RESEARCH REPORT

Innovative On-Site Wastewater Treatment

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INNOVATIVE ON-SITE WASTEWATER TREATMENT

by

J.D. Mooers and D.H.Waller

April 1998

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PURPOSE

This research was undertaken to demonstrate rational design and sound construction techniques for septic tank absorption field systems and for the demonstration of wastewater nutrient removal technologies. The control of flow to the treatment system was demonstrated using water conservation. Non-mechanical treatment of household wastewater was was demonstrated using a Waterloo biofilter, a denitrification filter and recirculation of effluent. Sampling at each point in the treatment process, and in the groundwater below the disposal bed,allowed for the characterization of each of the treatment components.

ACKNOWLEDGEMENTS

Preparation of this report depended on the cooperation of the Nova Scotia Canada Mortgage and Housing Corporation office, in particular Brian Hudson and John Healey provided invaluable assistance in site location. The cooperation of the Nova Scotia Department of the Environment is gratefully acknowledged. The system design was carefully prepared by Environmental Design Management Ltd. and Dr. Craig Jowett and installed with care by Steve Barry. Special thanks goes to the homeowners who were patient and cooperative with our system monitoring and maintenance efforts and to Steve Sauver for his diligent field work in all aspects of this project from site assessment through monitoring.

EXECUTIVE SUMMARY

Problems with failing residential septic systems are common across the country. System failures are evidenced by surface break-outs, back-ups into homes, and contamination of groundwater supplies. The underlying causes are inadequate site assessment, especially in marginal soils, outdated design practices, poor construction, excessive water usage and lack of maintenance.

This paper documents the results of a project that attempts to address these concerns with the installation of an innovative on-site disposal system in rural Nova Scotia involving a design based on a permeameter assessment, water conservation, septic tank insulation and filtering, a Waterloo Biofilter, re-circulation of effluent, a de-nitrification filter, and ultimate disposal in a Nova Scotia contour trench.

Rational design and sound construction techniques for a septic tank absorption field system are documented The disposal bed design procedure is elaborated in Appendix B. The Nova Scotia permeameter, an inexpensive soil conductivity assessment tool, was used to assess in-situ permeability as part of the field design. A schematic of the device, the necessary equations, and the operating instructions are documented in Appendix A. Water conserving fixtures at this site reduced flows to the wastewater disposal system by 30% over average household usage values. Water conservation should be treated as an additional level of insurance for the investment made in the disposal system and not as a reason to reduce the infiltration field. The use of a septic tank filter, provided some chemical renovation of the effluent, but more importantly prevented large solids from leaving the septic tank.

After passing through the Waterloo biofilter the effluent showed concentration reductions of 95% for BOD, 58% for phosphorous, 19% for suspended solids, and 94% for fecal coliforms. Total nitrogen reduction in the effluent was 38% in the pump chamber, 44% after the biofilter, and 60% after the denitrification filter. The installation of a recirculating system after the biofilter increased the total nitrogen removal in the pump chamber and after the biofilter and denitrification filter to 69%, 72%, and 69% respectively. Minimal impact on the down gradient aquifer was measured.

Systems that treat the septic tank effluent before disposal to the subsurface remove some of the burden of treatment from the soil. These systems are important for sites where the hydraulic or treatment capabilities of the soil are compromised. However, these systems are more complex than conventional gravity disposal systems and require a higher level of operation and maintenance, particularly if they are to be used with a higher design loading rate. Technical design aspects of the Waterloo Biofilter that resulted in the high levels of maintenance during this demonstration have been addressed by the designer. The denitrification filter design used is not likely to provide a commercially viable product without further design work to reduce the levels of maintenance required for regular operation.

Résumé

Il arrive couramment que les installations d'assainissement individuelles connaissent des défaillances qui se caractérisent par des éruptions en surface, des refoulements dans les maisons et la contamination de la nappe d'eau souterraine. Les causes profondes de ces problèmes sont liées à une évaluation inadéquate de l'emplacement, particulièrement dans les sols marginaux, ainsi qu'à des pratiques de conception désuètes, à une mauvaise construction, à une consommation d'eau excessive et à un manque d'entretien.

On présente ici les résultats des travaux de recherche qui ont été réalisés pour pallier ces défaillances par la mise en œuvre d'une installation novatrice d'assainissement sur place dans une zone rurale de la Nouvelle-Écosse. L'installation comportait les éléments suivants : une évaluation au perméamètre, une réduction de la quantité d'eau dans l'effluent, l'isolation de la fosse septique, un filtre à la sortie de la fosse, un biofiltre Waterloo, un système de recirculation de l'effluent, un filtre de dénitrification, ainsi que l'évacuation de l'effluent vers une tranchée de niveau, technique mise au point en Nouvelle-Écosse.

Dans le rapport dont il est question ici, des techniques rationnelles de conception et de construction des champs d'épandage sont décrites. L'annexe B présente un processus détaillé en matière de conception de champs d'épandage. Un perméamètre, outil peu coûteux mis au point en Nouvelle-Écosse, a permis de mesurer la perméabilité du sol. En outre, l'annexe A présente un schéma du dispositif, les équations nécessaires aux calculs ainsi que les consignes d'utilisation. Grâce à l'installation d'appareils à faible débit, l'effluent vers la fosse septique a diminué de 30 % comparativement au débit moyen courant des ménages. Le fait de diminuer la quantité d'eau acheminée vers la fosse septique doit être considéré comme une précaution additionnelle, vu les coûts d'immobilisation de l'installation d'assainissement, et non pas comme une mesure permettant de réduire les dimensions du champ d'épandage. Le filtre pour l'effluent de la fosse septique a certes amélioré la composition chimique de l'effluent, mais ce qui importe plus encore, c'est qu'il a empêché le passage de grosses particules solides.

À la sortie du biofiltre Waterloo, l'effluent affichait une diminution de 95 % de la teneur en DBO, de 58 % en phosphore, de 19 % en solides en suspension et de 94 % en coliformes fécaux. La réduction de l'azote totale a été de 38 % à la sortie de la chambre de la pompe, de 44 % à la sortie du biofiltre et de 60 % à la sortie du filtre de dénitrification. L'installation d'un système de recirculation en aval du biofiltre a augmenté de 69 % le taux de rétention de l'azote totale dans la chambre de la pompe et de 72 % et 69 % respectivement, en aval du biofiltre et du filtre de dénitrification. Les répercussions sur la nappe d'eau avoisinante ont été faibles.

Les installations qui traitent l'effluent des fosses septiques avant qu'il ne soit acheminé vers le champ d'épandage facilitent le traitement naturel dans le sol. Ces installations sont donc d'une importance capitale, surtout dans le cas des emplacements où en raison de la composition du sol la qualité hydraulique et la capacité de traitement sont limitées. Ces installations sont toutefois plus complexes que les installations d'assainissement

gravitaires courantes et requièrent donc un niveau plus élevé de surveillance et d'entretien, surtout si elles doivent fonctionner à des niveaux supérieurs de charge. Les caractéristiques techniques du biofiltre Waterloo qui ont entraîné des frais élevés d'entretien ont depuis été modifiées par le concepteur. Il est peu probable que le modèle de filtre de dénitrification s'avère un produit viable dans le commerce sans lui apporter des améliorations permettant de réduire son niveau d'entretien courant.

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Introduction

In Nova Scotia approximately 40% of the population dispose of household wastewater into septic tank and infiltration field systems. The 1990 U.S. Census shows approximately 25% of homes are on on-site systems. Where the population density is sparse these systems pose minimal environmental impact. In an era of decreasing funding for large infrastructure projects, we are increasingly seeing densely developed suburban areas and small communities relying on on-site wastewater disposal. Of particular concern are areas sensitive to increased nutrient loadings. Where systems are properly designed, installed, and operated, some of the nutrient effects may be mitigated in the subsurface, however, where systems fail they can directly impact receiving waters.

Some of the underlying causes of system failure are inadequate site assessment, especially in marginal soils, outdated design practices, poor construction techniques, and excessive water usage in modern homes.

In an effort to overcome and better understand these common sources of system failure, and to minimize the environmental impact of wastewater disposal, a novel on-site wastewater treatment system was constructed at a single family dwelling, based on rational design, sound construction techniques, and innovative technologies. The suitability of the soil was assessed using a combination of test pits and in-situ assessment of hydraulic conductivity. The home was fitted with water conserving fixtures to substantially reduce domestic water flows and to allow for an appropriate match between the wastewater flow and the host soil long-term infiltration rate. In combination with the reduced hydraulic loading on the disposal bed a number of technologies were used to reduce the treatment burden on the soil below the bed. These included insulating the septic tank and pump chamber to improve biological breakdown of the effluent, a septic tank filter to reduce the movement of solids to the disposal field, an aerated biofilter to provide aerobic breakdown, a denitrification filter to enhance nitrogen removal, and the use of a Nova Scotia contour trench disposal field. The system was constructed to allow for the monitoring of the household water usage and for the sampling of the effluent at each step in the flow path. Four piezometers, one in the disposal field, and three installed down gradient of the field allowed for the monitoring of shallow groundwater impacts.

Materials and Methods

Site Description

The site chosen for this project was selected with the assistance of the Nova Scotia office of Canada Mortgage and Housing Corporation. The lot has 38 m frontage and is 76 m deep, located in Lower Cornwall, Lunenburg County, Nova Scotia. The site slopes to the back at approximately 10% and is well treed along the back of the lot. The water supply is a drilled well in the front yard between the house and the road (see Figure 1).

Soil Investigation

Two 1.5 m deep inspection pits were dug with a back-hoe along the proposed disposal bed location and inspected by an on-site design engineer, a Department of the Environment engineer, and the local environmental health inspector.

The upper soil horizon, just below the organic layer, is a medium orange brown poorly sorted till with a sandy silt matrix. It has 50% sub-angular to sub-rounded clasts from fine gravel to approximately 5cm diameter. The clast origins are varied including a range from siltstone to granite.

The lower soil horizon is a medium grey till. The matrix is similar to the upper horizon in texture. The clasts are slightly less abundant and smaller (only up to 1cm). The clasts are predominately siltstone.

The hydraulic conductivity of the soil adjacent to both test pits was determined using a single level ponded height permeameter. Using a hand auger a

0.6 m hole was drilled at each location and tested with the permeameter. This is an easy-to- build and use device based on the assessment methodology developed at Guelph University (Reynolds, 1992). The two permeameter tests gave values of 5.7×10^{-6} and 1.4×10^{-5} m/sec. Test results, construction designs, and use instructions are shown in Appendix A.

Measuring the soil depth, from the bottom of the organic layer to the level in the test pits that showed some seepage of groundwater, gave a total useable depth of approximately 1.2m. Based on the soil conductivity and the total useable soil depth a partially trenched contour design (C2) was selected for installation. A detailed description of the design process is shown in Appendix B.

Water Conservation

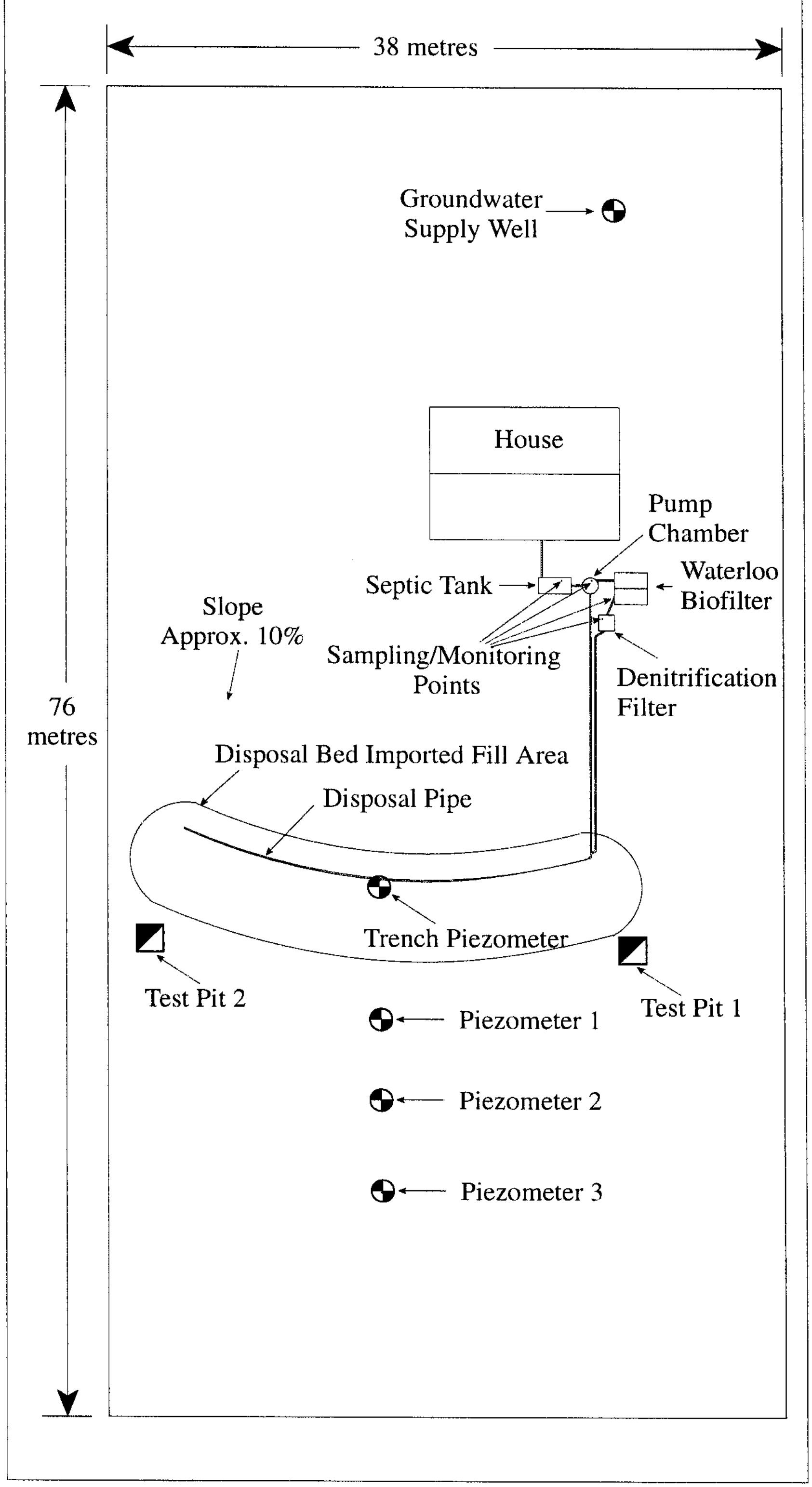
The house has one bathroom. It is equipped with a 6 litre per flush low flow toilet and a low flow shower head. All sinks are equipped with low flow aerators. Water meters on the main inflow line and the exterior tap line allow for the monitoring of all water use and permit calculation of the system loading rate.

Tank, Pump Chamber

The wastewater disposal system is installed as shown in Figure 1. The system consists of an 3600 L single chamber septic tank fitted with inlet and outlet tee's. The outlet tee contains a removable effluent filter installed as shown in Figure 2. The septic tank was insulated on the top and sides with sheets of rigid foam insulation (Figure 3). A 2.5cm perforated PVC pipe was inserted into the tank through a small hole drilled on the top to allow for regular sampling and temperature monitoring of the effluent.

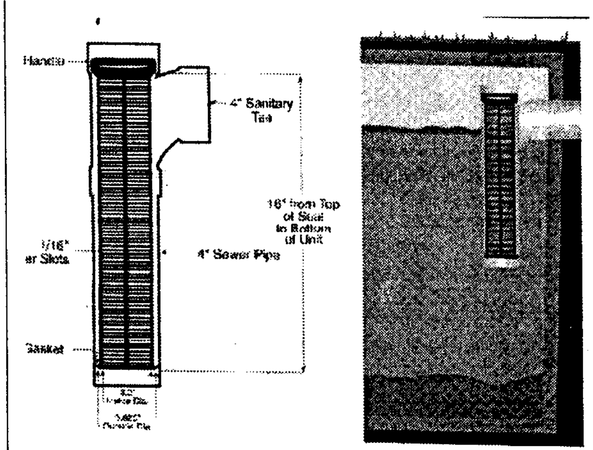
A 900 L pump chamber is installed immediately downstream from the septic tank (Figure 3). An effluent pump, controlled by float switches and a timer, passes all effluent from the pump chamber into the Waterloo biofilter. A timer activates the pump and doses the biofilter for 30 seconds every

Figure 1:
Plan View of Lot and Disposal System



15 minutes. A low level float switch prevents the pump from running while the chamber is dry and a high level float activates the pump to prevent effluent from flowing out of the pump chamber high level outfall. The high water level outfall from the pump chamber is connected directly to the disposal bed. It is planned that the pump be removed at the end of the project at which time the high level outfall will serve to operate as a standard gravity feed to the disposal bed. The top surface of the pump chamber is covered with a layer of rigid foam insulation. A 2.5 cm perforated PVC pipe was inserted into the chamber through a small hole drilled on the top to allow for regular sampling and temperature monitoring of the effluent.

Figure 2:
Septic Tank Filter (Zabel, 1996)



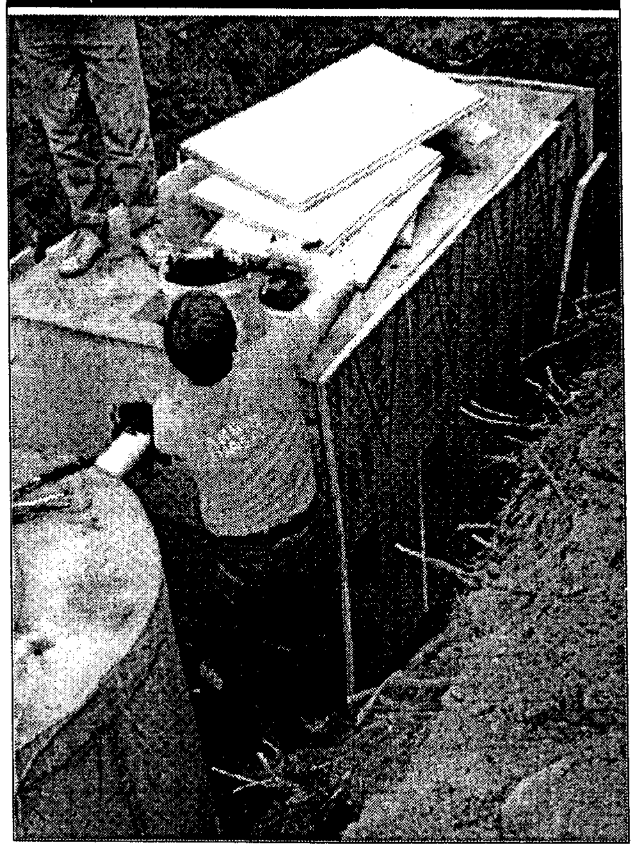
Biofilter

The Waterloo biofilter is structurally an insulated plywood box 1.8m x 1.8m and 1.2m high capped by a hinged roof as shown in Figure 4. The box is insulated with a 8cm layer of extruded foam on all interior surfaces. The filter media is a 1m deep bed of 5cm square polyurethane foam blocks. The air circulation system in the biofilter consists of perforated PVC pipes near the bottom of the foam bed connected to a blower above the filter bed. This provides air circulation as air is drawn down through the filter media and back up through the pipes and blower. Effluent is sprayed evenly across the top of the filter through a system of nine upward facing large-orifice helical spray nozzles. A layer of webbing prevents the nozzles from sinking into the foam media. A collection pipe in the floor of the biofilter allows treated effluent to gravity flow out of the filter. A "U" trap in the pipe leaving the filter allows for sample collection and temperature monitoring. Two vents near the peak of the roof provide fresh air entry points.

Denitrification Filter

Effluent flows by gravity from the biofilter to a buried denitrification filter. The denitrification filter is a 0.9m x 0.9m bottomless plywood box pushed into a concrete base as shown in Figure 5. A collection pipe in the base passes effluent by gravity to the disposal bed. The denitrification media is a 0.3m thick layer of hardwood pellets supported on a plastic screen mesh. The sawdust layer is topped with a 5cm thick sheet of polyurethane foam meant to limit oxygen migration into the wood pellet layer. Effluent enters the denitrification filter at the top of the box, runs off a splash plate, into the foam, and

Figure 3:
Septic Tank Insulation



then through the filter media. A 2.5cm perforated PVC pipe was inserted in the box through a small hole drilled in the top to allow for regular sampling.

A "U" extension was added to the outflow pipe from the filter in July of 1996, as shown in Figure 5. This ensured that the wood pellet layer was submerged and operating in an anaerobic manner.

Contour Field

Effluent flows by gravity into one end of the disposal pipe. The disposal field is a C2 (raised and imported fill covered) contour trench (Nova Scotia Guidelines, 1988). A cross section of the design is shown in Figure 6. Field dimensions were determined using the results of soil analysis and the design procedure shown in Appendix B. The disposal trench is 24m long, 1.2m wide and 0.2m deep at the toe. Seven centimeters of clean sand were laid in the bottom of the trench. As effluent is dosed to the bottom of the trench an organic slime layer, known as the biomat, will eventually develop at the crushed rock and sand interface, and become a hydraulic barrier that sets the long term acceptance rate (LTAR) of the effluent. As the LTAR of the biomat is dependent on the host soil, using a consistent layer of sand with an estimated LTAR, allows for the use of the LTAR as a design variable. In Nova Scotia this results in a design loading rate of $32L/m^2/day$. Fifteen centimeters of clean gravel was placed on top of the sand layer, then the disposal pipe, and then 7 more centimeters of clean gravel. The trench was covered with a geotextile barrier, to prevent fill migration into the gravel pore spaces, on top of which was placed the imported fill buffer to a depth of 30cm, 1.5m upslope of the bed, across the bed, and extending 4.6m below the toe of the bed. A 2.5cm open bottomed observation well was installed to the bottom of the contour trench to allow for the monitoring of ponding levels within the disposal bed and the withdrawal of samples should effluent be present.

Monitoring Wells

Three piezometers were installed down gradient of the disposal bed (see Figure 1). The piezometers were constructed from 5cm schedule 40 tubing with 1.5m geotextile wrapped screens on the bottom. Once the wells were in place the boreholes were backfilled with sand and sealed off with bentonite. Piezometer 1 was installed to a depth of 9.1m. The upper soil horizon, as described in section 2.12 extended down to 2.4m. The lower horizon extended to the bottom of the well. Piezometer 2 and 3 were finished to depths of 8.2m and 9.1m respectively.

Monitoring Schedule

Samples were collected weekly for the first 5 weeks of system operation until it became apparent that the pump had failed due to improper installation. After replacing the pump and the control panel sampling resumed on a monthly

Figure 4:
Waterloo Biofilter Configuration

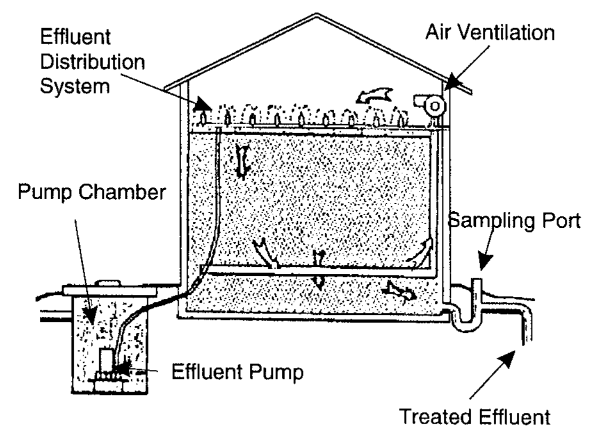


Figure 5:
Denitrification Filter Schematic

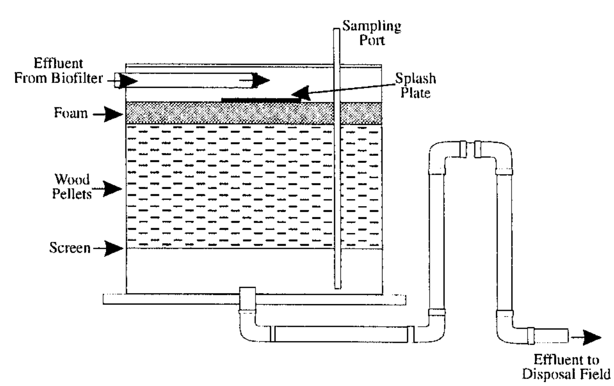
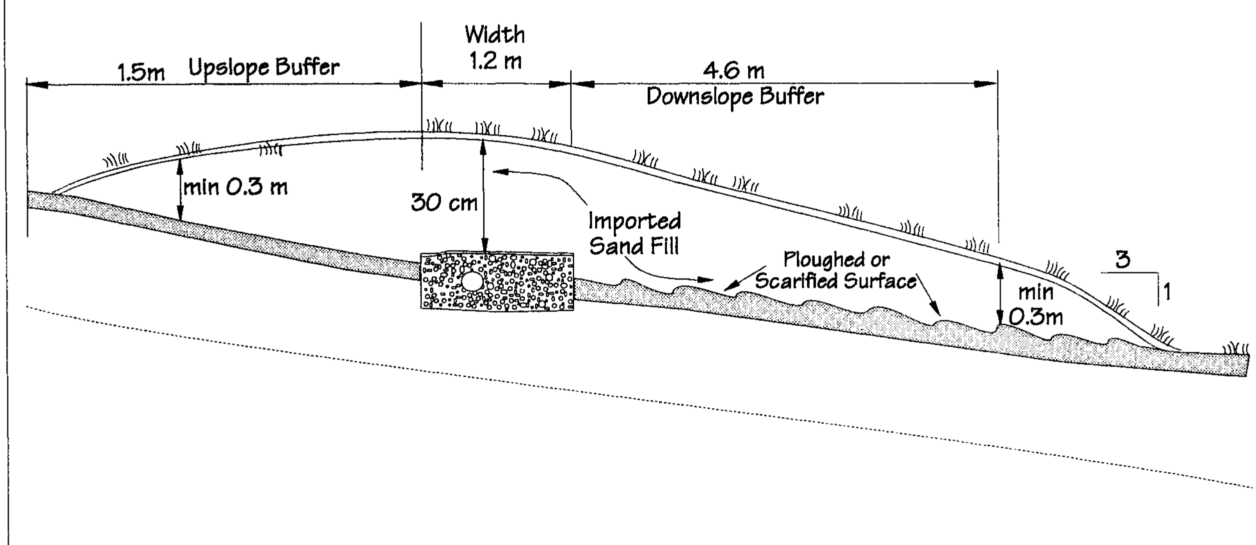


Figure 6:
C2 Contour Disposal Field Cross-Section



basis. Samples were collected using a hand-held vacuum pump. They were taken directly to the Environmental Chemistry Laboratory at the Victoria General Hospital and analyzed for ammonia, nitrate+nitrite, Kjeldhal nitrogen, suspended solids, biological oxygen demand, total phosphorus, pH, and total and fecal coliforms. Monthly samples continued until the October 1995 after which sampling and testing was conducted bi-monthly to mid-October 1996.

Recirculation

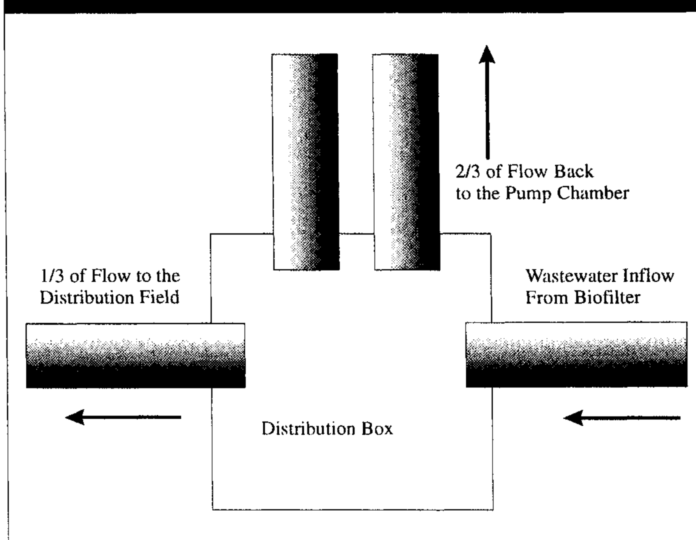
In an attempt to remove as much nitrogen as possible from the wastewater stream a recirculation system was installed. Effluent in which the ammonia has been converted to nitrates, such as that leaving the biofilter, can be further cleansed of nitrogen through the process of denitrification (conversion of nitrates to N_2 gas). Conditions that promote denitrification include a low oxygen environment in which a carbon source is present. Such conditions exist in the septic tank. One strategy for reducing nitrogen is thus to recirculate nitrified effluent back into the septic tank.

In mid-November 1996 a distribution box was installed between the biofilter and the denitrification filter, as shown in Figure 7 . This sent two thirds of the effluent back to the pump chamber and allowed one third to proceed on through the denitrification filter to the disposal field. Five sample sets were collected from January through November 1997.

Results/Discussion

The results of the chemical sampling episodes conducted between February 23rd 1995 to October 15th 1996, as well as the 5 post-recirculation samples collected from January 15th to the November 27th 1997, are shown in Appendix C. The average parameter values and percent

Figure 7:
Plan-View Schematic of Recirculation System



removals for the two periods are summarized in Tables 1 and 2. Samples were taken from within the septic tank and the pump chamber and after flow through the biofilter and denitrification filter. The piezometer in the disposal bed has remained consistently dry, therefore no analysis was possible. For the purposes of the statistical comparisons, values that were at the detection limit, for example <0.1 mg/L, are shown as 0.1 mg/L in the tables in Appendix C. Removal efficiencies have been calculated, using the average values, as percent reduction from the septic tank effluent values.

Table 1:
Average Parameter Values and % Removal Efficiencies

Parameter (mg/L) % Removal	Initial Monitoring Septic Tank	Pump Chamber	Biofilter	Denit. Filter	P1	P2	P3
BOD	1260	558	58	38	15.2	3.7	4.8
		56%	95%	97%	99%	100%	100%
Total Phosphorus	50	20	21	19	0.1	0.1	0.1
		61%	58%	62%	100%	100%	100%
Suspended Solids	142	135	115	29	2.9	3.7	2.8
		5%	19%	79%	98%	97%	98%
Ammonia	175	117	61	58	0.1	0.2	0.1
		33%	65%	67%	100%	100%	100%
TKN	220	138	82	64	0.1	0.3	0.2
		38%	63%	71%	100%	100%	100%
Total Coliforms (counts/100ml)	3.1E+07	3.5E+07	3.8E+06	1.7E+06	392	57	281
		-13%	88%	95%	100%	100%	100%
Fecal Coliforms (counts/100ml)	2.8E+07	1.2E+07	1.6E+06	4.4E+05	7	4	22
		56%	94%	98%	100%	100%	100%

Table 2:
Recirculation Parameter Values and % Removal Efficiencies

Parameter (mg/L) % Removal	Post- Recirculation Septic Tank	Pump Chamber	Biofilter	Denit. Filter	P1	P2	P3
BOD	464	207	29	14	9.0	6.8	13.3
		55%	94%	97%	98%	99%	97%
Total Phosphorus	13	12	16	13	0.1	0.1	0.1
		8%	-18%	-2%	99%	99%	99%
Suspended Solids	113	39	38	16	2.4	2.2	2.9
		65%	66%	86%	98%	98%	97%
Ammonia	81	44	19	30	0.1	0.4	0.1
		46%	77%	63%	100%	100%	100%
TKN	101	56	33	34	0.4	0.5	0.3
		45%	67%	66%	100%	99%	100%
Total Coliforms (counts/100ml)	1.1E+07	6.6E+06	1.3E+06	7.9E+04	0	10,000	144
		40%	88%	99%	100%	100%	100%
Fecal Coliforms (counts/100ml)	5.4E+06	2.6E+06	2.5E+05	1.8E+04	0	6,000	0
		52%	95%	100%	100%	100%	100%

Chemical Monitoring

BOD

The average STE (septic tank effluent) BOD value of 1260 mg/L, as shown in Table 1, is high relative to typical residential effluent BOD values which are in the range of 200 to 300 mg/L (USEPA, 1980). The reason for these high values is unknown. However, as can be seen in Appendix C the STE BOD results have gradually decreased and approached the normal range over time. The removal efficiency increases from 56% in the pump chamber to 95% after the biofilter and 97% in the outflow from the denitrification filter. With the exception of the Oct. 5th 1995 sample event, BOD values in the groundwater piezometers have remained below 10 mg/L and have been predominately below the detection limit.

The average post recirculation STE BOD value is 464 mg/L. The pump chamber shows a 55% reduction in BOD levels. The final 3 samples taken from the pump chamber (Appendix C) are all below 50 mg/L, indicating significant BOD reductions that may be attributable to the recirculation. Percent removal efficiencies increase to 94% after the biofilter and 97% after the denitrification filter.

Nitrogen

Nitrogen in the septic tank is predominately in the form of ammonia. Ammonia and nitrate+nitrite are related in the wastewater disposal treatment by the process of nitrification. Under aerobic conditions, such as in the biofilter, ammonia is converted to nitrite and then nitrate by the action of nitrifying bacteria. If the nitrified effluent is subsequently passed through an anaerobic environment which contains a source of carbon, it can be converted to nitrogen gas and thus removed from the waste stream.

As can be seen in Appendix C, STE ammonia and kjeldhal nitrogen (TKN) values are above typical domestic levels; total nitrogen levels are commonly in the range of 35 to 100 mg/L (USEPA, 1980). As with the BOD, there is no apparent reason for the high levels. The ammonia values show a steadily decreasing trend, during the initial monitoring period, to levels typical of domestic effluent . The average STE ammonia value is 175 mg/L. This is reduced by 33% in the pump chamber and by 65% after the biofilter. Ammonia values are still in the 50 to 60 mg/L range after the biofilter indicating partial nitrification. Nitrification results from biofilters at other locations have been significantly better (Jowett, 1995). It is suspected that the high levels of BOD in this effluent have hindered the nitrification process. As expected ammonia that passes the biofilter is relatively unchanged by the denitrification filter. The groundwater piezometers begin to show ammonia levels slightly above the detection limit after November 9th 1995. In particular P2 has showed consistent low levels with an average of 0.4 mg/L.

Nitrifying and denitrifying bacteria are difficult to establish in cold temperatures. The initial February and March sampling sessions show high ammonia values persisting in the effluent even past the denitrification filter. As the weather warms, ammonia values can be seen to decrease as conversion to nitrite and nitrate in the biofilter occurs (Appendix C).

After the establishment of recirculation, ammonia values are reduced considerably in the pump chamber and after the biofilter. The final 3 measurements in the pump chamber are all below 30 mg/L and 3 of the final 5 measurements after the biofilter are below 10 mg/L. The average ammonia value after the denitrification filter is 30 mg/L, an increase from levels after the biofilter.

The average nitrite+nitrate STE level is 0.1 mg/L. There is little increase of this level in the pump chamber. The value after the biofilter is 41 mg/L, indicating some nitrification. Average values are reduced to 24 mg/L after the denitrification filter. Of note in Appendix C is the response of the nitrate+nitrite values to the operation of the denitrification filter. The missing samples on Oct. 5/95 resulted from a clogged filter. On excavation it was revealed that the wood chip layer (see Figure 5) had expanded, forced the foam into the in-flow pipe, and completely filled the filter resulting in very slow saturated flow. The filter had obviously been working in this state for some time. Some of the wood chips were removed and the filter was returned to the original design state (unsaturated flow). It is interesting to note that in this relatively more aerated state the filter no longer removed nitrates. On July 17/96 the pipe leaving the filter was equipped with an up-turn, returning the filter to a saturated state. Nitrate+nitrite levels for all subsequent samples suddenly dropped from the 40-60 mg/L range to less than 2 mg/L.

The average STE TKN (total kjeldhal nitrogen) level is 220 mg/L. Table 1 shows that TKN is reduced by 38% in the pump chamber, 63% in the biofilter, and decreases 71% after flow through the denitrification filter. Post recirculation TKN STE concentrations are 101 mg/L. Removal in the pump chamber is 45%, a slight increase over initial removal levels, however, biofilter and denitrification removal levels remain virtually unchanged.

Combined nitrogen values, from all the nitrogen forms analyzed in this study, yield the total nitrogen component (TN=TKN + NO_3 + NO_2). Examining the TN values at the various sample points allows the calculation of a simple nitrogen balance. The percent removal of TN from the values measured in the septic tank, are 38% in the pump chamber, 44% after the biofilter, and 60% after the denitrification filter. Post-recirculation values are 41% in the pump chamber, 21% after the biofilter and 66% after the denitrification filter.

Total Phosphorous

Total phosphorous analysis includes both the soluble and particulate phosphorous. Total phosphorous values in the STE average 50 mg/L, higher that typical for residential effluent (USEPA, 1980). Values during the winter and spring of 1995 are quite high but as with the BOD and TKN levels approach normal domestic numbers near the end of the initial monitoring period. Phosphorus levels are reduced by 61% in the pump chamber, however, no significant reduction takes place onward through the system. The implementation of the recirculation system produced no significant results on the phosphorous levels. Phosphorous values are below the detection limit in the groundwater piezometers indicating a lack of mobility most likely due to adsorption in the soil below the disposal bed.

Suspended Solids

Suspended solid (SS) concentrations differ little between the septic tank and the pump chamber with average levels of 142 and 135 mg/L respectively. Even with the presence of the septic tank filter the suspended solids removal is only 5%. SS concentrations are still high, average 115 mg/L after the biofilter, with only a 19% removal from levels in the STE. This increases to 79% after the denitrification filter. Much higher levels of solids reduction have been achieved by the biofilter at other locations (Jowett, 1995).

After the recirculation system was installed the suspended solids content of the pump chamber was dramatically reduced to an average of 39 mg/L a 65% reduction from the STE value of 113 mg/L. Suspended solids levels in the groundwater piezometers showed no significant change over the study period.

pH

pH values show a small gradual increase from the STE to the pump chamber and the biofilter and then a slight decrease after the denitrification filter. The implementation of the recirculation system produced no noticeable change in the values or pattern.

Coliform Bacteria

The STE average values for the total and fecal coliform, respectively are 3.1×10^7 and 2.8×10^7 counts/100 ml, as shown in Table 2. The total coliforms average increases slightly in the pump chamber, however, they are reduced by 88% after flow through the biofilter. Fecal coliforms are reduced by 56% in the pump chamber and 94% after the biofilter. The denitrification filter further increase the removal of the total coliforms to 95% and the fecal's to 98%. Installation of the recirculation system resulted in reduced concentrations in both total and fecal coliforms in the pump chamber and after the denitrification filter. Other numbers remained essentially unchanged.

Table 3:
Water Use

Water Meter (imp. gal)			
Date	House	Ext.Tap	Rate (m^3/day)
19-Sep-94	697260	50005	
14-Nov-94	702465	50031	0.42
3-Feb-95	709984	50055	0.42
27-Apr-95	718730	50059	0.48
5-Oct-95	748482	50319	0.83
9-Apr-96	770395	50321	0.53
13-Jun-96	778113	50335	0.54
27-Nov-97	873530	50757	0.81

Table 4:
Water Levels

Date	Water Level (m below casing top)		
	Piez 1	Piez 2	Piez3
31-Aug-94	4.27	3.02	2.99
19-Sep-94	3.58	2.87	2.82
3-Oct-94	3.52	2.79	2.75
11-Oct-94	3.40	2.70	2.68
24-Oct-94	3.42	2.69	2.64
2-Nov-94	3.20	2.41	2.38
17-Feb-95	2.21	1.54	1.59
23-Feb-95	2.36	1.71	1.76
29-Mar-95	2.45	1.76	1.79
27-Apr-95	2.03	1.45	1.46
1-Jun-95	1.89	1.87	2.55
29-Jun-95	2.40	1.74	1.79
27-Jul-95	2.90	2.19	2.18
31-Aug-95	3.43	2.71	2.68
5-Oct-95	3.71	3.00	2.95
9-Nov-95	1.70	1.12	1.22
14-Dec-95	1.74	1.15	1.26
22-Feb-96	1.89	1.14	1.18
9-Apr-96	1.75	1.13	1.20
13-Jun-96	2.34	1.66	1.70
7-Aug-96	2.48	1.81	1.82
15-Oct-96	2.06	1.38	1.44
15-Jan-97	3.36	1.69	1.70
19-Mar-97	2.24	1.54	1.58
11-Jun-97	2.67	1.96	1.96
9-Sep-97	3.75	3.05	3.00
27-Nov-97	2.04	1.41	1.45

Water Use / Water Level

Water meter readings show that the average daily water use is 580 L/day (see Table 3). Typical flow values are approximately 205 L/c/day (45 igal/c/day), which for a family of four would be approximately 820 L/day. Conservation measures have resulted in a flow rate 30% lower than average use rates even though a portion of this daily flow is also used for backwash of the water conditioner.

The use of water conserving fixtures is one possible explanation for some of the higher than average chemical concentrations in this effluent. Laak (1986) estimates 85 g/c/day of BOD for average household wastewater streams. Using these numbers this household should contribute 340 g/c/day of BOD (340,000 mg/c/day). Assuming even mixing and no removal by settling of particles or biodegradation, and dividing the daily contribution by the daily flow, 580 L/day, we obtain a concentration of 586 mg/L BOD. Although these assumptions are unrealistic this estimated value is close to the range of measured BOD levels. To show the effect of conservation on chemical concentration, using the same assumptions as above, if the flow rate is the more typical 820 L/day the BOD concentration drops to 414 L/day.

Water levels as measured in the 3 piezometers are shown in Table 4. The seasonal variation of groundwater levels can be seen in all wells with the highest water levels in the spring and lowest levels in the fall.

Temperature

Table 5 shows the temperature of the STE and the groundwater in piezometers 1, 2, and 3. The average STE temperature is 17°C. Winter temperatures are approximately 15°C and summer temperatures are as high as 24°C. The shallow groundwater shows a seasonal variation of temperature with lows of near 3°C in the spring and highs of 10°C in the late fall.

From Table 5 it can be seen that the septic tank water temperature remained warm even during the winter months. Tests of tank temperature at other locations in Nova Scotia have shown winter tank temperatures as low as 6°C. However, it must be remembered that the tank temperatures taken in this study are snapshots. Readings taken either a short time before or after may have varied considerably depending on the quantity and temperature of the water use within the house. It is expected that the benefits of increased biological activity in the septic tank, as a result of the warmer temperatures from insulation, would be even greater in a climates, such as central and western Canada, that have much greater temperature extremes than Nova Scotia.

Table 5:
Temperatures

	Water Temp (C)			
Date	ST	Piez 1	Piez 2	Piez3
29-Mar-95	10.4	4.7	3.3	3.0
27-Apr-95	14.2	3.5	2.9	2.8
27-Apr-95		6.4	6.7	6.4
1-Jun-95	13.1	6.5	6.4	5.6
29-Jun-95	18.4	7.2	6.5	6.7
27-Jul-95		7.5	8.0	8.2
31-Aug-95	22.9	8.0	7.6	8.1
5-Oct-95	20.4	8.0	8.9	9.0
9-Nov-95	16.9	9.2	9.2	9.1
21-Feb-96	15.4	5.7	4.8	4.8
9-Apr-96	16.4	3.8	3.4	2.8
13-Jun-96	24.2	7.3	7.2	6.8
7-Aug-96	22.6	9.1	9.0	7.4
15-Jan-97	6.9	7.2		4.2
19-Mar-97	17.6	6.1	3.7	2.8
11-Jun-97	19.0	6.5	6.4	6.1
9-Sep-97	20.3	10.1	9.7	10.1

System Operation

The initial installation of the effluent pump and float switches at this site was wired incorrectly resulting in power being applied to the pump at all times. Thus the pump operated until the chamber was dry and continued to operate until the thermal protection within the pump shut it down. When the pump cooled it would start to operate again and the cycle would be repeated until the pump failed sometime in the fall of 1994. The pump and float switches were replaced and wired correctly in November of 1994. Observations by Dr. Jowett that the biofilter treatment performance would be optimized if it was given multiple small doses over the day as opposed to the 4 doses provided by the original pump switch, resulted in the purchase and installation of a programmable pump control panel in February of 1995.

The expansion of the wood pellets, used as the carbon source in the denitrification filter, caused the filter to clog in October of 1995. Corrective measures included excavation of the system and removal of the excess wood pellets. It is estimated that the expansion factor is roughly 100% when wet. Future filter designs should take this into account when filling the filter with dry pellets. The difference between the clogged (saturated) and unclogged (unsaturated) flow in terms of nitrate removal is significant. In July of 1996 the denitrification filter was once again excavated and an inverted U-trap was placed on the outflow pipe which raised the level of the effluent within the filter resulting once again in anaerobic flow. Future design refinements of this filter will ensure that anaerobic conditions are maintained to assure optimum nitrate removal.

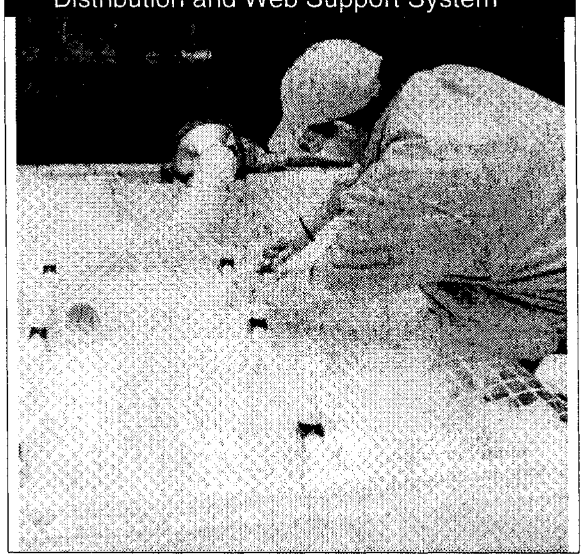
The homeowner had two complaints about the biofilter; the noise and the smell. The noise resulted from the thump of the pump control relay switch in the controller box installed on the side of the biofilter. For a period of the system operation, the high level float switch was malfunctioning and instead of engaging the pump and running until the low level effluent switch shut the pump off, it caused the relay switch and pump to repeatedly click on and off. This eventually resulted in a failure of the pump in March of 1997. Both the pump and the float switch were replaced. Examination of the float switch revealed that some moisture had migrated up the electrical cord leading to the switch and was most likely responsible for the repeated activation of the relay switch.

The homeowner reported that the smell from the biofilter was apparently quite strong at times. During field visits an effluent odour was apparent within a metre of the biofilter. Discussion with Dr. Craig Jowett indicated that adjusting the fan to a slower speed had reduced the odour at other installations. Recommendations for future installations are to place a rheostat on the fan electrical line in the control box and to place the biofilter farther from the dwelling. The biofilter at this site is approximately 2 metres from the rear side of the house.

During the project a large degree of settling of the foam blocks in the biofilter was noted; the volume was reduced by approximately 50%. Although the frame that supports the distribution nozzles was on webbing that kept it from sinking into the foam, as shown in Figure 8, with the settling of the entire frame the hose from the pump became kinked reducing the flow volume and resulting in a period of almost continuous pump operation. The frame was propped up which partially restored the flow through the nozzles, however, a few had become permanently clogged. Since the installation of this unit, design modifications by Dr. Jowett that have addressed these concerns include a deeper filter bed (1.5 m enclosure), downward facing spray nozzles, modular ventilated baskets in the enclosure, vents which can be fitted with carbon to absorb odours, and basic 50% recirculation to the septic tank.

The difficulty of obtaining a sample from after the biofilter and after the denitrification filter (Figures 4 and 5) necessitated the installation of sample traps. Chemistry of results taken from these traps may have varied from flow-through samples. Future sampling designs should incorporate a flow-through collection arrangement.

Figure 8:
Distribution and Web Support System



Samples taken on either side of the septic tank filter, in the septic tank and in the pump chamber, show an average reduction of 56% for BOD, 61% for phosphorous, 5% for suspended solids, and 33% for ammonia. These samples were drawn from the main sections of septic tank and pump chamber and are not necessarily due to the effects of the filter alone. Some of the difference in the effluent chemistry between the septic tank and the pump chamber may be the result of differing biological regimes in these two areas.

The septic tank filter was inspected after one year of operation. When the tank was uncovered it was observed that scum layer was almost to the top of the outflow tee. When the filter was pulled out a solid plug of scum fell out of the center of the filter and the level of the top of the scum layer in the tank dropped approximately 3 inches. The filter was partially clogged, with what appeared to be lint and grease, and restricting flow. Although it is disconcerting to have the filter partially clogged after one year of operation, on the positive side the filter was doing what it was designed to do: preventing large solids from moving out of the septic tank. It is recommended that the tank and the filter at this site be inspected at no greater than a three-year frequency.

One of the questions raised by this research is whether a smaller disposal field can be constructed because the flow has been reduced through water conservation. Current designs in Nova Scotia are based on a flow rate of approximately 1m^3/day per household. Although the length of the system is calculated using the daily flow rate it is often constructed as long as is reasonably possible on the site to provide a margin of safety and to help ensure long term operation. Increased flow can result from social gatherings, increased occupancy, addition of appliances such as whirlpool tub or water treatment systems, or malfunction of existing fixtures such as leaking taps or toilets. Water conservation should be treated as an additional level of insurance for the investment made in the disposal system and not as a reason to reduce field size. The initial savings, in terms of material and construction costs, are far outweighed by the advantages of long-term system operation and future flexibility of a full size disposal system.

A related question concerns allowing a higher loading rate, and thus a reduced field size, for pretreated wastewater in which most of the organic material has been removed. The organic material in wastewater is food for microorganisms. The higher the organic content, often expressed as BOD and SS, in the wastewater, the more microorganisms and their associated slimes that are produced and the more likely a soil surface is to clog. An increased organic load can also result from either a large volume of low BOD-SS or a small volume of high concentration wastewater.

Investigations of the long-term acceptance rate (LTAR) of various chemical concentrations of wastewater have shown that over time all systems approach a loading rate that is a small percentage of the initial rate (Laak, 1986, Siegrist 1987, Tyler, 1994). Exceptions to this are tap water and very highly pretreated waters, which tend to maintain LTAR's close to initial levels. Most administrative design codes are based on numbers that are in the range of the LTAR for the soils in a particular area. Loading rates in Wisconsin vary from 3.3 to 0.8 cm/day depending on soil type, texture and structure. The current Nova Scotia loading rate is 3.2 cm/day for all soil types. The use of a consistent sand layer in the bottom of the trenches and beds allows for the use of a consistent loading rate as the biomat develops on the sand layer and not the host soil.

Previous investigators (Siegrist 1987, and Tyler 1994) have suggested the potential for loading rate increases for systems using highly pretreated wastewater. Tyler (1994) has suggested a range of loading rate increases for pretreated effluent depending on the type of soil. Relatively impermeable soils would use a loading rate increase factor of 2 with a range extending up to a factor of 16 for permeable sandy soils.

It is difficult to assess if the pretreatment examined in this report would allow for a higher loading rate at this site. During this research project no ponding was detected in the disposal field and the impact on the down gradient wells was minimal, indicating that it may have been a possibility. However, any system design that relies on an increased loading rate should be approached with caution. Such a design would require strict operational control to ensure that the hydraulic and organic loading rates were never exceeded due to a malfunction of the pretreatment unit.

Summary

A residential on-site wastewater disposal system was constructed using a contour trench design based on flow principles instead of rule of thumb estimates. The system design involved a soil permeability analysis using an inexpensive permeameter. Construction specifications and conductivity equations are shown in Appendix A. It should be noted however, that in Nova Scotia, as in many other jurisdictions, that soil analysis from a test pit provides the primary information necessary for a rational system design as outlined in Appendix B.

Water conserving fixtures reduced the hydraulic load on the system by 30% over average literature values although this may have contributed to higher than normal concentrations of measured parameters in the STE.

Insulation of the septic tank gave average STE temperatures of 17°C. The septic tank effluent filter provided some chemical renovation of the effluent, but most importantly prevented the movement of lint and large solids from the tank into the disposal field. Rinsing the clogging solids off the filter and replacing it in the septic tank is much easier than removing the same material from the disposal bed pipes, gravel, or soil pores.

After passing through the biofilter the effluent showed removal rates of 95% for BOD, 58% for phosphorous, 19% for suspended solids, and 94% for fecal coliforms. Biofilter operational concerns with the effluent distribution system, foam settling, and odour have been addressed by Dr. Jowett in subsequent system designs. One of the benefits of a system of this design is that it allows for effluent collection, should further removal or monitoring of nutrients or contaminants be required prior to surface or subsurface disposal.

The necessity of ensuring anaerobic conditions in the design of the denitrification filter was demonstrated. The denitrification filter reduced the post biofilter total nitrogen load of the effluent a further 15% to 35% prior to subsurface disposal and would be a valuable system component where the receiving environment is nutrient sensitive. However, the denitrification filter design used is not likely to provide a commercially viable product without further modifications to reduce the levels of maintenance required for regular operation.

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Appendix A

Permeameter Test Results

Pit 1

Soil selection number: 2

a (cm) = 6   H (cm) = 10   X or Y (cm^2) = 51.53   R (cm/min) = 1

	K (cm/s)	Phi (cm^2/s)
Maximum	9.8E-04	1.1E-02
Best estimate	5.7E-04	4.8E-03

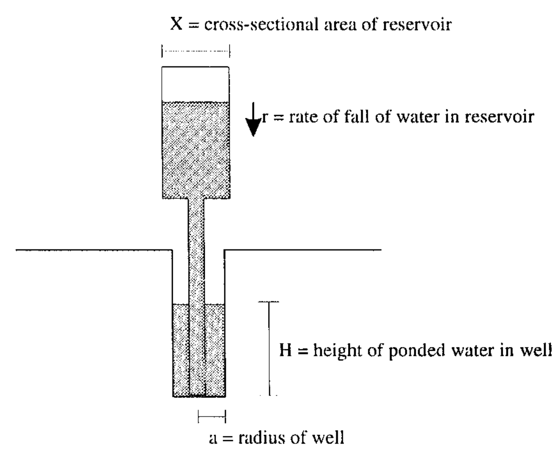
Pit 2

Soil selection number: 2

a (cm) = 4   H (cm) = 10   X or Y (cm^2) = 51.53
R (cm/min) = 1

	K (cm/s)	Phi (cm^2/s)
Maximum	1.4E-03	1.5E-02
Best estimate	7.9E-04	6.5E-03

Figure A1:
Calculation Components



Permeameter calculations are based on the work of D. Elrick and W. Reynolds. For an in depth review of this technique see: Reynolds, W.D. et al., 1992, An assessment of the single-head analysis for the constant head well permeameter, Can. J. Soil Sci. 72: 489-501.

As noted in the above reference this method of calculation gives field-saturated conductivity (Kfs), indicating that the soil pores may contain some entrapped air. Thus Kfs can be roughly half the truly saturated hydraulic conductivity.

Also noted in the above reference the use of the curves to determine the c value strictly speaking only applies for soils at about field capacity or drier and when the wetting front from the test holes does not appear at the soil surface.

Calculations are based on:

$Q = Xr$

$A = (2 \Pi H^2)/C + (\Pi a^2)$

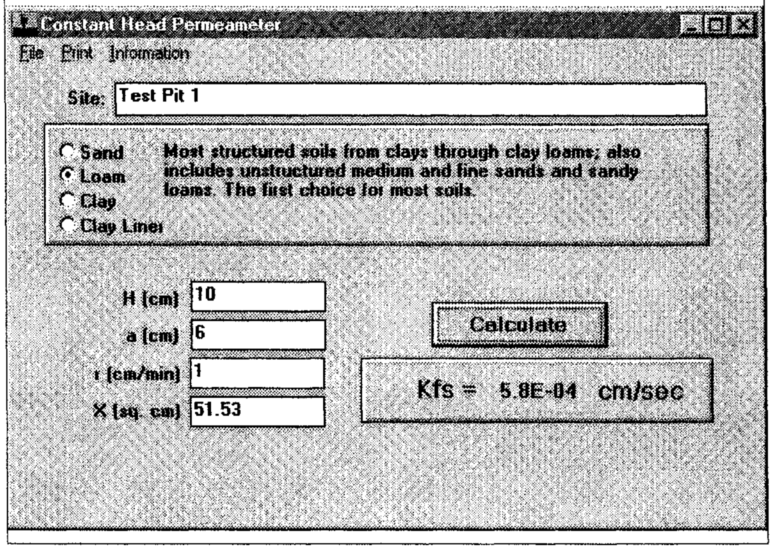
$B = 2 \Pi H / X$

$Kfs = Q/(A + B/alpha)$

Where alpha is determined from the soil type (the alpha value for sand =0.36cm-1, loam =0.12 cm-1, clay =0.04 cm-1, and clay liner =0.01 cm-) and C is determined from empirical curves (in above reference) and the variables a, H, X, and r as shown in Figure A3.

A windows based program that automatically performs the above calculations(Figure A1-2) has been developed. For information contact J. Mooers (jordan.mooers@dal.ca). Permeameter construction specifications are shown in Figure A1-3.

Figure A2:
Permeameter Calculation Program



IN-SITU PERMEAMETER OPERATING INSTRUCTIONS

1. Loosen the reducing connector. Release and extend the small diameter tube fully. Re-tighten the connector.

2. Auger the hole to a depth of 18-20 inches, using a standard dutch auger giving a hole of radius 4 cm. Care should be taken to locate the hole in a location the will most closely represent the conductivity values of the area in question. Attention should be paid to any soil condition that may cause an erroneous high reading such as macro-pores due to excessive insect or rodent activity, roots, clay or gravel lenses, or soil cracks.

3. If the soil is fine grained and wet it may be necessary to remove the smear layer, that may develop when auguring, from the sides of the hole with a brush. (A windshield snow brush with bristles cut to 1/4 inch length will do).

4. Stand the device upside down, fill with water to the air inlet hole and insert the rubber stopper.

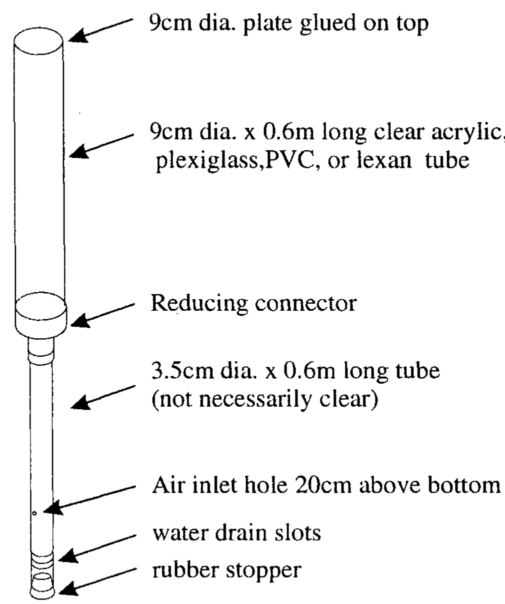
5. Invert the device rapidly and insert in the hole until the rubber stopper rests on the bottom.

6. The constant head in the well is allowed to come to equilibrium, usually 5 to 30 minutes depending on soil type. The time interval and the reservoir water levels are recorded until the rate of fall becomes constant for at least 3 successive readings.

7. The rate of fall of water in the reservoir is calculated in cm/min (note: if the fall of water in the reservoir is fast the original readings can be done in cm/sec and then converted to cm/min).

8. The constant values describing the permeameter and the hole, as shown in formulas above, are used to calculate the field hydraulic conductivity.

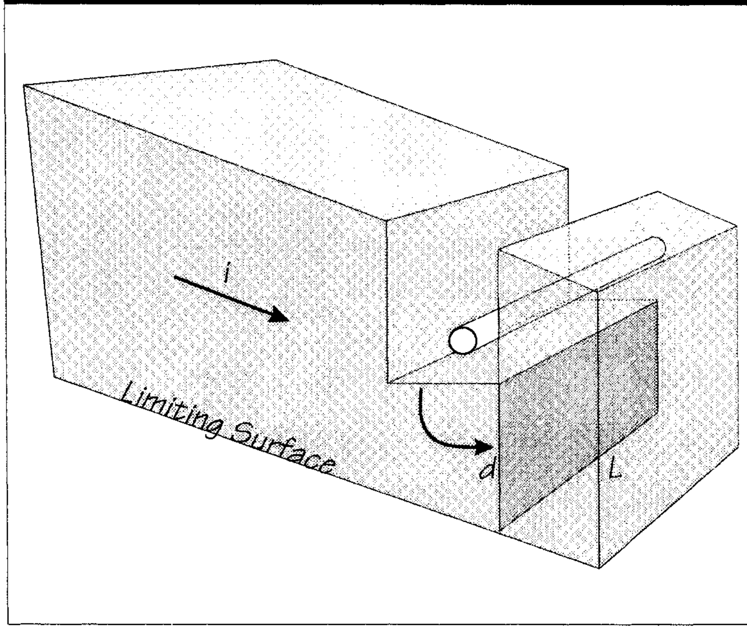
Figure A3:
Construction Specifications



Appendix B.

Disposal Field Design

Figure B1:
Trench Length Calculation Parameters



1. Proper design of the soil absorption system is critical to the successful disposal and treatment of household effluent. One important aspect in a rational design is the site evaluation which includes the determination of the host soil's hydraulic characteristics. During the evaluation of this lot the house location, well location and lot boundaries were laid out and the lot slope was measured.

2. A back hoe was used to dig two test pits at the proposed location of the disposal bed. Soil type and depth to high seasonal groundwater table were determined. Examination of the soils showed the upper horizon to be a sandy silt.

It must be emphasized that there is no deterministic relationship between hydraulic conductivity and soil type, due to a number of factors such as compaction, a table of approximate relationships is listed in the Nova Scotia On-Site Wastewater Disposal Guidelines (1988). The sandy silt conductivity range is listed as 2×10^{-6} to 8×10^{-6} m/sec. A similar table in the Ontario Sewage Systems Regulations (1990) gives a conductivity range for sand silt mixtures as 1×10^{-5} to 1×10^{-7} m/sec. As discussed in Appendix A of this report, two permeameter tests were conducted near the test pits at the soil depth of the disposal trench and gave values of 5.7×10^{-6} to 1.4×10^{-5} m/sec. The lower value was used for design purposes as this will yield the maximum trench length and the safest design.

Measuring the soil depth, from the bottom of the organic layer to the level in the test pits that showed some seepage of groundwater, gave a total useable depth of approximately 1.2m. The standard Nova Scotia fully trenched contour design (C1 system), requires 0.675m of trench depth. This would leave only 0.525m of trench below the disposal bed and above the limiting surface (seasonal high groundwater). Calculation of the required bed length indicated that a system based on this depth would exceed the lot boundaries. A partially trenched contour design (C2) requires only 0.3m depth leaving 0.9m as the design depth for the determination of system length.

3. After the determination of the site slope, soil conductivity, and soil depth, the length of trench,"L" shown in Figure A2-1, necessary to pass the daily volume of effluent can be calculated. As this volume is assumed to pass through the soil slice "d x L", shown in Figure A2-1 it is first necessary to calculate the area of this slice using the following information and steps:

- hydraulic conductivity = K = 5.7×10^{-6} m/sec
- hydraulic gradient = i = 0.1
- soil depth = d = 0.6 m
- loading rate = q = 0.91 m^3/day (200 igal/day)

1. Determine the velocity of the effluent: Velocity $= V = K\,i = 5.7 \times 10^{-7}$ m/sec

2. Determine soil area necessary to carry daily flow: $A = q / V = 18.6\ m^2$

3. Calculate length of soil slice based on area: $L = A/d = 20.6$ m

Therefore a slice of soil of 0.9m deep and 20.6m long will pass 910 L/day of liquid.

The length available for the disposal trench was 24m (80 ft). Standard design practice is to make the field as long as reasonably possible as a safety measure to help absorb periods of peak flow. A length of 24m was accepted as the best length of field based on the soil characteristics.

4. Determine the width of the bottom of the disposal trench based on a design long term acceptance rate (LTAR). As discussed in Section 2.7 a standard Nova Scotia system has a 7.5 cm layer of sand laid on top of the host soil. Effluent passing through the gravel will form a biomat on the layer of sand. As the sand is a consistent specification the LTAR of the biomat tends to be consistent. A standard household system LTAR is $0.031 m^2$/L/day (1.5 ft^2/gal/day). The width of the disposal trench, "W" in slice W x L in Figure A2-2 can be calculated using the following information and steps:

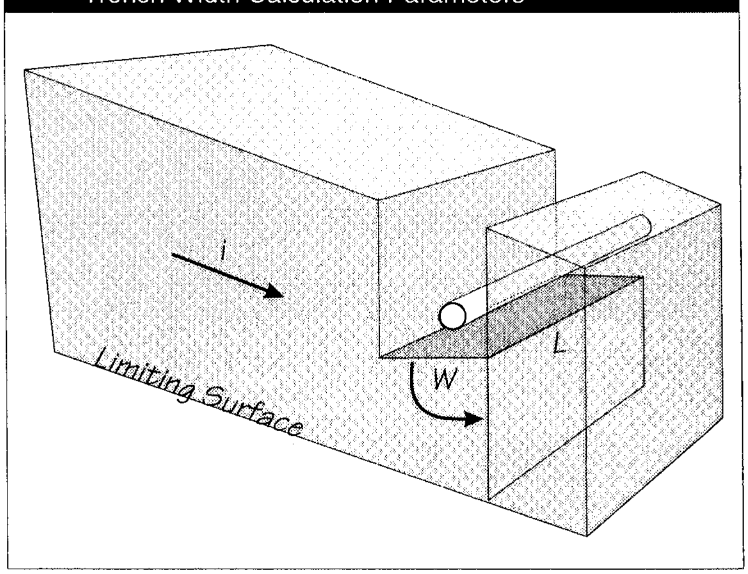
- loading rate = $q_2 = 0.91\ m^3$/day
- LTAR = $0.031 m^2$/L/day
- length L = 24m

1. Determine the area of W x L = q * LTAR = $27.9 m^2$

2. Calculate the trench width = A / L = 1.4m

The trench bottom dimensions are 24m long x 1.4m wide. The rest of the C2 system is constructed as shown in Figure 6 and as documented in the Nova Scotia On-Site Wastewater Disposal Guidelines (1988).

Figure B2:
Trench Width Calculation Parameters



Appendix C

Chemical Analysis Results

Total Phosphorus

DATE	T	PC	B	DN	P1	P2	P3
3-Oct-94	9.8		5.1	9	0.1	0.1	0.1
11-Oct-94	15		4.8	8.2	0.1	0.1	0.1
17-Oct-94	16		4.9	3.7	0.1	0.1	0.1
24-Oct-94	16		5	5.8	0.15	0.1	0.1
2-Nov-94	20		5.2	5.5	0.1	0.1	0.1
17-Feb-95	17		20		0.1	0.1	0.1
23-Feb-95	59	18	18	24	0.1	0.1	0.1
29-Mar-95	111	21	26	24	0.1	0.1	0.1
27-Apr-95	97	16	11	17	0.1	0.1	0.1
1-Jun-95	144	6.2	6.2	9.8	0.1	0.1	0.1
29-Jun-95	62	24	22	22	0.1	0.1	0.1
27-Jul-95	74	33	33	26	0.1	0.1	0.1
31-Aug-95	38	28	26	26	0.1	0.1	0.1
5-Oct-95	37	25	18		0.1	0.1	0.1
9-Nov-95	23	22	19	19	0.1	0.1	0.1
14-Dec-95	19	17	22	15.8	0.1	0.1	0.1
22-Feb-96	16	14	21	15	0.1	0.1	0.1
9-Apr-96	18	17	17	15	0.1	0.1	0.1
13-Jun-96	16	17	31	18	0.1	0.1	0.1
7-Aug-96	23	19	33	21	0.1	0.1	0.1
15-Oct-96	14	16	13	13	0.1	0.1	0.1
Average	50	20	21	19	0.1	0.1	0.1
% Removal		61%	58%	62%	100%	100%	100%
15-Jan-97	13	14	20	13	0.1	0.1	0.1
19-Mar-97	13	12	22	14	0.1	0.1	0.1
12-Jun-97	12	12	12	15	0.1	0.1	0.1
9-Sep-97	11	11	12	13	0.1	0.1	0.13
27-Nov-97	17	12	12	12	0.25	0.1	0.1
Average	13	12	16	13	0.1	0.1	0.1
% Removal		8%	-18%	-2%	99%	99%	99%

Nitrate+Nitrite

DATE	T	PC	B	DN	P1	P2	P3
3-Oct-94	0.05		0.48	0.05	0.05	0.05	0.05
11-Oct-94	0.05		0.56	0.05	0.05	0.05	0.05
17-Oct-94	0.06		0.51	0.05	0.05	0.05	0.05
24-Oct-94	0.05		0.59	0.05	0.05	0.05	0.05
2-Nov-94	0.05		0.81	0.05	0.05	0.05	0.05
17-Feb-95	0.05		0.05	0.05	0.05	0.05	0.05
23-Feb-95	0.05	0.05	0.05	0.05	0.05	0.05	0.05
29-Mar-95	0.05	0.05	0.05	0.05	0.05	0.05	0.05
27-Apr-95	0.05	0.05	0.23	7.9	0.05	0.05	0.05
1-Jun-95	0.05	0.05	21	0.05	0.05	0.05	0.05
29-Jun-95	0.05	0.05	79	27	0.05	0.05	0.05
27-Jul-95	0.13	0.05	88	6	0.05	0.05	0.27
31-Aug-95	0.16	0.16	47	9	0.05	0.05	0.05
5-Oct-95	0.05	0.05	38		0.05	0.05	0.05
9-Nov-95	0.17	0.16	56	61	0.24	2.8	1.34
14-Dec-95	0.05	0.05	29	57	3.3	6.5	8.8
22-Feb-96	0.05	0.05	37	58	0.2	0.05	4.5
9-Apr-96	0.05	0.05	47	43	0.05	0.05	4.2
13-Jun-96	0.31	0.32	73	62	0.05	0.05	1.44
7-Aug-96	0.4	0.54	59	0.22	0.05	0.05	1.05
15-Oct-96	0.33	0.31	39	0.88	0.05	0.05	2.95
Average	0.1	0.1	41	24	0.3	0.7	1.7
15-Jan-97	0.25	0.25	78	1.37	0.05	0.05	3.00
19-Mar-97	0.05	0.05	128	0.05	0.05	0.05	0.12
12-Jun-97	0.06	9	11	0.05	0.05	0.05	0.05
9-Sep-97	0.05	0.05	3	0.05	0.05	0.05	0.05
27-Nov-97	0.05	12	14	0.05	0.99	0.07	0.05
Average	0.1	4.2	47	0.3	0.2	0.1	0.7

Ammonia

DATE	T	PC	B	DN	P1	P2	P3
3-Oct-94	75.0		0.05	0.05	0.05	0.05	0.05
11-Oct-94	3.8		0.05	0.1	0.05	0.05	0.05
17-Oct-94	97.2		0.3	0.05	0.05	0.05	0.05
24-Oct-94	91.0		0.2	0.05	0.05	0.05	0.05
2-Nov-94	25.8		0.1	0.9	82.0	0.1	0.1
17-Feb-95	84.9		109.0		0.05	0.05	0.05
23-Feb-95	250	86	96	108	0.05	0.05	0.1
29-Mar-95	148	79	122	106	0.05	0.05	0.1
27-Apr-95	235	87	50	30	0.1	0.05	0.05
1-Jun-95	188	37	11	0	0.05	0.05	0.05
29-Jun-95	233	158	44	48	0.05	0.05	0.1
27-Jul-95	278	175	37	30	0.05	0.05	0.05
31-Aug-95	202	151	93	102	0.05	0.05	0.05
5-Oct-95	194	148	55		0.05	0.05	0.05
9-Nov-95	149	140	65	61	0.1	0.2	0.1
14-Dec-95	147	128	73	61	0.1	0.4	0.1
22-Feb-96	106	104	63	59	0.2	0.3	0.1
9-Apr-96	143	139	62	60	0.1	0.3	0.1
13-Jun-96	112	121	48	48	0.05	0.3	0.1
7-Aug-96	151	124	55	49	0.05	0.3	0.1
15-Oct-96	84	82	39	46	0.1	0.4	0.1
Average	175	117	61	58	0.1	0.2	0.1
% Removal		33%	65%	67%	100%	100%	100%
15-Jan-97	74	81	3	30	0.05	0.38	0.08
19-Mar-97	88	84	51	40	0.05	0.47	0.15
12-Jun-97	74	12	10	43	0.05	0.35	0.09
9-Sep-97	71	28	23	16	0.05	0.33	0.07
27-Nov-97	100	14	8	21	0.05	0.34	0.06
Average	81	44	19	30	0.1	0.4	0.1
% Removal		46%	77%	63%	100%	100%	100%

KjeldhalNitrogen

DATE	T	PC	B	DN	P1	P2	P3
3-Oct-94	101		2.3	20	0.2	0.3	0.60
11-Oct-94	135		2.2	18	0.3	0.3	0.30
17-Oct-94	143		2.3	14	0.1	0.1	0.1
24-Oct-94	124		2.6	16	0.5	0.2	0.20
2-Nov-94	130		2.4	14	0.1	0.1	0.10
17-Feb-95	90		109		0.05	0.05	0.05
23-Feb-95	263	106	100	117	0.1	0.1	0.10
29-Mar-95	297	100	129	108	0.1	0.1	0.10
27-Apr-95	268	96	96	81	0.1	0.1	0.20
1-Jun-95	364	46	21	50	0.1	0.1	0.1
29-Jun-95	275	182	47	54	0.1	0.1	0.10
27-Jul-95	335	209	44	41	0.1	0.1	0.1
31-Aug-95	224	168	94	102	0.1	0.1	0.20
5-Oct-95	217	165	61		0.1	0.1	0.10
9-Nov-95	174	158	68	68	0.2	0.4	0.20
14-Dec-95	156	139	101	60	0.2	0.5	0.20
22-Feb-96	126	121	108	59	0.2	0.4	0.20
9-Apr-96	158	158	75	61	0.1	0.5	0.30
13-Jun-96	136	148	110	50	0.2	0.5	0.40
7-Aug-96	209	168	131	60	0.1	0.5	0.20
15-Oct-96	103	100	40	48	0.1	0.6	0.40
Average	220	138	82	64	0.1	0.3	0.2
% Removal		38%	63%	71%	100%	100%	100%
15-Jan-97	97	103	46	32	0.2	0.6	0.30
19-Mar-97	110	105	61	45	0.1	0.5	0.20
12-Jun-97	87	15	14	50	0.1	0.5	0.20
9-Sep-97	89	38	33	20	0.5	0.6	0.50
27-Nov-97	123	18	11	24	1.0	0.5	0.30
Average	101	56	33	34	0.4	0.5	0.3
% Removal		45%	67%	66%	100%	99%	100%

Biochemical Oxygen Demand

DATE	T	PC	B	DN	P1	P2	P3
3-Oct-94	337		6	745	27	20	8.3
11-Oct-94	471		15	481	11.4	6.6	8.1
17-Oct-94	425		5	221	5	2.0	3.2
24-Oct-94	301		3	111	5.4	2.0	2.0
2-Nov-94	348		11	97	4.1	4.9	2.0
17-Feb-95	304		17		2.0	2.0	2.0
23-Feb-95	1310	366	8	50	21	2.2	2.0
29-Mar-95	2140	402	27	15	2.0	2.0	2.0
27-Apr-95	1890	367	78	14	2.9	3.5	4.7
1-Jun-95	3030	127	25	174	3.2	3.7	2.0
29-Jun-95	1400	712	83	62	3.5	3.5	8.11
27-Jul-95	1720	521	11	26	6.1	5.7	2
31-Aug-95	1040	925	26	25	9.5	3.3	2
5-Oct-95	1310	895	44		157	13	26
9-Nov-95	892	788	19	14	2.7	2	2
14-Dec-95	635	525	146	6	2	2	2
22-Feb-96	635	458	139	35	4.2	5.3	4.4
9-Apr-96	816	595	44	17	3.2	3.1	8.5
13-Jun-96	765	588	78	17	2.3	2.6	2
7-Aug-96	768	562	117	57	6.2	2.2	2.1
15-Oct-96	546	538	21	24	2.1	2	3.1
Average	1260	558	58	38	15	4	5
% Removal		56%	95%	97%	99%	100%	100%
15-Jan-97	460	517	76	8	14	11	8
19-Mar-97	452	444	13	13	4	4	2
12-Jun-97	451	7	9	22	6	3	2
9-Sep-97	429	46	28	6	15	3	3
27-Nov-97	529	19	17	18	7	12	52
Average	464	207	29	14	9	7	13
% Removal		55%	94%	97%	98%	99%	97%

Suspended Solids

DATE	T	PC	B	DN	P1	P2	P3
3-Oct-94	54.8		15.2	72	2	5.2	22
11-Oct-94	94		11.3	52	3	12	3
17-Oct-94	80		5	33	1.2	2.7	2
24-Oct-94	98		5	52	3	7.2	4
2-Nov-94	100		20	31	16	9	6
17-Feb-95	80		31		2	3.5	2.5
23-Feb-95	103	76	9	20	1.5	3	2
29-Mar-95	76	110	64	38	1.2	1.2	1.2
27-Apr-95	175	96	88	23	2.5	5.5	6.5
1-Jun-95	91	45	51	22	2.5	1.5	3.5
29-Jun-95	185	272	13	59	2	3.5	2
27-Jul-95	175	235	43	38	8	8.5	3
31-Aug-95	236	243	36	18	3	4	4
5-Oct-95	160	132	46		4	3	4.7
9-Nov-95	156	125	29	18	2.4	2.4	2.4
14-Dec-95	177	96	320	2.4	1.2	4.4	1.6
22-Feb-96	227	143	360	14	2.8	6.8	2.4
9-Apr-96	145	140	126	14	2	2	2.5
13-Jun-96	103	103	324	7	1.5	1.5	1.5
7-Aug-96	3	3	152	116	8	6	2
15-Oct-96	111	207	58	18	1.5	1.5	3
Average	142	135	115	29	3	4	3
% Removal		5%	19%	79%	98%	97%	98%
15-Jan-97	92	74	37.3	12	2.5	3.0	3.0
19-Mar-97	84	63	64	14	2.0	1.5	1.5
12-Jun-97	66	10	26	19	1.2	1.2	1.2
9-Sep-97	140	42	52	14	2.5	1.5	2.0
27-Nov-97	183	7	11	20	4.0	4.0	7.0
Average	113	39	38	16	2	2	3
% Removal		65%	66%	86%	98%	98%	97%

DATE	pH T	pH PC	pH B	pH DN	pH P1	pH P2	pH P3	Chloride T	Chloride PC	Chloride B	Chloride DN	Chloride P1	Chloride P2	Chloride P3
3-Oct-94	7.8		7.7	6.5	7.3	7.7	7.9							
11-Oct-94	7.5		7.3	6.8	7.9	7.8	8.1							
17-Oct-94	7.7		7.6	7.3	7.9	8.1	8.1							
24-Oct-94	7.9		8.1	7.6	7.9	8.0	8.0							
2-Nov-94	7.9		8.1	7.6	7.8	8.2	8.1							
17-Feb-95	7.5		8.0		8.0	8.8	8.4							
23-Feb-95	6.9	7.3	8.3	7.3	7.7	7.9	8.0	244	90	86	100	11	13	20
29-Mar-95	6.9	7.7	8.3	7.8	7.8	8.2	8.1							
27-Apr-95	6.6	7.4	8.1	7.9	7.8	7.8	7.7							
1-Jun-95	6.4	7.3	6.9	7.0	7.9	8.1	8.1	502	90	93	97	28	22	18
29-Jun-95	6.8	7.4	7.1	7.5	7.8	8.2	8.1	293	214	190	178	38	27	34
27-Jul-95	6.6	7.4	6.1	7.2	7.7	8.5	8.0	365	227	181	170	32	29	37
31-Aug-95	6.8	6.9	7.7	7.7	7.9	8.1	8.2	255	202	197	204	36	32	36
5-Oct-95	6.5	6.6	7.5		7.7	7.9	7.6	250	214	160		43	35	35
9-Nov-95	6.6	6.8	7.4	6.7	7.5	7.9	7.9	185	174	161	158	52	47	42
14-Dec-95	7.1	7.2	7.3	6.8	7.6	7.9	7.8	167	161	148	147	39	45	48
22-Feb-96	7.0	7.0	7.3	6.7	7.7	8.0	7.9	142	128	143	143	23	27	35
9-Apr-96	6.9	7.0	7.2	6.9	7.3	8.0	7.7	166	164	152	153	25	29	39
13-Jun-96	6.9	7.0	6.6	6.6	7.6	7.9	7.7	140	154	140	141	24	33	47
7-Aug-96	7.3	7.1	7.1	6.8	7.7	8.1	7.8	215	192	171	158	27	31	44
15-Oct-96	7.0	7.1	7.4	7.4	8.6	8.1	7.8	140	133	115	117	35	34	45
Average								236	165	149	136	32	31	37
% Removal									30%	37%	42%	87%	87%	84%
15-Jan-97	6.96	7.02	6	7.06	7.24	7.84	7.83	120	130	125	131	36	33	25
19-Mar-97	7.07	7.00	6.98	6.94	7.28	7.90	7.82	129	122	141	130	30	40	36
12-Jun-97	6.99	7.34	7.28	6.92	7.05	7.96	7.72	143	137	138	118	34	31	40
9-Sep-97	6.62	7.29	7.50	6.51	7.31	7.65	7.48	139	131	132	118	50	43	46
27-Nov-97	6.66	7.02	7.70	6.54	6.82	7.61	7.46	137	124	124	125	77	57	50
Average								134	129	132	124	45	41	39
% Removal									4%	1%	7%	66%	70%	71%

DATE	Total Coliforms T	PC	B	DN	P1	P2	P3	Fecal Coliforms T	PC	B	DN	P1	P2	P3
3-Oct-94	2.0E+08		1.5E+02	2.0E+08	A	A	P							
11-Oct-94	2.0E+08		2.0E+04	7.2E+07	A	P	P	2E+08		0	2200000	A	A	A
17-Oct-94	2.0E+08		2.6E+02	2.0E+07	P	P	P	6.5E+07		30	3800000	A	A	A
24-Oct-94	2.5E+08		0.0E+00	8.7E+06	350	65	46	1.8E+08		0	3000000	4	0	0
2-Nov-94	3.0E+07		7.0E+01	6.5E+06	2000	36	38	5300000		10	0	14	0	0
17-Feb-95														
23-Feb-95	3.5E+07	3.4E+07	4.5E+05	1.9E+06	1640	820	4000	2.3E+06	1.0E+07	2.0E+04	1.7E+05	76	56	320
29-Mar-95	1.2E+08	1.4E+08	2.6E+06	2.0E+06	820	12	26	4.6E+07	8.9E+07	1.8E+05	5.0E+04	24	2	6
27-Apr-95	1.5E+07		4.1E+06	3.2E+06	1380	0	2	3.8E+06	1.8E+06	1.2E+06	7.2E+05	0	0	0
1-Jun-95	1.6E+07	1.9E+07	4.3E+06	5.5E+05	4	0	4	3.0E+08	1.5E+06	1.6E+05	2.0E+04	0	0	0
29-Jun-95	8.4E+07	6.3E+06	1.5E+06	3.4E+06	1860	0	4	2.4E+06	3.7E+06	8.0E+04	9.0E+04	0	0	0
27-Jul-95	9.9E+06	1.4E+06	2.5E+07	7.3E+05	2	0	34	8.3E+06	4.9E+05	2.0E+07	3.6E+05	0	0	0
31-Aug-95	5.4E+07	9.1E+07	3.1E+06	1.9E+06	42	0	6	2.1E+06	8.2E+06	6.0E+04	1.6E+04	0	0	0
5-Oct-95	3.0E+07	2.5E+07	9.5E+06		0	4	0	3.4E+06	5.9E+06	5.1E+05		0	0	0
9-Nov-95	1.8E+07	1.1E+07	6.1E+05	7.0E+04	0	0	0	3.9E+06	2.8E+06	2.2E+05	3.5E+03	0	0	0
14-Dec-95	9.8E+06	8.4E+06	5.3E+05	9.0E+02	74	0	28	1.5E+06	1.3E+06	2.0E+04	0.0E+00	0	0	0
22-Feb-96	3.6E+05	3.5E+05	4.6E+05	1.0E+04	0	0	18	1.2E+05	6.0E+04	6.0E+01	0.0E+00	0	0	0
9-Apr-96	1.0E+07	7.9E+06	2.9E+05	2.5E+05	0	0	0	4.5E+06	3.6E+06	4.0E+04	5.0E+04	0	0	0
13-Jun-96	2.0E+07	3.0E+07	2.0E+04	8.0E+03	60	12	16	4.0E+06	4.5E+07	1.0E+04	5.0E+03	0	0	0
7-Aug-96		1.0E+08		7.2E+06	0	4	82		4.0E+06		4.6E+06	0	0	0
15-Oct-96	9.7E+06	8.5E+06	9.6E+05	1.9E+06	0	4	0	4.7E+06	4.1E+06	2.8E+05	8.3E+04	0	0	0
Average	3.1E+07	3.5E+07	3.8E+06	1.7E+06	392	57	281	2.8E+07	1.2E+07	1.6E+06	4.4E+05	7	4	22
% Removal		-13%	88%	95%	100%	100%	100%		56%	94%	98%	100%	100%	100%
15-Jan-97	9.5E+06	9.0E+06	8.0E+05	6.5E+03	0	0	600	1.7E+06	2.4E+06	2.0E+04	3.5E+03	0	0	0
19-Mar-97	6.2E+06	4.9E+06	3.7E+03	4.4E+02	0	0	64	1.6E+06	1.7E+06	5.5E+02	2.0E+01	0	0	0
12-Jun-97	5.6E+06	7.5E+05	1.9E+04	1.5E+05	0	0	26	1.7E+06	5.0E+04	1.5E+04	3.1E+03	0	0	0
9-Sep-97	1.4E+07	9.2E+06	5.0E+06	7.0E+04	0	5.0E+04	30	9.1E+06	5.1E+06	1.1E+06	6.0E+04	0	3.0E+04	0
27-Nov-97	2.0E+07	9.2E+06	5.0E+05	1.7E+05	0	0	0	1.3E+07	3.7E+06	1.6E+05	2.3E+04	0	0	0
Average	1.1E+07	6.6E+06	1.3E+06	7.9E+04	0	1.0E+04	144	5.4E+06	2.6E+06	2.5E+05	1.8E+04	0	6.0E+03	0
% Removal		40%	88%	99%	100%	100%	100%		52%	95%	100%	100%	100%	100%