TREATMENT OF SANITARY WASTES AT
INTERSTATE HIGHWAY REST AREAS
PHASE I

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BY

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JOINT HIGHWAY RESEARCH PROJECT
PURDUE UNIVERSITY AND
INDIANA STATE HIGHWAY COMMISSION

Final Report

Phase I

TREATMENT OF SANITARY WASTES AT INTERSTATE HIGHWAY REST AREAS

TO: J. F. McLaughlin, Director   September 7, 1972
Joint Highway Research Project   File: 9-11-6

FROM: H. L. Michael, Associate Director   Project: C-36-67F
Joint Highway Research Project

The attached Final Report on Phase I of the HPR Part II Research Study titled "Treatment of Sanitary Wastes at Interstate Highway Rest Areas" is submitted for acceptance as fulfillment of the objectives of that phase of the Study. The proposal for Phase III of this Study has been based on the findings of this Phase I and the construction of two prototype systems incorporating the design and methods studied in this research was found to be warranted.

The results of this Study could have a major impact on the design and cost of waste treatment facilities at highway rest areas. The application of the findings is the emphasis of Phase III of this Study which has been submitted for approval. If the evaluation of the prototype systems to be constructed in Phase III proves them to be practical, this Study could result in great savings to the ISHC and other highway departments.

The Report will be submitted for review, comment, and acceptance by the ISHC and the FHWA.

Respectfully submitted,

Harold L. Michael
Associate Director

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Final Report
Phase I
TREATMENT OF SANITARY WASTES AT INTERSTATE HIGHWAY REST AREAS

by

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The contents of this report reflect the views of the author who is responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the Federal Highway Administration. This report does not constitute a standard, specification, or regulation.

Purdue University
West Lafayette, Indiana
September 7, 1972

Treatment of Sanitary Wastes
at Interstate Highway Rest Areas

INTRODUCTION

Since the beginning of wastewater production many methods have been devised to prevent the pollution of our natural water resources, including both surface and underground waters. Among the many treatment methods presently used, the activated sludge system including its various modifications seems to be the most widely used method.

Although wastes from large numbers of people can be handled reasonably, there has been no adequate solution for the treatment of the wastes from single dwellings, camping areas, picnic areas, highway rest areas and other sources with small waste flow volumes. The most common methods for the treatment of low volume wastes have been septic tanks and percolation fields, seepage pits, pre-fabricated package plants, oxidation ponds and other aerobic treatment methods (1) (2) (3) (4) (5). The Indiana State Highway Department predominately employs the extended aeration method for treatment of the sanitary wastes from interstate highway rest areas. However, these methods are considered to be inadequate due to the widely fluctuating hydraulic and organic loading conditions which cause poor effluent quality and because of the difficulties in providing operation and maintenance during freezing non-use periods. Aside from

these problems there is also the need for large land areas at rest stops, the need for proximity to a stream and water source, and the esthetic unattractiveness of the completed waste treatment system.

The purpose of the research was thus to develop a reasonable and effective method to treat the wastewaters from highway rest areas under the unusual and varying loading and operational parameters present.

Basic Considerations

In the activated sludge system removal of organics in a wastewater is accomplished by converting the dissolved and suspended substances into settleable materials which can be removed from the wastewater by sedimentation for further treatment. A main disadvantage of the activated sludge system is that an appurtenant treatment facility is necessary in order to treat the harvested solids. The traditional method for the treatment of waste activated sludge is by anaerobic digestion, in which the excess activated sludge, usually together with primary sludge, is stabilized anaerobically. Recently, much effort has been devoted to the stabilization of waste activated sludge by aerobic digestion especially with small scale operations. When compared to anaerobic digestion, aerobic digestion systems afford several benefits, including: minimal supervision, simpler equipment, lower initial cost, and lower maintenance costs (6). Aerobic digestion of primary sludge or mixtures of primary and activated sludge has been investigated by Moriarty(7), Woodley (8), and Lawton and Norman (9). In each case, they concluded that the aerobic digestion of primary and activated sludge was more complete

than anaerobic digestion. Ritter (10) has reported design data for sludge digestion using diffused air. Based on his studies, it can be concluded that the use of aerobic digestion eliminates the operational and economic disadvantages associated with anaerobic digestion of waste activated sludge.

To avoid the separate sludge digestion problems, the idea of total oxidation has been developed. Early in 1953, Porges, et.al., (11) suggested that an activated sludge plant could be operated with return of all biological solids to the aerator. They suggested that a point would be reached wherein net auto oxidation would balance net synthesis of new biological sludge, permitting the system to operate at or near equilibrium. The main difficulty of the application of this idea has been the practicality of collecting the activated sludge in a final settling tank. The economics involved with the construction of large final clarifiers needed to effect a satisfactory liquids/solids separation presents a significant disadvantage. Gaudy, et.al., (12) tried to overcome this difficulty by using a cellophane type reactor in which a plastic membrane, permeable to soluble substances but not to cells, was employed. Their attempt failed due to operational problems. However, the use of a membrane ultrafiltration method is feasible (13) (14) (15) and warrants further investigation.

Preliminary studies performed by Stiefel, et.al., (16) reported that wastes with biochemical oxygen demands (BOD) ranging up to 10,000 mg/l could be successfully treated by maintaining a MLVSS level of 25,000 mg/l in an aerator. Other advantages of keeping a high MLSS or MLVSS in an aerator will be discussed later.

Figure 1 - Pilot Plant For The Research

Figure 2 - Detail of the pilot plant

Figure 3 - Variations in pH and MLSS concentration

Extended aeration is an incomplete application of the total oxidation concept. In this system, solids are collected in a final settling tank and returned to the aerator. Hence the MLSS concentration in the aerator is higher than those of other systems, thus keeping the food to microorganism ratio low (17). Since the successful return of biological solids from a final settling tank to the aerator depends on the performance of the final settling tank, it is not always possible, to achieve high quality effluents.

In this research a synthetic fabric filter was employed to replace the conventional final settling tank so that a positive means to prevent the loss of MLSS into the effluent from the treatment system could be achieved. This method allowed complete application of the total oxidation concept in the treatment of the sanitary wastes at highway rest areas.

Experimental Procedure

1) Equipment

A test scale unit, shown in Figure 1, was set up in the pilot plant room in the Civil Engineering Building of Purdue University, W. Lafayette, Indiana. This unit was constructed of a steel tank with a volume of 264 gallons (3 ft. diameter and 5 ft. height). Both the inside and outside surfaces of the tank were painted with coal tar to prevent corrosion. Inside the tank a synthetic fabric filter bag was supported on a polyethylene coated No. 10 wire mesh. The fabric used in the unit was made by GAF and had a total thickness of approximately 0.25 in. The filter bag was arranged in a saw-tooth fashion, as shown in Figure 2, to maximize the surface

Figure 4 - BOD and SS concentrations influent and effluent

area, and thus increase the ratio of the surface area to aeration chamber volume. The total surface area of the filter bag was 133 ft.² (excluding the bottom surface) and the total volume of the aeration chamber (inner part of the filter bag) was 19.7 ft.³ (147 gallons), thus making the volume between filter and tank wall 117 gallons. Therefore, the ratio of the filter surface area to aeration chamber volume was 6.8 ft.³/sq.ft. The inlet pipe had a diameter of 2 inches and was installed with a valve to control the influent flow rate. The outlet of the tank was located near the bottom and a weir box was attached to control the outlet level as shown in Figures 1 and 2.

2) Sewage Source

The sewage source was the toilet wastes from the Purdue Civil Engineering Building. Since the sewage from the building originated mainly from the use of toilets, it was anticipated that the influent would have similar characteristics to those of highway rest area wastes. Reasonableness of this assumption was confirmed by the analysis of the many sewage samples from some of the interstate highway rest areas in Indiana during the summer of 1971. Some characteristics of the influent sewage are given in Table 1.

The sewage from the building was collected by gravity in a sump in the basement of the building and pumped up to the pilot plant on the second floor of the building by a centrifugal pump. When the sewage from the building was not enough for the research the street sewer valve in the basement was opened to get sufficient sewage from a street sewer. It was found that the sewage taken

from the street sewer was weaker in organic content.

3) Air Supply

At the start of the research an air blower was installed at the pilot plant site. However, after only a couple of days the blower was replaced with a laboratory compressed air supply line because of the noise problem. Initially, air was diffused into the mixed liquor through twenty stone air diffusers attached to a plastic tube ring which was located on the bottom of the filter bag. After about one month of operation, air was not uniformly distributed because part of the air tube was clogged with sewage due to incomplete sealing of the tube. Therefore, six additional diffusers were installed at one side of the filter bag to supply air uniformly. The air flow rate was measured to be about 2.1 ft.³/min. This flow rate was maintained throughout the remainder of the research.

4) Operation

It was decided to operate the pilot plant on the basis of a batch-feed schedule since this method would simulate the conditions present in the rest areas. The pilot plant was fed with raw sewage once every hour during daytime hours only. Sometimes, conditions dictated deviation from this feeding schedule. The operation procedure was as follows:

- (1) Measure the sewage levels both inside and outside of the filter.
- (2) Close the outlet valve
- (3) Turn on sewage source

- (4) Take influent sample
- (5) Turn off sewage source when the mixed liquor level in the aeration chamber reached to within 2 inches from the top of the tank.
- (6) Measure the sewage level of the outside of the filter
- (7) Open outlet valve
- (8) Take effluent sample

Both influent and effluent were sampled every feeding time so as to obtain flow-graduated composite samples. The mixed liquor was sampled just before the initial feeding of each day's operation. The temperature of the pilot plant room had been measured and was found to range between 20°C and 30°C.

Since the feeding rate was controlled by the filterability of the filter material the detention time in the aeration chamber was controlled by adjusting the level of the effluent weir box. The feeding rate was increased by lowering the level of weir box as this caused greater head difference between the inside and outside of the filter and thus shorter liquid retention times.

The following determinations were carried out during the research:

For influent samples-----pH, SS, VSS, BOD (filtered and
unfiltered)

For effluent samples-----pH, SS, BOD

For aeration contents samples-----SS, VSS

Results and Discussion

The operation of the pilot plant started after the filter bag (aeration chamber) was filled with MLSS which were prepared by mixing sewage with return sludge (about 40 gallons) obtained from the

month	Number of days operated	pH				BOD				SS				Loading		
		detention time in aeration chamber (hr)	Influent	Effluent	Influent (mg/l)	Effluent (mg/l)	removal efficiency (%)	Influent (mg/l)	Effluent (mg/l)	removal efficiency (%)	MLSS (mg/l)	MLVSS (% of MLSS)	LbBOD 1 day	LbBOD 11,000 ft ³	MLVSS	
11-71	16	avg	15.8	7.7	7.8	260	10.4	96	186	3.9	4900	77	24.8	0.116		
		max	23.8	8.1	8.2	820	739.1		500	16.0	6230	79	81.3	0.358		
		min	14.7	6.8	7.4	103	2.9		56	1.0	3960	75	10.4	0.036		
12-71	19	avg	15.3	7.8	7.9	248	5.0	98	230	1.8	7760	74	23.4	0.069		
		max	20.5	8.7	8.3	652	23.3		805	5.5	8670	79	49.7	0.174		
		min	9.4	7.1	7.5	33	1.8		39	0.0	6560	72	5.8	0.020		
1-72	10	avg	7.7	8.0	7.6	280	11.2	96	255	0.4	12765	75	56.1	0.088		
		max	11.3	8.8	7.7	674	36.0		520	1.5	15110	76	126.8	0.172		
		min	5.4	7.3	7.4	161	3.0		82	0.0	10880	74	29.5	0.046		
2-72	7	avg	11.8	8.1	7.7	190	5.0	97	136	1.0	18060	76	39.4	0.029		
		max	14.0	8.4	7.8	398	8.2		230	1.8	18370	77	51.8	0.059		
		min	9.2	7.6	7.6	72	2.1		48	0.5	17670	76	7.7	0.009		
3-72	16	avg	9.8	7.8	7.6	191	5.3	97	148	0.9	19300	75	31.0	0.032		
		max	12.8	8.4	7.9	288	9.9		235	2.5	20960	77	42.4	0.044		
		min	6.6	6.0	7.1	125	2.2		25	0.0	17910	73	18.2	0.020		
4-72	21	avg	8.3	8.2	7.8	171	4.2	97	142	1.1	21680	75	31.1	0.029		
		max	11.5	8.7	8.1	320	14.9		382	3.0	23110	76	63.1	0.057		
		min	6.3	7.5	7.6	42	1.2		56	0.0	19270	74	7.0	0.007		
5-72	23	avg	5.7	8.1	7.8	180	5.7	97	174	1.8	24600	76	39.7	0.032		
		max	8.5	9.2	8.0	7846	717.6		425	3.5	28650	80	717.1	0.165		
		min	5.3	6.4	7.2	49	1.4		70	0.0	21670	74	11.7	0.010		
TOTAL	112	avg	10.5	7.9	7.8	213	6.3	97	180	1.6	16320	76	33.1	0.053		
		max	23.8	9.2	8.3	7846	736.0		805	16.0	28650	80	717.1	0.165		
		min	5.3	6.0	7.1	33	1.2		25	0.0	3960	72	5.8	0.007		

Table 1 - Summary Data From The Research

West Lafayette, Indiana sewage treatment plant. The actual plant performance research began November 15, 1971 and continued to May 31, 1972.

1) Treatment efficiency

During the research period the pilot plant was fed an average sewage flow of 302 gallons per day (over a maximum period of 12 hours) for 112 days. The influent flow rate ranged from 123 to 523 GPD, thus making the detention time in the aeration chamber range from 5.3 hours to approximately 24 hours.

The pH of the incoming sewage fluctuated from 6.0 to 9.2 with an average value of 7.9 as shown in Table 1 and Figure 3. After treatment the pH of effluent ranged from 7.1 to 8.3 with an average of 7.8.

As shown in Table 1 the average BOD values for the influent and effluent were 213 and 6.3 mg/l respectively, thus affecting an average BOD removal of 97%. Figure 4 shows the fluctuations in BOD's of the influent and effluent. During the research period it had been noticed when the influent contained high concentrations of detergents and waxes due to building cleaning the effluent showed very high BOD values (in these cases the influent appeared a white milky color and gave a typical wax and soap odor). If those cases were not included in the BOD removal efficiency calculation; the efficiency would be higher. Since the sanitary wastes from highway rest areas do not contain very high concentrations of detergents the BOD removal efficiency would be even greater.

The suspended solids removal efficiency was excellent, approach-

ing 100% as shown in Table 1 and Figure 4. As the effluent SS concentrations were very low, they could not be represented clearly in Figure 4. Since the SS concentrations of an effluent from an activated sludge system are contributed mainly by the activated sludge particles due to inadequate performance of the final clarifier (18) (19), the use of the fabric filter for the almost complete removal of MLSS from the effluent in this system should produce an effluent with a very high quality.

In summary, the quality of the effluent from the system was so high that it is reasonable to consider that it may be recirculated for toilet flushing or disposed of after chlorination without fear of causing any pollution of a receiving stream.

2) Filter Performance

Though the use of a filter can produce high quality effluents, there is a limitation in the flow rate through the filter just like filters utilized for other purposes. This is the main disadvantage of the use of the filter. The fabric filter used for this research satisfied all the requirements for the filters suggested by Kaswell (20), and McCabe and Smith (21), except one, blinding. This blinding phenomenon of the filter material caused limitations in the flow rate through the system.

To determine the flow rate through the filter both continuous-- and batch-feed studies were conducted. On December 26 and 31, 1971, the reactor was fed sewage continuously during day-time hours and flow rates measured at various head differences. The results of the continuous-feed studies are shown in Figure 5. It can be concluded

that the flow rate is almost proportional to head loss through the filter; which is reasonable since the flow velocity through the filter was in the laminar region.

The batch-feed studies were performed numerous times during the research period and representative data is shown in Figure 6. The batch-feed studies were carried out by fixing the outlet level at 10 inches from the top of the tank. The aeration chamber, after an overnight rest period (during which no sewage was fed), was filled with sewage up to 2 inches from top of the tank utilizing the feeding intervals of 1 hour, 1/2 hour, or 1/4 hour for each case and then allowed to drain down for a given feeding interval period. This procedure was repeated and the volume of sewage fed was summed to calculate flow rate and then plotted in Figure 6. From this figure it can be concluded that the flow rates of batch-feed study approach those of the continuous-feed study (compare Figure 6 with Figure 5). The flow rate through the filter is not affected by MLSS concentration within the conditions given in this research (compare curve A with curve B, curve C with curve D, and curve E with curve F). It can be noticed that after a rest period, flow rate through the filter can be increased, but by continuing sewage feeding the filter is clogged with solids in the aeration chamber due to pressure difference between the inside and outside of the filter. It is interesting that the final flow rate through the filter increased with the increase of MLSS concentration. It is postulated that after a long period of operation of the filter MLSS forms larger flocs which, when attached onto surface of the filter

Figure 5 - The flow rate through the filter
under continuous feeding conditions

Figure 6 - The flow rate through the filter under conditions of different feeding intervals and MLSS concentrations

forms a fine mat which prevents the passage of small particles which may clog the filter. It is also possible that a micro-biological slime layer is formed on or in the filter cloth which causes a more uniform flow rate by preventing the passage of small particles.

To determine the blinding rate of the filter, batch-feed studies were done and results are shown in Figure 7. The studies were carried out by filling the aerator with sewage at different times during a day of operation. For curve 1, the aerator had not been fed with sewage overnight (for 13 hours), thus making the head difference between the inside and outside of the filter very small (0.2 in.). The aerator was then filled with 24 gallons of sewage to bring the level in the filter up to within 2 inches of the top of the tank (feeding time was 20 seconds). This volume was thence allowed to drain and the effluent flow rate measured. From the curve it is concluded that the flow rate through the filter can be increased by providing a rest period without feeding with sewage. In any case, however, it was established that a sustained filter rate of 0.004 gal/day/sq.ft. of filter bag area was possible.

It was noticed that the high concentrations of detergents and waxes in the influent deteriorated not only the quality of effluent, but also the filter flow rate. The flow rate decreased drastically when the influent contained high concentrations of detergents and waxes. This effect decreased as the MLSS concentration increased, but was not eliminated completely.

Data taken on Dec. 15, 1971
Feeding Schedule : approx. once in an hour

curve	Time measured	previous rest period (hrs)	sewage feeding rate	$\Delta H = 10 - H_i$
①	08:10	13	24 gal/20 sec. = 72 GPM	0.2
②	19:10	2	16 gal/15 sec. = 64 GPM	3.2
③	15:30	1	9.5 gal/8 sec. = 71 GPM	4.7

Figure 7 - The blinding trend of the filter

3) Accumulation of solids in the filter bag.

As shown in Figure 3 and Table 1 the MLSS concentration in the filter bag increased from the initial value of 3960 mg/l up to 26,000 mg/l after about 6 1/2 months operation. During this period the maximum MLSS concentration occurred on May 24, 1972 with a value of 28,650 mg/l. It should be mentioned here that the MLSS concentration curve in Figure 3 shows only the general trend, because a significant amount of solids had deposited on the inner surface of the filter between the top of the filter and liquid surface. During the sludge-build up period the sludge had shown a brown color and given an earthy odor. According to Caron (22) this means that the activated sludge in the filter bag had remained in a healthy condition.

The increase in MLSS concentration would be due to:

- (1) Increase in the concentration of microorganisms such as bacteria, fungi, and protozoa
- (2) Accumulation of cell products which are non-biodegradable volatile matter
- (3) Accumulation of inert material in the influent such as grit, lignin or other nondegradable organics

During the research period the average ash content of the influent suspended solids ranged from zero to 66% with an average value of 22%. If it is assumed that all the ash content of the influent suspended solids had remained in the aeration chamber, the inert material accumulated would be about 11 pounds. The total flow through the treatment unit over the test period was calculated to be 33,800 gallons, based on the average daily load-

ing rate.

$$0.0338 \text{ MG} \times 8.34 \times 180 \text{ mg/l} \times 0.22 = 11 \text{ pounds}$$

Microorganisms in the activated sludge system multiply by utilizing the organics in the wastewaters. Based on the data given in Table 1 and the assumptions given below the calculations can be made.

ASSUMPTIONS:

- (1) 60% of the BOD added was converted to cells
- (2) The fixed solids in the influent SS are retained in the filter bag
- (3) The ratio between BOD due to SS and SS themselves is assumed to be 1
- (4) The amount of solids deposited on the inner surface of the filter below mixed liquor surface is negligible
- (5) There is no solids deposit on the bottom of the filter bag

$$\text{Total BOD added} = 213 \text{ mg/l} \times 8.34 \times 0.0338 \text{ MG} = 60.0 \text{ lbs.}$$

$$\text{Cells produced} = 60.0 \times 0.6 = 36.0 \text{ lbs.}$$

$$\text{Filtered BOD added} = 0.57 \times 60.0 = 34.2 \text{ lbs.}$$

(filtered BOD of the influent had an average value of 57% of the total influent BOD)

$$\text{BOD due to SS} = \text{total BOD} - \text{filtered BOD}$$

$$= 60.0 - 34.2 = 25.8 \text{ lbs.}$$

$$\text{Total SS added} = 180 \times 8.34 \times 0.0338 = 50.7 \text{ lbs.}$$

$$\text{Total VSS added} = 0.78 \times 50.7 = 39.6 \text{ lbs. (VSS content in the influent had an average value of 78% of the total SS).}$$

$$\text{Fixed SS} = \text{total SS} - \text{VSS} = 50.7 - 39.6 = 11.1 \text{ lbs.}$$

Volatile SS not oxidized =

Total VSS - VSS oxidized by microorganisms

$$= 39.6 - 25.8 = 13.8 \text{ lbs.}$$

Total solids increase in the aeration chamber = 36.0 lbs.

microorganisms from BOD + 11.1 lbs. of fixed SS = 13.8 lbs.

of VSS not oxidized = 60.9 lbs. of MLSS.

The initial MLSS = $3960 \text{ mg/l} \times 8.34 \times 123 \times 10^{-6}$ gallons
aeration chamber volume = 4.1 lbs.

Expected total MLSS = $60.9 + 4.1 = 65.0$ lbs.

Actually, the MLSS concentration on the final day (May 31, 1972)
was 26,000 mg/l and the aeration chamber volume was 123 gallons.

$$\text{Actual MLSS} = 26,000 \times 8.34 \times 123 \times 10^{-6} = 26.7 \text{ lbs.}$$

There was a solids deposit on the inner surface of the filter
between the top of the filter and the mixed liquor surface. The
moisture content by weight and the thickness of the solids layer
were measured to be about 85% and 1/2 inches, respectively. The
surface area covered with solids deposit was calculated to be
22 ft.²

$$\text{Solids deposited} = 22 \text{ ft.}^2 \times 1/12 \times 1/2 \text{ ft.} \times 62.4 \text{ lbs./ft.}^3 \\ \times (1-0.85) = 8.6 \text{ lbs.}$$

$$\text{Total actual solids content} = 26.7 + 8.6 = 35.3 \text{ lbs.}$$

$$\text{Destruction of solids due to endogeneous respiration} = \\ 65.0 - 35.3 = 29.7 \text{ lbs.}$$

This means that 29.7 lbs. of cells had been destructed
during the 199 days period (from November 15, 1971 to May 31, 1972)

$$\text{Viable cells remained in the system} = 36.0 - 29.7 = 6.3 \text{ lbs.}$$

This is the estimated amount of viable cells in the final MLSS.

$$\% \text{ viability of the final MLSS} = \frac{6.3}{26.7} \times 100 = 23.6\%$$

$$\% \text{ viability of the final MLVSS} = \frac{6.3}{26.7 \times 0.76} \times 100 = 31.0\%$$

Therefore, it is speculated that the viability of the MLSS in the filter bag was very low compared to that of the conventional activated sludge system.

The average endogenous respiration rate, b , may be calculated as follows under the assumption that during the period the average viability was 50%.

$$\text{Average } b = \text{MLVSS destroyed} / (\text{average MLVSS}) \times (\text{time period})$$

$$\text{Average MLVSS} = 16,320 \times 8.34 \times 123 \times 10^{-6} \times 0.76 = 12.7 \text{ lbs.}$$

$$\text{Viable MLVSS} = 12.7 \times 0.5 = 6.35 \text{ lbs.}$$

$$b = \frac{29.7}{6.35 \times 199} = 0.0235$$

Generally, the b value for conventional activated sludge system is considered to be about 0.1, but for a system where sludge age is very old such as in extended aeration systems, the b value should be low. Gaudy, et.al., (12) has stated that the endogenous activity of the extended aeration sludge is approximately 5 to 10% of that for young cells of the same origin. Kountz, et.al., (23) and Stewart, et.al., (17) reported that the endogenous loss in a total oxidation system was about 2% per day of the total sludge. Thus, the above calculated b value, 0.0235 which represents 2.35%/day, seems reasonable. It was reported that the rate of endogenous respiration was primarily a function of sludge age (24) and that the biological activity approached a minimum after total sludge age

of 120 days. Sludges with such ages are stabilized, having an earthy, slightly musty odor, (b).

Washington, et.al., (15) has stated that the decrease in sludge activity is partly due to the increase of metabolic end products. Therefore, Washington et.al., (25) has concluded that total oxidation is not possible. In addition to the accumulation of non-biodegradable metabolic end products, the non biodegradable part, such as gum, of the influent SS will accelerate the accumulation of sludge in the aeration chamber. Since the non biodegradable metabolic end products contain nitrogen, nitrogen determination of an activated sludge cannot be used to measure active mass, (25).

The ability of this system using a fabric filter to keep a high MLSS concentration will bring many advantages. Though it was calculated that only 23.6% of the MLSS was viable, the actual active cell concentration was higher than those of other activated sludge systems. It was previously mentioned that high MLSS concentration makes the food to microorganism ratio; thus producing a high quality effluent. Symons and McKinney (26) have stated that the application of the total oxidation concept to wastewater treatment makes it possible to remove carbon from the waste more completely and to utilize the nitrogen nutrient more thoroughly in the waste by microorganisms. It was previously mentioned that an activated sludge process having a high concentration of MLVSS could effectively treat wastes with a high BOD concentration (13) (16). The use of pure oxygen would make this more theoretically acceptable (27) (28) (29) (30). McKinney and O'Brien (31) mentioned that it should be poss-

ible to treat domestic wastewater in 2.5 to 3 hour aeration period provided high MLSS concentrations are maintained. Ludzack and Noran (32) reported that high unit solids, thus low load ratios, increased activated sludge tolerance chloride changes and high chloride concentrations.

Since many researchers have reported that a bulked sludge produces an effluent with a higher quality than does a dense sludge (33) (34) (35) the use of a fabric filter for such cases would be another advantage of this system.

CONCLUSIONS

From the results of this study the conclusions are as follows:

1. The system can produce an effluent with a very high quality when fed with domestic sanitary wastes. The average BOD and SS removal efficiency were 97 and 99%, respectively
2. Therefore the system can be used effectively for the treatment of sanitary wastes at the interstate highway rest areas
3. During the pilot plant research period of 6 1/2 months no deterioration or clogging of the filter was observed
4. The effluent was odor-free, clear and sparkling. Thus, it may be reused for toilet flushing
5. It seems that after a long period of operation sludge wasting may be necessary due to the continuous accumulation of inert and non biodegradable volatile materials (about once per year on mens restroom facilities and about once every 4 to 6 months on womens restroom facilities)
6. The system can tolerate organic or hydraulic shock loads
7. The use of the system for wastewater treatment would be more economical than the existing type treatment facilities since the system does not require primary or secondary sedimentation, produces a more stabilized sludge. Reduced sludge amounts are also realized.

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