 44 ft) down-gradient of the mound system. These down-gradient monitoring wells were multilevel well nests consisting of four 1.9 cm. (3/4 in.) PVC wells with 45 cm (1 1/2 ft), 0.02 cm (0.01 in) slotted points. The shallowest well in the well nest was placed with half of the screen above the water table to allow for the annual fluctuations of the water table (Figure 3.22).

Two additional wells were installed for VOC sampling. These wells are constructed from 3.2 cm (1 1/4 in) PVC and are fitted with 91 cm (36 in) slotted, 0.02 cm (0.01 in) slot size, PVC screens. Well DGE V was installed on October 26, 1993 when the water table was unusually high. The water table dropped below this well and well DGE V2 was installed on November 17, 1994. These wells were approximately 10.5 m (34 ft) down-gradient of the mound system and are adjacent to well nest DGE.

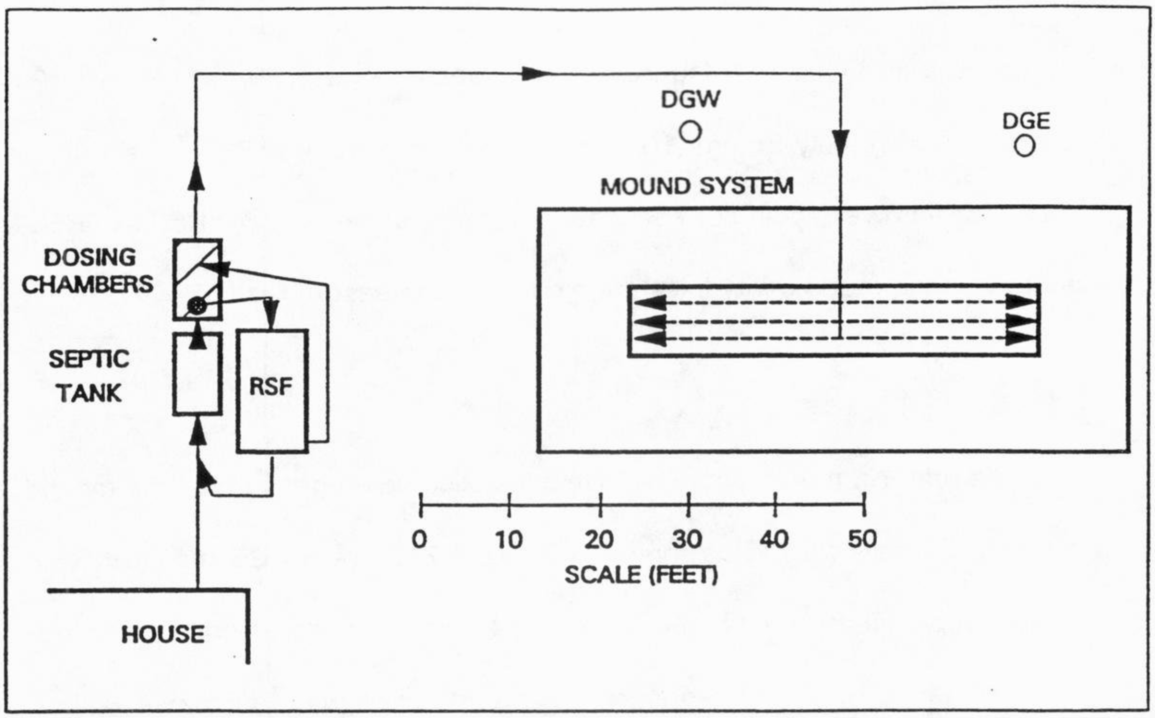


Figure 3.21 Location of down-gradient monitoring wells at Site #2.

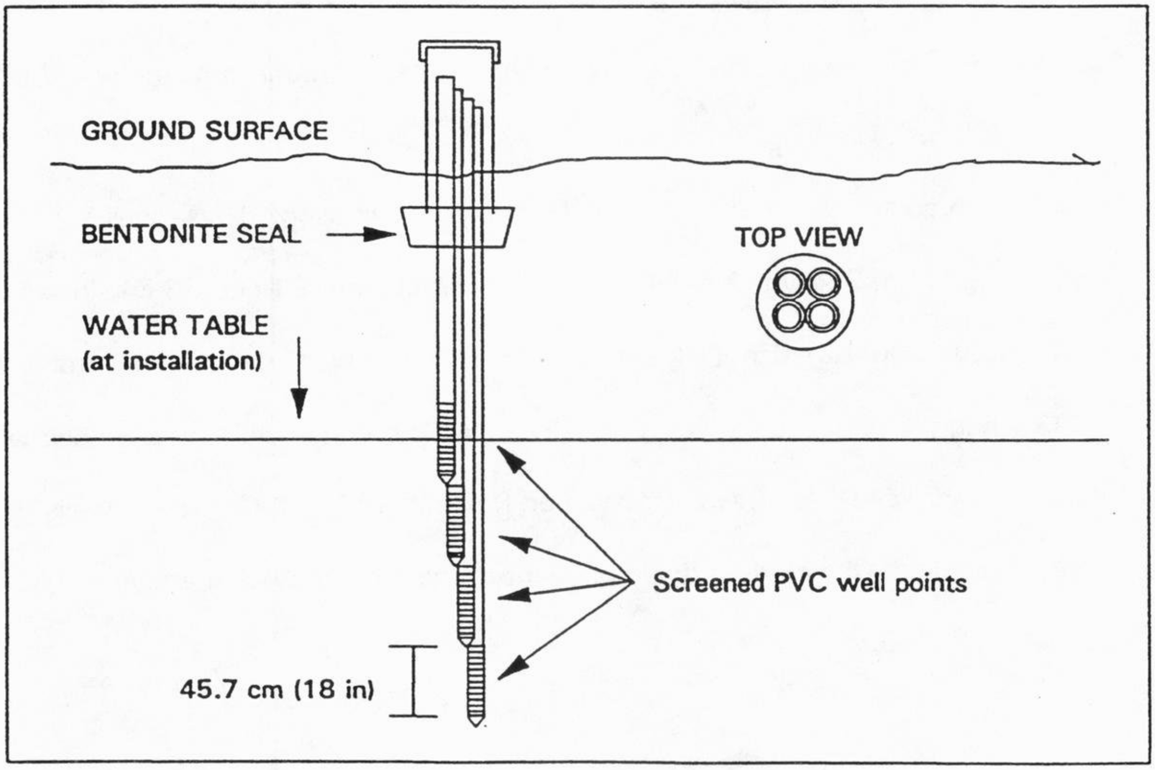


Figure 3.22 Site #2 DGW and DGE well design.

3.53 Groundwater Sampling Techniques

All monitoring wells were sampled using a 0.5 l/min peristaltic pump powered by a twelve volt battery. Samples were extracted through polypropylene sampling tubes and field filtered through an in line 0.45 micron membrane filter. The samples were then collected in 250 ml polypropylene bottles with polyethylene caps, both of which where double rinsed with sample water after several well volumes had been purged from the well. Samples were then placed in coolers containing ice packs and transported to the Environmental Task Force Laboratory (Lab State ID No. 750040280) at the University of Wisconsin-Stevens Point for analysis and storage at 4°C. Water table depth was measured using an audible popper attached to a measuring tape. The measuring tape accuracy was to the hundredth of a foot.

Groundwater samples were collected at the two locations prior to the initiation of the denitrification project. The two sites were each used previously for different projects and thus the previous sampling schedule and chemical analyses of the samples varied from one site to the other. Since the beginning of the denitrification project in the summer of 1991, groundwater samples were taken from both sites on at least a bimonthly basis. However, beginning in 1993 the down-gradient multiport wells at Site #1 (A,B,C,D,E) were monitored biannually. The samples from all the wells were analyzed for nine inorganic water quality indicators. These included pH, electrical conductance, alkalinity, total hardness, NH_4^+-N, NO_3^--NO_2^--N (nitrate-N), sodium, chloride, and fluorescence.

Samples for VOC analyses were collected from the VOC wells at both sites on a quarterly basis. Samples were collected by lowering a 30.5 cm (36 in) TEFLON bailer into the wells after purging several volumes from the well. These samples were placed in 40 ml glass vials with a TEFLON cap and a 0.5 ml hydrochloric acid preservative and kept on ice until they were brought to the laboratory for analysis.

4. RESULTS

4.1 Study Site #1

This section describes the results of the two projects involving the design and optimization of the RSF system for reducing nitrogen loading to groundwater at Site #1. The initial project involving the design and initial monitoring of the RSF system was conducted by Osesek and Shaw (unpublished) between September 1, 1991 and August 31, 1993. Initial investigations at Site #1 conducted from September 1, 1991 to July 16, 1992 involved monitoring of the septic tank and groundwater quality characteristics. The RSF system was installed on July 16, 1992 but did not become fully operational until October 27, 1992 when problems with the electrical controls for the pump were corrected. After this time the RSF system and effluent loading to groundwater were monitored during study periods #1 and #2.

The goal of the project conducted by this author between September 1, 1993 and June 29, 1995 was to determine the optimal nitrogen removal capability of the RSF system. To do this variations were made in the recirculation rates and in the frequency and duration of pumping events. During this time 23 study periods (A-W) of varying duration were observed.

4.11 Temperature Variations

The temperatures observed in the septic tank and sand filter at Site #1 are shown in Figure 4.1. Sand filter temperatures ranged from 2.5°C (February 4, 1994) to 19.2°C (August 9, 1994). Septic tank temperatures ranged from 3.3°C (February 25, 1995) to 19.1°C (July 15, 1994). In general the septic tank is colder than the RSF in the spring and summer months because the sand filter is approximately 0.9 meters (3 feet) below the ground surface and is subject to solar radiation. In the fall and winter the reverse is true. The temperatures at this site are relatively low compared to Site #2 because of the shallow depth of the RSF, the relatively low water usage, and the homeowners use of only cold water to do laundry.

Temperature readings for the four sets of thermocouple wires located in the RSF at Site #1 can be seen in Figure 4.2. Temperature readings for the various depths for each set of thermocouple wires can be seen in Figure 4.3 (a - d). There were no appreciable differences horizontally (east, west, north, south) within the sand filter. This suggests fairly uniform waste application. Some variation was encountered with depth in the RSF with the deepest probe being the warmest in the winter.

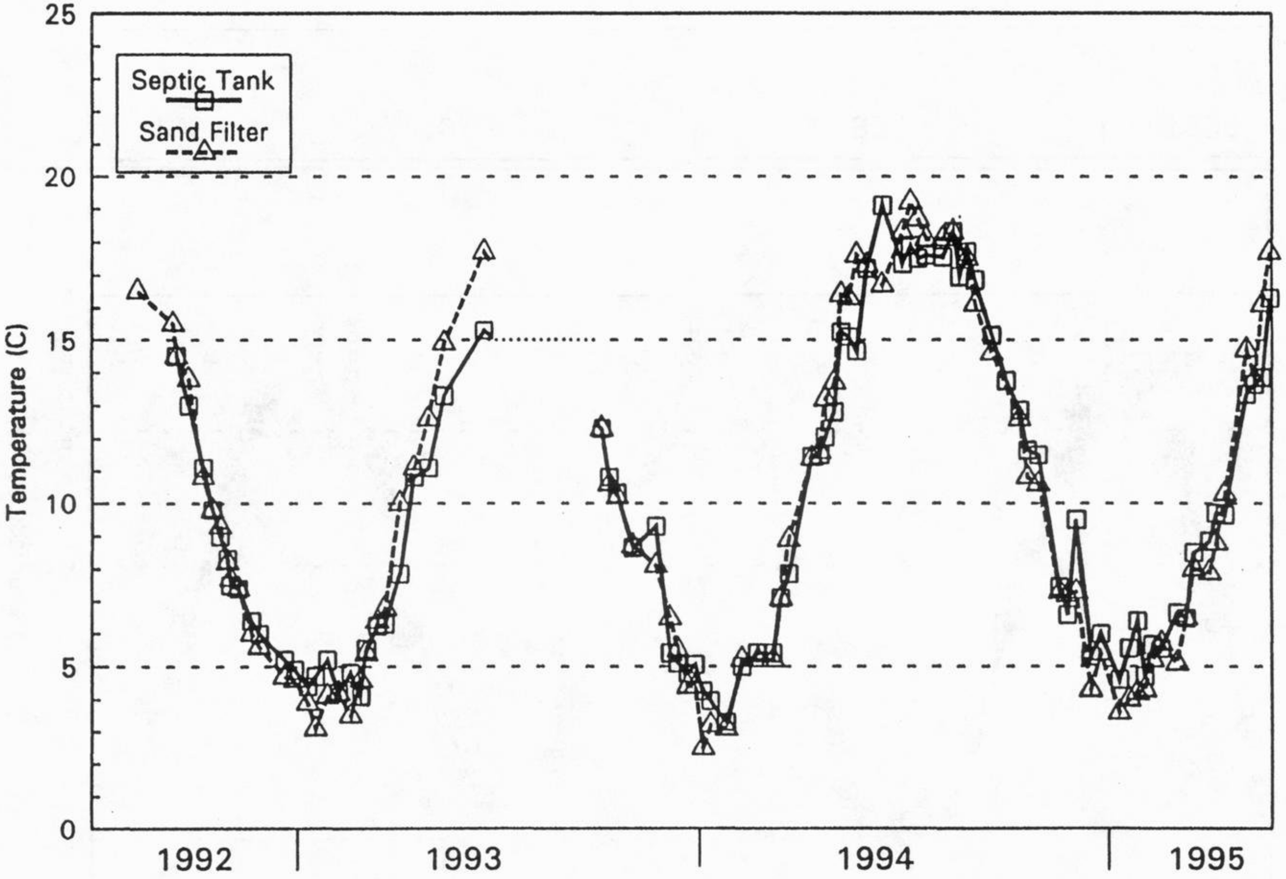


Figure 4.1 Temperature readings in the septic tank and pump chamber of the RSF, 1992-1995, Site #1.

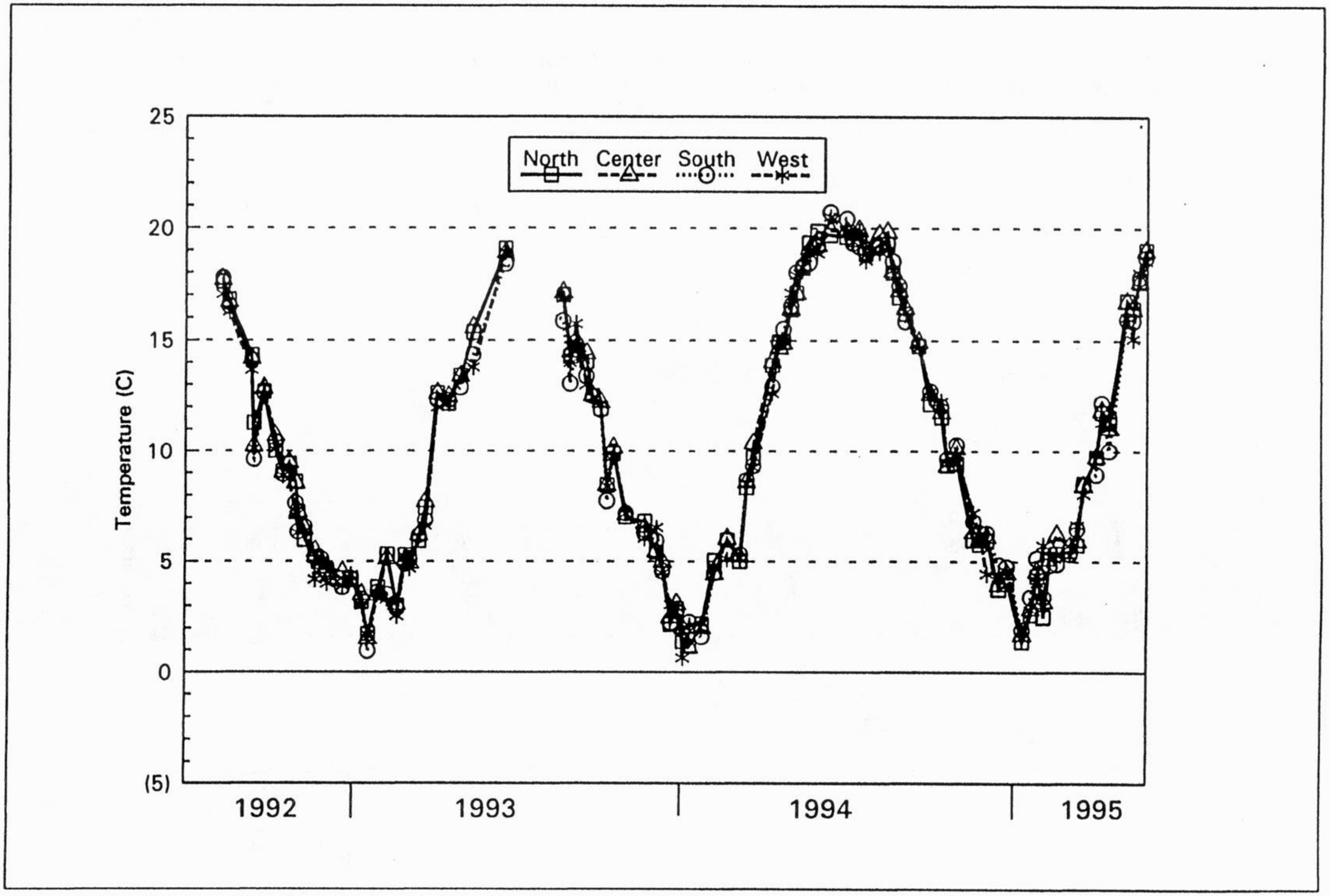
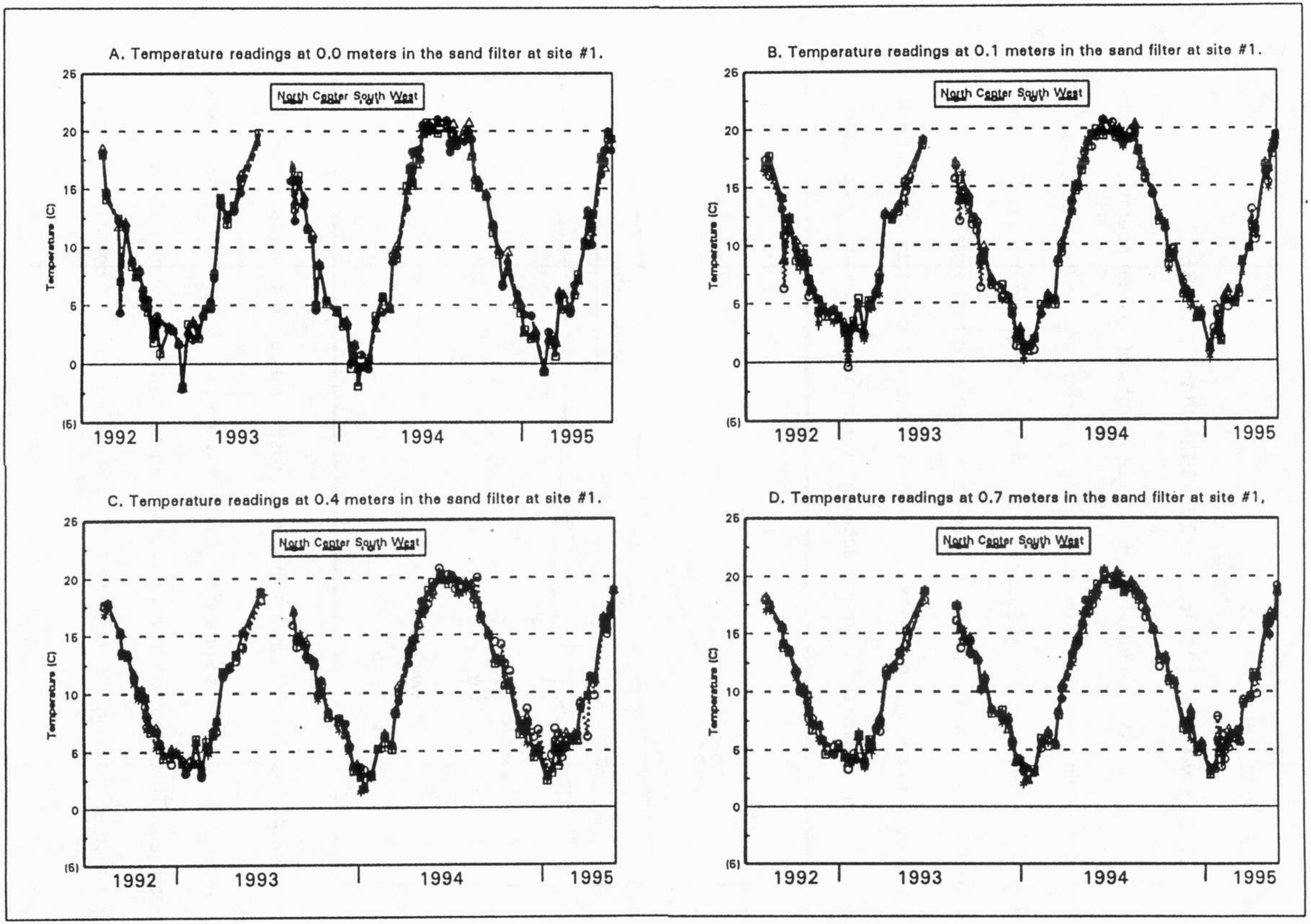


Figure 4.2 Average temperature readings for all depths in the RSF 1992-1995, Site #1.


A. Temperature readings at 0.0 meters in the sand filter at site #1.

B. Temperature readings at 0.1 meters in the sand filter at site #1.

C. Temperature readings at 0.4 meters in the sand filter at site #1.

D. Temperature readings at 0.7 meters in the sand filter at site #1,

Figure 4.3 Temperature readings at various depths in the RSF at Site #1.

4.12 Flow Conditions

4.121 Study Period #1 (October 27,1992 -June 8, 1993).

A problem which lasted the first eight months of system operation resulted from retrofitting the RSF to the existing septic system at Site #1. As shown in Figure 4.4, when effluent was supposed to be pumped out to the drainfield from the RSF, all of the effluent should have been discharged to the drainfield. However, while the entire length of the existing pipe from the septic tank to the drainfield was pitched downward, a smaller segment within the total length was pitched back towards the RSF. This apparently occurred when the original septic system was installed. This condition caused an unknown amount of effluent which was pumped to the drainfield to return to the RSF through the "T" connection (Figure

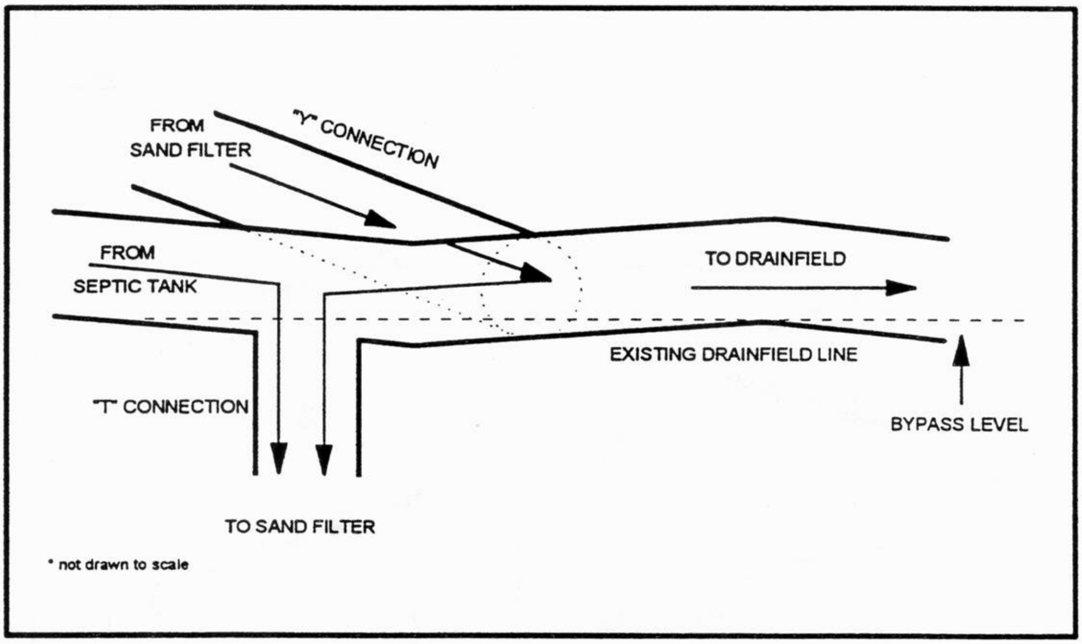


Figure 4.4 Side profile of upward pitch in the existing line to the drainfield at Site #1.

4.4). As a result, the effluent levels in the RSF remained at the bypass level; the level where the effluent level in the RSF is equal in height to the highest elevation of the pipe between the "Y" connection and the drainfield. This created difficulty in determining the portions of the effluent that received treatment in the sand filter or that bypassed the system.

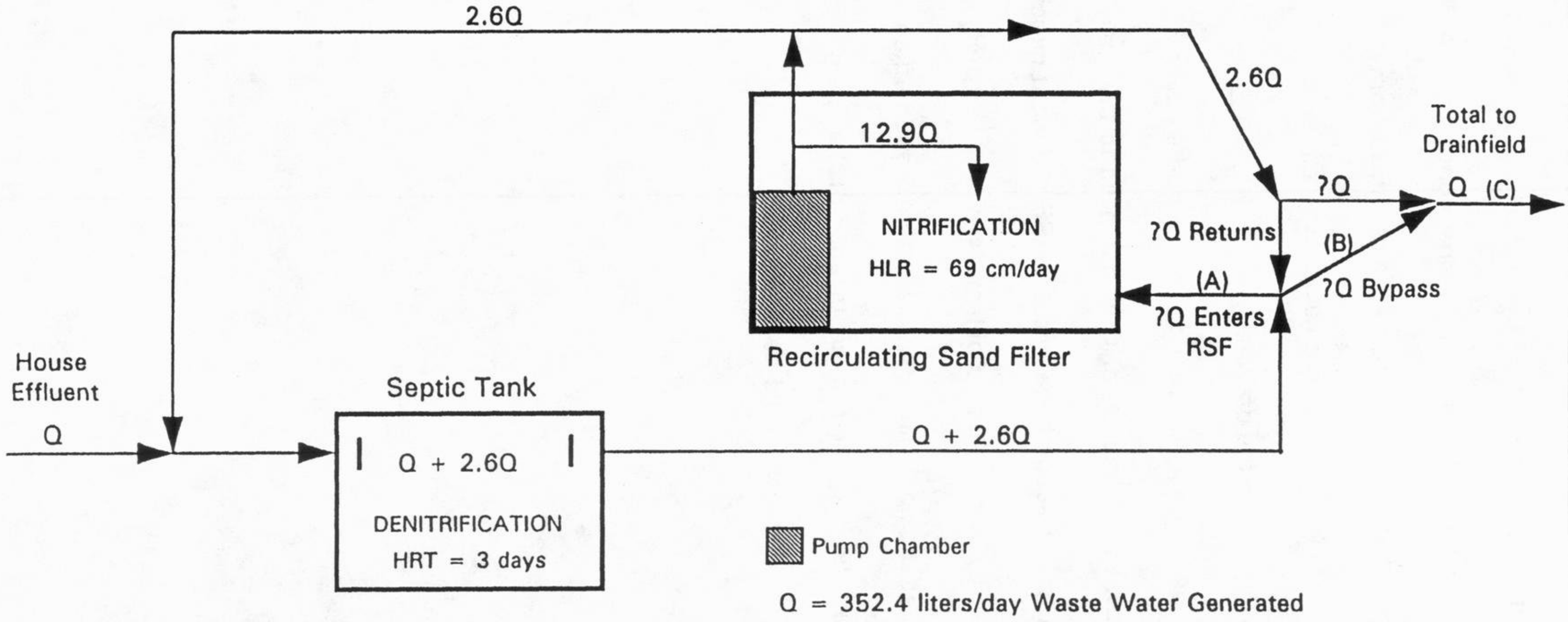
The flow control 'valves' were sized to pump 18.9 liters/min (lpm) (5.0 gallons/min (gpm)) to the top of the sand filter, 3.8 lpm (1.0 gpm) back to the septic tank, and 3.8 lpm (1.0 gpm) to the drainfiled. The timer controls were set to a 5 minute ON interval with a 25 minute DELAY interval. With these design settings, the maximum pumping time would be 240 minutes/day or 48 cycles/day if effluent levels in the RSF kept the DELAY interval float up. By pumping effluent from the RSF to the drainfield the effluent levels in the RSF should drop and result in fewer pumping cycles/day. The average household water usage at Site #1 during study period #1 was 352.4 lpd (93.1 gpd). Assuming that all of the household water used entered the RSF and that all of the 3.8 lpm (1.0 gpm) pumped to the drainfield entered the drainfield and did not return to the RSF, the pump in the pump chamber of the RSF should have been on an average of 18.5 cycles/day (Eq 8) or running a total of 92.5 min/day.

$$\textbf{352.4 liters/day / (3.8 liters/min * 5 min/cycle) = 18.5 cycles/day} \qquad \textbf{(7)}$$

The expected hydraulic conditions for these design settings are a sand filter recirculation rate (SFR), defined as *the ratio of flow pumped to the top of the sand filter to the forward flow (Q),* of 5.0Q. A septic tank recirculation rate (STR), defined as *the ratio of flow pumped to the septic tank to the forward flow (Q),* of

1.0Q. A drainfield rate, defined as *the ratio of flow pumped to the drainfield (Q*) to the forward flow (Q),* of 1.0Q. The expected hydraulic loading rate (HLR) to the top of the sand filter was 26.7 cm/day (6.5 gpd/ft^2) based on a sand filter area of 6.54 m^2 (70.4 ft^2) and an expected flow to the top of the sand filter of 1,748 lpd (461.8 gpd). The expected hydraulic retention time (HRT) in the septic tank was approximately 5 days based on the liquid storage volume of the septic tank (3,785 l or 1,000 gal) and the forward flow (Q) plus the expected flow recycled back to the septic tank 1.0Q.

However, due to the upward pitch in the drainfield line, effluent levels in the RSF remained high and the upper (DELAY interval) float never came into effect; therefore, the pump ran through the maximum 48 cycles/day or 240 minutes/day. This resulted in larger amounts of effluent being applied to the top of the RSF and to the septic tank relative to the amount pumped to the drainfield than was intended. Approximately 912 lpd (241 gpd) were pumped to the septic tank and drainfield and 4,536 lpd (1,198 gpd) to the top of the RSF. The actual hydraulic conditions were an SFR of 12.9Q, an STR of 2.6Q, an HLR of 69 cm/day (17 gpd/ft^2), and an HRT of 3 days. The hydraulic conditions observed during study period #1 are higher than the expected conditions because the pump continued to operate since effluent levels in the RSF did not drop. Both the flow ratios of the effluent pumped from the RSF to the drainfield (Q*) and the effluent bypassing the system are unknown since it is not known how much of the effluent pumped to the drainfield returned to the sand filter. A diagram of the flow schematics throughout the first study period at Site #1 can be seen in Figure 4.5.



(A) - An unknown mixture of the septic tank effluent and effluent directed to the drainfield that did not overcome the upward pitch in the drainfield line enters the RSF.

(B) - An unkown portion of the septic tank effluent bypassed the RSF and went directly to the drainfield.

(C) - The amount of effluent applied to the drainfield (Q) is an unknown mixture of bypassed effluent and pump chamber effluent directed to the drainfield that did not reenter the RSF.

HLR - Hydraulic Loading Rate to top of sand filter ((Flow pumped to top of sand filter) / Area of sand filter (6.54 m^2)).

HRT - Hydraulic Retention Time in the septic tank ((Liquid Storage Volume (3785 liters)) / Total Flow into Septic Tank).

Figure 4.5 Flow schematic for study period #1 (before the installation of the "Dam") at Site #1

This condition was rectified on June 8, 1993 by inserting a "dam" in the existing line to the drainfield between the "T" and "Y" connections (Figure 4.6 and Figure 4.7). This "dam" prevented effluent pumped to the drainfield from reentering the RSF by allowing it to accumulate to a height where it would overcome the upward pitch in the pipe to the drainfield.

The average household water usage (Q) during study period #1 was 352 lpd (93 gpd). However, it is unknown what fraction of Q was applied to the drainfield after treatment in the RSF and what fraction bypassed the RSF and went directly to the drainfield from the septic tank. Due to these problems, the data for study period #1 are difficult to interpret and not included in the determination effluent treatment by the RSF system at Site #1. After the installation of the "dam", the amounts of effluent applied to the drainfield could be determined.

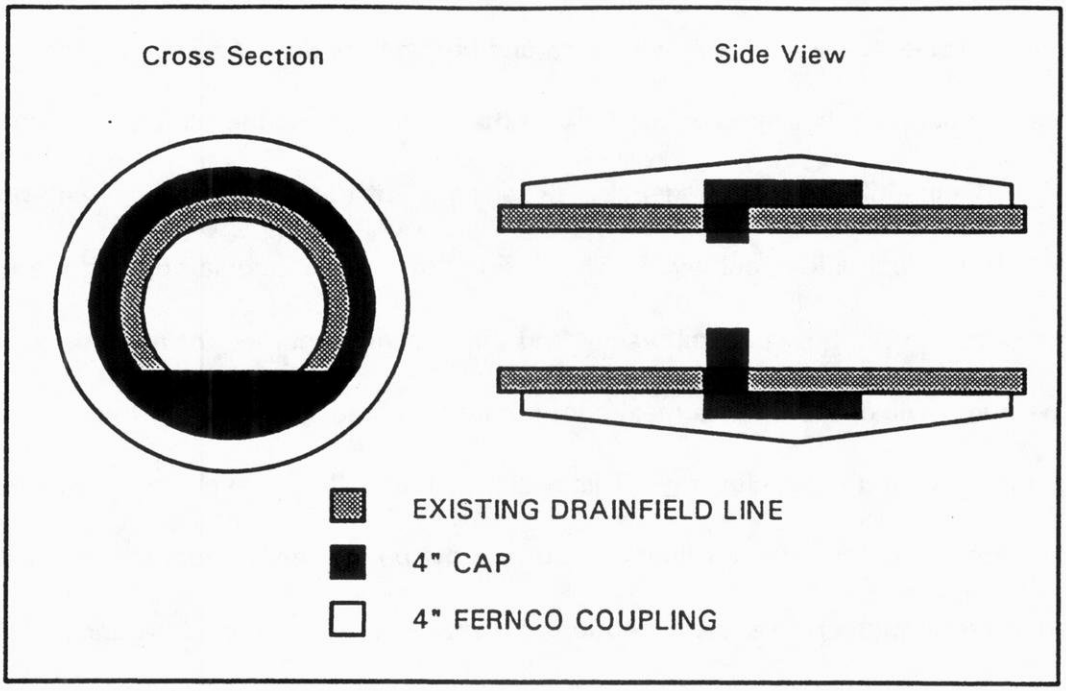


Figure 4.6 Cross section and side view of the "Dam" installed in the drainfield line.

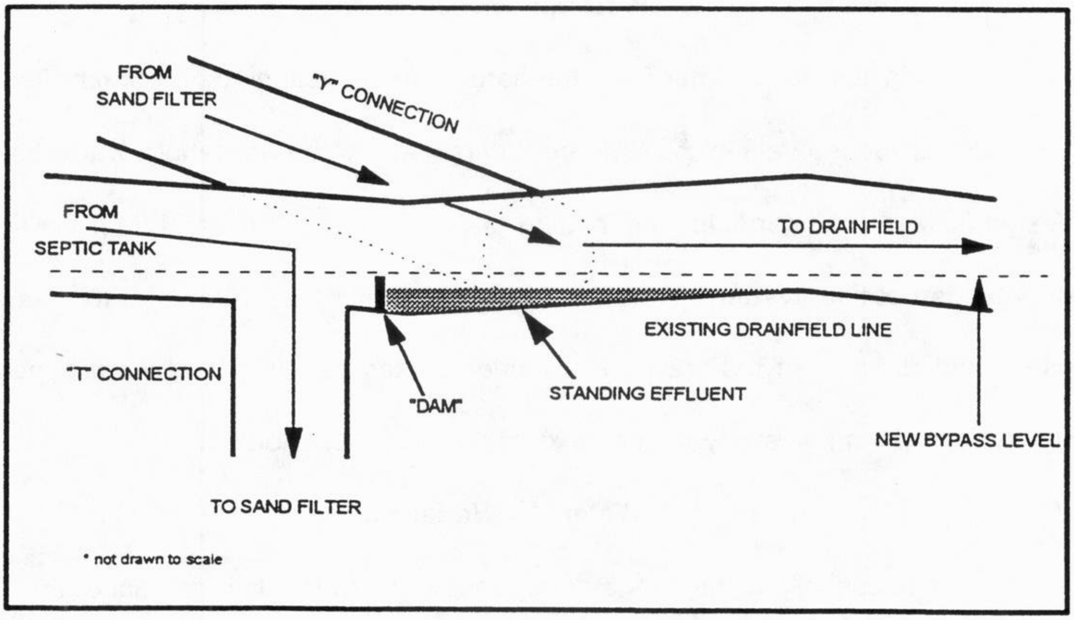


Figure 4.7 Side profile of the drainfield line with the "Dam" installed.

4.122 Design Flow Conditions

Table 4.1 shows the flow rates and programmable timer settings which were used to achieve the variations in flow to the sand filter, septic tank, and drainfield after the installation of the "dam". These represent the design settings and not the actual flow conditions achieved. The design flow rate is calculated from the flow rate through the flow control 'valve' and the maximum number of minutes per day the pump will operate. The maximum number of minutes per day is based on the ON interval and 48 cycles/day. This assumes that effluent levels in the RSF keep the upper (DELAY interval) float up and that the pump runs through two cycles/ hour every hour. However, exceptions are made for study periods A and H since the programmable timer settings allow only 36 cycles/day and 24 cycles/day, respectively.

Problems Encountered

The design flows differ from the actual flows depending on effluent levels in the RSF. Effluent levels in the RSF are determined by the household water use, the design flow to the drainfield, and loading to the bottom of the sand filter via the gravity distribution system. Problems in matching the design flows to the water usage and clogging of the gravity distribution system resulted in effluent bypassing the system during all study periods except #2, A, E, G, and J..

Water Use Variations

The household water usage determines effluent levels in the sand filter and thus controls the DELAY interval float which operates the pumping events. When

Table 4.1 Settings of the programmable timers, design flow rates, and actual duration of pump operation at Site #1 for all study periods after the installation of the "dam".

Study Period	Programmable Timer Setting		Design Pump Time* (min/day)	Flow Control Valve Setting			Design Flow Rates			Actual Cycles/day	Actual Pump Time (min/day)
	ON (min)	DELAY (min)		RSF (Liters/min)	Septic Tank (Liters/min)	Drainfield (Liters/min)	RSF (Liters/day)	Septic, Tank (Liters/day)	Drainfield (Liters/day)		
#2	5	25	240	15.1	8.5	3.8	3624	2044	908	20.3	203.0
A	11.5	25	552**	14.2	8.5	3.8	3804	2279	1014	9.9	113.7
B	5	25	240	14.2	9.0	3.8	3407	2158	908	17.6	87.9
C	5	25	240	14.2	9.0	1.9	3407	2158	454	20.9	104.7
D	0	0	0	0.0	0.0	0.0	0	0	0	0.0	0.0
E	5	25	240	14.2	9.0	1.9	3407	2158	454	47.5	237.6
F	5	25	240	14.4	5.3	1.9	3452	1272	454	39.6	197.9
G	5	25	240	14.4	3.8	1.9	3452	908	454	45.4	227.1
H	5	55	120**	14.4	3.8	1.9	1726	454	227	23.9	119.3
I	5	25	240	14.4	3.8	1.9	3452	908	454	44.5	222.5
J	5	25	240	14.4	5.3	1.9	3452	1272	454	45.2	226.0
K	5	25	240	11.4	4.9	1.9	2725	1181	454	46.7	233.3
L	5	25	240	9.5	11.4	1.9	2271	2725	454	21.2	105.8
M	5	25	240	5.7	4.3	1.9	1363	1022	454	20.1	100.4
N	5	25	240	9.5	0.0	1.9	2271	0	454	15.5	77.3
O	4	26	192	9.5	4.7	1.9	1817	908	363	15.0	60.0
P	4	26	192	18.9	4.7	0.9	3634	908	182	42.6	170.5
Q	4	26	192	18.9	4.7	0.0	3634	908	0	39.8	159.2
R	4	26	192	18.9	4.7	0.0	3634	908	0	?	?
S	4	26	192	18.9	4.7	0.0	3634	908	0	48.0	192.8
T	4	26	192	18.9	5.7	0.0	3634	1090	0	48.0	194.2
U	4	26	192	15.1	7.6	1.9	2907	1454	363	44.9	179.6
V	3	27	144	15.1	7.6	1.9	2180	1090	274	48.0	144.0
W	5	25	240	15.1	7.6	1.9	3634	1817	454	48.0	240.0

*Based on maximum cycles per day; 2 cycles per hour = 48 Cycles/day (Assumes that effluent levels in RSF keep the upper (DELAY) float up)

**Exceptions: Study Period A (36 Cycles/day) and Study Period H (24 Cycles/day) water usage is less than the amount of effluent pumped to the drainfield, wastewater levels in the RSF drop and the pump does not operate until levels rise and activate the DELAY interval. This results in fewer cycles/day and actual flows which are lower than the design flows. In concept, the design flows to the drainfield should be oversized with respect to expected water usage to keep the number of pumping events and thus energy costs down and prevent effluent from bypassing the RSF. By setting the design flow to the drainfield higher than the expected water usage, effluent levels in the pump chamber of the RSF should be lowered and require fewer pumping cycles. However, a minimum number of pumping cycles are required for effective wastewater treatment in the RSF. The variability of household water usage at Site #1 can be seen in Figure 4.8.

Flows to Drainfield

Flows in 13 of the 24 study periods were sufficiently designed to accommodate anticipated water usage. However, the flow to the drainfield was under designed during 11 of the 24 study periods (Table 4.2). Under designed flows to the drainfield occurred when the design flow to the drainfield was insufficiently sized compared to the household water usage. This resulted in untreated effluent bypassing the system. Over the course of this study this occurred because two major errors were made in the design settings.

1) Flows to the drainfield were under designed compared to variable water usage.
2) Reductions of the frequency and duration of pumping events without increasing the size of the flow control valve to the drainfield

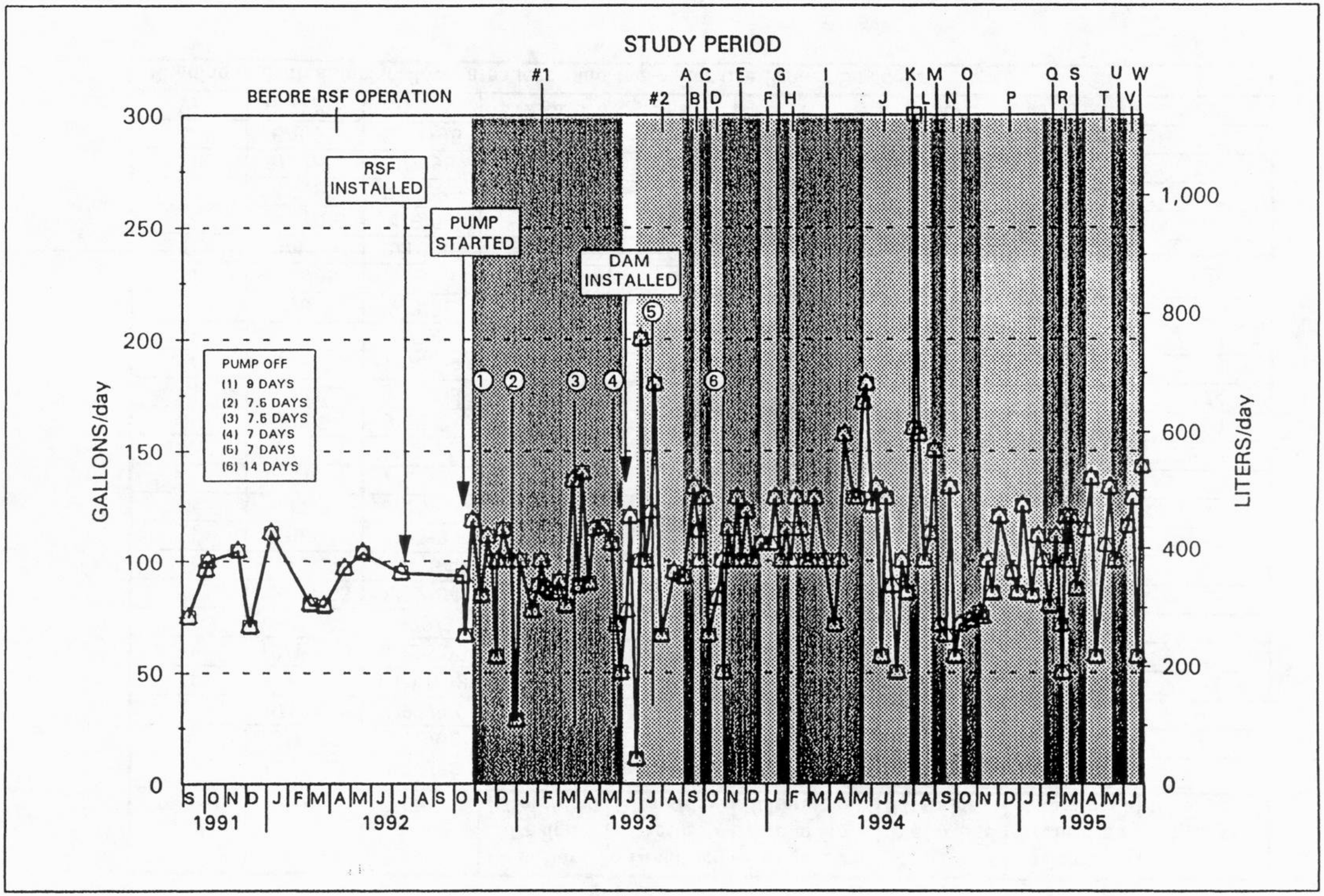


Figure 4.8 Water use at Site #1 from 1991 to 1995.

Table 4.2 Comparison of design and actual flow rates to the drainfield with water usage for all study periods after the installation of the "dam" at Site #1.

Study Period	Start	End	Flow Rate To Drainfield		Design Over Actual Flow (Liters/day)	Water Use (Liters/day)	Design Over Water Use (Liters/day)	Bypass (Liters/day)
			Design (Liters/day)	Actual (Liters/day)				
#2	6/30/93	8/31/93	908	385	523	385	385	0
A	9/1/93	9/16/93	1014	430	584	428	586	0
B	9/17/93	9/23/93	908	333	576	430	478	97
C	9/24/93	10/11/93	454	198	256	372	82	175
D	10/12/93	10/26/93	0	0	0	347	-347	347
E	10/27/93	12/23/93	454	450	5	389	65	0
F	12/24/93	1/12/94	454	375	79	447	7	72
G	1/13/94	2/4/94	454	430	25	396	58	0
H	2/5/94	2/11/94	227	226	2	487	-260	261
I	2/12/94	5/20/94	454	421	33	447	7	26
J	5/21/94	7/22/94	454	433	21	399	55	0
K	7/23/94	8/9/94	454	441	13	779	-324	337
L	8/10/94	8/25/94	454	201	254	401	53	201
M	8/26/94	9/16/94	454	190	264	364	90	174
N	9/17/94	10/6/94	454	146	308	330	124	184
O	10/7/94	11/7/94	363	114	250	284	79	171
P	11/8/94	2/3/95	182	161	20	382	-200	220
Q	2/4/95	3/3/95	0	0	0	296	-296	296
R	3/4/95	3/10/95	0	0	0	416	-416	416
S	3/11/95	4/6/95	0	0	0	406	-406	406
T	4/7/95	5/11/95	0	0	0	412	-412	412
U	5/12/95	6/7/95	363	340	23	408	-45	69
V	6/8/95	6/21/95	274	274	0	351	-79	78
W	6/22/95	6/28/95	454	454	0	541	-87	82

Shading indicates design flow rates less than the water use (Under Designed).

The flow to the drainfield was under designed compared to the water usage during study period K. The average water usage made an unexpected increase from 399 to 779 lpd (105 to 206 gpd) between study periods J and K. The design flow to the drainfield was 454 lpd (120 gpd) during study period J and was not changed during study period K. This resulted in 324 lpd (85.6 gpd) of untreated effluent bypassing the RSF and going directly to the drainfield. This also occurred during study period W where the design flow to the drainfield was 454 lpd (120 gpd) and the average water use was 541 lpd (143 gpd) therefore, 82 lpd (23 gpd) bypassed the system. Although these increases in water usage occurred during summer months they cannot be attributed to irrigational uses since the water meter measures only the water used within the household. The increases may be due to visitors at the site.

Under designed flows to the drainfield also occurred due to problems with the electrical controls for the pump in the pump chamber and errors made in the setting of the flow control valve to the drainfield. During study period D the timer control panel for the pump in the pump chamber of the RSF was removed for a design modification. As a result, the pump was off-line and therefore the flow to the drainfield was 0 lpd (0 gpd), under designed and 347 lpd (91.7 gpd) bypassed the system. During study periods Q, R, S, and T the flow control valve to the drainfield was mistakenly set to 0 Liters/day (0 gpd) and bypass resulted.

During study periods H, P, U, and V the flow to the drainfield was under designed when the frequency and duration of pumping events were reduced without increasing the size of the orifice in the flow control 'valve' to the drainfield.

For example, during study period H the DELAY interval was increased to 55 minutes, meaning the pump would run through 24 instead of 48 cycles/day. The flow to the drainfield should have been doubled to accommodate the change in pumping event frequency and duration. However, the flow to the drainfield was left at 9.5 Liters/cycle or 227 lpd (60 gpd). The average household water use was 487 lpd (128.6 gpd) and 260 lpd (68.6 gpd) of bypass resulted.

Gravity Distribution System

The loading of effluent to the bottom of the RSF via the gravity distribution system may have been inhibited as evidenced by study periods B, C, F, I, L , M, N, and O (Table 4.2). During these time periods the design flows to the drainfield were sufficiently sized over the water use. However, 26 to 201 lpd (6.8 to 53.1 gpd) of untreated effluent bypassed the system since the actual flows were less than the water usage. Based on observations made of effluent levels in the pump chamber of the RSF and the inspection pipe of the gravity distribution system, this bypass was occurring due to partial blockage of the gravity distribution system.

It was concluded that the gravity distribution system, to the bottom rock storage layer, was not an effective means of loading effluent to the RSF without prior filtration. It was suspected that partial clogging of the holes in the distribution system resulted in effluent bypassing the system. Observations of water levels in the gravity distribution system's inspection pipe and the pump chamber of the RSF showed effluent levels in the inspection pipe to be 2.5 to 5 cm (1 to 2 in) below the bypass level while levels in the RSF were approximately 50.8 to 52.1 cm (19.5 to 20.5 in) below the bypass level. The gravity distribution system should drain completely into the sand filter and should not contain 25.4 cm (10 in) of effluent above the holes located in the bottom of the pipe when effluent levels in the sand filter are 16.5 cm (6.5 in) below the bottom of the pipe.

Figure 4.9 shows the levels at which effluent bypasses the system or activates the DELAY interval. At the bypass level the RSF should contain 73.7 cm (29 in) of effluent. Using the dimensions of the sand filter (365.8 cm x 185.4 cm, or 144 in x 73 in) and an average porous volume of 39%, the volume of effluent in the sand filter at the bypass level should be 1950 Liters (515 gal) (Eq 8). However, the effluent levels in the RSF never reached this level. Instead effluent levels remained near or slightly below the level at which the float activates the DELAY interval. The level at which the DELAY interval becomes activated is 24.1 cm (9.5 in). Using the dimensions of the sand filter and an average porous volume of 39% the volume of effluent needed to activate the DELAY interval is 637 Liters (168.3 gal) (Eq 9).

$$(365.8 * 185.4 * 73.7)\ \text{cm}^3 * (\text{m} / 100\ \text{cm})^3 * 1000\ \text{Liters/m}^3 * .39 = 1950\ \text{Liters} \qquad (8)$$

$$(365.8 * 185.4 * 24.1)\ \text{cm}^3 * (\text{m} / 100\ \text{cm})^3 * 1000\ \text{Liters/m}^3 * .39 = 637\ \text{Liters} \qquad (9)$$

As effluent was directed to the drainfield levels in the sand filter would drop below the delay interval level and result in fewer pumping cycles/day and actual flows which were lower than the design flows. If complete blockage of the gravity distribution system had occurred, effluent levels in the sand filter would remain below the DELAY level. However, a fraction of the household water was making it through the gravity distribution system and reactivating the DELAY interval. Since only a fraction of the household water was entering the RSF the number of

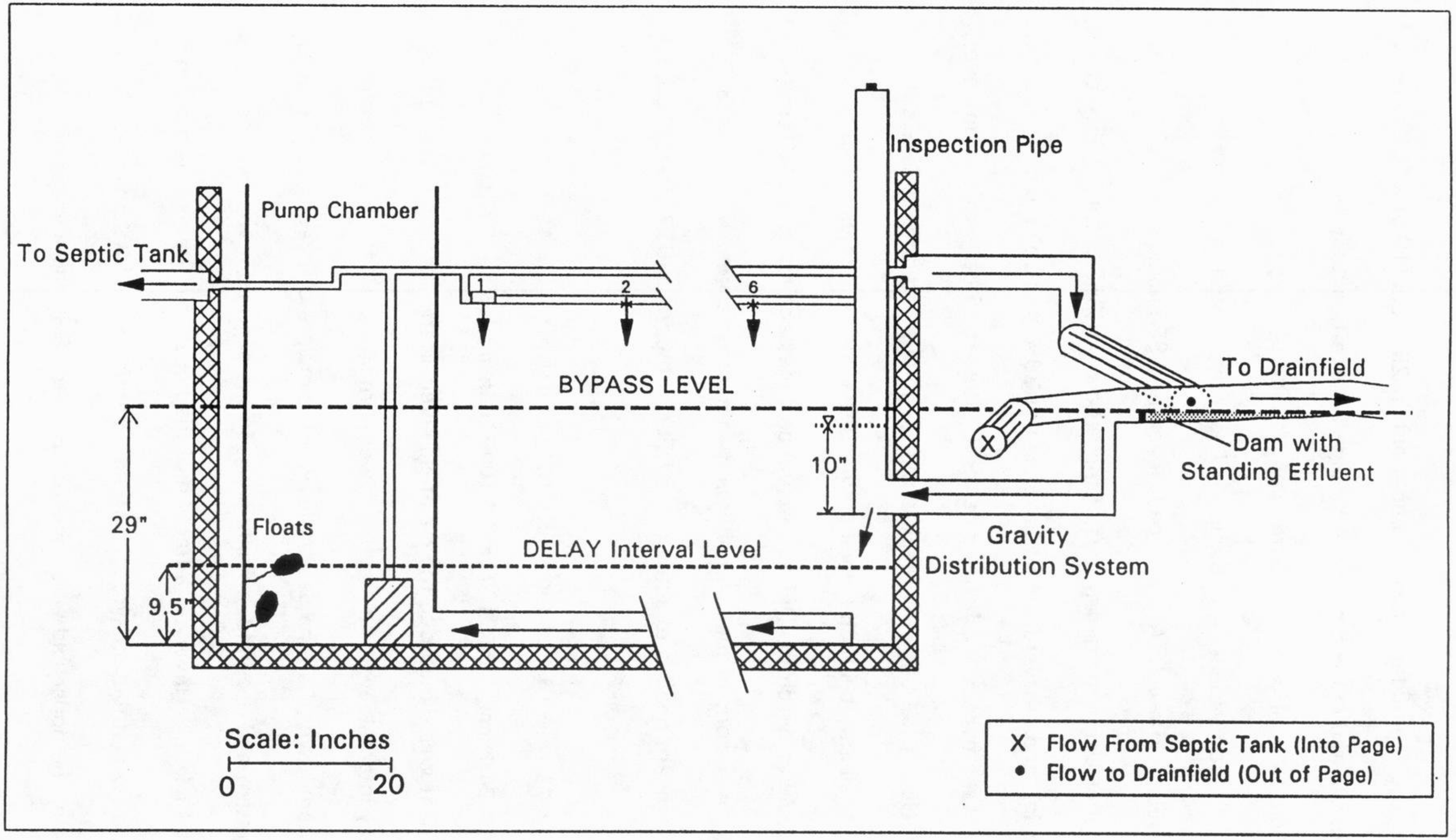


Figure 4.9 Cross section of RSF at Site #1 showing bypass and delay interval levels and also problem with partial clogging of gravity distribution system (inspection pipe) (Note. longitudinal axis of RSF has been abrieviated)

cycles/day were reduced and therefore, the actual flows were less than the design flows. This was not realized to be a problem until March 3, 1995 (study period Q) when the gravity distribution system was cleaned by running a four-inch plastic brush through it. Masses of organic solids and human hair were removed from the distribution system by this method. It is suspected to have alleviated this problem since the levels in the RSF increased and as a result the number of cycles/day increased, however, it is unknown if this totally alleviated this problem in the remaining study periods (R - W) since the drainfield flows were under designed and bypass resulted.

Blockage of the gravity distribution system was not constant over the time periods before cleaning. Study periods #2, A, E, G, and J were sufficiently designed and the actual flows matched or exceeded the water usage, no flow bypassed the system.

4.123 Actual Flow Conditions

The actual flow rates (average flow rate (lpd)) are determined from the flow rates through the flow control 'valves' (lpm) and the average actual pumping time (minutes/day) (Table 4.1). The average flow rates, the hydraulic loading rate (HLR) in cm/day, and the hydraulic retention time (HRT) in days are summarized for all study periods in Table 4.4. Table 4.5 contains the sand filter ratio (SFR), septic tank ratio (STR), drainfield ratio (Q*), and bypass ratio (BPR). Flow ratios are defined as *the ratio of flow going to the specified area to the forward flow (Q) through the system*. The forward flow (Q) through the system is defined as *the* *average household water usage*. The average flow conditions were used in the mass balance calculations.

Initially, problems were encountered with pump failures; however, these problems were latter resolved through the redesign of the timer control panel. Grains of sand occasionally clogged the orifices of the flow control 'valves'. The flow conditions and samples that represent these occasions are not included in the flow rate and concentration averages used in the mass balance calculations. These occasions and the cause for their omission are included in Table 4.3.

Table 4.3 Dates omitted from the flow rate and concentration averages used in the mass balance calculations at Site #1.

Date(s)	Study Period	Cause for Omission
10/28/92 to 11/6/92	#1	Pump Off 9 days
12/14/92 to 12/22/92	#1	Pump Off 7.5 days
3/16/93 to 3/23/93	#1	Pump Off 7.5 days
5/12/93 to 5/19/93	#1	Pump Off 7 days
7/8/93 to 7/16/93	#2	Pump Off 8 days
10/12/93 to 10/26/93	D	Pump Off 14 days, Redesign of Timer Control Panel
2/5/94	H	Drainfield Orifice Clogged
6/2/94	J	Septic Tank Orifice Clogged
7/15/94	J	Septic Tank Orifice Clogged
5/5/95	T	Septic Tank Orifice Clogged

Table 4.4 Average hydraulic conditions for all study periods after the installation of the "dam" at Site #1.

Study Period	Start	End	Forward Flow (L/day)	Flow to Top of Sand Filter (L/day)	HLR [a] (cm/day)	Flow to Septic Tank (L/day)	HRT [b] (days)	Flow to Drainfield (L/day)	Bypass to Drainfield (L/day)
#2	6/30/93	8/31/93	385	1,533	23	863	2.5	385	0
A	9/1/93	9/16/93	428	1,614	25	969	2.7	430	0
B	9/17/93	9/23/93	430	1,247	19	790	3.1	332	97
C	9/24/93	10/11/93	372	1,486	23	941	2.9	198	174
D	10/12/93	10/26/93	347	0	0	0	10.9	0	347
E	10/27/93	12/23/93	389	3,373	52	2,136	1.5	450	0
F	12/24/93	1/12/94	447	2,810	43	1,030	2.6	374	72
G	1/13/94	2/4/94	396	3,223	49	860	3.0	430	0
H	2/5/94	2/11/94	487	1,693	26	452	4.0	226	261
I	2/12/94	5/20/94	447	3,159	48	842	2.9	421	26
J	5/21/94	7/22/94	399	3,248	50	1,191	2.4	433	0
K	7/23/94	8/9/94	779	2,649	41	1,148	2.0	441	337
L	8/10/94	8/25/94	401	1,002	15	1,202	2.4	201	201
M	8/26/94	9/16/94	364	570	9	428	4.8	190	174
N	9/17/94	10/6/94	330	732	11	0	11.5	147	184
O	10/7/94	11/7/94	284	568	9	284	6.7	113	170
P	11/8/94	2/3/95	382	3,228	49	807	3.2	161	220
Q	2/4/95	3/3/95	296	3,012	46	753	3.6	0	296
R	3/4/95	3/10/95	416	?	?	?	?	0	416
S	3/11/95	4/6/95	406	3,650	56	913	2.9	0	406
T	4/7/95	5/11/95	412	3,675	56	1,103	2.5	0	412
U	5/12/95	6/7/95	408	2,719	42	1,359	2.1	340	68
V	6/8/95	6/21/95	351	2,190	33	1,095	2.6	273	77
W	6/22/95	6/28/95	541	3,667	56	1,833	1.6	454	83

[a] HLR = Hydraulic Loading Rate to top of RSF (Based on total amount of effluent applied to top 6.54 m^2 area of RSF).

[b] HRT = Hydraulic Residence Time within Septic Tank (Liquid Storage Volume (3,785 L) / Total Flow into Septic Tank).
Total Flow into Septic Tank = (Forward Flow + Flow to Septic Tank).

Table 4.5 Flow ratios for the average hydraulic conditions for all study periods after the installation of the "dam" at Site #1.

Study Period	Start	End	Forward Flow (Q) (liters/day)	Sand Filter Ratio (SFR)	Septic Tank Ratio (STR)	Drainfield Ratio (Q*)	Bypass Ratio (BPR)	Forward Flow (Q)
#2	6/30/93	8/31/93	385.0	4.0	2.3	1.0	0.0	1.0
A	9/1/93	9/16/93	428.1	3.8	2.3	1.0	0.0	1.0
B	9/17/93	9/23/93	430.2	2.9	1.8	0.8	0.2	1.0
C	9/24/93	10/11/93	372.5	4.0	2.5	0.5	0.5	1.0
D	10/12/93	10/26/93	347.0	0.0	0.0	0.0	1.0	1.0
E	10/27/93	12/23/93	389.3	8.7	5.5	1.2	0.0	1.0
F	12/24/93	1/12/94	447.2	6.3	2.3	0.8	0.2	1.0
G	1/13/94	2/4/94	396.3	8.1	2.2	1.0	0.0	1.0
H	2/5/94	2/11/94	486.8	3.5	0.9	0.5	0.5	1.0
I	2/12/94	5/20/94	447.4	7.1	1.9	0.9	0.1	1.0
J	5/21/94	7/22/94	399.0	8.1	3.0	1.1	0.0	1.0
K	7/23/94	8/9/94	778.7	3.4	1.5	0.6	0.4	1.0
L	8/10/94	8/25/94	401.3	2.5	3.0	0.5	0.5	1.0
M	8/26/94	9/16/94	364.0	1.6	1.2	0.5	0.5	1.0
N	9/17/94	10/6/94	330.5	2.2	0.0	0.4	0.6	1.0
O	10/7/94	11/7/94	284.3	2.0	1.0	0.4	0.6	1.0
P	11/8/94	2/3/95	381.6	8.5	2.1	0.4	0.6	1.0
Q	2/4/95	3/3/95	295.6	10.2	2.5	0.0	1.0	1.0
R	3/4/95	3/10/95	416.4	?	?	?	1.0	1.0
S	3/11/95	4/6/95	406.2	9.0	2.2	0.0	1.0	1.0
T	4/7/95	5/11/95	411.9	8.9	2.7	0.0	1.0	1.0
U	5/12/95	6/7/95	408.4	6.7	3.3	0.8	0.2	1.0
V	6/8/95	6/21/95	351.3	6.2	3.1	0.8	0.2	1.0
W	6/22/95	6/28/95	540.8	6.8	3.4	0.8	0.2	1.0

Flow Ratios: Ratio of (Flow or top of RSF, to septic tank, to drainfield, bypassing system) to the Forward Flow (Q)

Flow Ratios are based on Forward Flow (Q) rather than Q* which is the amount of effluent pumped to the drainfield from the RSF. The difference between Q and Q* is the amount bypassing the RSF system and going directly to the drainfield.

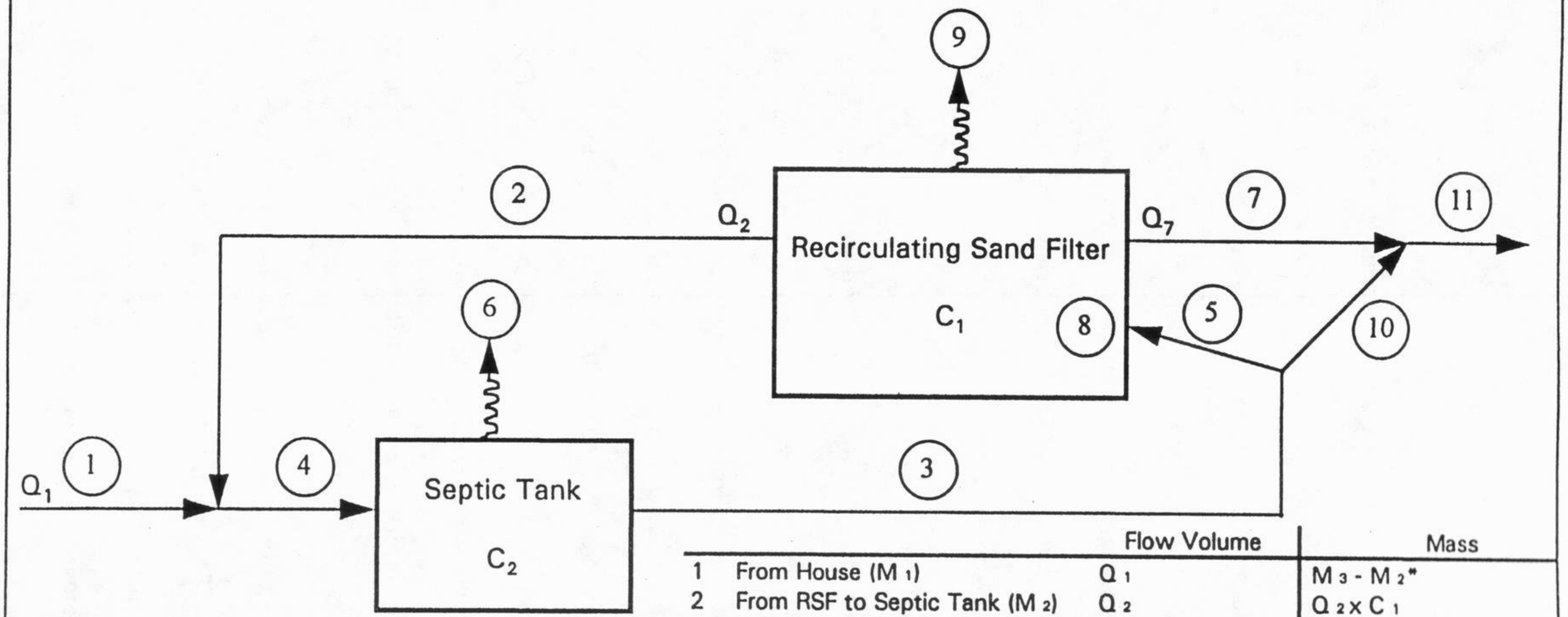
4.13 Mass Balance Calculations

Mass balances were produced for 19 of the 24 study periods at Site #1. Mass balance calculations could not be compiled for six study periods, #1, D, Q, R, S, and T, because the flow conditions used in the mass balances could not be determined. These study periods will be discussed separately from the 19 used in the determination of the optimal operating conditions for the system at Site #1. Mass loadings and removals for several water quality characteristics for these 19 study periods and are included in the appendices.

Assumptions

Mass loadings and removals throughout the system were calculated using the chemical data from the samples collected [from the pump chamber of the RSF (C_1) and the septic tank (C_2)], and the flows from the household water meter (Q_1), the flow recycled back to the septic tank (Q_2), and the flow pumped to the drainfield (Q_7). There are three known values obtained from these measurements which become the basis for the mass balance calculations: the mass from the RSF to the septic tank (M_2), the mass in the septic tank outflow (M_3), and the mass pumped from the RSF to the drainfield (M_7). A schematic for the mass balance calculations can be found in Figure 4.10.

No samples could be collected from the point of wastewater entry to the system after the RSF was installed. Therefore, the actual concentrations of chemical species in the household waste was unknown. The average concentrations in the septic tank before RSF installation could be used to represent



		Flow Volume	Mass
1	From House (M_1)	Q_1	$M_3 - M_2$*
2	From RSF to Septic Tank (M_2)	Q_2	$Q_2 \times C_1$
3	Septic Tank Outflow (M_3)	$Q_1 + Q_2$	$(Q_1 + Q_2) \times C_2$
4	Total to Septic Tank (M_4)	$Q_1 + Q_2$	$M_1 + M_2$
5	From Septic Tank to RSF (M_5)	$Q_3 - (Q_1 - Q_7)$	$(Q_3 - (Q_1 - Q_7)) \times C_2$
6	Removed in Septic Tank (M_6)	0	$M_4 - M_3$
7	From RSF to Drainfield (M_7)	Q_7	$Q_7 \times C_1$
8	Portion of Mass from House Entering Septic Tank (M_8)	$Q_1 - Q_{10}$	$M_1 - M_{10}$
9	Removed in Sand Filter (M_9)	0	$M_5 - (M_2 + M_7)$
10	Bypassing System (M_{10})	$Q_1 - Q_7$	$(Q_1 - Q_7) \times C_2$
11	Total to Drainfield (M_{11})	$Q_7 + Q_{10}$	$M_7 + M_{10}$

Measured Concentrations	Measured Flows
C_1 Sand Filter	Q_1 Household Water Meter
C_2 Septic Tank	Q_2 Recycle to Septic Tank
	Q_7 Pumped to Drainfield

* Total N includes Kjeldhal N only.

Figure 4.10 Schematic for the mass balance calculations at site #1 the household conditions, but these were highly variable over time, therefore the household concentration was estimated as follows.

An assumption was made in order to estimate the concentrations of chemical species in the household waste since there was no sample point between the household and the septic tank. The concentrations in the septic tank are a combination of household waste and recirculated flow from the sand filter. The assumption made was that no transformations or losses of chemicals occur in the household waste as it passes through the septic tank. Therefore, the mass of chemical coming from the house is equal to the mass flowing from the septic tank to the sand filter less the mass pumped to the septic tank from the sand filter.

A further assumption was made for total nitrogen (TN). Household waste is primarily NH_4^+-N and organic-N; there is no nitrate-N in household waste because anaerobic conditions exist and none was detected in samples collected from 1991 to 1992. Total kjeldahl nitrogen (TKN) consists of NH_4^+-N and organic-N; therefore, the TN in the household waste is *the mass of TKN in the septic tank less the mass of TKN in the septic tank from the sand filter*.

The use of these assumptions in the mass balance calculations indicates that the removal of chemical mass, with the exception of TN, occurs only in the sand filter. Accordingly, no chemicals are removed in the septic tank except TN as nitrate-N (through denitrification) since it was assumed that no nitrate-N exists in household waste. The implication that chemical mass is not removed in the septic tank may be a limitation in the interpretation of these results based on mass and percent removal, but not for TN removal which was the primary focus of this study.

Due to the problems encountered with bypass of the sand filter and the lack of a sample point between the "dam" and the soil absorption system, the total mass to the drainfield cannot be accurately determined during most study periods at Site #1. Since there was no sample point, the mass bypassing the RSF was based on the concentrations in the septic tank and the flow assumed to be bypassing the sand filter. The total mass to the drainfield is the mass pumped from the pump chamber of the sand filter and the mass bypassing. Therefore, during study periods with bypass the calculated total mass loading to the drainfield is the best estimate that can be made. During study periods #2, A, E, G, and J no bypass occurred, as a result the mass applied to the drainfield is known and the total loading to the drainfield is an accurate estimate for these study periods.

Due to mass bypassing the system, the mass removed by the RSF system was calculated in two ways: 1) based on the mass applied to the RSF to compare the treatment efficiencies resulting from the changes made in the recirculation rates and 2) based on the total mass applied to the drainfiled to determine the loading to groundwater.

4.14 BOD$_5$, COD, and Total Phosphorous

The concentrations of BOD$_5$, COD, and total phosphorous (TP) in the septic tank and pump chamber of the RSF from 1992 to 1995 at Site #1 are included in Figures 4.11, 4.12, and 4.13. Samples for these characteristics were collected for 15 of the 19 study periods used to evaluate the various levels of treatment achieved at this site. Samples were not available for study periods C, G, H, and W.

Table 4.6 shows BOD$_5$ removal for the RSF system based on the mass applied to the RSF and also based on the total mass being applied to the drainfield accounting for system bypass. The estimated mass of BOD$_5$ from the house was calculated using the mass balance assumptions discussed earlier. Using these assumptions, 28.4 to 385 grams/day (0.063 to 0.849 lbs/day) of BOD$_5$ were generated in the household. BOD$_5$ removal ranged from 80.8% to 99.0% based on the mass applied to the RSF for the various flow conditions used. BOD$_5$ removal ranged from 40.9% to 98.2% when including system bypass.

The estimated average concentration of BOD$_5$ in the household waste was 293.1 mg/l (range 63.5 to 1238 mg/l). The average BOD$_5$ measured in the septic tank before the system was installed was 270.6 mg/l (range 203 to 337 mg/l). The average measured BOD$_5$ in the septic tank effluent after the system was installed was 114.3 mg/l (range 20.0 to 630.0 mg/l). The average measured BOD$_5$ in the sand filter effluent applied to the drainfield was 13.3 mg/l (range 2.8 to 34.0 mg/l).

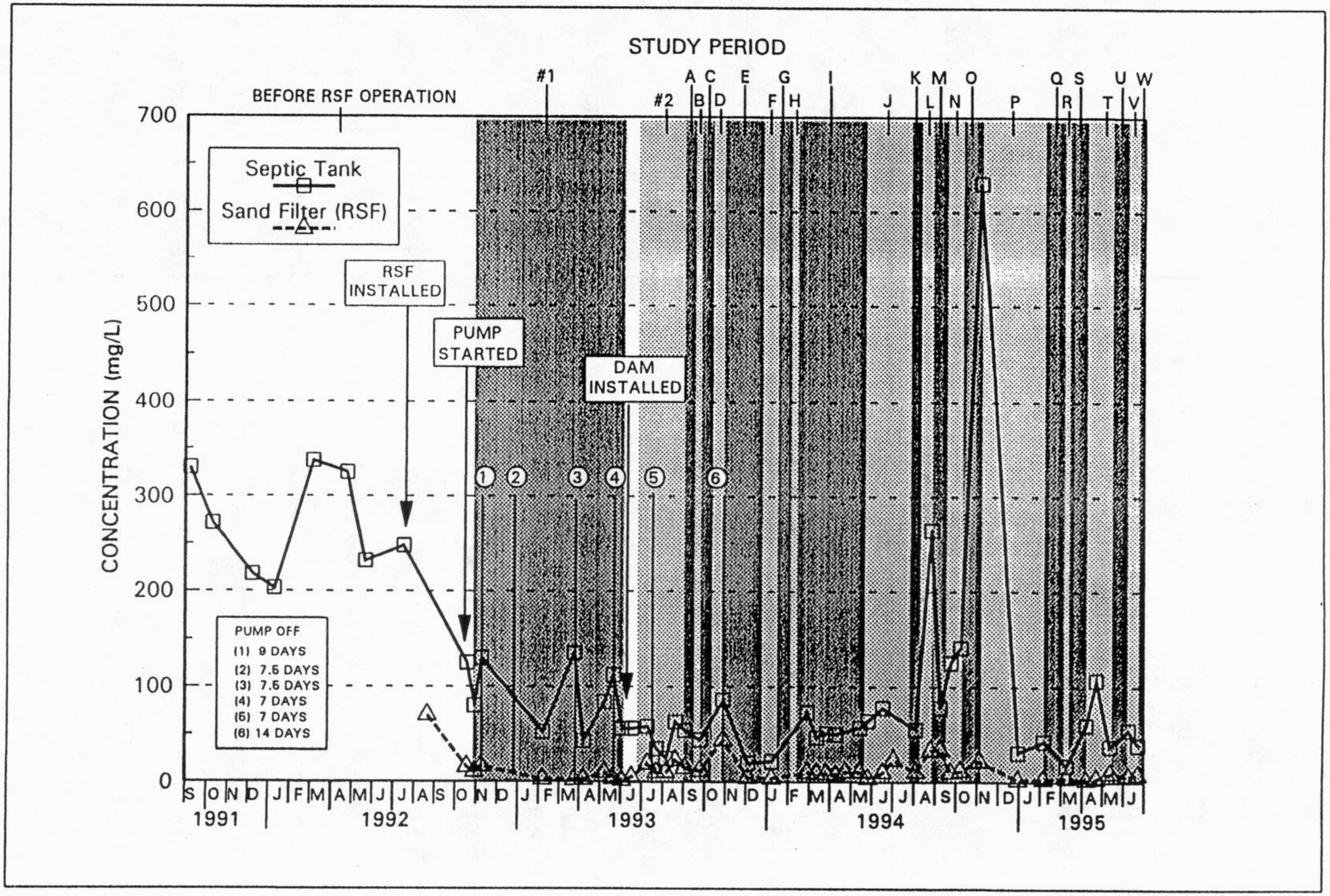


Figure 4.11 BOD$_5$ concentrations in the septic tank and pump chamber of the RSF at site #1 from 1991 to 1995.

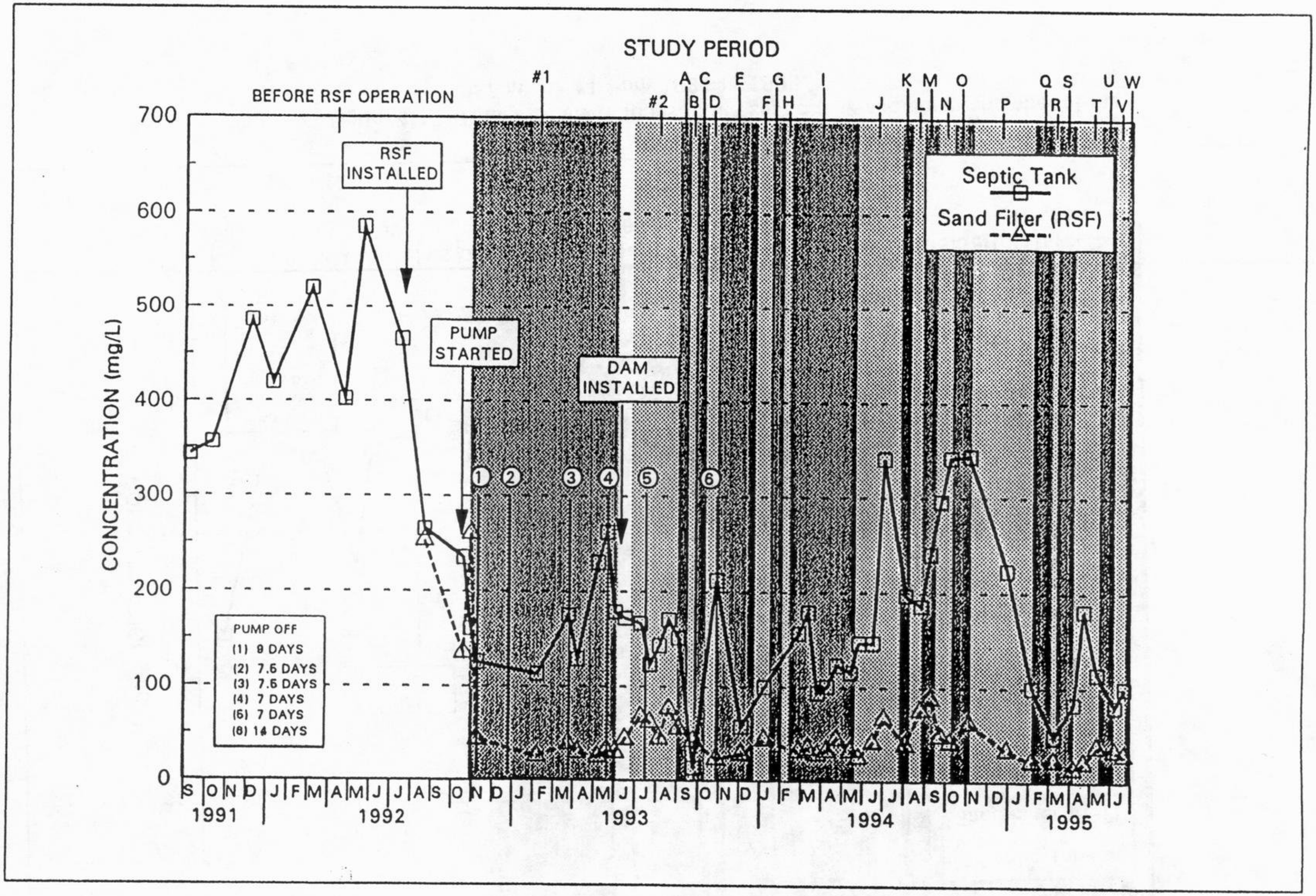


Figure 4.12 COD concentrations in the septic tank and pump chamber of the RSF at site #1 from 1991 to 1995.

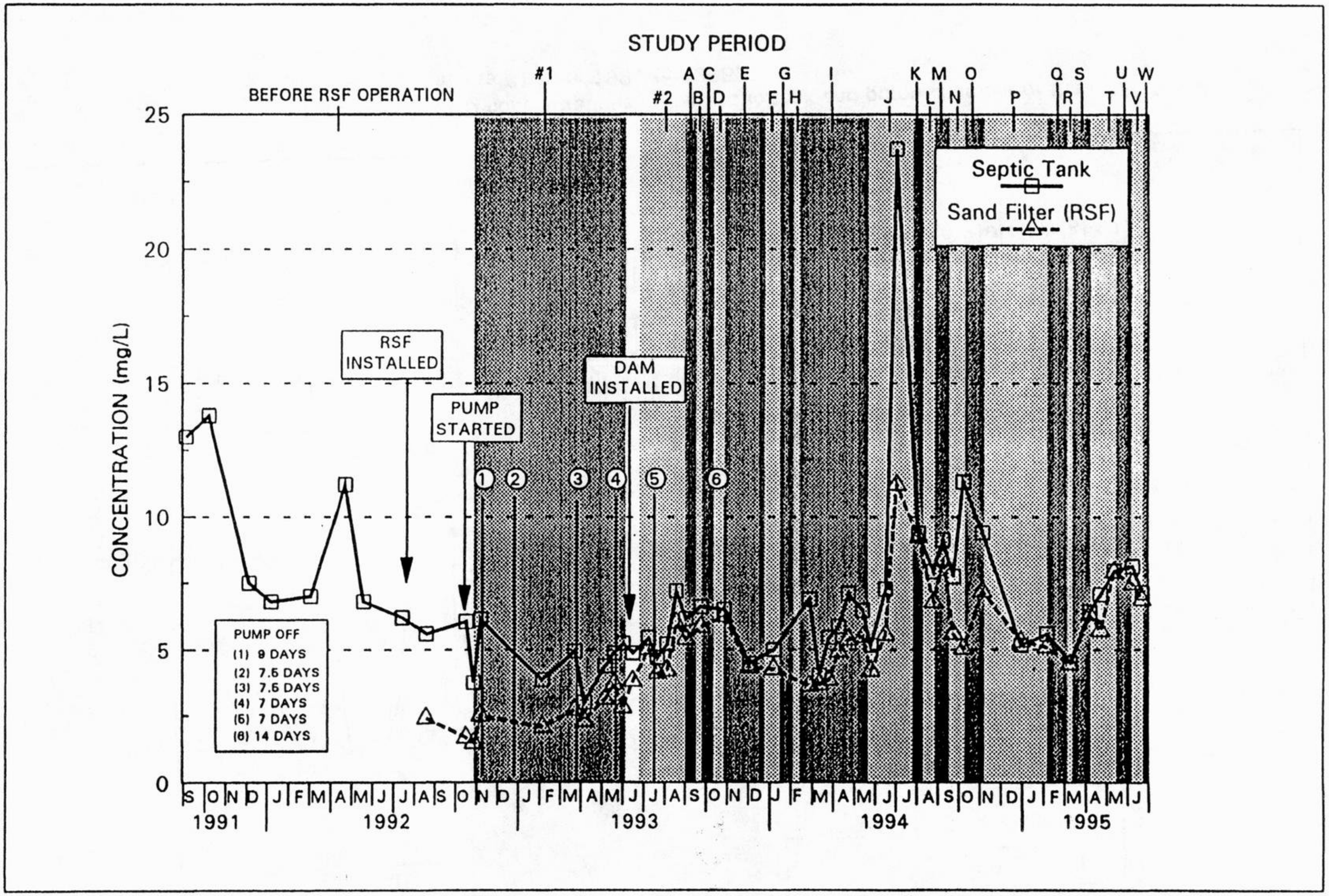


Figure 4.13 Total P concentrations in the septic tank and pump chamber of the RSF at site #1 from 1991 to 1995.

Table 4.6 BOD_5 removal by the RSF system at Site #1. (Exclude study periods #1, D, Q, R, S, and T)

Study Period	Start	End	Mass From House (grams/day)	Mass From House Entering RSF (grams/day)	Mass Pumped from RSF to Drainfield (1) (grams/day)	Percent Removed (1)	Total Mass to Drainfield(2) (grams/day)	Percent Removed (2)
#2	6/30/93	8/31/93	45.3	45.3	6.0	86.8	6.0	86.8
A	9/1/93	9/16/93	60.9	60.9	6.5	89.4	6.5	89.4
B	9/17/93	9/23/93	44.8	40.5	3.9	90.4	8.2	81.6
C	9/24/93	10/11/93	NS	NS	NS	NS	NS	NS
E	10/27/93	12/23/93	40.2	40.2	1.9	95.4	1.9	95.4
F	12/24/93	1/12/94	28.4	26.8	1.5	94.4	3.1	89.1
G	1/13/94	2/4/94	NS	NS	NS	NS	NS	NS
H	2/5/94	2/11/94	NS	NS	NS	NS	NS	NS
I	2/12/94	5/20/94	63.5	62.1	4.1	93.4	5.5	91.3
J	5/21/94	7/22/94	272.9	272.9	5.0	98.2	5.0	98.2
K	7/23/94	8/9/94	93.6	75.0	4.8	93.6	23.4	75.0
L	8/10/94	8/25/94	385.1	332.1	6.6	98.0	59.6	84.5
M	8/26/94	9/16/94	47.2	33.7	6.5	80.8	20.0	57.6
N	9/17/94	10/6/94	43.9	19.6	1.6	91.7	26.0	40.9
O	10/7/94	11/7/94	351.6	243.9	2.5	99.0	110.2	68.7
P	11/8/94	2/3/95	41.5	33.4	0.5	98.6	8.6	79.3
U	5/12/95	6/7/95	87.3	83.6	2.2	97.3	6.0	93.2
V	6/8/95	6/21/95	47.6	44.6	2.2	95.1	5.2	89.0
W	6/22/95	6/28/95	NS	NS	NS	NS	NS	NS

(1) Removal based on mass applied to the RSF, does not account for mass bypassing the system.
(2) Removal based on total mass applied to the drainfield, accounts for system bypass.
NS No Sample

Table 4.7 shows the COD removal for the RSF system based on the mass applied to the RSF and also the total mass applied to the drainfield accounting for system bypass. It should be noted that the results for study period B are misleading because the COD concentrations measured in the septic tank (13.8 mg/l) were less than in the sand filter (42.5 mg/l).

The estimated mass of COD from the house was calculated using the mass balance assumptions discussed earlier. Using these assumptions, 84.6 to 334 grams/day (0.187 to 0.736 lbs/day) of COD was generated in the household. COD removal ranged from 82.7 to 96.7% based on the mass applied to the RSF for the various flow conditions used. COD removal ranged from 38.8% to 93.7% when including system bypass.

The estimated average COD concentration in the household waste was 364 mg/l (range 217.5 to 707 mg/l). The average concentration of COD in the septic tank before the system was installed was 447.5 mg/l (range 344 to 585 mg/l). The average measured COD in the septic tank was 158 mg/l (range 13.8 to 344 mg/l). The average COD measured in the sand filter effluent directed to the drainfield was 47.3 mg/l (range 27.7 to 88.2 mg/l).

Table 4.8 shows the removal of TP based on the mass applied to the RSF and the total mass applied to the drainfield accounting for system bypass. The estimated TP generated in the household ranged from 2.08 to 10.83 grams/day (0.005 to 0.024 lbs/day). TP removal ranged from 16.9 to 74.3%, based on the mass applied to the RSF for the various flow conditions used. TP removals ranged from 8.3% to 74.3% when including system bypass.

Table 4.7 COD removal by the RSF system at Site #1. (Excludes study periods #1, D, Q, R, S, and T).

Study Period	Start	End	Mass From House (grams/day)	Mass From House Entering RSF (grams/day)	Mass Pumped from RSF to Drainfield (1) (grams/day)	Percent Removed (1)	Total Mass to Drainfield (2) (grams/day)	Percent Removed(2)
#2	6/30/93	8/31/93	134.5	134.5	23.6	82.4	23.6	82.4
A	9/1/93	9/16/93	157.0	157.0	23.7	84.9	23.7	84.9
B	9/17/93	9/23/93	-16.7	-18.1	14.1	178.0	15.5	192.4
C	9/24/93	10/11/93	NS	NS	NS	NS	NS	NS
E	10/27/93	12/23/93	84.6	84.6	11.1	86.8	11.1	86.8
F	12/24/93	1/12/94	102.4	95.1	16.5	82.7	23.8	76.8
G	1/13/94	2/4/94	NS	NS	NS	NS	NS	NS
H	2/5/94	2/11/94	NS	NS	NS	NS	NS	NS
I	2/12/94	5/20/94	136.3	133.0	14.5	89.1	17.8	86.9
J	5/21/94	7/22/94	282.4	282.4	17.8	93.7	17.8	93.7
K	7/23/94	8/9/94	334.4	267.8	17.4	93.5	84.0	74.9
L	8/10/94	8/25/94	207.2	170.0	15.2	91.0	52.4	74.7
M	8/26/94	9/16/94	152.3	110.6	16.8	84.8	58.5	61.6
N	9/17/94	10/6/94	105.3	46.9	6.6	86.0	65.0	38.3
O	10/7/94	11/7/94	178.0	119.2	6.9	94.2	65.7	63.1
P	11/8/94	2/3/95	169.7	134.0	4.5	96.7	40.2	76.3
U	5/12/95	6/7/95	96.0	90.6	11.4	87.4	16.8	82.5
V	6/8/95	6/21/95	157.0	157.0	23.7	84.9	23.7	84.9
W	6/22/95	6/28/95	NS	NS	NS	NS	NS	NS

(1) Removal based on mass applied to the RSF, does not account for mass bypassing the system.
(2) Removal based on total mass applied to the drainfield, accounts for system bypass.
NS No Sample

Table 4.8 Total Phosphorous removal by the RSF system at Site #1. (Excludes study periods #1, D, Q, R, S, and T).

Study Period	Start	End	Mass From House (grams/day)	Mass From House Entering RSF (grams/day)	Mass Pumped from RSF to Drainfield (1) (grams/day)	Percent Removed (1)	Total Mass to Drainfield (2) (grams/day)	Percent Removed(2)
#2	6/30/93	8/31/93	2.88	2.88	1.88	34.6	1.88	34.6
A	9/1/93	9/16/93	3.43	3.43	2.32	32.3	2.32	32.3
B	9/17/93	9/23/93	3.43	2.78	1.94	30.2	2.59	24.5
C	9/24/93	10/11/93	NS	NS	NS	NS	NS	NS
E	10/27/93	12/23/93	2.08	2.08	1.68	19.4	1.68	19.4
F	12/24/93	1/12/94	3.01	2.64	1.59	39.8	1.95	35.0
G	1/13/94	2/4/94	NS	NS	NS	NS	NS	NS
H	2/5/94	2/11/94	NS	NS	NS	NS	NS	NS
I	2/12/94	5/20/94	3.91	3.76	1.89	49.7	2.05	47.7
J	5/21/94	7/22/94	10.83	10.83	2.79	74.3	2.79	74.3
K	7/23/94	8/9/94	7.47	4.30	4.07	5.5	7.23	3.2
L	8/10/94	8/25/94	4.55	2.96	1.36	54.0	2.95	35.2
M	8/26/94	9/16/94	3.64	2.05	1.59	22.6	3.18	12.7
N	9/17/94	10/6/94	3.14	1.40	0.78	44.1	3.52	19.7
O	10/7/94	11/7/94	3.29	1.69	0.81	51.8	2.42	26.5
P	11/8/94	2/3/95	2.21	1.02	0.84	18.0	2.03	8.3
U	5/12/95	6/7/95	4.21	3.66	2.54	30.6	3.09	26.6
V	6/8/95	6/21/95	2.82	2.26	1.88	16.9	2.44	13.6
W	6/22/95	6/28/95	NS	NS	NS	NS	NS	NS

(1) Removal based on mass applied to the RSF, does not account for mass bypassing the system.

2) Removal based on total mass applied to the drainfield, accounts for system bypass.

NS No Sample

The estimated average concentration of TP in the household waste was 9.80 mg/l (range 5.36 to 27.15 mg/l). The average TP concentration in the septic tank before the system was installed was 9.00 mg/l (range 6.20 to 13.80 mg/l). The average TP concentration measured in the septic tank effluent was 7.46 mg/l (range 4.48 to 12.05 mg/l). The average TP concentration in the sand filter effluent applied to the drainfield was 6.18 mg/l (range 4.25 to 9.22 mg/l).

Two mechanisms are likely responsible for the removal of TP in the RSF system, adsorption and precipitation of free-phosphate (PO_4^{-3}). Samples collected were analyzed only for TP, no analyses were performed to distinguish organic-P from PO_4^{-3}. The adsorption of PO_4^{-3} in the sand filter is probably influenced by the HLR and the adsorptive capacity of the media in the RSF. The precipitation of PO_4^{-3} in the RSF is probably influenced by pH and the presence of iron, calcium, or organic solids in the RSF.

Due to the variability of TP loading to the system from the household, the different HLRs used, and the different pHs observed, it is difficult to determine which process is more important in controlling TP removal by the RSF system. If TP removal is desired, different sand filter media should be selected to achieve better removal than were found in this study.

4.15 Nitrogen

4.151 Nitrification

Chemical data from samples collected on July 1, 1994 from the lysimeter in the unsaturated sand layer of the RSF, the monitoring well screening the saturated rock storage unit of the RSF, the pump chamber of the RSF, and the septic tank at Site #1 are shown in Table 4.9. These chemical data indicates that the concentrations of chemical species are not uniform within the sand filter. While the lysimeter shows a nitrate-N production of 27.0 mg/l, the monitoring well and pump chamber show nitrate-N concentrations of <0.2 and 8.4 mg/l, respectively. The decrease in concentrations between the unsaturated and saturated bottom portions of the RSF imply denitrification in the bottom portion of the RSF. Therefore, it is impossible to estimate the amount of nitrate-N produced in the RSF using nitrate-N concentrations measured in the pump chamber of the RSF.

Table 4.9 Concentrations of water quality characteristics in the sand filter monitoring well, lysimiter, pump chamber, and the septic tank at Site #1 as measured on 7/1/94.

Location	Nitrate-N mg/l	NH_4^+-N mg/l	TKN mg/l	BOD_5 mg/l	pH Std Units
RSF Lysimeter	27.0	9.5	10.5	-	-
RSF Well	<0.2	33.3	42.2	>120	7.26
Pump Chamber	8.4	6.5	9.0	23.5	6.93
Septic Tank	<0.2	50	61.9	>400	6.84

Table 4.10 illustrates the changes in ammonia (NH_3-N) and nitrate-N in single pass (non-recirculating) intermittent sand filters (ISFs) for a variety of hydraulic loading rates as documented in previous studies by Ronayne et al. (1982) and Cagle

and Johnson (1994). As undiluted, unrecirculated septic tank effluent (STE) is applied to the ISFs, NH_3-N concentrations are reduced by nitrification and/or volatilization. In a single pass ISFs reduce the NH_3-N concentrations by 90 to 99 %. Assuming that no volatilization occurs, the reduction in NH_3-N concentrations is due entirely to nitrification and results in the production of 40.1 and 43.2 mg/l of nitrate-N. Nitrate-N concentrations in the effluent from ISFs are lower than the concentrations produced, implying approximately 28% denitrification by ISFs. Ball (1994) reports that ISFs of this design are capable of up to 50% denitrification, depending upon temperature.

Table 4.10 Changes in NH_3-N and Nitrate-N concentrations from single pass intermittent sand filter systems [adapted from Ronayne et al. (1982), and Cagle and Johnson (1994)].

Study	STE NH_3-N (mg/l)	ISFE NH_3-N (mg/l)	%Reduction NH_3-N (mg/l)	Nitrate-N Produced (mg/l)	ISFE Nitrate-N (mg/l)	Percent Denitrification
1.	40.6	0.3	99	40.1	29.1	28
2.	47.8	4.6	90	43.2	31.1	28

1. Ronayne et al. (1982)
2. Cagle and Johnson (1994)

STE Septic tank effluent
ISFE Intermittent sand filter effluent

The reductions of NH_4^+-N in an RSF system with multiple passes should be similar to the reductions observed in a single pass ISF. If compete conversion of NH_4^+-N to nitrate-N in the RSF system occurred, no NH_4^+-N would be detected in the sand filter; the conversion in the sand filter was not complete since average NH_4^+-N concentrations between 2.5 and 22.6 mg/l were detected in the pump chamber of the RSF and average NH_4^+-N concentrations detected in the septic tank ranged between 7.5 and 53.2 mg/l. When using the septic tank concentrations, the conversion of NH_4^+-N to nitrate-N is 57 to 67% complete; only half of the reduction seen in ISFs. However, septic tank concentrations are diluted by reduced concentrations being recycled from the RSF. The undiluted, unrecirculated influent household NH_4^+-N concentrations estimated from the mass balance calculations should be used to compare the reductions of single pass ISFs and the RSF system.

Table 4.11 shows the theoretical changes in NH_4^+-N and nitrate-N concentrations in the RSF system at Site #1. The conversion of NH_4^+-N to nitrate-N is estimated from the influent household NH_4^+-N concentrations and the NH_4^+-N concentrations measured in the pump chamber of the RSF. A range of 25.0 to 87.7 mg/l of nitrate-N were produced in the sand filter, a theoretical NH_4^+-N conversion of 56 to 96%, with hydraulic loading rates (HLRs) between 9 and 56 cm/day. NH_4^+-N removal by volatilization is not likely to be significant at the near neutral pH values seen in the sand filter (6.56 to 7.68 Std. Units) at Site #1 despite the high degree of air to water contact provided by pumping effluent to the top of the sand filter. Therefore, the reduction in NH_4^+-N is due entirely to nitrification in the RSF.

Nitrate-N concentrations in the pump chamber of the RSF are lower than the concentrations estimated to be produced implying 38 to 96% denitrification in the RSF. Denitrification in the RSF system at Site #1 is greater than the denitrification seen in ISFs due to differences in the designs of the two systems. The design of the RSF system includes a saturated rock storage unit for denitrification, whereas ISFs are designed to drain completely.

Table 4.11 Theoretical changes in NH_4^+-N and Nitrate-N concentrations in the RSF for the study periods used in the determination of effluent treatment by the RSF system at Site #1. (Excludes study periods #1, D, Q, R, S, and T).

Study Period	HLR (cm/day)	Sand Filter Ratio (SFR)	Estimated in Household Waste		Average Measured in RSF			Theoretical		
			TKN (mg/l)	NH_4^+-N (mg/l)	TKN (mg/l)	NH_4^+-N (mg/l)	Nitrate-N (mg/l)	Percent Conversion NH_4^+-N * (%)	Nitrate-N Produced (mg/l)	Percent Denitrification Nitrate-N (%)
#2	23	4.0	64.6	42.5	13.3	10.4	8.5	76	32.1	74
A	25	3.8	57.5	48.2	11.2	9.0	8.2	81	39.2	79
B	19	2.9	71.7	64.7	13.5	11.9	10.9	82	52.8	79
C	23	4.0	112.7	85.8	9.3	8.9	3.5	90	77.0	96
E	52	8.7	70.0	35.0	5.7	2.5	11.4	93	32.5	65
F	43	6.3	67.1	53.1	5.6	4.2	18.7	92	48.9	62
G	49	8.1	88.5	56.2	5.4	4.5	14.8	92	51.7	71
H	26	3.5	86.5	65.8	7.6	6.0	13.5	91	59.8	77
I	48	7.1	86.2	45.1	7.2	5.3	12.5	88	39.8	69
J	50	8.1	110.6	93.3	6.6	5.6	11.4	94	87.7	87
K	40	3.4	55.1	45.3	7.6	6.5	12.2	86	38.8	69
L	15	2.5	58.6	52.3	14.2	13.5	4.4	74	38.8	89
M	9	1.6	76.9	62.6	26.2	22.6	5.8	64	40.0	86
N	11	2.2	67.5	53.2	13.8	10.4	20.5	81	42.8	52
O	9	2.0	109.6	44.7	26.4	19.7	15.5	56	25.0	38
P	49	8.5	84.2	68.1	4.5	3.0	16.2	96	65.0	75
U	42	6.7	82.7	67.3	6.5	4.9	10.9	93	62.4	83
V	34	6.2	66.7	60.2	7.0	4.6	8.7	92	55.6	84
W	56	6.8	66.7	42.3	5.7	5.4	7.0	87	36.9	81

* Theoretical conversion of NH_4^+-N to Nitrate-N. Does not account for NH_4^+-N volatilization in the RSF.

Figure 4.14 and Figure 4.15 show the relationships of the hydraulic loading rate (HLR) and sand filter ratio (SFR) to the theoretical conversion of NH_4^+-N in the RSF. In general, a greater percentage of NH_4^+-N is converted at higher hydraulic loading rates and sand filter ratios. However, nitrification in the RSF is also dependent on temperature and alkalinity levels.

The reported minimum temperature for nitrification to occur is 5°C (Shammas, 1986). Temperature and nitrate-N concentrations measured in the pump chamber of the RSF are shown in Figure 4.16. The lowest temperature observed in the sand filter was 2.5°C on (February 4, 1994) and is below the minimum temperature required for nitrification; however, on this date, 13.1 mg/l of nitrate-N were measured in the sand filter. Temperature readings in the RSF were also below 6°C between January and March of 1993, 1994, and 1995, and during these three winters nitrate-N levels in the RSF again remained above 10 mg/l despite the low temperatures.

Nitrate-N levels below 10 mg/l were detected during pump failure periods (5) and (6) and also during fall of 1993 and the summer of 1994 even though temperatures which were favorable for nitrification. These concentrations are likely the result of denitrification in the RSF, or lower SFRs.

The variations in the nitrate-N concentrations measured in the RSF make it difficult to determine the effect of temperature on nitrification. However, despite temperatures below the minimum required for nitrification, nitrate-N concentrations detected in the sand filter were greater than 10 mg/l, indicating that some degree of nitrification still occurred.

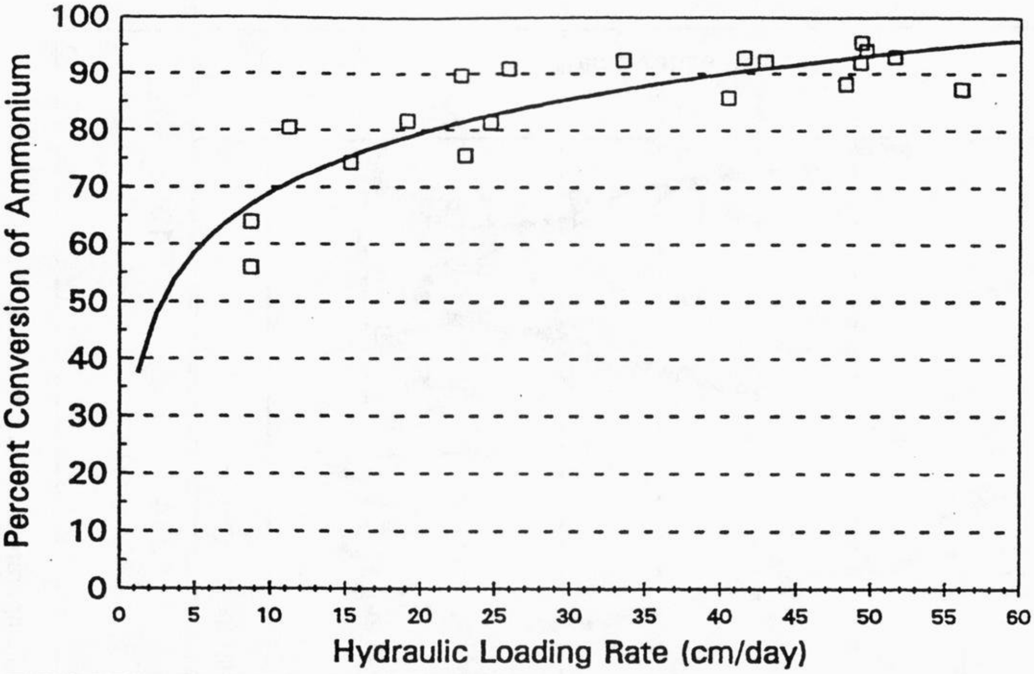


Figure 4.14 Percent conversion of ammonium to nitrate-N in the sand filter vs hydraulic loading rate at Site #1.

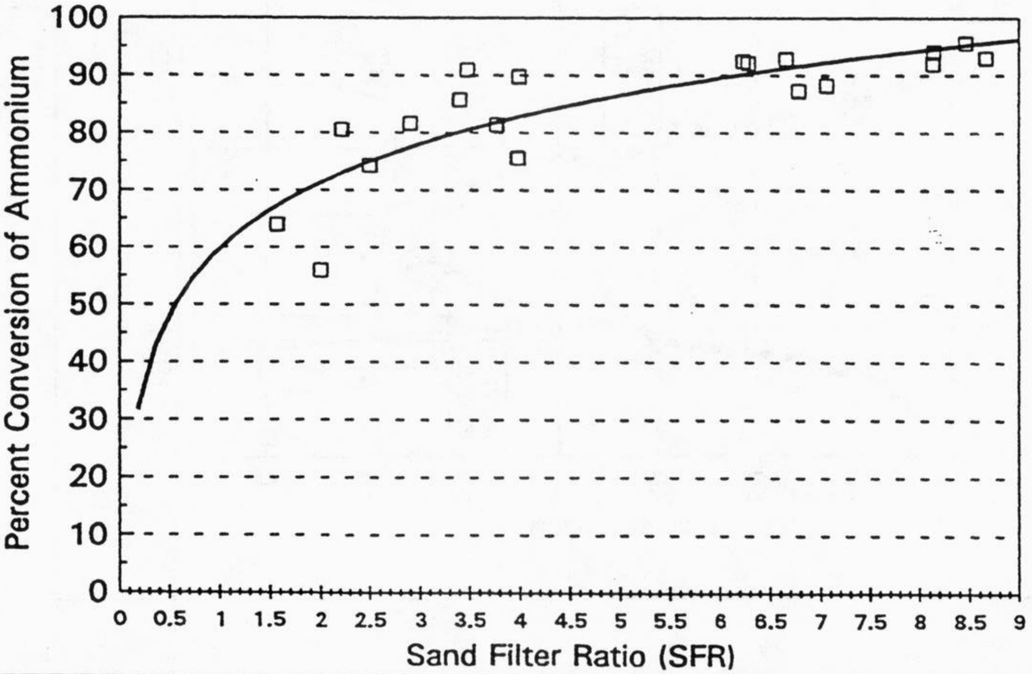


Figure 4.15 Percent conversion of ammonium to nitrate-N in the sand filter vs sand filter ratio (SFR) at Site #1.

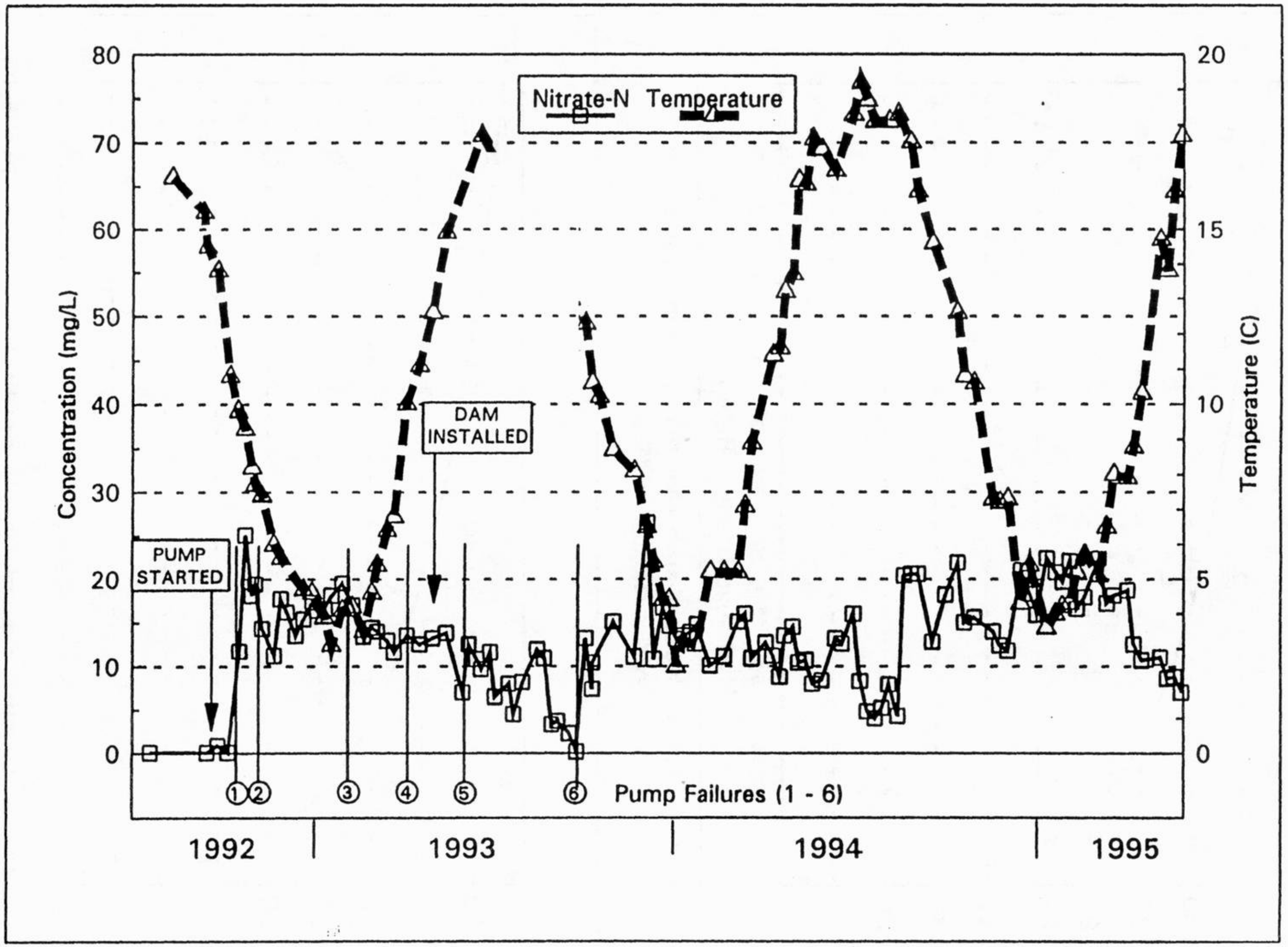


Figure 4.16 Nitrate-N concentrations and Temperature in the pump chamber of the RSF at Site #1 from 1992-1995.

The minimum alkalinity level required for nitrification is 40 mg/l as $CaCO_3$ (Sandy, 1987). Figure 4.17 shows the alkalinity and total hardness concentrations in the septic tank and pump chamber of the RSF from 1992 to 1995. Alkalinity in the pump chamber of the RSF has remained above 100 mg/l ($CaCO_3$) since the system began operating. The limestone within the rock storage unit of the RSF continues to provide an adequate supply of alkalinity for nitrification in the RSF. The average alkalinity and total hardness concentrations in the septic tank before the system was installed were 381.4 mg/l (range 340 to 432 mg/l) and 154.5 mg/l (range 124 to 180 mg/l), respectively.

It is impossible to determine the amount of alkalinity destroyed by nitrification in the RSF because: 1) total hardness and alkalinity concentrations in the septic tank and sand filter are a mixture of treated effluent and household loading, and there is no way to differentiate between them, 2) alkalinity increases due to denitrification in the septic tank and/or sand filter cannot be accounted for, 3) organic solids in the effluent samples buffer the alkalinity endpoint, and 4) alkalinity consumption by sulfur reactions cannot be accounted for (no analyses were performed for sulfate or H_2S).

Alkalinity increases in the RSF due to denitrification cannot be accounted for since samples were not collected from the rock storage unit; therefore, it is impossible to construct a statistical model for nitrification. However, because some nitrification still occurred despite low temperatures, and because alkalinity levels were within the range of the microorganisms involved, it appears that the variation in the conversion of NH_4^+-N was due mostly to the changes made in recirculation

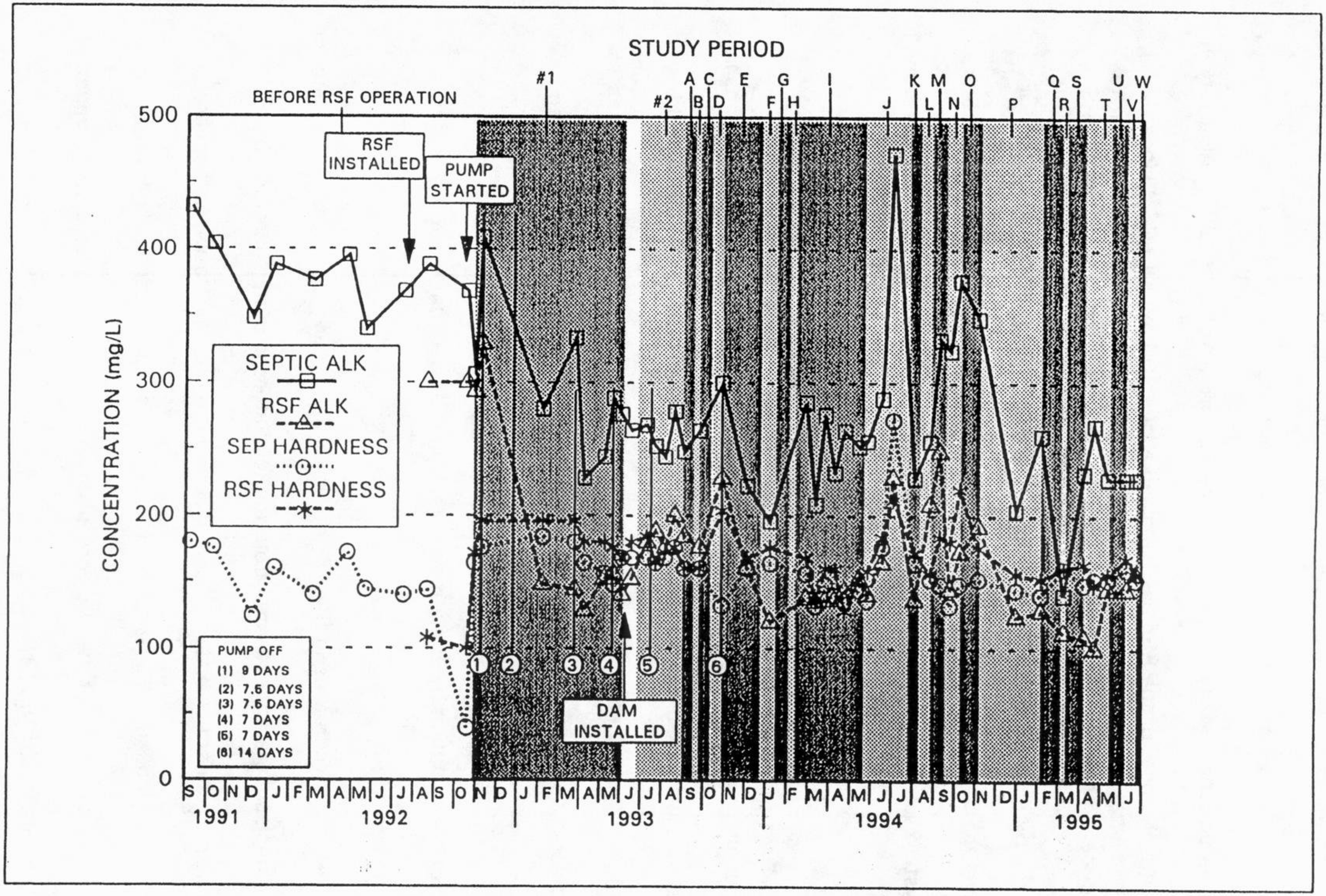


Figure 4.17 Alkalinity and Total Hardness in the septic tank and pump chamber of the RSF at Site #1 from 1991 to 1995.

rates to the top of the sand filter.

4.152 Denitrification

Table 4.12 shows the removal of nitrate-N in the septic tank by the denitrification process for the 19 study periods used in determining treatment efficiency. It is assumed that denitrification of the nitrate-N produced in the sand filter occurs prior to pumping; thus, the concentrations pumped to the septic tank are the concentrations measured in the pump chamber of the RSF. Average nitrate-N concentrations between 0.2 and 9.4 mg/l imply incomplete denitrification during some study periods. Denitrification is assumed to be 100% complete when nitrate-N concentrations are below the limit of quantitation (<0.2 mg/l). Denitrification in the septic tank did not occur during study period N when no flow was pumped from the RSF to the septic tank (STR = 0.0Q, HRT = 11.7 days).

Denitrification of the nitrate-N pumped from the sand filter to the septic tank ranged from essentially complete down to only half complete (49.9% during study period F) with STRs between 0.9 and 5.5Q and HRTs of 6.8 to 1.5 days. In theory, if the STR is too high, denitrification could be inhibited by the excessive loading of aerobic effluent recycled from the sand filter. Unfortunately, samples were not collected for DO, thus the STR at which aerobic conditions developed in the septic tank cannot be determined.

The highest average nitrate-N concentration in the septic tank was 9.4 mg/l (range 3.6 to 15.1 mg/l) during study period F (December 24, 1993 through January 12, 1994). This high nitrate-N level occurred with a STR of 2.3Q, which was not the highest ratio used during this study: STRs between 3.3 and 5.5Q also had detectable nitrate-N levels ranging between 3.3 and 4.2 mg/l. However, denitirification in the septic tank is also dependent on temperature, pH, and carbon availability.

Table 4.12 **Average removal of Nitrate-N in the septic tank by the denitrification process for the study periods used in the determination of effluent treatment at Site #1, (does not include study periods #1, D, Q, R, S, or T).**

Study Period	Septic Tank Ratio (STR)	HRT (Days)	$NO_3^- + NO_2^-$-N Sand Filter (mg/l)	$NO_3^- + NO_2^-$-N Septic Tank (mg/l)	Percent Denitrficication (%)
#2	2.3	2.5	8.5	0.2	97.6
A	2.3	2.8	8.2	0.4	94.7
B	1.8	3.2	10.9	0.7	93.6
C	2.5	3.0	3.5	<0.2	100.0
E	5.5	1.5	11.4	4.2	62.9
F	2.3	2.63	18.7	9.4	49.9
G	2.2	3.1	14.8	0.4	97.5
H	0.9	4.1	13.5	<0.2	100.0
I	1.9	3.0	12.5	2.0	83.9
J	3.0	2.4	11.4	0.5	95.4
K	1.5	2.0	12.2	<0.2	100.0
L	3.0	2.4	4.4	<0.2	100.0
M	1.2	4.9	5.8	0.2	97.1
N	0.0	11.7	20.5	<0.2	0
O	1.0	6.8	15.5	0.3	98.4
P	2.1	3.3	16.2	0.8	95.3
U	3.3	2.2	10.9	1.9	82.5
V	3.1	2.7	8.7	<0.2	100.0
W	3.4	1.6	7.0	0.2	97.1

NOTE Denitrification is assumed to be 100% complete for Nitrate-N <0.2 mg/l

The reported minimum temperature for denitrification in land treatment systems is approximately 2 to 5°C (Crites et Al., 1981). Temperature and nitrate-N concentrations in the septic tank from 1992 to 1995 can be seen in Figure 4.18. It should be noted that during study period R nitrate-N levels reached 16.4 mg/l in the septic tank with an average temperature of 4.7 °C. This high nitrate-N level was due to the excessive loading of aerobic RSF effluent being recirculated to the septic tank due to a problem with the electrical controls for the pump in the pump chamber.

The lowest temperature in the septic tank was 3.3°C on February 25, 1994, near the minimum temperature for denitrification to occur. On this date 3.0 mg/l nitrate-N occurred in the septic tank, indicating that temperature may be inhibiting denitrification.

Denitrification rates are depressed below a pH of 6.0 and above a pH of 8.0 with the highest rates occurring within a pH range of 7.0 to 7.5 (EPA 1975). Figure 4.19 shows the pH in the septic tank from 1992 to 1995 at Site #1. After the installation of the "dam", pH values ranged outside the optimal range but did not fall within the ranges at which denitrification would become depressed. Therefore, denitrification within the septic tank was not inhibited by pH.

Table 4.13 shows the average percent denitrification, temperature, STR, BOD_5, COD, and pH measured in the septic tank for the study periods used in determining optimal effluent treatment by the RSF system at Site #1. Values of percent denitrification below 100% indicate that denitrification was inhibited during some study periods. Denitrification could not occur during study period N when no flow was directed to the septic tank.

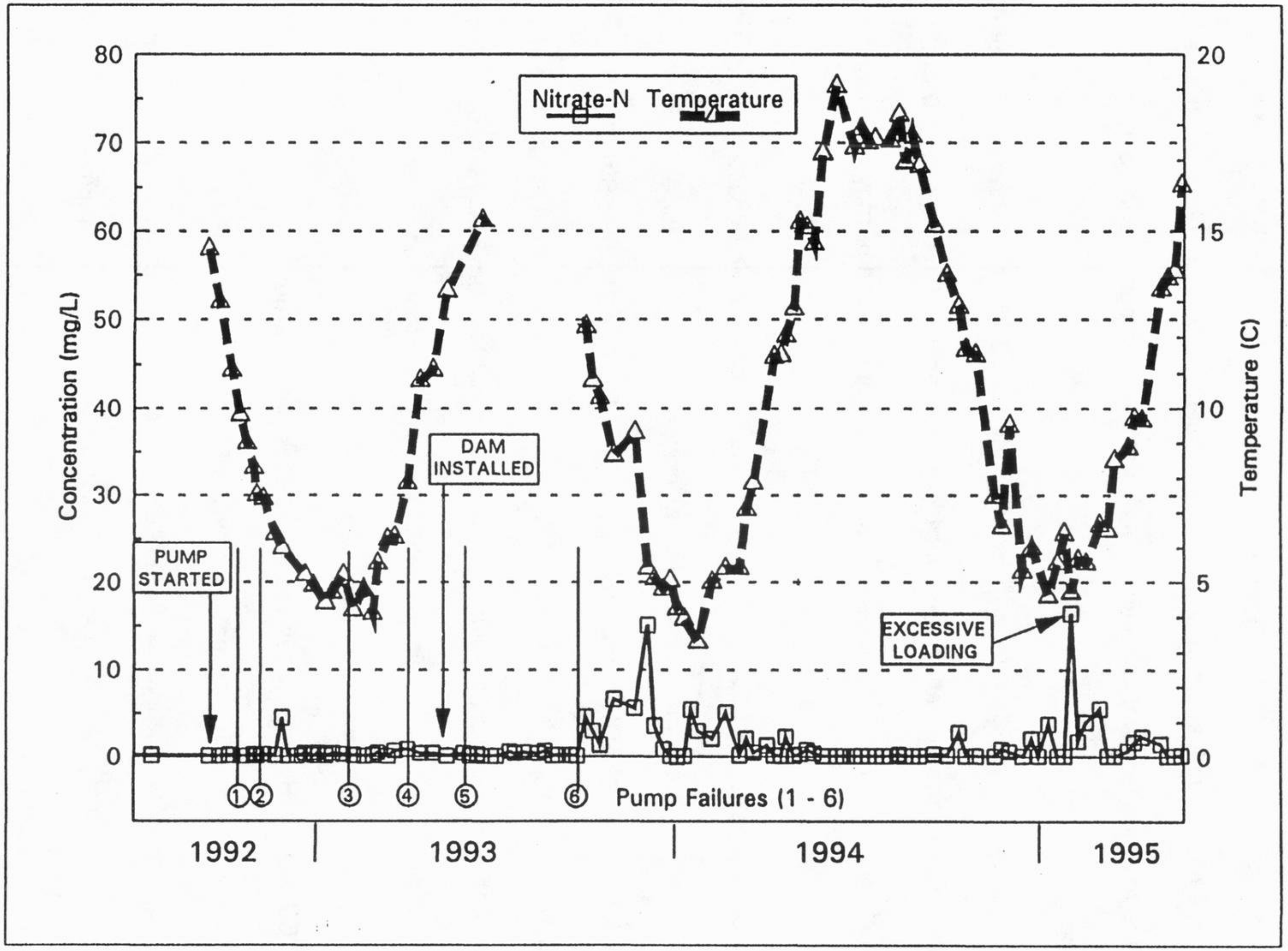


Figure 4.18 Nitrate-N concentrations and Temperature in the septic tank at Site #1 from 1992-1995.

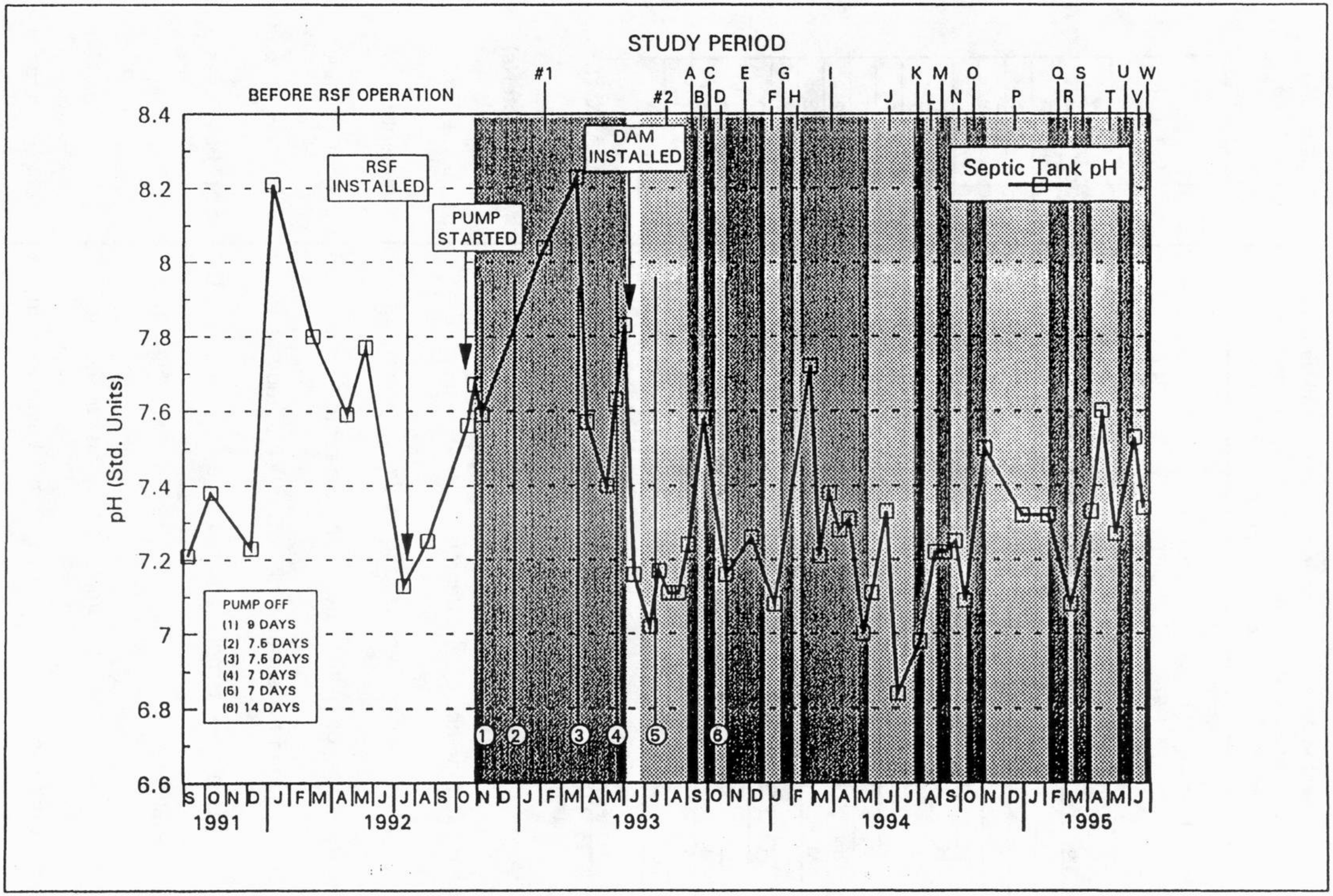


Figure 4.19 pH in the septic tank at Site #1 from 1991 to 1995.

Table 4.13 Average nitrate-N, temperature, septic tank ratio (STR), BOD$_5$, COD, and pH in the septic tank at Site #1 for the study periods used in determination of optimal effluent treatment by the RSF system

Study Period	Percent Denitrification (%)	Temperature (°C)	STR	BOD$_5$ (mg/l)	COD (mg/l)	pH Std. Units
#2	97.6	14.3	2.3	47	150	7.13
A	94.7	NS	2.3	54	151	7.24
B	93.6	NS	1.8	44	14	7.58
C	100.0	NS	2.5	NS	NS	NS
E	62.9	10.3	5.5	20	58	7.26
F	49.9	5.3	2.3	22	100	7.08
G	97.5	4.7	2.2	NS	NS	NS
H	100.0	4.0	0.9	NS	NS	NS
I	83.9	7.1	1.9	56	128	7.32
J	95.4	15.2	3.0	181	211	7.09
K	100.0	17.6	1.5	55	197	6.98
L	100.0	17.6	3.0	265	186	7.22
M	97.1	17.9	1.2	78	240	7.22
N	0	17.2	0.0	133	319	7.17
O	98.4	14.5	1.0	630	344	7.50
P	95.3	8.9	2.1	37	162	7.32
U	82.5	11.5	3.3	55	80	7.53
V	100.0	13.8	3.1	39	100	7.34
W	97.1	16.3	3.4	NS	NS	NS

Note Denitrification is assumed to be 100% complete for Nitrate-N <0.2 mg/l
NS Indicates no sample available

A multiple regression model was constructed to determine which factor(s) had the most influence on the degree of denitrification in the septic tank. Twelve of the 19 study periods were used to determine optimal effluent treatment by the RSF system could be used to determine the factor(s) that have the most influence on denitrification in the septic tank at Site #1. Study periods A, B, C, G, H, N, and W were omitted since temperature readings and/or BOD$_5$, COD, and pH analyses were not available.

The model yielded a multiple regression coefficient $R^2=0.81$ ($p=0.09$). The model does not allow any conclusions to be drawn with any statistical significance

($p<0.05$). However, this is the best model that be constructed for denitrification at Site #1 because many study periods were omitted for reasons previously discussed. Probability values (p) of the variable coefficients suggest that temperature explained the most variation in denitrification in the septic tank (Table 4.14).

Table 4.14 Variables, coefficients, and probability values for the multiple regression model of denitrification in the septic tank at Site #1.

Variable	Coefficient	p-value
Intercept	-436.9	0.14
Temperature	2.0	0.15
HRT	-9.8	0.19
STR	-4.8	0.41
BOD_5	-0.04	0.43
COD	0.22	0.30
pH	70.4	0.30

Incomplete denitrification in the septic tank can be attributed primarily to the low temperatures observed in the septic tank at Site #1. The low temperatures encountered in a typical Wisconsin winter may affect the nitrogen removal capability of the RSF system at Site #1; however, this site had low hydraulic loading and lower incoming temperatures than would most domestic systems.

As noted previously, denitrification does not occur solely in the septic tank. Denitrification of the nitrate-N produced in the sand filter ranged between 38 and 96% complete. It is suspected that more denitrification occurs in the sand filter when the saturated thickness in the bottom portion of the RSF is greater.

4.153 Total Nitrogen

The loading to and removal of TN from the system at Site #1 was obtained from mass balances of the 19 study periods used to determine optimal nitrogen removal. Due to the problems encountered with the design and actual flows, some untreated effluent bypassed the RSF and went directly to the drainfield during 15 of these 19 study periods. Hence, removals were calculated based on the mass applied to the RSF to make comparisons of the treatment efficiency for the various flow conditions used (Table 4.15). The total mass applied to the drainfield was calculated taking into consideration the bypass, and is used to determine the impacts on groundwater (Table 4.16). TN and chloride concentrations in the septic tank and sand filter from 1991 to 1995 are shown in Figure 4.20 and Figure 4.21, respectively.

TN removal based on the mass applied to the RSF ranged from 49.1% to 92.8% over the course of the study, with the various flow conditions described earlier. TN removal based on the mass applied to the drainfield ranged from 21.8% to 83.7%. These ranges are different because of the untreated effluent that bypassed the system.

The estimated TN concentration in the household waste was calculated from the mass balance calculations. From these calculations the average TN concentration in the household waste for the 19 study periods used in the determination of treatment efficiency was 78.1 mg/l (range 55.1 to 112.7 mg/l). The average TN concentration in the septic tank before the system was installed was 76.0 mg/l (range 58.0 to 96.0 mg/l). The average TN concentration measured in the septic tank effluent after system installation was 35.1 mg/l (range 19.8 to 68.8 mg/l). The average concentration of TN in the sand filter effluent applied to the drainfield was 21.7 mg/l (range 12.7 to 41.9 mg/l).

Table 4.15 Total Nitrogen removal for the RSF system at Site #1 based on mass applied to the RSF

Study Period	Start	End	Mass From House Entering RSF* (grams/day)	Mass Removed in Septic Tank (grams/day)	Mass Removed in RSF (grams/day)	Mass From RSF to Drainfield (grams/day)	Mass Removed by System (grams/day)	Percent Removal Based on Mass Applied to RSF* (%)
#2	6/30/93	8/31/93	24.8	7.1	9.4	8.4	16.4	66.2
A	9/1/93	9/16/93	24.6	7.4	8.9	8.3	16.3	66.1
B	9/17/93	9/23/93	27.4	7.8	11.6	8.1	19.3	70.5
C	9/24/93	10/11/93	35.2	3.0	29.6	2.5	32.6	92.8
E	10/27/93	12/23/93	27.2	13.7	6.8	6.7	20.6	75.6
F	12/24/93	1/12/94	27.5	5.4	13.1	9.1	18.4	67.0
G	1/13/94	2/4/94	35.0	12.2	14.8	8.0	27.0	77.2
H	2/5/94	2/11/94	29.4	6.0	18.6	4.8	24.7	83.8
I	2/12/94	5/20/94	37.6	7.9	21.3	8.3	29.3	77.9
J	5/21/94	7/22/94	44.1	12.8	24.2	7.2	37.0	83.7
K	7/23/94	8/9/94	33.8	13.8	11.3	8.7	25.1	74.2
L	8/10/94	8/25/94	18.4	5.1	9.5	3.7	14.7	79.7
M	8/26/94	9/16/94	19.3	2.3	10.9	6.1	13.3	68.7
N	9/17/94	10/6/94	9.9	0.0	4.9	5.0	4.9	49.1
O	10/7/94	11/7/94	19.4	4.2	10.5	4.7	14.7	75.7
P	11/8/94	2/3/95	25.3	12.1	9.9	3.3	22.0	86.8
U	5/12/95	6/7/95	32.0	11.5	14.6	5.9	26.1	81.5
V	6/8/95	6/21/95	21.7	9.4	8.1	4.3	17.5	80.3
W	6/22/95	6/28/95	34.4	12.4	16.2	5.8	28.6	83.2

*Reductions based on mass applied to RSF, does not account for mass bypassing the system.

Table 4.16 Total Nitrogen removal for the RSF system at Site #1 based on mass applied to drainfield.

Study Period	Start	End	Mass From House (grams/day)	Mass Removed in Septic Tank (grams/day)	Mass Removed in RSF (grams/day)	Mass From RSF to Drainfield (grams/day)	Mass Bypassing RSF (grams/day)	Total to Drainfield (grams/day)	Mass Removed by System* (grams/day)	Percent Removal Based on Mass Applied to Drainfield* (%)
#2	6/30/93	8/31/93	24.8	7.1	9.4	8.4	0.0	8.4	16.4	66.2
A	9/1/93	9/16/93	24.6	7.4	8.9	8.3	0.0	8.3	16.3	66.1
B	9/17/93	9/23/93	30.8	7.8	11.6	8.1	3.4	11.5	19.3	62.7
C	9/24/93	10/11/93	41.9	3.0	29.6	2.5	6.8	9.3	32.6	77.9
E	10/27/93	12/23/93	27.2	13.7	6.8	6.7	0.0	6.7	20.6	75.6
F	12/24/93	1/12/94	30.0	5.4	13.1	9.1	2.5	11.5	18.4	61.5
G	1/13/94	2/4/94	35.0	12.2	14.8	8.0	0.0	8.0	27.0	77.2
H	2/5/94	2/11/94	42.1	6.0	18.6	4.8	12.7	17.5	24.7	58.5
I	2/12/94	5/20/94	38.5	7.9	21.3	8.3	1.0	9.2	29.3	76.0
J	5/21/94	7/22/94	44.1	12.8	24.2	7.2	0.0	7.2	37.0	83.7
K	7/23/94	8/9/94	42.9	13.8	11.3	8.7	9.1	17.8	25.1	58.5
L	8/10/94	8/25/94	23.5	5.1	9.5	3.7	5.1	8.8	14.7	62.5
M	8/26/94	9/16/94	28.0	2.3	10.9	6.1	8.6	14.7	13.3	47.3
N	9/17/94	10/6/94	22.3	0.0	4.9	5.0	12.4	17.4	4.9	21.8
O	10/7/94	11/7/94	31.1	4.2	10.5	4.7	11.7	16.4	14.7	47.3
P	11/8/94	2/3/95	32.2	12.1	9.9	3.3	6.8	10.2	22.0	68.4
U	5/12/95	6/7/95	33.8	11.5	14.6	5.9	1.8	7.7	26.1	77.2
V	6/8/95	6/21/95	23.4	9.4	8.1	4.3	1.7	6.0	17.5	74.5
W	6/22/95	6/28/95	36.1	12.4	16.2	5.8	1.7	7.5	28.6	79.2

* Reductions based on mass applied to drainfield, accounts for mass bypassing the RSF and going directly to drainfield.

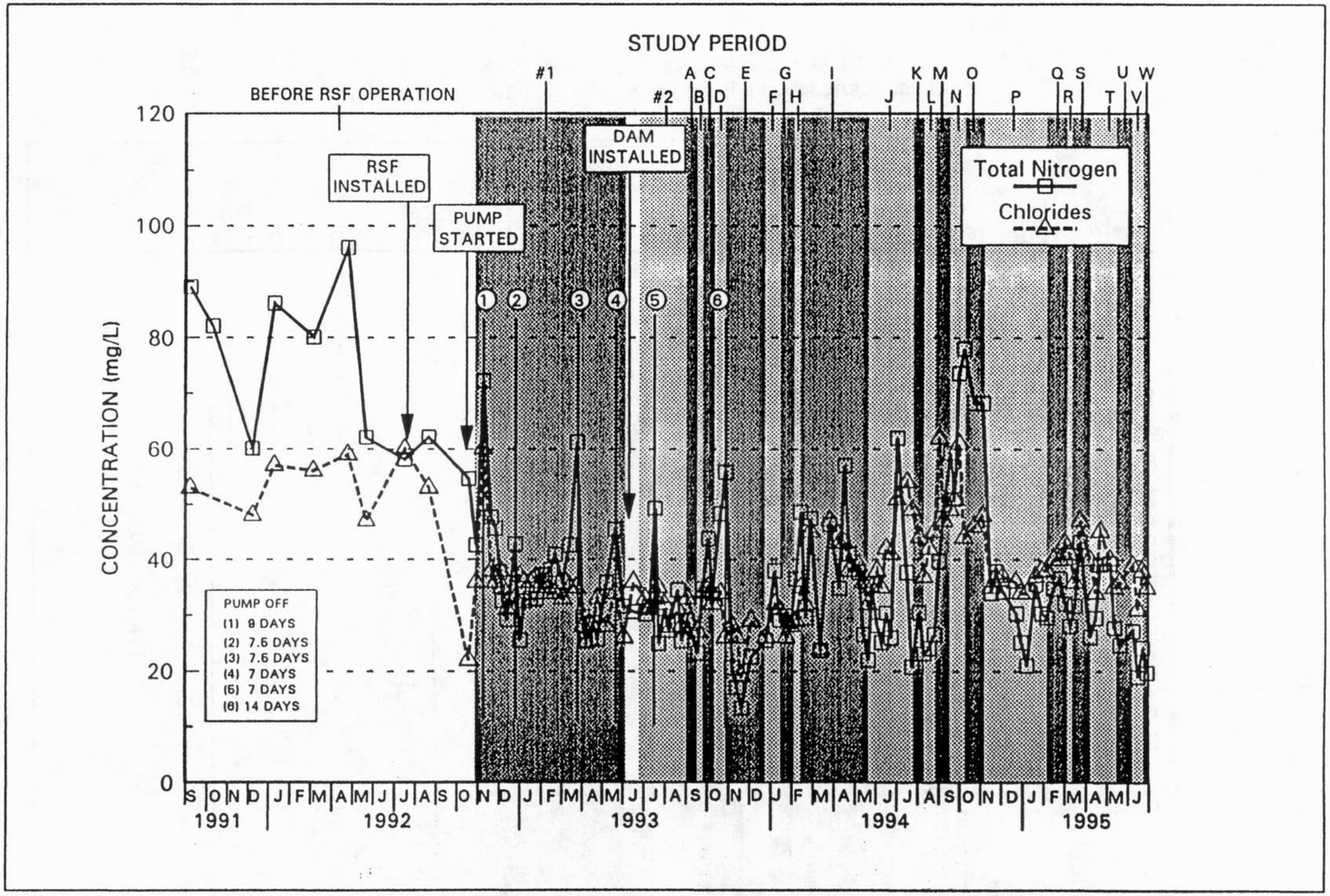


Figure 4.20 Total Nitrogen and Chloride concentrations in the septic tank at Site #1 from 1991 to 1995.

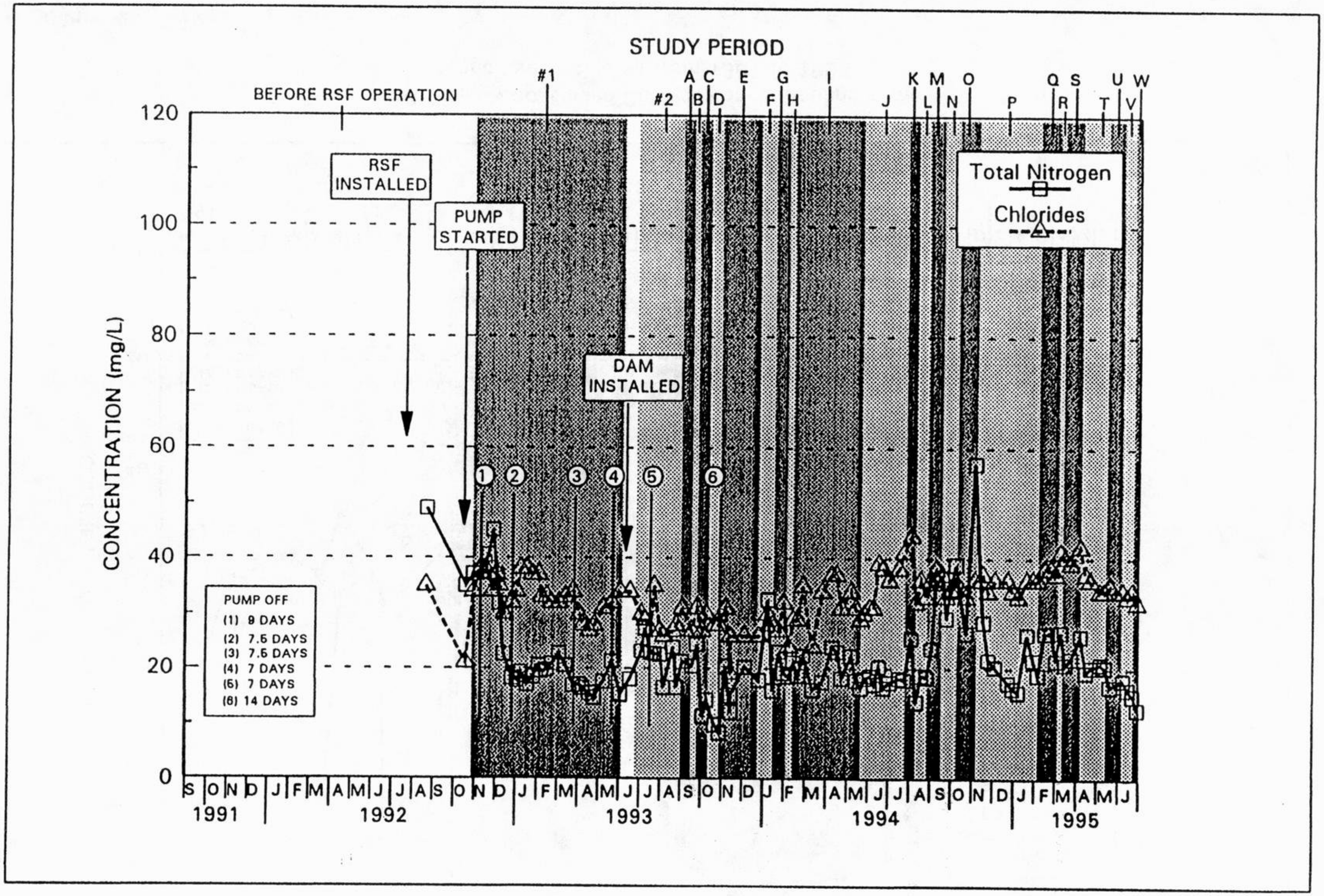
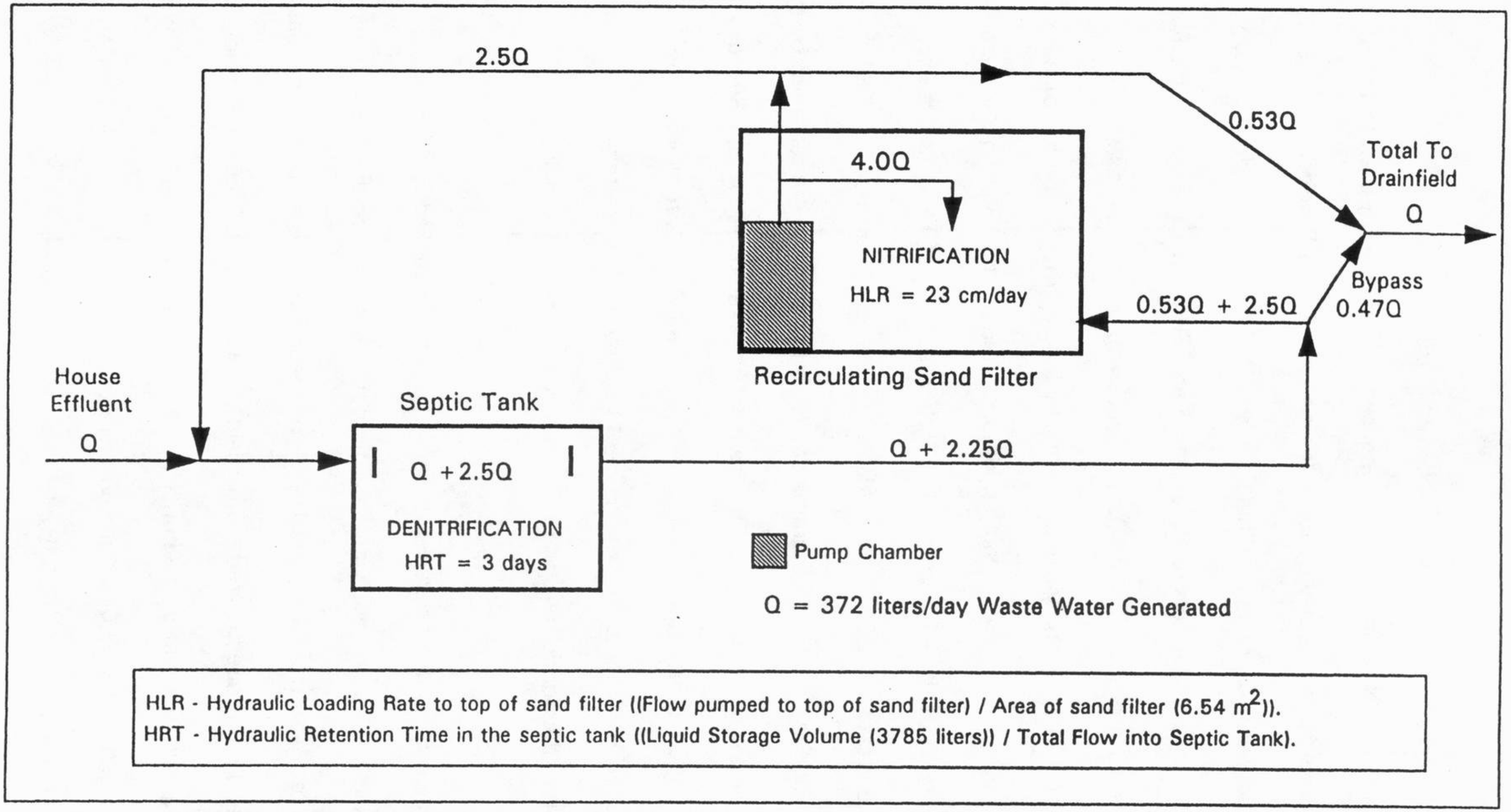


Figure 4.21 Total Nitrogen and Chloride concentrations in the sand filter at Site #1 from 1991 to 1995.

Maximum Removal

A maximum TN removal of 92.8% based on the mass applied to the RSF was accomplished during study period C (September 24, to October 11, 1993). The household water usage during this time was 372 lpd (98.3 gpd). The flow conditions used to achieve this removal where an SFR of 4.0Q, an STR of 2.5Q, a drainfield ratio (Q*) of 0.53Q, with 0.47Q bypassing the system as shown in Figure 4.22. These flow conditions yielded a hydraulic loading rate to the top of the sand filter of 23 cm/day (5.6 gpd/ft^2) and a hydraulic retention time within the septic tank of 3.0 days. Of the 41.9 grams/day (0.097 lbs/day) of TN estimated to be in the household waste, only 35.2 grams/day (0.078 lbs/day) entered the RSF. Of this mass applied to the RSF, 3.0 grams/day (0.066 lbs/day) were removed by the septic tank and 29.6 grams/day (0.065 lbs/day) were removed in the RSF, for a total removal of 32.6 grams/day (0.072 lbs/day) or 92.8% TN removal. The estimated household TN concentration was 112.7 mg/l. The average TN concentration in the septic tank and RSF effluents were 38.8 and 12.8 mg/l, respectively.

The results for study period C based on the mass applied to the RSF, indicate excellent TN removal. Only a portion of the mass from the house entered the RSF and received treatment; 16.1% of the mass from the household bypassed the system and went directly to the drainfield (Table 4.17). Of the 41.9 grams/day (0.092 lbs/day) coming out of the household 32.6 grams/day (0.072 lbs/day) were removed, 2.5 grams/day (0.006 lbs/day) were pumped to the drainfield from the RSF, while 6.8 grams/day (0.015 lbs/day) bypassed the system.



Q = 372 liters/day Waste Water Generated

HLR - Hydraulic Loading Rate to top of sand filter ((Flow pumped to top of sand filter) / Area of sand filter (6.54 m^2)).
HRT - Hydraulic Retention Time in the septic tank ((Liquid Storage Volume (3785 liters)) / Total Flow into Septic Tank).

Figure 4.22 Flow shematics for study period C at Site #1

Table 4.17 Percent daily loading and removal of household Total N mass for the RSF system at Site #1.

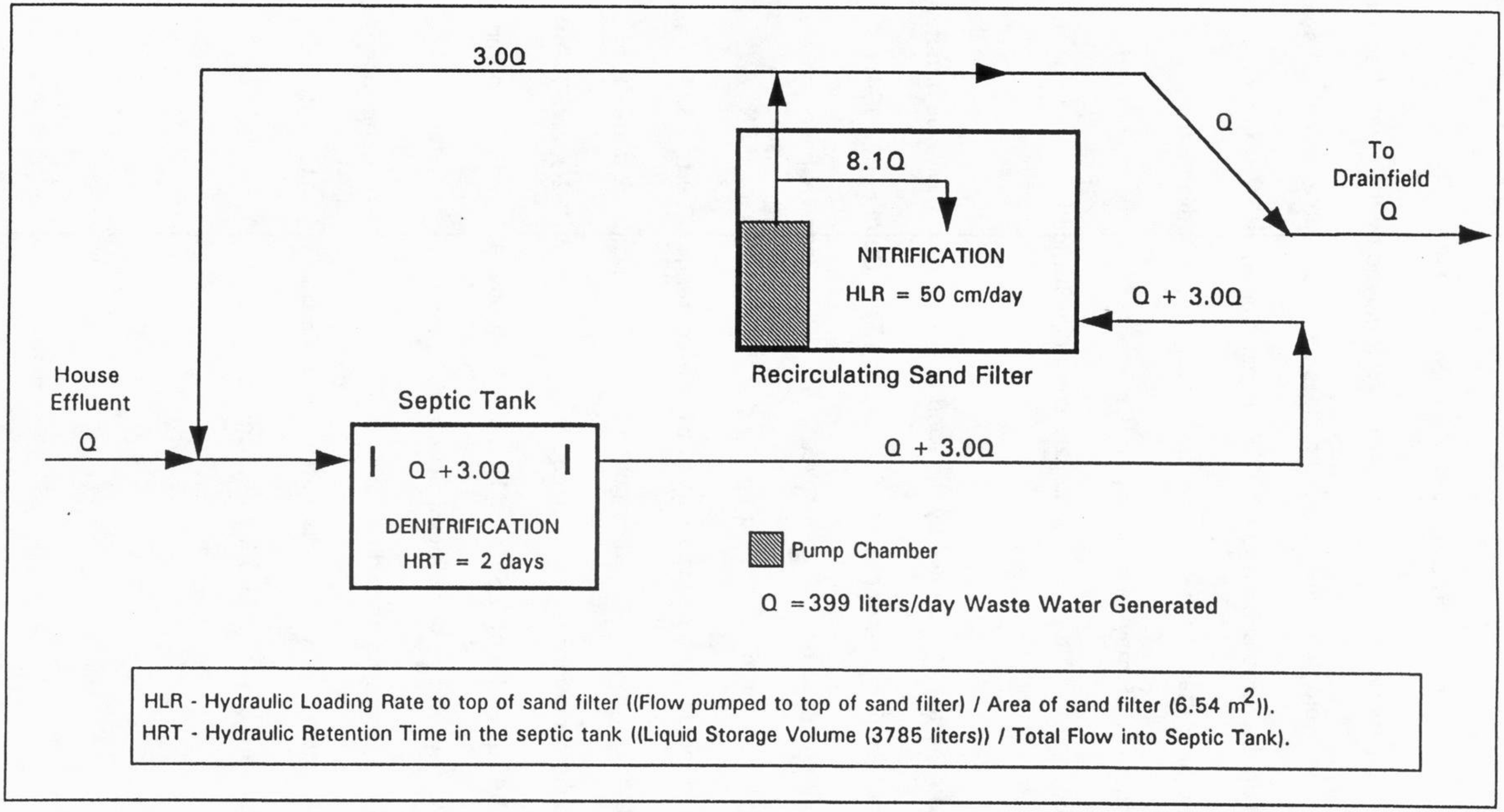
Study Period	Start	End	Mass From House (grams/day)	% Mass Removed in Septic Tank	% Mass Removed in RSF	% Mass From RSF to Drainfield	% Mass Bypassing RSF	% Mass Total to Drainfield	% Mass Removed by System*
#2	6/30/93	8/31/93	24.8	28.5	37.9	33.9	0.0	33.9	66.2
A	9/1/93	9/16/93	24.6	30.0	36.1	33.9	0.0	33.9	66.1
B	9/17/93	9/23/93	30.8	25.2	37.5	26.3	11.0	37.3	62.7
C	9/24/93	10/11/93	41.9	7.2	70.6	6.0	16.1	22.1	77.9
E	10/27/93	12/23/93	27.2	50.5	25.1	25.4	0.0	24.4	75.6
F	12/24/93	1/12/94	30.0	17.9	43.6	30.3	8.2	38.5	61.5
G	1/13/94	2/4/94	35.0	34.9	44.2	22.8	0.0	22.8	77.2
H	2/5/94	2/11/94	42.1	14.3	44.3	11.3	30.1	41.5	58.5
I	2/12/94	5/20/94	38.5	20.6	55.4	21.5	2.5	24.0	76.0
J	5/21/94	7/22/94	44.1	29.0	54.8	16.3	0.0	16.3	83.7
K	7/23/94	8/9/94	42.9	32.2	26.3	20.3	21.2	41.5	58.5
L	8/10/94	8/25/94	23.5	21.8	40.6	15.9	21.6	37.5	62.5
M	8/26/94	9/16/94	28.0	8.3	39.1	21.7	30.9	52.6	747.4
N	9/17/94	10/6/94	22.3	0.0	21.8	22.6	55.5	78.2	21.8
O	10/7/94	11/7/94	31.1	13.6	33.7	15.2	37.5	52.7	47.3
P	11/8/94	2/3/95	32.2	37.7	30.7	10.4	21.2	31.6	68.4
U	5/12/95	6/7/95	33.8	33.9	43.3	17.5	5.2	22.8	77.2
V	6/8/95	6/21/95	23.4	40.1	34.5	18.3	7.2	25.5	74.5
W	6/22/95	6/28/95	36.1	34.2	45.0	16.0	4.8	20.8	79.2

* Reductions based on mass applied to drainfield, accounts for mass bypassing the RSF and going directly to drainfield.

By accounting for the mass bypassing, the TN removal is reduced from 92.8% to 77.9%, since 9.3 grams/day (0.021 lbs/day) or 3.4 kg/year (7.5 lbs/year), instead of 2.5 grams/day (0.006 lbs/day)or 0.9 kg/year (2.0 lbs/year), were entering the drainfield.

The highest TN removal that occurred when the system was operating without any bypass, was 83.7 %, achieved during study period J (April 21, to July 22, 1994). The average household water use (Q) was 399 lpd (105.4 gpd) with an SFR of 8.1Q, an STR 3.0Q, an HLR of 50 cm/day (4.8 gpd/ft^2), and an HRT of 2.0 days. The flow schematics for study period J at Site #1 can be seen in Figure 4.23. Of the 44.1 grams/day (0.097 lbs/day) of TN generated in the house daily, 7.2 grams/day (0.016 lbs/day) or 2.6 kg/year (5.7 lbs/year) entered the drainfield. The estimated TN concentration in the household was 110.6 mg/l. The average TN concentration in the septic tank and sand filter effluents were 32.2 and 18.0 mg/l, respectively.

Study periods #2, A, E, and G did not have any bypass and TN removals of 66.2, 66.1, 75.6, and 77.2% were achieved. The estimated TN concentrations in the household for study periods #2, A, E, and G were 64.6, 57.5, 70.0, and 88.5 mg/l, respectively. The average TN concentrations in the septic tank for study periods #2, A, E, G were 29.3, 25.8, 19.8, and 32.0 mg/l, respectively. Average TN concentrations in RSF effluent pumped to the drainfield for study periods #2, A, E, and G were 21.8, 19.4, 17.1, and 20.2 mg/l, respectively. While above the 10 mg/l nitrate-N standard the concentrations in the RSF represents an average of 76% nitrogen removal by the system.



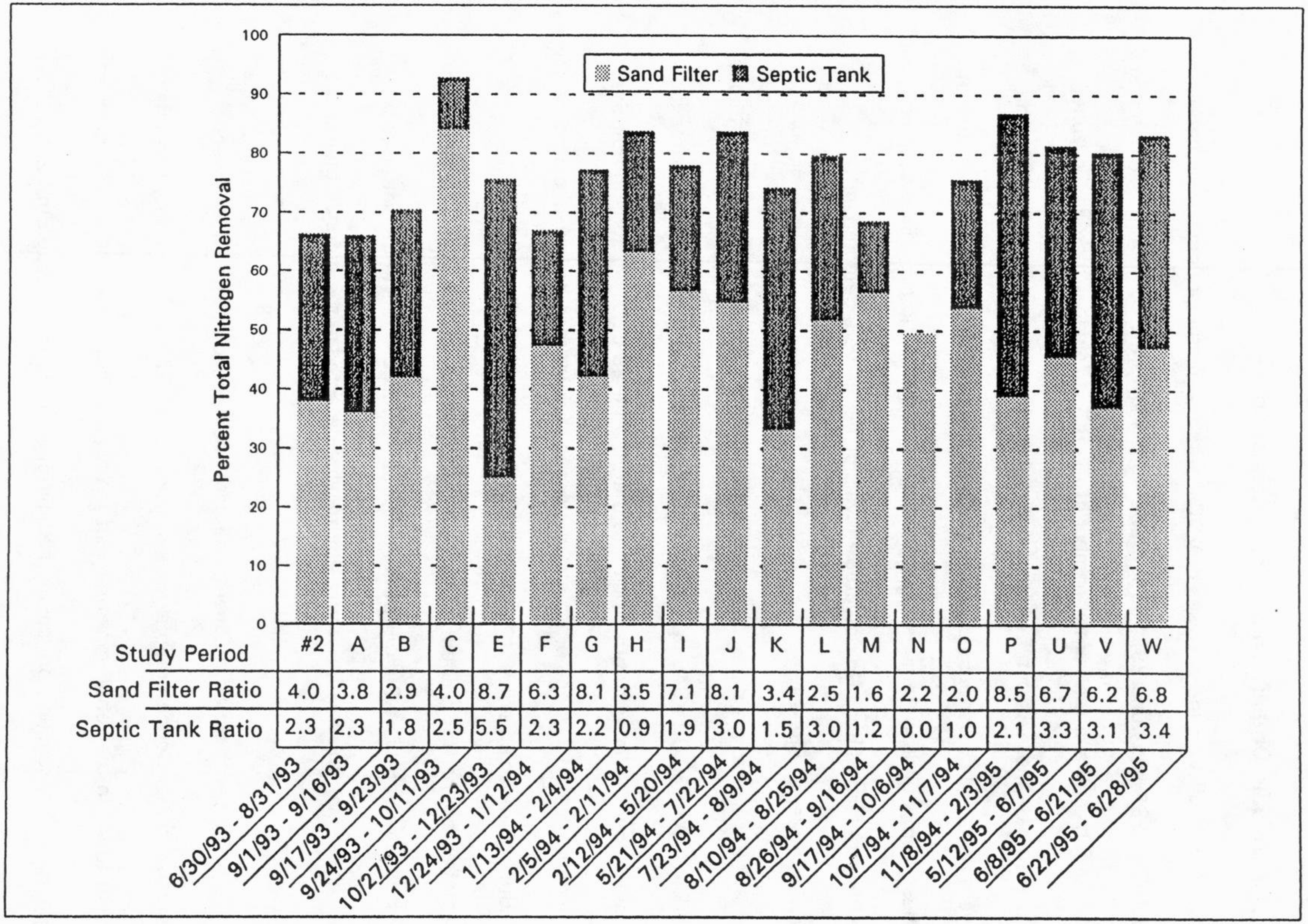
HLR - Hydraulic Loading Rate to top of sand filter ((Flow pumped to top of sand filter) / Area of sand filter (6.54 m^2)).
HRT - Hydraulic Retention Time in the septic tank ((Liquid Storage Volume (3785 liters)) / Total Flow into Septic Tank).

Figure 4.23 Flow schematic for study period J at Site #1

Removal in Sand Filter and Septic Tank

Figure 4.24 shows the percentage of the TN removed by the sand filter and septic tank, which were determined from the mass balance calculations of 19 study periods based on the mass applied to the RSF. In general, more TN was removed within the sand filter than in the septic tank. Although the volatilization of ammonia within the RSF may be occurring, it is suspected that the majority of the TN removed by the sand filter occurred largely due to denitrification within the rock storage unit.

To determine the TN removal by the sand filter alone, no flow was directed to the septic tank during study period N (September 17, to October 6, 1994). Denitrification in the sand filter alone resulted in 49.1% TN removal based on the mass applied to the sand filter. During this 19 day period the average TN concentration in the septic tank increased to 67.5 mg/l (range 50.9 to 78.0 mg/l). The average forward flow (Q) for study period N was 330 lpd (87.2 gpd). The average hydraulic conditions for study period N were an SFR of 2.2Q, an STR of 0Q, a Q* of 0.45Q, a BPR of 0.55Q, an HLR of 11 cm/day (1.1 gpd/ft^2), and an HRT of 11.5 days. With no recycling of flow to the septic tank, this TN concentration represents household conditions. The average TN concentration in the septic tank before the system was installed was 76.6 mg/l (range 58 to 96 mg/l) with a forward flow of 371.3 lpd (98.1 gpd).



Study Period	#2	A	B	C	E	F	G	H	I	J	K	L	M	N	O	P	U	V	W
Sand Filter Ratio	4.0	3.8	2.9	4.0	8.7	6.3	8.1	3.5	7.1	8.1	3.4	2.5	1.6	2.2	2.0	8.5	6.7	6.2	6.8
Septic Tank Ratio	2.3	2.3	1.8	2.5	5.5	2.3	2.2	0.9	1.9	3.0	1.5	3.0	1.2	0.0	1.0	2.1	3.3	3.1	3.4
	6/30/93 - 8/31/93	9/1/93 - 9/16/93	9/17/93 - 9/23/93	9/24/93 - 10/11/93	10/27/93 - 12/23/93	12/24/93 - 1/12/94	1/13/94 - 2/4/94	2/5/94 - 2/11/94	2/12/94 - 5/20/94	5/21/94 - 7/22/94	7/23/94 - 8/9/94	8/10/94 - 8/25/94	8/26/94 - 9/16/94	9/17/94 - 10/6/94	10/7/94 - 11/7/94	11/8/94 - 2/3/95	5/12/95 - 6/7/95	6/8/95 - 6/21/95	6/22/95 - 6/28/95

Figure 4.24 **Percent Total N removal in the septic tank and sand filter at Site #1. Percent removal based on the household mass applied to the RSF. Does not include mass bypassing the RSF.**

4.16 Volatile Organic Compound Attenuation

Samples for the analyses of VOCs were taken on eight occasions between July 8, 1993 and June 29, 1995. Table 4.18 shows the compounds detected in the septic tank and the pump chamber of the RSF. Toluene, sec-butylbenzene, dichloromethane, tert-butylbenzene, 1,4-dichlorobenzene, and o-xylene/styrene were detected on various occasions in the system at Site #1. Concentrations of these compounds were generally much higher in the septic tank, indicating removal within the sand filter. No detectable levels of VOCs were detected in the groundwater at Site #1.

For all sample dates toluene was found in the septic tank in concentrations ranging from 20.0 to 345 µg/L. Toluene concentrations in the sand filter ranged from below the limits of quantitaion (0.5 µg/L) to 64.9 µg/L. Assuming that the concentration of Toluene present in the septic tank represents household conditions diluted by recycled effluent from the RSF, 81 to 100% of the toluene was removed by the RSF system. The Maximum Contaminant Level (MCL) for toluene in drinking water is 1.0 mg/l (USEPA, 1995). Toluene levels in the septic tank never reached this level, nor was any toluene detected in the groundwater at Site #1.

The second most frequently detected compound at Site #1 was sec-butylbenzene. Concentrations ranging from 1.7 to 52.1 µg/L sec-butylbenzene were found in the septic tank on six of the eight sample dates. Concentrations in the sand filter ranged from below the limit of quantitation (0.5 µg/L) to 2.1 µg/L.

Table 4.18 Volatile Organic Compounds detected in the septic tank and pump chamber of the RSF at Site #1

Sample date	July 8, 1993		October 19, 1993		January 21, 1994		April 5, 1994		August 16, 1994		February 3, 1995		May 19, 1995		June 29, 1995	
Compound (ppb)	ST	RSF	ST	RSF	ST	RSF	ST	RSF	ST	RSF	ST	RSF	ST	RSF	ST	RSF
	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND
Toluene	24.0	NS	53.9	ND	20.0	1.8	36.0	1.7	345	64.9	50	ND	57.8	2.1	135	6.3
sec-Butylbenzene	15.1	NS	52.1	ND	ND	ND	ND	ND	9.3	1.5	0.5	ND	1.7	ND	2.1	2.1
Dichloromethane	5.5	NS	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND
tert-Butylbenzene	ND	NS	1.7	ND	2.7	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND
Bromobenzene	ND	NS	1.0	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND
1,4-Dichlorobenzene	ND	NS	ND	ND	3.7	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND
o-Xylene/Styrene	ND	NS	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	2.6
Temperature (°C)	NS	NS	NS	NS	4.8	4.4	5.4	5.2	17.8	18.7	4.6	3.6	9.7	10.3	16.3	17.7

NS No Sample Taken
ND Compound not detected

ST Septic Tank
RSF Pump Chamber of RSF

The RSF system removed between 0 and 100% of the sec-butylbenzene present in the septic tank. An MCL for sec-butylbenzene has yet to be issued by USEPA. No sec-butylbenzene was detected in the groundwater at Site #1.

Dichloromethane, tert-butylbenzene, and bromobenzene were each detected in the septic tank on at least one sample date. It is unknown what percentage of the 5.5 µg/L of dichloromethane detected in the septic tank on July 8, 1993 was removed, since a sample was not obtained from the sand filter on this date. One-hundred percent removal of tert-butylbenzene occurred for the samples of October 19, 1993 and January 21, 1994. It also appears that 100% removal of bromobenzene and 1,4-dichlorobenzene occurred for the samples of October 19, 1993 and January 21, 1994, respectively.

4.17 Study Periods Omitted From Evaluation of Effluent Treatment

Study Period #1

During study period #1 the problem which resulted from retrofitting the RSF to the existing septic system at Site #1 made it impossible to determine the portions of the effluent applied to the drainfiled from the sand filter or bypassed directly from the septic tank. As a result, mass balance calculations could not be produced. Table 4.19 shows the ranges, averages and percent reductions of several water quality characteristics for the septic tank and pump chamber of the RSF during study period #1 at Site #1. The reductions during study period #1 are based on the concentrations observed in the septic tank prior to the installation of the RSF system at Site #1. In the worst case scenario all of the effluent applied to

Table 4.19 Averages, ranges and percent removals of several water quality characteristics for the septic tank and pump chamber of the RSF at site #1, Study period #1.

	Septic Tank BEFORE RSF Installed 9/6/91 to 7/15/92			Septic Tank AFTER RSF Installed * 10/27/92 to 6/8/93				Sand Filter * 10/27/92 to 6/8/93			
	Range Min mg/L	Range Max. mg/L	Avg. mg/L	Range Min mg/L	Range Max. mg/L	Avg. mg/L	Red. %	Range Min mg/L	Range Max. mg/L	Avg. mg/L	Red. %
BOD$_5$	203	337	270.6	43	84.0	63.1	76.7	1.6	11.0	5.7	97.9
COD	344	585	447.5	112.8	229.8	161.8	63.8	25.5	260.0	74.5	83.4
NO_2+NO_3-N	<0.2	<0.2	<0.2	<0.2	4.5	0.4	NA	0.9	25.1	15.3	NA
NH_4^+	53.8	77	62.8	16.1	42.2	27.7	55.9	1.1	34.5	7.0	88.9
TKN	58	96	76.6	21.0	47.5	34.5	55.0	2.0	36.2	8.1	89.4
Total N	58	96	76.6	25.5	47.5	34.9	54.4	14.6	45.1	23.4	69.5
Total P	6.20	13.80	9.00	3.00	5.30	4.10	54.4	1.50	3.10	2.40	73.3
Alkalinity	340	432	381.4	228.0	306.0	266.8	30.0	128.0	293.0	172.2	54.9
Total Hardness	124	180	154.5	156.0	184.0	167.2	-8.2	168.0	196.0	179.2	-16.0
Chlorides	47	60	47.5	25.0	38.0	33.4	29.7	27.0	38.0	33.3	29.9
pH	7.13	8.21	7.54	7.40	8.04	7.70	-2.12	6.56	7.64	7.27	3.58
Conductivity	NS	NS	NS	594	731	658	NA	473	746	597	NA
Sodium	31.1	40.1	34.2	29.1	38.0	32.0	6.4	27.8	45.0	32.7	4.4
Fluorescence	222	420	324	85	195	128	60.4	49	129	77	76.1

Shading indicates percent reduction

(*) Does not include first two sampling periods after RSF was installed, or pump failure periods > 5 days.

(-) Indicates an increase.

the drainfield came from the septic tank; TN concentrations applied to the drainfield were 54.4% less than those being applied to the drainfield without the RSF system.

Incorrect Design Flow Settings

The flow rate from the RSF to the drainfield was mistakenly set to zero during study periods Q, S, and T, making it impossible to determine how much effluent was treated by the RSF and how much bypassed the system. Therefore, mass balances were not produced for these study periods.

Study Period D

During study period D (October 12 to 26, 1993) the timer control panel for the sand filter pump was removed on October 12, 1993 for a design modification. With the timer control panel removed, effluent was not recirculated through the system. When water levels in the sand filter reached the bypass level septic tank effluent bypassed the sand filter and went directly to the drainfield. Since no effluent was recirculated during this time, it was assumed that nitrogen was not being removed by the system. Therefore, a mass balance was not produced. A simplified timer control panel was installed on October 26, 1993 and effluent was once again recirculated through the system.

The concentrations of chemical species in the septic tank during study period D represent household conditions, because the system was not operating and no flow was directed back to the septic tank. The average concentrations of several water quality characteristics observed during study period D and those observed in the septic tank prior to the installation of the RSF system are included in Table 4.20.

Table 4.20 Concentrations of several water quality characteristics in the septic tank and pump chamber of the RSF from before the installation of the system and also during study period D at Site #1.

	Septic Tank BEFORE RSF Installed 9/6/91 -7/15/92			Septic Tank AFTER RSF Study Period D 10/12/93 - 10/26/93			Sand Filter AFTER RSF Study Period D 10/12/93 - 10/26/93		
	Range			Range			Range		
	Min mg/l	Max. mg/l	Avg. mg/l	Min mg/l	Max. mg/l	Avg. mg/l	Min mg/l	Max. mg/l	Avg. mg/l
BOD_5	203	337	270.6	86.0	86.0	86.0	44	44	44
COD	344	585	447.5	212	212	212	24	24	24
NO_3^--NO_2^--N	<0.2	<0.2	<0.2	<0.2	0.2	<0.2	0.2	2.3	1.3
NH_4^+-N	53.8	77	62.8	41.0	47.0	41.0	6.2	7.2	6.7
TKN	58	96	76.6	48.0	55.6	51.8	7.5	8.2	7.9
TN	58	96	76.6	48.2	55.6	51.8	8.4	9.8	9.1
TP	6.20	13.80	9.00	6.50	6.50	6.50	6.25	6.25	6.25
Alkalinity	340	432	381.4	300	300	300	160	160	160
Total Hardness	124	180	154.5	132	132	132	168	168	168
Chlorides	47	60	47.5	26.0	34.0	30.0	28	28	28
pH	7.13	8.21	7.54	7.2	7.2	7.2	7.4	7.4	7.4
Conductivity	NS	NS	NS	741	741	741	535	535	535
Sodium	31.1	40.1	34.2	29.1	29.1	29.1	33.6	33.6	33.6
Fluorescence	222	420	324	138	138	138	99	99	99

The forward flows (Q) before the system was installed and during study period D were 371 lpd (98.4 gpd) and 347 lpd (91.7 gpd), respectively. The hydraulic residence times (HRT) in the septic tank before the installation of the RSF and study period D were 10.2 and 10.9 days, respectively.

The concentrations observed in the septic tank during study period D were less than those observed before the system was installed. The concentrations may have been lower due to less loading from the household, as evidenced by the lower Q observed during study period D. Another possibility is that the HRT of 10.9 days may have not been long enough to reestablish household conditions in the septic tank, as only 12 days elapsed and concentrations in the septic tank may still have been diluted by recycled treated effluent.

Study Period R

During study period R (March 4 to 10, 1995) the electrical relay to the pump in the sand filter failed. The relay failure caused the pump in the sand filter to recirculate effluent continuously through the system. The relay failure may have been caused by the presence of water in the box that houses the electrical components for pump operation. Melting snow contributed water which infiltrated through a broken rubber seal on the access panel of the box. The exact date of the relay failure could not be determined, and it is unknown how long this condition existed between March 3 and 8. Therefore, the amount of flow being recirculated through the system was unknown and a mass balance could not be produced. The system was shut off on March 8, 1995 until a new 240-volt/12 amp solid state relay was installed on March 10, 1995.

The loading to and removal of nitrogen within the system during study period R is unknown. However, inspection of chemical data collected on March 8, 1995 shows the results of the excessive loading of aerobic RSF effluent to the septic tank. Nitrate-N levels in the septic tank increased to 16.4 mg/l, indicating incomplete denitrification (Table 4.21). Approximately 25% denitrification of the 22.1 mg/l of nitrate-N pumped from the RSF to the septic tank occurred, indicating that it is important not to over circulate RSF water to the septic tank.

Table 4.21 Concentrations of chemical species in the Septic Tank and Sand Filter before, during, and after the system failure of study period R (All values mg/l).

	Study Period	Sample Date	NO_3^--NO_2^--N	NH_4-N	TKN	TN	Chloride
Septic Tank	Q	3/1/95	<0.2	31.0	32	32.1	43
	R	3/8/95	16.4	9.9	11.6	28.0	41
	S	3/15/95	1.8	17.8	29.9	31.7	36
Sand Filter	Q	3/1/95	19.7	1.5	1.8	21.5	37
	R	3/8/95	22.1	4.25	4.76	26.9	41
	S	3/15/95	16.6	2.52	4.2	20.8	40

4.18 Groundwater Impacts

The groundwater quality at Site #1 has been monitored between 1988 and 1995 in three different studies conducted through the University of Wisconsin-Stevens Point. The first study conducted by Shaw et al. (1993), investigated septic system and lawn chemical impacts at Site #1 (referred to as Site REE) from 1988 to 1991 as part of a study designed to determine subdivision impacts on groundwater quality. With homeowner cooperation Site #1 was selected for the denitrification studies because the plume from the soil absorption system was well identified.

This section describes the changes in groundwater quality resulting from the installation of the RSF system through June 1995. The figures include a separation indicating conditions before the installation of the RSF (BEFORE RSF) and conditions after the installation of the RSF (AFTER RSF).

4.181 Installation of Monitoring Wells

The following is a description of the installation of the monitoring wells at Site #1. From 1988 to 1995, 37 wells were installed to allow sampling from various locations in the aquifer (Figure 3.17 and 4.27). Of these 37 wells, four were single depth skimming (REE UG, REE DG, REC V, and REC V2), two were well nests screened at 3 depths (REW S-D, and REC S-D), and five were multiport wells with five to seven ports each (RSDS A-E). The elevations and screened intervals of these wells are included in Table 4.22.

Table 4.22 Elevations of the monitoring wells and screen lengths at Site #1.

Well	Installed	Point Depth ft	Point Elevation ft MSL	Screen Length ft	Casing Elevation ft MSL
REE UG	Summer, 1988	23.11	1092.69	3	1115.80
REE DG	Summer, 1988	23.12	1092.44	3	1115.56
REW S	Summer, 1989	19.35	1096.29	1	1115.64
REW M	Summer, 1989	20.08	1094.84	1	1115.64
REW D	Summer, 1989	22.06	1093.58	1	1115.64
REC S	Summer, 1989	19.25	1096.00	1	1115.25
REC M	Summer, 1989	20.56	1094.69	1	1115.25
REC D	Summer, 1989	22.06	1093.19	1	1115.25
REC V	Oct. 26, 1993	17.74	1097.34	3	1115.08
REC V2	Dec. 1, 1994	21.05	1094.55	3	1115.60
RSDS A	Summer, 1990	19 - 23	1092.28	0.83 - 1	1115.28
RSDS B	Summer, 1990	19 - 25	1090.10	0.83 - 1	1115.10
RSDS C	Summer, 1990	19 - 23	1092.12	0.83 - 1	1115.12
RSDS D	Summer, 1990	19 - 23	1091.89	0.83 - 1	1114.89
RSDS E	Summer, 1990	19 - 23	1091.86	0.83 - 1	1114.86

Originally one up-gradient well (REE UG) and one down-gradient well (REE DG) were installed at Site #1 in the summer of 1988 (VanRyswyk, 1996). Fluctuations in the water table elevations and chloride concentrations for wells REE UG and REE DG are shown in Figures 4.25 and 4.26, respectively. The average background chloride concentration found in the up-gradient well (REE UG) between October 4, 1988 and June 29, 1995 was 2.9 mg/l (range <1.0 to 8.0 mg/l). The average chloride concentration in the down-gradient well (REE DG) over the same time period was 4.1 mg/l (range <1.0 to 9.0 mg/l). In this discussion, any monitoring well with a chloride concentration of 10 mg/l or greater is considered to be located in the plume originating beneath the soil absorption system.

The average chloride concentration of 4.1 mg/l in well REE DG indicates that it is not in the plume; however, this average could be skewed low due to the fact that REE DG's screened interval was often more then 1.9 m (6.3 ft) below the

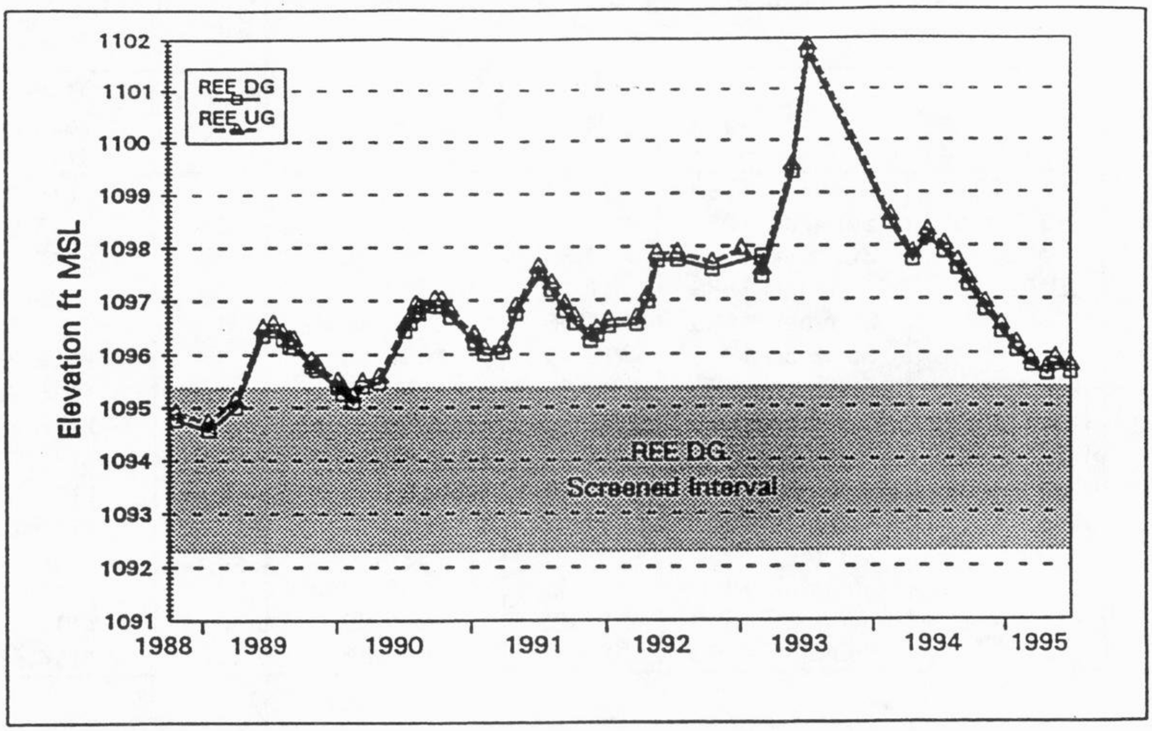


Figure 4.25 Watertable fluctuations: wells REE DG and REE UG, 1988-1995.

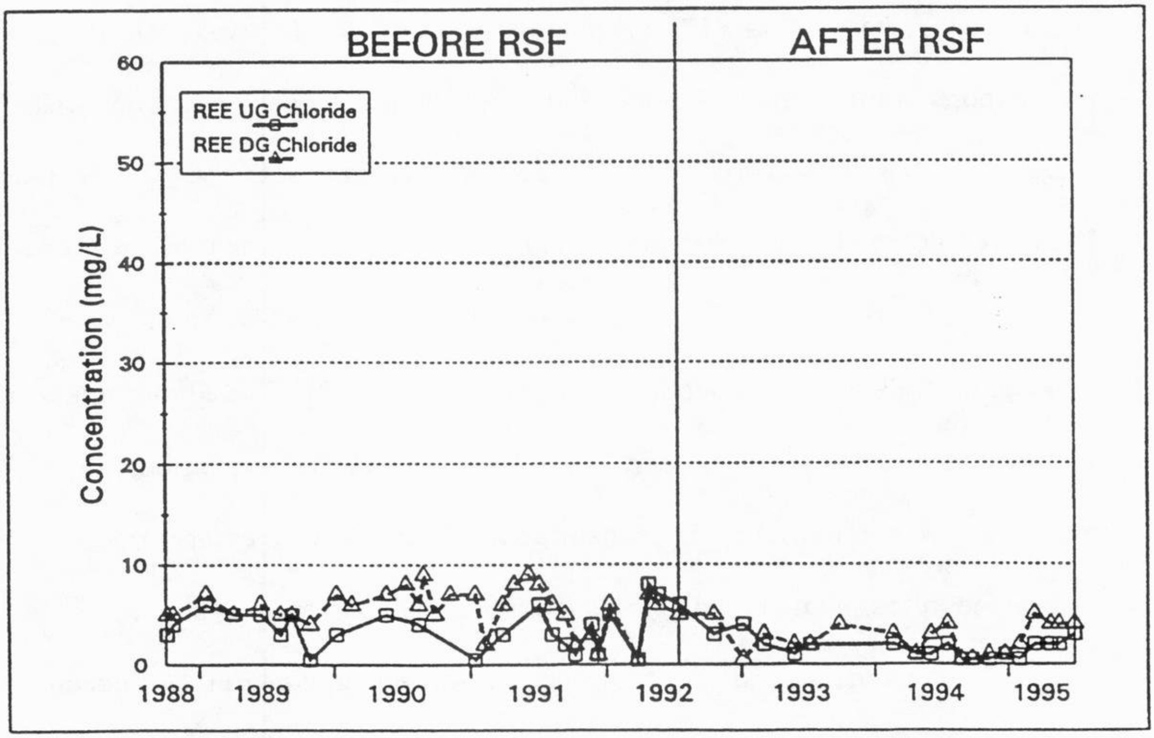


Figure 4.26 Chloride concentrations in wells REE DG and REE UG, 1988-1995.

water table, where contaminant concentrations are expected to be highest. To test this hypothesis, chloride data were evaluated for two time periods when the screened interval of REE DG did intersect the water table: October 20 through March 31, 1988 and also between January 5 through March 12, 1990. The average chloride concentrations in the up-gradient well (REE UG) for these time periods were 5.0 and 3.0 mg/l, respectively. The average chloride concentrations in the down-gradient well (REE DG) for these time periods were 5.7 and 7.0 mg/l respectively. This supports the hypothesis that well REE DG is not located in the plume.

Well nests REW and REC were installed down-gradient of the drainfield in the summer of 1989 in an effort to better define the plume. The initial investigation indicated that well nest REC had chloride concentrations in excess of 10 mg/l and was located in the plume while well nest REW was not. It was suggested by VanRyswyk (1996) that preferential discharge was occurring within the first few meters of the drainfield (Figure 4.27). Reneau et al. (1989) reported that this phenomenon is not uncommon in highly permeable soils. In theory, the discharging zone of a soil absorption system should progress towards the end of the drainfield as a biological mat develops.

A transect of multiport wells (RSDS A-E) was installed in the summer of 1990 in an attempt to intersect the contaminant plume 38.1 m (125 ft) down-gradient of the drainfield (Figure 4.27). Results from these wells offered more evidence that discharge from the drainfield was creating a relatively narrow and

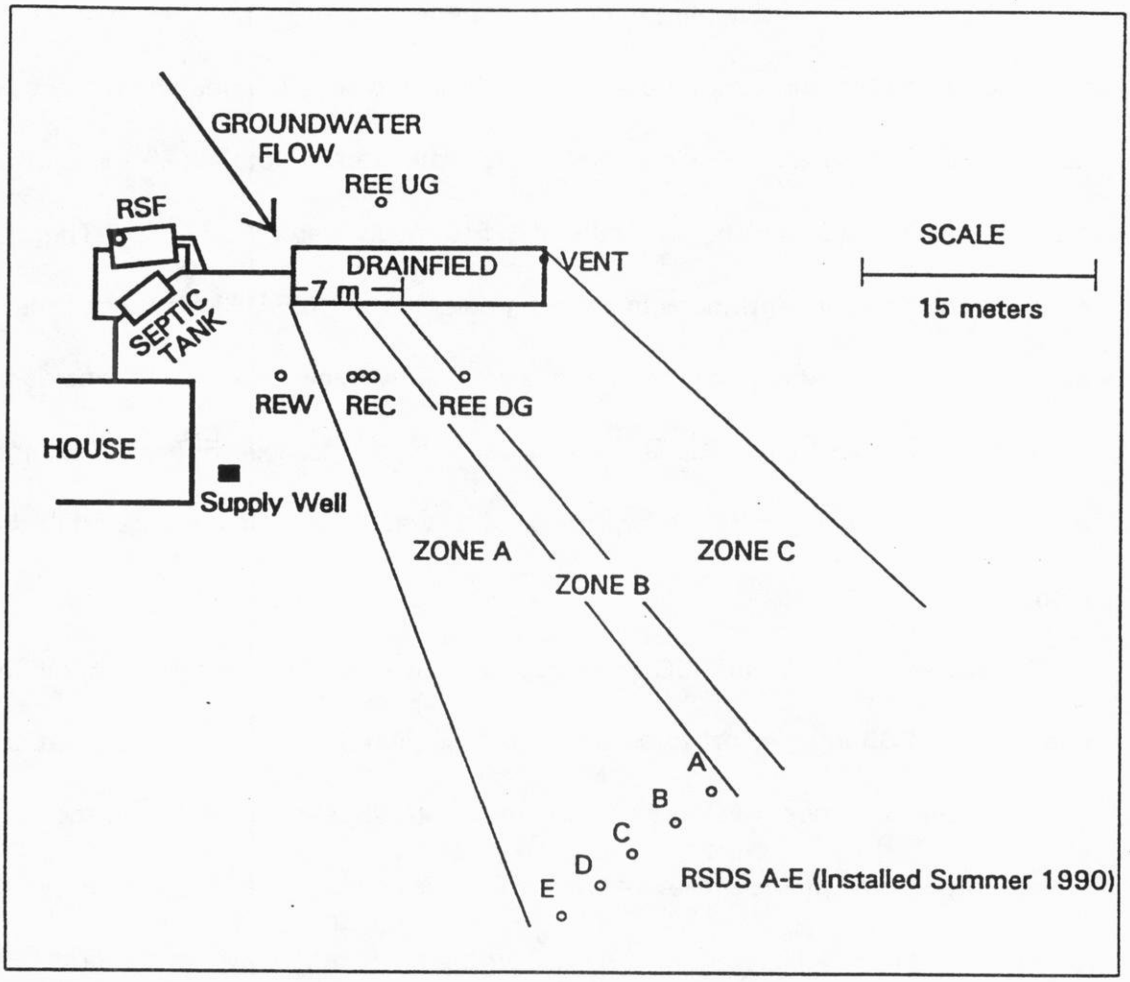


Figure 4.27 Preferential discharge of the drainfield at Site #1. Zone A is known to be impacted, Zone B may be impacted and zone C is not believed to be impacted.

thin plume. These wells were also used to compare nitrate-N: chloride ratios (N:Cl) with occurring in the saturated zone. VanRyswyk (1996) concluded that no significant attenuation of nitrogen occurred over the roughly 33.1 m (110 ft) flow distance between these wells. Denitrification is not likely to be significant at the shallow depths of this highly permeable, unconfined aquifer which is aerobic and lacks a sufficient carbon supply.

In order to determine the VOC impacts on groundwater and to sample nitrate-N during high water table conditions, two additional wells were installed. Well REC V was installed October 26, 1993 during the high water table levels encountered in 1993. The water table stayed within the three foot screened interval of this well until September, 1994 when water table levels dropped by at least 0.2 m (0.5 ft). In response to the lowering of the water table well, REC V2 was installed December 1, 1994. The water table stayed within the three foot screened interval of this well through the end of this project (June 29, 1995).

4.182 Water Table Fluctuations.

The water table at Site #1 fluctuated between 4.1 and 6.1 m (13.6 to 20.0 ft) below the ground surface at well nest REC. This amounts to approximately 3.2 to 5.2 m (10.6 to 17.0 ft) of unsaturated soil beneath the soil absorption system. VanRyswyk (1996) and Osesek (unpublished) observed a plume thickness of approximately 1.0 m (3.3 ft) at well nest REC. The water table fluctuated up to 1.4 m (4.7 ft) above the top of the shallowest port of well REC between the falls of 1993 and 1994 (Figure 4.28). When the water table was above the screened interval of the well, the most concentrated portion of the plume was not captured.

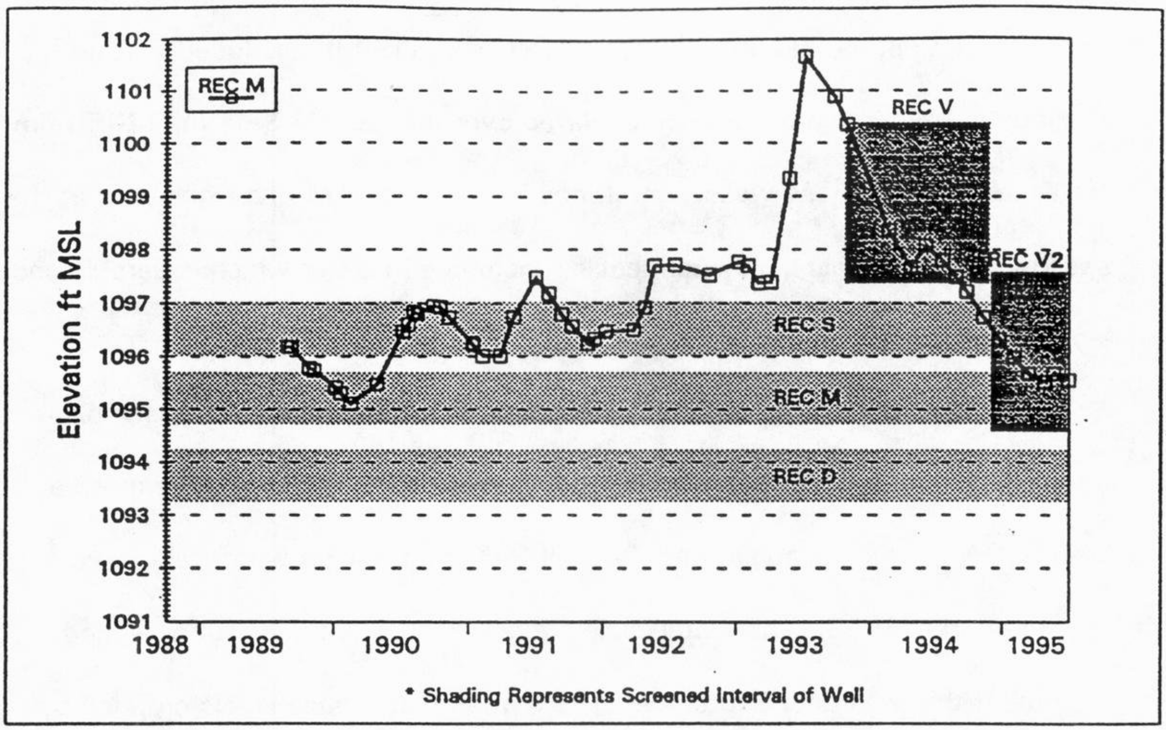


Figure 4.28 Watertable fluctuations and screened interval of each well at well nest REC, 1988-1995, Site #1. 4.6 m (16 ft) down-gradient.

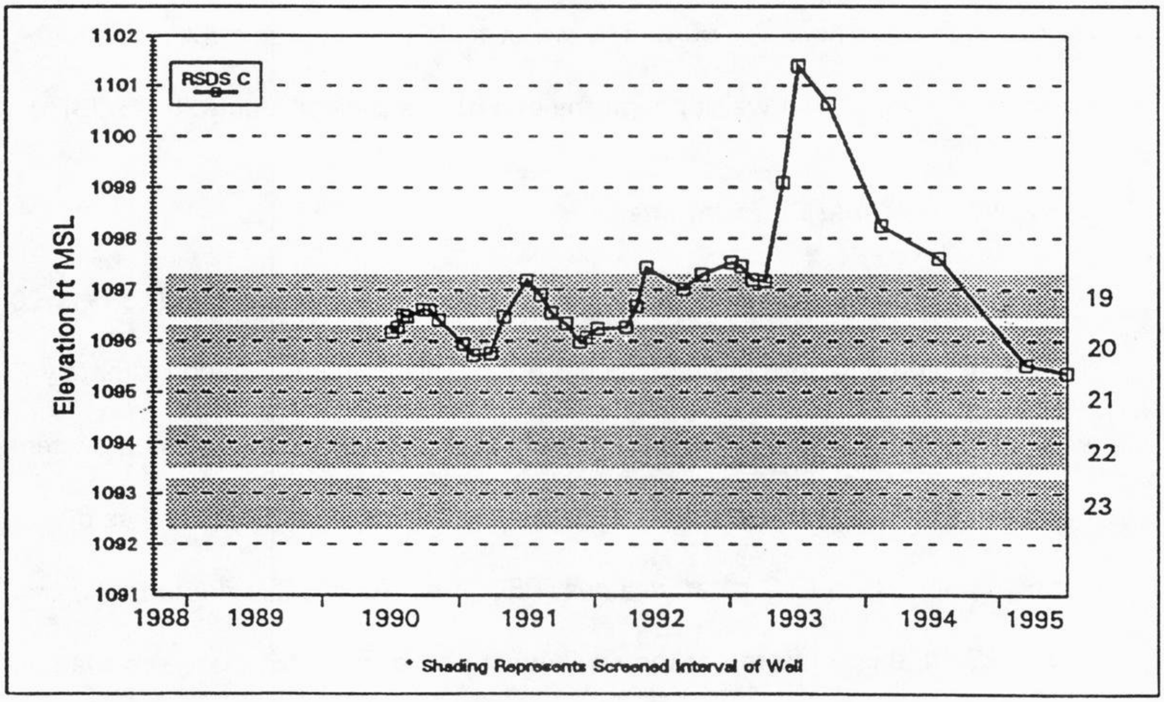


Figure 4.29 Watertable fluctuations and screened interval of each port for multiport well RSDS C, 1988-1995, Site #1. 38.1 meters (125 ft down-gradient).

However, the addition of wells REC V and REC V2 did allow shallow samples to be collected when the water table when levels were above REC S (Figure 4.28).

In the transect of multiport wells, well RSDS C was the most impacted. The water table fluctuations and screened interval of well RSDS C is shown in Figure 4.29. Since wells REC and RSDS C were both located within the plume they are used to determine any changes in groundwater quality resulting from the enhanced effluent treatment provided by the RSF.

4.183 Nitrate-N Reductions

Well Nest REC (4.6 meters down-gradient)

The concentrations of nitrate-N and chlorides observed in wells REC are shown in Figures 4.30 and 4.31, respectively. As previously stated, when the water table was above the screened interval of the wells the most concentrated portions of the plume were not captured, making it difficult to compare the conditions before and after the installation of the RSF. However, if sample dates of similar water table levels within the screened elevations are compared, conclusions regarding the changes in groundwater quality resulting from nitrogen removal by the RSF system can be made.

Tables 4.23 and 4.24 show the average nitrogen (TN or nitrate-N) and chloride concentrations in the septic tank, the RSF, well nest REC, and up-gradient well REE UG. The groundwater averages were obtained from the sample date(s) which had similar water table elevations. Shading in the table indicates that water table elevations were within the screened interval of the well. Samples for groundwater conditions were collected less frequently than septic tank and RSF

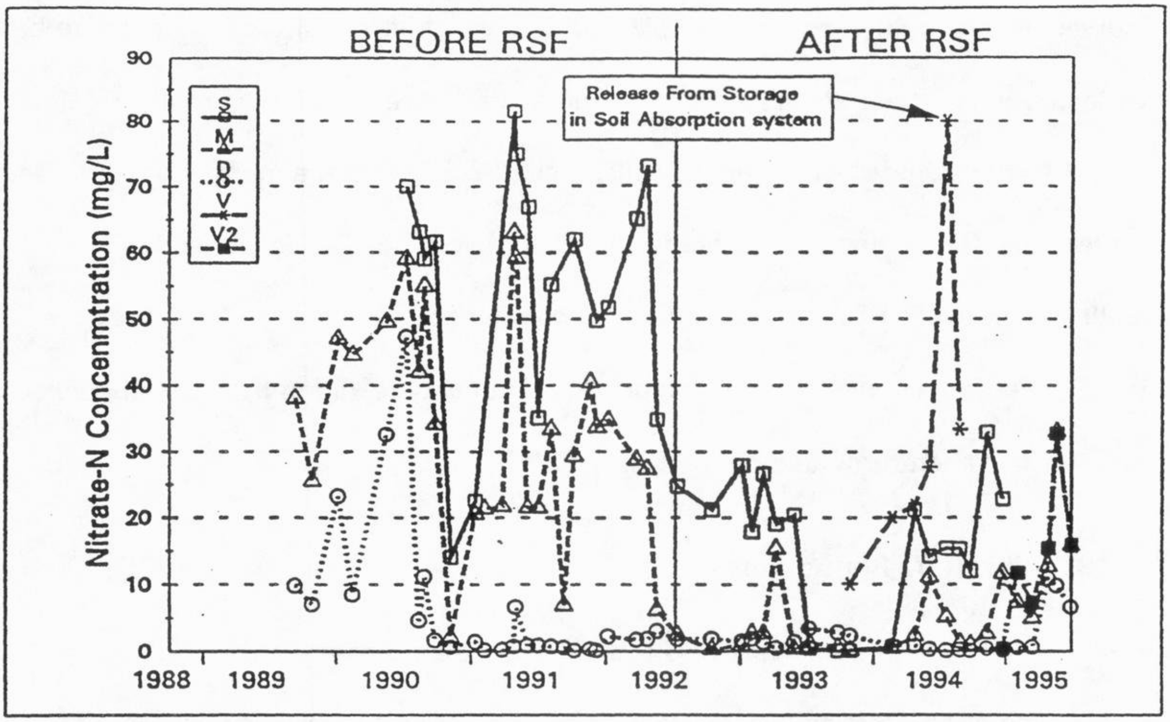


Figure 4.30 Nitrate-N concentrations in well nest REC, 1988-1995, Site #1. 4.6 m (16 ft) down-gradient.

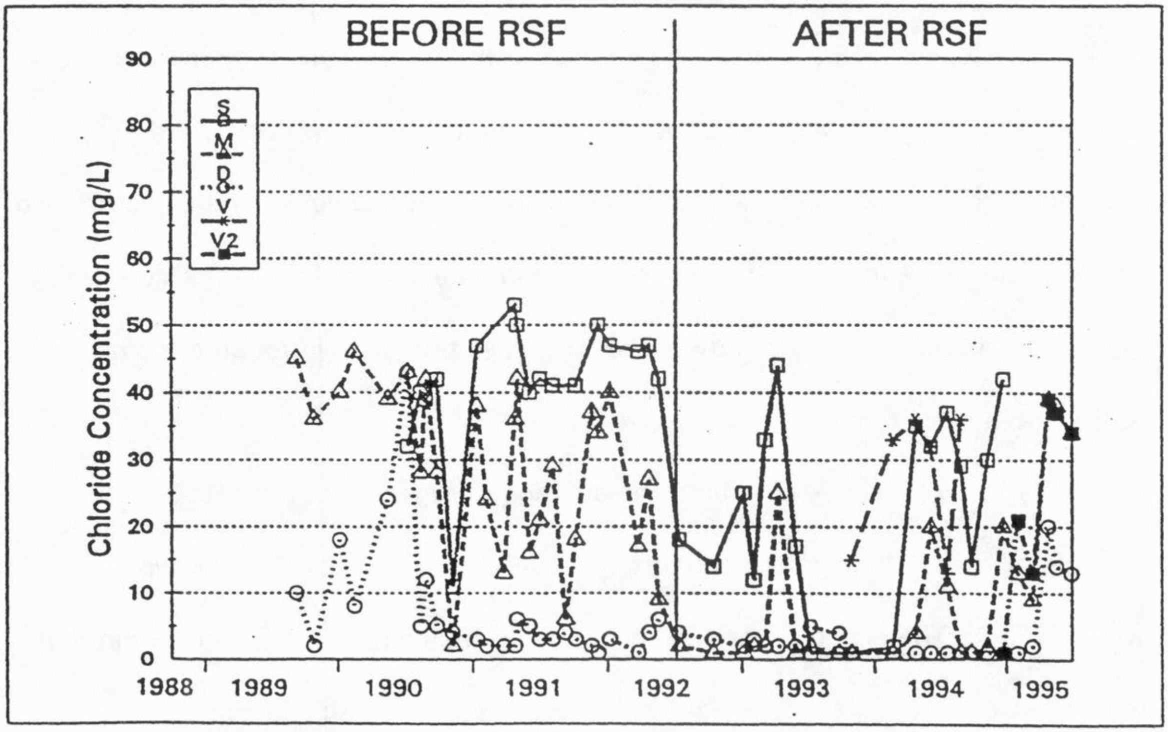


Figure 4.31 Chloride concentrations in well nest REC, 1988-1995, Site #1. 4.6 m (16 ft) down-gradient.

Table 4.23 Average Nitrogen Concentration (TN or Nitrate-N) in the septic tank, RSF, well nest REC, and well REE UG at Site #1, 1989 to 1995.

			Total N (mg/l) Discharge		Nitrate-N (mg/l) Well					
	Start	End	Septic Tank	RSF	REC S	REC V	REC V2	REC M	REC D	REE UG
B	8/28/89	9/8/89			-			38	9.8	1.3
E	10/26/89	11/08/89			-			25.5	7.0	0.2
F	1/5/90	4/26/90			-			47	21.4	0.3
O	7/5/90	6/3/91			57.1			39.6	8.3	0.7
R	7/1/91	8/5/91			45.2			27.5	0.8	0.8
E	9/9/91	4/23/92	82.3		60.3			32.3	1.0	0.9
	5/20/92	7/15/92	60.0		29.8			4.3	2.4	2.3
A	10/16/92	4/6/93	40.5	26.1	22.5			4.4	1.4	1.2
F		5/26/93	33.9	17.9	20.4			0.4	1.4	0.5
T		7/7/93	30.8	16.5	0.5			1.3	3.4	0.9
E	9/24/93	10/28/93	34.8	18.6	0.1	10.1		0.4	2.6	NS
R		2/18/94	30.1	18.0	0.7	20		0.4	0.8	1.5
	4/24/94	8/24/94	34.1	31.4	16.5	40.6		5.0	0.4	0.8
R		9/22/94	49.9	30.7	12.0	-		1.0	0.1	0.6
S	11/7/94	12/21/94	53.0	20	27.9	-		7.2	0.3	1.2
F		1/31/95	28.0	24.1	-	-	11.7	7.3	0.5	1.1
	3/13/95	6/29/95	30.1	19	-	-	17.8	16.7	7	1.1

Note: Shading indicates average values for water table levels within the screened intervals of the well ports, the upper most portion of the contaminant plume.

Table 4.24 Average Chloride Concentrations in the septic tank, RSF, well nest REC, and well REE UG at Site #1, 1989 to 1995.

			Chloride (mg/l) Discharge		Chloride (mg/l) Well					
	Start	End	Septic Tank	RSF	REC S	REC V	REC V2	REC M	REC D	REE UG
B	8/28/89	9/8/89			-			45.0	10	4.0
E	10/26/89	11/08/89			-			36.0	2	1
F	1/5/90	4/26/90			-			41.7	16.7	4
O	7/5/90	6/3/91			39.3			30.5	9.4	2.5
R	7/1/91	8/5/91			41.5			25.0	3.0	4.5
E	9/9/91	4/23/92	54.7		40.2			28.8	2.2	3.1
	5/20/92	7/15/92	53.5		30.0			5.5	5.0	6.5
A	10/16/92	4/6/93	35.7	30.9	25.6			6.1	2.4	3.0
F		5/26/93	30.8	31.4	17.0			1	2.0	1
T		7/7/93	31.7	28.6	1			2	5.0	2.0
E	9/24/93	10/28/93	29.7	27.3	1	15.0		1	2.5	NS
R		2/18/94	28.5	32.4	2	33		1	1	2.0
	4/24/94	8/24/94	38.9	36.0	33.3	34.7		8.9	1	1.2
R		9/22/94	52.3	31.5	14.0	-		1	1	1
S	11/7/94	12/21/94	43.0	27.3	36.0	-		11.0	1	1
F		1/31/95	36.2	38.0	-	-	21	13.0	1	1
	3/13/95	6/29/95	38.9	30.3	-	-	30.8	30.0	12.3	2.3

Note: Shading indicates average values for water table levels within the screened intervals of the well ports, the upper most portion of the contaminant plume.

samples; therefore averages for the septic tank and sand filter were made for all the sample dates prior to the last date which was used for the groundwater. Both the concentrations in the septic tank and sand filter are listed due to the problems encountered with effluent bypassing the RSF and the uncertainty in the concentrations applied. When bypass is occurring the concentration applied to the soil absorption system is likely to be in-between the concentrations observed in the septic tank and sand filter.

In the literature the ratios of the chloride concentrations in on-site system effluent to the chloride concentrations in the down-gradient wells (Cl:Cl) have been used to determine the degree of mixing/dilution that occurs as the effluent enters the groundwater and moves to the monitoring wells (Shaw and Turyk, 1992). Also reported in the literature is the ratio of TN concentrations in the on-site system effluent to the nitrate-N concentrations observed in the down-gradient wells (TN:N). Comparison of the Cl:Cl and TN:N ratios can indicate if nitrogen and chloride respond in a similar fashion. Differences in the ratios could be attributed to nitrogen removal via denitrification in the soil absorption system and/or in the groundwater. If the ratios are not appreciably different, it has been concluded that nitrogen and chloride respond in a similar fashion. It is widely accepted that chloride is conservative, non-reactive in nature; any changes in chloride concentrations are due to dilution/mixing only. When it has been determined that nitrogen and chloride respond in a similar fashion the ratio of nitrate-N concentrations in monitoring wells to the chloride concentrations in the monitoring wells (N:Cl) have been used to determine any attenuation of nitrogen as it moves through the soil and/or groundwater.

Prior to the installation of the RSF the TN:Cl and Cl:Cl ratios ranged between 1.3 and 1.8 over the 4.6 m (16 ft) distance to well nest REC. The effluent bypassing the system and water table fluctuations make it difficult to determine the amount of dilution/mixing after the installation of the RSF. However, when the ratios for dilution/mixing were compared between similar water table elevations as those seen before the installation of the RSF, no appreciable difference was found. Since it appears that nitrogen and chloride responded the same when similar water table elevations were compared, the N:Cl ratio can be used to compare the changes in groundwater quality resulting from the installation of the RSF system.

Nitrogen:Chloride (N:Cl) ratios were determined for the average concentrations in the septic tank, the RSF, well nest REC, and up-gradient well REE UG (Table 4.25). A reduction in the N:Cl ratio between the conditions observed before and after the installation of the RSF are due to denitrification by the RSF system. The ratios averaged 1.5 before and 0.7 after the installation of the RSF. When the shaded values are compared to the discharging unit (septic tank or RSF) it appears that no appreciable changes in nitrogen occurred between the RSF and monitoring wells during most of the sampling periods. TN removal for all the study periods used to determine effluent treatment by the RSF system was 65.3%. The average N:Cl ratios in well nest REC, which represents only sampling periods when the water table was within the screened interval of the monitoring wells, indicate a

Table 4.25 Average Nitrogen:Chloride (TN or N:Cl) ratios for the septic tank, RSF, well nest REC and well REE UG at Site #1.

			Discharge [1][a]		Well [2][b]						WT ft above 1096 ft *
	Start	End	Septic Tank	RSF	REC S	REC V	REC V2	REC M	REC D	REE UG	
B	8/28/89	9/8/89			-	-	-	0.8	1.0	0.3	0.18
E	10/26/89	11/08/89			-	-	-	0.7	3.5	0.2	-0.24
F	1/5/90	4/26/90			-	-	-	1.1	1.3	0.1	-0.67
O	7/5/90	6/3/91			1.5	-	-	1.3	0.9	0.3	0.61
R	7/1/91	8/5/91			1.1	-	-	1.1	0.3	0.2	1.32
E	9/9/91	4/23/92	1.5		1.5	-	-	1.1	0.5	0.3	0.53
	5/20/92	7/15/92	1.1		1.0	-	-	0.8	0.5	0.4	1.7
A	10/16/92	4/6/93	1.1	0.8	0.9	-	-	0.7	0.6	0.4	1.55
F		5/26/93	1.1	0.6	1.2	-	-	0.4	0.7	0.5	3.33
T		7/7/93	1.0	0.6	0.5	-	-	0.7	0.7	0.5	5.66
E	9/24/93	10/28/93	1.2	0.7	0.1	0.7	-	0.4	1.0	NS	4.62
R		2/18/94	1.1	0.6	0.4	0.6	-	0.4	0.8	0.8	2.36
	4/24/94	8/24/94	0.9	0.9	0.5	1.2	-	0.6	0.4	0.7	1.78
R		9/22/94	1.0	1.0	0.9	-	-	1.0	0.1	0.6	1.21
S	11/7/94	12/21/94	1.2	0.7	0.8	-	-	0.7	0.3	1.2	0.53
F		1/31/95	0.8	0.6	-	-	0.6	0.6	0.5	1.1	-0.01
	3/13/95	6/29/95	0.8	0.6	-	-	0.6	0.6	0.6	0.5	0.39

Note: Shading indicates averages values for water table levels within the screened intervals of the well ports.

- Indicates no sample available (Well not yet installed or insufficient volume in well)

1 Average values for the groundwater samples based on date(s) on which samples were collected.

2 Average values for the septic tank and sand filter are based on the all the sample dates prior to the last date on which the groundwater sample(s) were collected.

* Average height of the water table above 1096 ft MSL (the point elevation of REC S). Values between 0.01 and 1 are within the screened interval of REC S.

a Total Nitrogen:Chloride

b Nitrate-N:Chloride

53% reduction of nitrate-N. It would be expected that the reduction seen in the groundwater would have been greater if bypass had not occurred.

An increase in the N:Cl ratio is apparent for well REC V for the sample dates between April 24 and August 24, 1994. The ratio seen in the monitoring well is greater than the ratio seen in the septic tank or RSF. This can be attributed to the release of nitrogen from storage in the soil absorption system as the BOD_5 loading was decreased by secondary treatment in the RSF. Unfortunately, the study design did not include detailed monitoring of the of the conditions within and immediately beneath the soil absorption system. This, combined with the elevated water table levels and bypass of the system, make it impossible to quantify any release of nitrogen which may have occurred when the plume was not captured by the monitoring wells.

Despite the nitrogen removal which was achieved by the RSF system, nitrate-N concentrations at 4.6 m (16 ft) down-gradient of the drainfiled still exceed the drinking water standard of 10 mg/l. TN concentrations applied to the drainfield by the discharging unit (septic tank or RSF) were not below 10 mg/l due to the relatively high household concentrations entering the system reflecting the conservative water usage by the residents. However, if bypass of the RSF had not occurred, these concentrations would have been considerably lower.

Multiport Wells RSDS (38.1 meters Down-gradient)

From 1990 to 1991 samples were collected monthly from wells RSDS (A-E) by VanRyswyk (1996). From 1992 to the fall of 1993 the frequency of sample collection was reduced to approximately once every two months by Osesek (unpublished). This project reduced the frequency of sample collection even further to once every six months. This resulted in insufficient data collected after the installation of the RSF to draw any significant conclusions regarding its effect on down-gradient groundwater quality.

The water table fluctuations in the down-gradient multiport wells RSDS (Figure 4.29) obscure any changes in groundwater quality that resulted from the installation of the RSF system at Site #1. In 1993 the water table rose approximately 1.2 m (4 ft) above the screened elevation of the shallowest well port of all wells in transect RSDS (A-E). Following a peak in 1993, the water table continued to drop through 1994 and stabilized in the summer of 1995.

The average hydraulic gradient between wells REC and RSDS C was 2.26 x 10^{-3}. The gradient varied with the water table fluctuations between July, 1990 and June 1995, decreasing from 2.91 x 10^{-3} to 1.09 x10^{-3}. Assuming an effective porosity (n_e) of 0.30 and a hydraulic conductivity (K) of 0.05 cm/sec (142 ft/day) the estimated average linear velocity (v) is 0.32 m/day (1.1 ft/day). The estimated travel time for the 33.5 m (110 ft) distance is 103 days (range 80 to 214 days), or 3.4 months (range 2.6 to 7.0 months). In general, the travel time increased from 1990 to 1995.

The concentrations of nitrate-N and chlorides observed in the transect of multiport wells located 38.1 m (125 ft) down-gradient of the soil absorption system both before and after the installation of the RSF system are included in Figures 4.32 and 4.33, respectively. Travel times between well REC and RSDS C were not accounted for in determination of the average values. It should be noted 22 sample dates were used to determine the average values for before the installation of the RSF: only seven samples dates were used for calculation of the averages for after the installation of the RSF.

Due to the water table fluctuations above the screened intervals of the wells, it is difficult to compare the values seen in well nest REC and multiport well RSDS C. In general, the concentrations observed in wells RSDS are less than those seen in well REC, due to dispersion and further dilution of the contaminant plume as the groundwater moves the additional 33.5 m (110 ft) down-gradient. As seen in Figures 4.32 and 4.33, the concentrations in the plume at 38.1 m down-gradient before the installation of the RSF system were greater than those seen after the installation of the RSF system; average nitrate-N concentrations were reduced by an average of at least 50%. The number of well ports exceeding the 10 mg/l nitrate-N standard were reduced from twelve to two. If bypass of the RSF had not occurred, it would be expected that no well port would have exceeded the standard at this distance.

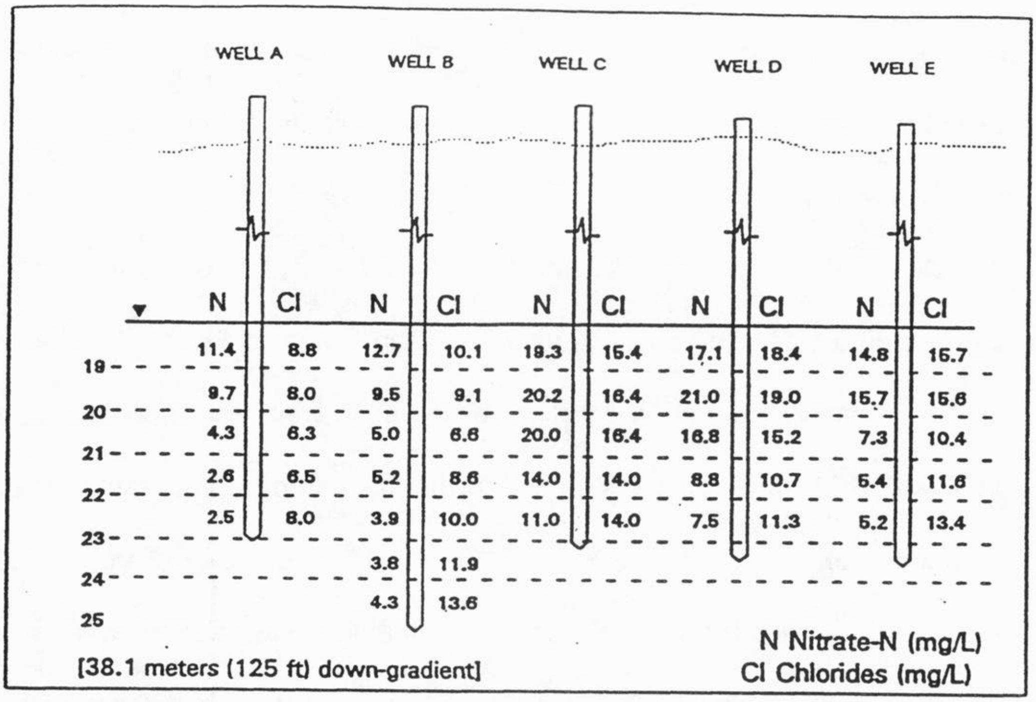


Figure 4.32 Average nitrate-N and chloride concentrations in RSDS multilevel wells BEFORE the installation of the RSF. Includes sampling dates from June 20, 1990 to August 28, 1992.

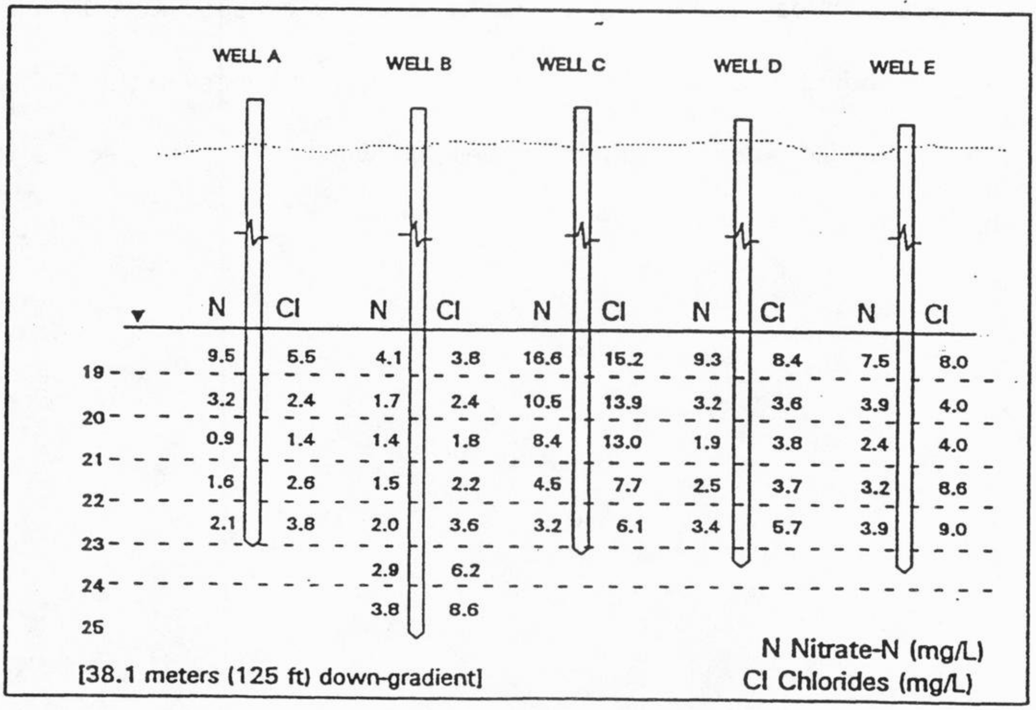


Figure 4.33 Average nitrate-N and chloride concentrations in RSDS multilevel wells AFTER the installation of the RSF. Excludes sampling dates between July 7, 1993, and July 18, 1994.

4.184 Phosphate

Figure 4.34 depicts phosphate concentrations in well nest REC located 4.9 m (16 ft) down-gradient of the drainfield. Phosphate concentrations began to increase significantly in 1994 and continued to rise through June, 1995. The movement of phosphate though soil is strongly dependent on the number and types of adsorption sites available on the soil particles. The continuous loading of phosphate to the soil causes the adsorption sites to become saturated and phosphate is allowed to move down-gradient. The hydraulic loading to the soil absorption system was approximately 378.5 lpd (100 gpd) from 1988 through 1995. The original conventional septic system was installed at Site #1 in the summer of 1982. Since no significant levels of phosphate were detected in well nest REC prior to 1994, it appears that the breakthrough of phosphate at 4.9 m (16 ft) occurred after 12 years of system usage. Up-gradient concentrations in well REE UG showed no considerable increase in reactive phosphate (Figure 4.35).

There are three causes for uncertainty in the explanation of phosphate breakthrough at 4.9 m (16 ft) down-gradient. (1) TP concentrations in the septic tank and sand filter reached respective maximum values of 23.7 mg/l and 11.2 mg/l on July 1, 1994. These were the highest values observed between 1991 and 1995. (2) Groundwater samples were not analyzed for phosphate between the summer of 1991 and the summer of 1993. (3) The sample taken in 1993 occurred when the water table was 1.4 m (4.6 ft) above the screened interval of REC S. Breakthrough may have already occurred at this time, but the sample is not

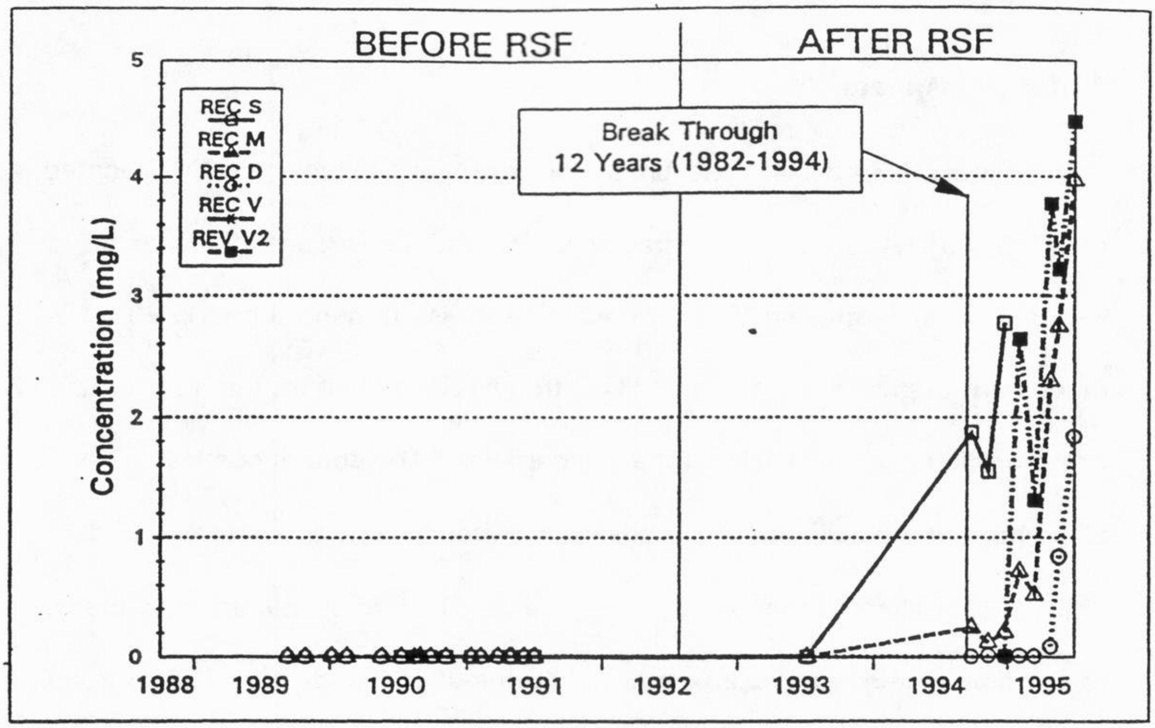


Figure 4.34 Phosphate concentrations in well nest REC, 1988-1995. 4.9 meters (16 ft) downgradient of the drainfield.

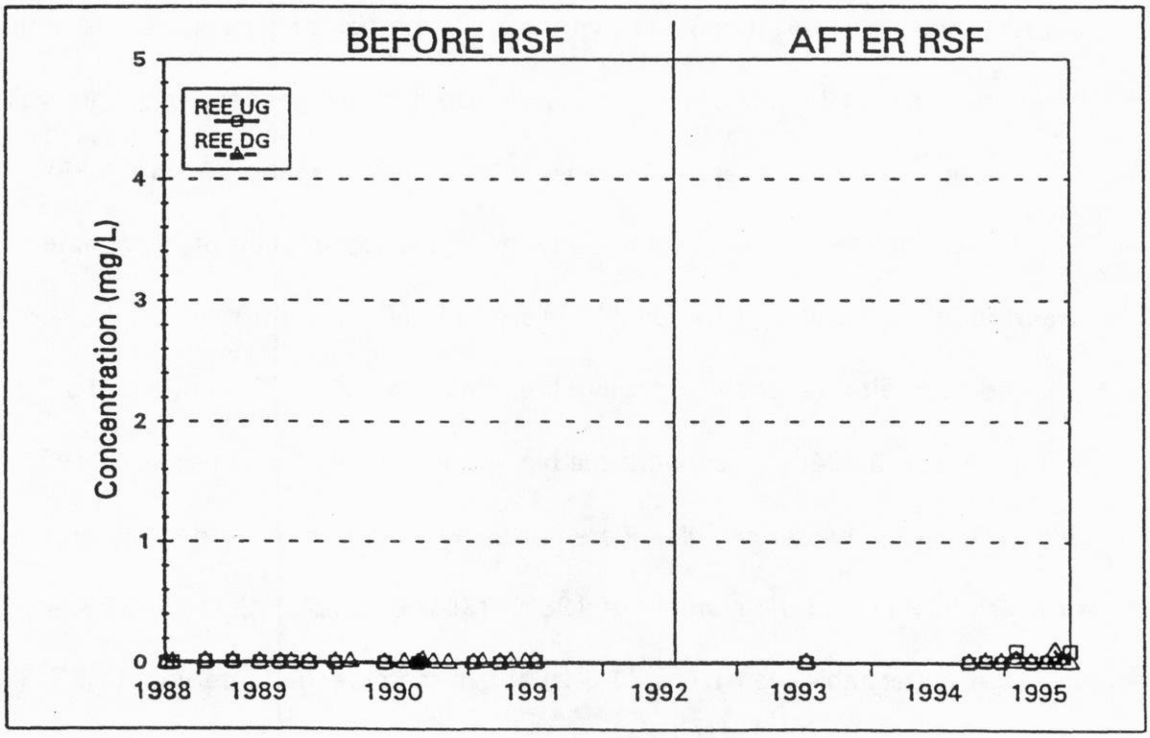


Figure 4.35 Phosphate concentrations in wells REE DG and REE UG, 1988-1995.

representative of the plume since the plume thickness at this distance was only 1.0 m (3.3 ft) below the water table.

Figure 4.36 depicts reactive phosphate concentrations in multiport well RSDS C located 38.1 m (125 ft) or 33.5 m (110 ft) down-gradient of the drainfield or well nest REC, respectively. A slight increase in reactive phosphate concentrations is apparent. However, these concentrations are still only slightly above the quantitation limit of 0.002 mg/l, compared to concentrations above 4 mg/l observed in well REC V2. It is suspected that the volume of soil between well nest REC and multiport well RSDS C is adsorbing the reactive phosphate, therefore breakthrough at 38.1 m (125 ft) down-gradient of the drainfield has not yet occurred.

4.185 Volatile Organic Compounds

Samples were collected from either well REC V or REC V2 on five occasions between July, 1993 and June, 1995. Samples collected were representative of water table levels within the screened intervals of the wells. No VOCs were detected above the limits of quantitation by EPA Method 8021. It appears that the soil absorption system or the 3.2 to 5.2 m (10.6 to 17.0 ft) of unsaturated soil beneath the soil absorption system were effective at removing any VOCs which were not removed by the sand filter.

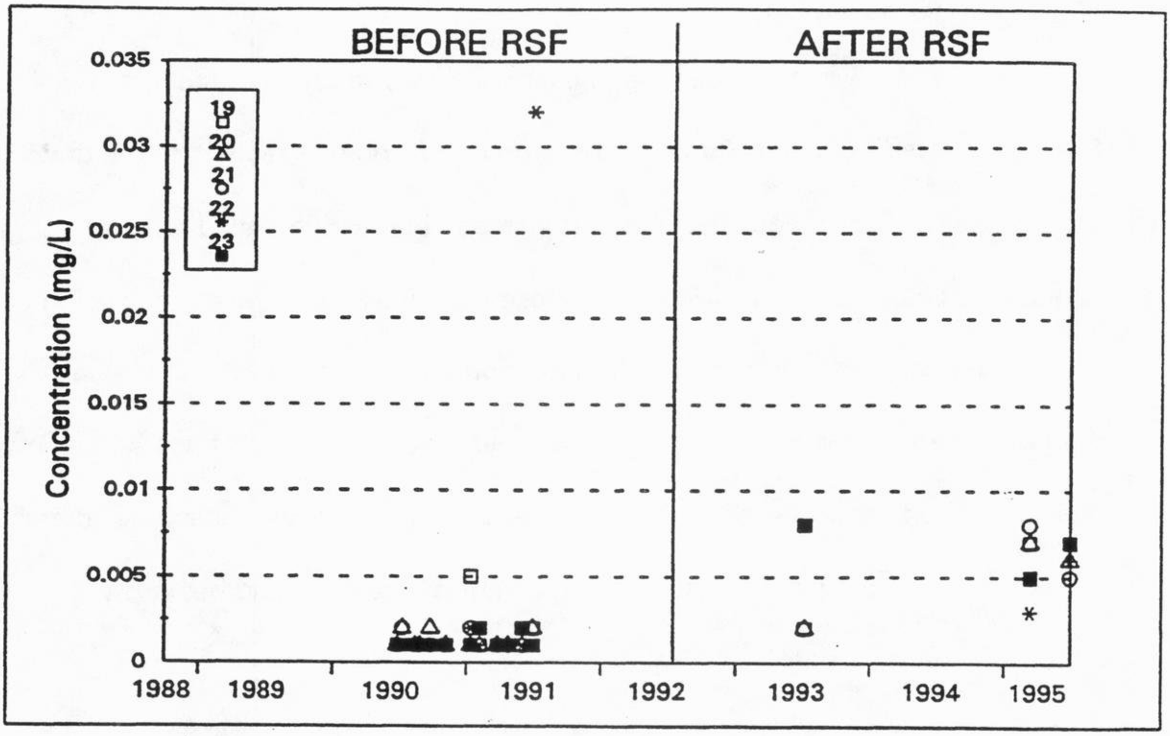


Figure 4.36 Phosphate concentrations in multiport well RSDS C, 1988-1995. 38.1 meters (125 ft) down-gradient.

4.2 Study Site #2

This section describes the results of the two projects involving the design and optimization of the RSF system for reducing nitrogen loading to groundwater at Site #2. The initial project involving the design and initial monitoring of the RSF system was conducted by Osesek and Shaw (unpublished) between September 1, 1991 and August 31, 1993. Prior to the installation of the RSF investigations at Site #2 conducted from September 1, 1991 to August 12, 1992 involved monitoring of the original dosing chamber and groundwater quality characteristics. After the installation of the RSF system on August 13, 1992 both the RSF system and effluent loading to groundwater were monitored during study periods #1, #2, and #3.

The objective of the project conducted by this author between September 1, 1993 and June 29, 1995 was to determine the optimal nitrogen removal capability of the RSF system. To do this, variations were made in the recirculation rates and in the frequency and duration of pumping events. During this project 19 study periods (A-S) of varying duration were observed.

4.21 Temperature Variations

The temperatures observed in Dosing Chamber #1, the pump chamber of the sand filter, and Dosing Chamber #2 from 1992 to 1995 are shown in Figure 4.37. Temperatures in the pump chamber of the sand filter ranged from 9.5°C (January 5,1995) to 21.1°C (June 22, 1994). Temperatures in Dosing Chamber #1 ranged from 10.7°C (February 13, 1995) to 20.5°C (June 22, 1994). Temperatures in Dosing Chamber #2 ranged from 9.0°C (February 4, 1994) to 18.7°C (July 15, 1994). The temperatures at this site are greater than those observed at Site #1 due to the greater water usage and the deeper placement in the ground of the RSF and dosing chambers.

Unlike the previous site, only one set of thermocouple wires was located in the RSF, so horizontal variations were not determined. Temperature readings for all depths of the thermocouple wires are shown in Figure 4.38. Again, the deepest probe is the warmest in winter, approximately 5°C warmer than the top.

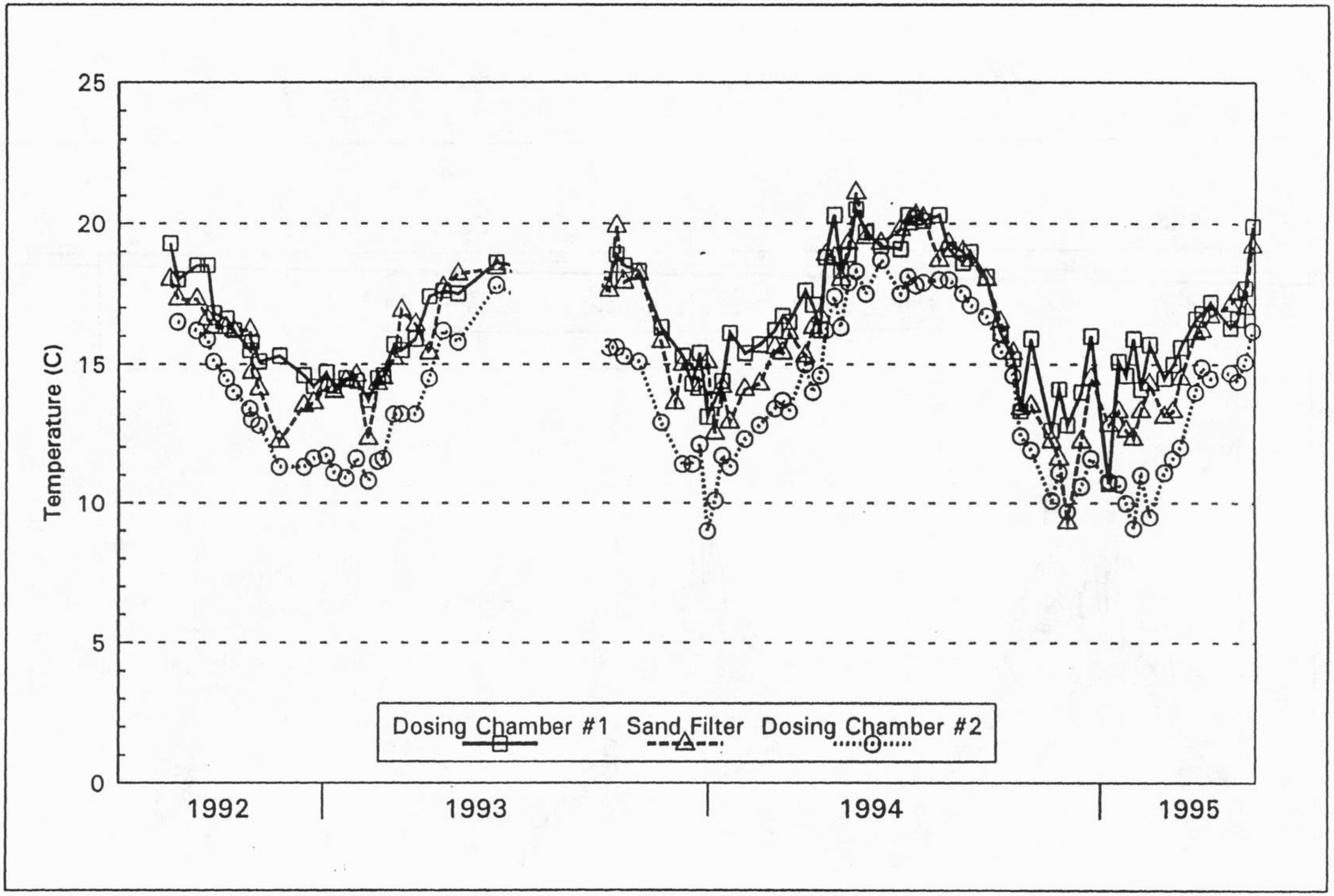


Figure 4.37 Temperature readings in dosing chamber #1, the pump chamber of the sand filter, and dosing chamber #2, at Site #2, 1992-1995.

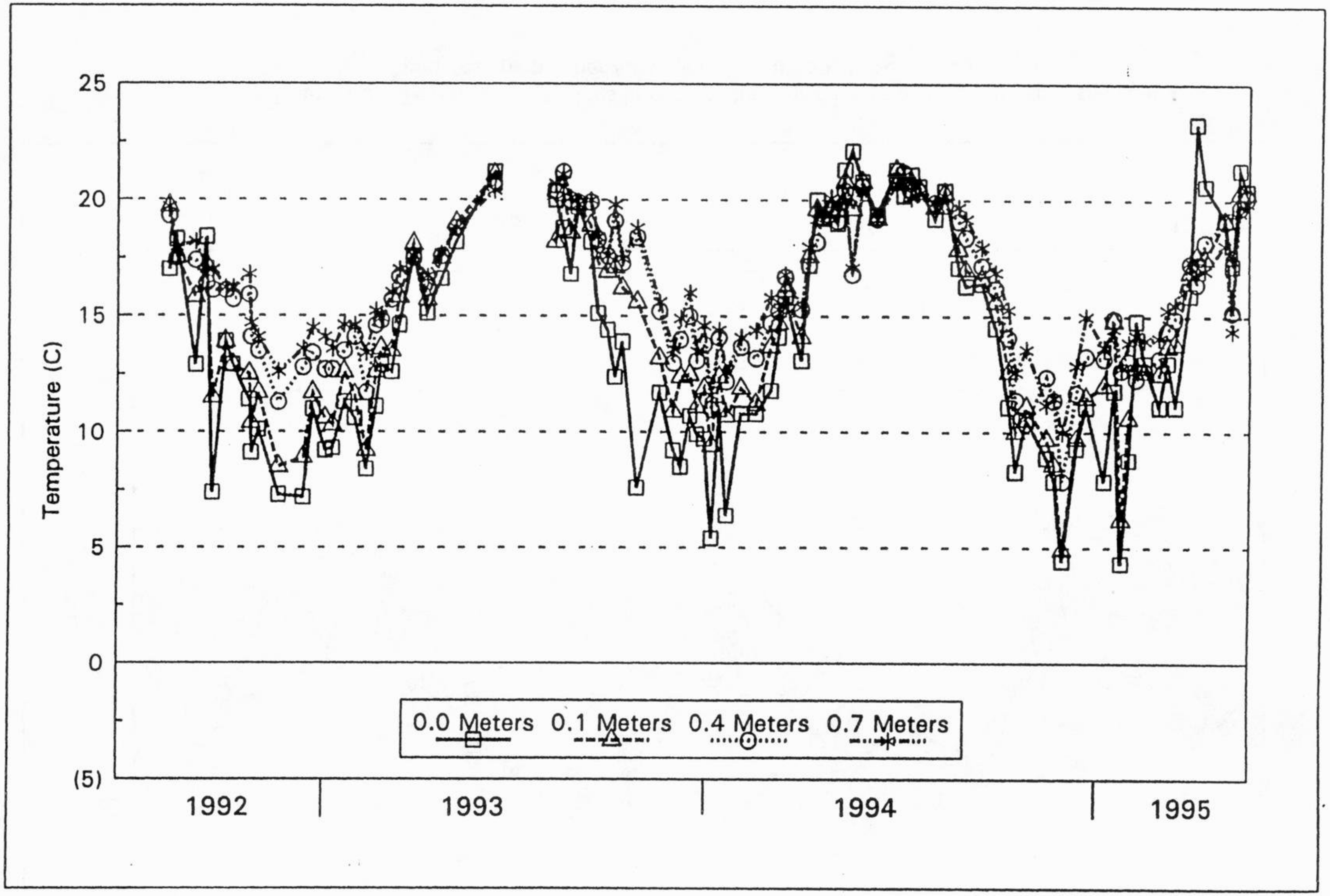


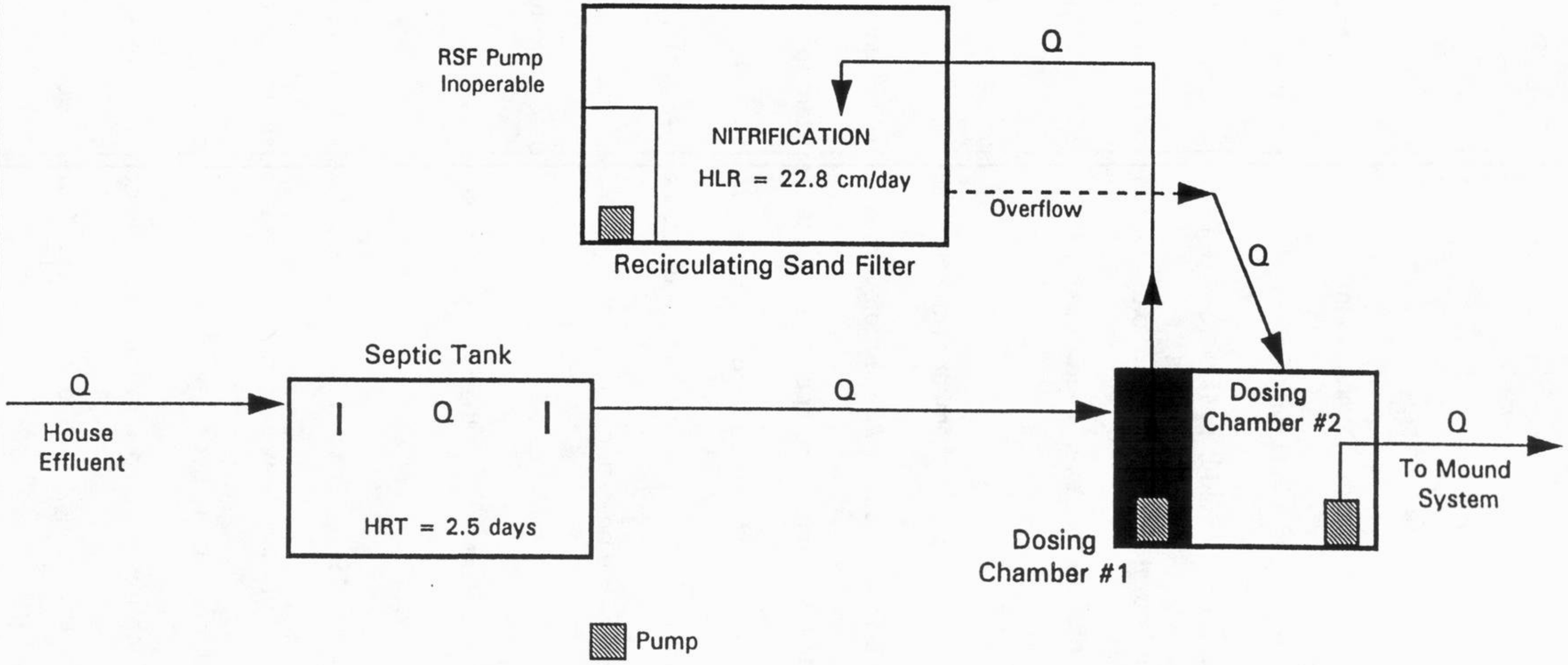
Figure 4.38 Temperature readings for all depths in the RSF, 1992-1995, Site #2.

4.22 Flow Conditions

In the design of this system wastewater is applied to the top of the RSF from two distributions systems: from Dosing Chamber #1, and from the pump chamber of the RSF. If the pump in Dosing Chamber #1 were to malfunction, untreated septic tank effluent could bypass directly to Dosing Chamber #2 through the overflow pipe built into the sewage ejector pit. Over the course of monitoring, the pump in Dosing Chamber #1 never failed, thus no untreated household effluent bypassed the system.

When the pump in the pump chamber of the RSF fails to operate, effluent flows by gravity into Dosing Chamber #2 via the overflow level built into the RSF. From Dosing Chamber #2 effluent is pumped to the mound system for disposal. The conditions seen during study period #1 of the initial project represent treatment in the RSF for the time period from the installation of the RSF, August 13, 1992 to the time the pump was made operational, November 5, 1992. The concentrations in Dosing Chamber #1 during study period #1 represent undiluted household effluent that was not recirculated from the RSF. A flow schematic for study period #1 is included in Figure 4.39.

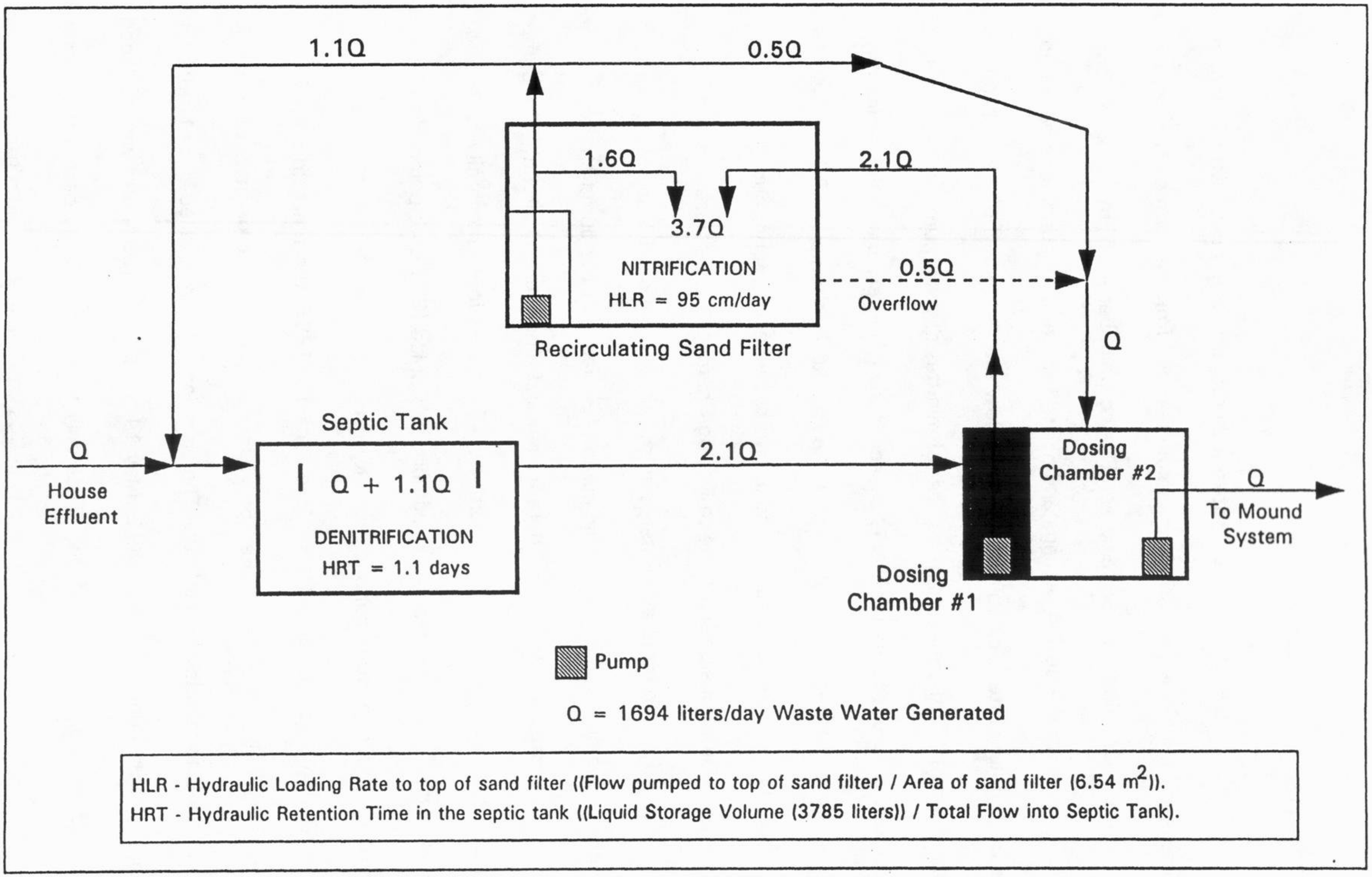
After November 5, 1992, when the timer controls were operational, effluent from the pump chamber of the RSF was pumped simultaneously to the top of the sand filter for nitrification, back to the septic tank for denitrification, and to Dosing Chamber #2 from which it was pumped to the mound system for disposal. The flow schematic for the scenario during study period #2 is shown in Figure 4.40.



Q = 1491 liters/day Waste Water Generated

HLR - Hydraulic Loading Rate to top of sand filter ((Flow pumped to top of sand filter) / Area of sand filter (6.54 m^2)).
HRT - Hydraulic Retention Time in the septic tank ((Liquid Storage Volume (3785 liters)) / Total Flow into Septic Tank).

Figure 4.39 Flow schematic for study period #1 at Site #2.



HLR - Hydraulic Loading Rate to top of sand filter ((Flow pumped to top of sand filter) / Area of sand filter (6.54 m^2)).

HRT - Hydraulic Retention Time in the septic tank ((Liquid Storage Volume (3785 liters)) / Total Flow into Septic Tank).

Figure 4.40 Flow schematic for study period #2 at Site #2.

4.221 Design Flow Conditions

Table 4.26 shows the flow rates and programmable timer settings which were used to achieve the variations in flow pumped from the pump chamber of the RSF: to the sand filter, septic tank, and Dosing Chamber #2 from 1992 to 1995. These represent the design settings and not the actual flow conditions achieved. The design flow rate is calculated from the flow rate through the flow control 'valves' (lpm) and the maximum number of minutes/day the pump will operate. The maximum number of minutes/day is based on the ON interval and 48 cycles/day. This assumes that volume of effluent in the RSF keeps the upper (DELAY interval) float up and that the pump runs through two cycles/hour every hour.

A different design flow rate to Dosing Chamber #2 was used for study periods J and K. To more accurately control the amount of effluent being pumped to Dosing Chamber #2, an additional pump was placed in the pump chamber in the RSF. This pump operated on a different float and timer control circuit. The pump was set to deliver 14.2 lpm (3.75 gpm) for 15 minutes four times daily to Dosing Chamber #2. This amounts to a maximum of 852 lpd (225 gpd) pumped to Dosing Chamber #2 during study periods J and K.

During study period R (May 12 to June 13, 1995) the timer controls for the pump in the pump chamber of the RSF were inoperable due to damage caused when an electrical relay burned out. The electrical relay had failed once before on June, 15 1994 during study period E (May 14 to July 15, 1994) and was replaced two days latter on June 17, 1994. Since only two days elapsed before the pump was turned back on, the concentrations in Dosing Chamber #1 did not have time to

Table 4.26 Settings of the programmable timers, design flow rates from the pump in the pump chamber of the RSF, and actual duration of pump operation for all study periods at Site #2.

	Programmable Timer Setting		Design	Flow Control Valve Setting			Design Flow Rates				Actual
Study Period	ON (min)	DELAY (min)	Pump Time* (min/day)	RSF (Liters/min)	Septic Tank (Liters/min)	DC #2 (Liters/min)	RSF (Liters/day)	Septic Tank (Liters/day)	DC #2 (Liters/day)	Actual Cycles/day	Pump Time (min/day)
#1	5	25	240	0	0	0	0	0	0	0	0
#2	5	25	240	11.36	7.57	3.79	2,725	1,817	908	48.0	240.0
#3	5	25	240	18.93	11.36	4.73	4,542	2,725	1,136	48.0	240.0
A	5	25	240	18.93	11.36	4.54	4,542	2,726	1090	48.0	240.0
B	8	22	384	18.93	11.36	4.54	7,268	4,361	1744	36.3	290.1
C	8	22	384	17.98	17.60	4.54	6,905	6,759	1744	30.9	247.0
D	5	25	240	17.98	17.60	6.81	4,315	4,225	1635	43.0	214.8
E	5	25	240	17.98	22.71	6.62	4,315	5,451	1590	48.0	240.0
F	5	25	240	26.50	22.71	6.44	6,359	5,451	1544	33.6	168.2
G	5	25	240	26.50	22.71	0	6,359	5,451	0	48.0	240.0
H	5	25	240	23.47	26.50	0	5,633	6,359	0	48.0	240.0
I	5	25	240	22.71	13.63	0	5,451	3,271	0	48.0	240.0
J	5	25	240	24.23	13.25	14.20 **	5,814	3,180	852 **	41.5	207.7
K	5	25	240	25.55	9.08	14.20 **	6,132	2,180	852 **	42.3	211.5
L	5	25	240	25.74	9.08	0	6,178	2,180	0	48.0	240.0
M	5	25	240	25.74	0	0	6,178	0	0	48.0	240.0
N	4	26	192	25.74	0	0	4,942	0	0	48.0	192.0
O	4	26	192	25.74	9.08	0	4,942	1,744	0	48.0	192.0
P	4	26	192	25.74	11.36	0	4,942	2,180	0	48.0	192.0
Q	4	26	192	18.93	11.36	4.54	3,634	2,180	872	48.0	192.0
R	0	0	0	0	0	0	0	0	0	0	0
S	3	27	144	18.93	13.25	4.54	2,725	1,908	654	48.0	144.0

* Based on maximum cycles per day; 2 cycles per hour = 48 Cycles/day (Assumes that effluent levels in RSF keep the upper (DELAY) float up)

** Additional pump in pump chamber of RSF pumping 14.2 Lpm to Dosing Chamber #2 for 15 minutes every six hours.

reestablish household conditions. Nonetheless, the data for June 15, 1994 were omitted from the averages for study period E.

The relay failure which occurred on May 12, 1995 caused extensive damage to the wiring in the timer control panel. This damage was repaired on June 13, 1995. Since flow was not pumped from the RSF to the septic tank for approximately 30 days, household concentrations were reestablished in Dosing Chamber #1. Thus, the data obtained between May 12 and June 13, 1995 is treated as a distinct study period, study period R.

Water Use Variations

The household water usage determines effluent levels in the sand filter and thus, controls the DELAY interval float which operates the pumping events. When the water usage exceeds the maximum design flow pumped from the sand filter to Dosing Chamber #2, overflow to Dosing Chamber #2 occurs. When water levels in the RSF are at the overflow level the pump runs through the maximum cycles/day. When water usage is less than the flow pumped to Dosing Chamber #2, water levels in the RSF drop below the DELAY interval float and the pump does not operate until water levels in the RSF rise. This results in less than maximum cycles/day and actual flows which are lower than design flows. The variability in the household water usage at Site #2 can be seen in Figure 4.41.

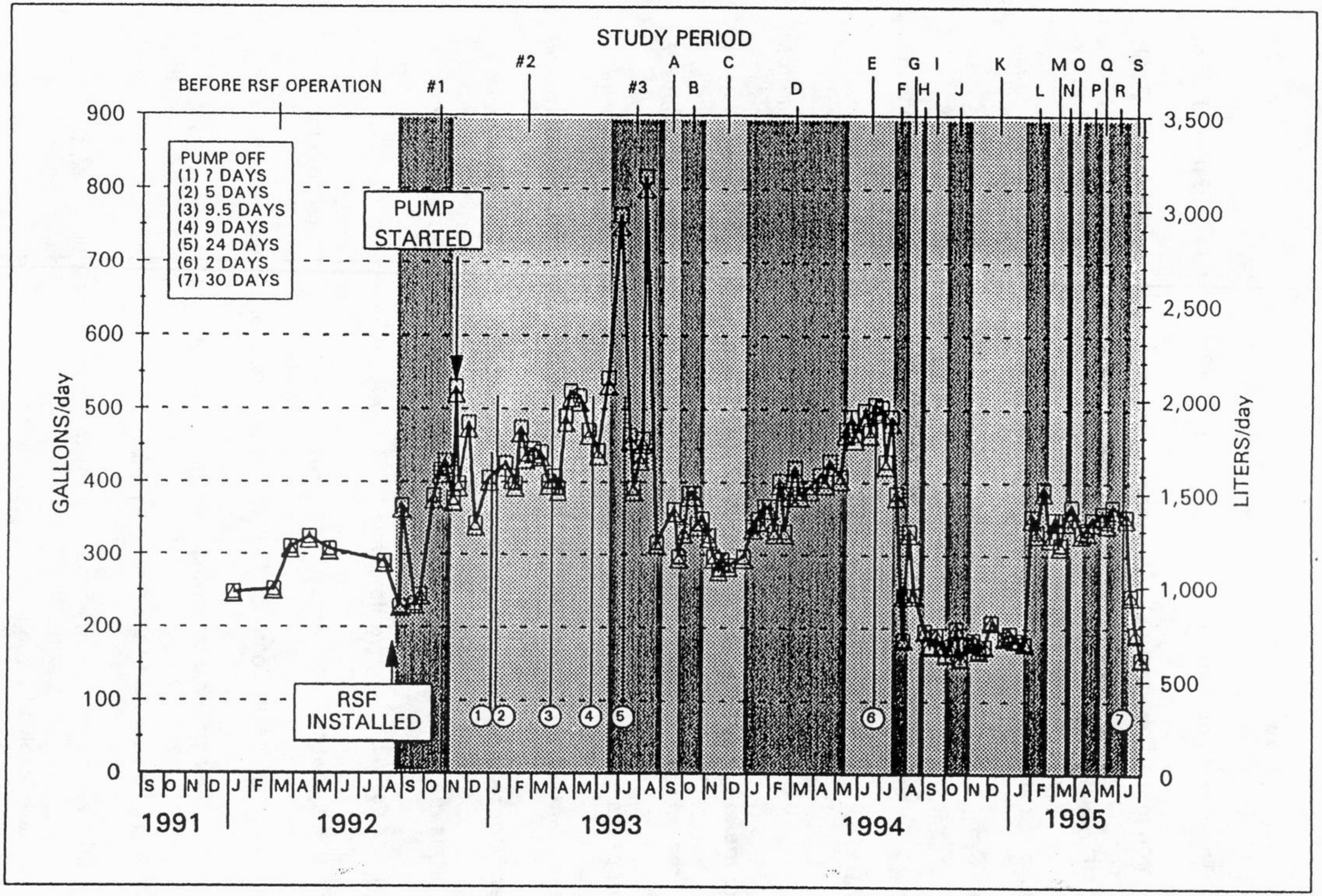


Figure 4.41 Water use at Site #2 from 1991 to 1995.

4.222 Actual Flow Conditions

The actual flow rates [average flow rates (lpd)] are determined from the flow rates through the flow control 'valves' and the average actual pumping time (Table 4.26). The average flow rates, the hydraulic loading rate (HLR) in cm/day, and the hydraulic retention time (HRT) in days are summarized for all the study periods at Site #2 in Table 4.27.

Table 4.28 contains the flow ratios for the average hydraulic conditions at Site #2. The forward flow (Q) through the system is defined as *the average household water usage*. The sand filter ratio (SFR) at Site #2 is defined as *the total flow to the top of the RSF to the forward flow (Q)*. The total flow to the top of the RSF is *the sum of the amounts pumped from the pump chamber of the RSF plus the amount of recycled to the septic tank and the forward flow (Q) from the house into the septic tank*. The septic tank ratio (STR) is *the ratio of the amount treated sand filter effluent recycled to the septic tank to the forward flow (Q)*. The discharge ratio (DR) is *the ratio of the flow pumped from the RSF to Dosing Chamber #2 to the forward flow(Q)*. The overflow ratio (OFR) is *the ratio of effluent exiting the RSF via the built in overflow level to the forward flow (Q)*. The total amount of effluent entering Dosing Chamber #2 is *the amount discharged from the pump in the RSF plus the amount overflowing,* this equals the forward flow (Q) pumped from Dosing Chamber #2 to the mound system.

Initially, problems were encountered with pump failures; however, these problems were resolved through the redesign of the timer control panel. Grains of sand occasionally clogged the orifices of the flow control 'valves'. Sample

Table 4.27 Average hydraulic conditions for all study periods at Site #2.

Study Period	Start	End	Forward Flow (L/day)	Total Flow to Top of Sand Filter (L/day)	HLR [a] (cm/day)	Flow to Septic Tank (L/day)	HRT [b] (days)	Flow to DC #2 (L/day)	Overflow to DC #2 (L/day)
#1	8/13/92	11/5/92	1,491	1,491	23	0	2.5	0	1,491
#2	11/6/92	6/15/93	1,694	6,236	95	1,817	1.1	908	786
#3	6/16/93	8/31/93	1,604	8,872	136	2,725	0.9	1,136	468
A	9/1/93	9/23/93	1,370	8,638	132	2,725	0.9	1,090	280
B	9/24/93	10/26/93	1,318	10,102	154	3,294	0.8	1,318	0
C	10/27/93	12/23/93	1,122	9,910	152	4,347	0.7	1,122	0
D	12/24/93	5/13/94	1,463	9,106	139	3,781	0.7	1,463	0
E	5/14/94	7/15/94	1,826	11,592	177	5,451	0.5	1,590	236
F	7/16/94	8/9/94	1,082	9,360	143	3,820	0.8	1,082	0
G	8/10/94	8/23/94	859	12,670	194	5,451	0.6	0	859
H	8/24/94	9/1/94	804	12,796	196	6,359	0.5	0	804
I	9/2/94	9/29/94	665	9,386	144	3,271	1.0	0	665
J	9/30/94	11/7/94	676	8,459	129	2,752	1.1	676	0
K	11/8/94	1/18/95	698	8,024	123	1,921	1.4	698	0
L	1/19/95	2/22/95	1,327	9,685	148	2,180	1.1	0	1,327
M	2/23/95	3/15/95	1,278	7,456	114	0	3.0	0	1,278
N	3/16/95	3/23/95	1,389	6,331	97	0	2.7	0	1,389
O	3/24/95	4/6/95	1,257	7,943	121	1,744	1.3	0	1,257
P	4/7/95	5/5/95	1,317	8,439	129	2,180	1.1	0	1,317
Q	5/6/95	5/11/95	1,310	7,124	109	2,180	1.1	872	438
R	5/12/95	6/13/95	1,364	1,364	21	0	2.8	0	1,364
S	6/14/95	6/28/95	749	5,383	82	1,908	1.4	654	95

[a] HLR = Hydraulic Loading Rate to top of RSF (Based on total amount of effluent applied to top 6.54 m^2 area of RSF).

[b] HRT = Hydraulic Residence Time within Septic Tank (Liquid Storage Volume (3,785 L) / Total Flow into Septic Tank).
Total Flow into Septic Tank = (Forward Flow + Flow to Septic Tank).

Table 4.28 Flow Ratios for the average hydraulic conditions for all study periods at Site #2.

Study Period	Start	End	Forward Flow (Q) (L/day)	Sand Filter Ratio (SFR)	Septic Tank Ratio (STR)	Discharge Ratio (DR)	Over Flow Ratio (OFR)	Forward Flow (Q)
#1	8/13/92	11/5/92	1,491	1.0	0.0	0.0	1.0	1.0
#2	11/6/92	6/15/93	1,694	3.7	1.1	0.5	0.5	1.0
#3	6/16/93	8/31/93	1,604	5.5	1.7	0.7	0.3	1.0
A	9/1/93	9/23/93	1,370	6.3	2.0	0.8	0.2	1.0
B	9/24/93	10/26/93	1,318	7.7	2.5	1.0	0.0	1.0
C	10/27/93	12/23/93	1,122	8.8	3.9	1.0	0.0	1.0
D	12/24/93	5/13/94	1,463	6.2	2.6	1.0	0.0	1.0
E	5/14/94	7/15/94	1,826	6.3	3.0	0.9	0.1	1.0
F	7/16/94	8/9/94	1,082	8.6	3.5	1.0	0.0	1.0
G	8/10/94	8/23/94	859	14.7	6.3	0.0	1.0	1.0
H	8/24/94	9/1/94	804	15.9	7.9	0.0	1.0	1.0
I	9/2/94	9/29/94	665	14.1	4.9	0.0	1.0	1.0
J	9/30/94	11/7/94	676	12.5	4.1	1.0	0.0	1.0
K	11/8/94	1/18/95	698	11.5	2.8	1.0	0.0	1.0
L	1/19/95	2/22/95	1,327	7.3	1.6	0.0	1.0	1.0
M	2/23/95	3/15/95	1,278	5.8	0.0	0.0	1.0	1.0
N	3/16/95	3/23/95	1,389	4.6	0.0	0.0	1.0	1.0
O	3/24/95	4/6/95	1,257	6.3	1.4	0.0	1.0	1.0
P	4/7/95	5/5/95	1,317	6.4	1.7	0.0	1.0	1.0
Q	5/6/95	5/11/95	1,310	5.4	1.7	0.7	0.3	1.0
R	5/12/95	6/13/95	1,364	1.0	0.0	0.0	1.0	1.0
S	6/14/95	6/28/95	749	7.2	2.5	0.9	0.1	1.0

Flow Ratios: Ratio of (Total Flow to top of the RSF, to septic tank, discharged to Dosing Chamber #2 (DC #2), and Overflow to DC #2) to the Forward Flow (Q).

acquisition was also a problem when dosing chamber hatches were frozen. These occasions are not included in flow rate and concentration averages used in the mass balance calculations (Table 4.29).

Table 4.29 Dates omitted from the flow rate and concentration averages used in the mass balance calculations at Site #2..

Date(s)	Study Period	Cause for Omission
? to 12/30/93	#2	Pump Off an Unknown Number of days
12/30 to 1/4/93	#2	Pump Off 5 days
3/13/93 to 3/23/93	#2	Pump Off 9.5 days
5/10/93 to 5/19/93	#2	Pump Off 9.0 days
7/2/93 to 7/26/93	#3	Pump Off 24 days, Redesign of Timer Control Panel
11/18/93	C	Dosing Chamber #2 Orifice Clogged
11/23/93	C	Dosing Chamber #2 Orifice Clogged
6/15/94 to 6/17/94	E	Pump Failure, Electrical Relay Replaced
11/5/94	K	Unable to Obtain Sample for DC #2, Hatch Frozen

4.23 Mass Balance Calculations

Mass balances were produced for all 22 study periods at Site #2. Unlike Site #1, an additional sample point (Dosing Chamber #2) is available between the pump chamber of the RSF and the mound system. This additional sample point allows the loading to the mound system to be determined with more accuracy than at the previous site.

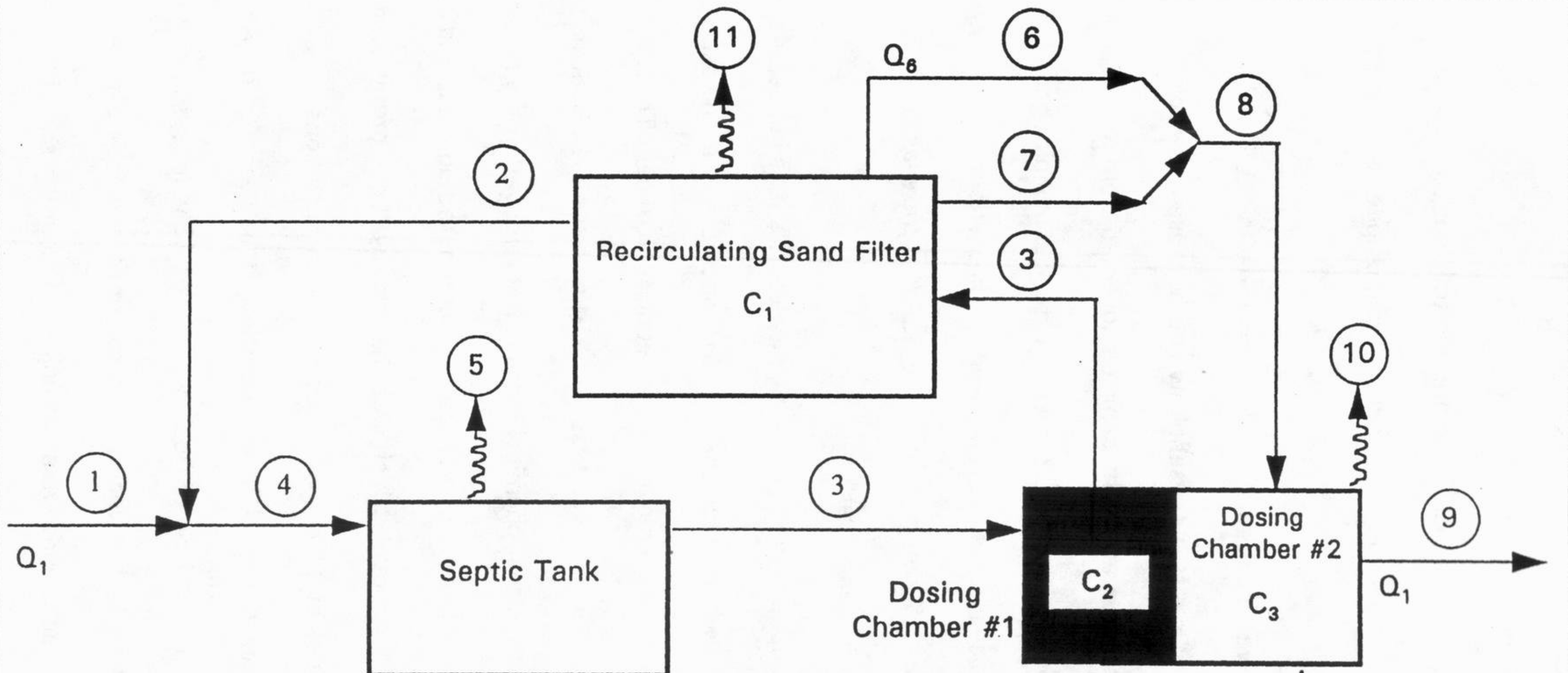
Assumptions

Mass loadings and removals throughout the system were calculated using the chemical data collected [from the pump chamber of the RSF (C_1), Dosing Chamber #1 (C_2), and Dosing Chamber #2 (C_3)] and the flows from the household (Q_1), the flow recycled back to the septic tank (Q_2), and the flow pumped to Dosing

Chamber #2 from the RSF (Q_6). There are four known values obtained from these measurements which become the basis for the mass balance calculations: the mass from the RSF to the septic tank (M_2), the mass from Dosing Chamber #1 to the top of the RSF (M_3), the mass from the RSF to Dosing Chamber #2 (M_8), and the mass applied from Dosing Chamber #2 to the mound system (M_9). A schematic for the mass balance calculations can be found in Figure 4.42. Mass balance calculations for the 22 study periods at Site #2 are included in the appendices.

No samples were available from the point of wastewater entry to the system after the RSF was installed and the pump became operational. Therefore, the actual concentrations of chemical species in the household waste was unknown. The average concentrations in the original dosing chamber obtained before the installation of the system could be used to represent household conditions, but these were variable over time because the water usage at the Site #2 was highly variable over the course of monitoring. The daily water usage at Site #2 between 1991 and 1995 ranged between 596 lpd (157 gpd) and 3,090 lpd (816 gpd). The household concentrations were estimated, due to this variability, as follows.

An assumption was made in order to estimate the concentrations of chemical species in the household waste since there was no sample point between the household and the septic tank effluent flowing into Dosing Chamber #1. The concentrations in the septic tank effluent flowing into Dosing Chamber #1 are a combination of household waste and recirculated flow from the sand filter. The assumption made was that no transformations or losses of chemicals occur in the



		Flow Volume	Mass
1	From House (M_1)	Q_1	$M_3 - M_2$*
2	From RSF to Septic Tank (M_2)	Q_2	$Q_2 \times C_1$
3	Septic Tank Outflow (M_3)	$Q_1 + Q_2$	$(Q_1 + Q_2) \times C_2$
4	Total to Septic Tank (M_4)	$Q_1 + Q_2$	$M_1 + M_2$
5	Removed in Septic Tank (M_5)	0	$M_4 - M_3$
6	From RSF to DC #2 (M_6)	Q_6	$Q_6 \times C_1$
7	Over Flow to DC #2 (M_7)	$Q_1 - Q_6$	$(Q_1 - Q_6) \times C_1$
8	Total to DC #2 (M_8)	Q_1	$M_6 + M_7$
9	DC #2 to Mound(M_9)	Q_1	$Q_1 \times C_3$
10	Removed in DC #2 (M_{10})	0	$M_8 - M_9$
11	Removed in RSF (M_{11})	0	$M_3 - (M_8 + M_2)$

Measured Concentrations	Measured Flows
C_1 Sand Filter	Q_1 Household Water Use
C_2 Septic Tank	Q_2 Recycle to Septic Tank
C_3 Dosing Chamber #2	Q_6 Pumped to DC #2

Figure 4.42 Schematic for the mass balance calculations at Site #2.

household waste as it passes through the septic tank. Therefore, the mass of chemical coming out of the house is equal to the mass in the septic tank effluent applied to the RSF from Dosing Chamber #1 less the mass pumped to the septic tank from the sand filter.

A further assumption was made for TN. Household waste is primarily NH_4^+-N and organic-N; there is no nitrate-N in household waste because anaerobic conditions exist and none was detected in the samples collected from the original dosing chamber before the installation of the system. Total kjeldahl nitrogen (TKN) consists of NH_4^+-N and organic-N; therefore, the TN in the household waste is *the mass of TKN in the septic tank effluent applied to the RSF less the mass of TKN in the septic tank effluent from the sand filter.*

The use of these assumptions in the mass balance calculations indicates that the removal of chemical mass, with the exception of TN, occurs only in the sand filter. Accordingly, no chemicals are removed in the septic tank except TN as nitrate-N through denitrification since it was assumed that no nitrate-N exists in household waste. The implication that chemical mass is not removed in the septic tank may be a limitation in the interpretation of these results based on mass and percent removal but not for TN removal, which was the primary focus of this study.

A sample point did not exist which would allow the concentrations overflowing to Dosing Chamber #2 via the built in overflow to be determined. As a result, the concentrations in the overflow to Dosing Chamber #2 were assumed to be the same as the concentrations observed in the pump chamber. However, as seen at the previous site, concentrations were not uniform within the sand filter and it is unlikely that the concentrations in the over flow to Dosing Chamber #2 were the same as those seen in the pump chamber of the RSF. This causes the mass balances to predict an unrealistic mass increase in Dosing Chamber #2 during some study periods, when the concentrations measured in Dosing Chamber #2 are greater than those in the RSF. However, TN removal in Dosing Chamber #2 by denitrification is likely to be minimal as the effluent in Dosing Chamber #2 is characteristically aerobic and depleted in available carbon.

Despite the apparent discrepancies in Dosing Chamber #2 during some study periods, the mass balances still provide a good estimate of the loading to groundwater. The mass discharged from the system is *a known value based on concentrations measured in Dosing Chamber #2 and the average flow rate applied to the mound system*. The mass removed by the system is *the difference between the estimated mass from the household* (M_1) *and the mass discharged* (M_9). Since the concentrations in Dosing Chamber #2 are the actual concentrations being applied to the mound system, the mass balances at Site #2 provide a better estimate of the mass removed by the system than do the mass balances at Site #1.

4.24 BOD$_5$, COD, and Total Phosphorous

The concentrations of BOD$_5$, COD, and total phosphorous (TP) in Dosing Chamber #1, the pump chamber of the RSF, and Dosing Chamber #2 from 1991 to 1995 at Site #2 are included in Figures 4.43 to 4.45. These data were collected for 20 of the 22 study periods used to evaluate the various levels of treatment achieved at this site. Samples were not available for study periods N and H.

Table 4.30 shows BOD$_5$ removal for the RSF system at Site #2. The estimated mass of BOD$_5$ from the house was calculated using the mass balance assumptions. Using these assumptions, 123 to 485 grams/day (0.272 to 1.069 lbs/day) of BOD$_5$ were generated in the household. BOD$_5$ removal ranged from 79.3% to 98.9% for the various flow conditions used.

The estimated average concentration of BOD$_5$ in the household waste was 198.6 mg/l (range 118 to 276 mg/l). The BOD$_5$ measured in the original dosing chamber before the system was installed averaged 174.7 mg/l (range 68 to 292). The average concentration of BOD$_5$ in Dosing Chamber #1 after system became operational was 83.8 mg/l (range 28 to 245 mg/l). The average concentration of BOD$_5$ in the RSF was 11.6 mg/l (2 to 43 mg/l). The average concentration of BOD$_5$ applied to mound system from Dosing Chamber #2 was 9.0 mg/l (range 2 to 44 mg/l).

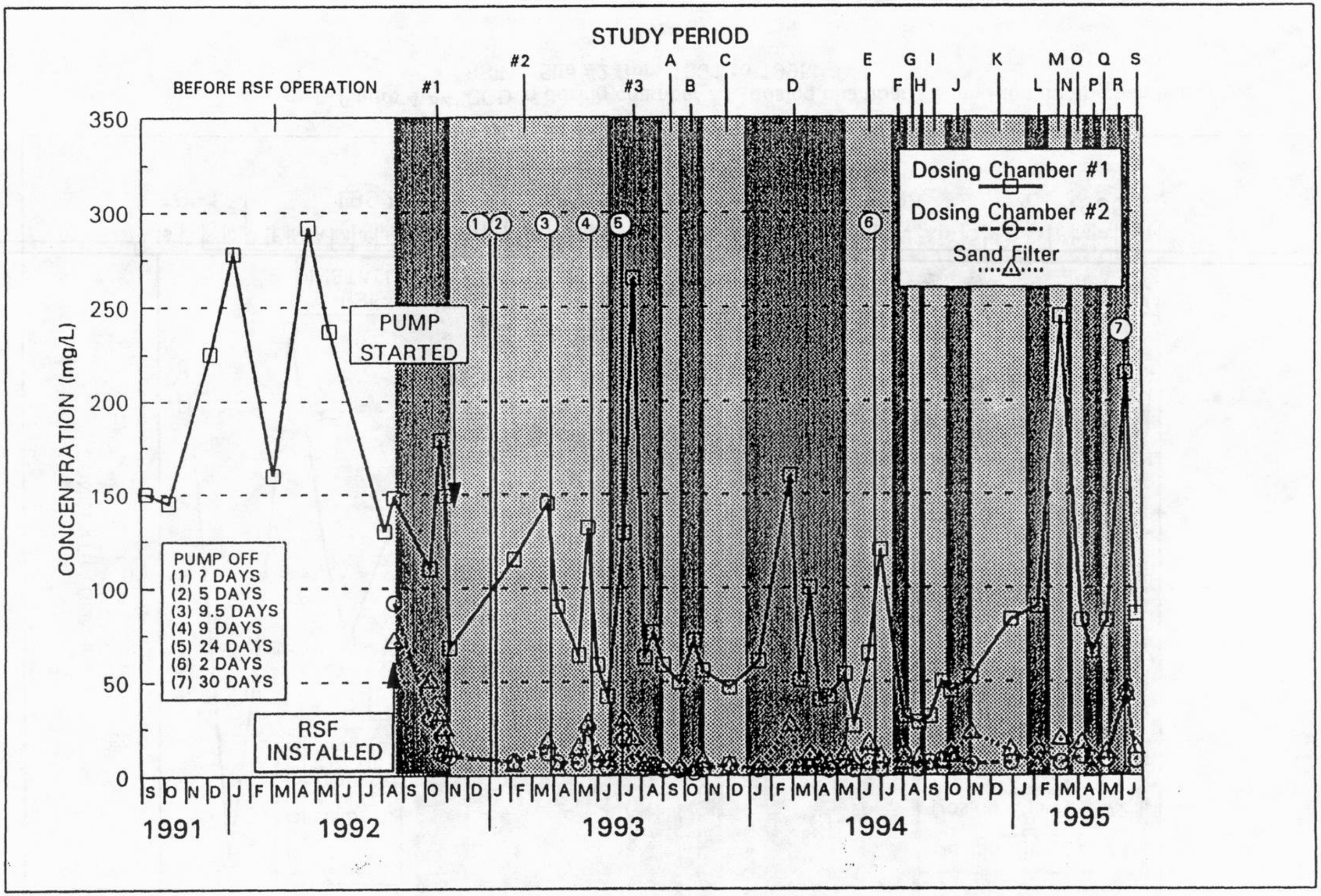


Figure 4.43 BOD in dosing chamber #1, dosing chamber #2, and the pump chamber of the RSF at Site #2 from 1991 to 1995.

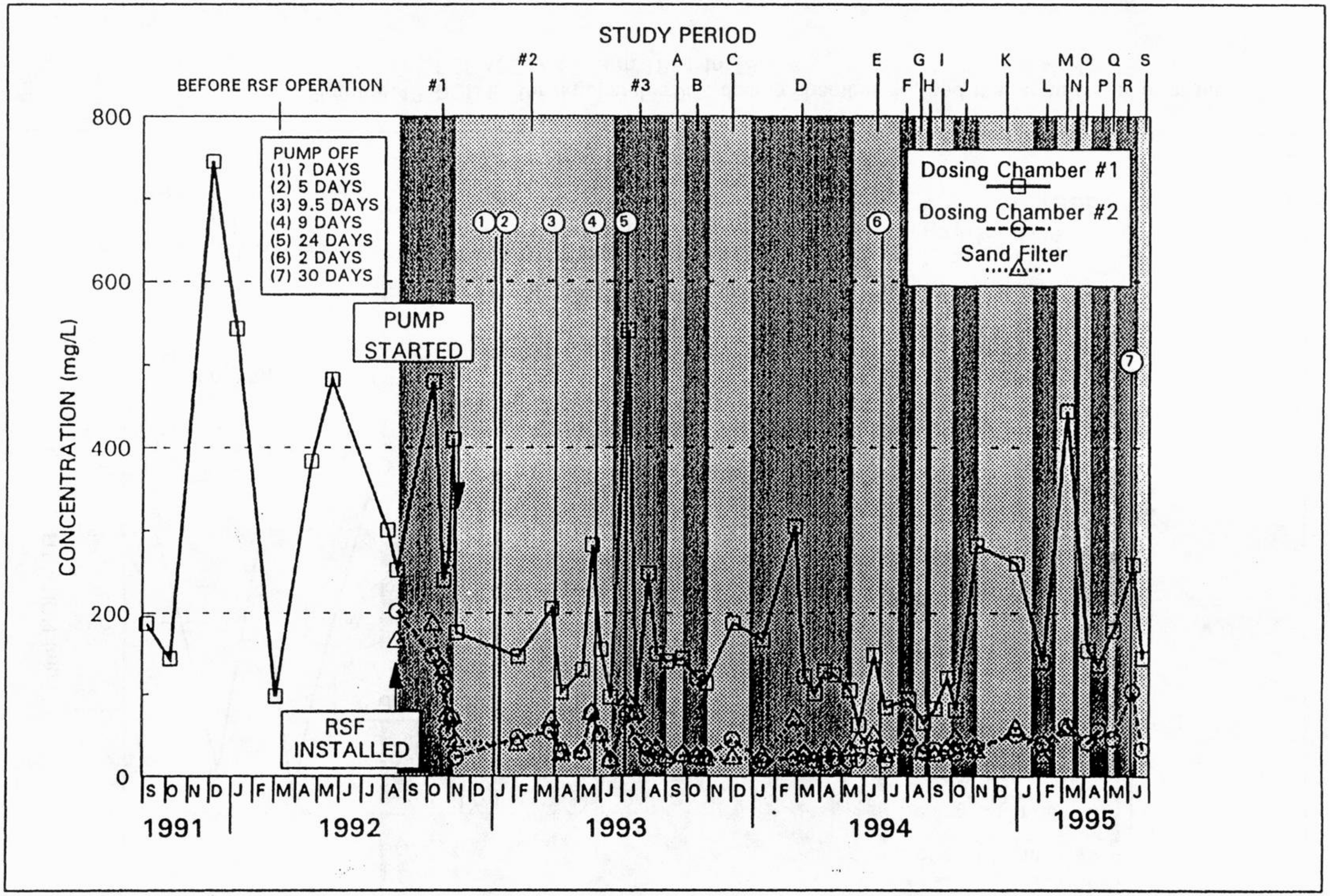


Figure 4.44 COD in dosing chamber #1, dosing chamber #2, and the pump chamber of the RSF at Site #2 from 1991 to 1995.

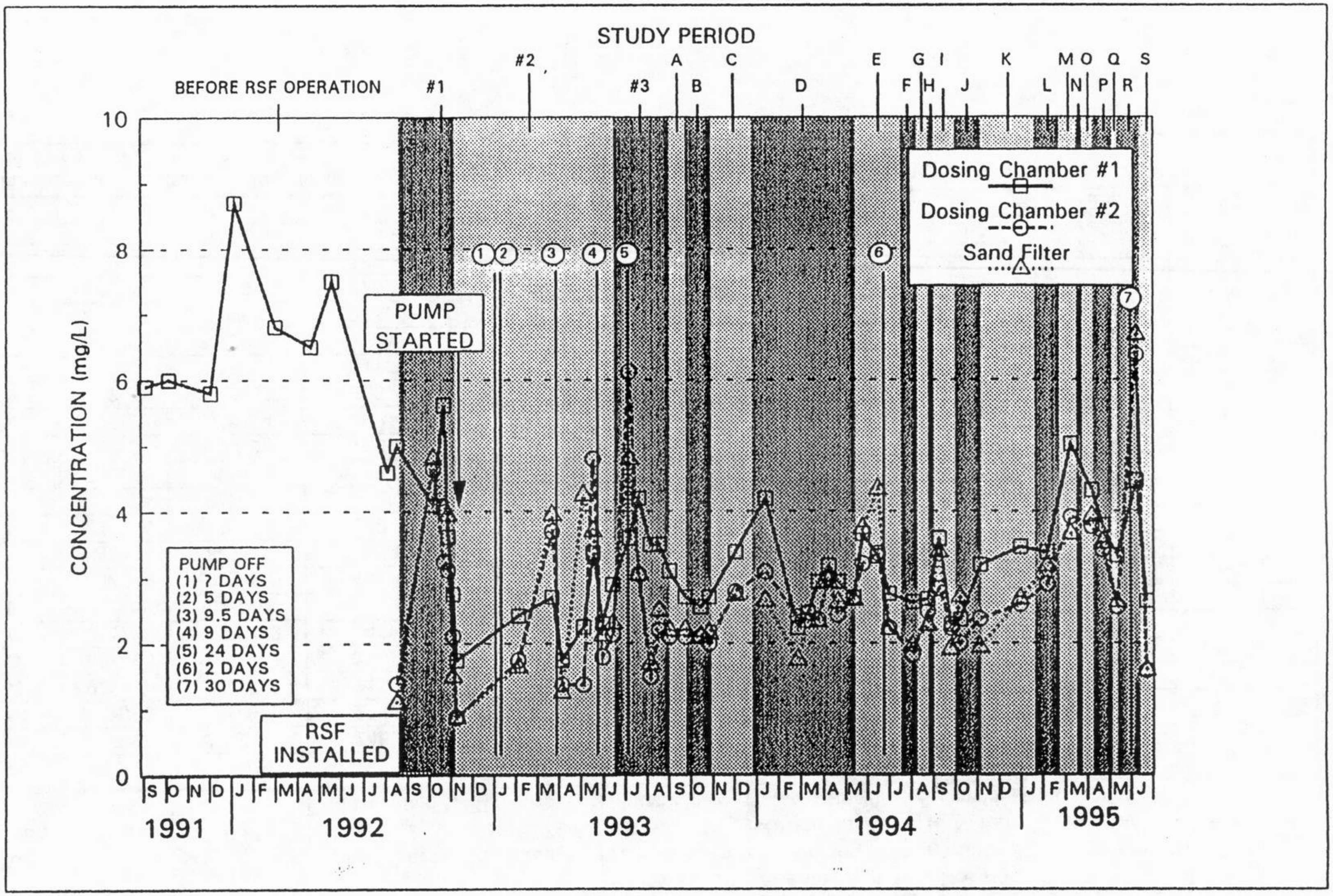


Figure 4.45 Total P in dosing chamber #1, dosing chamber #2, and the pump chamber of the RSF at Site #2 from 1991 to 1995.

Table 4.30 BOD_5 removal by the RSF system for all study periods at Site #2.

Study Period	Start	End	Mass From House (grams/day)	Removal in Septic Tank (grams/day)	Removal in Sand Filter (grams/day)	Removal in Dosing Chamber #2 (grams/day)	Dosing Chamber #2 to Mound (grams/day)	System Removal (grams/day)	Percent Removed
#1	8/13/92	11/5/92	188.6	0.0	144.3	20.9	23.4	165.2	87.6
#2	11/6/92	6/15/93	245.0	0.0	231.3	2.2	11.5	233.5	95.3
#3	6/16/93	8/31/93	274.8	0.0	268.4	0.2	6.3	268.6	97.7
A	9/1/93	9/23/93	213.1	0.0	208.4	0.7	4.0	209.1	98.1
B	9/24/93	10/26/93	279.0	0.0	272.6	3.4	3.0	276.0	98.9
C	10/27/93	12/23/93	241.0	0.0	236.8	-2.5	6.6	234.3	97.3
D	12/24/93	5/13/93	343.7	0.0	329.3	4.1	10.2	333.4	97.0
E	5/14/94	7/15/93	484.9	0.0	469.4	10.0	5.5	479.4	98.9
F	7/16/94	8/9/93	127.9	0.0	121.1	-5.0	11.8	116.1	90.8
G	8/10/94	8/23/93	128.7	0.0	121.2	4.6	2.9	125.8	97.7
H	8/24/94	9/1/94	NS	NS	NS	NS	NS	NS	NS
I	9/2/94	9/29/94	143.7	0.0	140.5	-1.9	5.1	138.6	96.4
J	9/30/94	11/7/94	123.3	0.0	112.4	5.1	5.7	117.5	95.3
K	11/8/94	1/18/94	192.4	0.0	183.3	4.2	4.9	187.5	97.5
L	1/19/95	2/22/95	305.4	0.0	299.1	-10.2	16.5	288.9	94.6
M	2/23/95	3/15/95	313.1	0.0	289.1	15.3	8.7	304.4	97.2
N	3/16/95	3/23/95	NS	NS	NS	NS	NS	NS	NS
O	3/24/95	4/6/95	218.8	0.0	198.1	8.2	12.6	206.2	94.3
P	4/7/95	5/5/95	225.8	0.0	222.8	-8.4	11.5	214.3	94.9
Q	5/6/95	5/11/95	262.9	0.0	247.8	4.1	11.0	251.9	95.8
R	5/12/95	6/13/95	293.3	0.0	235.3	-2.7	60.7	232.6	79.3
S	6/14/95	6/28/95	201.8	0.0	191.3	4.5	6.0	195.8	97.0

(-) Indicates an increase in dosing chamber #2 relative to the sand filter due to higher concentrations being detected in dosing chamber #2
NS No Sample Available

Table 4.31 shows COD removal for the RSF system at Site #2. The estimated mass of COD from the house was calculated from the mass balance calculations. Approximately 253 to 931 grams/day (0.558 to 2.053 lbs/day) of COD were generated in the household. COD removals ranged from 59.7% to 95.7% for the various flow conditions used.

The estimated average concentration of COD in the household waste was 413.5 mg/l (range 194 to 830 mg/l). The average COD concentration measured in the original dosing chamber before the system was installed was 348.6 mg/l (range 98 to 745 mg/l). The average concentration measured in Dosing Chamber #1 after the system became operational was 174.7 mg/l (range 66 to 444 mg/l). The average concentration in the RSF was 44.4 mg/l (range 22 to 116 mg/l). The average concentration of COD applied to the mound system from Dosing Chamber #2 was 43.4 mg/l (range 20 to 104 mg/l).

Table 4.32 shows TP removal for the RSF system at Site #2. The estimated mass of TP was calculated from the mass balance calculations. Approximately 2.90 to 9.00 grams/day (0.006 to 0.020 lbs/day) of TP were generated in the household. TP removal ranged from 17.5 to 70.1% for the various flow conditions used. It should be noted that the results for study period R are misleading because the TP concentrations in Dosing Chamber #1 (4.47 mg/l) were less than those seen in the sand filter (6.68 mg/l) or Dosing Chamber #2 (6.37 mg/l). Thus, the concentrations seen during study period R are not used in determining the concentration averages and ranges.

Table 4.31 COD removal by the RSF system for all study periods at Site #2.

Study Period	Start	End	Mass From House (grams/day)	Removal in Septic Tank (grams/day)	Removal in Sand Filter (grams/day)	Removal in Dosing Chamber #2 (grams/day)	Dosing Chamber #2 to Mound (grams/day)	System Removal (grams/day)	Percent Removed
#1	8/13/92	11/5/92	520.4	0.0	348.4	29.5	142.4	378.0	72.6
#2	11/6/92	6/15/93	409.6	0.0	351.0	0.5	58.1	351.5	85.8
#3	6/16/93	8/31/93	694.5	0.0	645.9	9.0	39.6	654.9	94.3
A	9/1/93	9/23/93	515.4	0.0	481.6	1.5	32.3	483.1	93.7
B	9/24/93	10/26/93	468.0	0.0	439.3	-4.0	32.6	435.3	93.0
C	10/27/93	12/23/93	930.8	0.0	905.7	-26.1	51.3	879.5	94.5
D	12/24/93	5/13/93	663.0	0.0	614.6	17.7	30.7	632.3	95.4
E	5/14/94	7/15/93	355.0	0.0	294.0	23.9	37.1	317.9	89.6
F	7/16/94	8/9/93	279.8	0.0	228.4	5.5	45.9	234.0	83.6
G	8/10/94	8/23/93	252.7	0.0	227.2	0.0	25.5	227.2	89.9
H	8/24/94	9/1/94	NS	NS	NS	NS	NS	NS	NS
I	9/2/94	9/29/94	302.1	0.0	282.2	-1.9	21.8	280.3	92.8
J	9/30/94	11/7/94	515.9	0.0	489.7	3.9	22.3	493.6	95.7
K	11/8/94	1/18/94	568.8	0.0	528.0	4.7	36.1	532.7	93.7
L	1/19/95	2/22/95	439.3	0.0	407.9	-22.6	54.0	385.3	87.7
M	2/23/95	3/15/95	567.4	0.0	493.7	-6.5	80.3	487.2	85.9
N	3/16/95	3/23/95	NS	NS	NS	NS	NS	NS	NS
O	3/24/95	4/6/95	367.6	0.0	299.5	15.2	52.9	314.7	85.6
P	4/7/95	5/5/95	389.5	0.0	337.5	-23.3	75.3	314.1	80.7
Q	5/6/95	5/11/95	530.3	0.0	473.6	-5.0	61.7	468.6	88.4
R	5/12/95	6/13/95	353.3	0.0	194.8	16.2	142.3	211.0	59.7
S	6/14/95	6/28/95	319.4	0.0	293.9	0.0	25.5	293.9	92.0

(-) Indicates an increase in dosing chamber #2 relative to the sand filter due to higher concentrations being detected in dosing chamber #2
NS No Sample Available

Table 4.32 Total Phosphorous removal by the RSF system for all study periods at Site #2.

Study Period	Start	End	Mass From House (grams/day)	Removal in Septic Tank (grams/day)	Removal in Sand Filter (grams/day)	Removal in Dosing Chamber #2 (grams/day)	Dosing Chamber #2 to Mound (grams/day)	System Removal (grams/day)	Percent Removed
#1	8/13/92	11/5/92	5.96	0.0	0.60	0.45	4.92	1.04	17.5
#2	11/6/92	6/15/93	3.91	0.0	0.35	1.02	2.54	1.37	35.0
#3	6/16/93	8/31/93	9.00	0.0	5.63	0.32	3.05	5.95	66.1
A	9/1/93	9/23/93	5.88	0.0	2.87	0.14	2.88	3.00	51.1
B	9/24/93	10/26/93	5.11	0.0	2.31	0.11	2.70	2.41	47.2
C	10/27/93	12/23/93	6.67	0.0	3.61	-0.07	3.13	3.54	53.1
D	12/24/93	5/13/93	6.12	0.0	2.52	-0.25	3.85	2.27	37.1
E	5/14/94	7/15/93	7.12	0.0	1.67	0.46	4.98	2.13	29.9
F	7/16/94	8/9/93	5.36	0.0	3.24	0.14	1.98	3.38	63.0
G	8/10/94	8/23/93	4.47	0.0	2.52	-0.18	2.13	2.34	52.4
H	8/24/94	9/1/94	NS	NS	NS	NS	NS	NS	NS
I	9/2/94	9/29/94	2.90	0.0	1.14	0.08	1.68	1.22	42.1
J	9/30/94	11/7/94	3.15	0.0	1.58	0.09	1.48	1.66	52.9
K	11/8/94	1/18/94	3.93	0.0	2.07	0.06	1.81	2.12	54.0
L	1/19/95	2/22/95	4.94	0.0	0.74	0.36	3.85	1.09	22.1
M	2/23/95	3/15/95	6.42	0.0	1.75	-0.35	5.01	1.41	21.9
N	3/16/95	3/23/95	NS	NS	NS	NS	NS	NS	NS
O	3/24/95	4/6/95	6.13	0.0	1.20	0.19	4.74	1.39	22.7
P	4/7/95	5/5/95	5.58	0.0	0.94	0.13	4.50	1.08	19.3
Q	5/6/95	5/11/95	5.91	0.0	2.48	0.09	3.34	2.57	43.5
R	5/12/95	6/13/95	6.10	0.0	-3.01	0.42	8.69	-2.59	-42.5
S	6/14/95	6/28/95	3.98	0.0	2.79	0.00	1.19	2.79	70.1

(-) Indicates an increase in dosing chamber #2 relative to the sand filter due to higher concentrations being detected in dosing chamber #2
Also indicates an increase in the sand filter relative to dosing chamber #1 due to higher concentrations being detected in the sand filter.
during Study Period R.

NS No Sample Available

The average TP concentration estimated in the household waste was 4.6 mg/l (range 2.3 to 5.9 mg/l). The average TP concentration in the original dosing chamber before the system was installed was 5.6 mg/l (range 2.8 to 8.7 mg/l). The average TP concentration in Dosing Chamber #1 after the system became operational was 3.3 mg/l (range 2.2 to 5.0 mg/l). The average TP concentration in the RSF was 2.7 mg/l (range 1.6 to 3.9 mg/l). The average concentration of TP applied to the mound system from Dosing Chamber #2 was 2.6 mg/l (range 1.5 to 3.9 mg/l).

As noted previously in the discussion for Site #1, the variability of TP loading to the system from the household, the different hydraulic loading rates used, and the different pHs observed make it impossible to determine whether adsorption or precipitation of phosphate control TP removal within the sand filter at Site #2.

4.25 Nitrogen

4.251 Nitrification

Chemical results from sample data collected on July 4, 1994 from: the lysimeter in the unsaturated sand layer of the RSF, the monitoring well screening the saturated rock storage unit of the RSF, the pump chamber of the RSF, Dosing Chamber #1, and Dosing Chamber #2 are shown in Table 4.33. Results for this sample date indicate that the concentrations of chemical species are not uniform within the sand filter. The lysimeter showed 6.8 mg/l of nitrate-N produced in the top sand layer of the RSF, while the monitoring well and pump chambers showed concentrations of 3.8 mg/l and 3.6 mg/l, respectively. The decrease in concentrations between the unsaturated and saturated bottom portions of the RSF imply denitrification in the saturated rock storage layer. Therefore, it is not possible to estimate the nitrate-N produced in the RSF using the concentrations measured in the pump chamber of the RSF.

Table 4.33 Concentrations of water quality characteristics in the sand filter lysimeter, monitoring well, pump chamber, in Dosing Chamber #1, and Dosing Chamber #2 at Site #2 as measured on July 1, 1994.

Location	Nitrate-N mg/l	NH_4^+-N mg/l	TKN mg/l	BOD_5 mg/l	pH Std Units
RSF Lysimeter	6.9	<0.01	0.8	-	-
RSF Well	3.8	4.2	4.5	35.8	7.36
RSF Pump Chamber	3.6	1.87	3.7	9.0	6.98
Dosing Chamber #1	0.4	5.10	9.3	120	6.85
Dosing Chamber #2	3.0	1.95	3.3	3.0	7.25

Table 4.34 shows the theoretical conversion of NH_4^+-N to nitrate-N in the RSF. The conversion of NH_4^+-N is estimated from the influent household concentrations and the NH_4^+-N concentrations measured in the pump chamber of the RSF. Approximately 8.8 to 61.9 mg/l of nitrate-N were produced in the sand filter, a theoretical conversion of 42 to 98%, with HLRs ranging from 21 to 196 cm/day. NH_4^+-N removal by volatilization is not likely to be significant at the near neutral pH values seen in the sand filter (6.33 to 7.89 Std. Units) at Site #2, despite the high degree of air to water contact provided by pumping effluent to the top of the sand filter. Therefore, the reduction in NH_4^+-N is due predominantly to nitrification in the RSF. Concentrations in the pump chamber are lower than the

Table 4.34 Theoretical changes in NH_4^+-N and Nitrate-N concentrations in the RSF for the study periods used in the determination of effluent treatment by the RSF system at Site #2.

Study Period	HLR (cm/day)	Sand Filter Ratio (SFR)	Estimated in Household Waste		Average Measured in RSF			Theoretical		
			TKN (mg/l)	NH_4^+-N (mg/l)	TKN (mg/l)	NH_4^+-N (mg/l)	Nitrate-N (mg/l)	Percent Conversion NH_4^+-N * (%)	Nitrate-N Produced (mg/l)	Percent Denitrification Nitrate-N (%)
#1	23	1.0	28.7	22.1	18.9	12.9	0.5	9.2	42	95
#2	95	3.7	27.4	18.3	6.3	4.8	4.7	13.5	74	65
#3	136	5.5	45.4	26.6	4.9	3.9	5.7	22.7	85	75
A	132	6.3	42.3	28.9	2.5	2.0	6.9	26.9	93	74
B	154	7.7	44.9	28.7	1.9	1.1	8.8	27.6	96	68
C	152	8.8	40.2	9.9	1.7	1.1	4.0	8.8	89	54
D	139	6.2	33.5	18.3	2.7	1.5	4.8	16.9	92	72
E	177	6.3	18.1	13.7	4.2	3.3	2.9	10.4	76	72
F	143	8.6	38.3	28.9	6.1	4.4	3.3	24.4	85	87
G	194	14.7	29.1	19.3	1.9	0.9	1.4	18.4	95	93
H	196	15.9	32.8	30.8	1.6	0.5	1.5	30.3	98	95
I	144	14.1	44.1	41.8	2.7	1.0	3.4	40.8	98	92
J	129	12.5	64.7	34.7	6.9	3.8	3.1	30.9	89	90
K	123	11.5	92.1	68.9	7.7	7.0	5.9	61.9	90	90
L	148	7.3	43.4	37.4	7.2	3.8	5.9	33.6	90	83
M	114	5.8	40.4	33.1	7.9	7.0	8.5	26.1	79	67
N	97	4.6	36.4	26.0	12.2	8.1	3.0	17.9	69	83
O	121	6.3	32.8	27.0	7.3	5.0	5.0	22.0	82	77
P	129	6.4	38.8	26.1	4.5	3.0	5.2	23.1	88	77
Q	109	5.4	40.6	26.9	6.2	4.3	5.6	22.6	84	75
R	21	1.0	32.1	26.7	15.5	12.5	0.6	14.3	53	96
S	82	7.2	54.2	43.4	4.9	2.3	8.0	41.2	95	80

* Theoretical conversion of NH_4^+-N to Nitrate-N. Does not account for NH_4^+-N volitilization in the RSF.

concentrations produced, implying 54 to 96% denitrification in the saturated rock storage unit of the RSF.

Figures 4.46 and 4.47 show the relationship of the hydraulic loading rate (HLR) and sand filter ratio (SFR) to the theoretical conversion of NH_4^+-N in the RSF. In general, a greater percentage is converted at higher hydraulic loading rates and sand filter ratios. Variations in temperature and/or alkalinity may account for the variations in NH_4^+-N conversion at similar recirculation rates.

The reported minimum temperature for nitrification to occur is 5°C (Shammas, 1986). Temperature and nitrate-N concentrations measured in the pump chamber of the RSF are shown in Figure 4.48. Nitrate-N concentrations measured in the pump chamber of the RSF were below 1.0 mg/l when the pump was not operating. This is due to denitrification of the nitrate-N produced when pumping effluent to the top of the sand filter from Dosing Chamber #1.

The lowest temperature observed in the pump chamber was 9.5°C (January 5, 1995), which is significantly higher than the minimum temperature required. On this date 8.1 mg/l of nitrate-N were measured in the sand filter. It is suspected that temperature did not have a significant effect on nitrification at Site #2 due to temperatures above the minimum required.

The minimum alkalinity level required for nitrification is 40 mg/l as $CaCO_3$ (Sandy, 1987). Figures 4.49 and 4.50 show the alkalinity and total hardness concentrations in Dosing Chamber #1, the pump chamber of the RSF, and Dosing Chamber #2 from 1992 to 1995. Alkalinity in the pump chamber of the RSF has remained above 80 mg/l ($CaCO_3$) since the system began operating. The limestone within the rock storage unit of the RSF continues to provide an adequate

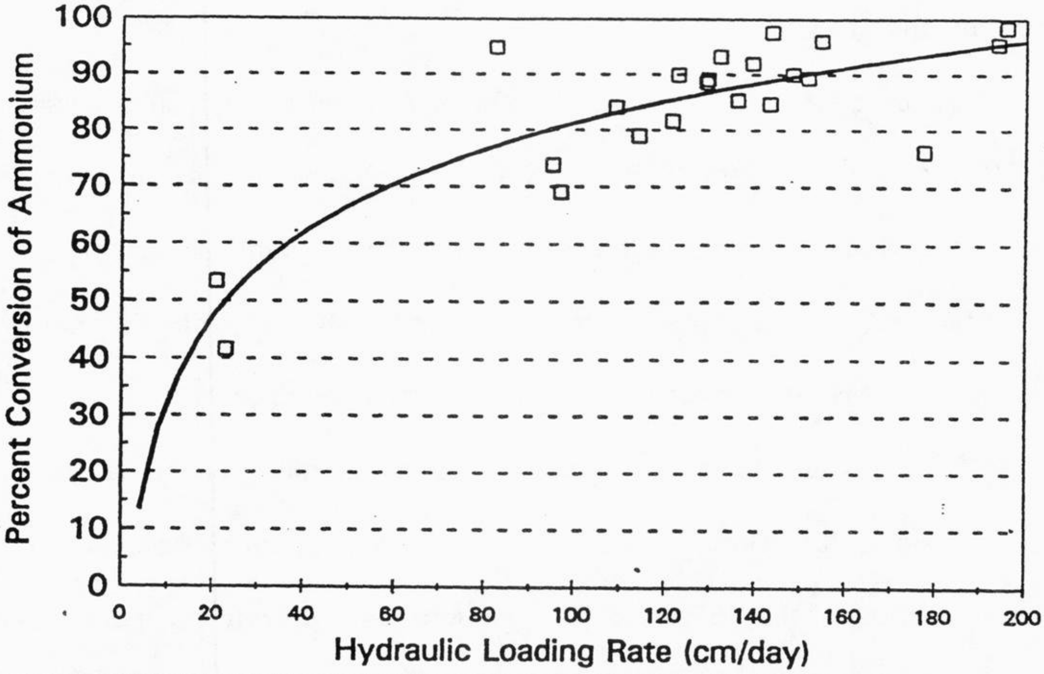


Figure 4.46 Percent conversion of ammonium to nitrate-N in the sand filter vs hydraulic loading rate at Site #2.

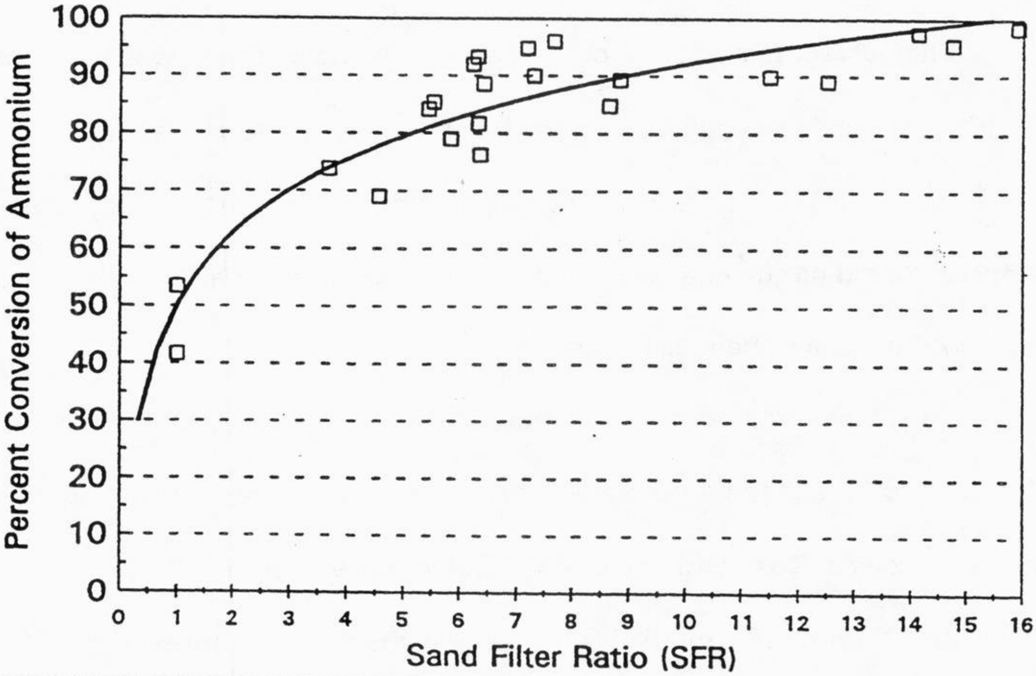


Figure 4.47 Percent conversion of ammonium to nitrate-N in the sand filter vs sand filter ratio (SFR) at Site #2.

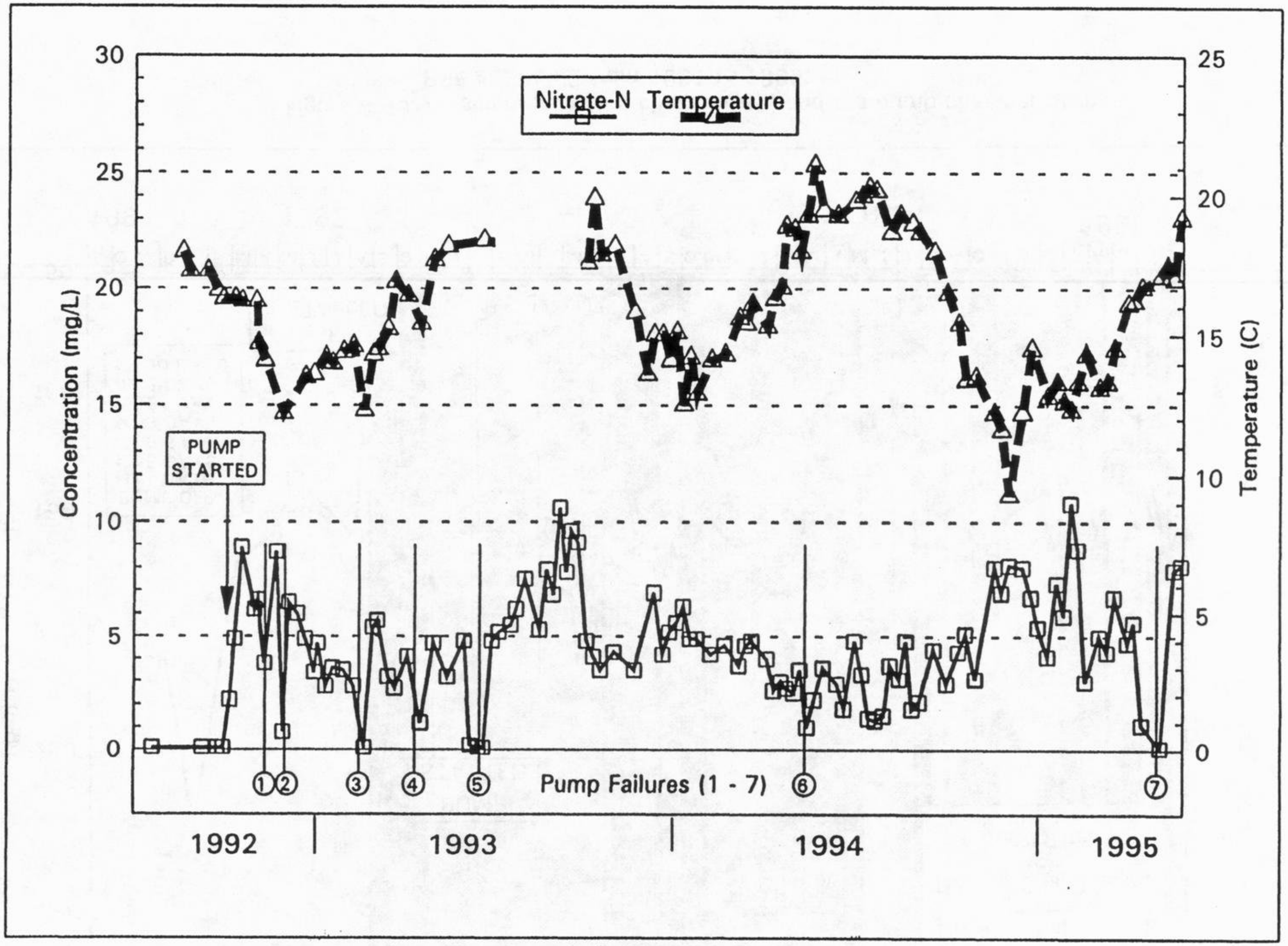


Figure 4.48 Nitrate-N concentrations and Temperature in the pump chamber of the RSF at Site #2.

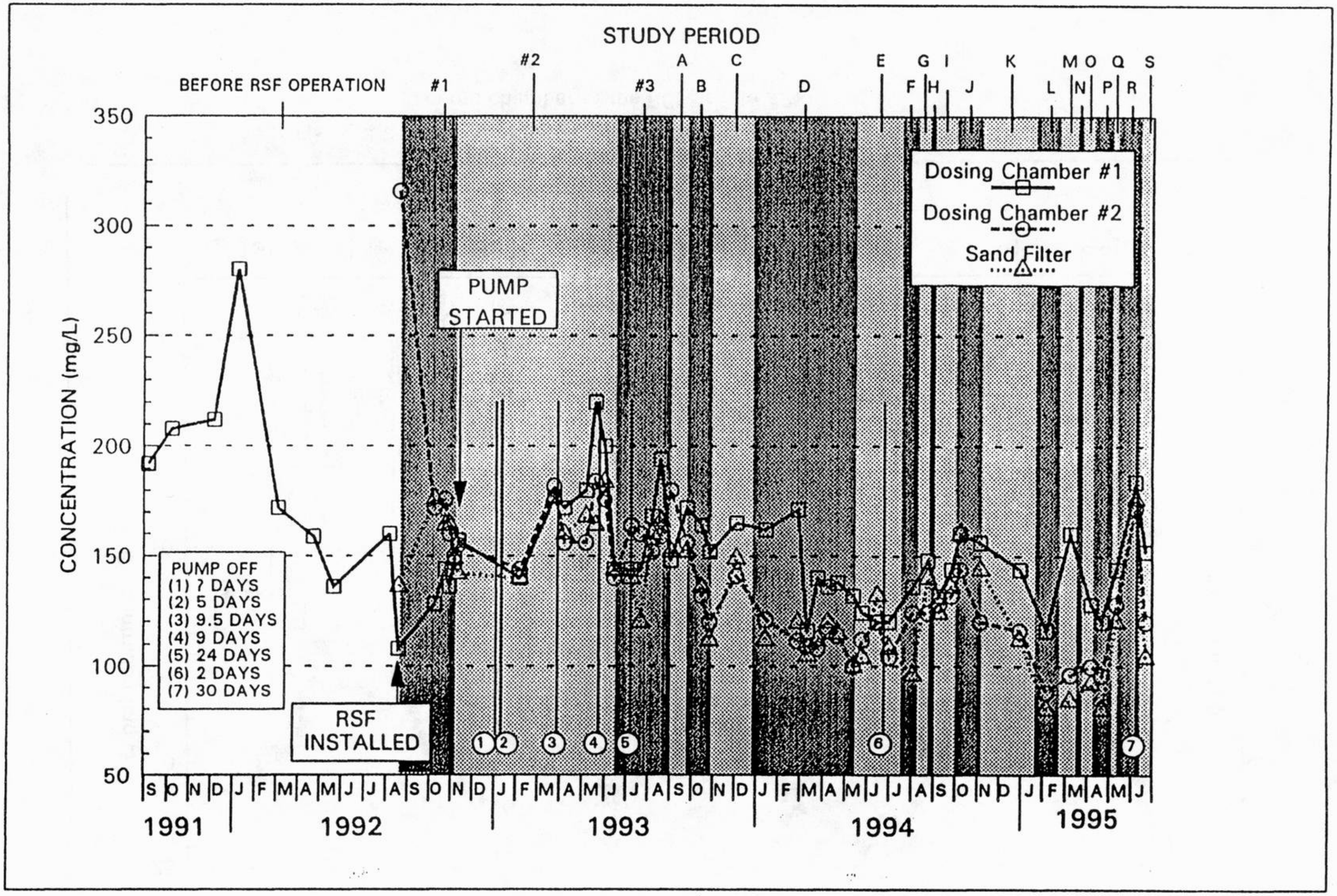


Figure 4.49 Alkalinity in dosing chamber #1, and the pump chamber of the RSF at Site #2 from 1991 to 1995.

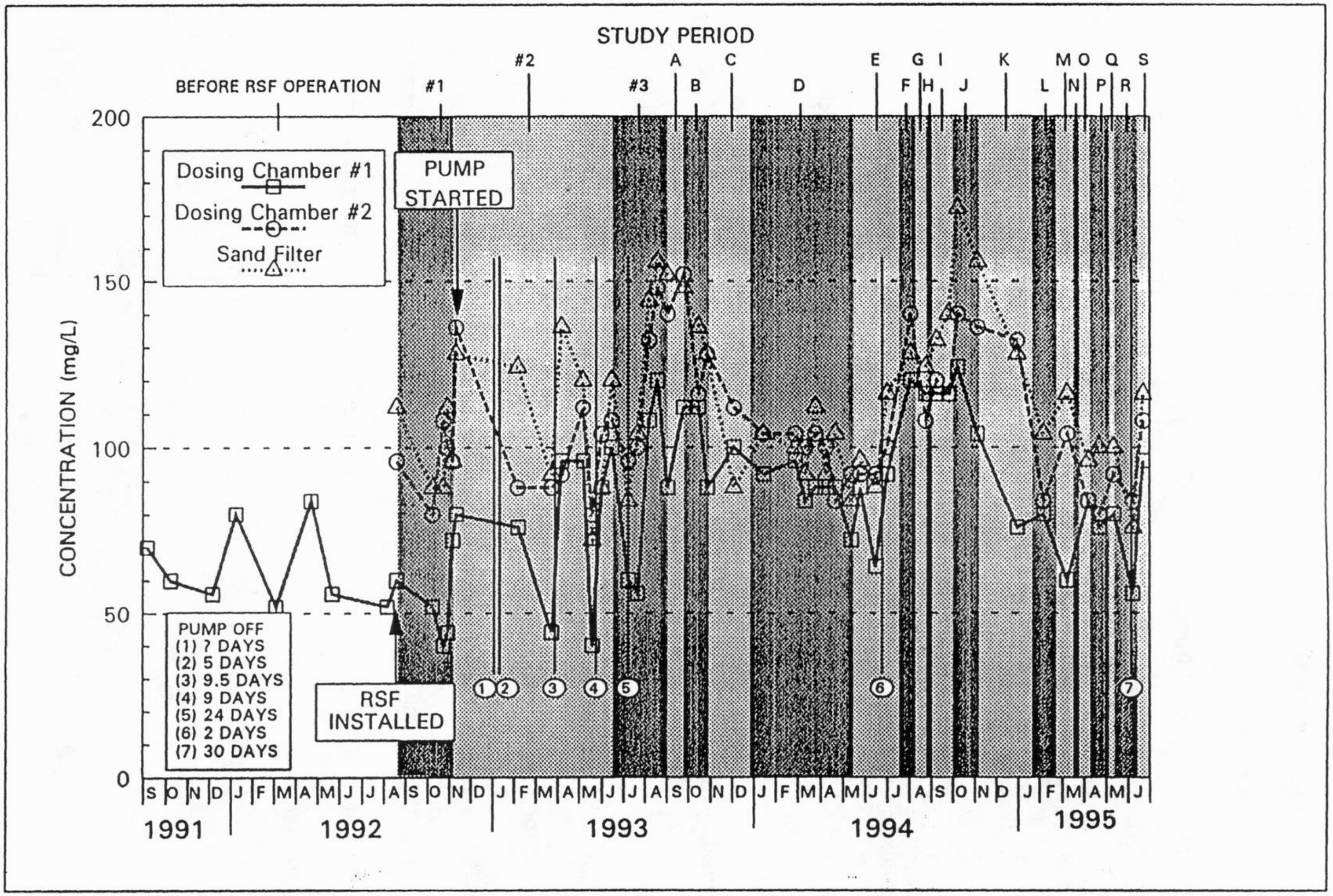


Figure 4.50 Total Hardness in dosing chamber #1, and the pump chamber of the RSF at Site #2 from 1991 to 1995.

supply of alkalinity for nitrification in the RSF. The average alkalinity and total hardness concentrations in the original dosing chamber before the system was installed were 167.8 mg/l (range 108 to 280 mg/l) and 59.8 mg/l (range 40 to 84 mg/l), respectively. It should be noted that alkalinity concentrations are greater than total hardness concentrations because organic solids in the effluent samples buffer the alkalinity endpoint.

It is impossible to determine the amount of alkalinity destroyed by nitrification in the RSF because: 1) total hardness and alkalinity concentrations in the septic tank and sand filter are a mixture of treated effluent and household loading, there is no way to differentiate between them, 2) alkalinity increases due to denitrification in the septic tank and/or sand filter cannot be accounted for, 3) organic solids in the effluent samples buffer the alkalinity endpoint, and 4) alkalinity consumption by sulfur reactions cannot be accounted for (No analyses were performed for sulfate or H_2S).

The variations in NH_4^+-N conversion are due predominately to changes which were made in the recirculation rate to the top of the sand filter. No variation can be attributed to temperature or alkalinity since they were within the range of the microorganisms.

4.252 Denitrification

Table 4.35 shows the removal of nitrate-N in the septic tank by the denitrification process for all study periods at Site #2. It is assumed that denitrification of the nitrate-N produced in the sand filter occurs prior to pumping to the septic tank; thus, the concentrations pumped to the septic tank are the concentrations measured in the pump chamber of the RSF. Average nitrate-N concentrations between 0.3 and 1.0 mg/l imply incomplete denitirification in the septic tank during some study periods. Denitrification is assumed to be 100% complete when nitrate-N concentrations are below the limit of quantitation (<0.2 mg/l). Also, denitrification in the septic tank did not occur during study periods #1, M, N, and R, when no flow was directed back to the septic tank (STR = 0.0Q, HRTs 2.7 to 3.0 days). In theory, if the STR is too high, denitirification in the septic tank will be inhibited by the development of aerobic conditions. Unfortunately, DO measurements are not available.

Denitrification of the nitrate-N pumped from the sand filter to the septic tank ranged from essentially complete down to only 66.7% (study period E) with STRs between 1.4 and 1.9Q and HRTs of 1.1 to 0.5 days. The highest average nitrate-N concentration within the septic tank was 1.0 mg/l (range <0.2 to 3.4 mg/l) during study period E. This high nitrate-N level occurred with an STR of 3.0Q, which was not the highest ratio used during this study. STRs between 1.6Q and 6.3Q also had detectable nitrate-N concentrations ranging between 0.3 and 0.7 mg/l. However, denitirification in the septic tank is also dependent on temperature, pH, and carbon availability.

Table 4.35 Average removal of nitrate-N in the septic tank by the denitirification process at Site #2.

Study Period	Septic Tank Ratio (STR)	HRT (Days)	$NO_3^- + NO_2^-$-N Sand Filter (mg/l)	$NO_3^- + NO_2^-$-N Septic Tank (mg/l)	Percent Denitrficication (%)
#1	0.0	2.5	0.5	<0.2	0
#2	1.1	1.1	4.7	<0.2	100
#3	1.7	0.9	5.7	<0.2	100
A	2.0	0.9	6.9	<0.2	100
B	2.5	0.8	8.8	0.3	96.1
C	3.9	0.7	4.0	<0.2	97.0
D	2.6	0.7	4.8	0.7	86.2
E	3.0	0.5	2.9	1.0	66.7
F	3.5	0.8	3.3	<0.2	100
G	6.3	0.6	1.4	0.3	81.5
H	7.9	0.5	1.5	<0.2	100
I	4.9	1.0	3.4	0.2	95.5
J	4.1	1.1	3.1	0.2	93.6
K	2.8	1.4	5.9	<0.2	100
L	1.6	1.1	5.9	0.5	91.9
M	0.0	3.0	8.5	<0.2	0
N	0.0	2.7	3.0	<0.2	0
O	1.4	1.3	5.0	<0.2	100
P	1.7	1.1	5.2	<0.2	100
Q	1.7	1.1	5.6	<0.2	100
R	0.0	2.8	0.6	<0.2	0
S	2.5	1.4	8.0	<0.2	100

NOTE Denitrification is assumed to be 100% complete for Nitrate-N <0.2 mg/l
NS Indicates no sample available for study periods

The reported minimum temperature for denitrification to occur is 2 to 5°C (Crites et al., 1981). Temperature and nitrate-N concentrations in the septic tank effluent measured in Dosing Chamber #1 are shown in Figure 4.51. The lowest temperature recorded in Dosing Chamber #1 was 10.7°C (February 13, 1995) and is significantly above the minimum required temperature of 5°C. On this date 0.2

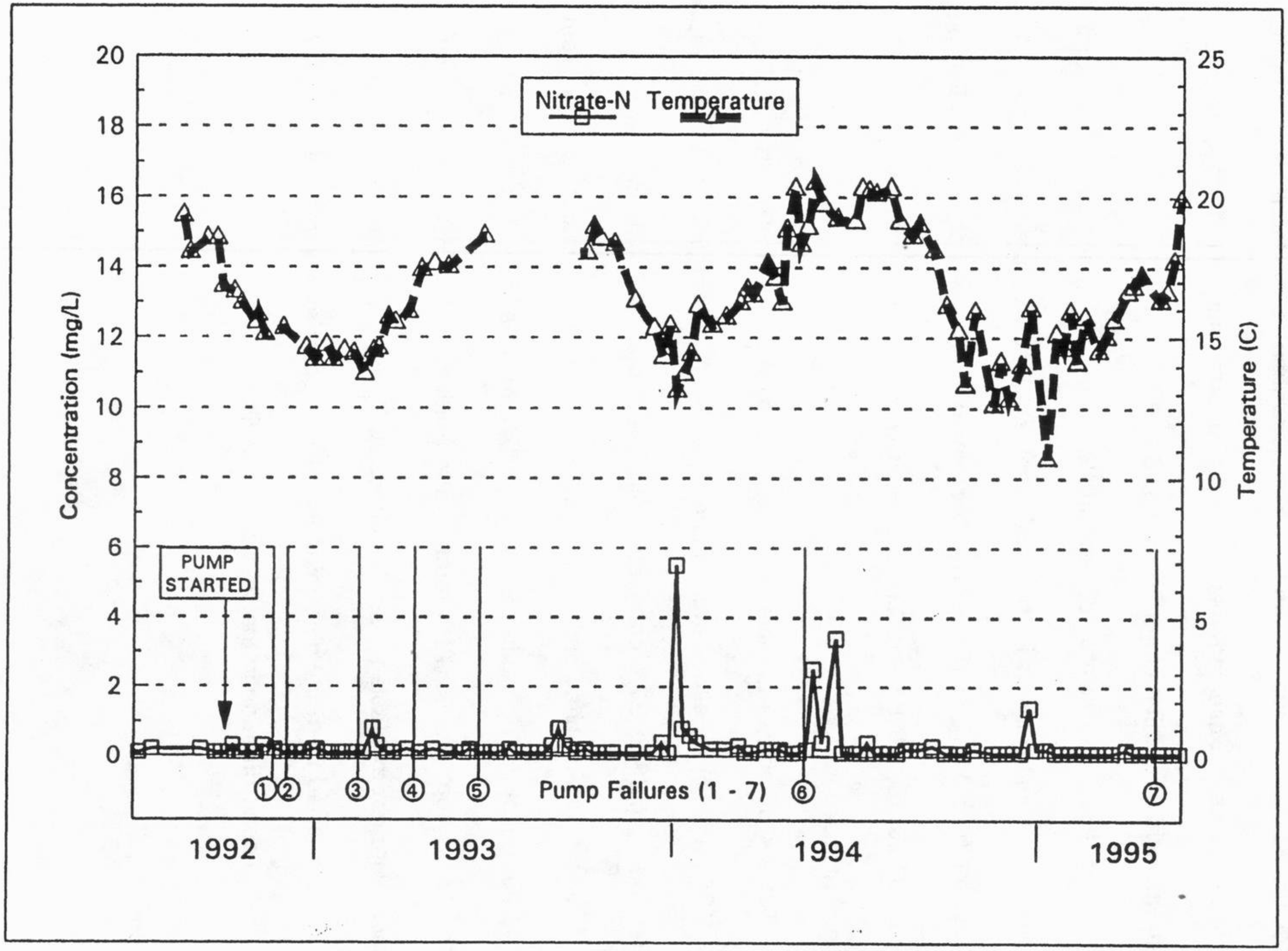


Figure 4.51 Nitrate-N concentrations and Temperature in dosing chamber #1 Site #2.

mg/l of nitrate-N were measured in Dosing Chamber #1. Since the measured nitrate-N concentration was at the limit of quantitation for nitrate-N (<0.2 mg/l) and the temperature observed was above the reported minimum it appears that temperature did not inhibit denitrification at Site #2.

Denitrification rates are depressed below a pH of 6.0 and above a pH of 8.0. Figure 4.52 shows pH levels in the septic tank effluent measured in Dosing Chamber #1 from 1992 to 1995. The fluctuation in pH levels observed in Dosing Chamber #1 (6.50 to 7.61 Std. Units) are within the range required by the microorganisms.

Table 4.36 shows the average percent denitrification, temperature, STR, BOD_5, COD, and pH for all the study periods at Site #2. Denitrification in the septic tank ranged from 66 to 97% complete, values less than 100% indicate that denitrification was inhibited during these study periods. Denitrification in the septic tank did not occur during study periods #1, M, N, or R, when effluent was not directed to the septic tank. Also temperature readings and/or BOD_5, COD, and pH analyses were not available for study periods A, B, and H. Thus, these study periods are omitted from the determination of the factor(s) which have the most influence on denitrification in the septic tank at Site #2.

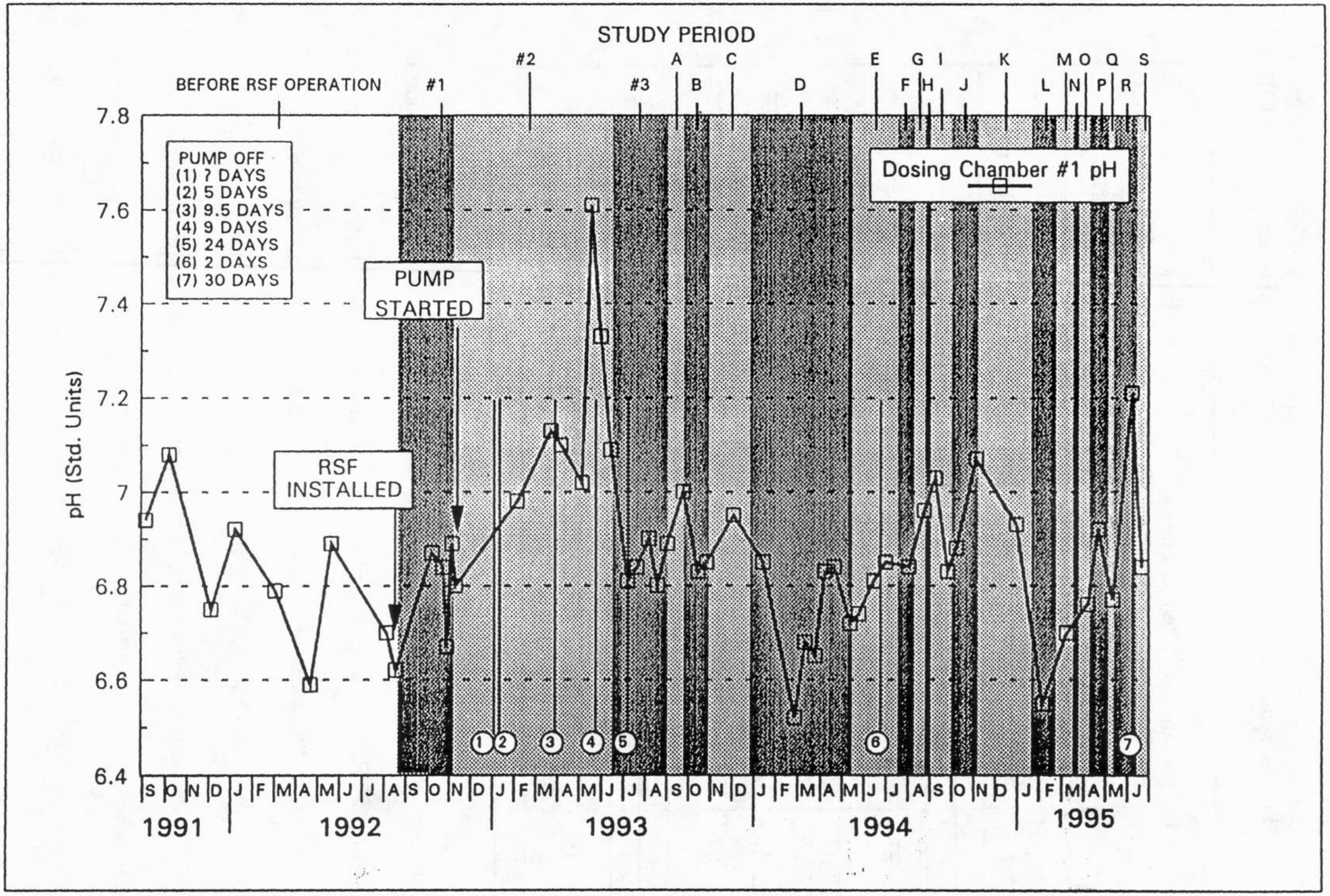


Figure 4.52 pH in dosing chamber #1 at Site #2 from 1991 to 1992.

Table 4.36 Average percent denitrification, temperature, septic tank ratio (STR), BOD_5, COD, and pH in the septic tank at Site #2 for all study periods between 1992 and 1995.

Study Period	Percent Denitrification (%)	Temperature (°C)	STR	BOD_5 (mg/l)	COD (mg/l)	pH Std. Units
#1	0	17.9	0.0	126	349	6.82
#2	100	15.4	1.1	74	135	7.05
#3	100	18.6	1.7	66	179	6.86
A	100	NS	2.0	54	142	6.95
B	96.1	NS	2.5	64	117	6.84
C	97.0	18.0	3.9	47	188	6.95
D	86.2	15.5	2.6	72	150	6.73
E	66.7	19.0	3.0	73	74	6.80
F	100	19.7	3.5	31	94	6.84
G	81.5	20.2	6.3	28	65	6.96
H	100	NS	7.9	NS	NS	NS
I	95.5	19.3	4.9	40	101	6.93
J	93.6	17.7	4.1	48	181	6.98
K	100	14.0	2.8	83	260	6.93
L	91.9	13.9	1.6	90	140	6.55
M	0	14.9	0.0	245	444	6.70
N	0	15.7	0.0	NS	NS	NS
O	100	14.5	1.4	82	154	6.76
P	100	15.7	1.7	66	136	6.92
Q	100	16.8	1.7	82	179	6.77
R	0	16.8	0.0	215	259	7.21
S	100	18.1	2.5	86	145	6.84

A multiple regression model was constructed to determine the influence of temperature, STR, BOD_5, COD, and pH on the percent denitrification in the septic tank. Only 15 of the 22 study periods were used for the reasons previously described. The model yielded a multiple regression coefficient $R^2 = 0.85$ $(p=0.005)$. Probability values (p) of the variable coefficients show with statistical significance $(p<0.05)$ that the HRT, STR, and BOD_5 explained the most variation in the degree of denitrification in the septic tank (Table 4.37). However, the HRT, STR, and BOD_5 concentration in the septic tank are not independent of each other.

Table 4.37 Variables, coefficients and probability values for the multiple regression model of denitrification in the septic tank at Site #2.

Variable	Coefficient	p-value
Intercept	273.64	0.024 **
Temperature	0.19	0.865
HRT	29.17	0.003 **
STR	-5.52	0.008 **
BOD_5	-0.62	0.003 **
COD	0.07	0.086
pH	-23.96	0.127
** Statistical Significance ($p<0.05$)		

The household water usage determines the STR, HRT, and BOD_5 concentration in the septic tank. As the water usage increases the HRT and BOD_5 concentration decrease. Increased water usage also causes the pump in the RSF to recycle more effluent from the sand filter to the septic tank, and this increases the STR. When the STR is increased, the volume of reduced BOD_5 recycled to the septic tank is greater and causes dilution of the influent household BOD_5. Therefore, incomplete denitrification in the septic tank at Site #2 can be attributed to reduced BOD_5, or reduced carbon availability in the septic tank. This probably has the most effect when a high degree of BOD_5 removal occurs in the sand filter and the household water usage increases.

The model also shows that temperature and pH did not have a significant effect on denitirification in the septic tank at Site #2. This supports the conclusions that temperature and pH levels were within the range of the microorganisms.

As noted previously, denitrification does not occur solely in the septic tank. Denitrification of the nitrate-N produced in the sand filter ranged between 54 and 96% complete. The degree of denitrification in the sand filter varies according to the changes in the thickness of the saturated bottom portion of the RSF. When no flow is pumped from the sand filter to Dosing Chamber #2, the saturated thickness is at its maximum level, equal to the overflow level to Dosing Chamber #2. The average degree of denitrification in the sand filter was 86% complete (range 68 to 96%) when the sand filter was at the overflow level (OFR = 1.0Q) during study periods #1, G, H, I, L, M, N, O, P, and R.

The average degree of denitrification in the sand filter was only 75% complete (range 54 to 90%) when the level of saturation in the RSF fluctuated below the overflow level (OFRs 0.0 to 0.5Q) in response to pumping from the pump chamber of the RSF during study periods #2, #3, B, C, E, F, J, K, and S. It is suspected that denitrification is less complete in the bottom of the sand filter during these study periods since less stagnation occurs in the rock storage unit of the RSF. This occurs because the level of saturation in the rock storage unit fluctuates due to the media's response to pumping from the pump chamber of the RSF. Decreased carbon availability probably accounts for the variations in denitrification in the rock storage unit of the sand filter during study periods with similar saturated thickness. Since few samples were collected from the monitoring well in the rock storage unit, variations in denitrification within the rock storage unit due to decreased carbon availability cannot be quantified.

4.253 Total Nitrogen

TN and chloride concentrations in Dosing Chamber #1, the pump chamber of the sand filter, and Dosing Chamber #2 from 1991 to 1995 are shown in Figures 4.53, 4.54, and 4.55. It should be noted that the concentrations in the septic tank

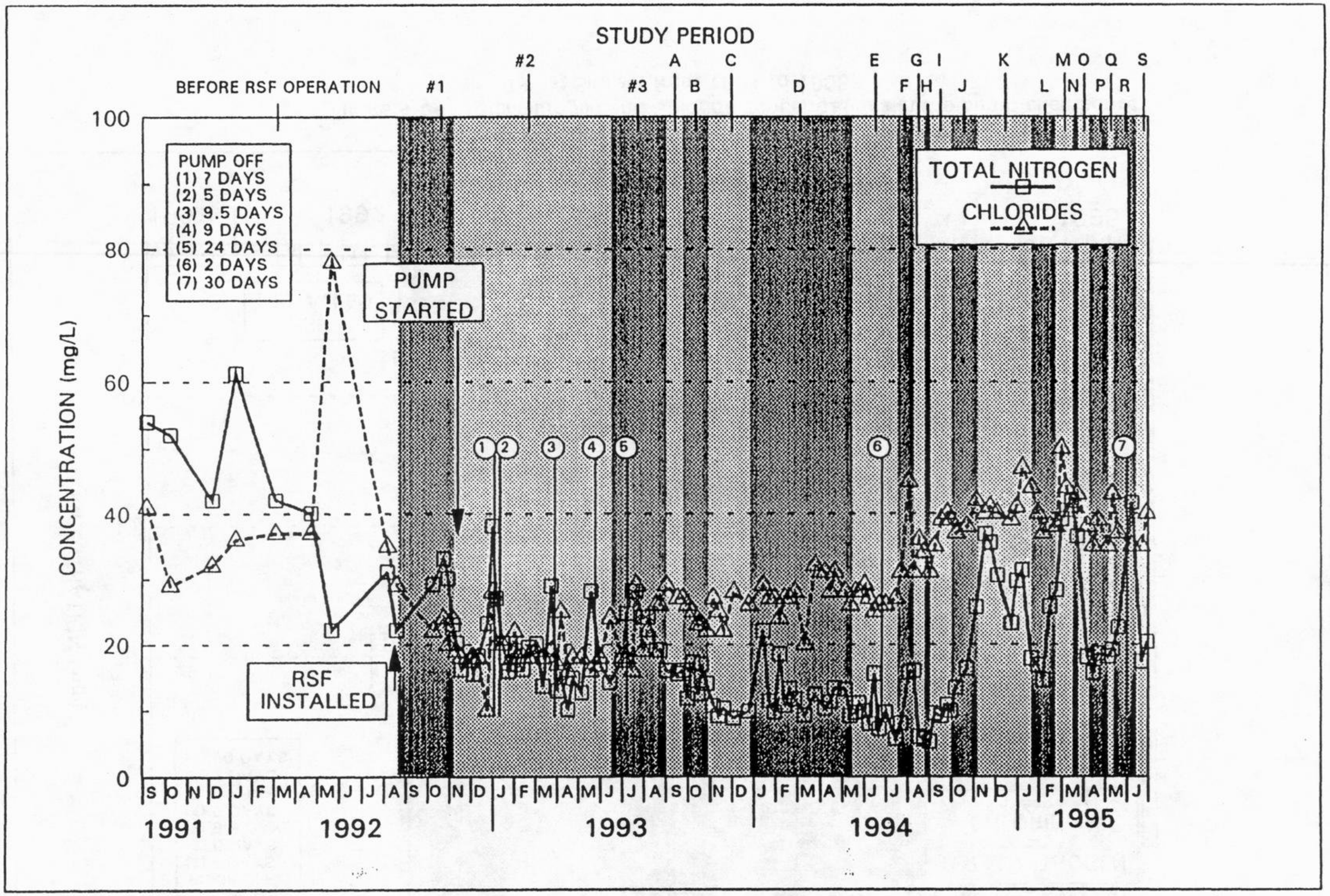


Figure 4.53 Total Nitrogen and Chloride concentrations in Dosing Chamber #1 at Site #2 from 1991 to 1995.

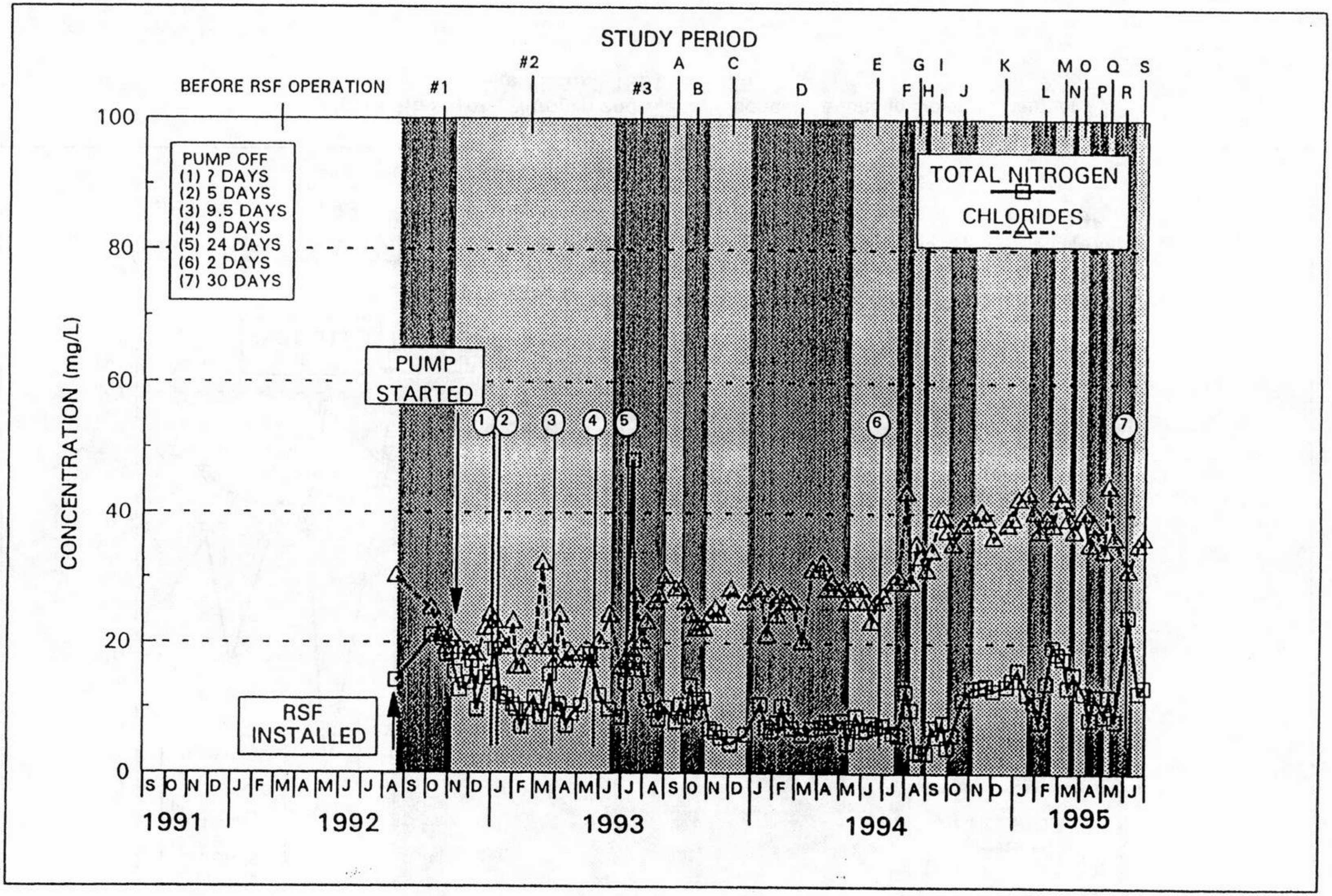


Figure 4.54 Total Nitrogen and Chloride concentrations in the pump chamber of the RSF at Site #2 from 1991 to 1995.

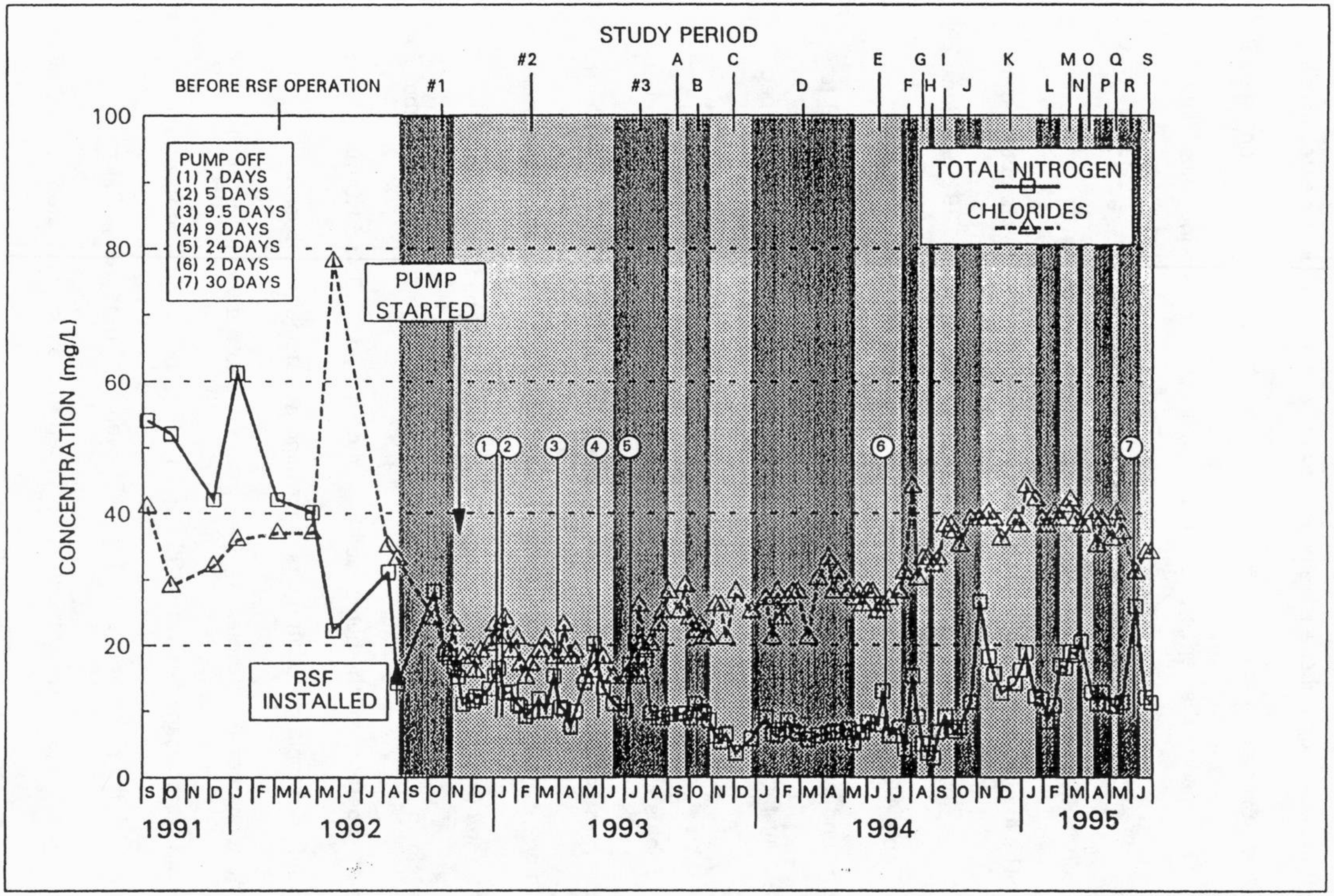


Figure 4.55 Total Nitrogen and Chloride concentrations in Dosing Chamber #2 at Site #2 from 1991 to 1995.

effluent in Dosing Chamber #1 are diluted by recirculated effluent treated in the sand filter. Exceptions are made for study periods #1, M, N, and R when effluent was not recycled to the septic tank. The samples collected during these time periods represent undiluted household concentrations and the mass balances predict these concentrations as the estimated household concentrations.

The loading and removal of TN to the system at Site #2 was obtained from the mass balances of all the study periods between August 13, 1992 and June 29, 1995 (Table 4.38). The reduction of the TN generated in the household and applied to the mound system from Dosing Chamber #2 at Site #2 ranged from 28.9 to 90.8% over the course of the study with the various flow conditions described earlier.

The estimated concentration of TN in the household waste was calculated from the mass balance calculations for all the study periods at Site #2. From these calculations the average TN concentration in the household waste was estimated to be 40.9 mg/l (range 18.1 to 92.1 mg/l). The average TN concentration in the original dosing chamber before the installation of the RSF and Dosing Chamber #1 before the operation of the pump was 37.1 mg/l (range 22.0 to 61.0 mg/l). The estimated household concentrations range slightly outside the observed concentrations before treatment by the system due to the variability of the household water usage between 1991 and 1995. The average concentration of TN pumped from the pump chamber of the RSF was 10.6 mg/l (range 3.1 to 19.4 mg/l). The average concentration of TN applied to the mound system from Dosing Chamber #2 was 11.5 mg/l (range 3.0 to 20.5 mg/l).

Table 4.38 Total Nitrogen removal by the RSF system for all study periods at Site #2.

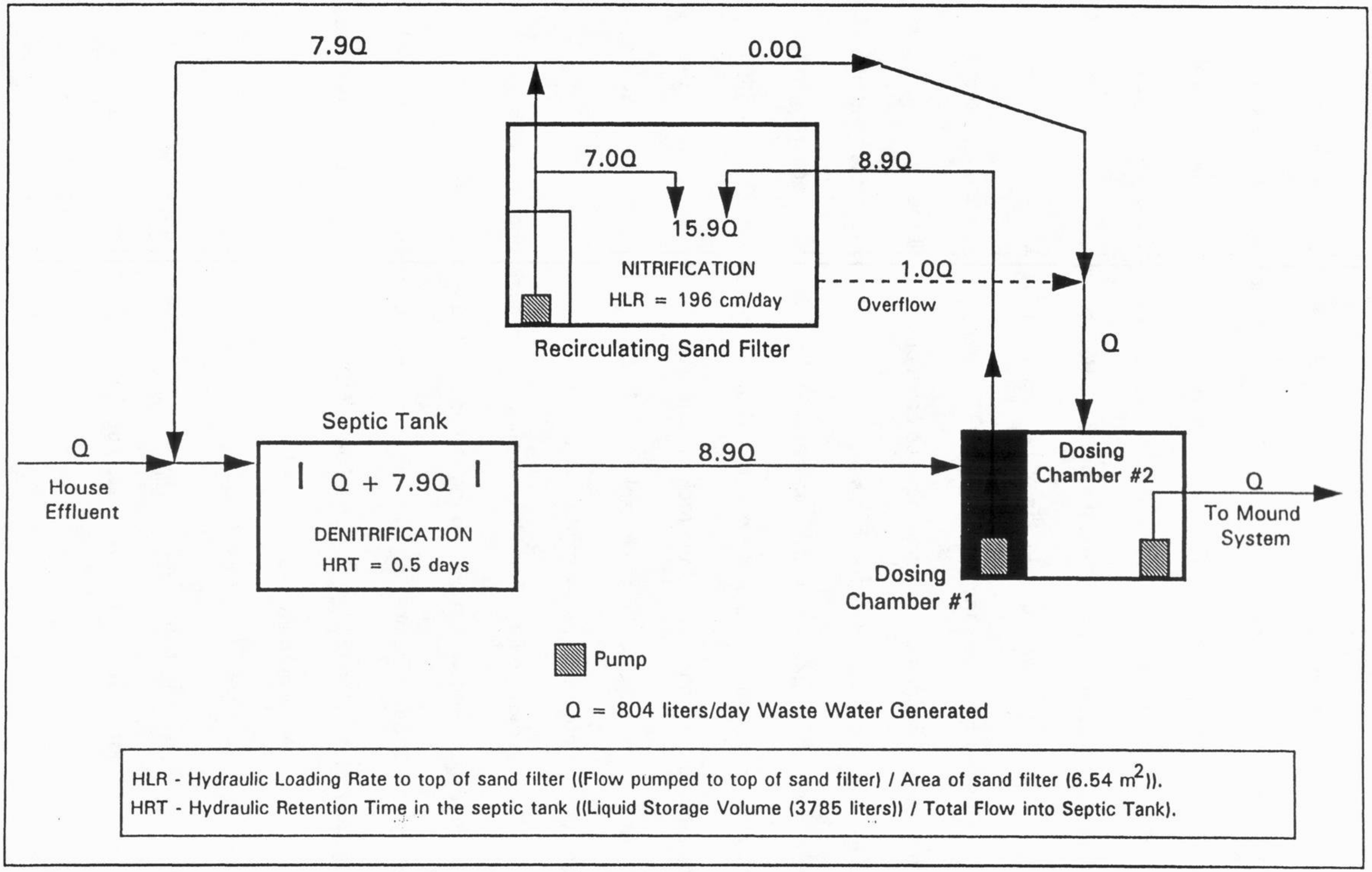
Study Period	Start	End	Mass From House (grams/day)	Removal in Septic Tank (grams/day)	Removal in Sand Filter (grams/day)	Removal in Dosing Chamber #2 (grams/day)	Dosing Chamber #2 to Mound (grams/day)	System Removal (grams/day)	Percent Removed
#1	8/13/92	11/5/92	42.8	0.0	14.0	-1.5	30.4	12.4	28.9
#2	11/6/92	6/15/93	46.5	8.2	19.7	-0.8	19.5	27.0	58.1
#3	6/16/93	8/31/93	72.8	15.1	40.7	-0.2	17.2	55.6	76.4
A	9/1/93	9/23/93	57.9	18.4	26.6	-0.1	13.0	44.9	77.5
B	9/24/93	10/26/93	59.2	27.6	17.5	1.4	12.7	46.6	78.6
C	10/27/93	12/23/93	45.1	16.8	21.9	0.1	6.3	38.8	86.1
D	12/24/93	5/13/93	49.0	14.5	23.6	0.3	10.7	38.4	78.2
E	5/14/94	7/15/93	33.1	8.5	11.6	-1.3	14.2	18.9	57.0
F	7/16/94	8/9/93	41.4	12.1	19.1	5.1	5.1	36.3	87.7
G	8/10/94	8/23/93	25.0	5.7	16.4	-0.9	3.8	21.2	84.9
H	8/24/94	9/1/94	26.4	8.8	15.0	0.1	2.4	23.9	90.8
I	9/2/94	9/29/94	29.3	10.3	15.0	-1.0	5.1	24.3	82.8
J	9/30/94	11/7/94	43.7	7.8	29.1	-3.4	10.2	33.5	76.7
K	11/8/94	1/18/94	64.3	11.1	43.7	-1.5	11.0	53.3	83.0
L	1/19/95	2/22/95	57.6	11.1	29.1	1.5	15.9	41.7	72.4
M	2/23/95	3/15/95	51.6	0.0	30.8	-2.0	23.0	28.6	55.4
N	3/16/95	3/23/95	50.6	0.0	29.6	-7.4	28.5	22.1	43.7
O	3/24/95	4/6/95	41.3	8.4	17.4	-0.6	16.1	25.2	61.0
P	4/7/95	5/5/95	51.0	11.0	27.3	-2.5	15.3	35.8	70.1
Q	5/6/95	5/11/95	53.1	11.9	25.8	1.4	14.0	39.1	73.6
R	5/12/95	6/13/95	43.8	0.0	22.0	-3.5	25.5	18.3	41.7
S	6/14/95	6/28/95	40.6	15.0	15.9	0.9	8.8	31.8	78.4

(-) Indicates an increase in dosing chamber #2 relative to the sand filter due to higher concentrations being detected in dosing chamber #2
NS No Sample Available

The mass balances predict an increase in TN within Dosing Chamber #2 during some study periods. This occurs when the concentrations measured in Dosing Chamber #2 are greater than those observed in the sand filter. The concentrations being pumped and/or overflowing to Dosing Chamber #2 were assumed to be those concentrations found within the pump chamber of the RSF. However, due to the variation of concentrations with location in the sand filter it is unlikely that the concentrations in the overflow to Dosing Chamber #2 were the same as those in the pump chamber. Unfortunately, too few samples were collected from the different locations in the RSF to predict the concentrations overflowing to Dosing Chamber #2, and therefore, any additional removal which may have occurred in can not be determined. It is suspected that denitrification would not be significant since the effluent in Dosing Chamber #2 is aerobic and depleted in available carbon.

Maximum Removal

A maximum TN reduction of 90.8% was accomplished during study period H (August 24 to September 1, 1994). The household water usage during this time was 804 lpd (212 gpd). The flow conditions used to achieve this removal were an SFR of 15.9Q, an STR of 7.9Q, a DR of 0.0Q, and an OFR of 1.0Q. These flow conditions yield a hydraulic loading rate to the top of the sand filter of 196 cm/day (48 gpd/ft_2) and a hydraulic retention time in the septic tank of 0.5 days. The flow schematic for study period H can be found in Figure 4.56. With no flow being directed to Dosing Chamber #2, the wastewater levels in the RSF were maintained at the overflow level and the pump ran through the maximum of 48 cycles/day.



Q = 804 liters/day Waste Water Generated

HLR - Hydraulic Loading Rate to top of sand filter ((Flow pumped to top of sand filter) / Area of sand filter (6.54 m^2)).
HRT - Hydraulic Retention Time in the septic tank ((Liquid Storage Volume (3785 liters)) / Total Flow into Septic Tank).

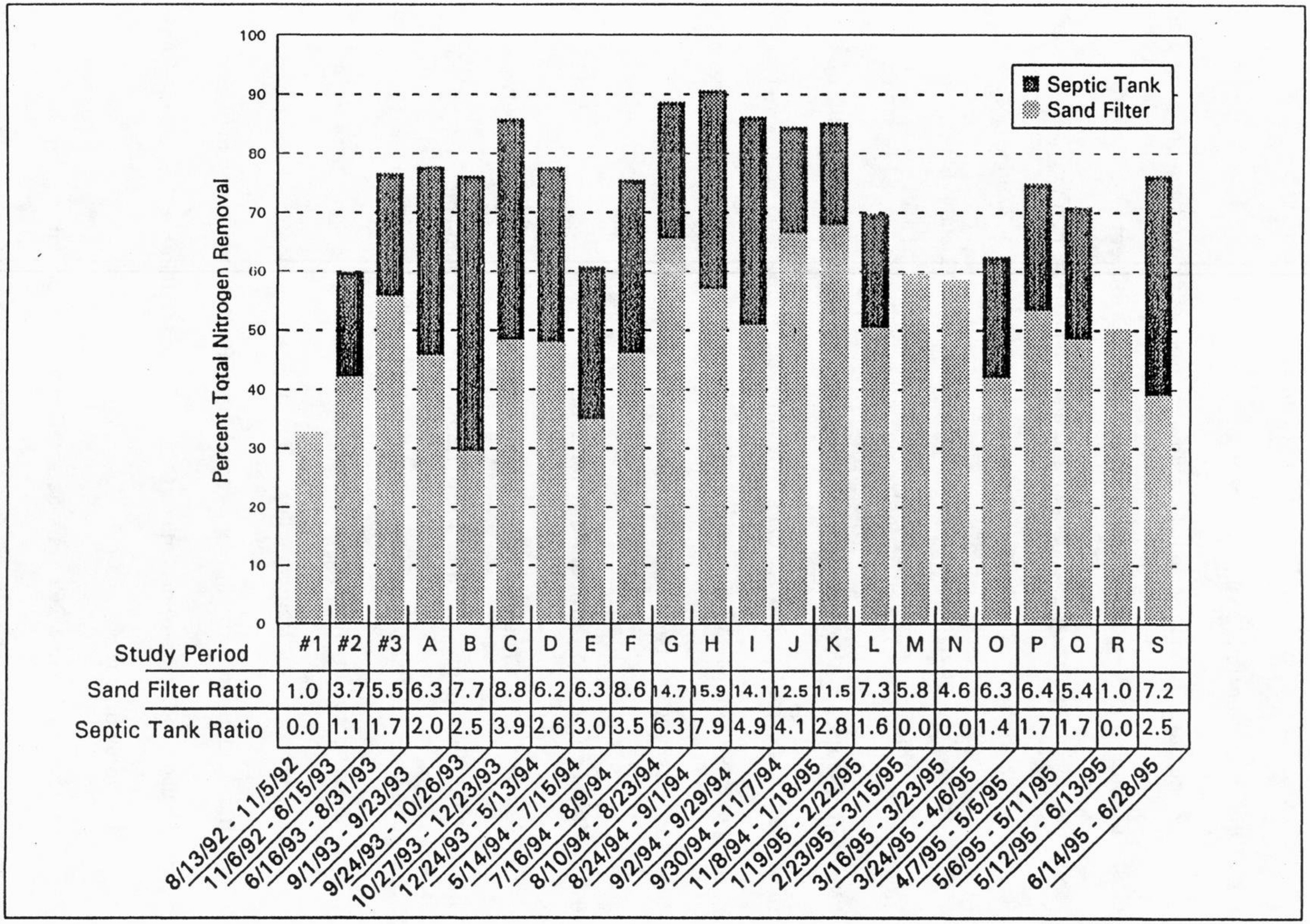
Figure 4.56 Flow schematic for study period H at Site #2.

Of the 26.4 grams/day (0.058 lbs/day) of TN estimated to be generated in the household only 2.4 grams/day (0.005 lbs/day) were applied to the mound system from Dosing Chamber #2. This would represent an annual loading to the mound system of only 0.8 kg/year (1.825 lbs/year) instead of 9.6 kg/year (21.2 lbs/year) being applied with out treatment by the RSF system.

Removal in Sand Filter and Septic Tank

Figure 4.57 shows the percentage of TN removed by the sand filter and septic tank determined by the mass balance calculations of all the study periods at Site #2. As also seen at the previous site, more TN is removed in the sand filter than in the septic tank. Although volatilization of NH_4^+-N may be occurring in the sand filter, it is suspected that the majority of the TN removed in the sand filter occurs by denitrification within the rock storage unit. Examination of study periods without flow recycled to the septic tank (#1, M, N, and R) reveals TN removal due entirely by treatment in the sand filter alone.

During study periods #1 and R the pump in the pump chamber was non-operational. Effluent was applied to the top of the sand filter in a single pass applied from Dosing Chamber #1. The forward flows (Q) during study periods #1, and R were 1,491 lpd (394 gpd) and 1,364 lpd (360 gpd), respectively. The sand filter ratio (SFR) for both study periods #1, and R is 1.0Q. The hydraulic loading rates (HLR) to the top of the sand filter were 23 cm/day (5.6 gpd/ft^2) and 21 cm/day (5.2 gpd/ft^2), respectively. Without flow recycled to the septic tank, the hydraulic retention times (HRT) were 2.5 and 2.8 days. TN removals of 28.9 and



Study Period	#1	#2	#3	A	B	C	D	E	F	G	H	I	J	K	L	M	N	O	P	Q	R	S
Sand Filter Ratio	1.0	3.7	5.5	6.3	7.7	8.8	6.2	6.3	8.6	14.7	15.9	14.1	12.5	11.5	7.3	5.8	4.6	6.3	6.4	5.4	1.0	7.2
Septic Tank Ratio	0.0	1.1	1.7	2.0	2.5	3.9	2.6	3.0	3.5	6.3	7.9	4.9	4.1	2.8	1.6	0.0	0.0	1.4	1.7	1.7	0.0	2.5
	8/13/92 - 11/5/92	11/6/92 - 6/15/93	6/16/93 - 8/31/93	9/1/93 - 9/23/93	9/24/93 - 10/26/93	10/27/93 - 12/23/93	12/24/93 - 5/13/94	5/14/94 - 7/15/94	7/16/94 - 8/9/94	8/10/94 - 8/23/94	8/24/94 - 9/1/94	9/2/94 - 9/29/94	9/30/94 - 11/7/94	11/8/94 - 1/18/95	1/19/95 - 2/22/95	2/23/95 - 3/15/95	3/16/95 - 3/23/95	3/24/95 - 4/6/95	4/7/95 - 5/5/95	5/6/95 - 5/11/95	5/12/95 - 6/13/95	6/14/95 - 6/28/95

Figure 4.57 Percent Total N removal in the septic tank and sand filter at Site #2.

41.7% were achieved by the RSF system at Site #2 in a single pass applied to the top of the sand filter from Dosing Chamber #1.

During study periods M and N the flow control 'valve' to the septic tank was closed off and no flow was directed to the septic tank. Effluent was applied to the top of the RSF from Dosing Chamber #1 and also from the sand filter. The forward flows through the system during study periods M and N were 1,278 lpd (338 gpd) and 1,389 lpd (367 gpd), respectively. The SFRs were 5.8Q and 4.6Q during study periods M and N. The HLRs to the top of the sand filter were 114 cm/day (28 gpd/ft^2) and 97 cm/day (24 gpd/ft^2). With no flow being directed to the septic tank, the HRTs were 3.0 and 2.7 days. TN removals of 55.4 and 43.7% were achieved by the RSF system in 5.8 and 4.6 passes applied to the top of the sand filter from Dosing Chamber #1 and the pump in the pump chamber of the RSF.

4.26 Volatile Organic Compound Attenuation

Samples for the analyses of VOCs were taken on eight occasions between July 8, 1993 and June 29, 1995. Table 4.39 shows the compounds detected in Dosing Chamber #1, the pump chamber of the RSF, and Dosing Chamber #2. VOCs were detected at one or all of the sample locations on seven of the eight sample occasions. No VOCs were detected at any sample location on August 16, 1994. It should be noted that the sample of May 19, 1995 was contaminated during handling and/or transport since VOCs were detected in the "trip blank". As a result, this study date is omitted from this discussion.

Fourteen different VOCs were detected in at least one of the sample locations at Site #2, double the number of VOCs detected at Site #1. The number and concentrations of the VOC species detected was greatest in Dosing Chamber #1 and reduced in the RSF and Dosing Chamber #2. This indicates removal of VOCs in the RSF.

The most frequently detected compound in Dosing Chamber #1 was 1,4-dichlorobenzene, with concentrations ranging from <0.7 to 39.8 µg/L. Concentrations of 1,4-dichlorobenzene in the RSF ranged between <0.7 to 4.3 µg/L, while those in Dosing Chamber #2 ranged between <0.7 to 2.8 µg/L. The lower concentrations in the RSF and Dosing Chamber #2 indicate 71 to 100% removal of 1,4-dichlorobenzene.

Toluene was the second most frequently detected compound in Dosing Chamber #1 with concentrations ranging from <0.5 µg/L to 49.9 µg/L. Toluene concentrations in the sand filter and Dosing Chamber #2 were never detected above the limits of quantitation (0.5 µg/L). This indicates complete removal of toluene by the RSF system.

The third most frequently detected compound in Dosing Chamber #1 was sec-butylbenzene (range <0.5 to 10.7 µg/L). No sec-butylbenzene was detected above the limits of quantitation (0.5 µg/L) in either the RSF or Dosing Chamber #2. This indicates complete removal of sec-butylbenzene by the RSF system.

Table 4.39 Volatile Organic Compounds detected in the Dosing Chamber #1, the pump chamber of the RSF, and Dosing Chamber #2 at Site #2

Sample date	July 8, 1993			October 19, 1993			January 21, 1994			April 15, 1994		
Compound (ppb)	DC#1	RSF	DC#2	DC#1	RSF	DC#2	DC#1	RSF	DC#2	DC#1	RSF	DC#2
Naphthalene	ND	ND	ND	2.0	ND	ND	ND	ND	ND	ND	ND	ND
Dichlorodifluoromethane	ND	ND	ND	ND	18.0	ND	ND	ND	ND	ND	ND	ND
n-Butylbenzene	1.0	ND	ND	ND	ND	ND	2.5	ND	ND	ND	ND	ND
1,1,1-Trichloroethane	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND
isopropylbenzene	1.5	ND	ND	ND	ND	ND	1.8	ND	ND	2.4	ND	ND
m+p Xylene	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	2.8	ND
Toluene	49.9	ND	ND	ND	ND	ND	8.5	ND	ND	10.0	ND	ND
sec-Butylbenzene	10.7	ND	ND	10.2	ND	ND	ND	ND	ND	ND	ND	ND
Dichloromethane	5.5	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND
1,4-Dichlorobenzene	39.8	ND	ND	10.8	1.6	1.2	9.7	3.7	2.8	10.1	4.3	2.4
o-Xylene/Styrene	ND	ND	ND	ND	ND	ND	ND	ND	ND	3.0	ND	ND
1,2,4-Trichlorobenzene	ND	ND	ND	ND	ND	ND	1.5	ND	ND	ND	ND	ND
Chloroform	ND	ND	ND	ND	ND	ND	7.3	ND	ND	ND	ND	ND
Isopropylbenzene	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND
Temperature (°C)	NS	NS	NS	NS	NS	NS	14.3	15.0	11.4	16.7	15.4	13.7

NS No Sample Taken
ND Compound not detected

DC#1 - Dosing Chamber #1
DC#2 - Dosing Chamber #2
RSF - Pump Chamber of RSF

(Continued)

Table 4.39 (continued)

Sample date	August 16, 1994			February 3, 1995			May 19, 1995				June 29, 1995		
Compound (ppb)	DC#1	RSF	DC#2	DC#1	RSF	DC#2	DC#1	RSF	DC#2	Trip B	DC#1	RSF	DC#2
Naphthalene	ND	ND	ND	ND	ND	ND	ND	ND	ND	1.3	ND	ND	ND
Dichlorodifluoromethane	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND
n-Butylbenzene	ND	ND	ND	ND	ND	ND	ND	1.4	1.3	ND	ND	ND	ND
1,1,1-Trichloroethane	ND	ND	ND	ND	ND	ND	ND	ND	ND	3.4	ND	ND	ND
isopropylbenzene	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND
m+p Xylene	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND
Toluene	ND	ND	ND	1.9	ND	ND	18.1	ND	1.0	ND	24.3	ND	ND
sec-Butylbenzene	ND	ND	ND	0.6	ND	ND	2.8	ND	ND	ND	3.1	ND	ND
Dichloromethane	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND
1,4-Dichlorobenzene	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND
o-Xylene/Styrene	ND	ND	ND	ND	ND	ND	1.9	ND	ND	ND	ND	ND	ND
1,2,4-Trichlorobenzene	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND
Chloroform	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND
Isopropylbenzene	ND	ND	ND	ND	ND	ND	1.4	ND	ND	ND	ND	ND	ND
Temperature (°C)	20.2	20.3	17.8	10.7	12.8	10.8	17.2	16.7	14.5	NS	19.9	19.2	16.2

NS No Sample Taken
ND Compound not detected

DC#1 - Dosing Chamber #1
DC#2 - Dosing Chamber #2
RSF - Pump Chamber of RSF
Trip B - Trip blank transported with samples.

4.27 Groundwater Impacts

The groundwater at Site #2 has been monitored between 1990 and 1995 in three studies conducted through the University of Wisconsin-Stevens Point. The first study, conducted by Shaw and Turyk (1992), investigated the mound system at this site as part of a study comparing nitrate-N loading to groundwater from different types of alternative soil absorption systems. Site #2 is referred to as site SN1 in Shaw and Turyk (1992). With homeowner cooperation Site #2 was selected for the denitrification studies because the plume from the mound system was well identified. This section describes the changes in the groundwater conditions resulting from the installation of the RSF system through June of 1995. The figures contain a separation indicating the conditions seen before the installation of the RSF (BEFORE RSF) and conditions after the installation of the RSF (AFTER RSF). The locations of the monitoring wells at Site #2 can be found in Figure 4.58.

4.271 Water Table Fluctuations

The water table at Site #2 fluctuated 2.0 m (6.2 ft) and was 1.4 to 2.0 m (4.5 to 11.2 ft) below the ground surface throughout the course of monitoring from October, 1990 to June, 1995 (Figure 4.59). Seasonal changes are again evident as is the severe fluctuation between the falls of 1993 and 1994.

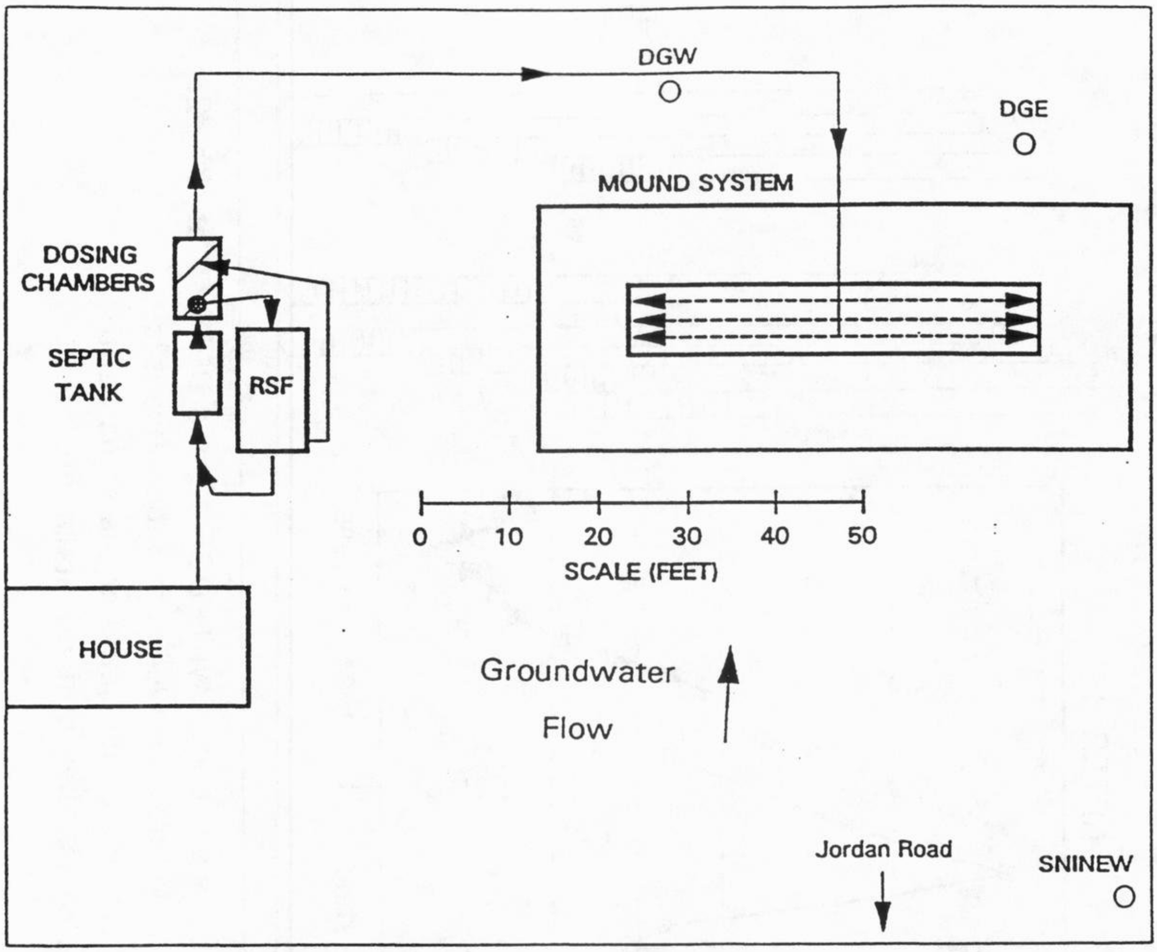


Figure 4.58 Location of monitoring wells at Site #2.

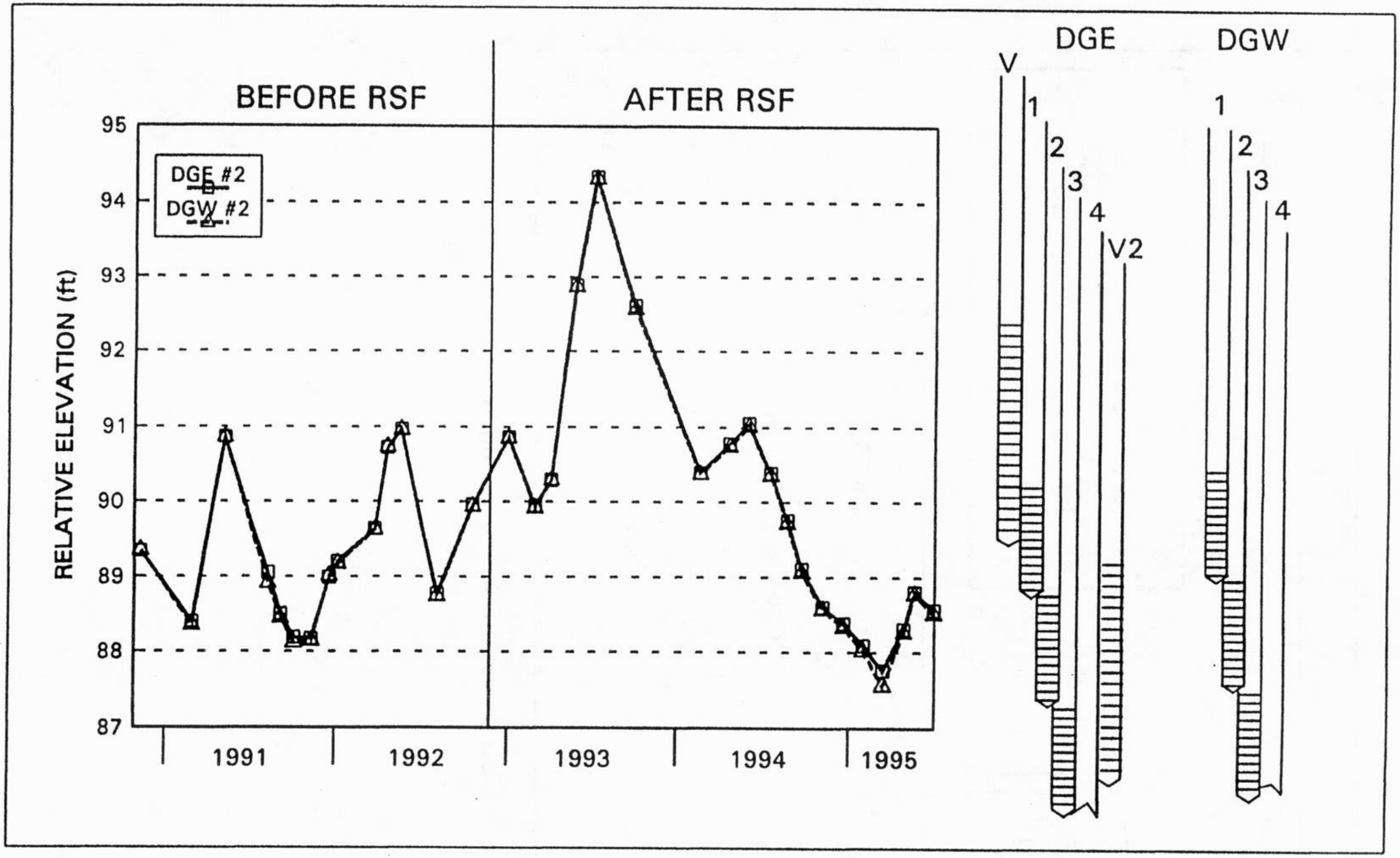


Figure 4.59 Watertable fluctuations as seen in wells DGE #2 and DGW #2 at site #2. Port #4 in each well shows no well screen because it is uncertain if the port was constructed at the same depth as port #3 or if it was broken during installation.

4.272 Nitrate-N Reductions

Figures 4.60, 4.61, 4.62, and 4.63 show the nitrate-N and chloride concentrations for all depths of down-gradient well nests DGE and DGW from 1990 to 1995. Tables 4.40 and 4.41 show the same data for all the sampling dates after the installation of the RSF. The plume is considered to have chloride concentrations greater than 10 mg/l. The sampling dates which had chloride concentrations less than 10 mg/l were omitted from the calculation of the average values for after the installation of the RSF because it is uncertain if the plume was within the screened interval of the well due to: 1) possible errors made in the water table measurements, or 2) possible errors made in obtaining the samples.

Table 4.42 shows the average nitrogen (TN or Nitrate-N), and average chloride concentrations: in wells DGE 1-4, DGE V or DGE V2, DGW 1-4, the up-gradient well SNINEW, and the concentrations applied to the mound system, for before and after the installation of the RSF. The average concentrations applied to the mound system were made for all the sample dates prior to the last date which was used in the groundwater averages. It should be noted that samples for groundwater conditions were collected less frequently than dosing chamber samples.

It is important to note that the chloride concentrations observed to be exiting the mound system are lower than the concentrations observed in the down-gradient monitoring wells (Table 4.42). The average concentrations of chloride after the installation of the RSF from October 20, 1992 to June 28, 1995 in Dosing Chamber #1, the RSF, and Dosing Chamber #2 were 28.5, 28.2, and 23.2 mg/l,

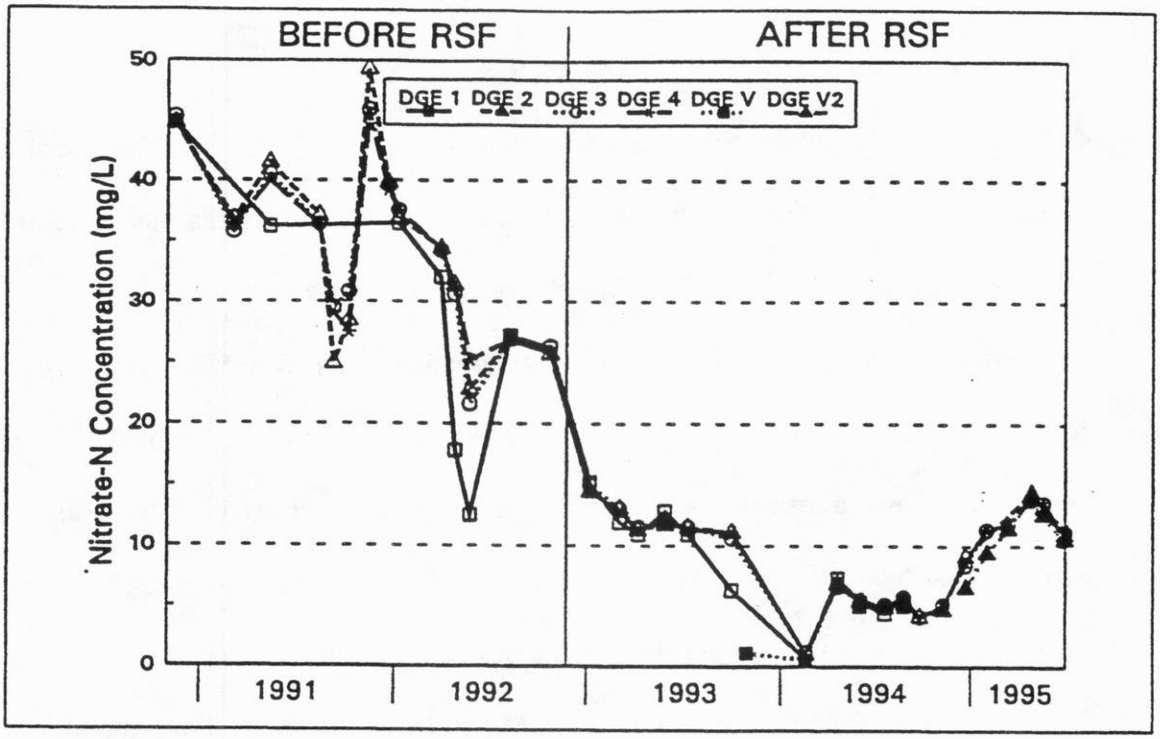


Figure 4.60 Nitrate-N concentrations in well nest DGE 1990 - 1995 Site #2

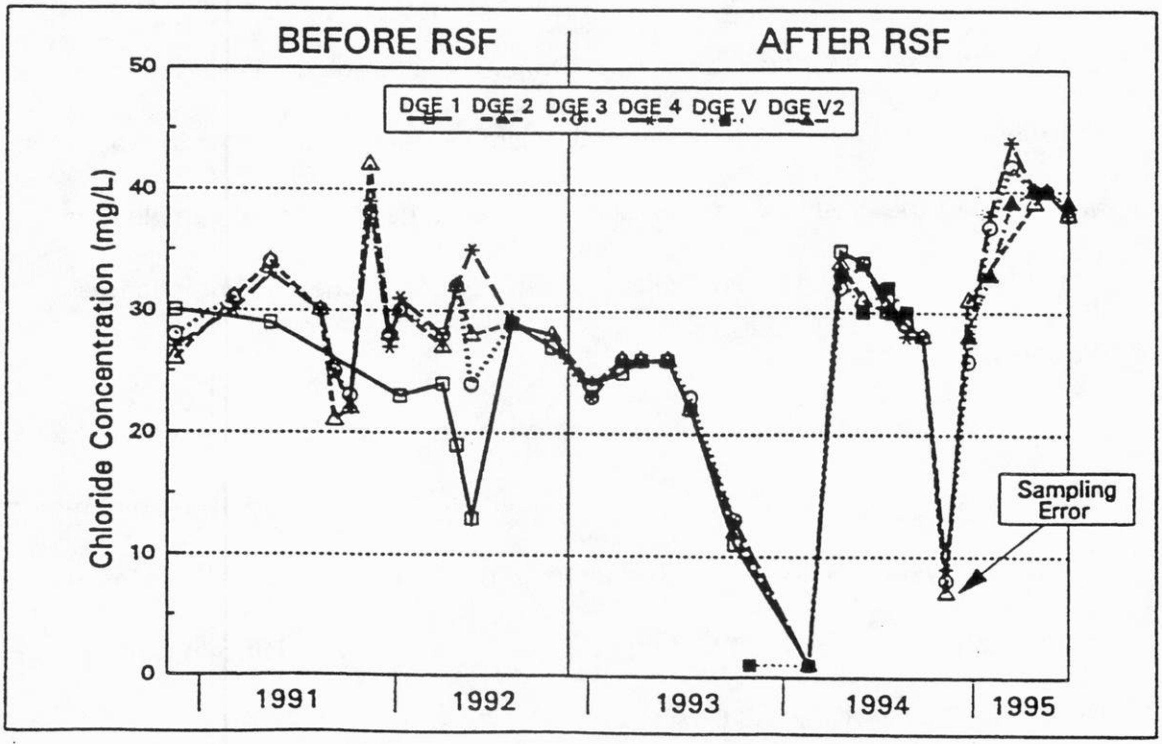


Figure 4.61 Chloride concentrations in well nest DGE 1990 - 1995 Site #2

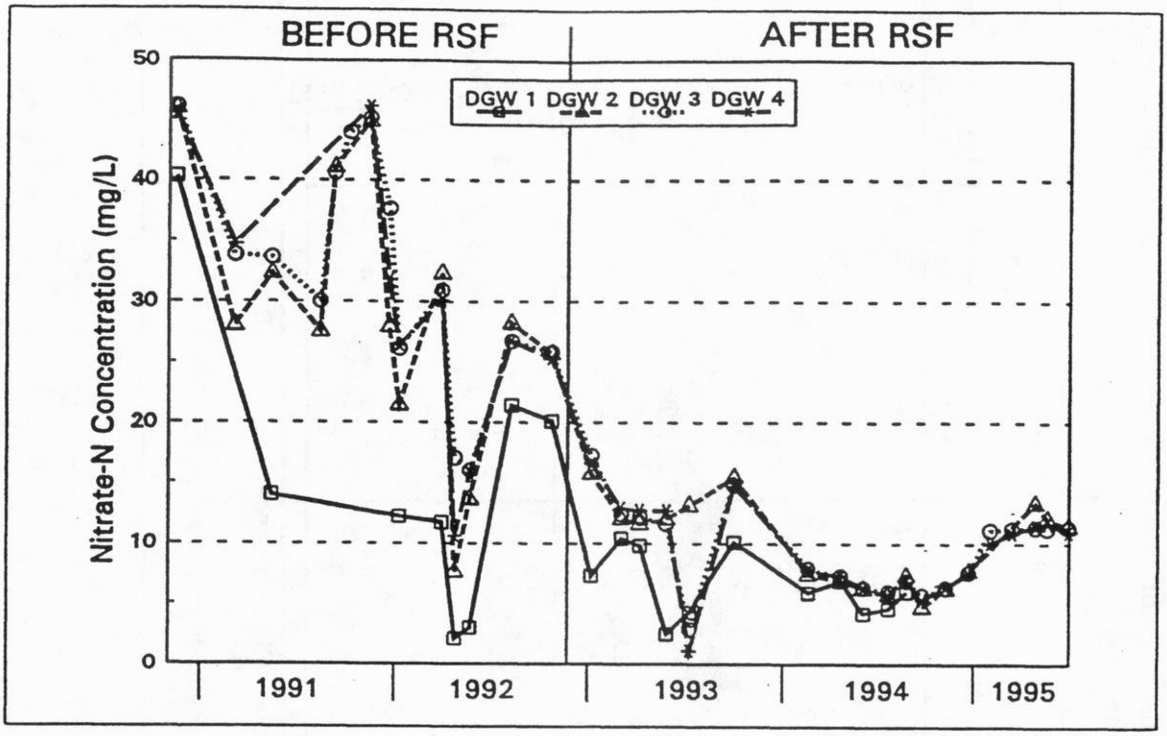


Figure 4.62 Nitrate-N concentrations in well nest DGW 1990 - 1995 Site #2

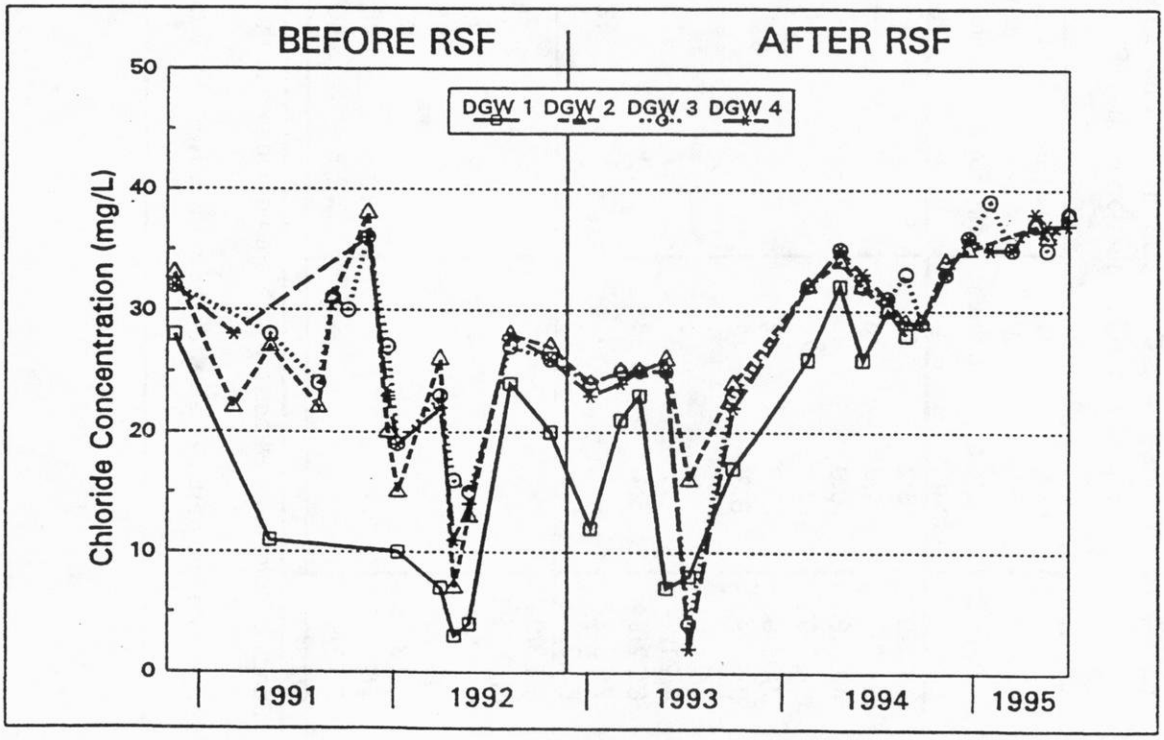


Figure 4.63 Chloride concentrations in well nest DGW 1990 - 1995 site #2

Table 4.40 **Nitrate-N and Chloride concentrations in groundwater monitoring wells DGE 1, V, V2, 2, 3, and 4 for all sample dates after the installation of the RSF system at Site #2.**

Date	DGE 1 Nitrate-N (mg/l)	DGE 1 Chloride (mg/l)	DGE V or V2 Nitrate-N (mg/l)	DGE V or V2 Chloride (mg/l)	DGE 2 Nitrate-N (mg/l)	DGE 2 Chloride (mg/l)	DGE 3 Nitrate-N (mg/l)	DGE 3 Chloride (mg/l)	DGE 4 Nitrate-N (mg/l)	DGE 4 Chloride (mg/l)
1/04/93	15.2	24	-	-	14.4	24	15.0	23	14.4	23
3/02/93	11.9	25	-	-	13.0	26	13.0	26	12.8	26
4/06/93	10.9	26	-	-	11.3	26	11.5	26	11.3	26
5/26/93	12.8	26	-	-	11.7	26	12.0	26	12.0	26
7/09/93	10.9	22	-	-	11.5	22	11.5	23	11.3	22
9/30/93	6.3	11	-	-	11.1	12	10.6	13	11.1	13
2/18/94	0.6	1	0.5	<1	1.0	1.0	9.8	2.0	1.0	<1
4/21/94	7.3	35	6.9	33	6.8	34	1.1	<1	6.7	33
6/02/94	5.4	34	5.0	30	5.4	31	6.6	32	5.6	34
7/19/94	4.4	31	4.9	32	4.9	30	5.4	34	4.9	30
8/23/94	5.5	29	5.0	30	5.6	29	5.1	30	5.8	28
9/23/94	-	-	-	-	4.1	28	5.7	29	4.1	28
11/07/94	-	-	-	-	4.7	7	4.1	28	5.0	9
12/21/94	-	-	6.5	28	9.1	31	5.1	8	8.7	28
1/31/95	-	-	9.4	33	-	-	8.4	26	11.4	38
3/13/95	-	-	11.4	39	-	-	11.3	37	11.7	44
4/26/95	-	-	13.9	40	14.3	39	11.7	42	14.0	40
5/19/95	-	-	12.5	40	13.1	40	13.8	40	13.2	40
6/29/95	-	-	10.5	39	10.9	38	13.5	40	11.3	38

Shading indicates well ports which are not located in the plume or have a Chloride concentration < 10 mg/l.

- Indicates no data available for the sample (well not yet installed or insufficient volume in well).

Table 4.41 Nitrate-N and chloride concentrations in groundwater monitoring wells DGW 1, 2, 3, and 4 for all sample dates after the installation of the RSF system at Site #2.

Date	DGW 1 Nitrate-N (mg/l)	DGW 1 Chloride (mg/l)	DGW 2 Nitrate-N (mg/l)	DGW 2 Chloride (mg/l)	DGW 3 Nitrate-N (mg/l)	DGW 3 Chloride (mg/l)	DGW 4 Nitrate-N (mg/l)	DGW 4 Chloride (mg/l)
1/04/93	7.3	12	15.7	24	17.2	24	16.6	23
3/02/93	10.4	21	12.0	25	12.4	25	12.9	24
4/06/93	9.8	23	11.6	25	12.2	25	12.7	25
5/26/93	2.5	7.0	12.0	26	11.6	25	12.7	25
7/09/93	4.3	8.0	13.1	16	2.9	4	1.1	2
9/30/93	10.1	17	15.4	24	14.8	23	14.7	22
2/18/94	5.9	26	7.4	32	7.9	32	7.7	32
4/21/94	6.9	32	6.9	34	7.3	35	7.3	35
6/02/94	4.2	26	6.2	32	6.3	32	6.3	33
7/19/94	4.6	30	5.7	30	6.0	31	5.7	31
8/23/94	6.0	28	7.3	29	6.9	33	6.9	29
9/23/94	-	-	4.7	29	5.7	29	5.3	29
11/07/94	-	-	6.2	34	6.3	33	6.4	33
12/21/94	-	-	7.7	35	7.6	36	7.5	36
1/31/95	-	-	-	-	11.1	39	10.0	35
3/13/95	-	-	-	-	11.2	35	10.8	35
4/26/95	-	-	13.3	37	11.3	37	11.5	38
5/19/95	-	-	12	36	11.2	35	11.7	37
6/29/95	-	-	11.4	38	11.3	38	10.9	37

Shading indicates well ports not located in the plume or have a Chloride concentration < 10 mg/l.

- Indicates no data available for the sample (well not yet installed or insufficient volume in well).

Table 4.42 **Average nitrogen and chloride concentrations, and nitrogen to chloride ratios (N:Cl) before and after the installation of the RSF system at Site #2.**

Well	BEFORE RSF			AFTER RSF								
	11/15/90 - 10/20/92			1/4/93 - 9/30/93			2/18/94 - 8/23/94			9/23/94 - 6/29/95		
	NO_3-N	Chloride	N:Cl	NO_3-N	Chloride	N:Cl	NO_3-N	Chloride	N:Cl	NO_3-N	Chloride	N:Cl
DGE - 1	29.1	24.3	1.2	11.3	22.3	0.5	5.7	32.3	0.2	-	-	-
DGE - V(2)	-	-	-	-	-	-	5.5	31.3	0.2	10.7	36.5	0.3
DGE - 2	34.3	29.1	1.2	12.2	22.7	0.6	5.7	31.0	0.2	10.3	35.2	0.3
DGE - 3	34.4	29.1	1.2	12.3	22.8	0.6	5.7	32.0	0.2	9.8	34.6	0.3
DGE - 4	34.3	29.7	1.2	12.2	22.7	0.6	5.8	31.3	0.2	10.6	36.5	0.3
DGW - 1	15.6	13.4	1.2	8.0	16.0	0.5	5.5	28.4	0.2	-	-	-
DGW - 2	29.0	23.8	1.2	13.3	24.8	0.6	6.7	31.4	0.2	9.2	34.8	0.3
DGW - 3	32.4	25.7	1.2	13.6	24.4	0.6	6.9	32.6	0.2	9.5	35.3	0.3
DGW - 4	29.3	23.9	1.2	13.9	23.8	0.6	6.8	32.0	0.2	9.3	35.0	0.3
SNINEW	0.2	131.4	0.0	<0.2	40.0	0.0	1.5	103.4	0.0	2.7	43.5	0.1
Entering MS	43.1*	40.6	1.2	12.7*	19.7	0.6	7.3*	27.3	0.3	13.1*	37.7	0.3

Average values exclude dates with chloride concentrations < 10 mg/l. Attributed to error in sampling or well missing the plume.

MS Mound System.

* Total Nitrogen Concentration. (Before values calculated from the original dosing chamber, After values calculated from Dosing Chamber #2)

\- Indicates no data available (well not yet installed or insufficient volume in well).

respectively. An unexplainable change occurs between the RSF and Dosing Chamber #2. It is widely accepted that chlorides are conservative, non-reactive in nature. However, chloride concentrations in Dosing Chamber #2 were consistently lower than the concentrations seen in Dosing Chamber #1 or the RSF. One logical explanation for decreased chloride concentrations in Dosing Chamber #2 would be infiltration or seepage of water into Dosing Chamber #2. While Dosing Chamber #2 is located fairly deep in the ground, no measurements where made to determine the exact depth; therefore it cannot be determined if water table elevations reached the elevation of Dosing Chamber #2.

A second clarification should be made regarding the chloride concentrations up-gradient of the mound system. The average chloride concentrations in the up-gradient well SNINEW were generally much higher than either the concentrations being applied to or seen down-gradient from the mound system. Figures 4.64 and 4.65 show the nitrate-N and chloride concentrations observed in up-gradient well SNINEW between 1990 and 1995. The up-gradient well is located only a few meters down-gradient of a road and may reflect the usage of road salt in the winter (Figure 4.58). Unfortunately, there were no up-gradient wells located any closer to the mound system and it is impossible to know the chloride concentrations immediately up-gradient; therefore, the amount of dilution that occurs as the groundwater moves between up-gradient well SNINEW and the mound system cannot be determined.

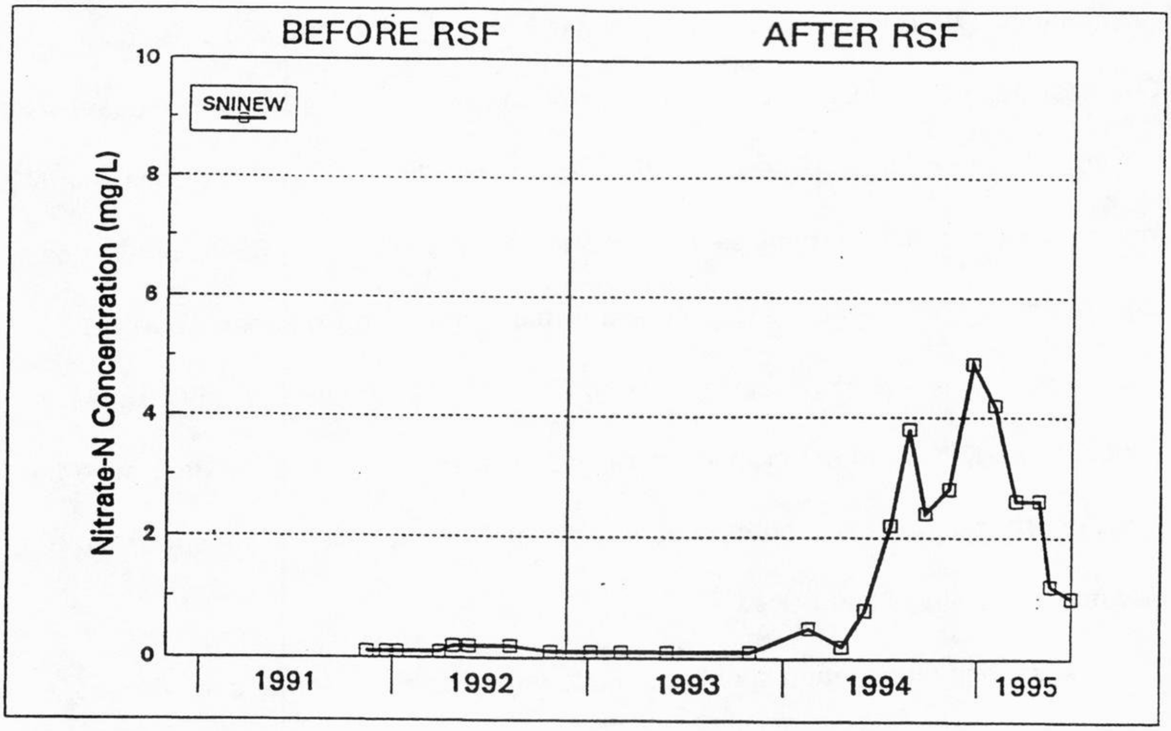


Figure 4.64 Nitrate-N concentrations in up-gradient well SNINEW 1990 - 1995
Site #2

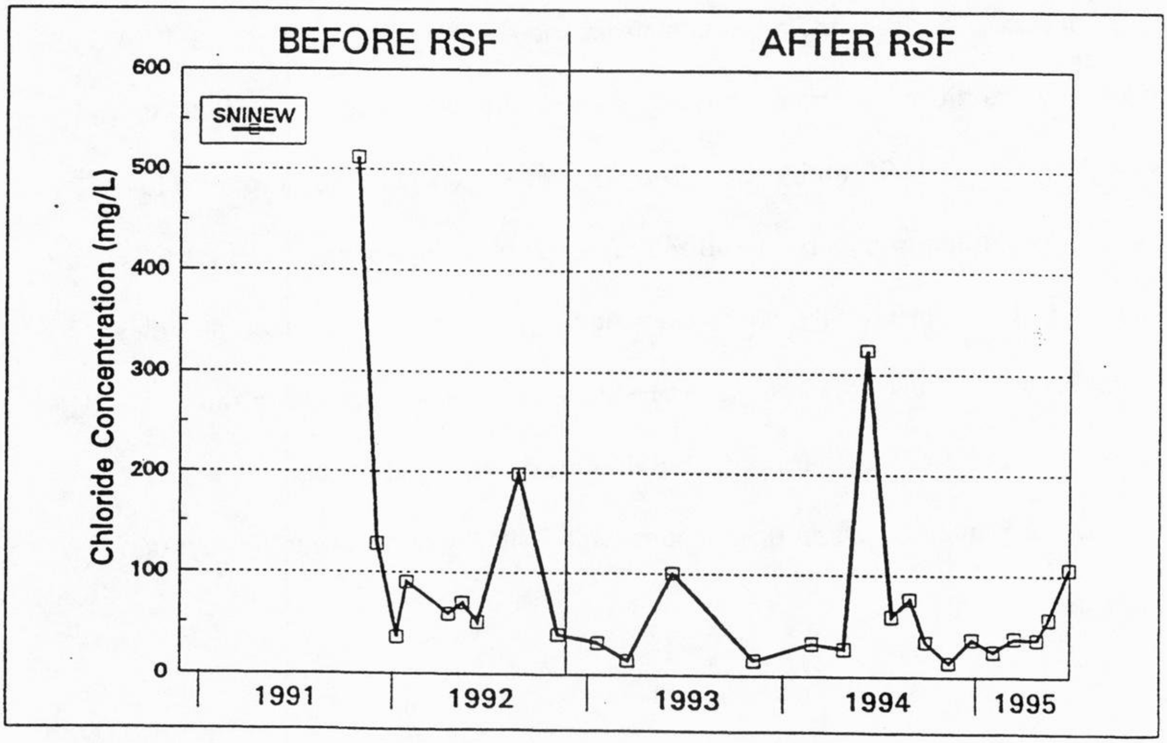


Figure 4.65 Chloride concentrations in up-gradient well SNINEW 1990 - 1995
Site #2

In the literature the ratio of the chloride concentrations in on-site system effluent to the chloride concentrations observed in the down-gradient wells (Cl:Cl) has been used to determine the degree of dilution/mixing of the effluent as it enters the groundwater and moves to the monitoring well (Shaw and Turyk, 1992). Also reported in the literature is the ratio of TN concentrations in the on-site system effluent to nitrate-N concentrations observed in down-gradient wells (TN:N). Comparison of the Cl:Cl and TN:N ratios can indicate if nitrogen and chloride respond in a similar fashion. Differences between the ratios could be attributed to nitrogen removal via denitrification in the soil absorption system and/or in the groundwater. Shaw and Turyk (1992) reported that the Cl:Cl and TN:N ratios averaged 1.4 (maximum 1.9) and 1.3 (maximum 1.6), respectively at Site #2 from 1990 to 1991. They concluded that both nitrate-N and chlorides behaved in a similar fashion as they moved into and through the groundwater and that no significant nitrogen removal occurred in the soil absorption system or groundwater.

Table 4.43 shows the TN:N and Cl:Cl ratios for before and after the installation of the RSF system at Site #2. The results observed before the installation of the RSF were similar to those previously documented. The ratios are larger for well nest DGW than DGE since it is located approximately 3 m farther down-gradient, hence more dilution/mixing occurs with greater distance. Also there is no appreciable difference between the TN:N and Cl:Cl ratios indicating that no denitrification occurs. However, after the installation of the RSF the Cl:Cl ratios indicate that it did not behave the same as nitrogen. The discrepancy in the Cl:Cl ratio before and after the installation of the RSF system results from the influence of the up-gradient chloride concentrations and the change seen in Dosing Chamber #2. Unfortunately, an adjustment for the up-gradient concentrations cannot be made since the chloride concentrations immediately up-gradient are not known.

Table 4.43 TN:Nitrate-N (TN:N) and Chloride to Chloride (Cl:Cl) ratios for before and after the installation of the RSF in wells DGE (1-4) and DGW (1-4) at Site #2, 1992 to 1995.

	Total Nitrogen : Nitrate (TN:N)			
Well	**Before RSF**	**After (1)**	**After (2)**	**After (3)**
DGE 1-4	1.3	1.1	1.3	1.3
DGW 1-4	1.6	1.0	1.1	1.4

	Chloride : Chloride (Cl:Cl)			
Well	**Before RSF**	**After (1)**	**After (2)**	**After (3)**
DGE 1-4	1.4	0.7	0.9	1.1
DGW 1-4	1.9	0.9	0.9	1.1

Before RSF 11/15/90 - 10/20/92
After (1) 1/4/93 - 9/30/93
After (2) 2/18/94 - 8/23/94
After (3) 9/23/94 - 6/29/95

Inspection of the TN:N ratios (Table 4.43) indicates that it reacted the same way before and after the RSF in periods (2) and (3). The TN:N ratio is less during after period (1) than before the RSF, reflecting that the down-gradient nitrate-N concentrations were greater than the TN concentrations applied to the mound system. This could be the result of nitrogen being released from storage in the soil absorption system as BOD$_5$ loading was reduced by secondary treatment in the RSF; similar results were seen at Site #1. Any release of nitrogen from storage in the soil absorption system resulting from the installation of the RSF cannot be quantified since the amount of dilution/mixing cannot be determined.

If it is assumed that the dilution/mixing factor did not significantly change after the installation of the RSF, then it can also be assumed that nitrogen and chloride behaved the same both before and after the installation of the RSF. Making this assumption enables a comparison of the nitrate-N to chloride (N:Cl) ratios before and after the installation of the RSF (Table 4.42). A reduction in the N:Cl ratio between the before and after periods is due to nitrogen removal by the RSF system. There is no appreciable difference between the N:Cl ratios applied to the mound system and those seen in the down-gradient monitoring wells. This has two implications: 1) no significant release or removal of nitrogen occurred in the soil absorption system or groundwater, and 2) the discrepancy caused by the elevated chloride concentrations did not have a significant effect on the nitrate and chloride ratio.

The effectiveness of the RSF in reducing nitrogen loading to groundwater can be determined by observation of the N:Cl ratios. These were determined for the average concentrations applied to the mound system and in wells DGE 1-4, DGE V or V2, and DGW 1-4 (Table 4.42). A decrease in the N:Cl ratio between the before and after periods is due to denitrification by the RSF system. The N:Cl ratio before the system was installed was 1.2, after the system was installed the ratio was reduced to 0.6, 0.2, and 0.3 in the down-gradient wells, a reduction of 64 to 77%. The reductions seen in the groundwater are similar to the reductions seen by the different levels of treatment achieved in the RSF for the various flow conditions used. The average TN removal determined from all the study periods at Site #2 was 70% (range 29 to 90%).

4.273 Phosphate

The original mound system at Site #2 was installed in 1983. The average hydraulic loading to the system from December 1991 to June 1995 was 1,337 lpd (range 596 to 3,090 lpd) [353 gpd (range 157 to 816 gpd).]. Wells DGE are DGW are 10.5 m (34 ft) and 13.5 m (44 ft), respectively, down-gradient of the mound system. Table 4.44 shows the ranges and average phosphate concentrations (mg/l PO_4^{-3}) for all depths of wells DGE, DGW, and the up-gradient well SNINEW for all sample dates between July 9, 1993 and June 29, 1995.

Table 4.44 Ranges and average phosphate concentrations for all depths and sample dates of wells DGE 1 - 4 , DGW 1- 4, and up-gradient well SNINEW from July 9, 1993 to June 8, 1995 at Site #2.

Well	PO_4^{-3} (mg/l) Min	Max.	Average
DGE 1-4	< 0.002	0.232	0.013
DGW 1-4	0.003	0.200	0.010
SNINEW	0.002	< 0.016	0.005

The average concentration of TP pumped from Dosing Chamber #2 to the mound system during this same time period was 2.71 mg/l. The average phosphate concentrations seen in wells DGE 1-4 and DGW 1-4 are much lower than the concentrations of TP in Dosing Chamber #2, indicating removal in the soil absorption system and in the volumes of soil between the mound system and the down-gradient wells. Thus, unlike the similarly aged soil absorption system at the pervious site, breakthrough of phosphate has not yet occurred at 10.5 or 13.5 m down-gradient of the soil absorption system at Site #2. However, there were no monitoring wells located 4.9 m (16 ft) down-gradient of the soil absorption system at Site #2 as there were at Site #1.

4.274 Volatile Organic Chemicals

Samples were collected from either well DGE V or DGE V2 on five occasions between October 1993, and June 1995. Samples collected were representative of water table levels within the screened intervals. No VOCs were detected above the limits of quantitation by EPA method 8021 on four of the five sample dates.

On May 19, 1995 7.3 µg/L of 1,1,1-trichloroethane and 1.7 µg/L of o-xylene/styrene were detected in the sample from well DGE V2. However, 3.4 µg/L of 1,1,1-trichloroethane, and 1.3 µg/L of naphthalene were detected in the 'trip blank' carried along with the samples. This indicates that the sample may have been contaminated during handling or transport.

No VOCs were detected in either well DGE V2 or the 'trip blank' on June 29, 1995 the final date on which samples were obtained. Excluding the data for the sample collected on May 19, 1995, it appears that either the mound system or the unsaturated soil beneath the mound system was effective at removing any VOCs that were not removed by the sand filter.

5. SUMMARY OF EFFLUENT TREATMENT BY RSF SYSTEMS

5.1 Study Site #1

Despite the problems with partial clogging of the gravity intake during part of the study, useful data was generated at this site.

The hydraulic loading rates (HLR) used at Site #1 ranged between 8 and 56 cm/day (2 and 14 gpd/ft^2). The sand filter ratios (SFR) used to achieve these loading rates ranged between 2.9 and 8.7Q. The septic tank ratios (STR) used ranged between 0.0 and 5.5Q. Table 5.1 shows the percent removal of BOD_5, COD, TP, and TN achieved by the various flow conditions for the RSF system at Site #1.

Table 5.1 Average, minimum, and maximum percent removal of BOD_5, COD, TP, and TN achieved by the various flow conditions at Site #1.

	Percent Removal		
	Average	Min.	Max.
BOD_5	94	81	99
COD	88	82	97
TP	35	5	74
TN	76	49	93

BOD_5 and COD removal was rather high, despite the variations in the amount of effluent applied to the top of the sand filter, averaging 94 and 88%, respectively. TP removal was highly variable (5 to 74%) reflecting changes in the household concentrations and the adsorptive capacity of the sand filter media. If phosphorous removal is desired, different sand filter media should be selected to achieve better removal than was found in this study.

Nitrogen is attenuated through nitrification in the sand filter followed by denitrification, in either the saturated rock storage unit of the RSF or in the septic tank. More nitrogen was removed in the rock storage unit of the RSF (48%) than in the septic tank (28%). Some variations in nitrogen attenuation were caused by temperature, carbon availability, and dissolved oxygen levels. Nitrogen removal was not significantly affected, despite conditions that were unfavorable for complete denitrification. TN removal by the RSF system averaged 76% (range 49 to 93%). Clogging of the gravity distribution system resulted in bypass of the system during some study periods; this created some uncertainty in the concentrations being applied to the soil absorption system. A maximum removal of 84% occurred when no effluent bypassed the sand filter. This removal was achieved with an average water usage (Q) of 339 lpd (105 gpd), an SFR of 8.1Q, an HLR of 50 cm/day (4.8 gpd/ft^2), an STR of 3.0Q, and an HRT of 2 days. Of the 44.1 grams/day of nitrogen generated in the household only 7.2 grams/day were applied to the drainfield.

The conversion of NH_4^+-N to nitrate-N was 90% complete, with an SFR of 3.5Q (HLR 25 cm/day). Loading rates higher than 3.5Q did not convert substantially more NH_4^+-N. While higher STRs resulted in conditions unfavorable for denitrification in the septic tank, the overall system performance was not affected since the sand filter removed a higher percentage of nitrogen than the septic tank.

Nitrate-N concentrations in the down-gradient monitoring wells adjacent to the soil absorption system were reduced by an average of 53%. If bypass of the system had not occurred, the reductions in the groundwater concentrations would have reflected the nitrogen attenuation by the RSF system. Due to dilution/mixing with groundwater, concentrations in down-gradient monitoring wells were approximately half of the concentrations being discharged. Nitrogen was not attenuated in the soil absorption system or groundwater. Nitrate-N concentrations at 38.1 meters (125 ft) down-gradient of the soil absorption system were further diluted than those seen adjacent to the soil absorption system. Before the installation of the RSF twelve ports exceeded the 10 mg/l nitrate-N standard; only two ports exceeded the nitrate-N standard after the installation of the RSF system. If bypass of the RSF system had not occurred, it would be expected that no shallow groundwater samples would have exceeded 10 mg/l at this distance down-gradient.

Phosphate breakthrough at 4.9 meters (16 ft) down-gradient of the soil absorption system occurred after approximately 12 years of usage with an average hydraulic loading of 378.5 lpd (100 gpd).

Toluene and sec-butyl benzene were the most frequently detected VOCs in the septic tank at Site #1. Concentrations were reduced by an average of 91% in the RSF, indicating physical and biological attenuation in the RSF. In general, VOCs were not completely removed at this site since the water usage and loading rates were relatively low. No VOCs were detected in the groundwater, indicating attenuation in the soil absorption system or in the unsaturated soil beneath the soil absorption system.

5.2 Study Site #2

This site used a pump in a sewage ejector pit to deliver septic tank effluent to the top of the sand filter. This design resulted in fewer problems than the design at Site #1. Water use at this site was greater than at Site #1, resulting in more dilute household effluent concentrations.

The hydraulic loading rates used at Site #2 ranged between 21 and 196 cm/day (5 and 48 gpd/ft^2). The SFRs used to achieve these loading rates ranged between 1.0 and 15.9Q. The STRs used ranged between 0.0 and 7.9Q. Table 5.2 shows the percent removals of BOD_5, COD, TP, and TN achieved by the various flow conditions for the RSF system at Site #2.

Table 5.2 Average, minimum, and maximum percent removal of BOD_5, COD, TP, and TN achieved by the various flow conditions at Site #2.

	Percent Removal		
	Average	Min.	Max.
BOD_5	95	79	99
COD	88	60	96
TP	42	18	70
TN	70	29	91

BOD_5 and COD removal was rather high, despite the variations in the amount of effluent applied to the top of the RSF; they averaged 95 and 88%, respectively. TP removal is again variable (18 to 70%), yet the average removal of 42% is higher than the removal seen at Site #1 due to the higher water usage and higher loading rates.

Nitrogen attenuation at Site #2 was achieved through the same processes as at Site #1. One significant difference at Site #2 was that temperature did not have an effect on nitrogen removal since levels were above the minimum level required for nitrification and denitrification. Some variation in nitrogen attenuation occurred due to carbon availability and dissolved oxygen levels. No problems of system bypass were encountered at Site #2. Again, more nitrogen was removed in the sand filter (48%) than in the septic tank (22%). The maximum removal of nitrogen was 91%. This removal was achieved with an average water usage of 804 lpd (212 gpd), an SFR of 15.9Q, an HLR of 196 cm/day (48 gpd/ft2), an STR of 7.9Q, and an HRT of 0.5 days. Of the 26.4 grams/day of nitrogen generated in the household, only 2.4 grams/day were applied to the mound system. These loading rates are relatively high and require a high amount of energy. SFRs around 8.5Q or an HLR of 100 cm/day (24 gpd/ft^2) would still achieve approximately 85% removal and are probably a more realistic goal for nitrogen removal by RSF systems since less energy input is required. It should be noted that higher loading rates were required to achieve results similar to those seen at Site #1 because the effluent was more dilute, reflecting the higher water usage at Site #2.

The conversion of NH_4^+-N to nitrate-N was 90% complete with as SFR of 6.0Q (HLR 100 cm/day). Loading rates greater than 6.0Q did not result in greater conversion and did not merit the additional energy input. Higher septic tank ratios may have effected the degree of denitrification in the septic tank due to aerobic conditions and decreased carbon availability. However, the overall system performance was not affected, as a higher percentage of nitrogen was removed in the sand filter. These problems seemed to arise at septic tank ratios exceeding 4.0Q, the ratio should be limited to eliminate similar problems.

Nitrate-N concentrations in the down-gradient monitoring wells were reduced an average of 73% by nitrogen attenuation in the RSF system. No attenuation of nitrogen occurred in the mound system or groundwater. While nitrate-N concentrations were generally near or below the 10 mg/l standard, it should be noted that the effluent at this site was relatively dilute due to the high water usage by the residents. Variations in water usage between different homes should be considered in efforts made to require nitrogen removal from on-site sewage disposal systems.

The number and concentrations of the VOC species detected were greatest in Dosing Chamber #1 and reduced in the RSF and Dosing Chamber #2. This indicates physical and biological attenuation of VOCs in the RSF. The most frequently detected compound in Dosing Chamber #1 was 1-4, dichlorobenzene. The RSF system removed 71 to 100% of the 1-4, dichlorobenzene prior to discharge to the mound system. Complete attenuation of non-chlorinated VOCs was achieved by the RSF system. VOC attenuation was more complete at Site #2 because the water usage and loading rates were higher. No VOCs were detected in the groundwater at Site #2, implying attenuation in the mound system or in unsaturated soil beneath the mound system.

5.3 Optimal Flow Conditions

Table 5.3 summarizes the flow conditions that achieved approximately 85% nitrogen removal by the RSF systems at Sites #1 and #2. Higher loading rates did achieve better nitrogen removal but, the author believes they do not merit the additional energy invested. It is difficult to make a direct comparison of the flow ratios at the two sites for two reasons: 1) the household water usage is higher and the initial application of effluent to the top instead of the bottom of the sand filter caused the HLR rate to be higher at Site #2, and 2) the HLR rate was lower at Site #1 since the water usage was less and the initial application of effluent was made to the bottom of the RSF.

Table 5.3 Comparison of study sites with flow conditions that will achieve approximately 85% removal of nitrogen.

	Site #1	**Site #2**
SFR	8.1Q	8.5Q
HLR cm/day (gpd/ft^2)	50 (4.8)	100 (24)
STR	3.0Q	3.5Q
HRT days	2	1

The SFR that will achieve approximately 85% removal of nitrogen is between 8.1 and 8.5Q. The HLR used to achieve this removal is between 50 and 100 cm/day (4.8 and 24 gpd/ft^2) and is variable due to differences in water usage and the method of initial application of effluent to the RSF.

The amount of effluent recycled to the septic tank should be limited to about 3.0Q. This will help to alleviate incomplete denitrification in the septic tank by maintaining anaerobic conditions and a sufficient carbon supply.

6. CONCLUSIONS and RECOMMENDATIONS

The following conclusions and design recommendations can be drawn from this research.

1) RSF systems can achieve up to 90 percent total nitrogen removal using relatively simple pump and flow regulation equipment.

2) Temperature did not appear to severely limit nitrification and denitrification despite the relatively cold temperatures observed. These systems could be utilized for nitrogen removal in Wisconsin.

3) The saturated rock storage unit of the RSF proved to be very effective at nitrogen removal; in fact it removed a higher percentage of nitrogen than did the recirculation to the septic tank. Systems aimed at nitrogen removal should include a sand filter that has a saturated rock storage unit.

4) The limestone storage unit appears to be able to maintain a pH near 7.0 within the wastewater, thus canceling the effect of nitrification on poorly buffered wastewater.

5) The method of applying septic tank effluent to the top of the RSF proved to be more reliable than the gravity distribution method to the bottom of the sand filter. Application of effluent should be made to the top of the RSF from a separate tank which incorporates a screened pump vault.

6) Applying effluent to the top of the RSF resulted in the development of a biological mat after approximately one year of operation. The mat was raked and did not redevelop over the next year and a half of monitoring. The laterals of the distribution systems required cleaning approximately once each year. This demonstrates that some type of routine maintenance would be required with these type of systems.

7) The pump chamber of the RSF should be screened to keep any sand or other solids from damaging the pump or clogging the orifices of either the flow control 'valves' or in distribution system laterals.

8) Sand filter ratios (SFR) between three and six times the daily water usage to the top of the sand filter resulted in 90% conversion of NH_4^+-N to nitrate-N.

9) An SFR approximately eight times the daily water usage achieved 85% total nitrogen removal. The author believes this to be the most practical removal for RSF systems based on energy costs of $45.00 per year at $0.07/KWhr.

10) The amount of effluent recycled to the septic tank from the RSF should be limited to three times the daily water usage to ensure compete denitrification.

11) The utilization of the septic tank as a carbon source was not always effective at promoting complete denitrification, especially when the loading rates to the septic tank from the sand filter were relatively high, resulting in dilution of the BOD_5 in the septic tank. Denitrification systems may require separate tanks to serve as a denitrification site. This would increase the monetary and space requirements of these systems.

12) RSF systems should be monitored once every six months to ensure that they are maintained to function properly.

13) A float style warning mechanism must be incorporated into RSF systems to alert homeowners of system failures.

14) These systems remove significant amounts of nitrogen and reduce groundwater nitrate-N impacts. Groundwater concentrations were reduced by an average of 73%. The reduction would have been greater if the systems had been operated at their optimal performance.

15) Phosphate breakthrough at 4.9 meters down-gradient of the conventional soil absorption system occurred after approximately 12 years of usage with an average hydraulic loading of 100 gallons/day.

16) Toluene, sec-butylbenzene, and 1-4-dichlorobenzene were the most frequently detected VOCs in the septic tanks. The RSF system is successful at reducing the VOCs being applied to the soil absorption systems. The unsaturated soil beneath the soil absorption systems removed any VOCs which were not removed by the RSF system.

17) RSF systems produce a high quality effluent with significantly reduced amounts of; solids, fecal coliforms, organic matter, and nitrogen (when incorporating denitrification). These abilities of a denitrifying RSF can help to lengthen the life-span of soil absorption systems and protect groundwater quality.

7. LITERATURE CITED

Ball, H. L. (1994). "Nitrogen reduction in an on-site trickling filter/upflow filter wastewater treatment system". Proceedings of Seventh-Nat. Symp. On Individual and Small Community Sewage Systems, ASAE Pub. 18-94. pp. 283-291.

Bremner, G. A., and K. Shaw. (1958). "Denitrification in soil. II Factors affecting denitrification". J. Agric. Sci. 51:40-52.

Cagle, W. A., and L. A. Johnson. (1994). "Onsite intermittent sand filter systems, a regulatory/scientific approach to their study in Placer County, California". Proceedings of Seventh-Nat. Symp. On Individual and Small Community Sewage Systems, ASAE Pub. 18-94. pp. 283-291.

Converse, J. C. (1992). "Principles of pretreatment systems for on-site waste disposal." University of Wisconsin, Madison, WI.

Canter, L. W., and R. C. Knox. (1986). Septic Tank System Effects on Ground Water Quality, Lewis Publishers, Inc., Chelsea, Michigan.

Crites, R. W., E. L. Meyer, and R. G. Smith. (1981). Process Design Manual for Land Treatment of Municipal Wastewater. EPA 625/1/81/013.

Davis, M. L., and D. A. Cornwall. (1985). Introduction to Environmental Engineering. McGraw-Hill Book Co., New York, N.Y.

Gaudy, A. F., and E. T. Gaudy. (1980). Microbiology for Environmental Scientists and Engineers. McGraw-Hill Book Co., New York, N.Y.

Grady, C. P. L., and H. C. Lim. (1980). Biological Wastewater Treatment. Marcel Dekker, Inc., New York, N.Y.

Hathaway, E. W. (1980). "Sources of toxic compound in household wastewater". EPA-600/2-80-128.

Kaplan, B. O. (1987). Septic Systems Handbook. Lewis Publishers Inc., Chelsea, Michigan.

Kristiansen, R. (1981a). "Sand-filter trenches for purification of septic tank effluent: I. The clogging mechanism and soil physical environment". J. Environ. Qual. 10:353-357.

Kristiansen, R. (1981b). "Sand-filter trenches for purification of septic tank effluent: II. The fate of nitrogen". J. Environ. Qual. 10:358-360.

Laak, R. (1981). "A passive denitrification system for on-site systems". Proceedings of Third National Symposium on Individual and Small Community Sewage Treatment. ASAE Pub. 1-82. pp. 108-115.

Laak, R., M. A. Parese, and R. Costello. (1981). "Denitrification of Blackwater with Greywater". J. Environ. Eng. Div., ASCE. 107:581-590.

Lamb, B. E., A. J. Gold, G. W. Loomis, and C. G. McKiel. (1990). "Nitrogen Removal for On-Site Sewage Disposal: A Recirculating Sand Filter/Rock Tank Design". Transactions of the ASAE, 33(2), pp. 525-531.

Lance, J. C. (1972). "Nitrogen Removal by Soil Mechanisms". J. Water Poll. Control Fed., 44:1352-1351.

Lawrence, C. H. (1973). "Septic tank Performance". Journal of Environmental Health, Vol. 36, No. 3, pp. 226-228.

Loudon, T. L., D. B. Thompson, L. Fay, and L. E. Reese. (1985). "Cold climate performance of recirculating sand filters". Proceedings of the Fourth National Symposium on Individual and Small Community Sewage Systems. ASAE Pub. 07-85. pp. 333-341.

Loudon, T. L., S. R. Wert, A. J. Gold, and T. A. McCarl. (1989). "Recirculating sand filters for cold regions". International Summer Meeting jointly sponsored by the American Society of Agricultural Engineers and the Canadian Society of Agricultural Engineering, Quebec, PQ, Canada. June 25-28, 1989. ASAE Paper No. 89-2173.

Reneau, R. B., C. Hagedorn, and M. J. Degen. (1989) "Fate and transport of biological and inorganic contaminants from on-site disposal of domestic waste water. J. Environ Quality. 18:135-144.

Rittman, B. F., and W. E. Langeland. (1985). "Simultaneous denitrification with nitrification in single-channel oxidation ditches". J. Water Poll. Control Fed. 57:300-308.

Romero, J. C. (1970). "The movement of bacteria and viruses through porous media". Groundwater. 8:37-48.

Ronayne, M. P., R. C. Paeth, and S. A. Wilson. (1982). Final Report: Oregon On-Site Experimental Systems Program. State of Oregon Department of Environmental Quality.

Sandy, A. T. (1987). "Nitrogen removal using a batch recirculating bottom ash filter", presented at Fifth Nat. Symp. on Individual and Small Community Sewage Systems, ASAE, Chicago, Illinois.

Sauer, D. K., and W. C. Boyle. (1977). "Intermittent sand filtration and disinfection of small wastewater flows". Proc. Second Nat. Home Sewage Treatment Symp., ASAE Pub. 05-77, pp. 164-174.

Sauer, P. A., and E. J. Tyler. (1991). "Volatile organic chemical (VOC) attenuation in unsaturated soil above and below an onsite wastewater infiltration system. Sixth-Nat. Symp. On Individual and Small Community Sewage Systems, ASAE Pub. 10-91.

Shammas, N. K. (1986). "Interactions of temperature, pH, and biomass on the nitrification process", J. Water Pollution Control Fed., 58(1), pp. 52-59.

Shaw , B., and N. Turyk. (1992). "A comparative study of nitrate-N loading to groundwater from mound, in ground pressure and at grade systems". Final Report: to WI DNR., 89pp.

Shaw, B., P. Arntsen, and W. VanRyswyk. (1993). "Subdivision impacts on groundwater quality", Final Report to WI DNR , 130 pp.

Shaw, B., and S. Osesek, (Unpublished M.S. Thesis). "Design and evaluation of two recirculating sand filters for nitrogen removal".

Sikora, L. J., and D. R. Keeney. (1974). "Laboratory studies on simulation of biological denitrification". Proceedings of the National Home Sewage Disposal Symposium. ASAE Pub. PROC-175. pp. 64-73.

Swanson, S. W., S. P., and Dix. (1986). "Performance of a recirculating sand filter utilizing bottom ash media". Presented at the Ninth Annual Technical Conference, WPCA of PA, Monroeville, PA.

U. S. Environmental Protection Agency. (1980). Design Manual--Onsite Wastewater Treatment and Disposal Systems. EPA 625/1-80-012, Cincinnati, Ohio.

U. S. Environmental Protection Agency. (1978). Management of Small Wastes Flows. EPA-600/2-78-173, Cincinnati, Ohio.

U. S. Environmental Protection Agency. (1975). Process Design Manual for Nitrogen Control. Office of Technology Transfer of the U.S. EPA

VanRyswyk, W. (1996). "Evaluation of conventional septic systems and their impact on groundwater quality in the central Wisconsin sand plain". MS Thesis. 117pp.

Viraraghavan, T. (1976). "Septic Tank Efficiency". ASCS Journal Environmental Engineering Division, Vol. 102, No. EE2, pp. 505-508.

Walker, W. G., J. Bouma, D. R. Keeney, and F. R. Magdoff. (1973). "Nitrogen Transformations During Subsurface Disposal of Septic Tank Effluents in Sands: I. Soil Transformations". J. Environ. Quality, 2(4), pp. 475-480.

Walker, W. G., J. Bouma, D. R. Keeney, and P. G Olcott. (1973). "Nitrogen Transformations During Subsurface Disposal of Septic Tank Effluents in Sands: II. Groundwater Quality". J. Environ. Quality, 2(4), pp. 521-525.

Warnock, R. G., and N. Biswas. (1981). "Study of columnar denitrification for application in an on-site system". Proceedings of the Third National Symposium on Individual and Small Community Sewage Treatment. ASAE Pub. 1-82.