UPGRADING EXISTING WASTEWATER TREATMENT PLANTS

TECHNOLOGY TRANSFER DESIGN SEMINAR

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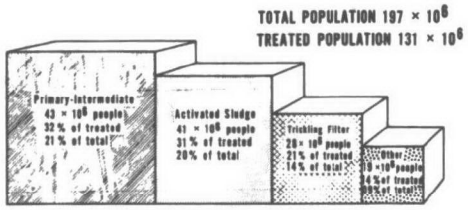
INTRODUCTION

It is estimated that an investment of 18.1 billion dollars will be required for the construction of municipal wastewater treatment facilities in the United States to meet the 1976 projected Federal State Water Quality Standards (1). Approximately 1/4 of this amount will be used for upgrading the performance of the existing wastewater treatment facilities.

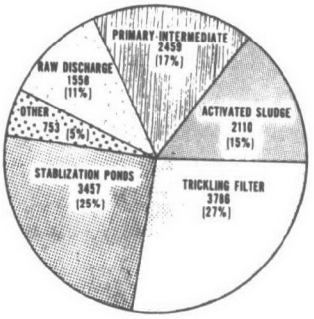
The distribution of these facilities according to population is presented in Figure 1. This figure indicates that 32% of the population receiving treatment is served by primary-intermediate treatment facilities, 31% by the activated sludge process and 21% by the trickling filter process.

FIG. 1



Treatment Classification by Population
EPA Survey 1968

FIG. 2



Total Facilities 14,123
EPA Survey 1968

Another way of looking at the in-ground municipal treatment plant investment is to examine the total number of existing plants and their distribution according to type. This is shown in Figure 2. Comparison of Figures 1 and 2 indicates that there are more trickling filter plants than activated sludge plants, but that more people are served by the activated sludge process. Simi-

lar comparisons can be made for the remaining treatment types.

The application of todays technology in upgrading plant performance includes (a) techniques that can be used to maintain the original treatment plant efficiency under increasing organic and/or hydraulic loading, (b) the addition of processes that can be used to increase overall plant removal efficiencies and (c) process additions or modifications for specific contaminant removal. Physically, these upgrading procedures may be applied ahead of the plant; as modifications of the treatment process itself; or as effluent polishing techniques.

Regardless of the techniques employed, cost effective treatment plant upgrading requires efficient use of available tankage and equipment, along with implementation of the best possible operating and maintenance programs. Initial investigations of potential upgrading situations must include a thorough examination of existing plant equipment and past performance history as well as a complete understanding of existing plant deficiencies and future treatment requirements. This information can then be used to formulate alternate courses of action, and finally to select the most effective solution.

PRE-PLANT CONSIDERATIONS

Infiltration and Flow Reduction

Historically, the construction of municipal wastewater treatment plants has been a "catch up" phenomenon. Under our current system of plant monitoring there has been little if any motivation to improve treatment plant efficiency until the plant is severely overloaded, many times to the point of routinely by-passing untreated or poorly treated wastewater.

A first step then, in upgrading many facilities, is to examine techniques to alleviate or reduce hydraulic overload. The first approach here is not at the plant. Control of groundwater infiltration, the reduction of extraneous surface sources into sanitary sewers, and the reduction of household water usage should all be thoroughly examined before considering any in-plant changes.

Nationally, there are nearly 3 billion feet of public sewer, the majority of which are constructed below the prevailing groundwater tables and therefore are subject to infiltration. Defective sewer pipe, faulty pipe joints and poor manhole construction are the principle causes of infiltration. In the United States infiltration flows average 15% of the total sewage flow, and during prolonged rainy periods amount to as much as 30% of the total flow. The entrance of extraneous surface water from such sources as roof leaders, manhole covers, cellar and foundation drains and other illegal connections can add another 20% to the total sewage flow (2).

While the adoption of tighter (200 gal/day/inch diameter/mile) infiltration specifications, better inspection during construction, improved construction methods and materials, and legal control of house laterals insepction, can virtually eliminate excessive infiltration for new systems, the reduction of excess flows in older systems is a perplexing and costly propositon. The use of closed circuit TV, dye testing and smoke testing have proven quite valuable in locating faulty joints and illegal connections. Studies have shown that TV inspection of 8" diameter sewer lines can be accomplished for about $0.20-0.30 per lineal foot, including labor, equipment, and supplies. Complete sealing of all joints can be accomplished at a speed of about 300 ft/day using a three man crew at a total cost including inspection of $1.70 per lineal foot. This is equivalent to a joint cost of $5.79. In one study (2), sewer sealing by this

method reduced the total sewage flow by 40%. Effective overall excess flow control will require reduction of extraneous surface sources in addition to adequate infiltration control. Control of infiltration and extraneous surface sources can be complimented by reduction of water usage in the home.

A recent EPA sponsored study (3) has shown that both water-saving devices and recycling systems for non-potable use in the home can significantly reduce the per capita sewage contribution from the average household. The installation of shallow-trap and dual flush toilets resulted in a toilet water usage savings of 20%, while flow restricting shower heads decreased the shower usage by as much as 35%. Wash-water recycle systems have decreased total household water usage from 24 to 35%.

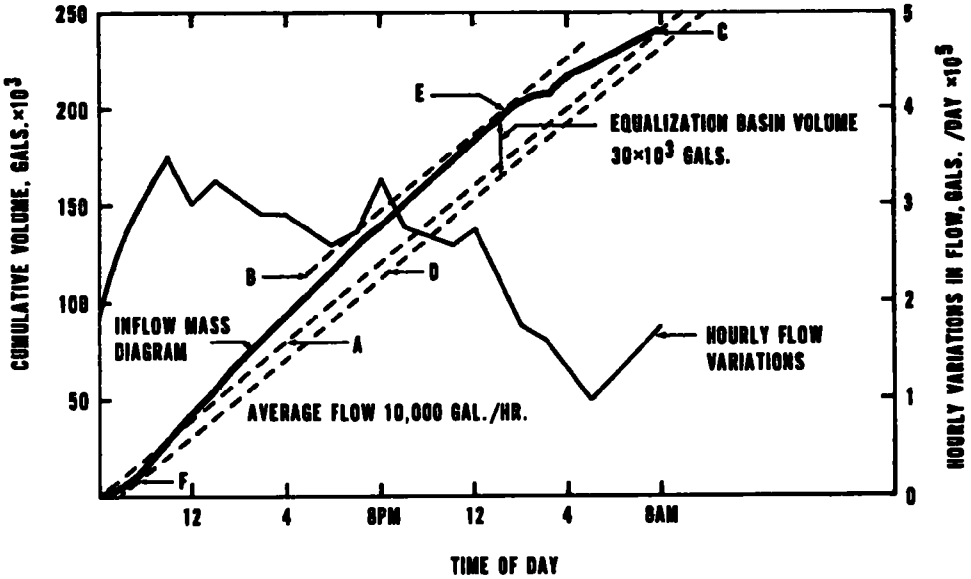
Flow Equalization

Equalization of the diurnal variation in incoming sewage flows to a treatment plant can relieve hydraulic overload, and if properly designed, can significantly dampen the variation in mass flow of contaminants into the plant. It has long been recognized that operation under these quasi-steady-state conditions is necessary and desirable for optimum operation of both biological and chemical-physical plants.

One way of determining the required equalization volume for a given plant is to plot an inflow-mass diagram of the hourly fluctuations in sewage volume for a typical day. This variation is shown in Figure 3 along with the associated inflow mass diagram. The ordinate of this diagram is obtained by accumulating the hourly flows for a given plant and converting them into volumes. The slope of line "A" in Figure 3 represents the average "outflow" pumping rate. The vertical distance between lines "B" and "D", which are drawn parallel to line "A" at the maximum and minimum points of the inflow mass diagram, represent the minimum required equalization volume for constant outflow. For most diurnal variations, this volume amounts to 12-15% of the average daily flow. Either a flow-through or side-line basin may be employed for flow equalization. The volume requirements are the same in either case. In the flow-through tank, all the flow passes through the equalization basin and much better mixing or mass-flow dampening is possible. In the side-line tank only the amount of flow over the average is retained. This scheme minimizes pumping requirements at the expense of less effective mixing. A flow diagram for a treatment plant employing flow equalization using a side-line tank is shown in Figure 4, and a summary of operating data for this plant is shown in Table 1.

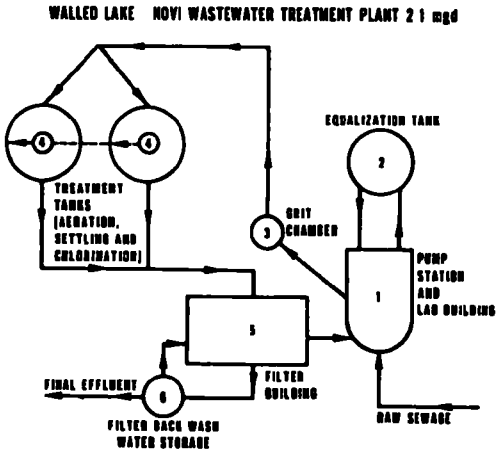
Flow equalization tanks should be designed as completely-mixed basins, using either diffused air or mechanical surface aerators. For systems using surface aeration equipment, the mixing requirements will vary from 0.02 to 0.04 HP/1000 gallons of storage volume for a typical municipal wastewater having a suspended solids concentration of 200 mg/l. Aeration to prevent septicity must also be provided. This equipment should be sized to supply 10-15 mg O_2/l/hr.

FIG. 3
EQUALIZATION REQUIREMENTS



Equalization Requirements

FIG. 4



Walled-Lake Novi Wastewater
Treatment Plant 2.1 mgd

TABLE 1
Performance Data - Walled Lake Novi
Plant

PERFORMANCE DATA WALLED LAKE-NOVI PLANT				
MONTH	REMOVAL %		EFF.	
	BOD	SS	BOD	SS
SEPT. 71	91.6	96.5	6	4.1
OCT. 71	98.6	98.2	2	3.4
NOV. 71	98.6	93.7	2.3	18.7
DEC. 71	98.8	92.3	* 2.3	19.0
JAN. 72	98.2	96.1	3.5	9.0

IN-PLANT MODIFICATIONS AND ADDITIONS

Clarification

Improvement in primary and secondary clarification can be accomplished either by structural modifications of the existing basin, by improved operation and sludge management practices, or by the addition of chemicals to improve coagulation and settling. Improper basin inlet design, high weir overflow rates, short circuiting and lack of surface skimming devices are the principle causes of sub-standard clarifier operation. Many of these deficiencies can be remedied by inexpensive structural changes to the existing clarifier such as the addition of scum baffles, strategically placed distribution plates, or additional weirs. West (4) has reported that increasing the sludge recirculation capacity, measuring effluent turbidity, and using a sludge blanket finder to control sludge wasting decreased the effluent BOD from 20 to 10 mg/l and the effluent suspended solids from 35 to 13 mg/l at a 3.5 mgd activated sludge plant in Sioux Falls, South Dakota. West (4) reports further that in St. Louis, Missouri the use of a sludge blanket finder, a turbidimeter, increased air supply and reduced sludge recirculation, decreased effluent BOD from 40 to 9 mg/l and effluent suspended solids from 92 to 16 mg/l in a conventional activated sludge plant.

The most recent innovation in clarifier design is the tube settler concept. A discussion of this subject by Hansen and Culp (5) and later papers by Culp, Hansen and Richardson (6), and Hansen, Culp and Stukenberg (7) have described the theoretical and practicable aspects of both the horizontal and steeply inclined tube settlers. Tube settlers have applicability for upgrading both primary and secondary clarifiers. Culp et.al. (8) has described the performance of several plant scale tube settler installations. Results of these studies indicate that overflow rates of up to 5,000 gpd/ft^2 can be maintained in primary clarifiers without sacrificing removal efficiency. Tube settlers in this application however, do not improve the basin efficiency much beyond the 40-60% removal range. Conley (9) has recommended the use of 1.0 gpm/sq.ft. as a maximum overflow rate and 35 lbs./sq.ft./day as a maximum solids loading for the design of secondary clarifiers with tube settlers. Since the flocculating and settling characteristics of sludge vary from plant to plant, each case should be evaluated separately for suitable design criteria.

Fouling due to attachment and growth of biological slime on the sides of the tubes is sometimes a problem. Some form of cleaning device (water jet or air) is required so that the solids build-up can be removed occasionally. The performance of clarifiers using tube settlers at various installations is summarized in Table 2. Little information is available at the present time to establish cost information on tube settlers. However, an estimating cost figure of 12 to 20 dollars/sq.ft. for tube settlers with an installation cost of 5 to 15 dollars/sq.ft. has been recommended by the manufacturer (9).

Chemical Addition

The use of chemicals to improve treatment efficiency, or for specific contaminant removal is now standard practice in many locations (11)(12). Chemicals commonly used are the salts of iron and aluminum, lime, and synthetic organic polyelectrolytes.

TABLE 2
Performance of Tube Settlers (10)

Plant Location	Type	Size mgd	Tube Location	Existing Facility SOR[1] gpm/sq ft	Existing Facility Eff SS mg/l	Operational Data Using Tube Settlers SOR[1] gpm/sq ft	Operational Data Using Tube Settlers Eff.SS mg/l
Philomath, Oregon	Trickling Filter	0 15	Secondary Clarifier	0.6	60-70	3 3-4 6	60-70
Philomath, Oregon	Trickling Filter	0 15	Primary Clarifier	0.84	40-45[2]	2 1-3 3	34-41[2]
Hopewell Township, Pennsylvania	Activated Sludge	0 13	Secondary Clarifier	0 34	60-70	2-3	27
Miami, Florida	Activated Sludge	1 0	Secondary Clarifier	1 3	500	1.7	33

[1]SOR - Surface Overflow Rate
[2]Percent removal rather than concentration

The metal salts (iron and aluminum) react with the alkalinity and soluble orthophosphate in the wastewater to precipitate the metal hydroxides or phosphates. Of equal importance in upgrading applications, is the ability of these hydroxides to destabilize colloidal particles that would otherwise remain in suspension. The destabilized colloids then flocculate and settle readily, thus improving the solids removal efficiency during clarification.

Iron and aluminum salts have been widely used in many locations to precipitate phosphorus from both activated sludge and trickling filter plants. The most common locations within the plant for adding these minerals are (1) the primary clarifier, (2) directly into the aerator of an activated sludge plant, and (3) before the final clarifier. Although minimum metal dosage is dictated by incoming phosphorus levels and desired removal efficiencies, this amount is normally sufficient for improved flocculation of suspended solids. Results of metal salt addition on suspended solids and BOD removal for some selected locations are shown in Table 3.

TABLE 3
Effect of Metal Salt Addition on Suspended Solids and BOD Removal Efficiency

Location	Type of Plant	Location of Mineral Addition	Metal and Dosage As Shown	Eff (COD) or BOD Before Metal Add mg/l	Total Phos Removal	Eff SS Before Metal Add mg/l	Eff (COD) or BOD After Metal Add mg/l	SS After Metal Add mg/l	(COD) or BOD Removal After Metal Add mg/l	SS Removal After Metal Add	Reference
Richardson, Texas	Trickling Filter std rate	Before Final Settlers	150 mg/l $Al_2(SO_4)_3$	20	93	20	7	10	-	-	(13)
Chapel Hill, North Carolina	Trickling Filter high rate	Before Final Settling	AL/P Mole Dosage 1 7/1	40	78	67	19	30	-	-	(13)
Texas City, Texas	Activated Sludge	Before Primary Settling	19 mg/l $FeSO_4$ as Fe	-	87	-	(85)	-	-	-	(14)
Wyoming, Michigan	Trickling Filter	Before Primary Before Final	15 mg/l Fe^{+3} 3-4 mg/l Fe^{+3}	-	80	-	-	75(1)	60-87		(12)
Lake Odessa, Michigan	Trickling Filter	Before Primary	15-25 mg/l Fe^{+2}	89(2)	82	78(2)	82(2)	62(2)			(14)
District of Columbia (Pilot Plant)	Activated Sludge Step Aeration	In Aerator	AL/P 2/1 wt Ratio	-	-	-	12	33	89(3)	68(3)	(15)
Manassas, Va	Activated Sludge high rate	In Aerator	Sodium Alum 14 B_3mg/l Al	-	61	-		41	(74)	69	(16)
	Three Sludge System - 2 Stage Activated Sludge Denitrification	In First Stage Aerator	Sodium Alum. 14 B_3mg/l Al	-	78	-	-	29	(91)	80	
Pomona, California	Conventional Activated Sludge	In Aerator	Alum Al/P Molar Ratio 1 9/1	(35 7)	74	9.8	(44 6)	29 7			(16)
	Conventional Activated Sludge	In Aerator	Iron Fe/P Molar Ratio 2 2/1	(35 7)	86	9 8	(38 6)	18 2			
			Fe/P 1 5/1	(35 7)	79	9 8	(37 2)	18 0			
			Fe/P 3/1	(35 7)	77	9 8	36 7	54 4			
Penn State	Conventional Activated Sludge	Aerator Effluent	Al/P wt Ratio 3/1	13	86	26	9	22			(17)

(1) removal across primary clarifier
(2) % removal
(3) removal across secondary treatment

In general, the addition of minerals in the dosage range shown will produce 80-90% total phosphorus removal. If added to the plant ahead of the primary, the addition of an anionic polymer is often recommended to minimize the carry-over of insolubilized phosphorus to the subsequent treatment stages. The addition of minerals to the aerator or aerator effluent of an activated sludge plant will generally result in more efficient coagulant utilization than when added to the primary because, during biological treatment, the complex phosphorus forms are hydrolyzed to the ortho form which is more easily precipitated. The addition of metal salts to the primary tends to dilute the resulting primary sludge. If added to the aerator, the metal has a weighting effect which results in a thickening of normal activated sludge.

The addition of metal salts to the aerator of an activated sludge plant produces moderate increases in suspended solids removal efficiency in some instances, but on many occasions results in significant increases in suspended solids carry-over. Mulbarger (16) states that this is due more to an incorrect ratio of volatile solids/aluminum added than to a strict overdose of chemicals. Results at Pomona, California indicate that increased turbidity is due to lowering the pH beyond the optimum limits for alum flocculation. The addition of 2 mg/l of polyelectrolyte prior to final clarification reduced the effluent turbidity to its previous value.

The addition of polyelectrolytes alone, in the 0.25-1.0 mg/l range has been shown to improve suspended solids removal of primary clarifiers from 37.7% to 64.7%, and BOD removal from 31.0% to 46.7% (10). The effectiveness of lime addition in increasing the efficiency of primary clarifiers at several locations is shown in Table 4.

TABLE 4
Effect of Lime Addition on
Primary Clarifier Performance

Location	Lime Added mg/l CaO	Percent Removal Before Lime Addition		Percent Removal After Lime Addition		Remarks
		BOD	SS	BOD	SS	
Duluth, Minnesota	75	50	70	60	75	—
	125	55	70	75	90	—
Rochester, New York	140	—	—	50	80-90	Jar tests
Lebanon, Ohio	145	—	—	66	74	Pilot plant

The addition of lime to primary clarifiers can nearly double the mass of primary sludge to be handled, depending on the alkalinity of the incoming wastewater, and will raise the primary effluent pH to the 9-11 range, depending on the lime dosage. The addition of lime at this location will decrease the organic loading to the subsequent biologically stage in many instances to where complete nitrification will occur. If this happens, the nitric acid and carbon dioxide produced by the nitrification and carbon oxidation reactions are adequate to maintain the pH within the aerator at a near optimum level for nitrification (12). A principal consideration in selecting lime addition to the primary clarifier as an upgrading technique is whether or not the existing sludge handling facilities are adequate and if not, whether alternate sludge handling schemes are possible.

Process Modifications

Examination of the various modifications of the activated sludge and trickling filter processes have identified situations where overall plant performance can be improved by process modifications within the plant, or by adding another biological stage in series with the existing plant.

Many such examples of in-plant modifications are contained in EPA process design manual for upgrading wastewater treatment plants (10) Three of these examples are shown below to illustrate the principles involved and the anticipated improvement in effluent quality.

(1) Upgrading Using Chemical Addition to Primary Clarifiers and Conversion of Conventional Activated Sludge to Contact Stabilization

This plant was originally a parallel activated sludge (1.2 mgd) and trickling filter plant. Flow into this plant at the time of upgrading had increased to 6.0 mgd and treatment efficiency had declined to 75% removal of BOD and SS.

The activated sludge portion of the plant was upgraded from 1.2 to 3.0 mgd by conversion of the primary settlers to final settlers, and by changing the existing aeration basin from conventional activated sludge to contact stabilization. This was accomplished by increasing the capacity of the mechanical aerators from 40 to 110 HP (60 HP in the contact basin and 50 HP in the stabilization basin) and by appropriately modifying the basins piping. This type of modification is possible since experience has shown that the contact stabilization process is well suited to a feed containing a high portion of BOD in the suspended or colloidal form. These alterations were designed to increase BOD removal and suspended solids from 75 to 90%. With this modification, the two stage filters were changed from parallel operation to series flow, and were again able to remove 70-80% of the primary effluent BOD. Overall trickling filter plant BOD removal was increased from 82 to 90%.

(2) Use of a Roughing Filter to Upgrade an Existing Low Rate Trickling Filter Plant

The existing single stage trickling filter plant was designed for an average flow of 0.7 mgd with 85% BOD and suspended solids removal. At the time of upgrading, the flow had increased to 1.15 mgd with some deterioration in suspended solids and BOD removal efficiency.

The plant was upgraded by replacing the existing comminuters and primary clarifiers with three 1.0 mgd Hydrosieve units. A plastic media roughing filter designed for an application rate of 2.5 gpm/ft^2 and an organic loading of 520 #BOD/1000 cu.ft. was installed following the Hydrosieve units and in series with the existing trickling filters to bring the plant capacity up to 2.3 mgd. The abandoned primary clarifiers were converted into sludge thickeners and the secondary clarifiers and chlorine contact chambers were enlarged to handle the increased flow. The anaerobic digester was upgraded to a high rate unit by the addition of a gas recirculation system for more efficient mixing. The upgraded facility now provides 85% removal of suspended solids and BOD at twice the original hydraulic design capacity.

(3) Upgrading a Contact Stabilization Package Plant to a "Modified" Completely Mixed Flow Pattern

The original plant was a 0.87 mgd contact stabilization plant designed for 2.6 and 6.5 hour detention times respectively, in the contact and stabilization zones. Investigation indicated the plant was performing poorly because of excess detention time in the contact zone. This increased detention time in the contact zone allowed partial stabilization of the adsorbed organics which resulted in a poorly settling mixed liquor. Before upgrading, the effluent BOD and suspended solids were 26 and 24 mg/l respectively.

To improve the plants performance, the influent piping was modified so that the raw wastewater was evenly distributed into what originally was the stabilization zone. No wastewater was introduced into the former contact zone. Mixing liquor in the upgraded system proceeded from the former stabilization zone through the former contact zone to the secondary clarifier.

The return sludge was introduced into the former stabilization zone at one point resulting in a "modified" completely-mixed flow pattern with an overall detention time of 9.1 hours at an average flow of 867,000 gpd. The upgrading procedure lowered the effluent BOD and suspended solids concentrations to 13 and 6 mg/l respectively for a total BOD reduction of 90% and suspended solids reduction of 96%.

The above examples illustrate only a few of the many process modifications that can be employed to maintain or moderately increase process efficiency under increased loadings. Recently, the use of oxygen aeration systems have gained wide recognition as another useful and efficient means of upgrading existing overloaded treatment plants. Because of the increased oxygenation capacity of these systems, a high level of biologically active solids can be maintained in the aeration tanks. This allows the biological capacity of a hydraulically or organically over-loaded plant to be doubled or tripled in size. Total plant capacity is increased by appropriately enlarging the final clarifiers as required by plant loading and sludge management considerations. A bonus advantage of the oxygen aeration system is that under proper loading conditions, significantly lower amounts of waste sludge is produced.

The largest application of oxygen aeration for upgrading the performance of an existing wastewater treatment plant is the 320 mgd Newtown Creek plant in New York City. One of 16-20 mgd aeration bays of this "modified air" plant has been converted to oxygen aeration to improve the BOD and suspended solids removal from 65 to 90%. This oxygen aeration system has been on stream at this location for one month and preliminary data indicate that target removal efficiencies can be met. The aeration basin contact time at average design flow of 20 mgd is 1.5 hours including 25% sludge recycle.

Another popular and successful treatment plant modification is the conversion of single stage biological systems to two stage systems. This is necessary in most instances to insure consistent nitrification which is required by many states now, either directly as a nitrogen control standard, or indirectly because of more stringent disinfection or very low total oxygen demand requirements.

Upgrading to two stage treatment systems has been successful using (1) two stage trickling filters, (2) trickling filters preceded by activated sludge, (3) activated sludge preceded by trickling filters, or by (4) two stage activated sludge plants. In each case, 60-70% of the BOD is removed during the first stage treatment. This lowers the organic loading sufficiently so that nitrification can proceed in the subsequent stage.

EFFLUENT POLISHING

The use of effluent polishing techniques is recognized as one of the most cost effective methods of upgrading existing plants to obtain increased organic and suspended solids removal. Overall plant performance can be improved from the 70-85% efficiency range to the 95-99% range depending on the process employed. Four unit processes are considered here for effluent polishing. They are: 1) filtration,(2) microscreening, (3) granular carbon adsorption, and (4) reverse osmosis.

Filtration

Simple filtration of secondary effluent provides a positive method of suspended solids control, and as such, is the most widely used and the most efficient single unit process for upgrading treatment plant performance today.

Contemporary filtration systems can be broadly classified as either deep-bed or surface filters. The most popular trend recently in deep-bed filter design is the use of the dual or tri-media filter. Here, the use of two or more layers of different media having increasing specific gravity with increasing bed depth allows gradation of the filter bed from coarse to fine in the direction of flow. This allows more efficient utilization of the total bed depth for solids storage than conventionally graded, single medium filters.

The approach taken in the development of surface filters on the other hand, is to allow filtration to take place on or near the top of relatively shallow single medium filters, and to optimize removal of the accumulated solids. In addition to the standard pressure and gravity surface filters, several innovative techniques for providing a continuous clean filter surface have been developed. The moving bed filter developed by Johns-Manville Products, the radial flow filter developed by the Dravo Company, the radial flow-external wash filter developed by the Hydromation Corporation, and the Hardinge traveling bed filters are typical examples.

The performance of each of the above filters in polishing secondary effluent will depend on such factors as surface loading rate, temperature, floc size and strength, degree of biological or chemical flocculation, media depth, grain size, solids loading, run length, and method of filter operation. Because of the numerous variables involved, and the ease of obtaining reliable design data from small pilot filters, it is recommended that final filter selection be based on pilot plant results where possible. Operating results from typical filter installations are shown in Table 5.

TABLE 5
Filtration Performance

Filter Type	Feed Type	Media Size mm	Filter Depth ft	Hydraulic Loading gpm/sq.ft.	SS Removal %	BOD Removal %	Effluent SS mg/l	Effluent BOD mg/l	Reference
Deep-Bed									
Gravity Downflow	T.F. Eff.	1.0-2.0	-	6	70	55	5-7	-	(18)
Gravity Downflow	T.F. Eff.	0.9-1.7	2-3	3	67	58	-	2.5	(19)
Pressure Upflow	T F Eff.	0.9-1.7	5	3	85	74	5 0	2.5	(19)
Pressure Upflow	A S Eff.	0.9-1.7	5	3	77	-	-	-	(19)
Pressure Upflow	A.S. Eff	1.0-2.0	5	2 2	50	62	7.0	6.4	(20)
Pressure Upflow	A.S Eff.	1.0-2.0	5	4.0	67	73	4 9	6.4	(20)
Pressure Upflow	A S Eff	1.0-2.0	5	4.9	56	65	5 7	7.1	(20)
Mixed Media	E A Eff.	0.25-2 0	2.5	5 0	74	88	4 6	2 5	(21)
Mixed Media	A S Eff			2.0	73	74	3.8	6.0	(20)
Mixed Media	A.S Eff			4.0	73	85	4.3	3.9	(20)
Mixed Media	A S. Eff	0.25-2 0	2 5	2 5					(22)
Surface Filters									
Moving Bed	T F Eff	(1)0 6-0 8	4.2	2	47	71	-	-	(23)
Moving Bed	T F Eff.	(2)0.6-0.8	4 2	2	67	80	-	-	(23)
Gravity Downflow	A S Eff	-	-	2 2	55	64	7 2	7.3	(20)
Gravity Downflow	A S Eff	-	-	4 0	69	70	4.8	7 4	(20)
Gravity Downflow	A S Eff	-	-	8.0	48	64	6 1	6 7	(20)
Gravity Downflow	A S Eff	0.9-1 7	2.0	1 6-4 0	72-91	52-70	-	-	(19)
Gravity Downflow	A S Eff	0.95	1 0	2.0	46	57	-	-	(24)
Gravity Downflow	A S. Eff	0.58	-	2 0-6 0	70	80	-	-	(25)
Gravity Downflow	C S Eff	0 45	1 0	5 3	62	78	5	4	(26)

T F - Trickling Filter E A - Extended Aeration
A.S. - Activated Sludge C.S - Contact Stabilization
(1) 100 mg/l alum & 0 2-0.75 mg/l anionic polymer added
(2) 200 mg/l alum & 0 2-0.75 mg/l anionic polymer added

As the table indicates, all of the filters mentioned produce an effluent with suspended solids and BOD_5 generally less than 7 mg/l. As a rule, the deep-bed filters are better suited to treating strong biological floc, will yield longer run lengths, and are less sensitive to solids loading than are the surface filters. The surface filters are better adapted to removing the more fragile chemical flocs, yield shorter run lengths, and require less backwash water than the deep-bed filters. A key element in efficient effluent polishing is to match correctly the type of filter used with the flocculant nature of the solids to be removed, and also to design the filter for solids-loading and run lengths that are compatible with normal operating schedules.

Microscreening

The microstrainer is another surface filtration device that has found increasing utility for polishing secondary effluents. The system consists of a specially woven stainless steel fabric mounted on the periphery of a partially submerged horizontal revolving drum. Influent enters through the upstream end of the drum and flows radially outwards through the fabric leaving the intercepted suspended solids behind. Microstrainers are available in sizes ranging from 5'0" diameter x 1'0" wide having a 0.06 to 0.6 mgd capacity to 10'0" diameter x 10'0" wide with a capacity of 4.0 to 12.0 mgd. Filtration efficiency depends primarily on fabric size and the character of the solids being removed. For wastewater polishing applications, these microstrainers are available with automatic controls to increase drum speed and backwash pressure to accommodate variations in flow, and to a lesser extent, variations in solids loading. One of the chief advantages of using a microstrainer for polishing secondary effluent is its low head requirement of 1 to 1-1/2 feet. Pumping secondary effluent prior to microstraining tends to shear the biological floc and decrease solids removal efficiency.

Microstrainers are washed continuously at 20-50 PSIG and require 4-6% of the filter throughput. Continuous backwash is advantageous for upgrading small plants since it eliminates sur-charging of the upstream units which must be considered when using conventional intermittently backwashed filters. Operational data from various microstrainer installations are presented in Table 6 (10).

TABLE 6
Microstrainer Performance

Location	Plant Size mgd	Feed Type	Fabric Opening microns	SS Removal percent	Effluent SS mg/l	BOD Removal percent	Effluent BOD mg/l	Backwash % of flow
Brampton, Ontario	0.1	A S.[1] Effluent	23	57	-	54	-	-
Lebanon, Ohio	Pilot	A S. Effluent	23	89	1.9	81	-	5.3
	Pilot	A S Effluent	35	73	7.3	61	-	5 0
Chicago, Illinois	3.0	A S Effluent	23	71	3.0	74	3 0	3.0
Lutton, England	3.6	Effluent from A.S. and T F[2]	35	55	7 3	30	-	3 0
Bracknell, England	7 2	T F Effluent	35	66	5 7	32	8.4	-

[1] A S. - Activated Sludge

[2] T F - Trickling Filter

Carbon Adsorption

The necessity to upgrade secondary effluent quality beyond the levels that can be obtained by implementation of the process modifications previously discussed, or by the application of tertiary filtration, will require a substantial investment in additional plant equipment as well as a 30-50% increase in operating cost. It is doubtful that this level of expenditure can be justified except as part of a major plant expansion where the expected lifetime of the new facility is sufficiently long to permit reasonable amortization of the high capital investment required. The situation is especially difficult for small plants in the 0.5 mgd range that serve rapidly growing urban areas.

The effectiveness of granular activated carbon for upgrading the treatment efficiency of larger plants is well established. Operating experiences at Pomona and South Lake Tahoe, California; Nassau County, New York and Colorado Springs, Colorado have left little doubt regarding process efficiency, operating cost and reliability of these systems. Operating results and principle design parameters for three of these locations are shown in Table 7 (10).

TABLE 7
Tertiary Granular Carbon Adsorption
Design Parameters and Operating Results

	Pomona		Lake Tahoe		Nassau County	
Operating Data						
Capacity	200 gpm		1,800 gpm		400 gpm	
Source of Waste	Domestic		Domestic		Domestic	
Secondary Treatment	Standard Activated Sludge		Standard Activated Sludge		High-rate Activated Sludge	
Pre-treatment	Chlorination		Coagulation & Filtration		Coagulation & Filtration	
Carbon Type	16 x 40 mesh		8 x 30 mesh		8 x 30 mesh	
Column Configuration	4-Stage Downflow		2-Upflow in Parallel		4-Stage Downflow	
Column Dimensions	6' dia. x 9' deep		12' dia. x 14' deep		8' dia x 6' deep	
Nominal Contact Time	36 minutes		13 minutes		24 minutes	
Loading Rate	7 gpm/sq.ft		8 gpm/sq.ft.		7.5 gpm/sq.ft	
Carbon Column Performance	Influent	Effluent	Influent	Effluent	Influent	Effluent
COD, mg/l	47	10	20-30	2-10	-	5
BOD, mg/l	-	-	5-20	2-5	-	-
Color, Pt-Co Units	30	3	20-50	5	-	-
Carbon Dosage	350 lbs/million gallon		250 lbs/million gallon		500 lbs/million gallon	

The capital cost of tertiary granular carbon systems will vary widely depending on the particular system design and pretreatment provided. Direct application of secondary effluent to downflow carbon adsorption columns as practiced at Pomona, California will result in a smaller capital investment than the tertiary system used at Lake Tahoe, but may increase operating costs due to more frequent column backwashing.

Operating cost will depend primarily on the organic loading and associated carbon dosages. Table 8 compares the actual capital and operating costs for the 7.5 mgd conventional activated sludge plant at South Lake Tahoe with the corresponding tertiary carbon adsorption costs at that location (27). Other investigators have estimated capital and operating costs for tertiary carbon adsorption systems that are considerably higher than those shown. The above data was used to estimate operating costs for carbon dosages of 350 #/mg and 500 #/mg. These higher dosages would be anticipated from most activated sludge effluents applied directly to adsorption columns.

The following tabulation of costs illustrates that at the 7.5 mgd scale the use of granular carbon adsorption with regeneration can increase total plant operating cost by 60% and require a capital expenditure as great as 30% of the original plant investment. Culp and Culp (27) estimate that the total costs for tertiary carbon adsorption for a 2.5 mgd plant would be 50% greater than for the 7.5 mgd Tahoe system. Little operating information is available on carbon adsorption operating costs below this level.

TABLE 8
Tertiary Granular Carbon Adsorption Costs

	Operating Cost $/mg	Capital Cost $/mg	Total Cost $/mg
Primary and Activated Sludge + Organic Sludge Handling and Chlorination 7.5 mgd	103	67.50	170 50
Granular Carbon Adsorption and Regeneration 7 5 mgd			
250#/mg	30	21.5	51 5
350#/mg	42*	21 5	63 5
500#/mg	60*	21 5	81 5

* estimated based on South Lake Tahoe Data

Recently some investigators have advocated adding powdered activated carbon directly into the aerator of an activated sludge plant to upgrade the organic removal efficiency. The DuPont Company has developed a PACT (powdered activated carbon treatment) process for this purpose. Preliminary data obtained in a parallel 0.45 mgd study showed that the application of 308 mg/l of powdered activated carbon into a completely mixed aerator of a conventional activated sludge plant reduced the soluble effluent BOD from 20 to 11 mg/l, for an overall BOD removal of 96% (28). In another test, the flow to one side of a parallel activated sludge plant receiving 295 mg/l of powdered carbon was more than doubled without sacrificing soluble BOD removal efficiency. Total BOD removal decreased from 98 to 96% with suspended solids increasing from 58 to 89 mg/l (28). These experiences indicate that powdered activated carbon addition to activated sludge plants will produce excellent soluble BOD

removal, but these plants must be followed by filtration, to produce high suspended solids removals.

The operating cost for the addition of 300 mg/l of powdered activated carbon on a once through basis would be about $220/mg which is prohibitively high for continuous use. Regeneration of this carbon could bring the cost down to about $70/mg which is competitive with granular carbon adsorption systems. Although a great deal of work has been completed on powdered carbon regeneration schemes (29,30,31, 32), they have not been successfully used for this application. In any case, it is doubtful that powdered carbon adsorption should be used to compete with biological oxidation in the aerator of an activated sludge plant except for the removal of "hard" or refractory BOD substances. The most practical application would be to use the powdered carbon as an operational tool to improve treatment efficiency during times of "biological upset" due to toxic materials, or during short periods of extremely heavy organic loads.

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