A FIELD STUDY OF A HOUSEHOLD PACKAGE TREATMENT PLANT

by

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ABSTRACT

The field operation of an individual household aerobic treatment plant was investigated. The treatment plant study was paralleled by a literature survey of current disposal system practices.

It was found that the Cromaglass type CA5-D aerobic wastewater treatment system produced, under current installation and operation practices, an average effluent of 85 mg/l BOD and 79 mg/l SS. Under similar circumstances, the type CA5-E plant produced an average effluent quality of 67 mg/l BOD and 56 mg/l SS. As neither type of plant operated with a VSS concentration of greater than 150 mg/l, the treatment process cannot be described as a modification of the activated sludge system.

The literature indicates that the mechanisms and processes occurring in wastewater disposal systems are inadequately understood. As a consequence, no adequate test has yet been developed which is capable of relating measurable soil conditions to suitability for a disposal field. However, several authors have suggested that average effluent BOD's of approximately 40 mg/l will be required to permit a significant increase above current absorption field hydraulic loadings.

Under present conditions of installation, service, operation, and effluent quality, it is probable that significant relaxation of design criteria for in situ and imported fill disposal fields cannot be permitted for the Cromaglass treatment systems.

Indexing Terms: aerobic wastewater treatment, effluent, BOD SS, wastewater, domestic wastewater treatment

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CHAPTER I

INTRODUCTION

For many years suburban development has been occurring outside of regions served by sewer systems. In the past, wastewater management in these areas has been supplied by the familiar septic tank-absorption field system. Where large lots are permissible and soil conditions acceptable, this system provides low cost, low maintenance treatment and disposal. However, smaller lot sizes and adverse soil or terrain characteristics, have created a demand for a treatment system capable of adequately disposing of wastewater under these constrained conditions.

Bailey and Wallman (1) present a review of a few of the many systems which have been offered in response to this demand.* Most of the systems utilize aerobic biological treatment but do not provide for disposal of the effluent. In general, aerobic treatment on an individual household scale has not had widespread success. Many of the proposed plants have not survived the first stages of the evolutionary design. Those that have proceeded to the workable plant stage have, in general, yet to overcome problems of reliability, service, installation, parts inventories and public acceptance.

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*Note: Because most of the systems proposed are of a patented, and therefore somewhat unique, design and because the operational characteristics have been tested under such varied circumstances no comparison with other household aerobic wastewater treatment systems will be made. However, interesting data does exist (22,23).

The poor survival statistics of these facilities are not surprising in light of the design criteria. The variation in wastewater characteristics from household to household and even hour to hour within one household is not only immense, but completely unpredictable. Combined with the technical problems of small scale treatment plant design, this makes design of a rugged reliable unit very difficult.

The only household-scale aerobic waste treatment plant currently available in British Columbia is the Cromaglass* Aerobic Wastewater Treatment System. Currently, no unified treatment and disposal system is available. The Cromaglass system in British Columbia has been undergoing evolutionary development for approximately four years. The current type CA5-E plants are the most sophisticated units to emerge from the sequence thus far. Undoubtably, additional changes will be made in the future.

Although Cromaglass plants have been in operation for a number of years, little information, other than that provided by the manufacturer, is available on their field operation. Normally the manufacturer does not participate directly in the installation, maintenance, or operation of the plants. As each of these functions can, and often is, performed by different groups, many people have an effect on the eventual operating conditions of the plant. It was the purpose of this study to investigate the performance of

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*Note: Cromaglass is the registered trademark of Northern Purification Services Ltd., North Vancouver, B.C.

treatment units that were subject to the normal sequence of sale, installation, hook-up to the tile field, and finally operation by the householder.

To ensure the data was representative of field experience, disruption of household activities was minimized. In addition, no extraordinary action was taken by regulatory, maintenance or manufacturing agencies during the sampling period.

The plants chosen for sampling were selected on the basis of the characteristics of the family occupying the house. None of the plants was examined before the selection of sampling sites was made. During the sampling period all of the thirty plants, from which the 15 tested plants were chosen, were visually examined. Generally, these examinations reinforced the contention that the sampled plants were representative of operations in that locality.

The field testing of treatment systems was complemented by a literature survey of disposal system design. The greater portion of the literature is directed toward the design of septic tank absorption systems. These include in situ fields, fields in imported material, wells or pits and sand beds. Some data for the design of evapotranspiration systems was also available. However, this material is not yet well documented nor proven.

Throughout this report, two points should be kept in mind. It should be remembered that as system complexity and system stress increase, cost will inevitably increase also. Secondly, it should be emphasized that the data in this report on treatment system operation and the background material on disposal systems from the literature provides, at best, a skeleton upon which to hang an overall design. As design criteria become more constrained there will be a greater need to fill in details.

CHAPTER 2

UNIT OPERATIONS

I. Cromaglass Aerobic Wastewater Treatment System

The treatment plants sampled during this survey were all Cromaglass Series CA5 units, manufactured by Northern Purification Services Ltd. The two latest designs in an evolutionary sequence were examined. In terms of actual configuration, however, three distinct types of plants were studied.

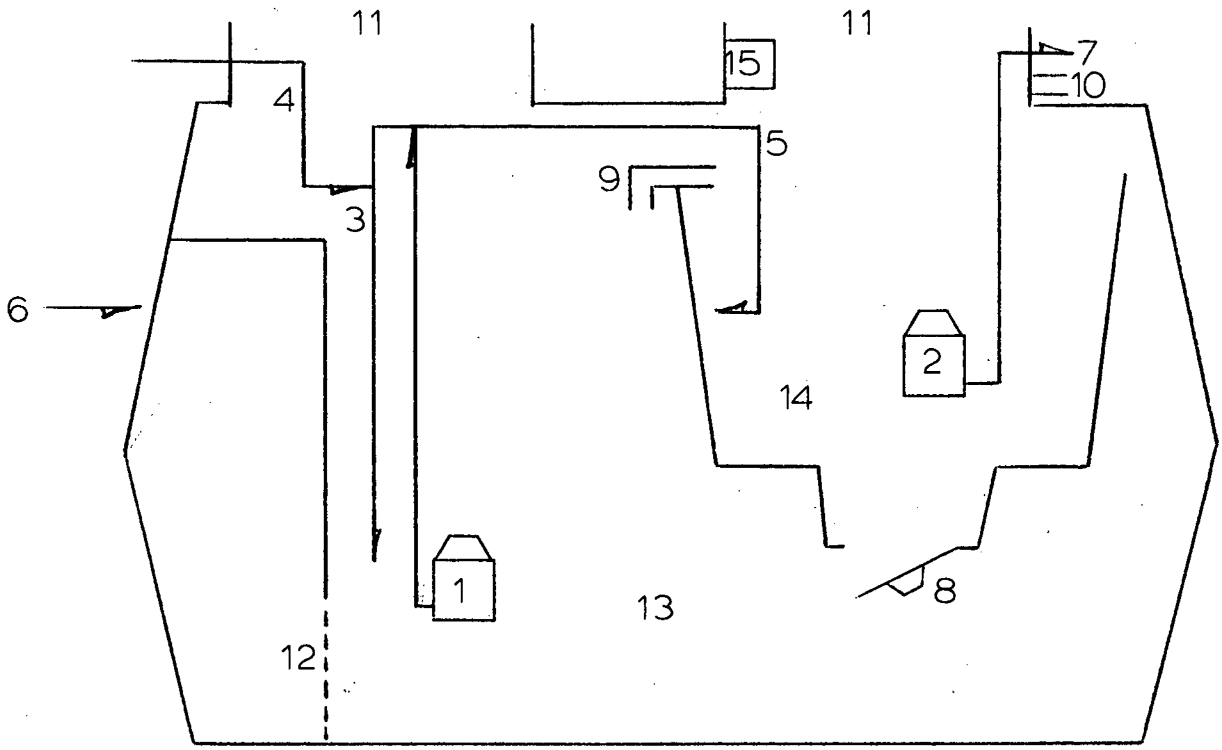
A. CA5-D

The CA5-D (type D) plant was sold up to December 1973. The general plant layout is illustrated in Figure 1. The outer shell and settling tank are of fiberglass construction and the pumps are fully submersible, sealed units. Under normal conditions the plant operates with a total liquid volume of approximately 330 Imperial gallons.

Influent enters the primary solids removal chamber from the house sewer. The wastewater then flows down through a screen into the aeration chamber. The aeration pump picks up mixed liquor and circulates it up through a bleed-off connection and a venturi and back into the aeration tank.

A plastic tube extending from the venturi to the basement of the house provides the air supply. Intimate contact of the air and wastewater is achieved as the mixture is injected back into the mixed liquor.

The bleed-off connection allows some of the mixed liquor to be continuously transferred to the settling tank.



1 aeration pump
2 discharge pump
3 venturi
4 air supply line
5 transfer line (to settling tank)
6 raw waste inlet
7 discharge line
8 float valve - for solids return
9 overflow
10 overflow (to disposal)
11 access manholes (with covers)
12 solids retention screen
13 aeration tank
14 settling tank
15 junction box

FIGURE 1

CROMAGLASS TYPE CA5-D AEROBIC WASTEWATER TREATMENT SYSTEM

When liquid levels in the aeration and settling tanks have equilibrated, a float operated valve, on the bottom of the settling chamber, opens to allow solids to return to the aeration chamber. The discharge pump is controlled by a float valve, so that discharge takes place only after the liquid level in the whole plant has reached the discharge depth.

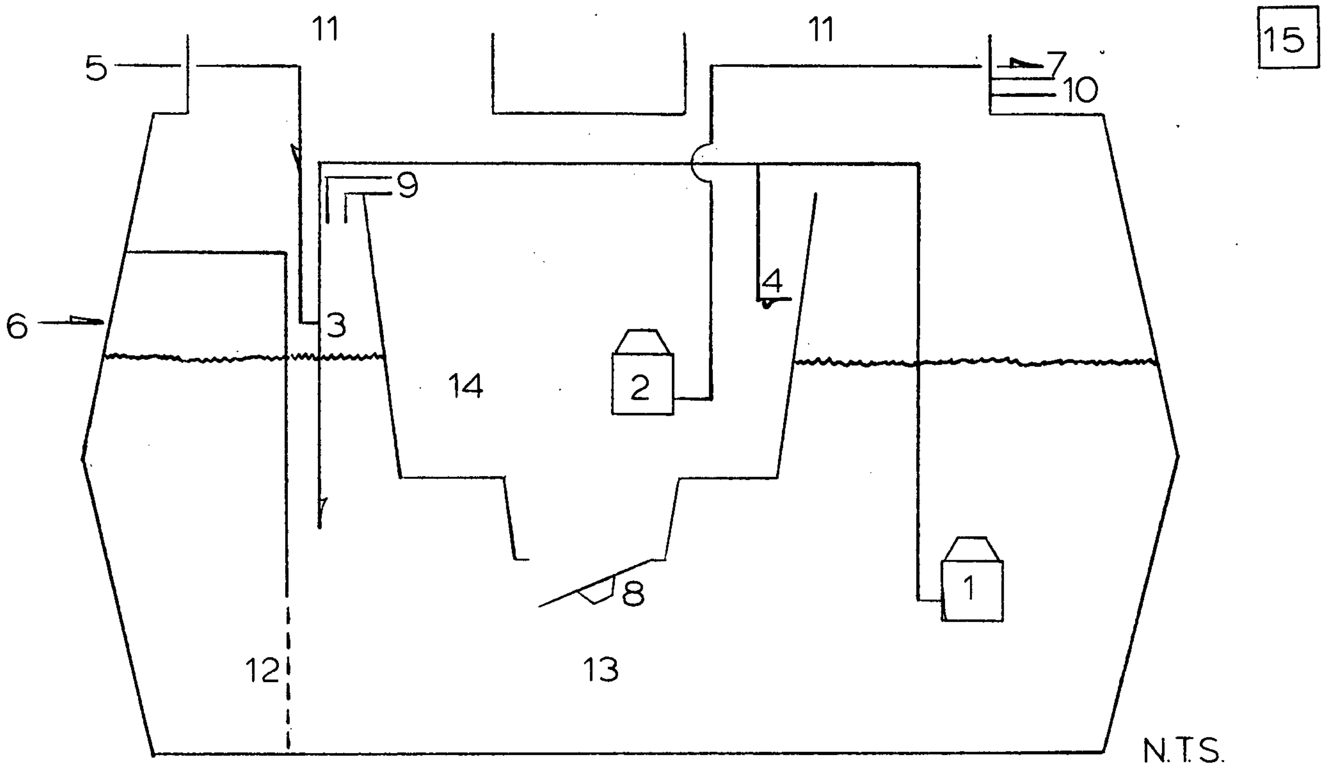
Continuous transfer from the aeration tank to the settling tank occurs at a rate of 1.5 to 4 gpm. The batch effluent discharge, of approximately 48 I gal, occurs in 1.5 to 3.0 minutes.

B. CA5-E

The type E units differ from the older type D plants primarily in their mode of operation. As figure 2 indicates, only the spatial arrangement of the components is altered. The other physical specifications are identical to the type D plants.

The significant change in the type E plants is the introduction of intermittent operation. Control over the aeration and discharge pumps is exercised by a remotely mounted timer.

The timer controls the mode of operation and the length of time the plant operates in a particular mode. These plants have two operational modes. One is the same as that which occurs in the type D plants, that is, continuous aeration and transfer to the settling tank. The other mode is complete quiescence throughout the plant. This second mode is intended to provide a quiescent settling period before discharge to disposal facilities. Discharge occurs near the



1 aeration pump
2 discharge pump
3 venturi
4 transfer line (to settling tank)
5 air supply line
6 raw waste inlet
7 discharge line
8 float valve - for solids return
9 overflow
10 overflow (to disposal)
11 access manholes (with covers)
12 solids retention screen
13 aeration tank
14 settling tank
15 remotely mounted timer control

FIGURE 2

CROMAGLASS TYPE CA5-E AEROBIC WASTEWATER TREATMENT SYSTEM end of the settling period just before aeration begins again.

Any desired length of settling or aeration period can be achieved by adjusting the timer. In addition to providing improved settling conditions, intermittent operation reduces power consumption and increases aeration pump life.

The other major change in the type E plants, spatial arrangement, was instigated to increase mixing efficiency in the aeration tank.

C. CA5-E(D)

This unit appeared as a result of a temporary shortage of timer controls. The physical configuration is that of a CA5-E plant but it operates continuously as does the CA5-D. These units are now being converted to CA5-E units with the installation of the timer control by the manufacturer.

As these plants functioned in the same way as the CA5-D units, it was expected that the effluent they produce would be similar to that of the type D plants. Houses numbered 1,2,4,9, and 15 had the CA5-E(D) plants. As Table 6 shows, these units do not appear to be significantly different than the type D plants in terms of effluent quality.

II. Treatment Design

A. CA5-D

The design is based on the extended aeration or aerobic stabilization principle with intermittent batch discharge. Input is uncontrolled and therefore also intermittent in nature. Provision has been made for the return of solids separated from the effluent.

The description of the unit operation as extended aeration is theoretical only. In fact, the operation of these plants is impossible to analyze within the framework of conventional biological treatment systems.

To begin with, the plants are subjected to a much more irregular "dosing" pattern than any larger scale system. Hydraulic loadings can vary from 0 to 20 gpm instantaneously, with great ranges in the total volume of a single input surge. Organic loads show even more dramatic peaks. As these plants have only a very rudimentary facility for providing load equalization, the use of any average values of influent characteristics can be completely misleading.

The insignificant amount of MLVSS (see Table 1) precludes the inclusion of these plants with any of the variations of the activated sludge process.

The timing of the batch discharge is controlled by the influent volume. Thus, the hydraulic residence time is also a function of inflow volume. The relatively small working volume of the plants fails to damp out fluctuating input volumes and thus average aeration time can be highly variable.

However, with these analytical shortcomings in mind, some generalizations can be made about the operations.

The 330 gallon operating volume provides an average hydraulic residence time of 44 hours for a family of four (see Appendix 2). This period is longer than the 18 to 36 hours generally recommended for larger plants (18). In the larger extended aeration plants, a MLVSS of 3000 to 6000 mg/l can be maintained under organic loadings of 10 to 25 lb BOD/day/1000 ft^3 of aeration tank volume.

The continuous aeration of the type D plants probably results in short periods of overloaded and relatively long periods of underloaded operation. Appendix 2 suggests an organic load of 0.106 lb BOD/capita/day can be expected on the average (without garbage grinding). Four persons would then be expected to provide 0.424 lb BOD/day. Based on a design organic loading range of 0.10 to 0.25 lb BOD/1000 ft^3/day a family of four would require between 265 and 105 gallons of aeration tank volume. The effective aeration chamber volume of the Cromaglass plants is approximately 220 gallons. Thus, for any household of less than five people, the unit will be operating at very low organic loadings most, if not all, of the time.

These low loading conditions and long hydraulic residence times could result in the destruction of volatile solids by auto-oxidation. The majority of the solids, formed during periods of high organic loading, are oxidized during the low load periods at mid-day and at night. The low TSS and VSS concentrations found in the mixed liquor (Table 1) are not, then, unexpected.

TABLE 1

MLVSS IN CA5-D PLANTS

House Number	No. of Samples	MLVSS, mg/l high	average	low
1	4	105	83	68
2	-	-	-	-
3	2	86	69	52
4	6	110	64	32
5	4	101	96	87
6	5	139	87	58
7	4	70	54	28
8	4	100	77	68
9	1	-	-	28

These low MLVSS concentrations emphasize that the system does not function by contacting incoming wastes with large concentrated volumes of active microorganisms. The treatment mechanism apparently consists of dilution of the relatively small volumes of incoming wastewater, to reduce the concentration, and then oxidation over a relatively long aeration period.

Both oxidation and discharge of solids in the effluent, contribute to the maintenance of low solids levels. The high-rate transfer from the aeration to settling tank (measured at approximately 2 to 4 gpm) induces a great deal of turbulence in the settling tank. The result is that virtually no solids removal is attained and consequently no accumulated solids are available for return to the aeration chamber. That this does occur can be seen from Table 2, which lists the mean difference between simultaneous effluent and mixed liquor samples.

TABLE 2

DIFFERENCE BETWEEN MIXED LIQUOR AND EFFLUENT SAMPLES

TYPE CA5-D PLANTS

parameter	mean difference*	standard deviation of difference
BOD	-0.6667 mg/l	20.1090
SS	-0.1220 mg/l	42.6639
VSS	-6.9048 mg/l	17.1520
COD	-15.7778 mg/l	35.2909

*mean difference = mean of (Settling Tank Concentration - Mixed Liquor Concentration)

The submersible pumps used in these plants operate at approximately 45 gpm. The shear within the pumps and in the venturi used for aeration is probably sufficient to disintegrate any large flocs of activated sludge. If the settling velocity of the resultant fine floc were slow enough, significantly greater solids concentrations may be discharged in the effluent.

The sewage inlet from the house discharges into a primary chamber, designed to provide some initial solids retention. The waste then flows through a screen into the aeration chamber. No attempt was made to measure the solids build-up in this chamber. However, it may provide some reduction in the solids loading on the system. The retained solids would theoretically decompose aerobically until small enough to escape through the screen into the aeration chamber. In light of the generally underloaded condition of these plants this primary treatment may not be desirable.

When the aeration pump was operating at full capacity, dissolved oxygen concentrations in the mixed liquor were in the range of 1.0 to 8 mg/l. Often, however, the aeration pumps would become partially clogged, resulting in a decrease in dissolved oxygen concentrations to the 0 to 5 mg/l range. Usually, a clogged pump would clear itself within a day or two but some remained partially clogged for a week. Occasionally the persistence of low dissolved oxygen conditions was sufficient to produce enough undesirable odor to cause the homeowner to call in service personnel.

Volatile solids usually made up 65 to 90 percent of the TSS but samples as low as 50 percent were recorded. As most of the plants had been operating from 6 to 15 months, the buildup of inorganic material does not appear to be a problem. This is probably the result of the discharge of solids with the effluent.

B. CA5-E

The physical and operational changes instituted in the CA5-E plants do apparently result in improved effluent quality (Chapter 3). The most significant change in terms of unit operations, the introduction of intermittent aeration, does have at least one undesirable side effect.

In the plants included in this survey, the timer was set to give approximately a two hour settling period. During this time, there is no interruption of the periodic inputs. Biological degradation of the existing and applied organic loads continues, with a resultant decrease in dissolved oxygen concentration. In some instances, this resulted in anaerobic conditions relatively soon after aeration ceased. The presence of anaerobic conditions was correlated with the loading of the plant to the extent that it only appeared in plants serving five or more people.

Dissolved oxygen concentrations returned to aerobic levels (5 to 7 mg/l) within a few minutes of the resumption of aeration. Thus, the oxygen deficit, created during the anaerobic period (which lasted as long as an hour), probably did not adversely affect the treatment process. The unpleasant odor however, was a source of householder complaints.

Without exception, discharge occurred from 15 to 45 minutes before aeration began. Because of the unpleasant odor associated with the anaerobic conditions prevailing at the end of the settling period, it would be advantageous to begin aeration immediately after the discharge.

During the aeration cycle, these plants function similarly to the CA5-D units and exhibit similar characteristics.

The average mixed liquor data, shown in Table 3, exhibits low VSS concentrations as were found in the type D plants. Even though the aeration pump was only operating for one hour out of every three, it appears the solids were effectively disintegrated.

TABLE 3

MIXED LIQUOR DATA - CA5-E PLANTS

house #	average BOD mg/l	average SS mg/l	average VSS mg/l
12	66	65	62
13	97	81	71
14	43	41	41

SUMMARY

The problems, afore discussed, concerning the analysis of the unit operations, manifest themselves when analyzing the effluent quality data also. Throughout these discussions, average values of parameters have been used, mainly because of a lack of data on influent conditions and the consequent difficulty of analyzing time series data. However, regardless of the influent characteristics, it is the quality of the effluent which governs disposal system design.

CHAPTER 3

EFFLUENT QUALITY AND GENERAL OBSERVATIONS

I. Sampling

The primary purpose of this investigation was to obtain data on the quality of effluent to be expected under field conditions. A total of fifteen households, using Cromaglass systems, were surveyed during June, July and August, 1974. The study was concentrated in the type D units as it was difficult to find type E plants which were suitable. However, five type E plants were included in the sampling program.

Effluent samples were taken from the settling tanks. This was necessary because, in all but two of the systems, the entire disposal field, including the distribution box, was inaccessable. Samples were taken approximately six to eight inches below the liquid surface. Generally, this depth is approximately four to six inches above the discharge pump intake. The sampling device had a lid which was opened only a small amount after the device had been submerged. This prevented the inflow of liquid above the desired sampling depth and allowed the inflow rate to be controlled. Turbulence in the settling tank was minimized by this procedure.

Two type D plants were sampled simultaneously from the distribution box and the settling tank. The results of this sampling, as shown in Table 4, indicated that the settling tank sampling procedure was representative of the effluent which enters the tile field.*

TABLE 4

EFFLUENT SAMPLING

Distribution Box		Settling Tank		Difference	
BOD mg/l	SS mg/l	BOD mg/l	SS mg/l	Δ BOD mg/l	Δ SS mg/l
102	46	103	82	-1	-36
110	94	105	100	5	-6
105	88	105	72	0	16
78	106	85	105	-7	1
81	39	80	46	1	-7
63	39	63	36	0	3
86	30	78	42	8	-48
197	49	143	48	54	-1
			average difference	7.5	-9.5
			standard deviation of difference	19.29	21.48

As noted earlier, the frequency of discharge of these plants is regulated by the inflow volume. Thus, it is not possible to predict when the units will discharge to the tile field. Without automatic sampling devices, it becomes very difficult to collect samples precisely when the plant

*Note: For simplicity and clarity most of the concepts presented are discussed in terms of BOD only. Many of the same comments apply to SS, COD, and other quality parameters.

discharges.

In order to develop a profile of the plants operation, over a 16 hour period, sampling of the type D units was carried out at one half hour intervals. In plants serving homes where moderate to high water use was occurring, this sampling period often coincided with the discharge cycle. However, when water use was low, several effluent samples may have been taken for each discharge to the tile field.

Obviously, if effluent quality was correlated with water use (hydraulic load), this procedure could bias the results considerably. However, as will be discussed later, it was found that hydraulic loading was not correlated with effluent quality. Thus, this sampling procedure should not bias the results.

It should be emphasized that most effluent samples were actually taken during discharge to the tile field.

During the course of the summer, many aeration pump failures were observed. The early package plants came with a metal housed aeration pump. These were later replaced by pumps with a plastic housing. These plastic pumps exhibited such a high failure rate that Northern Purification returned to the use of the metal housed pumps. Many of the pumps which failed during the summer were plastic units. During failure periods and for one week after replacement, these plants were not sampled. One unit was sampled during and just after the failure and replacement of a metal housed pump. Thus, the effluent values for house number four have several high values. Number four was not the only house to experience the failure of a metal pump. The practice of not sampling units during pump failure would tend to make the overall effluent quality presented in this report, higher than actually occurred.

Thus, if the results presented are biased, they would tend to be of somewhat better quality than can be expected in normal field operation.

For the type E units, only the results from samples taken during discharge to the tile field are reported.

In addition to a discussion of the effluent quality data, this chapter contains some general notes about field performance.

II. Overview

A summary of the effluent data is presented in Table 6. Some average values are given in Table 5.

TABLE 5

AVERAGE EFFLUENT QUALITY FROM ALL UNITS

	average		median	
	BOD mg/l	SS mg/l	BOD mg/l	SS mg/l
type D	85	79	80	75
type E	67	56	63	56

Note: Regulatory standards are often based on average effluent quality.

TABLE 6

EFFLUENT QUALITY

House number	Percent of the time that the effluent quality exceeds the stated level**						Number of samples	
	50/50 *		70/70		125/125			
	BOD	SS	BOD	SS	BOD	SS	BOD	SS
type D units								
1	100	67	87	52	17	2	46	42
2	93	93	93	71	0	29	14	14
3	68	92	49	46	0	0	37	37
4	69	51	64	40	51	29	39	35
5	93	91	73	64	0	9	15	11
6	64	78	14	50	0	6	22	18
7	12	47	6	47	0	24	17	17
8	100	100	89	58	0	0	19	19
9	0	75	0	0	0	0	4	4
15	0	0	0	0	0	0	1	1
type E units								
12	100	50	50	50	0	0	2***	2
13	100	100	67	33	0	0	3	3
14	0	0	0	0	0	0	2	2
16	100	0	100	0	0	0	1	1
17	50	100	0	0	0	0	2	2

* A 50/50 effluent quality was used by the British Columbia Health Branch in its Guidelines for Package Treatment Systems.

A 70/70 effluent quality corresponds to an 80% removal of BOD and SS from an influent of 350 mg/l (or 72% from an influent of 250 mg/l).

TABLE 6 - Cont.

A 125/125 effluent quality represents a 50% removal from an influent of 250 mg/l (septic tank effluent)

** for example the effluent from house number 3 exceeded 50 mg/l BOD 68% of the time

*** these are discharge samples only

The type E units produce a better quality effluent than do the older type D plants. However, neither type is capable of consistently producing effluents in the range of 30 to 40 mg/l BOD. The effluent of the type D plants is not dissimilar from that expected from a properly functioning septic tank (1, 15, 18).

The following discussion relates an attempt to analyze the data in more detail. The analysis is concentrated on the type D units first and then extrapolated to the type E plants.

A. Effluent Quality - Type D

The great variation of the data from hour to hour and day to day make it difficult to recognize any significant correlations with other events. Another compounding factor is the variation of lag times between influent events and effluent discharge, as the input flow rate varies. The only statement that can be made with certainty is that it is impossible to predict the effluent quality of a particular plant based on previous data, family activities, other units performance or other measurable criteria or combination thereof.

1. Specific Effluent Characteristics

The general effluent quality has been described in

Tables 5 and 6 in terms of BOD_5 and SS. In addition to these routinely performed analyses, some other tests were run intermittently to provide a more complete description of the effluent characteristics.

One of the anomalies of the data is the apparently unexplicable peaks which occur in a time series of samples. For example, in 80 percent of the sampling sequences one or two BOD values were significantly higher or lower than the average. These peaks were not correlated with influent events. One possible explanation was that nitrification was beginning at or near the five day mark in the BOD test. This would result in the odd high or low BOD depending on how the samples were treated. To test this hypothesis 5, 8 and 11 day BOD tests were run on the same samples. Although the results were inconclusive, an average increase of 54 percent above the 5 day values was found in the 11 day tests. That is, the BOD_5 is less than 50 percent of the ultimate BOD.

Effluent COD ranged from 109 to 330 mg/l and averaged 218 mg/l over all plants. The BOD to COD ratio ranged from 0.181 to 1.083 and averaged 0.432.

In the early stages of the program, an attempt was made to correlate TOC with BOD. As Figure 3 illustrates, no such correlation could be found. In addition, no correlation was found between TOC and BOD within a particular plant.

It can be concluded that neither the TOC nor the COD test can be used to estimate the BOD in the effluent of these plants.

With respect to temperature, values ranged from

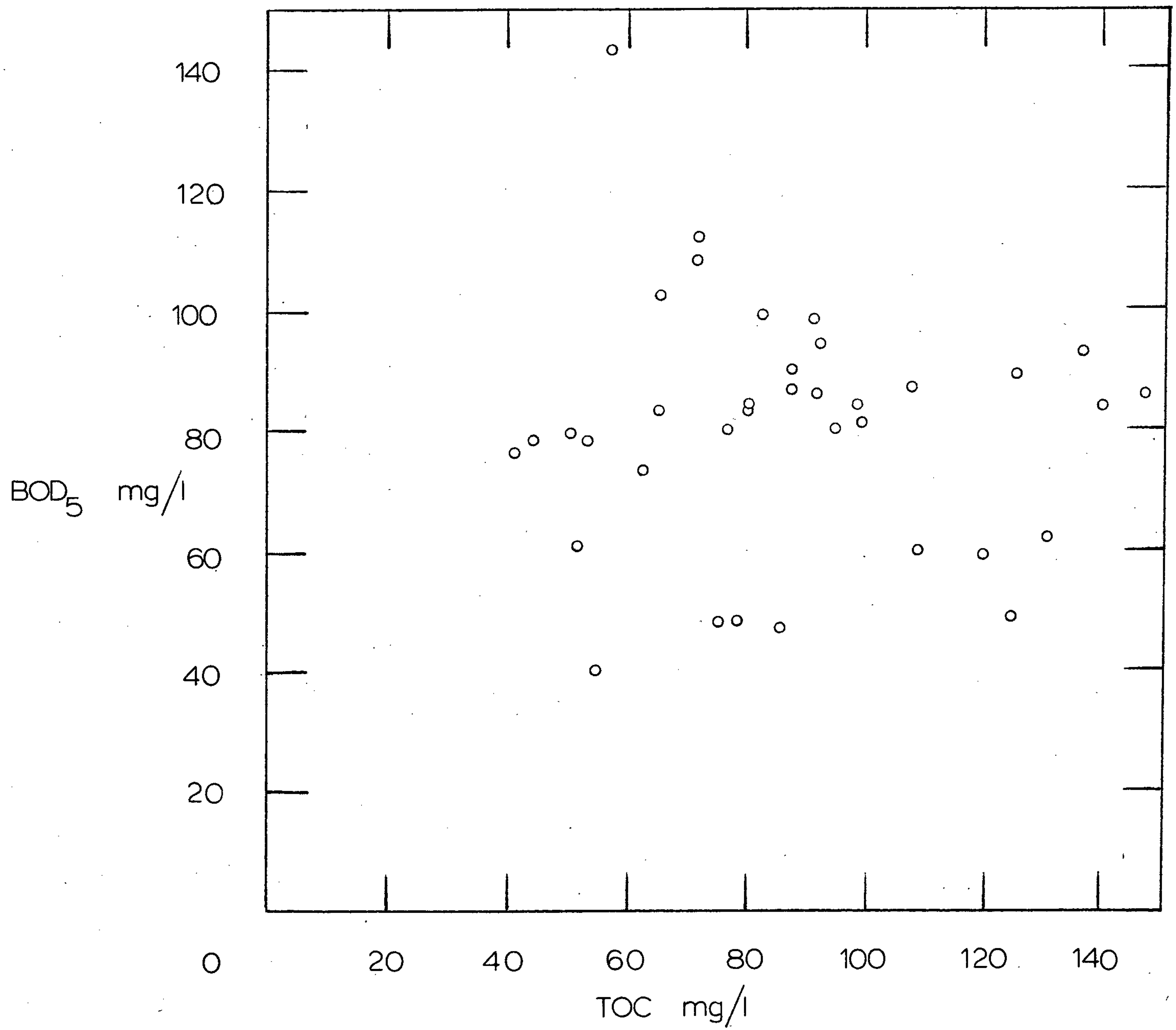


FIGURE 3 - BOD VERSUS TOC, TYPE D EFFLUENT (ALL D PLANTS)

22 to 31 °C with most plants averaging 25 °C.

2. Nutrients

Phosphorus analyses were not conducted as this data is well documented in other sources (16). It is reasonable to expect, as phosphorus compounds are not volatile and sludge removal is not practiced, that after a period of operation, a quasi-equilibrium state will be reached in which all the phosphorus entering the plant will be discharged.

To obtain an order-of-magnitude estimate of effluent nitrogen concentrations, tests were run on six samples, from two households. As Table 7 illustrates, significant nitrate and nitrite concentrations were recorded in some samples (see also discussion of nitrite in Appendix 3). As Table 7 shows, TKN to BOD ratios were significantly higher than that normally considered necessary for active microbial degradation. Thus, if groundwater contamination is a potential problem, precaution will be necessary in tile field design to ensure denitrification.

TABLE 7

EFFLUENT QUALITY - NITROGEN, mg/lN

House #	BOD_5 mg/l	NH_3 mg/l	NO_3 mg/l	NO_2 mg/l	ORN mg/l	TKN mg/l	BOD/TKN
3	82	23.3	1.3	1.9	8.7	32	2.6
	121	21.3	1.3	2.0	14.7	36	3.4
	104	19.3	2.5	2.9	13.7	33	3.2
6	66	22.5	0.05	0.103	14.5	37	1.8
	65	23.8	0.04	0.107	10.2	34	1.9
	60	23.8	0.05	0.129	9.2	33	1.8

3. Dissolved Oxygen

Generally, dissolved oxygen concentrations in the effluent were high. Unless the aeration pump was partially clogged, concentrations of 3 to 7 mg/l were normal. Pump clogging or failure resulted in anaerobic effluent after a relatively short time. Many of the plants had broken, leaking, or missing transfer line components, which often resulted in significant amounts of spray aeration. This additional aeration was sufficient to maintain high dissolved oxygen concentrations, even in plants with partially clogged aeration pumps.

No correlation could be found between daily average effluent quality and mixed liquor dissolved oxygen concentrations when the aeration pump was operating normally. The effect of aeration pump failure is discussed later.

4. Effluent Quality Versus Hydraulic Loading

Facilities were not available for sampling the influent for organic content or volume. However, bi-monthly water consumption figures were available for each household (Appendix 2). A plot of average effluent BOD versus average daily water consumption, Figure 4, illustrates no interpretable correlation.

The Cromaglass plants are designed to produce their best effluent quality within a range of daily average hydraulic loadings. That is, the manufacturer expects his units to produce an effluent BOD of approximately 40 mg/l when serving households discharging approximately 160 gpd. Depending on the turn down and overload capacities of the plant, good

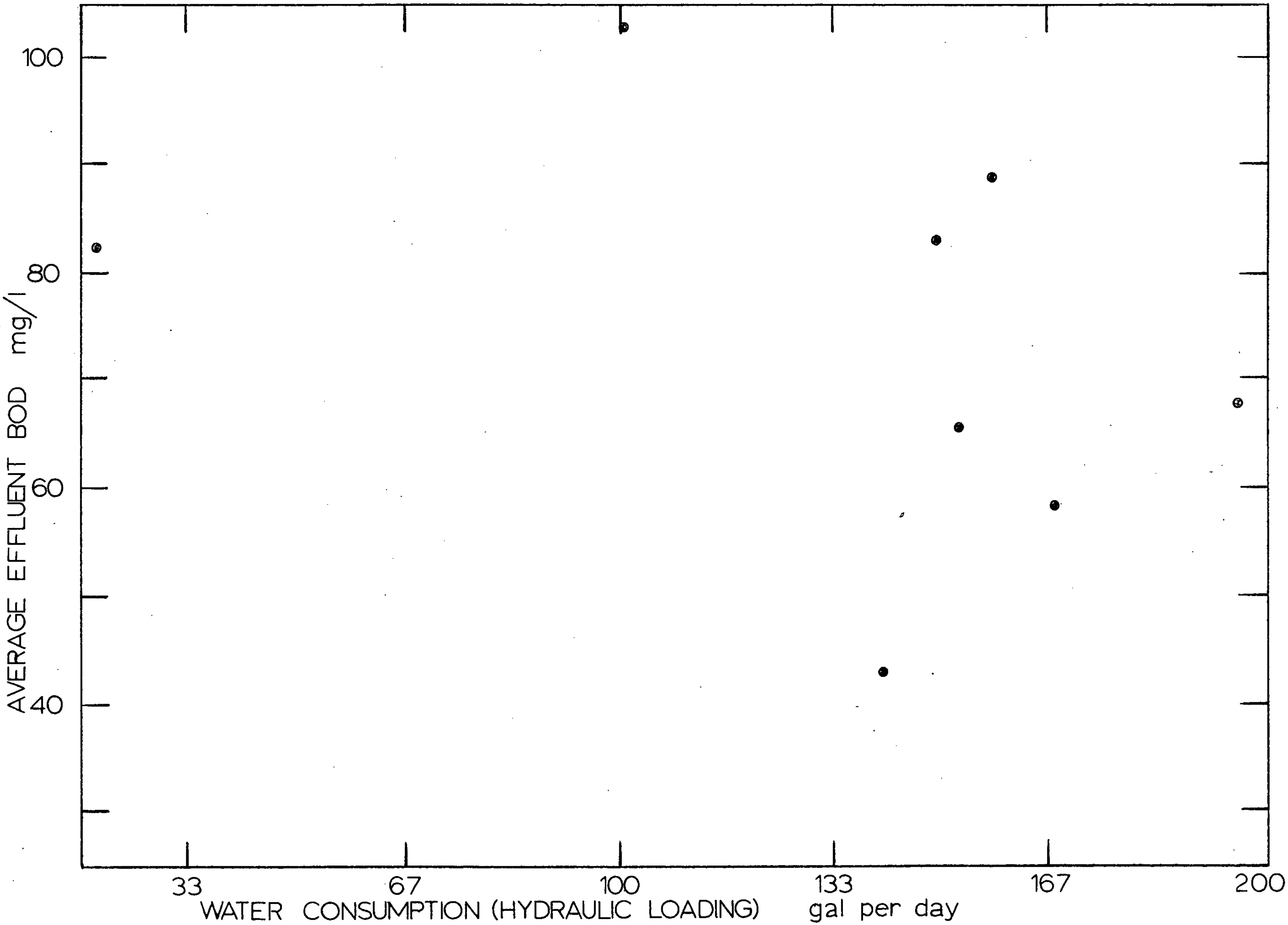


FIGURE 4 - EFFLUENT BOD VERSUS HYDRAULIC LOADING

treatment efficiency should be achievable over a range of hydraulic loadings. However, if the daily average input is sufficiently far removed from 160 gpd a decrease in treatment efficiency would be expected.

Although hydraulic loadings ranged from 20 to 195 gpd, significantly decreased treatment efficiencies were not observed at the extremities. Thus, no decision about the possible effects of hydraulic underloading or overloading can be made from these data.

5. Settling

Several attempts were made to measure settleable solids in both the effluent and the mixed liquor. The tests were conducted in 1000 ml plastic graduated cylinders. In all cases, insufficient solids settled to obtain a measurement. However, samples of the supernatant of some of these tests were analyzed. The results are reported in Table 8.

TABLE 8

RESULTS OF SETTLING OF TYPE D EFFLUENT

Sample	Settling Time	Unsettled		Settled		Percent Removal	
#	hours	BOD mg/l	SS mg/l	BOD mg/l	SS mg/l	BOD %	SS %
1	2	89	76	85	20	4	74
2	2	85	84	116	14	-36	83
3	1	92	68	94	25	-2	63
4	1	75	14	76	13	-1	7

These results suggest that for type D plants, where effluent solids concentrations are significant, the effluent could be improved by quiescent settling. However, BOD removals cannot be so affected.

6. Effect of Laundry on Effluent Quality

Automatic washing machines use large volumes of water and discharge wastewater at high rates. Because laundry discharges tend to shock load the treatment plant, an attempt was made to evaluate their effect on effluent quality. Many homeowners found it was necessary to spread their use of the laundry machine over the week to prevent excessive tile field eruption*. However, two housewives restricted almost all of their washing to one day of the week. Table 9 illustrates that laundry use apparently has little significant effect on BOD and an inconsistent effect on SS in the effluent. These data may also indicate that the plants would show good resistance to other shock hydraulic loads.

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*Note: Many homeowners experienced regular (often, essentially continuous) eruption of their absorption field systems. These eruptions are undoubtably attributable to many causes. It should be emphasized that the treatment plant manufacturer, Northern Purification Services Ltd., did not participate in the design or construction of any of the fields surveyed in this study.

TABLE 9

EFFECT OF LAUNDRY USE ON EFFLUENT QUALITY

House #	Wash Day average BOD mg/l	Wash Day average SS mg/l	Non-wash Day average BOD mg/l	Non-wash Day average SS mg/l
1	86	53	83	92
3	65	78	73	67

7. Aeration Pump Failure

One unit was sampled to examine the effluent quality during periods of aeration pump failure. The effects of such a failure are illustrated in Figure 5. The aeration pump in this plant had failed less than 24 hours before sampling began on July 9 and the pump was replaced before sampling started on July 10. The household consisted of two adults, one child and one infant. The failure of the pump was not recognized by a member of the household.

As the Figure shows, BOD increased immediately, but solids concentrations remained low. The low solids level may have occurred because the non-functioning aeration chamber acted as a settling tank. The soluble or colloidal BOD was not settled out and was carried into the 'discharge' chamber through the overflow.

After aeration resumed, the stored solids were probably resuspended by the aeration pump and hence resulted in the high solids concentrations on July 10. Some increase

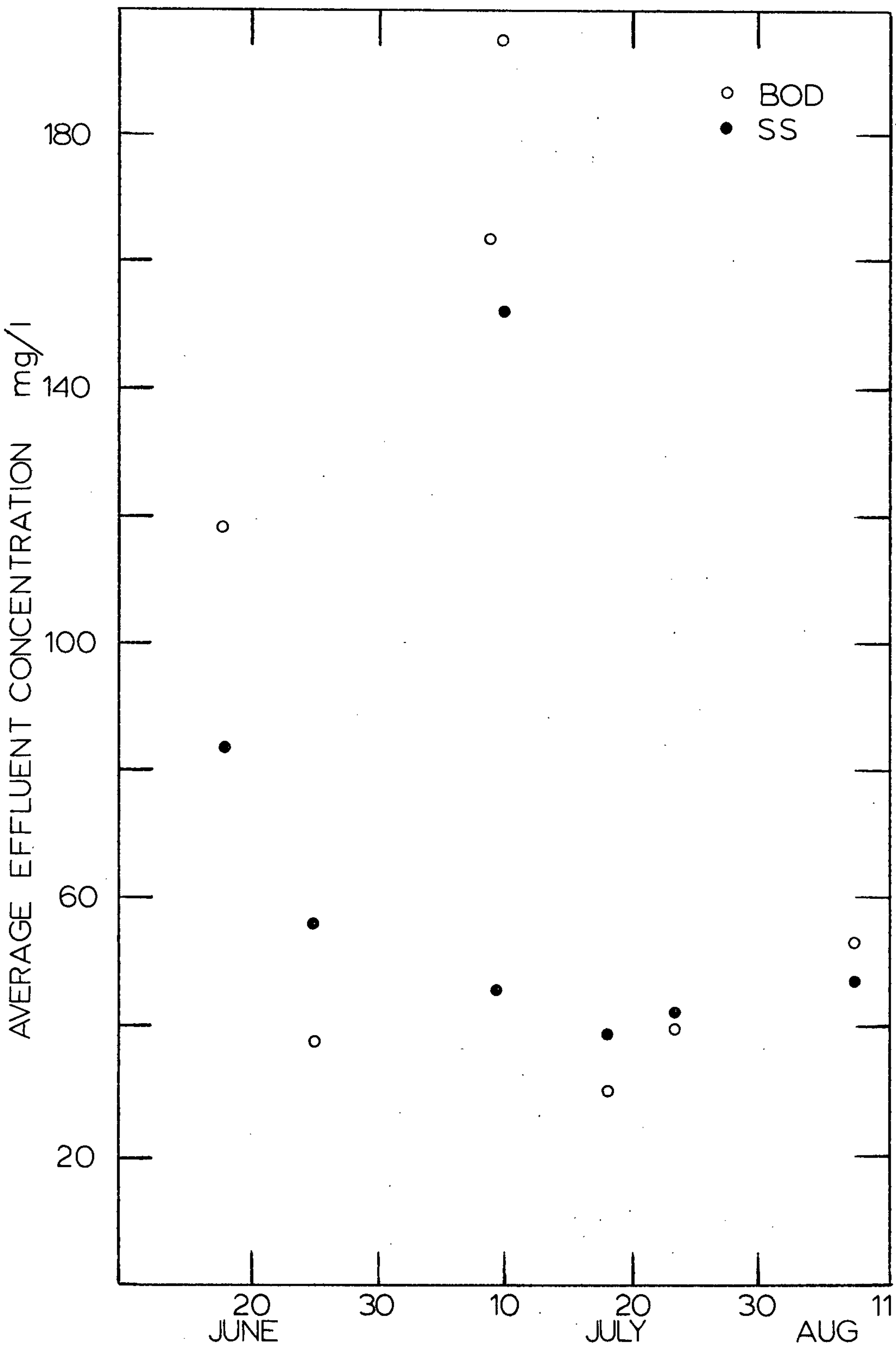


FIGURE 5 - EFFECT OF AERATION PUMP FAILURE ON EFFLUENT QUALITY

in BOD was also noted on July 10, probably indicating the BOD associated with the resuspended solids. By July 18, effluent quality had returned to normal.

The quick increase in BOD concentrations (it took less than 24 hours to reach 160 mg/l) emphasizes the need for early recognition of failure as well as the necessity of prompt aeration pump replacement. The very high BOD concentrations exhibited during failure periods could lead to severe oxygen deficits in small disposal fields. In addition, after a period of dosing with high BOD effluent, the field would receive two or three doses of effluent with very high SS concentrations. This may lead to conditions from which the field could not normally recover.

8. Other Data

It is very difficult to compare the operational and effluent characteristics of the many household waste treatment units on the market. In fact, very little data, collected under field conditions similar to those of this study, is available.

The only other effluent data available for the type D Cromaglass plants is that provided by the manufacturer. In addition to controlled testing, they sampled two plants which had been installed and maintained under factory supervision at private dwellings. After sampling each plant during six discharges over a two month period, they report the data shown in Table 10.

These significantly better results may be explicable in terms of the much improved (over general field practice) installation and maintenance procedures used by the factory personnel. Certainly, this quality of effluent is in an entirely different range in terms of tile field length requirements, than that reported earlier in this chapter.

TABLE 10

NPS EFFLUENT DATA ON CROMAGLASS TYPE CA5-D*

House	BOD, mg/l			SS, mg/l		
	high	average	low	high	average	low
A	60	34	21	115	60	15
B	51	36	18	47	33	23

* Data obtained through private communication with Northern Purification Services Ltd.

B. Effluent Quality: Type CA5-E

The sampling procedure discussed in the first pages of this chapter was modified slightly for the type E plants. Based on the improvement in quality obtained by settling the type D effluent, it was assumed the settling period would significantly improve the type E effluent. In order to obtain a sample of the best possible effluent quality, samples were taken closer to the liquid surface in the settling tank. In theory then, this "supernatant" sample would be at least as good, if not better than, a sample taken from a greater depth near the discharge pump inlet. However, the sampling point was far enough below the surface that floating materials were excluded.

1. Effect of Settling

As noted earlier, quiescent settling significantly reduced SS in the type D effluent. This improvement was not generally observed during the two hour settling period of the type E plants. Samples, taken at 15 to 30 minute intervals, as shown in Table 11, generally exhibited increased, not decreased, SS concentrations.

TABLE 11

EFFECT OF SETTLING IN CA5-E PLANTS

Effluent at Cessation of Aeration		Effluent after approx. 2 hours of settling	
BOD	SS	BOD	SS
70	57	76	80
31	16	33	41
34	35	45	31
76	22	94	39
97	54	105	51
66	65	65	63
72	48	40	61

One explanation, for the poor solids removal performance, may be related to the observation of floating solids in those plants which became anaerobic before discharge. This may indicate that a resuspension of the solids by rising gas was occurring. The increased or unchanged BOD concentrations are to be expected from the type D test results. Certainly, the quiescent period did not appear to consistently improve effluent quality.

The VSS to TSS ratio exceeded 0.875 and averaged 0.953 for the type E units as compared with an average of 0.788 for the type D plants. This may indicate that the settling period does decrease the concentration of inorganic SS relative to the organic solids in the effluent.

As with the type D plants, the inability of the settling tank portion of these units to remove solids from the mixed liquor means that little or no solids will be returned to the mixed liquor.

2. <u>Dissolved Oxygen</u>

As noted for the type D plants, dissolved oxygen concentrations were more than adequate during the aeration cycle. Continuing biological activity caused a decrease in D.O. concentrations during the settling period. In plants serving more than four persons, D.O. levels below 1.0 mg/l were measured at the end of the settling period. These low D.O. concentrations in the effluent put an extra stress on the tile field. In addition, the consequent odor is the source of complaints.

C. <u>General Observations</u>

In addition to the specific data on effluent quality, thus far discussed, some general observations were made.

Most of the houses to which the test units were connected had plumbing in the basement. To obtain gravity flow to the plant, it was placed in a large manhole or wooden box as much as six feet below ground level. Usually this meant that the tile field was located several feet above the discharge level of the plant. As the total volume of fifty feet (many of the tile fields had fifty feet of distribution pipe) of four inch diameter tile (27 gal) is only half the discharge volume of the unit (48 gal), a significant portion of the discharge would flow back into the unit after pumping stopped. This resulted in some units discharging as often as every ten minutes during high water use periods. Excessive pump wear is then due, not only to frequent operation, but also to the pumping of inorganic solids washed into the tank from the field.

The problem could be alleviated by installing a check valve between the pump and the distribution box. Alternatively, a twelve to eighteen inch drop from a high point in the discharge line to the distribution box could be constructed.

The location of units below ground level also led to ground water infiltration problems. Several units were observed with ground water showing between the access manholes. During high ground water periods a number of these units could easily be flooded. Drainage of the area around the plants is required to prevent overloading the discharge pumps. Additionally, collars, bonded to the plant and projecting to ground level, could be installed.

The failure of aeration pumps, if not quickly corrected, can be seriously detrimental to small tile fields. In almost all cases of aeration pump failure, the problem was first recognized by the technician doing the sampling or by the householder, only after the anaerobic odor had become intense. At no time were the warning systems responsible for the recognition of failure. Some units went as long as five or six days before the householder became aware of the problem.

If tile field length is to be minimized, a better system of pump failure warning is needed.

In respect to servicing, one of two policies must be adopted. If the present frequency of professional service is all that can be economically provided, the householder must be taught to inspect and service his own unit. Most importantly, he must be able to recognize and correct clogged aeration pumps and to maintain the cleanliness of the unit. There is no question that without this owner participation, the present service frequency is inadequate. The other procedure would exclude the householder almost completely and increase the dealer service to monthly at least. Monthly service may still be insufficient. This policy would require a good alarm system and the cooperation of the householder in responding to it.

One of the problems in analyzing the operation of the plants is the number of missing or damaged components and the variation in arrangement of pumps and chambers. Several plants had broken or leaking transfer lines which led to spray aeration and high dissolved oxygen levels. Missing transfer lines meant that the mixed liquor was jetted into the settling tank. This created more than normal turbulence and undoubtedly decreased settling efficiency. Different depths of the discharge pump in the settling chamber meant that some units scoured the bottom of the settling tank more thoroughly during discharge. This may have affected solids levels in the effluent.

In addition to sampling the treatment plants, approximately 30 disposal fields were inspected visually. These inspections were supplimented by interviews with the current householders. Of the thirty fields, 13 were reported to erupt regularly. These eruptions were generally confirmed by inspection. Four other householders had not occupied their houses long enough to comment. Thus, 13 of 26 fields were erupting regularly. At least four homeowners had installed major modifications to their fields. Undoubtably, there are a number of explanations for these failures, not the least of which is poor construction techniques.

Another major complaint was that of effluent washed out of neighbouring tile fields onto the adjoining lot. This was an acute problem in some areas during the high ground water level periods in the spring. Apparently, little has been done to isolate individual tile fields.

Two tile fields which had been built from gravel pit rakings, as prescribed by the Health Branch, were sampled for soil analysis. The probable drainage characteristics of both fields were described as "poor". Both fields erupted during the summer. However, one of the fields erupted only after the house had been occupied by ten people for one week. Apparently, another source of tile field soil should be recommended.

SUMMARY

In general, the data suggest that the plants installed under the inspection, supervision, and construction conditions prevailing in the last two years, are not capable of producing an effluent in the 30 to 40 mg/l BOD_5 range. In Chapter 4, it will be noted that literature suggests this relatively high quality effluent is required before significant deviations from current disposal field design practice can be permitted.

CHAPTER 4

DISPOSAL SYSTEMS - STATE OF THE ART

INTRODUCTION:

The survey described in this chapter was undertaken to review the current state-of-the-art in the design of absorption field disposal systems.

The premier question to be considered in this regard is: What is the purpose of a tile field? Conflicting opinions exist between those who are primarily concerned with keeping effluent below the ground surface (wastewater disposal) and those whose first priority is additional treatment. Certainly, both features are important.

This question could be considered by dividing tile fields into two categories. In one group are those fields which serve houses receiving municipal water supplies. The problem of groundwater pollution is then not of concern in the public health context unless the groundwater surfaces near the tile field. However, pollution of surface waters can occur. Many Ontario lakes (3) and the Okanagan lakes (20) in B.C., among others, are significantly affected by nutrients carried from soil disposal systems to surface waters by groundwater.

The second category would include those dwellings relying upon well water taken from a site relatively close to the tile field. Here groundwater pollution is an important factor. The current, generally recommended, minimum distance of one hundred feet from a tile field to a well may, in some soils, be insufficient to prevent water supply contamination (19).

One could carry on a discussion and formulate regulations for the two categories separately. However, many authors have warned of the undesirable and often unpredictable effects of blatant groundwater pollution (19, 20). This chapter will then, concentrate on systems which provide both protection of groundwater quality and adequate long term hydraulic capacity.

The discussion is oriented primarily toward absorption fields constructed in *in situ* soil or specified imported fill. Fields constructed with special facilities for introducing air are touched on near the end of the chapter.

It is important to clarify some basic concepts in a discussion of liquid application to soils. The following definitions will help to put the discussion on a common footing.

From McGauhey and Winneberger (17):

Porosity: is the fraction of the total soil volume which is void. In other words, it is the proportion of the soil volume which can be occupied by air and/or liquid.

Perviousness: refers to the size distribution of the void spaces involved in porosity. It is, however, measured by the rate of passage of liquid under standard conditions. This measure is called the permeability (or hydraulic conductivity, K) of the system.

The *infiltrative capacity* is the rate at which liquid will pass through the soil-water interface.

The *percolative capacity* is the rate at which water moves through the soil once it has passed the interface.

Both the infiltrative and the percolative capacity can be limiting. However, in tile field design, it is desirable to maintain a higher percolation than infiltration rate. This is necessary to prevent the problems associated with soil saturation discussed later.

Many factors have been shown to have a significant effect on the rate of passage of liquids into and through soil. Among these are

1. rate and pattern of application
2. chemical and biological composition of the liquid
3. temperature
4. soil texture, structure and composition
5. soil depth
6. depth to groundwater (and fluctuations of this depth)
7. soil moisture tension (degree of saturation)
8. slope of application surface
9. soil permeability, porosity and homogeneity
10. presence of swelling clays

Many of these factors are a function of the soil texture as it relates to pore size. As water is transported through the soil pores, the size and number of these pores has a great effect upon the water transport rate. Thus, any change in the number or size distribution of the pores will affect the rate of water transport. Adverse changes in soil pore size or distribution are termed clogging.

I. Soil Pore Clogging

There are three fundamental types of soil pore clogging; Physical, Chemical and Biochemical.

Physical Clogging

Physical clogging is due primarily to poor construction techniques. To a very large degree, this problem could be reduced by education of contractors, inspectors and householders. It can probably be blamed for the early failure of many otherwise adequately sized tile fields.

McGaughey and Winneberger (17) have summarized the causes of physical clogging as follows:

1. Compaction of the soil by superimposed loads
2. Smearing of soil surfaces by excavation equipment
3. Migration of fines by vibration of dry soil during site preparation
4. Migration of fines due to rainfall beating against surfaces
5. Washdown of fines perched on larger particles.

To these might be added:

1. Movement of fines by water pressure during high volume discharges to small tile fields.
2. Fines moved into and onto the field by flooding
3. Suspended solids in the applied effluent
4. Swelling of some clay fractions (a physio-chemical effect).

The most important effect of the physical clogging mechanism is the sealing of the trench bottom. Vibration and water flow carry the fines to the bottom of the trench where they contribute to the sealing of this surface. Thus, where even relatively small amounts of fines are present, their concentration on the bottom of the trench effectively eliminates this surface as an infiltration area.

It is apparent from the mechanisms of this type of clogging that it can be largely eliminated by:

1. Taking care during construction to avoid heavy loads on the site surface, minimize or correct smearing and minimize vibration.
2. To inspect the site soon after the tile is laid to prevent leaving the trenches open to the weather for extended periods.
3. Prevent groundwater flooding.
4. Maintain a low suspended solids in the applied effluent.

With care, early field failure due to physical clogging should be relatively rare.

Chemical Clogging

Chemical clogging is usually insignificant or prohibitive. The cause of most chemical clogging is related to the sodium exchange capacity of some clays. The addition of excess sodium causes these clays to become structurally unstable and pore clogging results. In most locations, this problem does not exist. However, where it is encountered, it usually means either sodium removal must be practiced, or *in situ* seepage beds are impossible.

It is interesting to note that chemical is the only form of clogging which can be estimated in laboratory experiments.

Biological Clogging

Biological clogging is primarily due to the growth of bacteria in the soil. This type of clogging can occur in two forms, depending on the soil particle size and the organic loading rate.

Surface clogging, in which a biological mat less than two centimeters thick forms, generally occurs in soils with an effective particle size of less than 0.1 mm regardless of the organic loading (17).

In coarser soils, a relatively permeable layer of biological growth has been observed (24, 26). The depth of this layer ranges from 5 to 35 cm. Although this type of formation reduces the infiltration rate, its presence often indicates the possibility of establishing a relatively good long term infiltration capacity. That is, the long term capacity of the system will be a relatively high percentage of its initial rate.

The destruction of biological mats is carried out by rotifers, ciliata and other higher organisms. These organisms do not function well under anaerobic conditions. Consequently, the extent of biological clogging is affected by oxygen content of the immediate soil.

The oxygen content of the soil is influenced by

1. oxygen content of the applied effluent
2. soil texture and pore size distribution
3. temperature
4. depth of the field
5. applied organic load

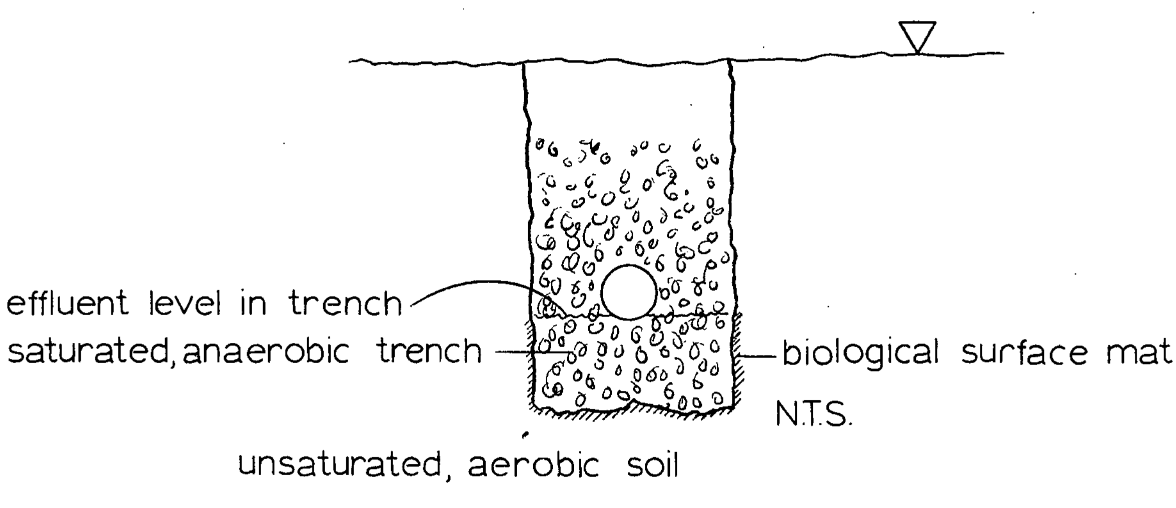
6. soil moisture tension

among others.

In order to maintain the pyramid of higher organisms necessary to control bacterial proliferation, it is essential to maintain aerobic conditions.

In addition to the clogging due to the presence of large numbers of bacteria, significant clogging occurs through the deposition of ferrous sulphide. Formed under anaerobic conditions, this compound is deposited as a slightly soluble crystal on the inside of the soil pores. Aerobic conditions are needed for its elimination.

Biological clogging is initiated by the organic overloading of the soil surface. The normal soil bacteria population expands rapidly in response to this increased food source. The presence of suspended solids in the effluent can cause immediate clogging of some pores. If the organic loading is maintained at a sufficiently high rate, either intermittently or continuously, the proliferation of bacteria will continue. Eventually a more or less stable population which is in equilibrium with the organic load is reached. The net effect is illustrated below.



As the infiltration rate is greatly reduced by the mat, the liquid which passes through the surface quickly percolates away. This leaves the soil under and around the trench aerobic and unsaturated but the contents of the trench remain anaerobic. Usually, even if the applied liquid has a significant D.O., it will quickly become anaerobic in the trench.

Thus, in soils where surface mat formation is likely to occur, every effort should be made to introduce as much air into the system as possible.

In most conventional septic tank systems, the field is dosed by gravity induced trickle flow. This flow tends to overload each increment of the field sequentially and thus induces rapid failure. If the organic load could be spread over the entire effective surface area of the field at once, the organic load per unit area would be minimized. This would result in a minimum demand on local oxygen reserves and a minimum production of bacteria per unit area. Biological clogging would be thus reduced and field life extended.

Obviously, higher dissolved oxygen levels and lower biochemical oxygen demands in the effluent would decrease the amount of anaerobic clogging. It has been illustrated (6) that the deposition of suspended solids from the effluent is important in initiating clogging. Thus, effluent quality is important from the first day of operation.

Even in beds which are dosed intermittently, the deposition of suspended solids on the trench bottom, in conjunction with the physical clogging discussed earlier, effectively seals this surface (except in very coarse soils).

Consequently, after a short period of operation, almost all of the applied effluent is being processed through the sidewall area of the trench.

In fine soils, the best operation, in terms of maintaining infiltrative capacity, may be to dose the field evenly and then wait until the applied organic material has been assimilated by the bacteria. The next dose would be applied after the bacterial population had been reduced to its initial level. A large effluent storage facility and a very large absorption field, in most soils, would be required to handle the wastes from one household, if this operation were adopted. This type of operation is approached in pump and siphon dosed fields. However, normally the hydraulic load is such that dosing must be carried out more frequently than this ideal operation.

One alternative suggestion (17) has been to construct a field of relatively coarse imported material. Here the build-up of a relatively large bacterial population leads to an in-depth biological layer. The surface crust is avoided yet sufficient bacteria are present to provide relatively good treatment. This sand bed technique also requires provision for good air circulation.

Other methods of reducing biological clogging include the alternating use of two or more beds and dosing field sections in sequence.

McGauhey and Winneberger (17) have suggested surface biological matting can be expected to be infiltration rate controlling in soils with an effective particle size of less than 0.1 mm (fine sand). Clogging occurs in these soils even at very low organic loadings and when apparently aerobic conditions exist. If soil analyses indicate that this effective particle size (0.1 mm) is not exceeded, they conclude that a conservative design hydraulic loading rate is 0.03 ft/day based on sidewall area.

Bouma (5) observed this surface clogging phenomena in much coarser soils, up to 4 mm in size. However, crusts in these coarser soils appear to maintain infiltration rates of 0.10 to 0.26 ft/day. McGaughey and Winneberger (17) have suggested using a conservative design figure of 0.06 ft/day in these soils.

Thomas (23) suggested that the long term ability of a soil to accept effluent may be controlled by the rate of oxygen diffusion to the loaded surface. His idea supports the contention that it is the biological mat which controls the infiltration rate and not the soil percolation characteristics *per se*.

Biological clogging may be accelerated by other affects. De Vries (26) reports on the effects of cold temperatures on surface dosed sand filters. Biological clogging, which did not occur at room temperature, was induced quickly at 4 °C. These results may be an indication of the inability of the biological system to process heavy organic loads at low temperatures.

Groundwater inundation of a previously unclogged bed can lead to the anaerobic conditions which cause clogging. Recession of the water table does not necessarily mean that the clogged bed has the ability to regenerate itself while still receiving effluent.

The consensus is that all seepage beds will suffer a loss of infiltration capacity with time. It has been shown (5, 17, 24, 26) that much of this lost capacity can be regained by 'resting' the bed, that is, by allowing the bed to drain and dry and allowing time for the accumulated organics and bacteria to be oxidized. The diffusion of air into the system will also oxidize any ferrous sulphide deposits. Not all of the lost capacity will be regained because of the deposition of resistant organic and inorganic materials.

The question of how long a rest period is required before a field is returned to operation is impossible to answer. Some of the factors which influence the rate of recovery are the amount of organic material accumulated in the bed, the rate at which the bed drains, rate of oxygen diffusion into the bed, temperature, moisture tension of the surrounding soil and so on. Rest periods of anywhere from a week to a year have been suggested for various values of the above mentioned factors. If the bed is to be operated on an intermittent basis, another disposal site will be required to operate in the intervening period.

Thus, in order to minimize failure through organic clogging, the following suggestions might be implemented:

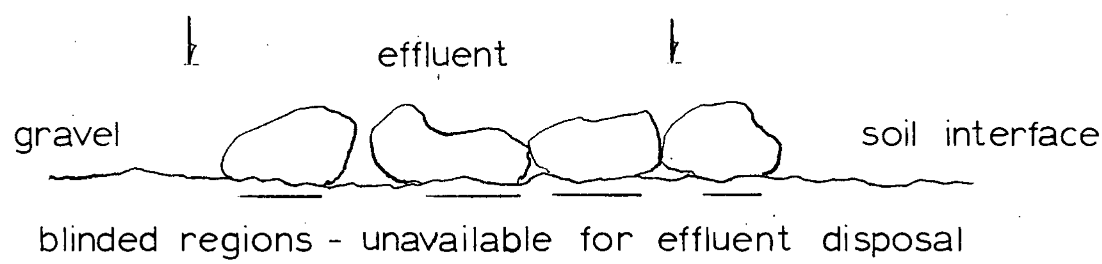
1. maintain an aerobic effluent, low in BOD and SS
2. discharge in such a fashion that the whole field receives an equal organic load per unit area

3. maintain aerobic conditions in the field by
   a. good bed design and construction
   b. allowing the field to rest periodically
   c. preventing groundwater inundation
4. constructing trenches with a high sidewall area to volume ratio

Effective use of each of the above criteria would increase both capital and operating costs above present day practice. Their implementation must be weighed against the cost of merely installing a very much overdesigned tile field.

One other point relevant to field design and clogging should be recognized. Most regulations stipulate that the trench around the distribution pipe be filled with gravel. This practice creates an abrupt change in particle size at the soil interface. Effluent moving away from the pipe receives little filtering until it reaches the soil interface. The total organic load is then concentrated on the interface.

A second result of this practice is the blinding of the soil surface as illustrated below.



Both of these conditions could be avoided by having an evenly graded particle size from the pipe to the wall. Although impossible, in a practical sense, a two or three layered system might be used.

This short description of some of the mechanisms of disposal field operation leads to the more practical questions of field design and sizing.

II. Tile Field Design

Tile field design is usually based upon the percolative capacity of the soil. As discussed previously, this is often influenced by the organic loading.

Traditionally, most regulatory agencies have defined the design hydraulic capacity as 0.5 to 5 percent of the percolation rate as measured by a percolation test (see Appendix 1). Table 12 contains some suggested figures.

As emphasized earlier in this chapter, the most important single factor affecting field design is usually the limitation of the infiltration rate by the biological mat. The literature generally suggests that it is inadvisable to design fields on the basis of maintaining aerobic conditions throughout the field. It is felt that the degree of reliability and control needed to maintain aerobic conditions in the field is not readily available. In addition, recovery from upsets or environmental influences can be slow.

Most authors agree that *in situ* fields are not suitable for soils with a percolation rate slower than 30 min/in. Also many authors, recognizing potential groundwater pollution problems, suggest rates faster than 3 min/in be modified unless there is a great depth to groundwater.

An acceptable loading rate will depend on the desired life of the system (7), overdesign factors, effluent

TABLE 12

PRESENT DESIGN SUGGESTIONS FOR SEPTIC TANKS

PERCOLATION RATE VS SUGGESTED HYDRAULIC LOAD

Percolation Rate min/in	Suggested design hydraulic loading Rate Igal/ft^2/day								
	Source 1	2	3	4	5	6a	6b	6c	7
1	0.56	0.52	2.08	0.75	0.25		1.85	1.39	1.47
2			1.67	0.75	0.25				1.47
3			1.33	0.75	0.25				1.47
4			1.17	0.75	0.25				0.76
5	0.54	0.46	1.08	0.75	0.25	1.05	1.04	0.78	0.76
10	0.50	0.38	0.83	0.75	0.25		0.79	0.60	0.76
15	0.45	0.31		0.75	0.25				0.50
20	0.41	0.25	0.60	0.75	0.25		0.62	0.46	0.50
30	0.35	0.17	0.40	0.75	0.25	0.42	0.52	0.39	0.50
40			0.35						0.42
45	0.28	0.08							0.42
60			0.30						0.37
90	0.15								
120	0.04								

1 Benhart (2) data for aerobic effluent
2 Benhart (2) data for anaerobic effluent
3 Winneberger (25) Arizona
4 Huddleston and Olson (13) 3% of percolation rate
5 Huddleston and Olson (13) 1% of percolation rate
6 B.C. Health Branch (calculated based on different areas)
7 Wisconsin Guide as substantiated by Bouma (5)

Note: McGauhey and Winneberger (17) make the following loading rate suggestions

effective soil size	hydraulic loading rate Igal/ft^2/day
$<$ 0.1 mm	0.10
0.1 to 1.0 mm	0.14
$>$ 1.0 mm	0.21

quality, reliability of the preceding treatment and other factors. Some authors suggest, and this author concurs, that because of the relatively low cost of _in situ_ field installation (and even installation in imported fill), that very conservative design loadings should be used. As Table 12 indicates, McGaughey and Winneberger have proposed the most conservative rates. The present Provincial Health Branch regulations, depending upon how they are calculated, appear to be similar to other regulatory agencies. If different, they tend to permit somewhat greater hydraulic loadings. Also, they are much less conservative than McGaughey and Winneberger's recommendations.

However, if the field size is restricted by lot size or if soil conditions are very poor, improved effluent quality will be required to permit a significant increase in loading rates. Even the rates suggested above for "full size" septic tank tile fields have not had complete success. The question of how much improvement in effluent quality is required, before significant reductions in the field length can be made, has not been adequately examined in the literature.

Thomas (24) observed surface clogging of sand filters when loading them at a rate of 0.67 ft/day (4.2 gal/ft^2/day). The average quality of the applied effluent is shown in Table 13. Infiltration rates declined from 3 min/in to 1500 min/in. (Note in Table 12 that the application rate is within the suggested range for absorption beds based on the initial infiltration rate). The sand had an effective size of 0.2 mm.

TABLE 13

APPLIED EFFLUENT QUALITY - THOMAS (24)

parameter	Thomas' value (average) mg/l	Cromaglass type D mg/l
COD	198	218
BOD	87	85
SS	50	79
VSS	40	
NH_3-N	24	22
ORG N	6.7	11.8
pH	7-8.1 units	7-7.9 units
T °C	2-21 °C	22-31 °C

It should be emphasized that the applied effluent was of similar quality to that of the Cromaglass type D plants. The major difference to be noted is that the Cromaglass effluent has a higher dissolved oxygen concentration. The effect of the increased D.O. may be more than offset by the greater tendency of underground systems to clog. Also, the effluent in Thomas' experiment was applied only once per day. From these data, it would appear that effluent of this quality cannot be applied at this rate without inducing clogging.

Other authors (2, 22) have suggested that average BOD and SS concentrations as low as 30 to 40 mg/l would be required before significantly higher loading rates are permissible.

When considering beds constructed in imported material, it is very important to measure infiltration rates after the material has been placed. Disturbing "natural" soil can reduce its infiltation and percolation rates from 20 to 500 times. Additionally, if the imported soil is placed on a relatively impermeable subsoil, sufficient storage must be provided in the imported fill to prevent eruption during periods of high loading. In coastal British Columbia areas, this may mean considerable depths of fill, below the actual seepage bed, will be required.

There appears to be no reason for increasing the loading rates in imported material over those for _in situ_ material of the same percolation rate.

Most authors caution against decreasing field size on the provision that aerobic conditions are maintained. As discussed earlier, organic matting occurs even under 'apparently aerobic' conditions. Once anaerobic conditions are initiated, they spread quickly and are slow to be eradicated. For _in situ_ beds and fields in imported fill, no allowance should be made for aerobic conditions.

Another response to constrained conditions for wastewater disposal on land, in addition to improved treatment, has been the development of some 'high rate' disposal systems. Two types of systems have been developed; sand beds, and evapotranspiration fields.

The sand bed consists of a small tile field, laid entirely in an excavation filled with sand. The effluent is applied to the field at a high rate by discharge through perforated tile near the surface of the field. The sand serves two functions. Firstly, it provides a large storage volume so that the applied effluent does not erupt, but instead, can percolate away constantly through the poorly permeable subsoil. Secondly, the sand acts as a biological filter and reactor to remove clogging constituents. Usually, special provision is made to introduce air into these beds.

The evapotranspiration field functions much the same as the sand bed. However, instead of relying on slow percolation through the subsoil, liquid is removed from the fields by evapotranspiration through a vegetative cover. In fact, sometimes these fields are sealed with a plastic sheet on the bottom.

However, both of these systems require a sufficiently high quality effluent such that clogging of the sand around the distribution pipes does not occur. Design criteria for these fields are not well documented and significant disparities exist. For example, Bernhart (2) suggests an evapotranspiration rate of 32 inches per year can be expected in Vancouver, but Oldham (21) estimates this figure to be 15 to 20 inches per year. Data on sand beds is similarly divergent.

SUMMARY

This brief review of some of the literature available on disposal field design indicates the widespread lack of information about design criteria. The mass of information which has been published of late on large scale surface disposal techniques, also indicates, to a significant extent, the lack of understanding of the mechanisms involved in ground disposal. Recognizing these problems, regulatory agencies have usually specified the use of conservative design figures.

The relatively recent demand for more "highly stressed" wastewater control systems has not been adequately researched. Few authors have ventured an estimation of the required improvement in treatment efficiency needed to allow increased disposal facility loadings. Even, with what is presently considered to be conservative design, a large number of systems fail. Until a better understanding of the operation of disposal fields is gained and a better procedure for field analysis of potential disposal areas is developed, the use of highly loaded fields will result in a significant percentage of failures.

CHAPTER 5

SUMMARY AND RECOMMENDATIONS

I. Introduction

It is not difficult to conclude that the effluent quality results of this survey indicate that the Cromaglass package plants are not consistently producing a high quality effluent. Whether these results are interpreted as being representative of average field operation may be open to question.

Every effort was made to exclude other than normal field conditions during the sampling periods. The householders were very cooperative in carrying on normal activities while the survey was underway. The regulatory and service agencies made no extra effort to review the plants; nor did the manufacturer call for any changes while the study was in progress. The manufacturer later stated that several of the units sampled were either incorrectly assembled or inadequately installed. Whether the same percentage of improperly functioning plants can be expected throughout the province is difficult to say.

The households participating in this study were largely middle class. The 1974 market value of these houses was in the $45,000 to $60,000 range. People in other socio-economic groups may have sufficiently different habits such that effluent quality may be improved, but this is open to speculation. However, it is unlikely, for economic reasons, that Cromaglass plants would be installed in very much less expensive houses.

All of the households surveyed, except one, were located in a single subdivision. However, the one plant from outside the district exhibited the same characteristics and problems as those within.

If it is necessary to obtain more information, to verify whether the results presented are indeed representative, two approaches could be used. It may be possible to repeat the work done during this project in a subsequent year. Alternatively, a similar program in another area might be undertaken. However, now that an investigation has been initiated, one of the most important aspects of further study will be to insure that normal field conditions and procedures are maintained.

As this study was largely independent of manufacturing and regulatory concerns, the conclusions presented below are those of an unbiased observer.

II. Unit Operations

It is difficult to formulate conclusions concerning unit operations from the data available. Some points can, however, be made:

1. Treatment does not appear to be effected through the activated sludge mechanism.
2. The settling tank in the type D plant appears to perform no treatment function.
3. The settling tank in the type E plant appears to perform no treatment function.

4. In dwellings occupied by more than four people, anaerobic conditions can often be expected at the end of the 2 hour settling period of the type E plants.
5. Based on literature values for the influent loading of extended aeration plants, it is evident that, on the average, the plants are underloaded. The auto-oxidation of microorganisms in underloaded units may be a major cause of the very low MLVSS concentrations. In order to increase organic and solids loadings, it may be desirable to require the installation of garbage grinders in houses with these plants. However, as seventy-five percent of the influent load is often received in a six hour period, conservative design is probably justified.

III. Effluent Quality and Disposal Systems

In light of the poor understanding of current absorption field design criteria, the literature indicates that effluent quality must consistently be below 40 mg/l BOD and 50 mg/l SS to permit significant reductions in present day septic tank absorption field requirements. The effluent quality, of neither the CA5-D nor the CA5-E plants surveyed in this study, was sufficiently high to allow any reduction in the size of _in situ_ or imported fill beds. Because of the accelerated rate of organic degradation in sand beds or certain specially constructed fields, some reduction in field length may be permissible.

IV. General Observations

1. As mentioned earlier, many aeration pump failures were noted during the summer of 1974. Since that time, the manufacturer has changed to a pump which he feels will provide the necessary service. However, this problem does underline a basic difficulty with any mechanical device - the necessity of maintaining parts inventories and supplies. The logistics of a back-up service to several thousand plants is considerable.
2. The frequency of service inspections needs to be increased, either through greater owner participation or more service inspections.
3. Regardless of how the service frequency is increased, a better alarm system is required.
4. Installation of a device to prevent back flow from the tile field to the unit should be mandatory.
5. Units installed below ground level require a good drainage network to prevent flooding of the unit.
6. Tile fields should be isolated from the surrounding area by collector drains.
7. At present, the following people are normally involved with the treatment and disposal system:
   1. manufacturer
   2. sales and service dealer
   3. installer of the treatment plant
   4. installer of the tile field (and subcontractors)
   5. contractor for the house
   6. Health inspector

7. householder (often inherits the product of the foregoing)
8. householder's lawyer

If a reasonable degree of control over effluent quality, service, and installation is to be maintained, some of these people must be eliminated. From the point of view of the manufacturer, regulatory agency, and the householder, it would be desirable to have a single concern who would be responsible for the installation and maintenance of the entire treatment and disposal system. In other words, the systems approach to the entire waste-management scheme is needed.

8. Under the present system, "one-shot" health inspection is not sufficient to ensure that health regulations have been met by all of the various contractors involved in the system installation and operation.
9. As Chapter 4 indicates, the soil absorption system is a complex, rather sophisticated device, which, if it is to operate at high rates, requires very careful construction techniques. The education of contractors and inspectors, to the degree of care needed in the preparation of these fields, is most important. The hap-hazard construction witnessed during this study only reinforces the contention that many field failures are probably due to poor construction technique.
10. Material recommended as fill for tile field construction needs to be carefully examined, not only _in situ_, but after removal, transport and reforming.

V. Householder

As a general policy, it is probably conservative to exclude the householder with respect to service, care, and management of the treatment and disposal systems. Obviously, cases do exist in which the householder is fully capable and willing to service his system. However, many homeowners "inherit" the plant from a contractor or previous owner with little knowledge of, or interest in, the system. In order to minimize problems throughout the life of the plant, it is strongly suggested that any system be as independent as possible from the householder, with respect to installation, operation, and maintenance.

SUMMARY

Regardless of the somewhat negative conclusions of this report, there remains a demand for high efficiency wastewater treatment in outlying areas. The theory of on-the-lot systems is fundamentally sound. It is based on a philosophy of low cost and little maintenance, justified on the basis of large overdesign factors. When these overdesign factors are eroded, in order to remain within the constraints of lot size and ground conditions, cost and maintenance requirements will increase. The recommendations discussed above must then be considered in light of their affect on the cost of wastewater management (14, 23). Obviously, the desire of rural Regional Districts to have individuals, rather than districts, assume wastewater management costs will be a major factor in this analysis.

However, the fundamental conclusion of this study, that improved treatment performance is apparently not yet capable of off-setting the problems associated with high-rate disposal systems, remains. Until these problems can be overcome, no significant relaxation of tile field overdesign factors should be permitted.

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APPENDICES

APPENDIX 1 - Percolation Testing

As noted in Chapter 4, one of the greatest difficulties in the design of disposal fields, is the lack of accurate design parameters. This problem is largely due to the inability to relate long term tile field operation to a simple, reliable field test procedure. For many years, the percolation test has been used to categorize soils.

The "percolation test" was first used by Ryon in the mid 1920's. He proposed and utilized a relatively standard test, in conjunction with some field failure data, to predict the suitability of various soils for seepage field installation. His test, much maligned and poorly understood, remains as virtually the only measure for field design in widespread use today.

However, it is almost universally agreed that this non-specific, poorly standardized, inadequately understood, often one-shot test has no simple correlation with the long term ability of a soil to accept effluent. Most authors feel that the test, properly performed, under standard conditions, will allow a very rough categorization of soils to be made. However, even when testing is carried out under very closely controlled conditions, by skilled technicians, the variability of the results make them difficult to interpret.

An examination of some of the inherent problems of the "standard percolation test" (SPT) will serve to underline these difficulties.

The SPT is usually one of two basic tests. The most common, the falling head test, apparently is less reproducible.

The static head test, although highly variable, is somewhat better.

At present, most agencies do not strictly define how a percolation test should be carried out. Depth and diameter of the hole, water levels, number of fillings before measurement, weather conditions, number of holes per site, quality of water used in the test, number of days upon which the test must be performed, and experience of testing personnel are but loosely outlined. The importance of careful definition and control of these parameters is illustrated in the literature.

Healy and Laak (11) compared a mathematical model with laboratory and field data. They make the following conclusions.

1. Percolation rate will not decrease significantly after two fillings of the test hole.

2. A percolation hole in any soil except sand or gravel will yield a wide range of percolation rates depending on the degree of saturation of the soil deposit at the time of the test. Confirmed with both field and laboratory testing.

3. Test hole shape can significantly affect percolation rates (up to factors of 6 or more).

4. Depth to the water table has a great effect on percolation rate even after the soils have been soaked twice (for water tables less than 6 ft.).

5. Percolation rates varied from 20 to 90 times the "permeability" (hydraulic conductivity, K) in laboratory tests. The lower the P.R./K ratios occurred at the faster percolation rates.

6. Disturbing the soil resulted in a permeability decrease of 10 to 200 times.

7. The saturated permeability was from 2 to 40 times greater than the unsaturated value. Laboratory data indicate that K_{sat} is probably 2 to 3 times K_{unsat}.

8. "The percolation rate is not a measure of the effective absorption properties of a soil deposit and should not

be used as a basis for determining allowable hydraulic loads on seepage beds.

Bouma (4) presents Table A1-1 as a comparison of some test methods carried out under standardized conditions (coefficient of variability for each test site in brackets)

TABLE A1-1

Soil type	Percolation Rate cm/day Falling Head	Percolation Rate cm/day Static Head	K_{sat} cm/day
silt loam	160 (41)	360 (35)	60 (25)
silt loam	200 (45)	600 (30)	28 (25)
silt loam	420 (82)	600 (30)	95 (30)
silt loam	1200 (30)	1200 (20)	80
loamy sand	1600 (30)	1800 (20)	300 (20)

Apparently the most reproducible test is the _in situ_ saturated hydraulic conductivity measurement. He also concludes that the percolation rate is significantly affected by soil moisture content and the depth of the water in the test hole.

Derr _et al_ (8) attempted to correlate various soil characteristics and other factors with percolation rates. They conclude

1. On the average, variability coefficients of 73 percent can be expected within an individual disposal site. However this variability can range from 0 to over 250%.

2. "The variability between individual tests within sites on the same soil is generally greater than the variability between individual tests at a site. Furthermore, the variability between site medians on different soil series is greater, but not generally significantly different, than the variation between site medians on the same soil series."

3. "The percolation test is influenced by a complex association of soil and environmental characteristics and the variability caused by this complexity reduces the reliability of percolation test results."

Winneberger (25) carried out an extensive analysis of percolation data collected in Arizona and southern California. He found percolation rates are significantly affected by:

1. The depth to which the test hole is filled
2. The surface area to volume ratio of the test hole
3. The method of digging the hole (manual versus power auger)
4. The person who performs the test
5. The device used to measure the water drop.

The extent of some of these effects is shown in Table A1-2.

However, even after presenting the data in Table A1-2, Winneberger suggests a standardized percolation test is still the best practicable method for categorizing soils.

In contrast, Bouma _et al_ (5), after an extensive study of several tile fields which had been operating for periods of a few months to 25 years, makes the statement, "Percolation test results, ..., do _not_ predict the infiltration rates as they occur from seepage trenches."

The consensus of the literature is that the SPT is a very poor method of determining soil suitability. However, there are two distinct reactions to this problem. The pollution control engineers, regulatory agencies, and construction people prefer a well defined percolation test; probably as much for historic reasons as any other. The soil scientists are in favour of the _in situ_ saturated hydraulic conductivity

TABLE A1-2

Test Parameters		Stabilized Percolation Rate		
1. Width of Hole				
narrow hole (dia 3.3")		33 in/hr (1.82 min/in)		
wide hole (dia 13")		9.6 in/hr (6.25 min/in)		
2. Method of Digging Hole		hand auger	power auger	
location	A	5.0 min/in	240 min/in	
	B	4.3	120	
	C	2.4	>60	
	D	0.8	80	
	E	10	240	
3. Depth of Water in Hole	depth of hole	first fill in/hr	second fill in/hr	
8"	18"	116	148	
14"	18"	292	204	
50"	54"	1860		
4. Personnel		personnel number #1	#2	#3
location	1	23 min/in	118 min/in	37 min/in
	2	15	59	172
	3	2	15	32
	4	130	91	161
	5	5	59	73
	6	2	12	24
	7	3	39	229
	8	4	22	34
	9	19	63	259

method. In terms of field performance, both experience difficulties.

The extent of permissible overdesign of the field will govern the degree of refinement required in the permeability testing procedure. If it is practically and economically feasible to install fields with a 50% overdesign, there is probably little need for a refined test. However, where small lot size and poor soil conditions exist, a good estimate of the long term percolation capacity is required.

In response to the apparent need for small systems with high loading rates, other tests have been researched. The most often suggested alternative is the *in situ*, saturated, double-tube, hydraulic-conductivity test. Bouma, de Vries and others have used this test.

A comparison of the SPT and the hydraulic conductivity test with actual tile field infiltration rates is illustrated in Table A1-3 (4).

TABLE A1-3

actual flow (Q/A) L/day	HC L/day	SPT L/day	Q/A / HC	Q/A / SPT
2500	261000	1392000	0.0096	0.0018
280*	19000	288000	0.0147	0.0010
800*	19000	288000	0.0421	0.0028
16	22.5	225	0.7111	0.0711

* Data from same field. Second value taken after resting

The important feature of Table A1-3 is the ratios between the test and the actual loading rate. For each test, excluding the last site, the measured rate to test value ratio varies by a factor of approximately three to four. Thus, although these ratios differ by an order of magnitude in each case, one does not appear to be significantly more reproducible than the other.

Some have argued that the hydraulic conductivity test is more difficult to perform than the SPT. However, when the SPT is properly performed, this does not appear to be a valid point.

French (9) suggested using the hydraulic conductivity test to establish loading rates for representative soils. Then, instead of performing infiltration rate tests at each site, samples of the soil profile would be analyzed. The rate determining layer could then be found from the hydraulic conductivity data on the nearest representative soil. The only difficulty with this approach is the great variety and non-homogeneity of British Columbia soils. This problem occurs to such an extent that several profiles would be required at each site. The expense and time required for this analysis may be prohibitive.

A similar approach has been suggested by Huddleston and Olson (13). They conducted SPT's on approximately fifteen recognized soil types. These soil types were well documented geographically on readily available soil survey maps. Thus, they proposed system design on the basis of the soil type designated on the survey map.

However, in order to provide for local differences in soil structure, they proposed that the percolation rate assigned to each soil type be the average of the measured percolation rates plus one standard deviation. And to add a further conservative feature to their design suggestions, they proposed that three categories of percolation rate would be used. Any soil with a percolation rate plus SD between 0 and 30 min/in is suitable for *in situ* bed construction. From 30 to 90 min/in imported fill must be used and for soils greater than 90 min/in, sand filters are specified. They feel then, that the use of the above criteria, in conjunction with soil survey maps, will eliminate altogether, the need for any on-site testing.

This proposal suffers from the same deficiency as French's. Certainly, in some areas it might be practical but, for use as a province wide standard, it is not suitable.

To the present, there does not appear to have been developed a practical test which is simply related to the long term capacity of a soil to accept effluent. The literature, for practical and traditional reasons, favours the use of the standard percolation test. The hydraulic conductivity test appears to be at least as good and perhaps better. However, all of the literature stresses the need for standardized test procedures carried out by trained, experienced personnel.

APPENDIX 2 - Waste Characteristics

Most of the data available in standard texts for per capita water consumption and liquid waste generation are based on averages computed for large municipal areas. They do not accurately reflect the actual use of water on an individual household basis.

Many agencies have used the value of 75 US gal per capita per day (approximately 61 Igal/c/day) as the basis for design regulations. This figure is used for the sizing of septic tanks, other treatment units and tile fields. Peak flow estimates are also much different for single family dwellings than municipal systems. Peaks of two to four times the average may be encountered in municipal systems where an individual household may discharge peaks of 6 to 10 times the daily average. Also, for a large portion of the day, the inflow to the single family unit is zero.

Table A2-1 illustrates some per capita per day flow rates for individual dwellings taken from the literature. The average of these is approximately 41 Igal/c/day. A family of four would then be expected to discharge an average of 164 Igal/day. Allowing an overdesign factor of approximately 2 indicates that conservative design could be based on 325 Igal/day.

For comparison, Table A2-2 contains the water consumption data for the houses sampled during this study plus some others in the neighbourhood (data from the GVWD). It should be noted that during the period when most of these water readings were collected, the majority of the homeowners

TABLE A2-1

WASTE WATER FLOW

information source	flow g/cap/day (Imp)	# of people
1. Septic Tank Performance		
C,H, Lawerence 36 #3 J.E.H.	25.8 measured	(6)
p226	40.8 measured	(5)
2. A.P. Bernhart	44	
3. Zanoni and Rutkowski	48	
4. Ligman Hutzler & Boyle	15 to 57	
literature review list	40.6	
various ref	41	
	17.7 to 43.6	
	25	
	34.1 to 69.1	
	10.8 to 48.3	
	48.3 to 114	
	39.1	
	29.1 to 41.6	
	41.6 to 74.9	
	23.3	
	40	
	37.0	
5. Bouma et al	26.4 measured	(3)
	41.6 measured	(3)
	36.1 measured	(3)
	34.7 measured	(6)
6. Zanoni and Rutkowski	54.1 measured	
	37.5 measured	
	20.8 measured	
	48.3 (average of 270 houses)	

average 40.9 Igal/c/day

TABLE A2-2

House Number	Dish Washer	Auto Washer	days of week used	No. of people	Average Water Use gal/month total	Average Water Use g/person/d	
1	None	Yes		3	3000	32	
2	Yes	Yes		2	600	10	low
3	Yes	Wringer	1	6	4600	25	
4	None	Yes	7	4	5800	46	
5	None	Yes	7	4	4400	36	
6	None	Yes	7	3	5000	54	
7	None	Yes	2	4	4200	34	
8	None	Yes		5	4700	30	
9	Yes	Yes		2	2000	33	
10	None	Yes	1	4	2800	23	
11	None	Yes		2	1900	30	
12	-	Yes		2	3700	60	
13	None	Yes		5	16200	105	high
14	Yes	Yes		2	5600	90	
15	None	Yes	7	5	8100	52	
16	Yes	Yes		2	3700	60	
17	None	Yes	7	9			
18	Little use	Yes	2	2	3500	57	
19	None	None	2		4734	76	
20	None	Yes	7	5	6300	41	
21	None	Yes	7	4	4300	35	
22	None	Yes		2	4000	65	
23	None	Yes		2	3200	52	
24	None	Yes		2	4900	79	
25	None	Yes		2	3800	62	
26	None	Yes	2	2	3115	50	
27	Yes	Yes		3	3600	39	
28	None	Little use		3	2500	27	
29	None	Yes	1	4	2800	23	

average excluding house #13 45.2

σ_D-standard deviation 19.28

were using very little water which did not enter the treatment system (the neighbourhood had few gardens and most of the data was collected during the winter when outside activities were at a minimum). The average of these figures, excluding one household which was using a lot of water for masonry construction, is 45.2 Igal/c/day. In this case, the family of four would be expected to use 181 Igal/day. Conservative design in this case would be for 360 Igal/day.

Figure A2-1 indicates, as expected, that per capita consumption rates are somewhat higher for houses with low populations but level off for households with more than three people.

The problem of peak loadings is difficult to design for. For example, flows can increase from zero to 20 gpm with the removal of the bathtub plug. Normally, as with municipal plants, peak loadings occur between 0800 and 1000 hrs. and 1700 and 1900 hrs.. However, erratic flow behaviour can occur at any time.

The same texts, which supply sewage flow rate data, also supply per capita organic loadings derived from municipal averages. Generally speaking, these figures are not applicable to individual households.

Ligman et al (16) conducted a detailed study of waste characteristics from rural and urban dwellings. Their results (which include data from an extensive literature survey) are summarized with others in Table A2-3.

From the figures in Table A2-3, it is evident that organic peaks occur and that these do not necessarily coincide

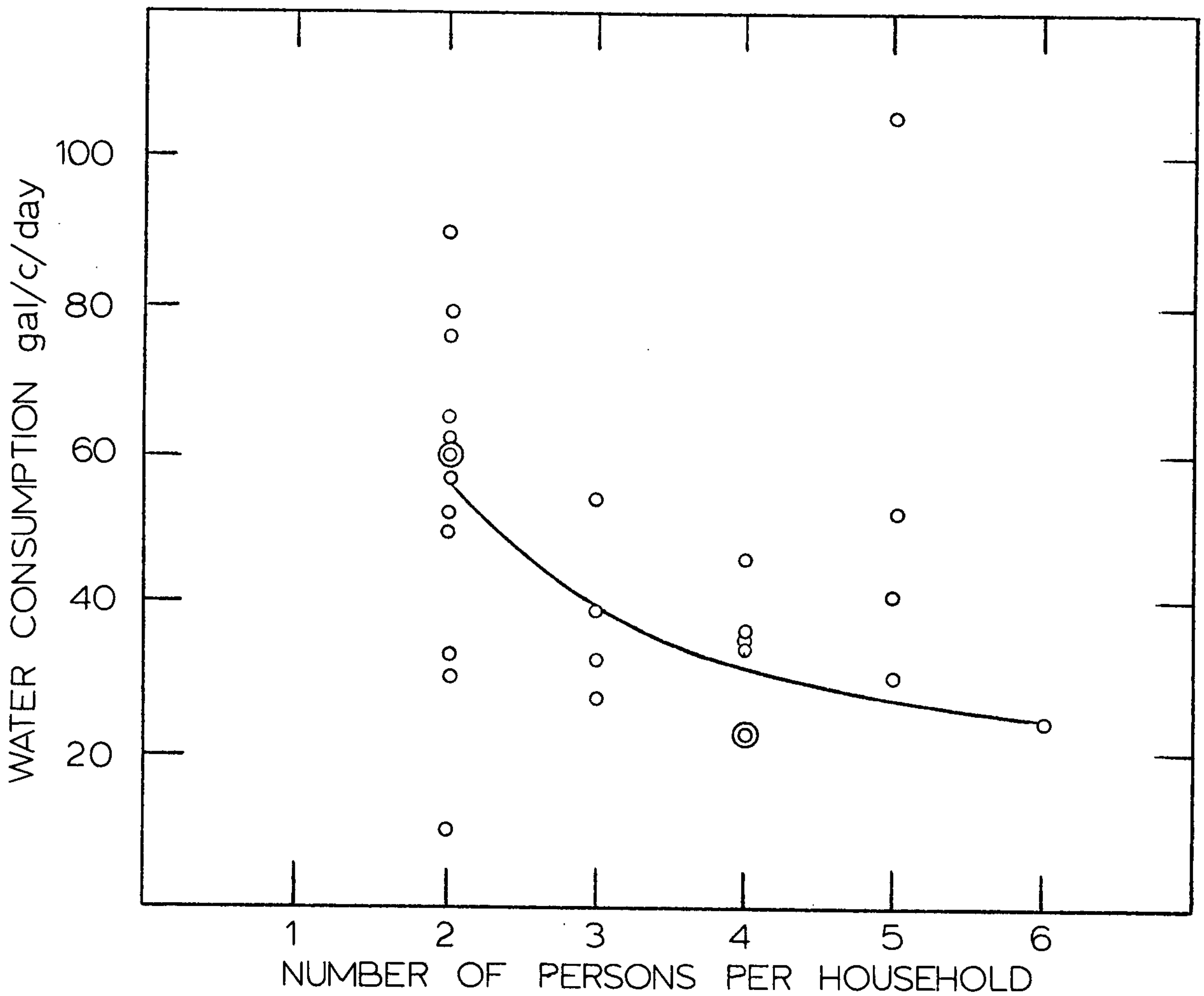
100
80
60
40
20
WATER CONSUMPTION gal/c/day
1
2
3
4
5
6
NUMBER OF PERSONS PER HOUSEHOLD

FIGURE A2-1

TABLE A2-3

HOUSEHOLD WASTE CHARACTERISTICS

1. ref Ligman *et al* (16)

Wastewater Event Characteristics in lbs per capita per day

Water event	BOD_5	Total Solids (dry wt)	SS	Total N	Total P	Fats (dry wt)
bath/shower	0.020	0.046	0.012	-	-	-
dishwashing	0.013	0.022	0.006	-	0.001	-
garbage disposal	0.068	0.135	0.096	0.002	-	0.017
clothes washer	0.021	0.088	0.016	-	0.005	-
Toilet						
feces	0.025	0.060	0.048	0.003	0.001	0.010
urine	0.023	0.132	-	0.034	0.002	-
paper	0.004	0.022	0.020	-	-	-
Average adult characteristics	0.174	0.505	0.198	-	0.009	-
Average adult characteristics without garbage	0.106	0.370	0.102	-	0.009	-

2. ref Benhart (2)

	lb/cap/day BOD	lb/cap/day SS
average adult characteristics without garbage grinding	0.082 - 0.132	0.115 - 0.143

with the hydraulic peaks. The average BOD concentration for an adult occupying a dwelling which does not have a garbage grinder can be calculated as follows:

$$0.106 \ \frac{\text{lb BOD}}{\text{cap day}} \times \frac{1}{41\text{-}45} \ \frac{\text{day cap}}{\text{Igal}} \times 10^6 \ \frac{\text{gal}}{\text{MG}} \times \frac{1}{10} \ \frac{\text{mg/l}}{\text{lb/MG}} \cong 260\text{-}235 \text{ mg/l}$$

Peak concentrations as high as 600 mg/l are not uncommon. However, a great proportion of the time, there is no influent to the treatment unit.

To achieve an effluent of approximately 50 mg/l, an eighty percent removal is required but to achieve 30 mg/l BOD a ninety percentage efficiency is needed. Considering the irregular influent pattern, treatment efficiency must range from zero to almost one hundred percent at various times during the day.

These erratic waste characteristics only compound the problems of small scale treatment design.

APPENDIX 3 - Testing and Analysis

Tests

The literature is far from agreement on which parameters are of importance when analyzing an effluent which is to be applied to the soil. Most authors agree DO, SS and nutrient concentrations have significant impacts on the soil in the disposal site. It has been suggested (2) that a ciliata count would be of value in determining how quickly clogging bacteria will be removed. However, the greatest diversity of opinion exists in how to measure the organic content of the effluent.

The standard five day BOD test is most often used; probably for traditional and comparatory reasons. As noted in Chapter 3, there appears to be very little correlation, if any, of BOD with either COD or TOC. In terms of soil clogging potential, the important organic measurement is oxygen utilization, while the effluent is in the vicinity of the tile field. However, in gravel soils, the important organic measurement may be the residual BOD discharged to groundwater.

In addition, there is the long standing dissatisfaction with the BOD_5 test in general. However, until it can be demonstrated that another parameter is better correlated with tile field results, the BOD_5 test will continue to be used.

Analyses

All samples were delivered to the Chemistry Laboratory of the Water Resources Service in Victoria for analysis. Samples were stored in coolers with ice packs if not delivered within eight hours of collection. All samples were turned over to the lab within twenty hours of collection. All analysis were carried out in accordance with the procedures adopted by the Water Resources Service.

The COD test procedure deserves special discussion. It is the policy of the Chemistry Lab to perform a starch-iodide test for residual chlorine compounds on <u>all</u> samples it receives for BOD analysis. A positive test indicates the presence of a strong oxidizing agent. Table A3-1 shows that the amount of oxidizing agent correlates well with the amount of nitrite in the sample. Orthotolidine-arsenite tests did not indicate the presence of Chlorine compounds. Thus, nitrite and not chlorine may be responsible for the positivie starch-iodide test.

TABLE A3-1

Sample	Mg/L $NO_2^=$ **	ml Cl_2 neutralized *
1	1.9	3.5
2	2.0	3.0
3	2.9	2.5
4	0.103	nil
5	0.107	nil
6	0.129	nil

* ml of the oxidizing compound indicated by the starch-iodide test

** The $NO_2^=$ test used by the Chemistry Lab is subject to interferences by sulphide and free chlorine and has a detection limit of 0.005 mg/l N for extracted samples

However, if a sample shows a positive starch-iodide test, irregardless of the cause, the Chemistry Lab neutralizes the sample with sodium sulfite and seeds the sample with settled municipal sewage. Table A3-2 indicates that a significant difference in BOD_5 exists between seeded and unseeded samples.

TABLE A3-2

Five day BOD in mg/l

sample number	seeded sulfite	No sulfite seeded	unseeded sulfite	No sulfite Nö seed
1	78	59	69	52
2	78	56	66	55
3	76	55	59	54
4	69	61	66	57
average of four samples	75	58	65	55

The addition of the sulfite has a much greater affect on the results than does the seeding.

The difficulty arises when comparing results from the Chemistry Lab and private laboratories. The private labs, if told a sample does not or should not contain any residual chlorine compounds, do not perform the starch-iodide test and consequently do not neutralize or seed. Thus, significant differences can result from these two different procedures.

Approximately eighty-five percent of the samples collected this summer had positive starch-iodide tests. As the provincial government is the testing and regulatory agency for these systems, the data, as would be presented by the Chemistry Lab under normal circumstances, will be used. In the majority of cases, this means the BOD test results are from samples which have been neutralized and seeded.