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Project Summary

Demonstration Physical Chemical Sewage Treatment Plant Utilizing Biological Nitrification

James F. Kreissl and Ronald F. Lewis

This study involved the design, construction and operation of a hybrid physical-chemical (P-C) biological treatment facility. Evaluation of this system was based on two factors: its utility as a transportable facility for interim high quality treatment of wastewaters at different locations and its value as a treatment concept to incorporate the best attributes of both methods (P-C and biological) of treatment.

Although the system produced a consistent, high quality effluent, its utility as a transportable system was only partially demonstrated and its viability as a treatment sequence could not be confidently stated due to several design and operational problems.

This summary presents an overview of this joint USEPA-DHUD project.

This Project Summary was developed by EPA's Municipal Environmental Research Laboratory, Cincinnati, OH, to announce key findings of the research project that is fully documented in a separate report of the same title (see Project Report ordering information at back).

Introduction

Experience has shown that a need exists for flexible and efficient sewage treatment units able to meet the ever more stringent regulations imposed by government. Urban fringe developments are particular problem areas that outstrip the service ability of urban sewage authorities.

Small housing, commercial, and public developments spring up beyond the range of central sewage transport systems and must have acceptable sewage treatment on a permanent or temporary basis. Often, the temporary nature of the treatment facilities compounds the problem, resulting in heavy capital equipment outlays prorated over relatively short time periods. The small size and nature of such development areas frequently provide flow variations that are not conducive to effective biological treatment. Daily, as well as seasonal, fluctuations may be extreme in both hydraulic and organic loadings. The need for treatment processes that can be placed in service quickly and with a minimum of delay to meet strict effluent limitations has long been recognized. Development areas on urban fringes frequently discharge to small streams with neither little dry weather flow nor periodic high rainy weather flow and effluent limitations are generally based on the most extreme low flow conditions.

This demonstration project was conducted to show that wastewater could be treated in a physical-chemical wastewater treatment plant employing a biological intermediate stage for oxidation of nitrogenous material to produce a high quality effluent and provide different treatment levels to meet a variety of effluent requirements. The physical-chemical plant chosen was of a modular design employing high-rate processes which normally facilitate a relatively speedy installation, a minimum amount of lag time to produce the desired effluent quality, and ease of transport for relocation to other critical areas when needed.

The plant was located in the drainage area of a planned residential development known as Beechgrove Village in the southern part of Kenton County, Kentucky. The wastewater was domestic in nature, with no commercial or industrial sources.

Facility Description

The facility consisted of a modular physical-chemical (P-C) wastewater treatment plant which was skid-mounted for ease of transport, an intermediate biological nitrification tower, an equalization tank and a sludge holding tank. Any of these ancillary units to the P-C plant could be constructed of materials which would facilitate relatively fast startup at a new location. Sizing of the plant components was based on a flow of 190 cu m/d (50,000 gallons per day). The treatment process sequence consisted of screening, flow equalization, chemical flash mix, flocculation, clarification, pH control, biological nitrification, filtration, granular activated carbon adsorbtion, and chlorination. Excess sludge from the clarifier was periodically transferred to the sludge holding tank from which settled solids were occasionally transported by truck to a disposal site. A treatment process flow schematic of the demonstration plant is shown in Figure 1.

Influent Flow Control

Wastewater for the demonstration plant was taken from an existing manhole above the lift station serving the Beechgrove Village development. A diversion dam within the manhole provided a flooded section from which the demonstration plant was fed. In the outlet pipe (0.2 m in diameter) an air-activated pinch valve was located in the flooded pipe which, when activated by liquid level controls in the equalization tank, opened and allowed diversion of the wastewater to the demonstration facility through the 845-m gravity line. The level controls were of the solid-rod type located directly in the main equalization basin. Signals from the electrodes were transmitted via a telephone circuit to a solid-state control relay located near the pinch valve. The relay in turn controlled a 3-way solenoid valve which was installed in an air pipe between the pinch valve and an air compressor. The air pressure in turn activated the pinch valve. Excess wastewater flow was discharged to the existing sewer from the manhole overflow.

Flow Equalization Tank and Screening

A bar screen with approximately 25 mm (1-in) openings was located in the influent structure of the flow equalization tank to remove larger objects that might damage the system. The flow equalization tank consisted of a rectangular 75.7-cu m (20,000-gal) prefabricated coated steel tank and incorporated a diffused-air system to ensure solids suspension and mixing and also to maintain aerobic conditions during storage. The buffer capacity of this tank allowed continuous operation during normal low flow conditions encountered at night. The tank also received filter and adsorber spent backwashes. In addition to the influent flow control liquid-level sensors, electrodes were also installed to provide emergency shut down of the remaining treatment processes in the event of low level conditions in the flow equalization tank. The wastewater was pumped from the flow equalization tank to the treatment unit by a constant-speed, progressive-cavity pump.

Chemical Clarification

Chemical clarification was achieved using hydrated line fed at a periodically adjusted constant rate in a 10 percent slurry form. Lime slurries were made up on a daily basis using commercial hydrated lime in 22.7 kg (50 lb) bags. A 1.363-cu m (360 gal) plastic tank served as the makeup and storage tank. Constant mixing of the lime tank was provided to maintain the slurry using a 0.37 kw (0.5 hp) constant-speed mixer. Lime slurry was fed to the 0.25-cu m (65-gal) flash-mix tank using a variable-speed, diaphragm-type slurry metering pump. A constant-speed mixer provided thorough mixing of the lime slurry with the incoming wastewater from the equalization tank. Theoretical detention time within the flash mix unit was 1.87 minutes at the design flow rate of 190 cu m/d (50,000 gal/d).

Flocculation was provided in a square-shaped 2.16-cu m (570-gal) tank. Agitation was carefully controlled using a variable-speed, vertical-shaft mixer. Theoretical detention time was 16.2 minutes at design flow rate.

Flow from the flocculation tank was introduced to the 17.83-cu m (4,710-gal) circular clarifier through a distribution box which channeled the flow to a peripheral-feed inlet near the bottom of the clarifier. A theoretical detention time of 135 minutes was available in the clarifier at the design overflow rate of 26 cu m/sq m/d (640 gal/sq ft/d). The design overflow weir rate was 21 cu m/m/d (1,700 gal/ft/d). The clarifier was equipped with motor-driven sludge raking and skimming and an effluent "V-notch" weir around the circumference of the tank.

pH Control

A neutralization step was necessary following lime clarification in order to prevent deposition of calcium carbonate in subsequent processes and to facilitate the biological nitrification process. For large-scale systems this is often accomplished by recarbonation of the high pH clarified wastewater with carbon dioxide (CO_2). For this facility, sulfuric acid was used because of the capital cost and space savings inherent in this approach.

Sulfuric acid was purchased in 49- or 57-liter (13- or 15-gal) plastic carboys, and the required solution was made up daily. A 0.3-cu m (80-gal) plastic tank was used for mixing and storage of the 20% sulfuric acid solution. A small mixer was installed in this tank to ensure the initial blending of the water and acid. A variable-speed chemical feed pump was used to transfer the solution to the 0.19-cu m (50-gal) neutralization tank for the lime-clarified effluent. Thorough mixing of the acid feed solution and high-pH effluent was provided. The tank was equipped with electrodes for pH measurement which provided signals to the pH control unit which, in turn, controlled the off-on operation of the acid feed pump. Experience demonstrated that acid added directly into the clarifier effluent piping upstream of the baffled neutralization tank (baffled to separate the mixing and sensing functions) were necessary to obtain satisfactory operation. The neutralization tank effluent was then pumped to the nitrification towers during most of the operational period, even though flexibility was available to

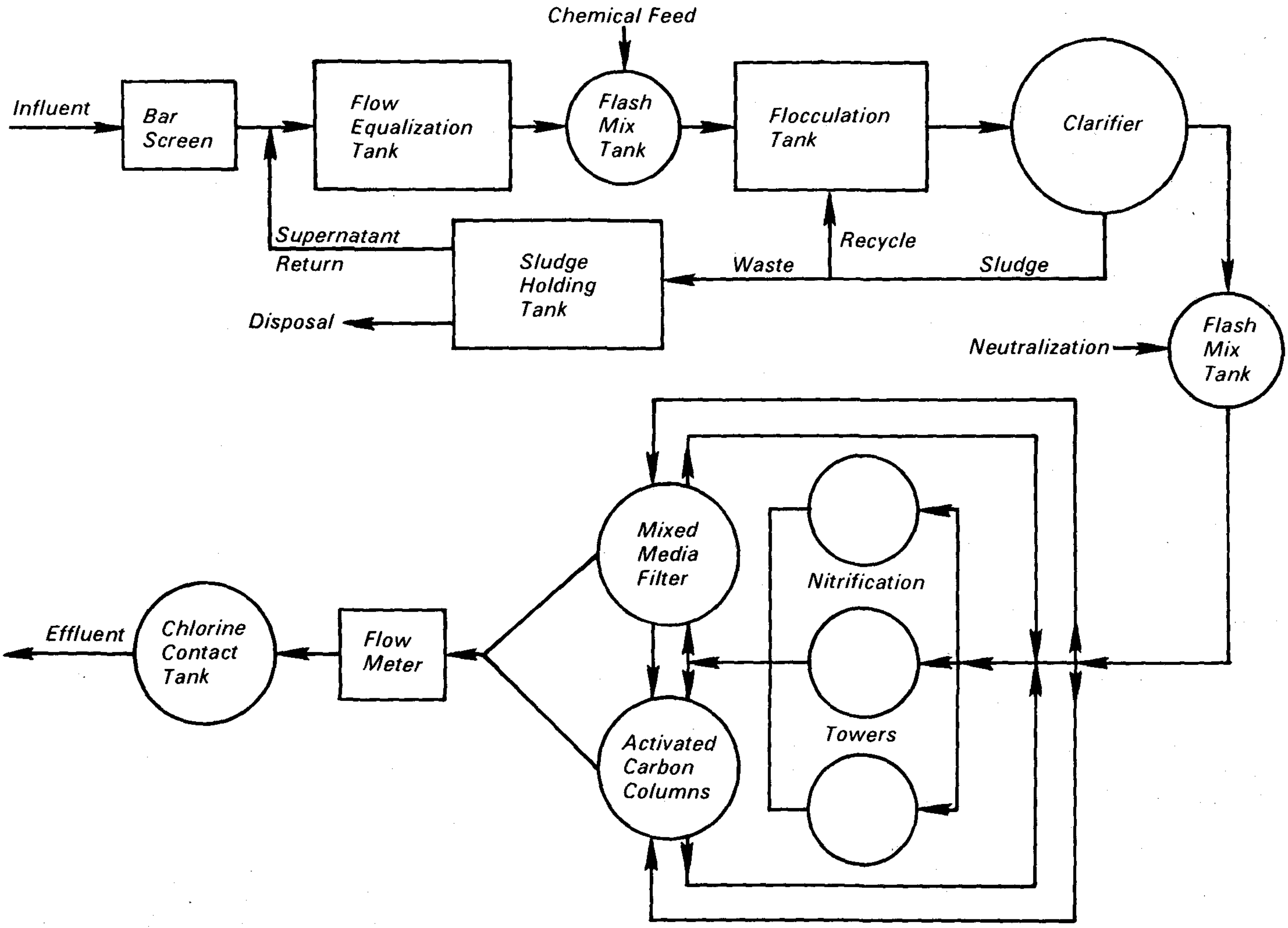

Figure 1. *Demonstration plant process flow schematic.*

vary the sequence of all subsequent processing steps.

Biological Nitrification

Three separate biological nitrification towers were constructed from 1.85-m (72-in) diameter concrete pipe sections. Overall height of each unit was 5.18-m (17-ft), and they were packed to a depth of 4.57-m (15-ft) with a high specific surface plastic media which was light-weight and provided 187 sq m/cu m (57-sq ft/cu ft) of surface area with 93 percent void space and a bulk density of 64 kg/cu m (4 lb/cu ft) by virtue of random packing in the towers.

The three nitrification units were designed for parallel operation, with adjustable flow rates to each unit. The system design allowed for total recycle of "seed" sludge in order to obtain a biological population capable of effecting nitrification within a reasonable period (4 to 6 weeks) after startup. Rotary distributor arms were used in each tower to provide uniform surface distribution. The underdrains from each tower discharged to a commom sump for pumping to the subsequent process. Based on the 7.88 sq m (84.8 sq ft) of surface area contained in the three towers, the design surface loading rate was 9.8 cu m/sq m/m (0.4 gal/sq ft/m).

As with all processes following clarification/neutralization steps, effluent from the nitrification towers could be directed to the dual-media filters or to the granular activated carbon adsorption towers; the former scheme was used throughout this study. System design was based on the presumption from earlier pilot studies that there would be a low net solids production associated with the nitrification towers so that intermediate clarification prior to filtration would not be required.

Dual-Media Filtration

The filter used was a downflow pressure system employing the dual-media concept, i.e., a 0.3-m (1-ft) layer of AWWA B 100 medium (0.45 to 0.55 mm effective size) sand overlain by a 0.3 m (1 ft) layer of anthracite, AWWA B 100 No. 1 (0.6 to 0.8 mm A 1.6-cu m) (424-gal) surge tank preceded the pressure filter and provided flow storage during the filter backwash cycle. In addition, the surge tank was equipped with liquid level sensors that served as controls for the pressure filter feed pump. Flow rate through the 0.66-sq m (7.1-sq ft) surface area filter was controlled by a variable orifice, pressure-compensated flow regulator at 12.2 cu m/sq m/h (5 gal/sq ft/m). Backwash operation was automatic with intervals between backwash cycles being operator selectable on a 24-hour time clock. The manufacturer suggested backwash initiation at 28 to 35 k Pa (4 to 5 psig) pressure differential across the pressure filter. This corresponds to a head loss of between 2.8 and 3.5 m (9.2 and 11.5 ft). Flexibility also existed for controlling the length of each backwash sequence. The chlorine contact tank served as the backwash source, and backwashing flow was regulated through a constant-flow control valve at a rate of 43 cu m/sq m/h (17.7 gal/sq ft/m). To facilitate backwash efficiency, a pre-backwash air source was provided. Spent backwash was returned to the flow equalization tank.

Granular Activated Carbon Columns

Two granular activated carbon columns were used to provide removal of dissolved organic matter. The two columns were operated in series with the first column being an upflow type and the second being of downflow design. Empty-bed contact time for each column was approximately 21.6 minutes. Each 1.22-m (4-ft) diameter by 2.44-m (8-ft) tall column contained approximately 1.22 m (4-ft) of granular activated carbon (Calgon Filtrasorb 300) underlain by a 0.3-m (1-ft) layer of selected gravel. Flow was introduced into the upflow column via a perforated distributor buried in the supporting gravel layer. In order to maintain a fluidized condition in the carbon bed at the liquid upflow rate of 6.8 cu m/sq m/h (2.8 gal/sq ft/m), a stream of air was also introduced at a manually controlled rate through a second perforated distributor within the gravel layer. Overflow from the upflow carbon column was screened prior to overflow to the surface of the downflow column. Backwash facilities similar to those for mixed media filtration were incorporated in the design of the downflow carbon column. The backwash flow rate was 19.5 cu m/sq m/h (8.0 gal/sq ft/m), and a surface wash was provided during the backwash cycle. The chlorine contact tank also served as the backwash water source for this operation, and spent backwash was returned to the equalization tank.

Effluent Subsystem

The effluent subsystem included a water meter for recording plant effluent flow and the chlorination facilities for disinfection of the final effluent. A 4.46-cu m (1,178-gal) chlorine contact tank provided a theoretical contact time of 33.6 minutes at the design flow. Chlorine was fed from a 45.4 kg (100 lb) liquid chlorine cylinder using a solution-feed, vacuum-operated gas chlorinator, mounted directly on the cylinder. The operating vacuum was provided by a hydraulic injector unit, with a close-coupled diffuser attached to a submersible pump mounted on the contact tank floor.

Sludge Handling Facilities

A 30.28-cu m (8,000 gal) rectangular sludge storage tank was provided to handle the excess lime sludge from the chemical clarification unit. As lime sludges generally show good settling properties, provisions were made in the storage tank to gravity thicken the sludge. Supernatant drawoff ports were placed at selected elevations along the upper section of the sludge storage tank to allow decanting of the supernatant during settling. The decent was returned to the flow equalization tank.

A diffused-air system was installed to prevent anaerobic conditions and excessive compacting and to facilitate removal of the thickened sludge. Couplings were installed at the bottom sludge draw-off valve to allow tank truck disposal of excess accumulated solids. The design and intent of the sludge storage-thickening unit was to aerate the sludge to prevent anaerobic conditions and to periodically stop aeration to permit thickening and subsequent supernatant drawoff. Withdrawal of thickened solids for disposal was permitted only during the aeration cycle to assist in fluidizing the tank contents for easier withdrawal.

Evaluation Factors

Sampling

Automatic composite samplers were used for collecting samples from the equalization tank (influent) and the effluent from the carbon adsorbers prior to chlorination (effluent). Also, periodic grab samples were taken of the clarifier effluent, neutralization tank effluent, nitrification tower effluent and filter effluent. All samples were refrigerated including samples for biochemical oxygen demand (BOD_5) and suspended solids (SS). Besides refrigeration, samples for chemical oxygen demand (COD), total organic carbon (TOC), total Kjeldahl nitrogen (TKN), ammonia nitrogen (NH_3-N) nitrite nitrogen (NO_2-N), nitrate nitrogen (NO_3-N), acid-hydrolyzable phosphate (AHP), and orthophosphate were further preserved by the addition of 2 ml of H_2SO_4 per liter of sample following collection. All testing was done in conformance with "Standard Methods for Examination of Water and Wastewater," Fourteenth Edition, 1975.

Construction and Start Up

Project planning and plant design and specifications were completed in February 1975. Due to the nature of the project and the equipment required, two separate contracts were awarded. One contract encompassed the skid-mounted physical-chemical treatment system, while the other covered site work, nitrification towers, the flow equalization tank, the sludge storage/thickener tank, and other miscellaneous items. All bids for both contracts were considerably in excess of the budget limits of the project. Negotiations with the low bidders coupled with numerous design changes resulted in the eventual signing of both contracts within the original budget estimates. The physical-chemical (P-C) plant was delivered in January 1976. The work scheduled in the second contract was to be completed in late November 1975, but due to financial difficulties on the part of the contractor and subsequent unanticipated requirements resulting from this problem, the construction phase and initial testing were not completed until late 1977.

Numerous problems were encountered during the initial attempts to check out the individual units in the system and to verify their proper operation. The treatment system was designed for above ground operation to allow for a short installation time and to facilitate movement of the system to another location, should the need arise. The P-C system was delivered to the site in January 1976. Because of the serious delays in completion of the other contract, this equipment was left at the site, unused, for two years including two winter seasons of unusually cold weather. Proper precautions were not taken to protect the units during this long period. Numerous pipes, valves, fittings, and pumps suffered substantial damage, requiring replacement or repair. Breakdowns encountered with pumps and motors continued to be a major problem during the entire operational phase of the project, probably caused by the long exposure.

Results

Overall Removals

Prior to system design 10 twenty-four hour composite samples of the raw wastewater gave the results presented in Table 1.

During the operational phase, the average BOD_5 of the equalization tank samples average characteristics were 159 mg/l of BOD_5, 368 mg/l of SS, and 435 mg/l of COD, as shown in Table 1 in parentheses. The significance of the differences between these values is not clear, since some are lower and some higher. Certainly, some changes could be due to the aerated flow equalization tank prior to the pumping to the clarification unit. Since the theoretical retention time in the equalization tank was between 5 and 10 hours, some biological oxidation could have occurred, and the recycle of certain streams from the treatment system to this tank would also account for some variance.

A summary of the treatment efficiencies achieved with each of the units (clarifier, nitrification tower, dual-media filtration, and carbon columns) is presented in Table 2. The percent of removal of BOD_5, COD, TOC, SS, acid-hydrolyzable phosphorous, and total nitrogen, is presented. The data represent paired samples where influent and effluent samples were taken from each unit and analyses performed. Thus, the percentage of removal within one unit is calculated from the difference of the influent and effluent of that unit. The percentage of cumulative removal is calculated from the difference between the clarifier influent and the effluent of that unit. As can be seen, the removals of BOD_5, COD, TOC and SS were excellent, with cumulative removals ranging from 88 percent for COD to 97 percent for suspended solids. The greatest amount of organic material and suspended solids was removed during the lime clarification.

Phosphorus removal from an influent concentration average of 12.3 mg/l was also excellent but the nitrogen removal was rather low. The major portion of the phosphorus was removed during the lime clarification. High lime feed with a higher pH (11.4 as compared to 10.7 for *the low lime feed) significantly increased* the removal of phosphorus, i.e., from 63 to 87 percent. Recycle or non-recycle of clarifier sludge had little influence on the removal of phosphorus.

Nitrification never properly developed during the course of the study, even though nitrogen removals averaged 40 percent from the average influent concentration of 38 mg/l during the last eight weeks of operation. The overall removal of nitrogen averaged 32 percent, with losses nearly equally split between the clarification, filtration, and carbon adsorption processes. In the first two processes these removals can be attributed to the organic nitrogen content of the solids removed. In the last process nitrogen removal appears to have been due to denitrification in the carbon beds.

Individual Process Performance

As noted in Table 2, the lime clarification step accounted for the major portion of removal of all pollutants measured. From the standpoint of defined secondary effluent quality, the *clarifier effluent nearly met the BOD_5:SS* requirement of 30:30, with actual values of 46:21. Organics, as measured by BOD_5, COD and TOC were removed by average rates of 66 to 77 percent, while 82 percent of the acid-hydrolyzable phosphorus was removed.

The nitrification tower, though seemingly ineffective in its intended role as measured by nitrogen series analysis, did provide significant additional removals of BOD_5, TOC and COD. The reasons for the apparent lack of nitrification remain somewhat mystifying based on earlier published data which indicated that the designed system should be able to oxidize 3.2 to 8.2 kg of NH_4-N/day (7 to 18 lb/d). Since the approximate loading was 5.0 kg of NH_4-N/day (11.0 lb/day), the resulting oxidation, as measured by NO_2-N and NO_3-N increase, of 0.27 kg/day (0.6 lb/day) was disappointing. This is especially true in light of favorable wastewater temperatures and BOD_5/

Table 1. *Wastewater Characteristics**

BOD_5	COD	TS	VTS	SS	VSS	DS	VDS	pH	Alkalinity as $CaCO_3$
239 (159)+	370 (435)+	974	467	411 (368)+	219	562	248	7.0	278

* *all analyses in mg/l, except pH*
\+ *operational phase averages*

Table 2. *Project Data Summary*

Subsystem	*Parameter Measured** BOD_5	COD	TOC	SS	AHP**	TN***
Clarifier						
% Removal	77	73	66	86	82	13
% Cumulative	77	73	66	86	82	13
Nitrification						
% Removal	44	13	42	Neg.	Neg.	Neg.
% Cumulative	84	76	80	85	80	11
Dual-Media Filter						
% Removal	—	40	5	71	17	11
% Cumulative	91	86	82	95	85	24
Carbon Columns						
% Removal	33	15	20	39	Neg.	10
% Cumulative	93	88	86	97	80	32

* *Data represents paired samples where influent and effluent analyses were performed.*
** *Acid-Hydrolyzable phosphorus*
*** *Total Nitrogen*

TKN ratios. Initial attempts to provide seeding to the nitrification towers were unsuccessful due to hydraulic deficiencies in the plant, but sufficient operating time was available for natural development of nitrifiers. The fact that no such development took place would appear to be due to either the unreliability of the neutralization step and/or the lack of recirculation.

The dual-media performed well in terms of solids removal and concomitant removals of organics, TKN and AHP associated with those solids. Suspended solids removals of 70 percent were achieved, along with COD, TOC, TKN and AHP removals of 40, 5, 11 and 17 percent, respectively. However, the media sizes were not well-suited to handling the 44 mg/l of SS (average) found in the filter influent. Therefore, filter runs were frequently as short as four hours, which represented a considerable O/M problem, because of the fine coal size provided with the filter.

The activated carbon columns were loaded very lightly during this study. The COD removed by the adsorbtion process had reached 0.18 lb. of COD per lb. of activated carbon by the end of the project, no apparent reduction in the rate of COD removal verified that the carbon had not been exhausted. The system was designed with the capability of removing spent carbon and adding fresh carbon. Denitrification of the nitrate produced by the nitrification towers did occur in the carbon columns, and no hydrogen sulfide problem was encountered.

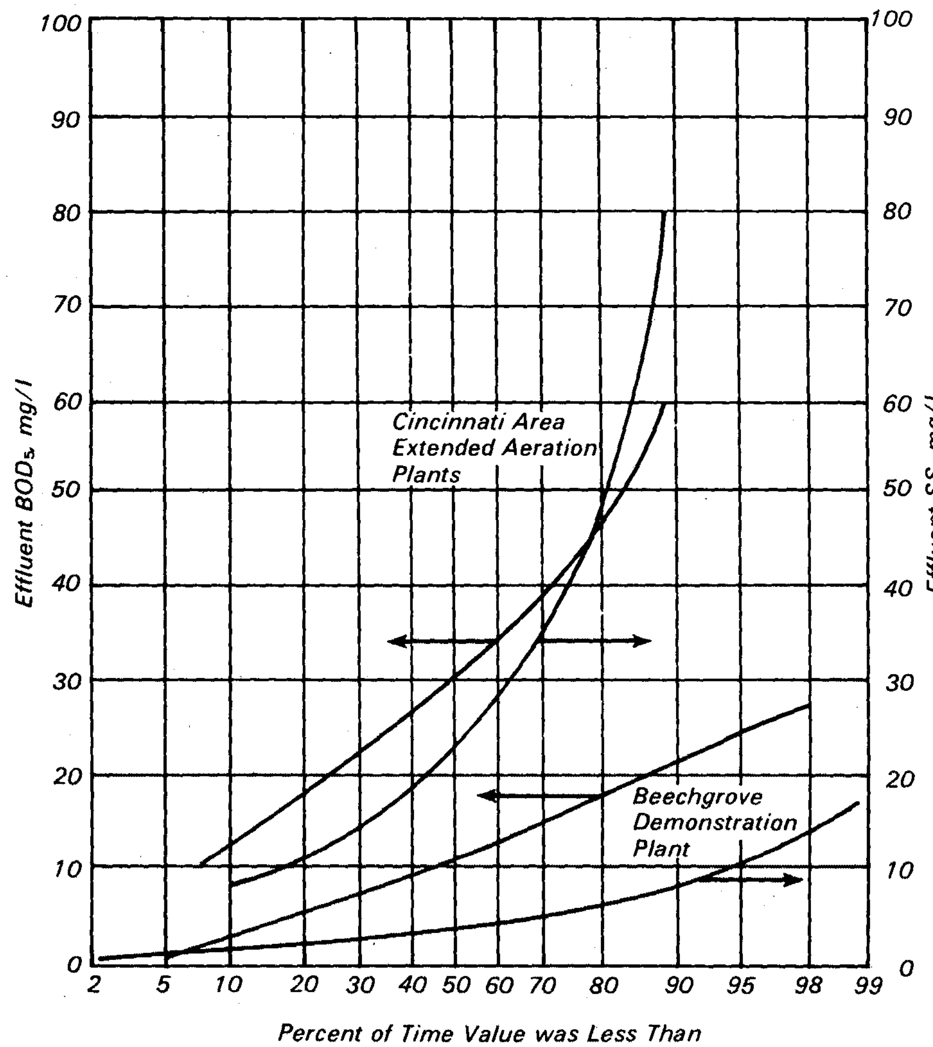

Figure 2. *Comparison of BOD_5 and SS reliability.*

Performance Reliability

In spite of the equipment and operational problems encountered, the hybrid (physical-chemical/biological) treatment plant, as designed, was able to produce a consistent, high-quality effluent, when compared to typical biological systems used to treat wastewaters from small communities. Figure 2 compares the reliability of this hybrid system for the removal of BOD_5 and SS versus extended aeration plants in the Cincinnati area. Since this hybrid plant also removes phosphorous, other biological systems would require ancillary treatment steps to provide comparable performance characteristics.

Operation and Maintenance

The normal operation and maintenance of this plant was more time consuming and complex than that associated with most biological treatment plants. Some of the work involved the sampling and laboratory testing required at the site for the experiments of this project and included sample preparation and delivery and preparation of logs and records. However, there were a number of pumps, tanks, mixing chambers, and backwash systems which had to be cleaned, adjusted, and occasionally repaired. The mixing of chemicals (lime slurry and acid neutralizer) required knowledge of the operations and caution to avoid chemical burns. One full time operator was required with additional manpower required for any unusual problem. Weekend coverage of the plant was also provided.

In order to properly conceive of the O/M requirements, it should be noted that an extended aeration package plant capable of handling the same design flow normally requires approximately 0.5 person-years/year. Therefore, the manpower required for the hybrid system was approximately three times that required for an extended aeration system. Likewise, increased chemical costs are inherent to the hybrid system design. The value of relatively instantaneous, high-quality effluent would have to be weighed against these increased O/M costs on a case-by-case basis. The question of initial cost comparison is far more difficult because of the transport-ability of the physical-chemical portion of the hybrid plant. Multiple use of such a system by a public or private entity at different sites would determine whether such a system would be economical.

Discussion

Two factors were intended for testing in this study, the technical feasibility of the treatment sequence and the concept of transportability. Although certain shortcomings arose in the testing of these factors, certain implications of the study are relevant to each.

The transportability concept is important to agencies such as DHUD in that the potential health and ecological dangers which often result from natural or man-made disasters might be minimized through prompt response with nearly instantaneous high quality treatment capability to meet most water quality limitations. To a lesser degree, an adjunct treatment capability for "boom towns" or other sudden population increases, which in recent times have been associated with energy development, could obviate the potential impacts on a fragile ecology due to sudden overload of existing sanitary facilities and infrastructure.

As noted earlier, the physical-chemical (P-C) portion of the hybrid treatment plant was skid-mounted and transportable from site-to-site by tractor trailor. The associated process needs, i.e., equalization and sludge handling, could quickly be provided at almost any site by excavation and lining or otherwise sealing of the soil to prevent seepage and/or introduction of debris to the wastewater, if such tankage is not already available. Therefore, a complete (P-C) unit could be quickly operable at such locations, assuming necessary power provisions at the plant site. The nitrification tower is an unlikely addition in the event that a nitrogen standard must be met, not because of its marginal performance during this study, but because its inherent lag time to reach proper nitrification is inconsistent with the otherwise quick startup potential of the unit. Therefore, the P-C system alone would serve the transportability function quite well if no nitrogen standard were in effect and offer the added benefits of phosphorus removal and consistently high quality performance. Introduction of a nitrogen standard would probably require the use of breakpoint chlorination or stripping towers in order to provide relatively instantaneous nitrogen removal consistent with the overall plant characteristics.

The technical feasibility of the hybrid facility's processing sequence is a separate issue. The concept of utilizing biological nitrification with physical-chemical processing was designed to overcome two basic weaknesses in the P-C treatment concept, i.e., high NH_4-N concentrations in the effluent and odors associated with the carbon adsorbers. The latter problem had been overcome by the addition of NO_3-N to the influent of carbon adsorbers in sufficient quantity to prevent H_2S formation. The hybrid facility was designed to utilize the nitrogen already in the wastewater by converting, all or part of, it to the NO_3-form prior to carbon adsorption. Although the perceptible nitrification was minimal, the total system did remove about 30 percent of the nitrogen in the wastewater, as opposed to the original estimate of 36 percent. The major difference in the actual *vs.* estimated effluent quality was the form of the nitrogen, i.e., NH_4-N rather than NO_3-N, which could result in a significant oxygen demand in the receiving stream.

The overall acceptability of a wastewater treatment system is based on a variety of factors including capital and O/M costs, labor requirements and performance characteristics. If one assumes that the reasons for poor nitrification tower performance can be easily overcome through improved neutralization and nitrification tower design, the hybrid design studied (with proper filter media) is capable of producing a high-quality effluent unmatched by either pure biological or pure physical-chemical systems, incorporating the positive features of both systems, e.g., compact size, reliability, resistance to toxic upset, improved toxics removal, phosphorus removal, non-odorous operation, and nitrogen reduction with ammonia removal.

The EPA authors ***James F. Kreissl*** *(also the EPA Project Officer, see below) and* ***Ronald F. Lewis*** *are with the Municipal Environmental Research Laboratory, Cincinnati, OH 45268.*

The complete report, entitled "Demonstration Physical Chemical Sewage Treatment Plant Utilizing Biological Nitrification," was authored by E. Brenton Henson of the Sanitation District No. 1 of Campbell and Kenton Counties, Covington, KY 41011 (Order No. PB 82-101 643; Cost: $9.50, subject to change) will be available only from:

National Technical Information Service
5285 Port Royal Road
Springfield, VA 22161
Telephone: 703-487-4650

The EPA Project Officer can be contacted at:

Municipal Environmental Research Laboratory
U.S. Environmental Protection Agency
Cincinnati, OH 45268

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