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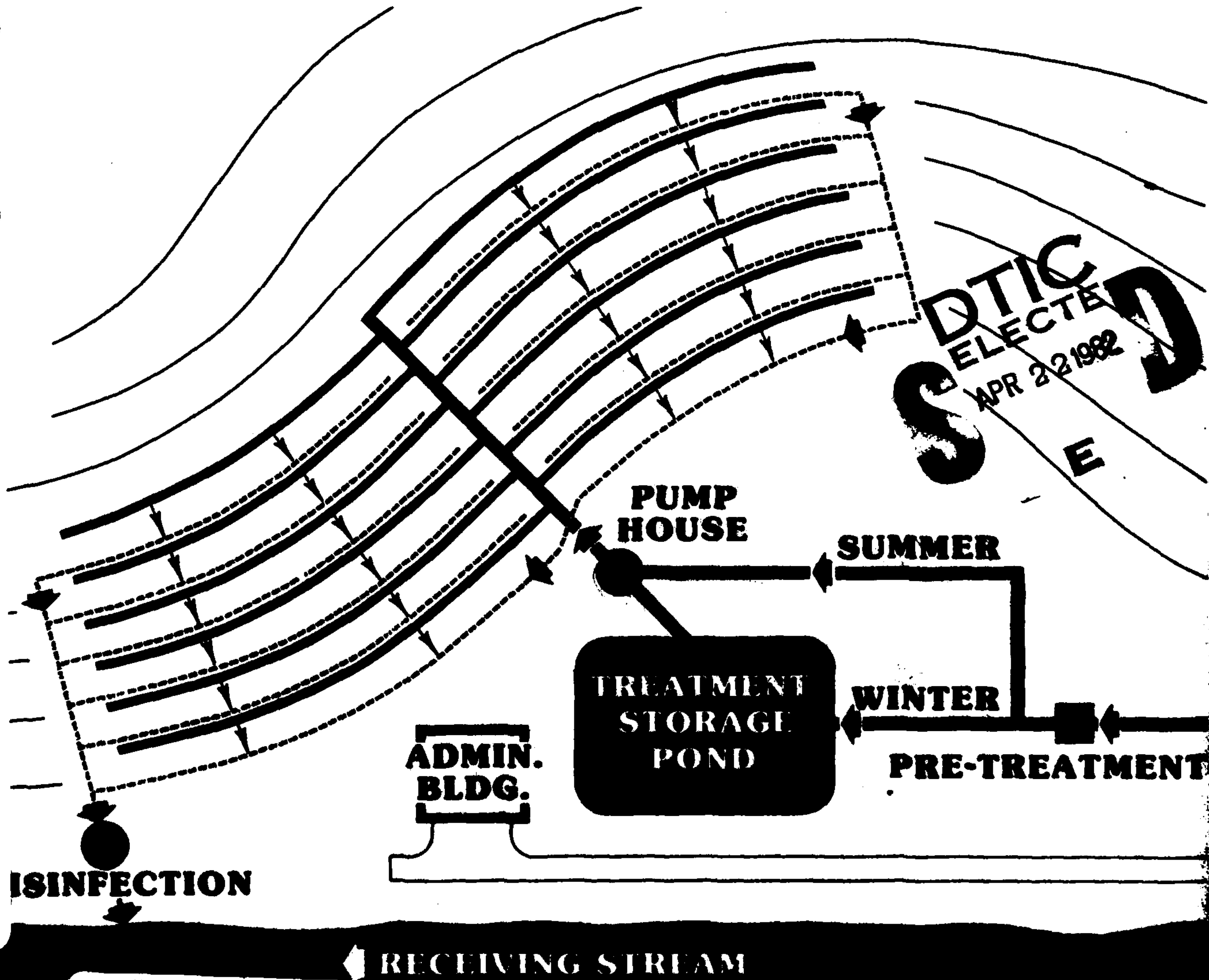
CRREL REPORT 82-2

US Army Corps of Engineers
Cold Regions Research & Engineering Laboratory

Development of a rational design procedure for overland flow systems



For conversion of SI metric units to U.S./British customary units of measurement consult ASTM Standard E380, Metric Practice Guide, published by the American Society for Testing and Materials, 1916 Race St., Philadelphia, Pa. 19103.

OVERLAND FLOW RESEARCH REPORTS

This is one of a series of reports on wastewater treatment by overland flow published by the U.S. Army Corps of Engineers under the Land Treatment of Wastewater Research Program. Other published and available reports on this topic are listed below.

Carlson, C.A., P.G. Hunt, T.B. Delaney (1974) Overland flow treatment of wastewater. *Misc. Paper Y-74-3*, U.S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi.

Chen, R.L. and W.H. Patrick, Jr. (1980) Nitrogen transformations in a simulated overland flow wastewater treatment system. *CRREL Special Report 80-16.*

Hoeppel, R.E., P.G. Hunt, T.B. Delaney (1974) Wastewater treatment on soils of low permeability. *Misc. Paper Y-74-2*, U.S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi.

Jenkins, T.F. et al. (1979) Prototype overland flow test data: June 1977-May 1978. *CRREL Special Report 79-35.*

Jenkins, T.F., D.C. Leggett, C.J. Martel and H.E. Hare (1981) Overland flow: Removal of toxic volatile organics. *CRREL Special Report 81-1.*

Lee, C.R. et al. (1976) Highlights of research on overland flow for advanced treatment of wastewater. *Misc. Paper Y-76-6*, U.S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi.

Martel, C.J., T.F. Jenkins and A.J. Palazzo (1980) Wastewater treatment in cold regions by overland flow. *CRREL Report 80-7.*

Peters, R.E., C.R. Lee and D.L. Bates (1981) Field investigations of overland flow treatment of municipal lagoon effluent. *Technical Report EL-81-9*, U.S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi.

Cover: Possible configuration of terraces and storage pond for hypothetical overland flow system.

CRREL Report 82-2

February 1982

Development of a rational design procedure for overland flow systems

C.J. Martel, T.F. Jenkins, C.J. Diener and P.L. Butler

Prepared for
OFFICE OF THE CHIEF OF ENGINEERS

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This report describes the development of a new design procedure for overland flow systems that is based on hydraulic detention time, a familiar concept in wastewater treatment process design. A two-year study was conducted at Hanover, New Hampshire, on a full-scale overland flow site to obtain performance data in relation to detention time. Kinetic relationships were developed for removal of biochemical oxygen demand, total suspended solids, ammonia, and total phosphorus. Also, an empirical relationship was developed to predict hydraulic detention time as a function of application rate, terrace length, and slope. These relationships were validated using published data from other systems. An example of how to use the new procedure and a comparison with the conventional design approach is included. A major advantage of the new procedure, which should significantly reduce site preparation costs, is that it allows overland flow systems to be designed for a wide range of site conditions as long as detention time requirements are met.

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PREFACE

This report was prepared by C.J. Martel, Environmental Engineer, of the Civil Engineering Research Branch, Experimental Engineering Division, T.F. Jenkins, Research Chemist, of the Earth Sciences Branch, Research Division, and by C.J. Diener, Civil Engineering Technician, and P.J. Butler, Physical Science Technician, of the Civil Engineering Research Branch, Experimental Engineering Division, U.S. Army Cold Regions Research and Engineering Laboratory. Funding for this research was provided by Corps of Engineers Civil Works Project CWIS 31732, *Land Treatment Management and Operation.*

Dr. Yoshisuke Nakano and Sherwood Reed of CRREL technically reviewed the manuscript of this report.

The authors acknowledge the assistance of the following CRREL personnel who contributed their skills to the successful completion of this project: Timothy Pangburn for data organization and analysis, Patricia Schumacher for conducting the water quality analyses and recording data, Anthony Palazzo for advice and assistance in the agronomic aspects of overland flow, and John Graham for assisting in harvesting the grass. The authors also acknowledge the personnel in the CRREL word processing center who patiently typed many drafts, and the editorial staff who greatly improved the readability of this report.

The authors are indebted to Donald Adrian, of the University of Massachusetts, who assisted in developing the hydraulic detention time relationship, Robert Peters and Richard Lee of U.S. Army Engineer Waterways Experiment Station who provided detention time data from the Utica, Mississippi, overland flow site, and Robert Smith of the University of California at Davis who also provided detention time data from his study.

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DEVELOPMENT OF A RATIONAL DESIGN PROCEDURE FOR OVERLAND FLOW SYSTEMS

C.J. Martel, T.F. Jenkins, C.J. Diener and P.L. Butler

INTRODUCTION

Background

An overland flow system consists of a series of grassy terraces which are carefully graded so that wastewater flows downslope in a thin sheet. Gated pipe, slotted troughs, or sprinklers are used to uniformly distribute the wastewater at the top of each terrace, and a ditch or channel collects the runoff at the base of the terrace (Fig. 1). When properly designed and managed, the quality of runoff from an overland flow system can easily meet secondary effluent standards.

Unlike most wastewater treatment systems, overland flow reduces the volume of water to be discharged. This reduction in volume is caused by losses through evapotranspiration and percolation as wastewater flows over the soil surface. Typically, the volume of runoff is 60 to 90% of that applied. Consequently, removal efficiency should be calculated on a mass rather than concentration basis.

Overland flow systems offer a number of advantages over conventional treatment. First and foremost is the lower cost of operation and maintenance. Highly skilled personnel are not needed to run the facility and energy requirements are significantly lower (Middlebrooks and Middlebrooks 1979). Overland flow systems are also very reliable and able to withstand large variations in strength of applied wastewater without system upset (Aly et al. 1979). Another important advantage is that no sludge is produced except by pretreatment processes. Overland flow systems can also provide

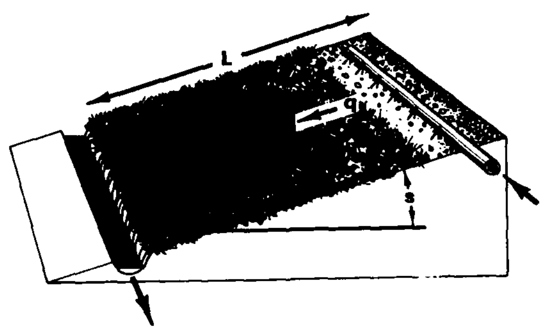

Figure 1. Concept drawing of overland flow system (where L is terrace length, q average overland flow rate and s the slope).

an economical return in the form of high quality forage crops. Palazzo et al. (in prep.) estimated the value of this crop to be \$858 ha^{-1} per year.

In spite of these advantages, not many overland flow systems have been built. One of the main reasons is the lack of a rational procedure for design. The procedure presented in U.S. EPA (1977) is based on general guidelines which are difficult for inexperienced engineers to interpret. For example, when untreated or primary effluent is applied, the designer is advised to select a hydraulic loading rate within a range of 6.4 to 20 cm wk^{-1}, depending on climate, degree of treatment and detention time. Without previous experience it is difficult for the designer to select a valid loading rate within such a wide range. Also, little information is given on how other variables such as terrace length and slope can affect performance. Thus, a more comprehensive and rational design procedure, which takes these factors into account, is needed to assure that discharge requirements are met.

The new design procedure developed in this report is based on reactor kinetics, a concept familiar to most environmental engineers. In the case of overland flow, the reactor is the soil surface where various physical, biological and chemical reactions take place. As in conventional process design, the controlling parameter is detention time. For overland flow, detention time is the average time a unit volume of water takes to travel from the top to the bottom of the terrace. The desired level of treatment can be achieved by controlling the length of time that wastewater remains in contact with the soil surface. With this approach, overland flow systems can be constructed for a wide range of site conditions as long as detention time requirements are met. This would significantly reduce site preparation costs.

Objectives

A design procedure based on detention time requires knowledge of two basic relationships. First is the hydraulic relationship among application rate, site characteristics and detention time. With this relationship the designer can determine the application rate needed to satisfy detention time requirements. Second is the kinetic relationship between detention time and removal of biochemical oxygen demand (BOD), total suspended solids (TSS), ammonia (NH_3-N) and total phosphorus (total P). With these relationships, the designer can determine the detention times needed to achieve the desired removal efficiency.

The specific objectives of this study were to

1. Develop a method which can be used to predict the hydraulic detention time.
2. Determine the removal kinetics for BOD, TSS, NH_3-N and total P.
3. Validate the detention time and kinetic relationships using data from other systems.
4. Provide an example using the new design procedure.

Scope

Data used in the development of the design procedure were obtained from the CRREL overland flow test site in Hanover, New Hampshire, during the 1978 and 1979 growing seasons (April through October). All the kinetic relationships were developed using primary effluent; use of this procedure for design of overland flow systems receiving a secondary or lagoon effluent will be discussed later in the report.

The hydraulic detention time relationship developed at CRREL was validated using data from the Utica, Mississippi, overland flow site (Peters et al. 1981) and the pilot scale system at the University of California, Davis (Smith et al. 1980). Data from several other domestic and foreign overland flow systems were used to validate the kinetic relationships.

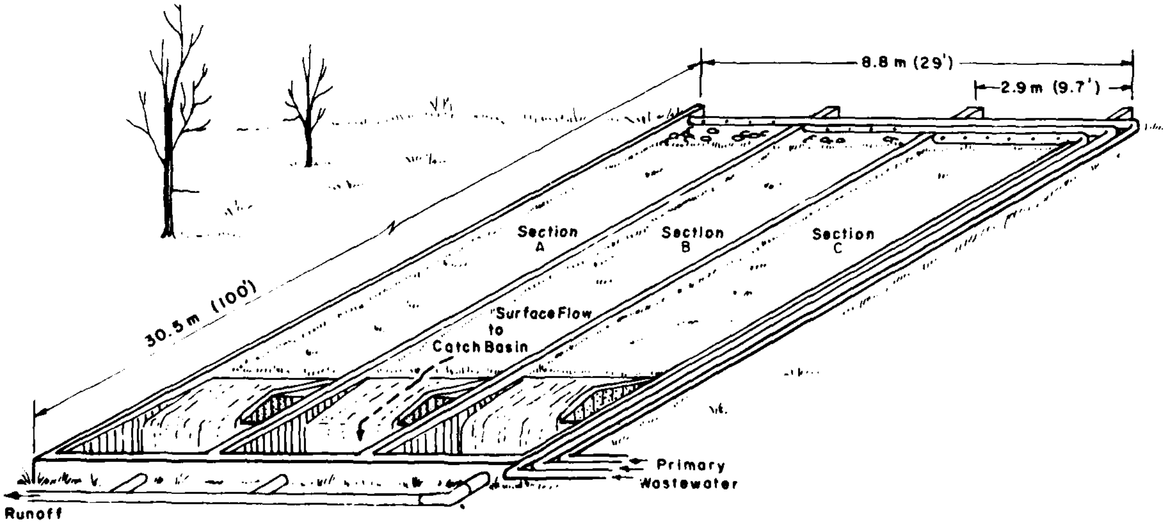

Figure 2. Schematic of CRREL overland flow test site.

DESCRIPTION AND OPERATION OF CRREL OVERLAND FLOW TEST SITE

The CRREL overland flow test site has been in operation since June 1977. This site is 30.5 m long x 8.8 m wide (0.03 ha) and graded to a 5% slope. It is subdivided into three equal sections designated A, B and C so that parallel studies can be conducted. A schematic of the site is shown in Figure 2.

Soil on the site is classified as a Hartland silt loam with sand, silt and clay contents of 5, 72 and 23% respectively. The cation exchange capacity is 5 meq/100 g and the pH is 7.1. Underlying the soil at a depth of 15 cm is a 30.0-mil-thick rubber membrane, which was installed to prevent downward percolation. The grass cover on the site is a mixture of many species including K-31 tall fescue, orchardgrass, Kentucky bluegrass and quackgrass (Palazzo et al. 1980). The grass was harvested on the average of once every six weeks during the growing season.

Undisinfected primary effluent was applied to the overland flow test site during the entire study. Perforated plastic pipe was used to distribute wastewater along the top of each section, and a bed of crushed stone placed beneath the pipe helped to uniformly disperse the flow. The quality of the primary effluent is shown in Table 1.

The application rate to each section was monitored and controlled by means of a constant head weirbox. Five application rates ranging from 0.35 to 1.20 m^3 hr^{-1} were tested. The application cycle was 7 hr on, 17 hr off for 5 days per week. At these application rates and this cycle, the equivalent hydraulic loading rates were 13.8 to 46.7 cm wk^{-1}. Each application rate was evaluated for a period of approximately 6 weeks. All sections were operated simultaneously at the same application rate. Because of leaks in the membrane along the outside boundary, wastewater applications to section A were discontinued during 1979.

Runoff was collected at the base of each section in individual galvanized steel catch basins. A small submersible sump pump located in each basin discharged the runoff into a drainage ditch. The volume of runoff was recorded by flowmeters attached to the discharge lines. During this study, the average runoff rate was 75, 87 and 89% of the application rate for sections A, B and C respectively.

All measurements of detention time and water quality sampling were conducted during periods of hydraulic steady-state operation. The hydraulic steady-state period began when the runoff rate stabilized and it terminated when application was stopped. A typical runoff hydrograph is shown in Figure 3. In this case the hydraulic steady state period began 1 hr after commencing application and terminated 6 hr later when the system was shut down. The amount of time needed to reach hydraulic steady state varied depending on antecedent moisture conditions.

Hydraulic detention time was determined by measuring the travel time of a chloride tracer. Chloride was selected as a tracer because it is conservative and easily analyzed. A tracer solution was made by dissolving 94.6 g of sodium chloride in 3 L of distilled water. The sodium chloride solution was added as a "slug addition" to the distribution chamber

Table 1. Quality of applied primary effluent.

Parameter	*Mean*	*Standard deviation*	*No. of observations*
BOD (mg L^{-1})	72	23	58
Total suspended solids (mg L^{-1})	59	30	98
Total Kjeldahl nitrogen (TKN) (mg L^{-1})	36	10	32
Ammonia (mg L^{-1} as N)	24	6	99
Nitrate (mg L^{-1} as N)	0.05	0.15	98
Total phosphorus (mg L^{-1} as P)	6.6	2.2	33
Orthophosphate (mg L^{-1} as P)	4.8	1.4	31
Turbidity (JTU*)	31	13	100
pH	7.2	0.1	100
Specific conductivity (µmhos/cm)	513	105	40
Chloride (mg L^{-1})	28	6	30
Fecal coliforms (no./100 mL)	2.5×10^6	1.7×10^6	14
Fecal streptococci (no./100 mL)	0.2×10^6	0.2×10^6	7

*Jackson turbidity units

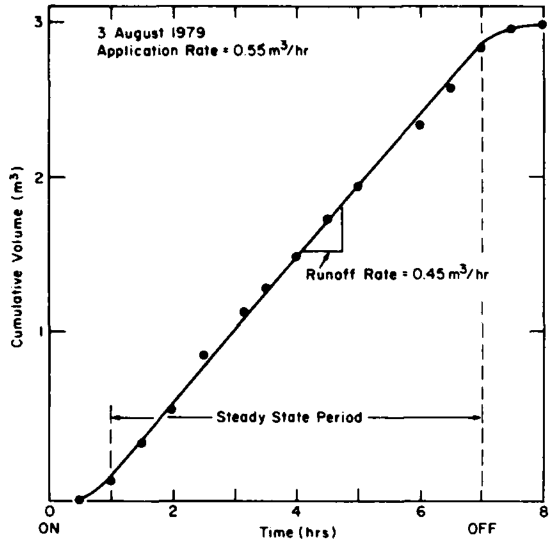

Figure 3. Typical runoff hydrograph.

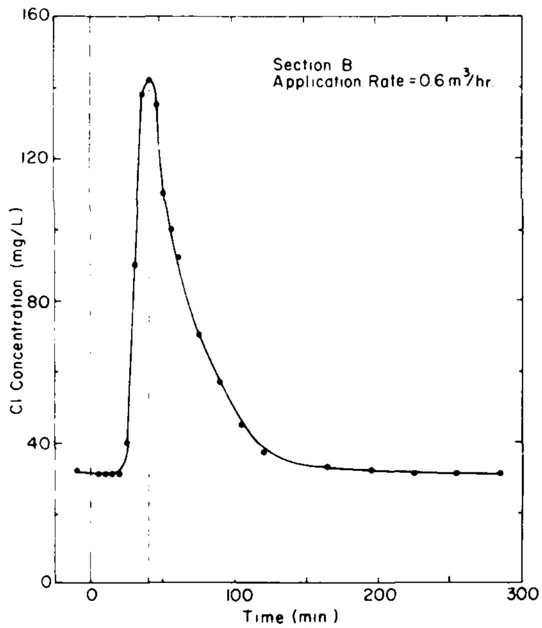

Figure 4. Typical chloride response curve for measuring detention time.

in the constant head weirbox. Composite samples were taken of the runoff at various intervals and analyzed for chloride. Chloride concentrations were then plotted vs time, and the peak of the response curve was chosen to represent detention time. The center of mass was not chosen to represent detention time because 36% of the tracer was lost in the percolate and plants. An example of a typical chloride response curve is shown in Figure 4. The detention time in this case was 40 min for an application rate of 0.6 $m^3\ hr^{-1}$. Altogether, 50 detention times were measured at the CRREL site during this study (see App. A).

Table 2. Parameters measured and frequency of analysis.

Parameter	*Analyses/wk**
BOD	3
TSS	5
TKN	1
NH_3-N	5
NO_3-N	5
Total P	1
PO_4	1
Turbidity	5
pH	5
Specific conductivity	1
Chloride	1
Fecal coliforms	1
Fecal streptococci	1

*Based on five application periods per week.

Flow-proportioned composite samples were taken of the applied primary effluent and runoff during each application period. Primary effluent samples were taken at fixed time intervals with an automatic composite sampler. Runoff samples were taken by small peristaltic pumps which were activated by relay switches during each operating cycle of the catch basin's sump pump. The water quality parameters measured and the frequency of analysis are shown in Table 2. Analtyical techniques are discussed in Appendix B.

HYDRAULIC DETENTION TIME

The hydraulic detention time on an overland flow terrace is dependent on many factors including application rate, slope, length of terrace, surface microtopography, soil infiltration rate, evapotranspiration and vegetation density. Of all these factors, only application rate is controllable by the operator. Slope and length of terrace are largely dependent on site characteristics. Surface microtopography is controllable to some extent by careful site preparation, but each terrace will develop different hydraulic pathways. Infiltration rate and evapotranspiration will vary from site to site depending on soil characteristics and climate conditions. Vegetation density, or surface roughness, varies with the type of vegetation and the maturity of the terrace. A mature terrace usually has a higher vegetation density and will normally have an organic mat near the soil surface caused by a buildup of grass clippings from previous harvests. This organic mat increases the resistance to flow which, in turn, increases detention time.

Theoretical development

At a well designed and operated overland flow site, water flows downslope as a thin sheet until it freefalls into a runoff collection ditch. Under these conditions, the overland flow system operates in the laminar flow regime (Kirkby 1978) where Reynolds numbers are less than 500 (Streeter 1966). At the CRREL site Reynolds numbers ranged between 38 and 226, which is well within the laminar flow regime. For the simplest case of laminar flow over a smooth surface, the average velocity ν_s can be described by the following equation (Nakano 1978):

$$\nu_s = \frac{g\,S\,d^2}{3\,\nu} \qquad (\mathrm{m\ s^{-1}}) \qquad (1)$$

where g = gravitational constant, 9.81 m s^{-2}
S = slope, m m^{-1}
d = average depth of flow, m
ν = kinematic viscosity, $m^2\ s^{-1}$.

For an overland flow system, resistance to flow will be greater because of the grass and vegetative litter. Therefore, the average overland flow velocity V will be lower than the smooth surface velocity ν_s and can be expressed as

$$V = a\,\nu_s \qquad (\mathrm{m\ s^{-1}}) \qquad (2)$$

where a is a resistance coefficient. Substituting eq 2 into 1, the velocity of flow over an overland flow terrace can be calculated by

$$V = a\left[\frac{g\,S\,d^2}{3\,\nu}\right], \text{ where } a < 1.0. \qquad (3)$$

If one assumes that most of the water flows in a relatively straight path downslope, the velocity V can also be expressed as

$$V = \frac{L}{\bar{t}} \qquad (4)$$

where L is the length of terrace in meters, and $\bar{t}$ the hydraulic detention time in seconds.

Also, from the continuity equation, the average depth of flow d can be determined by

$$d = \frac{Q\,\bar{t}}{L\,W} \qquad (5)$$

where Q is the average overland flow rate ($m^3\ s^{-1}$) and W the width of the terrace in meters.

Substituting eq 4 and 5 into eq 3 and rearranging terms, detention time can be calculated as follows:

$$\bar{t} = \left\{\frac{3\,\nu\,W^2}{a\,g\,S\,Q^2}\right\}^{1/3} L. \qquad (6)$$

In more convenient terms with the average detention time described in minutes ($\bar{T}$) and the average overland flow rate (q) in $m^3\ hr^{-1}$ of width, eq 6 becomes

$$\bar{T} = 5.65\left\{\frac{\nu}{ag}\right\}^{1/3} \frac{L}{S^{1/3}\,q^{2/3}}. \qquad (7)$$

Assuming a kinetic viscosity of $0.112 \times 10^{-5}\ m^2\ s^{-1}$ (at 15.6°C) and substituting the value of the gravitational constant g (9.81 m s^{-1}) eq 7 is reduced to

$$\bar{T} = 0.0274\,\frac{L}{a^{1/3}\,S^{1/3}\,q^{2/3}}. \qquad (8)$$

Determination of resistance coefficient, a

To determine a, eq 8 was evaluated using detention time data obtained from the CRREL overland flow test site. For each CRREL test section, the values of L and S are 30.5 and 0.05 m m^{-1} respectively. Substituting these values, eq 8 becomes

$$\bar{T} = \frac{2.27}{a^{1/3}\,q^{2/3}}. \qquad (9)$$

By plotting detention time vs the average overland flow rate on log-log paper, a can be determined from the line of best fit. This was done for the CRREL data shown in Figure 5. A regression analysis indicates good correlation (r = 0.78) between application rate and detention time. However, the standard deviation is large, indicating that detention time varied considerably for a given overland flow rate. For example, at an application rate of 0.2 $m^3\ hr^{-1}\ m^{-1}$ of width, the predicted detention time is 34 minutes. Within one standard deviation, detention times could range from 23 to 48 minutes. Most of this deviation appears to be caused by a difference in results obtained during the 1978 and 1979 growing seasons.

The detention times were generally higher in 1979 than 1978 for the same overland flow rate. A possible explanation for this difference is an increase in vegetation density during 1979 which caused an increase in resistance to flow. This conclusion is supported by the higher grass yields in 1979 than 1978 (Palazzo in prep.). Another reason for the increased detention times could be the presence

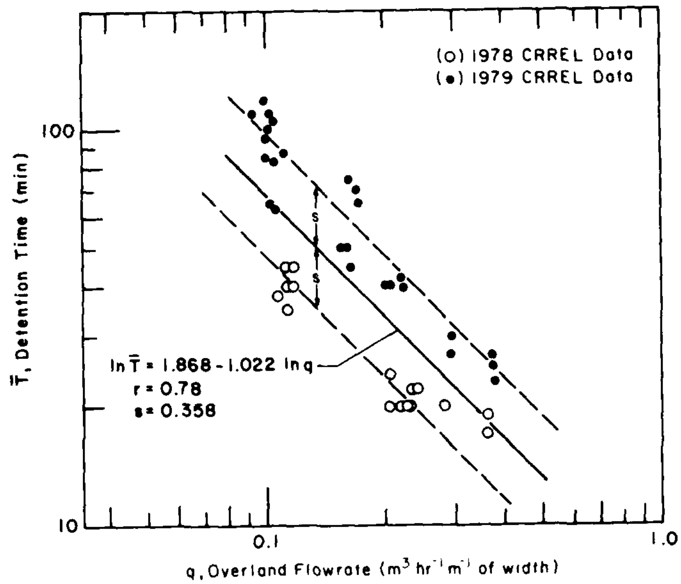

Figure 5. Overland flow rate vs detention time for CRREL overland flow test site.

of a thicker organic mat due to accumulated grass clippings from previous harvests. Tracer studies conducted during the 1980 growing season did not show a further increase in detention time. This observation suggests that the CRREL overland flow terraces reached full maturity or the maximum practical resistance coefficient after three years of operation.

The equation for the line of best fit shown in Figure 5 is

$$\ln \bar{T} = 1.868 - 1.022 \ln q \tag{10}$$

or

$$\bar{T} = \frac{6.48}{q^{1.022}}. \tag{11}$$

Substituting eq 11 for $\bar{T}$ in eq 9, an expression for the resistance coefficient is

$$a = 0.043\, q^{1.066} \approx 0.043\, q. \tag{12}$$

This expression indicates that the resistance coefficient a increases in direct proportion to the average overland flow rate. This relationship can be explained by the fact that, as the flow rate increases, the depth of flow also increases. On the irregular surface of most overland flow terraces, increasing the depth causes more surface area to be wetted, which increases the resistance to flow. This hypothesis is consistent with visual observations at the CRREL site and several other overland flow sites.

Substituting eq 12 back into eq 8, an empirical relationship that can be used for predicting detention time at the CRREL site is

$$\bar{T} = \frac{0.078\, L}{S^{1/3}\, q}. \tag{13}$$

This equation indicates that $\bar{T}$ is directly proportional to L and inversely proportional to q. Slope, being to the one-third power, is less significant although it cannot be considered negligible. For example, assuming L = 50 m and q = 0.2 $m^3\ hr^{-1}\ m^{-1}$, an increase in slope from 2 to 12% would decrease detention time from 72 to 40 minutes, a decrease of 44%.

Validation

To determine the validity of eq 13 for other systems, detention times were measured at two overland flow sites. The first site, located near Utica, Mississippi, was a research facility operated by the U.S. Army Engineer Waterways Experiment Station (WES). This site (no longer in operation) had 24 terraces, each 45 m long x 4.5 m wide and slopes of 2, 4 and 8% (Peters et al. 1981). Detention times times were measured using the same procedure developed at CRREL.

The second site is located indoors at the University of California at Davis. Each laboratory scale model terrace is 6 m long x 1.5 m wide and set at a 4% slope (Smith et al. 1980). Deionized water was used as the tracer and a response curve was developed by measuring specific conductance.

A combined total of 40 detention time measurements were taken at both sites. Measured detention times are shown in Table 3 along with the predicted detention times calculated from eq 13. The average difference between predicted and measured detention times was only 8 minutes. In most cases the measured detention time was longer than predicted, which allows an extra margin of safety in the design. In a Student's t distribution, the difference between measured and predicted detention time was not significant at the 95% level.

Although the average difference between predicted and measured detention time was not significant, individual differences were considerable. This is understandable, considering the variability of the surface microtopography from one terrace to another. Construction techniques, patterns of vegetative growth and harvesting operations are also factors which can change the hydraulic detention time.

KINETICS

Kinetic relationships describing removal of BOD, TSS, NH_3-N and total P were developed by taking several detention time measurements during each application period. The average detention time ($\bar{T}$) was then calculated along with the average percent removal on a mass basis for each constituent. All raw data used in this development can be found in Appendix A.

BOD removal

BOD is removed by sedimentation, filtration and biological oxidation (U.S. EPA 1977). The first two mechanisms are responsible for removing particulate BOD. The soluble BOD is oxidized by microorganisms which are probably similar to the attached biomass found in trickling filters. However, some soluble organic compounds are released from the plant-soil system, and as a result, runoff BOD concentrations below 3 to 5 mg/L cannot be expected (Overcash et al. 1976).

Temperature also has an effect on runoff BOD concentrations. Martel et al. (1980) found that BOD concentrations in the runoff exceeded 30 mg/L at soil temperatures at or below 4°C. However, temperature effects should not be a significant problem at full-scale facilities if wastewater is stored during the winter. In this study, temperature effects were nullified by selecting performance data obtained during the growing season only (April through October).

The experimental data obtained at CRREL and the University of California, Davis (Fig. 6) indicate that BOD removal can be expressed as a first-order equation in the form

$$\text{Percent removal} = (1 - A\, e^{-k\bar{T}})\, 100. \qquad (14)$$

The coefficients A and k, obtained by a least-squares fit to the data, were 0.52 and 0.03 min^{-1} respectively. The coefficient k is the average kinetic rate constant. The coefficient A can be interpreted as the non-settleable fraction of BOD in the applied wastewater while the remaining settleable fraction (0.48) is removed during the first few meters or minutes after wastewater is applied. This conclusion is supported by the BOD vs downslope distance data shown in Figure 7 where 44% of the BOD was removed within the first 5 m.

TSS removal

Total suspended solid (TSS) removal vs average detention time from the CRREL site is shown in Figure 8. The flat slope of the estimated line of best fit indicates that TSS removal changed little over the range of detention times tested. For example, at a detention time of 20 minutes, TSS removal was 86%. A three-fold increase in detention time (60 min) only increased removal by 6%.

The high solids removal efficiency of the overland flow process is due to the shallow depth of water and the long travel distance to the end of the terrace. Even minute particles with slow settling velocities are able to settle out before reaching the collection ditch. Also, grass and vegetative litter help to entrap and filter out particles. Data plotted in Figure 7 indicate that most of the suspended solids were removed within the first 5 m.

Because of rapid settling, a buildup of solids is apparent at the top of most overland flow terraces which receive raw or primary wastewater. At the CRREL site, solids deposition was heavy enough in some spots to smother grass growth. Similar condi-

Table 3. Measured and predicted detention times.

Location and date	Application rate ($m^3\ hr^{-1}$)	Average* overland flow rate ($m^3\ hr^{-1}\ m^{-1}$)	Slope (%)	Measured detention time, $\bar{T}_p$ (min)	Predicted detention time, $\bar{T}_p$ (min)	$\Delta\bar{T}$, $\bar{T}_p - \bar{T}_m$ (min)
Utica, Miss.						
13 July 78	0.584	0.097	4	75	108	33
14 July 78	0.584	0.097	4	60	108	43
21 July 78	0.584	0.097	4	75	108	33
21 July 78	0.584	0.097	4	90	108	18
20 July 78	0.435	0.072	4	130	146	16
2 Aug 78	0.292	0.049	8	255	170	-85
2 Aug 78	0.292	0.049	8	255	170	-85
3 Aug 78	0.435	0.072	8	150	116	-34
3 Aug 78	0.435	0.072	8	165	116	-49
17 Aug 78	0.435	0.072	8	120	116	- 4
17 Aug 78	0.435	0.072	8	135	116	-19
17 Aug 78	0.435	0.072	2	165	184	19
17 Aug 78	0.435	0.072	2	150	184	34
31 Jan 79	0.435	0.072	2	135	157	22
31 Jan 79	0.435	0.072	2	150	157	7
8 Feb 79	0.435	0.072	2	165	157	- 8
25 Jan 79	0.435	0.072	8	120	99	-21
2 Feb 79	0.435	0.072	8	105	99	- 6
2 Feb 79	0.435	0.072	8	165	99	-66
10 Feb 79	0.435	0.072	8	96	99	3
10 Feb 79	0.435	0.072	8	135	99	-36
24 Jan 79	0.584	0.113	4	105	93	-12
15 May 79	1.753	0.292	2	90	45	-45
22 May 79	0.435	0.072	2	330	184	-146
Aug 79	1.753	0.292	4	50	36	-14
Aug 79	0.876	0.146	4	135	72	-63
Aug 79	1.753	0.292	4	65	36	-29
Aug 79	1.753	0.292	8	48	29	-19
11 Feb 80	0.219	0.042	2	207	314	107
12 Feb 80	0.435	0.084	2	112	157	45
12 Feb 80	0.435	0.084	2	120	157	37
12 Feb 80	0.435	0.084	8	90	98	8
12 Feb 80	0.435	0.084	8	75	98	23
13 Feb 80	0.876	0.169	2	75	78	3
13 Feb 80	0.876	0.169	2	75	78	3
13 Feb 80	0.876	0.169	8	60	49	-11
13 Feb 80	0.876	0.169	8	60	49	-11
U. of Calif., Davis						
	0.118	0.079	4	21	17	- 3
	0.236	0.157	4	13	9	- 4
	0.345	0.236	4	11	6	- 5
					n	40
					mean	- 8
					std. dev.	43

*At Utica, 0.75 and 0.87 were used as overland flow coefficients for summer and winter respectively. A runoff coefficient of 1.0 was used at U. of Calif., Davis.

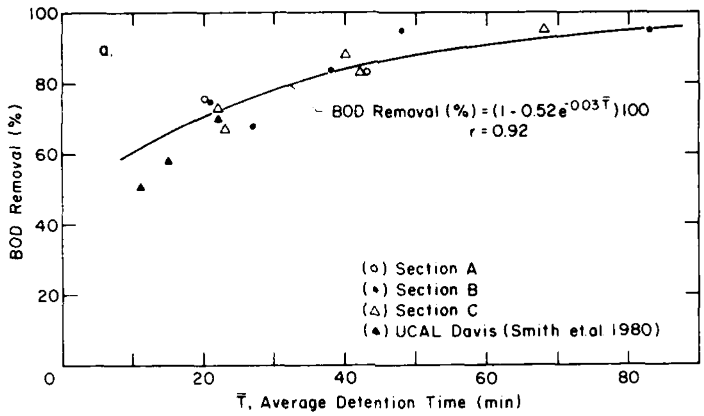

Figure 6. BOD removal efficiency vs detention time.

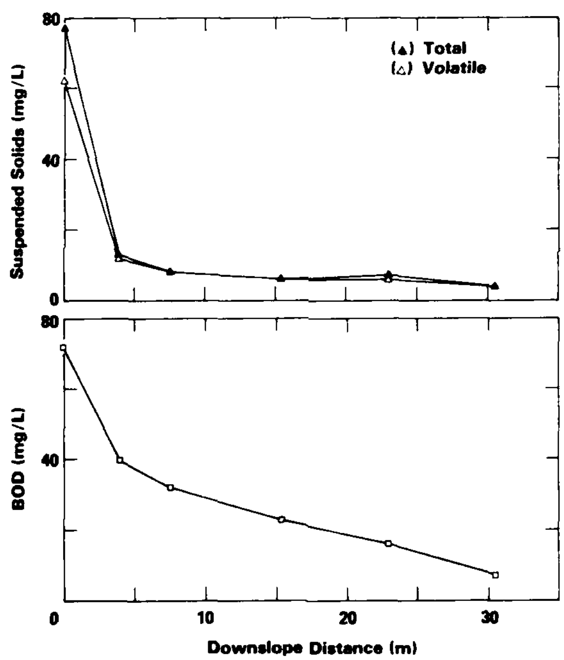

Figure 7. Concentration of BOD and suspended solids vs downslope distance on section C (25 June 1980).

tions were observed during a visit at the Easley, South Carolina, site. After years of application, solids buildup can be substantial and can cause an odor problem if these deposits are allowed to become anaerobic (Scott and Fulton 1979). However, this problem can be avoided by occasionally disking-in the solids and allowing enough reaeration time after each application. Disking should be done on only part of the terraces so that the rest of the system remains in service.

The solids removal relationship developed in this study (Fig. 8) applies to fecal types of solids only. Removal of algal solids found in lagoon effluent is more difficult to predict. Data from the Easley, South Carolina, site indicated that algae removal by overland flow is marginal (Pollock 1979). However,

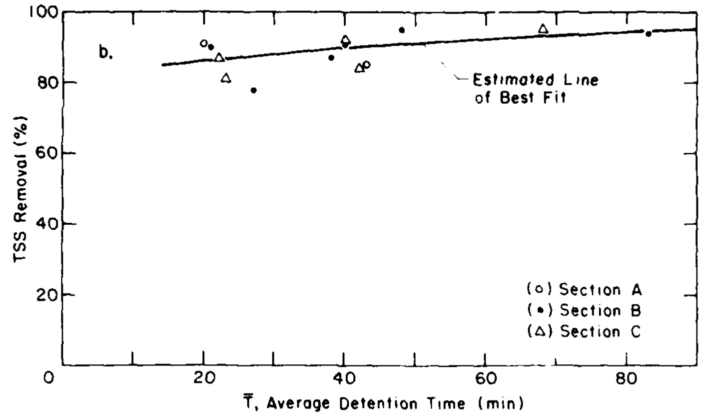

Figure 8. Total suspended solids removal efficiency vs detention time.

Peters et al. (1980) report removal of algae at low application rates.

Nitrogen removal

A number of mechanisms are involved in nitrogen removal, including volatilization, nitrification-denitrification, adsorption, plant uptake and soil storage. The ammonia form of nitrogen can be removed by any of the above mechanisms. Most of the organic nitrogen is initially removed by sedimentation and then incorporated into the soil or converted to ammonia by saprophytic bacteria. Nitrate is the most difficult form of nitrogen to remove (Jenkins et al. 1978, Walter 1974). Nitrate ions have little affinity for soil particles and thus are not retained on the overland flow terrace. For efficient removal of nitrogen, raw wastewater or primary effluent should be applied because these wastewaters contain very little nitrate.

This study focused on the kinetics of ammonia removal because it is the nitrogen form of most concern in discharge limitations. The correlation between ammonia removal and detention time obtained from CRREL data is shown in Figure 9. The first-order equation which closely fits these data (r = 0.91) is also shown in Figure 9. For ammonia removal the coefficients A and k were 0.81 and 0.03 min^{-1} respectively.

It is interesting to note that both BOD and ammonia removal equations (see Fig. 6 and 9) contain the same kinetic rate constant (k = 0.03 min^{-1}), suggesting that both BOD and ammonia removal are controlled

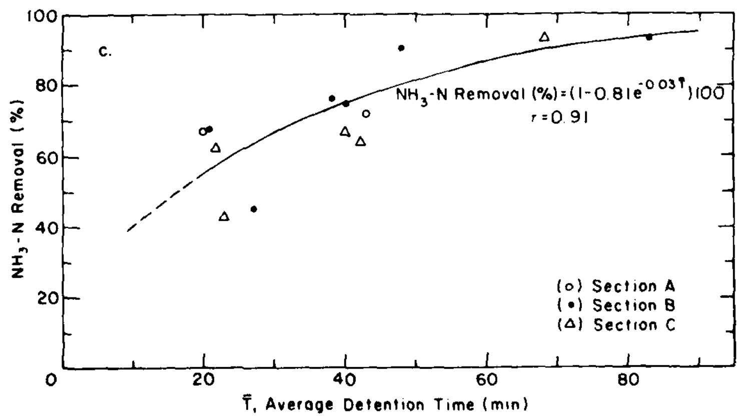

Figure 9. NH_3-N removal efficiency vs detention time.

by the same rate-limiting step. It is unlikely that both substrates would have the same removal rate constant; a more likely explanation is that removal rate is mass transport limited. In other words, the rate of mass transport from the bulk liquid to the active biomass and adsorption sites is the mechanism governing removal rate. This reasoning is reinforced by the fact that overland flow operates in a laminar flow regime, which reduces the opportunities for substrate contact with reactive sites.

Phosphorus removal

According to the *Process Design Manual for Land Treatment of Municipal Wastewater* (U.S. EPA 1977) phosphorus is removed primarily by sorption to soil particles. On overland flow terraces only surface exchange sites are available because most of the wastewater passes over the soil surface rather than through it. As a result, the exchange sites are used up rather quickly and the removal of phosphorus by overland flow systems is limited. Plant uptake is another mechanism capable of removing phosphorus. Palazzo et al. (1980) reported that forage grasses removed 54% of the applied phosphorus at the CRREL site.

As shown in Figure 10, our studies indicated that phosphorus removal did not change significantly over the range of detention times tested. Percentage removals ranged between 37 and 61% and averaged 53%. Analyses of runoff samples indicated that most of the total phosphorus was in the "ortho" form, which indicates that the phosphorus removed was tied up with particulate matter. As discussed earlier (see *TSS Removal*), particulate matter was easily removed by overland flow.

Phosphorus removal can be improved by adding alum to the wastewater prior to application. Thomas et al. (1976) increased phosphorus removal to 90% using this technique. Similar results were obtained at the Utica, Mississippi, overland flow site (Peters et al. 1981).

Validation

The kinetic relationships for removal of BOD, TSS and NH_3-N were validated by comparing the predicted removal to the actual removal reported at seven full-scale systems. Statistical analysis of these data indicated that the average differences between predicted and actual BOD, TSS and NH_3-N removal were only 1.9, -2.0 and 2.8%, respectively, for systems receiving primary or raw wastewater (see Table 4). However, when systems receiving pond effluent were evaluated, the predicted removals for BOD and TSS were 18 and 22% higher than actual (see Table 5). Higher predicted removals can be explained by the fact that pollutants remaining in pond effluent are generally less degradable, and thus more difficult to remove, than those in primary or raw wastewater. Also, there is a lower limit to the BOD and TSS concentration in the runoff. As discussed earlier, this limit is approximately 5.0 mg L^{-1}. Therefore, high removal efficiencies become more difficult to achieve as pollutant concentrations in the applied wastewater decrease.

The ammonia removal relationship (Fig. 9) appears to be valid whether primary or pond effluent is applied. The average differences between predicted and actual NH_3-N removal were only 2.8 and -4.5% for systems receiving primary and pond effluent, respectively.

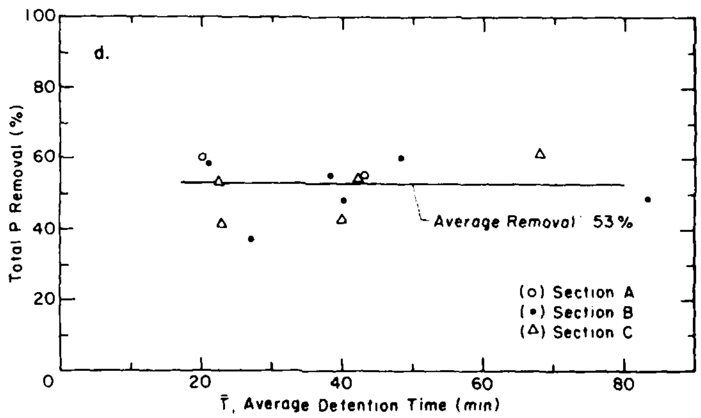

Figure 10. Total phosphorus removal vs detention time.

Table 4. Predicted vs actual removal efficiencies on overland flow systems receiving primary or raw wastewater.

			BOD				TSS				NH_3-N			
System	Calculated detention time (min)	Runoff fraction	Applied conc. (mg/L)	Predicted removal (%)	Actual removal (%)	Predicted - actual removal (%)	Applied conc. (mg/L)	Predicted removal (%)	Actual removal (%)	Predicted - actual removal (%)	Applied conc. (mg/L)	Predicted removal (%)	Actual removal (%)	Predicted - actual removal (%)
Ada, Oklahoma (Thomas et al. 1976)	222	0.50	150	99+	96	3	160	95+	97	-2	17.0	99+	98	1
	195	0.50	150	99+	97	2	160	95+	98	-3	17.0	99+	96	3
	171	0.50	150	99+	97	2	160	95+	98	-3	17.0	99+	97	2
Pauls Valley, Oklahoma (Hall et al. 1977)	294	0.50	117	99+	96	3	105	95+	97	-2	17.0	98	91	7
Werribee Farm, Australia (Scott and Fulton 1979)	626	0.80	507	99+	98	3	233	95+	93	2	31.0	99+	20*	–
Easley, South Carolina (Pollock, 1979)	59	0.70	200	91	91	0	186	92	97	-5	19.4	86	85	1
Paris, Texas (C.W. Thornthwaite, 1979)	138	0.60	480	99	99	0	181	95+	96	-1	–	–	–	–
Mean						1.9				-2.0				2.8
Std. dev.						1.3				2.2				2.5

*Wastewater was applied during the winter when crops were not actively growing.

Table 5. Predicted vs actual removal efficiencies on overland flow systems receiving pond effluent.

System	Calculated detention time (min)	Runoff fraction	BOD Applied conc. (mg/L)	BOD Predicted removal (%)	BOD Actual removal (%)	BOD Predicted - actual removal (%)	TSS Applied conc. (mg/L)	TSS Predicted removal (%)	TSS Actual removal (%)	TSS Predicted - actual removal (%)	NH_3-N Applied conc. (mg/L)	NH_3-N Predicted removal (%)	NH_3-N Actual removal (%)	NH_3-N Predicted - actual removal (%)
Utica, Mississippi	354	0.80	22	99+	95	4	30	95+	93	2	3.5	99	99	0
(Peters et al. 1981)	91	0.80	22	97	74	23	30	95+	71	24	3.5	95	95	0
Easley, South Carolina (Pollock 1979)	56	0.70	28	90	63	27	60	92	53	39	1.0	85	72	13
Carbondale, Illinois	17	0.83	27	69	40	29	22	86	—	—	4.5	51	47	3
(Hinrich et al.	17	0.83	18	69	73	-4	24	86	—	—	8.0	51	68	-17
1980)	17	0.83	70	69	85	-16	35	86	72	14	31.6	51	74	-23
	24	0.83	44	64	68	-4	34	84	51	33	29.6	43	38	5
	24	0.83	9	75	38	37	—	—	—	—	13.6	60	74	-14
	24	0.83	15	75	36	39	26	87	—	—	9.8	60	58	2
	24	0.83	20	75	31	44	24	87	65	22	16.8	60	74	-14
Mean						17.9				22.3				-4.5
Std. dev.						21.1				13.2				11.6

The results of this analysis indicate that the kinetic relationships for BOD and TSS removal are valid for overland flow systems receiving primary or raw wastewater only. The lowest applied concentration of BOD and TSS where these relationships hold is estimated to be 45 mg L^{-1}. Different kinetic relationships need to be developed for overland flow systems receiving pond effluent.

DESIGN PROCEDURE

The primary purpose of the proposed design procedure is to properly size the overland flow system so that it meets the quality requirements of the discharge permit. The three basic steps in this procedure are

1. *Determine the detention times required to* remove pollutants specified in the discharge permit.
2. Calculate the application rate needed to satisfy the longest or most critical detention time.
3. Calculate the land area required from the application rate and system design flow.

Step 1: Determine the detention time

The detention time required to achieve the desired removal of BOD, TSS and NH_3-N can be determined from Figures 6, 8, and 9. The phosphorus removal vs detention time relationship shown in Figure 10 is not used in this procedure because overland flow is not considered to be an efficient process for removing this pollutant. As noted earlier, phosphorus removal can be improved by chemical precipitation with alum.

To use Figures 6, 8, and 9, the percentage removal of BOD, TSS and NH_3-N must be calculated on a mass basis. Information needed for this calculation includes the design flow, an estimate of the applied wastewater concentrations, desired runoff concentrations and the runoff fraction.

Concentrations of BOD, TSS and NH_3-N in the applied wastewater will depend on the degree of pretreatment. In most cases it is advisable to use less pretreatment in order to reduce costs and take advantage of the excellent treatment capabilities of overland flow. If the design includes a storage pond, the diluting effect of this effluent when mixed with primary or raw wastewater should also be considered.

The desired runoff concentrations can be determined from the discharge permit. It should be noted that by satisfying the concentration limits specified in the discharge permit, the mass of pollutant discharged will be smaller than in conventional wastewater treatment systems. As explained earlier, less wastewater is discharged from overland flow systems than is applied.

The fraction of wastewater that reaches the runoff collection ditch can be estimated from local evapotranspiration and percolation rates. Typically, the runoff fraction will range from 0.6 to 0.9 depending on local climatic conditions and soil characteristics.

The detention time used for design should be the longest time determined from Figures 6, 8, and 9. For equal removal percentages, the controlling design parameter is ammonia removal followed by BOD and TSS removal. For example, if 90% ammonia, BOD and TSS removal is required, the detention times needed are 68, 57 and 40 minutes respectively. In this case, the design should be based on ammonia removal since it requires the longest detention time. However, in most cases ammonia removal is not required and BOD removal will be the controlling design parameter.

Step 2: Calculate the application rate

The annual application rate which will satisfy the detention time requirements determined in Step 1 can be calculated as follows:

$$Q_a = \frac{q}{r} Y \qquad (15)$$

where Q_a = annual application rate, $m^3\ yr^{-1}\ m^{-1}$
r = overland flow coefficient
Y = operating time, hr yr^{-1}.

The average overland flow rate q can be calculated by rearranging eq 13 so that

$$q = \frac{0.078\ L}{S^{1/3}\ \overline{T}} \text{ where } q < \frac{0.2}{S^{1/3}}\ . \qquad (16)$$

The values for L and S are selected by the designer based on the topography of the potential site. Natural terrain contours should be used to the maximum extent possible to minimize cut and fill operations. However, careful surface preparation will still be needed to ensure even flow distribution.

The upper limit on q in eq 16 is necessary to avoid a situation where the application rate is so high that it causes scouring. Typically this could occur when a design calls for a short detention time on a long terrace. The limitation placed on q was based on calculations of the scour velocity shown in Appendix C.

The overland flow coefficient r is the average fraction of applied wastewater flowing over the soil surface. The purpose of this coefficient is to convert the average overland flow rate q to an application rate (Q_a). It can be calculated from the relationship

$$r = \frac{1.0 + f}{2} \tag{17}$$

where f is the runoff fraction.

The annual operating time Y will vary depending on the application cycle and season. The application cycle is the number of hours per day and days per week that wastewater is being applied to the terraces. Application cycles normally range from 6 to 8 hours of continuous application per day, 5 to 7 days per week. Obviously, the land area required for the system can be reduced substantially by choosing a longer application cycle. No deleterious effects on performance were noted at Utica, Mississippi (Peters et al. 1981) when pond effluent was applied 24 hours per day, 7 days per week. However, shorter application cycles of 8 to 10 hours per day are recommended if raw wastewater or primary effluent is applied. Shorter application cycles will reduce the rate of solids accumulation at the top of the terrace and allow these solids to degrade more rapidly because of aeration during the off period.

The application season is the number of weeks per year that the system can be expected to operate. In southern areas the application season may extend throughout the year because short periods of cold weather will normally not affect performance (Aly et al. 1979). In northern areas, the application season usually coincides with the growing season. During the non-growing season, wastewater is stored in a pond or lagoon. Martel et al. (1980) found that the EPA-1 computer program provided a good estimate of the number of storage days needed for overland flow systems.

Step 3: Calculate the land area

Since the length of terrace has already been specified, the only remaining dimension needed to calculate the land area is width. Width can easily be determined by dividing the annual volume of wastewater applied by the annual application rate Q_a. If a storage pond is included in the design, the annual volume of wastewater applied should be adjusted to reflect the net volume of water entering or leaving the pond due to precipitation and evaporation.

The land area calculated by this procedure is only part of the total wetted area. Additional wetted area will be needed to handle wastewater flows during harvest operations. Each terrace should be harvested on a rotating basis. The length of the drying period before harvest will depend on local climatic conditions and should be long enough to allow harvesting machinery to drive over the terrace without rutting the surface. In most cases, a week should be adequate. An alternative to increasing the size of the wetted area would be to temporarily increase the application time to the remaining terraces.

Additional land will also be needed for buildings, access roads and buffer zones. Aly et al. (1979) indicated that this additional non-wetted area is usually 25% to 30% of the sum of wetted and non-wetted areas. The total area of the overland flow system should also include the land needed for a storage pond if necessary.

DESIGN EXAMPLE

The previously outlined design procedure can best be explained with an example. In this example, an overland flow facility is being considered for a small town in upstate New York. The design flow is 3785 m^3 day^{-1} (1.0 million gal. day^{-1}). Because of its northern location the facility will have a holding pond to store wastewater during the winter. The holding pond effluent and raw wastewater will be mixed prior to application. The expected quality of this mixture is 150 mg L^{-1} BOD and 100 mg L^{-1} TSS. The discharge limits for this facility are 20 mg L^{-1} BOD and 20 mg L^{-1} TSS. The soils in the area are slowly permeable so that the runoff fraction is expected to be only 0.6. The mass BOD and TSS removals required to meet the discharge limits are

$$\%\ \text{BOD removal} = \frac{(1.0 \times 150 - 0.6 \times 20)\ 100}{1.0 \times 150}$$

$$= 92\%$$

$$\%\ \text{TSS removal} = \frac{(1.0 \times 100 - 0.6 \times 20)\ 100}{1.0 \times 100}$$

$$= 88\%.$$

From Figure 6, the detention time needed to remove 92% of the BOD is 60 minutes. From Figure 8, the detention time needed to remove 88% of the TSS is 40 minutes. Since BOD removal is the limiting parameter, the design is based on a detention time of 60 minutes.

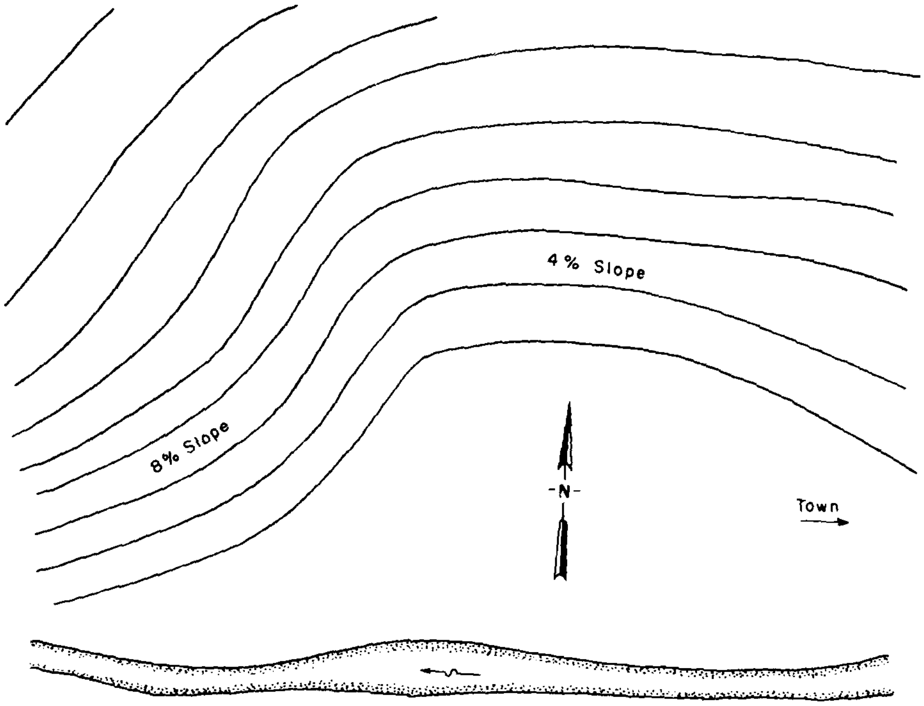

Figure 11. Topographic sketch of proposed overland flow site.

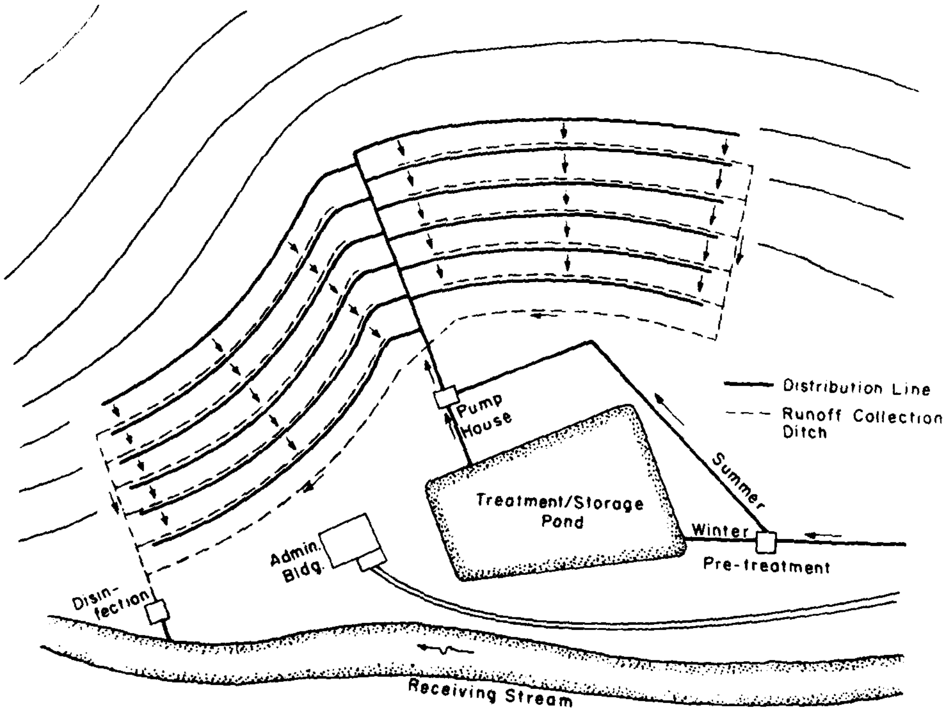

Figure 12. Possible configuration of terraces and storage pond for hypothetical overland flow system.

The proposed site for the overland flow system is on a hillside next to a river. As shown in Figure 11, the eastern half of the site has a general slope of 4% while the slope of the western half is 8%. The length of each terrace is assumed to be 50 m (160 ft). From eq 16,

$$q = \frac{0.078\,(50)}{(0.04)^{1/3}\,60}$$

$$= 0.190 \text{ m}^3 \text{ hr}^{-1} \text{ for the 4\% slope}$$

and

$$q = \frac{0.078\,(50)}{(0.08)^{1/3}\,60}$$

$$= 0.151 \text{ m}^3 \text{ hr}^{-1} \text{ m}^{-1} \text{ for the 8\% slope.}$$

With an application season of 28 weeks per year (April through October) and an application cycle of 10 hours daily for 5 days per week, the annual operating time is 1400 hrs. Also, the runoff coefficient calculated from eq 17 is 0.8. The annual application rate Q_a to each terrace can now be calculated from eq 15:

$$Q_a = \left(\frac{0.190}{0.8}\right) 1400$$

$$= 332.5 \text{ m}^3 \text{ yr}^{-1} \text{ m}^{-1} \text{ for the 4\% slope}$$

$$Q_a = \left(\frac{0.151}{0.8}\right) 1400$$

$$= 264.3 \text{ m}^3 \text{ yr}^{-1} \text{ m}^{-1} \text{ for the 8\% slope.}$$

If the annual design flow is 1.38×10^6 m^3 and the precipitation minus evaporation volume from the holding pond is 0.15×10^6 m^3, the total water volume applied to the overland flow site is 1.53×10^6 m^3 yr^{-1}. If this volume is divided equally, the wastewater applied to each half of the site is 0.77×10^6 m^3 yr^{-1}. With an annual application rate of 332.5 m^3 yr^{-1} m^{-1}, the total width of terrace needed on the 4% slope is 2,316 m. Likewise, the total width of terrace needed on the 8% slope is 2,913 m. If the length of each terrace is 50 m then the combined wetted area needed for treatment is 27 ha (67 acres).

Additional wetted area will be needed to handle wastewater flows during harvest operations. If three harvests per year and a drying period of one week per harvest are planned, the wetted area should be increased by 11% (3 wks/28 wks). Therefore the adjusted wetted area is 30 ha (74 acres). The non-wetted area needed for buildings, access roads and buffer zones would add another 10 ha based on the assumption that the wetted area is 75% of the sum of wetted and non-wetted areas (Aly et al. 1979).

The area needed for a storage pond is estimated to be 20.0 ha. This estimate is based on a storage volume of 667,750 m^3 which includes 567,750 m^3 for storing wastewater and 100,000 m^3 for storing precipitation during winter. The storage needed for wastewater was calculated by multiplying the design flow (3785 m^3 day^{-1}) by the number of storage days (150 days, November thru March). The depth of the storage pond was assumed to be 3.5 m. Summing the wetted, non-wetted and storage pond areas, the total area of the overland flow facility is 60 ha (148 acres). A possible configuration of the terraces for this overland flow system is shown in Figure 12.

COMPARISON WITH THE TRADITIONAL DESIGN APPROACH

As indicated earlier in this report the traditional design procedure is to calculate the wetted area based solely on hydraulic loading rate. For an overland flow system receiving primary or pond effluent, the procedure is to select a hydraulic loading ranging from 6.4 to 20 cm/wk. Using the same design flow and application season as in the previous design example, the required wetted area would range from 27 to 85 ha. Recall that the wetted area predicted by the rational procedure presented in this report was 27 ha, which is the least amount of land predicted by the traditional approach. Therefore a system designed according to the traditional procedure will require more land, especially if the designer selects a conservative loading rate. For example, it would be reasonable to assume that the designer would choose a loading rate in the middle of the range. For a hydraulic loading rate of 15 cm/wk, the land area required would be approximately 40 ha or 50% greater than that required by the rational procedure. In this case the cost of a traditionally designed system would be about 50% higher.

Beyond the monetary benefits just described, the rational procedure presented here is based on the fundamental process design concept of detention time. This concept is familiar to most designers, which makes overland flow more appealing as a treatment alternative. Also, this procedure allows the designer to tailor each site according to existing site conditions and discharge requirements.

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APPENDIX A: CRREL OVERLAND FLOW DATA

Applied and Runoff Water Quality from Section A of CRREL Overland Flow Test Site.[1]

Date	Average overland flow rate, q (m^3/hr m)	Detention time (min)	Steady state volume of wastewater, L Appl.	Steady state volume of wastewater, L Runoff	BOD (mg/L) Appl.	BOD (mg/L) Runoff	TSS (mg/L) Appl.	TSS (mg/L) Runoff	TKN (mg/L) Appl.	TKN (mg/L) Runoff	NH_3-N (mg/L) Appl.	NH_3-N (mg/L) Runoff	NO_3-N (mg/L) Appl.	NO_3-N (mg/L) Runoff
23 Aug 78	0.112	45	992	688	109	31	70	4	43	18	34	14	0.0	4.8
25 Aug	0.111		1860	1348	158	32	28	10	51	21	34	16	0.0	2.0
29 Aug							41	12	44	21	31	16	0.0	3.2
30 Aug	0.109		1302	901			57	15			28	13	0.0	2.3
1 Sept	0.114	40	1560	1079	66	15	51	14		-	36	17	0.0	2.3
5 Sept	0.097		1170	662			75	22			30	9	0.0	4.3
6 Sept	0.114	45	992	659	72	21	66	9	49	16	31	11	0.0	3.4
14 Sept	0.216	20	1020	753	45	14	91	7	-		31	17	0.1	1.0
15 Sept	0.203	20	1260	803	72	24	93	13			31	13	0.0	1.9
19 Sept	0.222		1440	1139			107	9	41	15	28	11	0.0	1.4
20 Sept	0.221		1140	893	42	16	112	5	-	-	34	14	0.0	2.1
21 Sept	0.218	20	1260	954	66	17	71	7			32	14	0.0	1.8
25 Sept	0.195		840	477			64	15	39	22	31	14	0.0	2.5
28 Sept	0.216		1020	754	51	18	52	13			19	14	0.2	1.8
29 Sept	0.216	20	1080	795	62	20	79	16	-		23	7	1.2	1.8
2 Oct	0.202		1050	776			127	22	-		32	10	0.0	1.6
3 Oct	0.201		1260	776			141	19	41	15	34	13	0.0	1.1
4 Oct	0.209	-	960	656	107	39	117	22		-	30	13	0.0	1.1
5 Oct	0.220		3540	2729	83	27	92	8	36	20	25	15	0.2	0.7
12 Oct	0.211		3600	2502	89	33	140	24	47	33	33	17	0.0	1.3
27 Oct	0.359	17	4900	3596	59	28	88	20	-	-	32	22	0.0	0.9
30 Oct	0.369		2400	1889			93	18	44	30	32	25	0.3	1.8

Date	Tot. P (mg/L) Appl.	Tot. P (mg/L) Runoff	PO_4 (mg/L) Appl.	PO_4 (mg/L) Runoff	Turb. (JTU) Appl.	Turb. (JTU) Runoff	pH Appl.	pH Runoff	Spec. cond. (µmhos/cm) Appl.	Spec. cond. (µmhos/cm) Runoff	Cl^- (mg/L) Appl.	Cl^- (mg/L) Runoff	F. coli. (#/100 mL) Appl.	F. coli. (#/100 mL) Runoff	F. strep. (#/100 mL) Appl.	F. strep. (#/100 mL) Runoff
23 Aug 78	7.8	5.5			43	11	7.2	7.1	603	568	36	52				
25 Aug	7.5	5.4	5.4	4.7	13	6	7.1	7.3								
29 Aug	9.0	5.3	6.6	5.1	18	7	7.3	7.4	693	562						
30 Aug					28	6	7.2	7.2	687	355						
1 Sept					29	6	7.5	7.4								
5 Sept					35	10	7.1	6.8	589	484						
6 Sept	10.4	5.5	7.6	5.4	63	6	7.4	7.5	574	532	42					
14 Sept					34	8	7.4	7.3	556	562						
15 Sept	8.7	5.1	6.5	4.3	50	9	7.0	7.0	547	570						
19 Sept	8.0	4.4			49	9	7.5	7.4	565	452						
20 Sept					35	7	7.1	7.2	582	464	34	30				
21 Sept					26	4	7.2	7.3	572	561						
25 Sept	10.5	7.4	9.1	7.0	27	10	7.2	7.0	597	490						
28 Sept		-			18	9	7.1	7.0	436	352						
29 Sept					29	8	7.1	7.2	503	528						
2 Oct					45	10	7.4	7.4	545	439						
3 Oct	9.7	5.9	5.7	4.7	42	8	7.1	7.1	554	432						
4 Oct					36	8	7.1	7.1	518	417	34	30				
5 Oct	5.3	3.2			33	5	7.0	7.1	485	421						
12 Oct	9.1	4.6	7.7	4.3	63	10	7.2	7.2	621	508						
27 Oct					50	12	7.4	7.4	764	721						
30 Oct	6.4	4.7	4.7	3.3	80	40	7.5	7.3	797	741						

[1] Because of leaks in the membrane, application to Section A was discontinued during 1979.

Applied and Runoff Water Quality from Section B of CRREL Overland Flow Test Site.

Date	Average overland flow rate, q (m³/hr m)	Detention time (min)	Steady state volume of wastewater, L Appl.	Steady state volume of wastewater, L Runoff	BOD (mg/L) Appl.	BOD (mg/L) Runoff	TSS (mg/L) Appl.	TSS (mg/L) Runoff	TKN (mg/L) Appl.	TKN (mg/L) Runoff	NH_3-N (mg/L) Appl.	NH_3-N (mg/L) Runoff	NO_3-N (mg/L) Appl.	NO_3-N (mg/L) Runoff	Tot. P (mg/L) Appl.	Tot. P (mg/L) Runoff	PO_4 (mg/L) Appl.	PO_4 (mg/L) Runoff	Turb. (JTU) Appl.	Turb. (JTU) Runoff	pH Appl.	pH Runoff	Spec. cond. (μmhos/cm) Appl.	Spec. cond. (μmhos/cm) Runoff	Cl^- (mg/L) Appl.	Cl^- (mg/L) Runoff	F. coli. (#/100 mL) Appl.	F. coli. (#/100 mL) Runoff	F. strep. (#/100 mL) Appl.	F. strep. (#/100 mL) Runoff
23 Aug 78	0.113	40	961	727			70	4	43	18	34	13	0.0	4.2	7.8	5.5			43	10	7.2	7.1	603	545						
25 Aug	0.107		1860	1253			28	9	51	18	34	14	0.0	2.0	7.5	4.7	5.4	3.5	13	5	7.1	7.3								
29 Aug							41	11	44	20	31	11	0.0	2.3	9.0	4.9	6.6	4.6	18	8	7.3	7.3	693	528						
30 Aug	0.109		1302	901			57	16			28	9	0.0	1.8					28	5	7.2	7.2	687	509						
1 Sept	0.113	35	1488	1139	66	16	51	11			36	11	0.0	2.2					29	6	7.5	7.3								
5 Sept	0.097		1151	670			75	21			30	8	0.0	2.1					35	8	7.1	6.8	589	480						
6 Sept	0.108	38	901	647	72	19	66	12	49	12	31	7	0.0	2.2	10.4	5.7	7.6	4.7	63	6	7.4	7.5	574	552	42					
7 Sept	0.111		360	284	84	11	113	3				5	-	1.0					52	4	7.3	7.3		499						
14 Sept	0.217				45		91				31		0.1						34		7.4		556							
15 Sept	0.206	24	1260	825	72	29	93	14	–		31	11	0.0	1.2	8.7	5.1	6.5	4.3	50	8	7.0	6.8	547	517						
19 Sept	0.224		1440	1162		-	107	9	41	12	28		0.0	1.5	8.0	4.0		-	49	7	7.5	7.4	565	432						
20 Sept	0.234		1140	1010	42	14	112	5			34	10	0.0	1.6					35	6	7.1	7.1	582	439	34	28				
21 Sept	0.231	20	1260	1083	66	15	71	5			32	11	0.0	1.3					26	3	7.2	7.3	572	531						
25 Sept	0.203		840	530	–		64	15	39	20	31	12	0.0	2.1	10.5	7.0	9.1	7.0	27	9	7.2	7.1	597	468						
26 Sept			1080	—	—	—	25	—			22		0.1						23		7.4		478							
28 Sept	0.221	–	1020	795	51	16	52	10			19	5	0.2	2.0		–			18	7	7.1	7.0	436	340						
29 Sept	0.225	20	1080	874	62	—	79				25	9	1.2	0.2					29	8	7.1	7.2	503	514						
2 Oct	0.210		1200	825	—	—	127	19			32	10	0.0	1.1					45	9	7.4	7.3	545	434						
3 Oct	0.208		1260	848	–		141	19	41	16	34	15	0.0	0.5	9.7	5.3	5.7	4.1	42	8	7.1	7.1	554	443						
4 Oct	0.225		960	776	107	37	117	20			30	11	0.0	0.8					36	8	7.1	7.0	518	405	34	29				
10 Oct	0.235		4080	3630	-	—	101	12	52	32	35	25	0.1	0.4	9.3	5.6	7.5		37	9	7.0	7.0	641	569						
13 Oct	0.229	-	3960	3350	83	25	163	17			31	21	0.0	0.5					62	11	7.2	7.5	614	557						
18 Oct	0.295		1800	1623	100	50	142	21	58	39	35	23	0.0	1.7	10.8		6.4	4.7	48	20	7.1	7.1	641	568						
24 Oct	0.266		2850	2029	–	-	103	28	61	30	34	17	0.0	1.7	10.4	6.0	5.3	4.3	35	10	7.0	7.1	636	548						
31 Oct	0.343		5100	3346	–	–	84	9	39	25	30	20	0.2	1.5	6.6	4.2	4.8	2.9	48	8	7.2	7.2	814	736						
22 May 79	0.208		3078	2349	—	—	33	6			19	5	0.2	1.0					26	7	7.2	7.3			23	18				
23 May	0.212		3186	2349	—	—	51				25	7	0.2	1.5					28	9	7.2	7.3			25	20				
31 May	0.206		3730	2897	59	15	49	11			20	2	0.2	0.7					25	6	7.3	7.4			27	16				
1 Jun	0.210		3051	2288	58	17	78	8	26	7	18	5	0.0	1.5	4.4	2.0	3.4	2.3	32	8	7.3	7.2			31	21				
6 Jun*	0.208	30	3260	2488	69	41	56	16	32	20	21	13	0.1	3.0	5.0	6.1			29	9	7.3	7.2	409	500	26					
7 Jun	0.208		3762	2871	62	16	44	7			22	12	0.1	2.7					26	5	7.3	7.3			25	30				
8 Jun	0.206		1546	1201	71	12	36	6			24	11	0.1	4.5					64	9	7.3	7.3								
11 Jun	0.201	40	3528	2593	-	-	54	8	28	15	20	10	0.1	4.2	4.4	3.5	4.6	4.8	57	13	7.1	7.2								
12 Jun	0.208		3819	3046			34	4			15	5	0.1	2.6					28	4	7.2	7.3			20	22				
13 Jun			3248		77	8	53	4			22	9	0.1	2.8					28	4	7.3	7.4								
14 Jun	0.206		3304	2571	89	8	51	5	32	13	25	11	0.0	2.8	7.2	5.6	5.8	5.2	31	4	7.3	7.3								
15 Jun	0.204		3641	2828	84	7	48	4			26	12	0.0	3.0					23	4	7.4	7.2	519	530						
18 Jun	0.193		1972	1491			43	5			25	9	0.0	5.0			4.4	4.6	24	4	7.3	7.1			27	28				
19 Jun	0.199		3234	2410			42	3	31	6	24	6	0.0	3.6	5.6	4.3			42	6	7.3	7.1					2.4×10^4	1.2×10^4		
21 Jun	0.182		3017	1932	114	8	111	5			20	3	0.1	3.3					52	4	7.4	7.4	397	292						
22 Jun	0.196		3186	2375	86	4	85	3			21	4	0.1	2.1					37	3	7.2	7.1								
25 Jun	0.109		1786	1457			49	1	27	10	37	12	0.1	2.9	9.4	5.5			24	2	7.3	7.1								
26 Jun	0.111		1947	1398			39	3			28	6	0.0	3.4			5.3	4.9	24	3	7.3	7.2					2.8×10^4	6.6×10^3		
27 Jun					49	5	35	2			21	3	0.0	3.3					17	2	7.2	7.0								
28 Jun	0.105	65	1873	1225	57	6	36	2			23	2	0.0	2.5					16	2	7.2	6.9								

Applied and Runoff Water Quality from Section B of CRREL Overland Flow Test Site (cont'd).

Date	Average overland flow rate, q (m^3/hr m)	Detention time (min)	Steady state volume of wastewater, l Appl.	Steady state volume of wastewater, l Runoff	BOD (mg/L) Appl.	BOD (mg/L) Runoff	TSS (mg/L) Appl.	TSS (mg/L) Runoff	TKN (mg/L) Appl.	TKN (mg/L) Runoff	NH_3-N (mg/L) Appl.	NH_3-N (mg/L) Runoff	NO_3-N (mg/L) Appl.	NO_3-N (mg/L) Runoff
29 Jun 79	0.110	–	1761	1253			28	2			-	2	1.2	2.2
2 Jul	0.117	–	1878	1646			40	2			18	3	0.1	1.3
3 Jul	0.099	–	2160	1302			40	3			21	2	0.1	1.5
5 Jul	0.097	–	1776	1015			36	4			22	2	0.1	3.5
6 Jul	0.101	-	2118	1342	29	4	19	3			23	2	0.0	3.5
9 Jul	0.102		1254	807			43	4			22	2	0.0	2.0
10 Jul	0.109		1896	1432			29	2			19	1	0.0	1.7
11 Jul	0.098		1812	1073	48	3	31	2			18	0	0.0	1.2
12 Jul	0.108		1800	1331	53	2	45	1			24	3	0.0	1.5
13 Jul	0.108	-	1932	1410	55	3	39	2	28	3	21	2	0.0	1.7
16 Jul	0.107		1566	1135			49	8			16	2	0.0	2.4
17 Jul	0.108		1794	1318			44	4			17	0	0.0	1.3
18 Jul	0.095		1728		55		63		20		16		0.0	
19 Jul	0.105		1800	1218	51	4	45	4			17	1	0.0	1.5
20 Jul	0.101	100	1800	1138			28	3			14	0	0.0	3.1
27 Jul*	0.105	–	1290	882	36	26	43	10	27	9	16	3	0.0	9.4
30 Jul	0.095		1254	669			37	6	22	8	15	2	0.2	9.5
31 Jul	0.093		1806				33				15		0.2	
1 Aug	0.100	85	1806	1124	40	4	61	3			16	1	0.1	3.0
2 Aug	0.174		2619	2266	39	4	26	3			16	1	0.0	2.7
3 Aug	0.173		2760	2245	47	4	23	2			15	1	0.2	3.3
6 Aug	0.160		2309	1589			29	4			17	1	0.1	3.5
7 Aug	0.160	50	2601	1884			36	4	31	5	23	2	0.0	3.3
8 Aug	0.154		1593	1045	88	5	82	3			20	1	0.0	2.8
9 Aug	0.144		2970		48		36				18	–	0.0	
13 Aug	0.134		3054				77				23		0.0	
14 Aug	0.166		3168	2596			51	2			16	1	0.0	1.8
15 Aug	0.164		2700	2027	64	2	74	2	21	3	15	1	0.0	2.0
16 Aug	1.162		2160	1609	61	3	50	2			14	1	0.0	2.0
17 Aug	0.169		3312	2556	85	4	60	3			25	3	0.0	5.0
20 Aug	0.174		3456	2582			128	2			25	5	0.0	5.1
21 Aug	0.176		3437	2651			43	3			21	5	0.0	1.7
22 Aug	0.177		2208	1720	76	7	47	4			20	4	0.0	1.7
23 Aug	0.179		3456	2766	61	5	46	3	38	9	21	5	0.0	2.2
24 Aug	0.180		3384	2878	72	6	44	3			23	6	0.0	1.8
27 Aug	0.176		3475	2685			31	3			21	4	0.0	2.7
28 Aug	0.159		3033	2149			57	2			21	3	0.0	2.3
29 Aug	0.162	45	3133	2463			39	2			21	3	0.0	2.4
30 Aug	0.166		1890	1463	89	4	37	2			21	3	0.0	2.8
31 Aug	0.155		3106	2185	54	3	31	2	21	4	17	2	0.0	3.8
4 Sept	0.149		1871	1222			36	4			24	2	0.0	5.2
5 Sept	0.155		2838	2104	71	4			29	4	23	2	0.0	4.1
7 Sept	0.154		3024	2333	86	3	107	1			18	1	0.0	2.7
10 Sept	0.153		2179	1609			41	6			21	3	0.0	6.4

Date	Tot. P (mg/L) Appl.	Tot. P (mg/L) Runoff	PO_4 (mg/L) Appl.	PO_4 (mg/L) Runoff	Turb. (JTU) Appl.	Turb. (JTU) Runoff	pH Appl.	pH Runoff	Spec. cond. (μmhos/cm) Appl.	Spec. cond. (μmhos/cm) Runoff	Cl^- (mg/L) Appl.	Cl^- (mg/L) Runoff	F. col. (#/100 mL) Appl.	F. col. (#/100 mL) Runoff	F. strep. (#/100 mL) Appl.	F. strep. (#/100 mL) Runoff
29 Jun 79			3.6	3.6	12	2	7.2	7.0	375	296	26	28				
2 Jul					26	5	7.2	7.2					1.2×10^6	6.5×10^3		
3 Jul					24	3	7.3	7.2								
5 Jul			2.7	2.7	19	4	7.1	6.9	397	267						
6 Jul					12	4	7.3	7.0								
9 Jul		–			28	1	7.2	6.9								
10 Jul					17	3	7.2	7.0					8.3×10^5	5.1×10^3		
11 Jul					24	3	7.2	7.1								
12 Jul					25	2	7.4	7.1	386	256						
13 Jul	4.0	2.5	3.7	2.7	22	2	7.4	7.2			23	15				
16 Jul					24	6	7.1	7.0								
17 Jul					23	4	7.2	7.2					2.7×10^6	1.8×10^4		
18 Jul	5.8		3.9		28		7.1		326		22	-	–	–		
19 Jul					24	3	7.2	7.2					–			
20 Jul					18	3	7.3	7.1								
27 Jul*	4.4	5.5	3.0	4.8	23	8	7.2	6.9	562	538	91	82				
30 Jul	4.1	4.5			25	5	7.1	6.9	569	630	90	103		–		
31 Jul					17		7.1						3.3×10^6			
1 Aug					22	2	7.2	7.2					–	–	–	
2 Aug					15	3	7.2	7.2								
3 Aug					17	2	7.3	7.2						–		
6 Aug					21	6	7.3	7.0			20	21				
7 Aug	4.4	2.6	2.7	1.9	24	4	7.4	7.0	378	301	24	32	1.2×10^6	5.3×10^4		
8 Aug					46	3	7.3	7.0				-				
9 Aug					20		7.3									
13 Aug					39		7.4									
14 Aug					26	2	6.9	7.0					3.8×10^6	1.7×10^4	3.3×10^4	1.6×10^3
15 Aug	3.9	2.2	2.9	2.5	28	2	7.1	7.0	303	225	20	17				–
16 Aug					27	2	7.3	7.0								
17 Aug					30	3	7.4	7.1			29	25				
20 Aug					37	6	7.4	7.2			31	23				
21 Aug					22	3	7.3	7.2			28	23	7.3×10^6	2.2×10^5	2.4×10^5	2.8×10^4
22 Aug					30	3	7.4	7.1								
23 Aug	4.5	2.5	3.7	2.7	22	3	7.3	7.1			28	26				
24 Aug					23	3	7.2	6.9								
27 Aug					26	6	7.1	6.9								
28 Aug					23	3	7.2	7.0					1.8×10^6	1.9×10^5	4.0×10^4	9.0×10^3
29 Aug					25	3	7.2	6.9								
30 Aug					32	3	7.2	7.0								
31 Aug	3.8	2.3	3.3	2.4	19	3	7.3	7.0			24	23				
4 Sept					39	5	7.2	6.9	–				2.3×10^7	6.0×10^5	1×10^4	5.3×10^4
5 Sept	5.3	2.1	4.6	2.4	25	3	7.2	6.9	399	261	24	20				
7 Sept					35	2	7.2	7.0								
10 Sept					27	6	7.1	6.8								

* First application day after harvest.

Applied and Runoff Water Quality from Section B of CRREL Overland Flow Test Site (cont'd).

Date	Average overland flow rate, q (m^3/hr m)	Detention time (min)	Steady state volume of wastewater, L		BOD (mg/L)		TSS (mg/L)		TKN (mg/L)		NH_3-N (mg/L)		NO_3-N (mg/L)		Tot. P (mg/L)		PO_4 (mg/L)		Turb. (JTU)		pH		Spec. cond. (μmhos/cm)		Cl^- (mg/L)		F. col. (#/100 mL)		F. strep (#/100 mL)	
			Appl.	Runoff	Appl.	Runoff	Appl.	Runoff	Appl.	Runoff	Appl.	Runoff	Appl.	Runoff	Appl.	Runoff	Appl.	Runoff	Appl.	Runoff	Appl.	Runoff	Appl.	Runoff	Appl.	Runoff	Appl.	Runoff	Appl.	Runoff
12 Sept 79	0.371	25	5779	4984	75	25	43	10	35	19	29	16	0.0	2.0	5.9	3.9	5.6	4.2	25	63	7.4	7.1	470	400	27	31	–			–
17 Sept	0.094	110	1218	697	–	–	–	–	–	–	–	–	–													–		–	–	–
18 Sept	0.155	50	2518	2080	–	–	–	–	–	–	–	–	–															–	–	
19 Sept	0.223	40	3599	2959	–	–	–	–	–	–	–		–		–															
20 Sept	0.295	27	–	–	–	–	–	–	–	–		–	–																	
24 Sept	0.372	27	7227	5889	–	–	71	5	34	17	26	13	0.0	1.4	5.8	3.6			42	12	6.9	7.0	438	400	28	35				
26 Sept	0.374	–	7346	6056	78	29	38	14	–	–	24	17	0.1	0.6	–	–		–	29	12	7.1	7.0								
1 Oct	0.373	–	7049	5780	–	–	54	13	29	13	29	13	0.0	1.7				–	35	13	7.0	7.2								
4 Oct	0.385	–	6950	6117	60	13	34	4	28	13	22	11	0.0	0.8	3.8	3.1	3.3	2.9	24	8	7.3	7.2			29	25				
9 Oct	0.387	–	6633	5884	–	–	54	24	39	29	32	19	0.0	0.7	7.7	6.2	6.9	5.3	36	18	7.2	7.2	545	480	38	34	3.7×10^6	1.8×10^6	1.7×10^5	1.1×10^5
11 Oct	0.381	–	7029	6021	91	40	56	10		–	28	24	0.0	1.0	–	–	–	–	32	15	7.3	7.3						–		
16 Oct*	0.377	–	7227	6072	–	–	83	25	–	–	37	25	0.0	0.3	–	–	–		45	26	7.3	7.2		–			3.5×10^6	1.8×10^6	9.6×10^4	1.2×10^5
18 Oct	0.380	–	6970	5946	128	58	71	11	43	27	35	21	0.1	3.9	7.4	5.0	5.7	4.8	37	18	7.2	7.1	545	529	35	36	–	–		
22 Oct	0.364	–	6282	6003	–	–	56	24		–	35	30	0.0	0.2	–	–	–	–	38	20	7.3	7.4	567	568						
23 Oct	0.306	–	4928	4178	–	–	65	5	39	25	28	21	0.0	0.7	6.2	5.1	5.2	5.1	35	14	7.2	7.3	452	429	28	27	2.4×10^6	1.9×10^6	6.1×10^5	2.7×10^5
24 Oct	0.237	–	794	677	–	–	161	39	–	–	29	15	0.0	2.0	–	–	–	–	92	35	7.0	7.5						–		
29 Oct	0.117	–	1572	1382	–	–	88	5	46	22	33	17	0.0	7.8	8.4	5.0	5.6	4.5	33	7	7.2	7.1	555	495	35	36	2.5×10^6	3.4×10^5	3.0×10^5	8.7×10^4
31 Oct	0.114	–	1758	1461	114	28	94	6	–	–	33	17	0.0	6.7	–	–	–	–	42	12	7.4	7.3					–	–		
2 Nov	0.114	–	1290	1066	101	16	67	7	–	–	30	15	0.1	5.5	–	–	–	–	37	8	7.2	7.1								–

* First application day after harvest.

Applied and Runoff Water Quality from Section C of CRREL Overland Flow Test Site.

Date	Average overland flow rate, q (m^3/hr m)	Detention time (min)	Steady state volume of wastewater, L Appl.	Steady state volume of wastewater, L Runoff	BOD (mg/L) Appl.	BOD (mg/L) Runoff	TSS (mg/L) Appl.	TSS (mg/L) Runoff	TKN (mg/L) Appl.	TKN (mg/L) Runoff	NH_3-N (mg/L) Appl.	NH_3-N (mg/L) Runoff	NO_3-N (mg/L) Appl.	NO_3-N (mg/L) Runoff	Tot. P (mg/L) Appl.	Tot. P (mg/L) Runoff	PO_4 (mg/L) Appl.	PO_4 (mg/L) Runoff	Turb. (JTU) Appl.	Turb. (JTU) Runoff	pH Appl.	pH Runoff	Spec. cond. (µmhos/cm) Appl.	Spec. cond. (µmhos/cm) Runoff	Cl^- (mg/L) Appl.	Cl^- (mg/L) Runoff	F. col. (#/100 mL) Appl.	F. col. (#/100 mL) Runoff	F. strep. (#/100 mL) Appl.	F. strep. (#/100 mL) Runoff
23 Aug 78	0.119	45	961	822	109	—	70	4	43	22	34	15	0.0	2.1	7.8	4.8	—		43	9	7.2	7.2	603	548	36	44				
25 Aug	0.112	—	1860	1378	—	—	28	11	51	22	34	17	0.0	1.1	7.5	4.3	5.4	3.6	13	7	7.1	7.4							—	
29 Aug	—		—	—		—	41	10	44	23	31	17	0.0	1.3	9.0	5.3	6.6	4.7	18	6	7.3	7.5	693	578				—		
30 Aug	0.114		1302	1011	—	—	57	16	—		28	16	0.0	1.0					28	7	7.2	7.4	687	603						
1 Sept	0.117	40	1488	1219	66	17	51	13		—	36	19	0.0	2.8					29	8	7.5	7.6								
5 Sept	0.100	—	1170	723			75	20		—	30	13	0.0	1.8					35	8	7.1	7.0	589	507						
6 Sept	0.113	40	901	734	72	20	66	14	49	18	31	10	0.0	1.0	10.4	5.4	7.6	4.8	63	9	7.4	7.5	574	604	42					
7 Sept	0.109	—	360	273	84	9	113	5				9		0.9					52	4	7.3	7.5		507						
14 Sept	0.224	22	1020	818	45	15	91	9	—	—	31	15	0.1	0.3					34	7	7.4	7.2	556	580						
15 Sept	0.214	20	1260	908	72	20	93	15	—		31	13	0.0	1.0	8.7	5.5	6.5	4.4	50	9	7.0	6.9	547	551						
19 Sept	0.247	—	1440	1427	—	—	107	8	41	15	28	10	0.0	0.8	8.0	4.1		—	49	8	7.5	7.5	565	450						
20 Sept	0.238	—	1140	1028	42	16	112	6	—	—	34	14	0.0	1.1				—	35	7	7.1	7.1	582	458	34	29				—
21 Sept	0.238	22	1260	1154	66	17	71	6	—	—	32	15	0.0	0.7					26	4	7.2	7.3	572	558	—				—	
25 Sept	0.215	—	840	617	—	—	64	20	39	23	31	16	0.0	1.2	10.5	7.4	9.1	6.9	27	11	7.2	7.2	597	486						
26 Sept	—	—	1080	—	—	—	25	—	—	—	22		0.1						23	—	7.4		478	—						
28 Sept	0.230	—	1020	871	51	15	52	11	—	—	19	7	0.2	1.2	—				18	7	7.1	7.0	436	347						
29 Sept	0.236	22	1080	973	62	20	79	16	—		25	12	1.2	0.8				—	29	8	7.1	7.2	503	522						
2 Oct	0.223	—	1200	950	—	—	127	21	—	—	32	14	0.0	0.9			—	—	45	10	7.4	7.4	545	440	—					
3 Oct	0.211	—	1260	882	—	—	141	19	41	18	34	16	0.0	0.4	9.7	4.9	5.7	4.3	42	9	7.1	7.1	554	448	—					
4 Oct	0.225	—	960	778	107	39	117	19	—	—	30	13	0.0	0.6	—	—	—	—	36	8	7.1	7.1	518	393	34	29				
11 Oct	0.232	—	3720	3218	89	26	100	20	47	26	31	19	0.0	0.7	8.3	4.8	4.8	5.7	61	13	7.3	7.1	566	503						
19 Oct	0.279	20	3300	2623	115	53	61	15	58	37	34	24	0.0	0.6	9.1	5.5	4.8	4.3	48	20	7.1	7.0	632	632					—	
25 Oct	0.281	—	1050	852	92	35	130	31	53	36	36	20	0.0	1.7	11.3	6.0	4.9	6.7	40	13	7.1	7.3	619	536		—				
2 Nov	0.365	19	3000	2283	86	25	97	9	39	28	28	18	0.0	1.7	6.7	3.3	4.5	2.4	43	8	7.3	7.4	777	769						
22 May 79	0.198	—	2916	2241		—	33	4	—		19	6	0.2	0.7				—	26	7	7.2	7.3	—	—	23	17			—	
23 May	—	—	3186	2349	—	—	51	—	—	—	25	7	0.2	1.5			—	2.8	28	9	7.2	7.3	—	—						
31 May	0.194	—	3463	2764	59	11	49	6	—	—	20	4	0.2	1.0		—		—	25	5	7.3	7.3	—							
1 Jun	0.200	—	2893	2183	58	15	78	7	26	7	18	4	0.0	1.3	4.4	2.0	3.4	1.8	32	7	7.3	7.3	—	—						
6 Jun*	0.203	40	3146	2466	69	40	56	14	21	13	32	19	0.1	2.0	5.0	5.2	—	—	29	8	7.3	7.3	409	479	26	—				
7 Jun	0.203	—	3630	2838	62	15	44	6	—	—	22	13	0.1	0.1	—		—	—	26	4	7.3	7.3	—	—	25	28				
8 Jun	0.203	—	1518	1187	71	11	36	6	—	—	24	12	0.1	3.2	—			—	64	10	7.3	7.4								
11 Jun	0.204	40	3402	2791	—	—	54	8	28	15	20	11	0.1	3.1	4.4	3.6	4.6	4.3	57	14	7.1	7.3			—					
12 Jun	0.215	—	3819	3279	—	—	34	3	—	—	15	7	0.1	1.3	—		—	—	28	4	7.2	7.3	—		20	22				
13 Jun	—	—	3132	—	77	8	53	5	—	—	22	12	0.1	2.0				—	28	4	7.3	7.3								
14 Jun	0.209	—	3245	2714	89	12	51	4	32	16	25	14	0.0	2.0	7.2	4.9	5.8	5.3	31	4	7.3	7.3								
15 Jun	0.200	—	3477	2860	84	7	48	3	—	—	26	14	0.0	2.5					23	4	7.4	7.5	519	516						
18 Jun	0.205	—	2009	1669	—	—	43	4	—		25	10	0.0	4.2	—		4.4	4.2	24	4	7.3	7.1			27	26				
19 Jun	0.206	—	3234	2618	—	—	42	3	31	14	24	10	0.0	2.9	5.6	5.0			42	7	7.3	7.1					2.4×10^6	4.6×10^3		
21 Jun	0.200	—	3074	2381	114	7	111	5	—	—	20	7	0.1	2.5					52	5	7.4	7.5	397	317						
22 Jun	0.205	—	3186	2657	86	8	85	5	—	—	21	8	0.1	1.5					37	3	7.2	7.1								
26 Jun	0.112	—	1915	1482	—	—	39	2	—	—	28	6	0.0	2.8			5.3	3.6	24	2	7.3	7.2					2.8×10^6	7.3×10^3		
27 Jun		—	—	—	49	4	35	2	—		21	3	0.0	2.9					17	2	7.2	7.0								
28 Jun	0.107	63	1842	1335	57	5	36	2	—		23	3	0.0	2.8					16	2	7.2	7.0								
29 Jun	0.113		1761	1334	—		28	2				3	1.2	2.2			3.6	3.0	12	2	7.2	7.0	375	335	26	28				

Applied and Runoff Water Quality from Section C of CRREL Overland Flow Test Site (cont'd).

Date	Average overland flow rate, q ($m^3/hr\ m$)	Detention time (min.)	Steady state volume of wastewater, L Appl.	Runoff	BOD (mg/L) Appl.	Runoff	TSS (mg/L) Appl.	Runoff	TKN (mg/L) Appl.	Runoff	NH_3-N (mg/L) Appl.	Runoff	NO_3-N (mg/L) Appl.	Runoff	Tot. P (mg/L) Appl.	Runoff	PO_4 (mg/L) Appl.	Runoff	Turb. (JTU) Appl.	Runoff	pH Appl.	Runoff	Spec. cond. (μmhos/cm) Appl.	Runoff	Cl^- (mg/L) Appl.	Runoff	F. col. (#/100 mL) Appl.	Runoff	F. strep. (#/100 mL) Appl.	Runoff
2 Jul 79	0.114	–	1878	1576	–	–	40	6	–	–	18	3	0.1	1.1	–	–	–	–	26	6	7.2	7.3	–	–	–	–	1.2×10^6	6.3×10^2	–	–
3 Jul	0.099	–	2160	1297	–	–	40	4	–	–	21	3	0.1	1.1	–	–	–	–	24	3	7.3	7.2	–	–	–	–	–	–	–	–
5 Jul	0.096	–	1776	975	–	–	36	6	–	–	22	2	0.1	3.4	–	–	2.7	2.5	19	4	7.1	7.0	397	269	27	22	–	–	–	–
6 Jul	0.099	–	2118	1258	29	6	19	3	–	–	23	2	0.0	3.2	–	–	–	–	12	4	7.3	6.9	–	–	–	–	–	–	–	–
9 Jul	0.099	–	1254	756	–	–	43	3	–	–	22	1	0.0	2.1	–	–	–	–	28	4	7.2	6.9	–	–	–	–	–	–	–	–
10 Jul	0.106	–	1896	1323	–	–	29	2	–	–	19	0	0.0	1.2	–	–	–	–	17	3	7.2	7.0	–	–	–	–	$.83\times10^6$	5.6×10^3	–	–
11 Jul	0.098	–	1812	1049	48	4	31	2	–	–	18	0	0.0	0.7	–	–	–	–	24	3	7.2	7.1	–	–	–	–	–	–	–	–
12 Jul	0.107	–	1800	1284	53	2	45	1	–	–	24	0	0.0	1.3	–	–	–	–	25	3	7.4	7.1	386	255	–	–	–	–	–	–
13 Jul	0.105	–	1932	1313	55	3	39	2	28	3	21	1	0.0	1.6	4.0	1.8	3.7	2.1	22	3	7.4	7.2	–	–	23	17	–	–	–	–
16 Jul	0.107	–	1514	1166	–	–	49	5	–	–	16	0	0.0	1.4	–	–	–	–	24	5	7.1	7.0	–	–	–	–	–	–	–	–
17 Jul	0.108	–	1734	1386	–	–	44	2	–	–	17	0	0.0	1.6	–	–	–	–	23	3	7.2	7.1	–	–	–	–	2.7×10^6	7.0×10^3	–	–
18 Jul	0.108	–	1728	1253	55	2	63	2	20	2	16	0	0.0	2.5	5.8	2.0	3.9	2.1	28	3	7.1	7.1	326	226	22	14	–	–	–	–
19 Jul	0.099	–	1740	1141	51	2	45	2	–	–	17	0	0.0	2.1	–	–	–	–	24	3	7.2	7.2	–	–	–	–	–	–	–	–
20 Jul	0.099	118	1740	1150	–	–	28	2	–	–	14	0	0.0	2.9	–	–	–	–	18	3	7.3	7.1	–	–	–	–	–	–	–	–
27 Jul*	0.100	–	1247	839	36	19	43	8	27	7	16	5	0.0	9.4	4.4	4.9	3.0	4.0	23	5	7.2	6.9	562	537	–	–	–	–	–	–
30 Jul	0.096	–	1254	690	–	–	37	4	22	5	15	6	0.0	11.3	4.1	3.3	–	–	25	4	7.1	6.9	569	651	–	–	–	–	–	–
31 Jul	0.107	–	1806	1297	–	–	33	2	–	–	15	3	0.2	5.8	–	–	–	–	17	2	7.1	7.1	–	–	–	–	3.3×10^6	1.2×10^4	–	–
1 Aug	0.105	105	1806	1248	40	3	61	2	–	–	16	2	0.1	3.8	–	–	–	–	22	2	7.2	7.2	–	–	–	–	–	–	–	–
2 Aug	0.168	–	2619	2091	39	4	26	3	–	–	16	1	0.0	3.2	–	–	–	–	15	3	7.2	7.2	–	–	–	–	–	–	–	–
3 Aug	0.164	–	2700	2056	47	4	23	2	–	–	15	1	0.2	3.8	–	–	–	–	17	2	7.3	7.1	–	–	–	–	–	–	–	–
6 Aug	–	–	2309		–	–	29	4	–	–	17	0	0.1	5.6	–	–	–	–	21	5	7.3	7.1	–	–	20	23		–	–	–
7 Aug	0.170	70	2601	2150	–	–	36	3	31	6	23	2	0.0	0.8	4.4	2.6	2.7	1.8	24	4	7.4	7.0	378	311	24	34	1.2×10^6	5.4×10^4	–	–
8 Aug	0.160	–	1593	1147	88	6	82	4	–	–	20	0	0.0	5.3	–	–	–	–	46	4	7.3	7.0	–	–	–		–	–	–	–
9 Aug	0.166	–	2970	2325	48	3	36	2	–	–	18	1	0.0	3.3	–	–	–	–	20	2	7.3	7.1	–	–		–	–	–	–	
13 Aug	0.167	–	3054	2541	–		77	3	–	–	23	3	0.0	1.5	–	–	–	–	39	4	7.4	7.1	–	–		–	–	–		
14 Aug	0.168	–	3168	2680	–	–	51	2	–	–	16	1	0.0	2.5	–	–	–	–	26	2	6.9	7.1	–	–	–	–	3.8×10^6	2.1×10^4	$.33\times10^5$	1.5×10^3
15 Aug	0.162	–	2640	2077	64	3	74	2	21	2	15	0	0.0	2.4	3.9	1.9	2.9	1.9	28	2	7.1	6.9	303	224	20	18	–		–	
16 Aug	0.159	–	2160	1540	61	3	50	2	–	–	14	0	0.0	1.9	–	–	–	–	27	2	7.3	7.1	–	–	–	–	–		–	
17 Aug	0.168	–	3240	2607	85	4	60	3	–	–	25	2	0.0	4.8		–	–	–	30	3	7.4	7.1	–		29	26	–	–	–	–
20 Aug	0.166	–	3168	2579	–		128	6	–	–	25	5	0.0	5.1	–	–	–	–	37	6	7.4	7.3	–		31	25	–	–	–	–
21 Aug	0.174	–	3294	2740			43	3		–	21	5	0.0	2.1	–	–	–		22	4	7.3	7.1	–	–	28	23	7.3×10^6	1.3×10^5	2.4×10^5	1.7×10^4
22 Aug	0.173	–	2116	1723	76	5	46	5	–	–	20	2	0.0	2.0					30	3	7.4	7.2	–	–		–				
23 Aug	0.175	–	3312	2774	61	5	46	3	38	7	21	3	0.0	2.2	4.5	2.4	3.7	2.3	22	3	7.3	7.1	–		28	26		–	–	–
24 Aug	0.168	–	3312		72	5	44	2		–	23	4	0.0	2.3	–			–	24	3	7.2	7.0					–			–
28 Aug	0.167	–	3033	2393			57	3		–	21	3	0.0	2.2	–	–		–	23	4	7.2	7.0	–				1.8×10^6	5.8×10^3	0.4×10^5	5.2×10^3
29 Aug	0.171	65	3204	2682	–	–	39	3	–		21	3	0.0	2.2	–	–		–	25	3	7.2	6.9				–		–		–
30 Aug	0.171	–	1890	1584	89	5	37	3		–	21	3	0.0	2.7	–		–		32	3	7.2	6.9	–							
31 Aug	0.166	–	3177	2488	54	3	31	2	21	3	17	2	0.0	3.5	3.8	1.9	3.3	2.1	19	2	7.3	6.9			24	23				
4 Sept	0.157	–	1914	1358	–		36	4		–	24	1	0.0	4.9					39	4	7.2	6.9					$.23\times10^6$	1.0×10^4	0.1×10^5	5.8×10^4
5 Sept	0.160	–	2838	2289	71	7			29	3	23	1	0.0	3.1	5.3	1.7	4.6	2.2	25	4	7.2	6.8	399	252	24	21				
7 Sept	0.164	–	3168	2506	86	3	107	2	–	–	18	1	0.0	2.3					35	2	7.2	7.0								
10 Sept	0.155	–	2262	1591			41	–	–	–	21	1	0.0	5.1			–		27	6	7.1	6.8								
11 Sept	0.356	–	6716	5260	–	–	–				24	11	0.0	2.1					34	13	7.3	7.1					2.1×10^6	8.5×10^4	1.5×10^5	6.5×10^4
13 Sept	0.385		1980	1744			–		36	16	24	10	0.1	2.6	7.3	4.3	5.7	4.7			7.3	7.0	423	403	46	39				
17 Sept	0.102	110	1260	812		–			–	–	–		–		–															
18 Sept	0.160	50	2651	2124		–	–				–	–		–																

Applied and Runoff Water Quality from Section C of CRREL Overland Flow Test Site (cont'd).

Date	Average overland flow rate, q (m^3/hr m)	Detention time (min.)	Steady state volume of wastewater, L		BOD (mg/L)		TSS (mg/L)		TKN (mg/L)		NH_3-N (mg/L)		NO_3-N (mg/L)		Tot. P (mg/L)		PO_4 (mg/L)		Turb. (JTU)		pH		Spec. cond. (µmhos/cm)		Cl^- (mg/L)		F. coli. (#/100 mL)		F. strep. (#/100 mL)	
			Appl.	Runoff	Appl.	Runoff	Appl.	Runoff	Appl.	Runoff	Appl.	Runoff	Appl.	Runoff	Appl.	Runoff	Appl.	Runoff	Appl.	Runoff	Appl.	Runoff	Appl.	Runoff	Appl.	Runoff	Appl.	Runoff	Appl.	Runoff
19 Sept 79	0.220	42	3660	2846	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–
20 Sept	0.295	30	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–
21 Sept	0.382	23	6277	5435	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–
25 Sept	0.373	–	3307	2709	–	–	40	7	33	18	25	14	0.3	0.7	5.0	3.7	–	–	31	13	7.1	6.9	444	369	27	26	–	1.3×10^6	–	1.1×10^5
28 Sept	0.382	–	7029	6054	89	39	52	20	–	–	27	17	0.0	1.4	–	–	–	–	35	18	7.3	7.0	–	–	–	–	–	–	–	–
3 Oct	0.396	–	7069	6622	87	29	46	5	–	–	22	14	0.0	0.4	–	–	–	–	39	18	7.2	7.3	–	–	–	–	–	–	–	–
5 Oct	0.384	–	7544	6588	61	17	32	7	28	15	23	14	0.0	0.8	4.8	3.1	3.7	3.1	24	8	7.2	7.1	388	327	23	21	–	–	–	–
10 Oct	0.386	–	5564	4906	77	31	36	10	35	21	27	17	0.0	1.7	5.7	4.6	4.6	3.7	28	14	7.4	7.1	491	412	32	28		–	–	–
12 Oct	0.386	–	6574	5823	102	37	39	9	–	–	30	23	0.0	1.0	–	–	–	–	32	15	7.3	7.1	–	–	–	–	–	–	–	–
17 Oct*	0.381	–	6851	5876	–	–	84	12	49	32	35	28	0.2	1.1	8.0	6.1	5.3	5.3	38	19	7.7	7.4	536	424	31	31	–	–	–	–
19 Oct	0.381	–	6712	5749	143	60	79	15	–	–	35	30	0.2	0.0	–	–	–	–	44	20	7.3	7.3	–	–	–	–	–	–	–	–
23 Oct	0.304	–	4928	4140	–	–	65	6	39	20	28	15	0.0	4.3	6.2	4.7	5.2	4.5	35	11	7.2	7.2	452	409	28	28	2.4×10^6	–	6.1×10^5	–
24 Oct	0.238	–	744	681	–	–	161	36	–	–	29	10	0.0	3.5	–	–	–	–	92	36	7.0	7.4	–	–	–	–	–	–	–	–
29 Oct	0.113	–	1572	1281	–	–	88	10	46	16	33	12	0.0	8.3	8.4	4.3	5.6	3.3	33	6	7.2	7.1	555	449	35	35	2.5×10^6	5.6×10^5	3.0×10^5	8.0×10^4
31 Oct	0.110	–	1758	1358	114	21	94	7	–	–	33	12	0.0	8.3	–	–	–	–	42	9	7.4	7.3	–	–	–	–	–	–	–	–
2 Nov	0.108	–	1204	1025	101	12	67	8	–	–	30	11	0.1	7.9	–	–	–	–	37	7	7.2	7.1	–	–	31	31	–	–	–	–

* First application day after harvest.

APPENDIX B: ANALYTICAL TECHNIQUES

Suspended solids analyses (total and volatile) were performed according to Millipore Bulletin AB312, (1975) in which the glass fiber filter technique (drying at 103°- 105°C for total solids and igniting at 550°C for volatile solids) was used. This is basically the same procedure as in *Standard Methods* (APHA 1975, p. 97), except that it is more specific. One modification of this procedure was to momentarily lift the filter from the holder after the sample had filtered through and then to replace it for approximately one minute. This was necessary to release a vacuum causing a film of water to remain directly under the filter. It was found that if not dried sufficiently, the filter would stick to the aluminum pan when dried in the oven. This modification, which proved to be very effective, was used in both filter preparation and sample filtration. The only other modification was that the filters were weighed to the nearest 0.01 mg instead of 0.1 mg as stated in the Millipore procedure.

Biochemical oxygen demand (BOD) measurements were obtained using the Winkler method, "Oxygen Demand (Biochemical)" in *Standard Methods* (1975, p. 543-549). Dissolved oxygen measurements (initial and five day) were made according to "Azide Modification" of the lodemetric Method, *Standard Methods* (1975, p. 443-445). Most reagents used were purchased (lot numbers and new bottles recorded) to ensure consistency. The phosphate buffer was prepared as stated in *Standard Methods* (1975, p. 545) in 100-mL amounts, refrigerated when not used and replaced approximately every month. Dilution technique 2 was used (p. 547). Reagents were added gently down the necks of the BOD bottles using repeater pipettes. Glucose-glutamic acid, dilution water and seeded blanks were run with each test.

Turbidity measurements were obtained using a Hach turbidimeter, model 2100A, according to the EPA approved procedure described in "Wastewater Analysis Handbook" (Hach Chemical Co. 1978, p. 592). Results were expressed in Jackson Turbidity Units (JTU). Samples with a turbidity reading greater than 40 JTU were *not* diluted. The turbidimeter was recalibrated before each sample, using the prepared latex standard.

Fecal coliforms and fecal streptococcus bacteria were enumerated according to procedures described in *Standard Methods* (1975, p. 937-939 and 944-945, respectively) and Millipore's "Biological Analysis of Water and Wastewater," Application Manual AM 302, p. 34-35. Millipore's 2-mL ampoules of M-FC broth were used for fecal coliform test and agar plates of *M-Enterococcus* were prepared for the fecal streptococcus test. One-hundred-milliliter volumes of varying sample dilutions (at least 3 different dilutions/sample) were filtered for each test. Millipore's incubator, which has the ability to maintain a critical narrow temperature range (44.5°C ± 0.2°C), was used.

The pH of the samples was taken using a Markson 1808 probe and an Orion 801 Ionalyzer. The probe was calibrated daily with pH 7.00 and 4.00 buffers. Values for pH were read after the probe was left in solution 1 minute to ensure consistency.

Nitrate-nitrogen was analyzed on the Technicon Auto Analyzer II (AAII) using Technicon's "Automated Cadmium Reduction Method." Two procedures were employed depending on amount of NO_3-N present. Technicon Industrial Method (T.I.M.) 246-731 ("3 in. Dialyzer Method") was used for samples ranging from 0 to 50 mg L^{-1} NO_3-N, and T.I.M. 271-73W ("24-in. Dialyzer Method") for samples between 0 to 1 mg L^{-1} NO_3-N.

Samples were tested for *ammonia-nitrogen* (NH_3-N) using the Technicon AAII. The "Salicylate/nitroprusside Method" was used according to T.I.M. procedure 329-74 W/A (revised Jan 1976) for NH_4-N ranges from 0 to 50 mg L^{-1}

The Technicon AAII "Block Digestion Method" was used for analysis of *Kjeldahl N* within the range of 0 to 50 mg L^{-1}. The digestion procedure used was T.I.M. 376-75 W/B (revised March 1977) and analysis procedure T.I.M. 329-74 W/B (also revised March 1977).

Chloride was analyzed on the Technicon AAII using the "Thiocyanate Method" for ranges of 0 to 35 mg L^{-1}, according to T.I.M. 99-70 W (1973).

Total P was analyzed on the Technicon AAII using the "Block Digestion Method" (molybdenum blue analysis for ranges of 0 to 10 mg L^{-1}). Digestion was carried out according to T.I.M. 376-75 W/B (1977) and analysis followed T.I.M. 329-74 W/B procedure (revised March 1977).

Ortho-P was analyzed using a Coleman Junion Spectrophotometer and the "Manual Molybdenum Blue Method" for ranges of 0 to 0.11 mg L^{-1}, according to Hach Chemical Company's "Water and Wastewater Analysis Procedure."

Specific Conductance was obtained using a resistivity bridge for ranges between 100 to 1000 μmhos cm^{-1}.

APPENDIX C: DETERMINATION OF SCOUR VELOCITY

An estimate of the scour velocity for overland flow systems was obtained from sedimentation theory. According to Metcalf and Eddy (1972), the horizontal velocity that will just produce scour in a settling basin can be determined from the following relationship:

$$V_s = \left[\frac{8k\ (s-1)\ gd}{f}\right]^{1/2} \tag{C1}$$

where V_s = horizontal scour velocity (m s^{-1})
s = specific gravity of particles
d = diameter of particles (m)
k = constant which depends on the type of material being scoured $\approx$ 0.06
g = gravitational constant = 9.81 m s^{-2}
f = Darcy-Weisbach friction factor = 0.025.

Assuming a particle size of 0.1 mm and a specific gravity of 1.1 the scour velocity was calculated to be 0.0434 m s^{-1} or 2.6 m min^{-1}. This appears to be a reasonable estimate based on the performance of the CRREL overland flow site. Suspended solids removal began to decrease slightly at a detention time of 20 minutes, which is equivalent to a velocity of 1.5 m min^{-1}.

The limiting overland flow rate that will produce a scour velocity (V_s) of 2.6 m min^{-1} can be determined from eq 13 as follows:

$$\overline{T} = \frac{0.078\ L}{1/3} = \frac{L}{V_s}.$$

Solving for q,

$$q = \frac{0.078\ V_s}{S^{1/3}} = \frac{0.078(2.6)}{S^{1/3}} = \frac{0.20}{S^{1/3}}.$$

Therefore, in order to avoid resuspension of solids and scour, the average overland flow rate should be limited to $0.20/S^{1/3}$ or less.

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Martel, C.J.
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