SMALL ACTIVATED SLUDGE TREATMENT PLANT DESIGN

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At the present time there are in the state of Massachusetts, 19 activated sludge treatment plants having a design capacity ranging from 5,000 to 100,000 gallons per day. These plants have no primary settling or primary sludge handling facilities. The raw sewage enters directly into the aeration tank and thence flows to a settling tank. The sludge from the settling tank is returned rapidly back to the aeration tank. In most cases chlorination is also provided. This type of treatment plant is called many names by the different manufacturers, some of which are Oxigest, complete mix, wet burning, Rated aeration, Aerobic digestion and Sparjair. In all cases, however, the unit is essentially an activated sludge treatment plant without primary treatment.

This paper will deal with the design of the treatment units, the influent works and the receiving stream.

What is Expected of the Treatment Units

Normally, the effluent is discharged to a small brook or stream which has a down-stream water use either presently or in the future of Class C which requires a minimum dissolved oxygen of 5.0 mg/1. Most of the small streams or brooks will reaerate rapidly but do not have the essential biota for pollution assimilation. Normally, the treatment unit is expected to produce better than 85% treatment. Most raw wastes being treated at the small plants have a biochemical oxygen demand (BOD) greater than 200 mg/1. Fifteen percent of a BOD of 200 mg/1 is 30 mg/1.

Let's assume an effluent BOD of 30 mg/1. What is the minimum stream flow capable of receiving this effluent? Using a general stream loading equation the maximum allowable BOD of the stream below the point of discharge varies between 3.3 and 15.8 mg/1, for a minimum dissolved oxygen of 5 and 2 mg/1 respectively. Since the receiving

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waters are in a Class C condition, we would use the minimum dissolved oxygen of 5 mg/1 and a maximum allowable BOD after dilution of 3.3 mg/1. Under these conditions, the stream should provide a dilution of 11.6 to 1. This dilution should be available during the time when the effluent is coming out of the plant. For a flow occurring during 16 hours, the dilution factor should be raised to 17.4 to 1. For example, a plant receiving 10,000 gallons per day in 16 hours would need a stream flow of 174,000 gallons per day. This would require a drainage area of at least 3 square miles. For a BOD of 20 mg/1 in the effluent, the stream must provide 109,000 gpd or about 2 square miles of drainage area. This all adds up to finding a good stream and not trying to put the effluent into a small brook which flows only during wet weather. It points out the need for a good field survey of the source or watershed of the stream and also the down-stream water use.

The BOD in the effluent is what we are primarily interested in. After determining this, we would then work out the percent of treatment required to attain the effluent concentration. The treatment unit then must reduce the BOD to at least 30 mg/1 and better yet 20 mg/1. The effluent must be very low in settleable solids and also visible solids.

The Influent Works

It is desirable that the influent sewers be laid at a slope which will be self cleansing. It cannot be assumed that high rates of flow will occur occasionally to remove deposits. For 10,000 gpd occurring in 16 hours, the rate of flow is 15,000 gpd. In order to provide self cleansing velocity at these low rates of flow, it would be necessary to lay a six-inch sewer at a slope of 10.5 feet per 1000 feet. In many areas, this slope is not available and it may be necessary to periodically flush the lines in order to prevent the accumulation of deposits. The necessity for flushing will be determined in large part by the amount of heavy foreign matter which enters the system. At some locations, only light organic material is carried and flushing may not be necessary while at other locations, industries, for example, there may be a small amount of heavier materials which find their way into the collection system.

Comminutor and Pumps

A pump is required only where gravity head is not sufficient. The pumping rate in large part controls the addition of organic material to the aeration tank and the overflow rate on the settling tank. It should operate as often as is feasible and at as low a rate as is practical. A 15,000 gpd rate of flow is 10.2 gpm and a 300% variation of this is 30 gpm.

The smallest pump which is in the normal production for raw sewage is a 50 gpm unit. As pointed out above, this pumping rate is greater than necessary for small collection systems. One way of using the 50 gpm pump and not overloading the treatment unit is to pump into a flow splitterbox. In this box, there is an adjustable fin which diverts part of the flow and returns it back to the wet well. By trial and error, this fin can be adjusted so that the design flow is directed through the treatment unit. The splitterbox has worked very effectively at some plants.

A comminutor is needed upstream from the wet well or at the inlet to the aeration tank, whenever it is felt that a shredder is necessary. It is questionable whether this expensive ($1800) piece of equipment is necessary at all plants. There are several plants operating well without one. At these latter, it is necessary to prevent the use of high wet strength brown paper towels etc. If a shredding device or a coarse screen is used, there should be an ample approach velocity, in order to provide sufficient energy to push the soft organic matter through the fine openings. Provision should be made for a free drop into the aeration tank while being aerated.

Design of the Treatment Unit

Aeration tank—the determination of the size of the aeration tank is one of the most controversial subjects in sanitary engineering today. The following is a list of some of the criteria presently in use: lbs of BOD/1000 cubic feet, lbs of BOD/lb of mixed liquor solids, time of aeration, sludge age, Eckenfelder's equations and lbs of BOD per 1000 lbs of solids per hour of aeration. The time of aeration, while an important individual factor, does not take into account the amount of organic material being added to the biological unit, thus it can be ruled out as the complete criterion. Sludge age is a measure of some of the many factors and seems to be useful at some plants but not at others. In Eckenfelder's formulation, it is necessary to know or assume a solids loading factor, the concentration of mixed liquor solids and the concentration of the return sludge. It is normally advisable to determine these by the operation of a pilot plant. If the mixed liquor solids concentration is known or has been assumed, the lbs of BOD/1000 cubic feet can be expressed as lbs of BOD/lb mixed liquor solids. In using the last two criteria and the lbs of BOD per 1000 lbs of solids per hour of aeration, it is necessary to assume a mixed liquor concentration.

Let's backtrack a little and see what the criterion should include. Since this is a biological treatment unit, it should include lbs of BOD or amount of organic matter being added per day or per unit of time. It has been stated that [1] "(a) area of contact surface or film and (b) opportunity for contact" are the controlling factors in biological sewage treatment. Area of contact surface or film can be indirectly expressed as lbs of BOD per 1000 lbs of mixed liquor solids and opportunity for contact can be expressed as hours of aeration.

The criterion lbs of BOD per 1000 lbs of suspended solids per hour of aeration has been correlated and formulated relative to efficiency. The curve for this formulation is shown on page 723 of "Water Supply and Waste Water Disposal" by Fair and Geyer. The efficiency equation is

$$P_2 = \frac{100}{1 + 0.03 \dfrac{(Y_0)}{(Wt)} 0.42}$$

By changing the units of expression, the $\frac{Y_0}{Wt}$ can be equated as follows:

$$Y_0 = \frac{2400\ (U_0)}{(\text{mg/1 of ss})\ t^2}$$

therefore

$$t^2 = \frac{2400\ (U_0)}{(\text{mg/1 of ss})\ (X)}$$

U_0 is BOD_{-5} raw in mg/1
X is value on X-axis—from curve
t is time of aeration in hours

Solving for t we can substitute it into

$$T = \frac{V}{Q}$$

$$V = \frac{Qt}{24}$$

[1] Fair & Geyer, Water Supply and Waste Water Disposal.

Where —Q is the raw waste flow
—T is detention time in days
t is detention time in hours
V is the volume of aeration tank

The curve is based on a voluminous number of sample results at plants throughout North America.

Let's compare this last criterion with lbs of BOD/lb of solids. (This may be expressed as per lbs of solids, per 100 or per 1,000 lbs of solids.) Take two wastes each requiring 90% treatment efficiency. The comparison of treatment plants using the two criteria is as follows:

TABLE I

				Based on Curve			Based on lbs BOD/lb of s.s.		
Waste	BOD raw mg/1	Flow mgd. Q	# BOD day	Effl. mg/1 90%	V for 90% mgd.	t hrs.	% removal Both Tanks 0.87	Effl. mg/1	t
A	400	1	3330	40	0.61	14.7	92.5	30	20.8
B	200	2	3330	20	0.87	10.4	90.0	20	10.4

Based on the curve, tank A. is 0.61 million gallons and tank B is 0.87 million gallons. Using lbs per lb of suspended solids both tanks would be the same size since there is the same weight of BOD and we assumed the same suspended solids concentration. If tank A were made 0.87 million gallons, the efficiency would increase to 92.5% but only 90% is needed. This would mean that tank A is 260,000 gallons over-designed or 0.26/0.61 = 42.5% too large. This is a considerable waste.

The efficiency formula has been compared with the results of a one week composite sampling at the Bedford nike site. The actual overall efficiency during the one week of around the clock sampling was 96.2%. The formula worked out to 97.7%.

At the present time, there is no efficiency formula which is extremely accurate for all wastes at all types of activated sludge plants. In the writer's opinion, the Fair-Thomas formula has the proper ingredients and with more data in the high percentage range is the type which should be used for the design of the small high efficiency units.

Suppose we assume that these criteria are fine guides but that the aeration tank can be made larger and that efficient operation will be more assured. This assumption has been shown to be in error. The oversized unit produces high nitrates and nitrites. In doing this, it breaks down nitrogenous material and particularly ammonia. The combination of these chemicals plus the low alkalinity in the sewage in this part of the country causes the pH to lower considerably, sometimes to 4.5. At the lower pH values, the desirable bacteria do not function well and a different biota is established. The solids do not settle well and the efficiency falls off appreciably. This condition may also occur to some extent in a properly designed highly efficient unit. The intermittent or continuous addition of ground limestone, calcium carbonate, has remedied the condition.

The Aeration Equipment

This equipment performs two functions, namely, oxygenation and mixing. The mixing should be sufficient to provide intimate contact between the biological floc, the organic matter and the oxygen. The mixing should also assure that there be no appreciable accumulation of solids on the bottom of the tank. The usual rule of thumb velocity to prevent accumulation is 1.0 to 1.5 fps. At most of the small plants this velocity is greatly exceeded. At one plant the excessive mixing, turbulence, which gave rise to voluminous foaming was reduced with a resultant decrease in foaming and a considerable increase in settleability. The control of mixing and turbulence is a matter which should be borne in the designer's mind and one which will likely be the subject of research in the future.

In these small high efficiency plants the amount of oxygen to be supplied is approximately equal to the ultimate BOD. The oxidizable COD is similar to the ultimate BOD and since oxidizable COD takes less time in the laboratory, it is frequently used as the basis for air supply. If the BOD_{-5} to oxidizable COD ratio is determined, the air supply can then be computed relative to the BOD_{-5}. For example: with normal domestic sewage, a 5% transfer efficiency of the aeration equipment, 15 pounds of oxygen per 1,000 cubic feet of air, and a BOD_{-5} to oxidizable COD ratio of 0.66 the amount of air is 1.4 cfm per pound of BOD_{-5} per day.

The amount of oxygen in solution and available for the biochemical reactions is dependent on many factors; some of which are—transfer efficiency of the diffusor device, dissolved oxygen in the tank, geometry of the tank and surface active chemicals in the solution. The design of the diffusor equipment can be based on the cfm per pound of BOD_{-5} figure. The air blower chosen should supply this output at about 2/3 of maximum speed. If the blower unit is equipped with a standard adjustable motor base and spring loaded adjustable sheave, the speed of the blower can be controlled by means of a hand wheel and screw over a range of 3 to 1. With this inexpensive variable control, the errors due to unmeasured variables can be compensated.

With mechanical aerating devices, the manufacturer's rated capacity is used. The manufacturer also recommends the dimensions of the aeration tank for a particular size of unit. Variability is accomplished by intermittent operation which is controlled by a time clock. It is to be noted that when the unit is off, mixing as well as oxygenation is stopped. The operational results of plants employing this type of control show a good effluent so continuous mixing is evidently not required at these plants. This may also indicate that less mixing is required in the aeration tank of any plant.

Settling Tank

The function of the settling tank is to separate the active solids from the liquid. The overall efficiency of the treatment plant is determined in large part by the settleability of the solids as they enter the tank and on the ability of the tank to remove the solids from the liquid. In the smaller treatment units the possibility of shortcircuiting through the tank is much higher than in larger settling tanks. Experience has shown that standard proven settling tank designs should be used. Novel or over-simplified settling tanks have proven very unsatisfactory. Because the tank is very small, the inlet zone and outlet zone are sometimes so close that there is little or no settling zone. Standard rectangular or circular tanks are recommended and any variation from normal design practices must be critically reviewed.

The surface area load on the settling tank should be less than 800 gpd per square foot. With the proper overflow rate, inlet arrangement and tank geometry, the detention time usually works out to 4 hours. It is normal practice to include the volume of the upper 1/3 of the sludge hopper when determining the detention time. The slope of the hopper of a nonmechanically cleaned settling tank should be at least 1.5 to 1. It has been found that the sludge tends to stick to the sides of the settling tank and that some method of periodically cleaning the sides is necessary. One company has provided a chain on the end of an off-set shaft which when manually rotated will drag along the surface of the hopper. This device is similar to what is used in cleaning the sloped sides of the settling chamber of an Imhoff Tank. The settling tank should also be equipped with a device for skimming. Since these treatment plants do not have primary settling, some of the solids which adhere to the mixed liquor are lighter than water and will cause the mixed liquor particles to float rather than to settle in the settling tank. The skimming can be accomplished with several devices; some of which are an air lift pump attached to a skimming funnel or pipe, scum trough which discharges back to the pump wet well or in the case of the down draft mechanical unit, a funnel and pipe which connects into the center of the aeration unit at a point above the level of the vortex. The tank should also be equipped with a scum baffle, standard adjustable v-notched weir plate and inlet baffle.

Some features which have not proven successful are two sludge hoppers in series, effluent weir trough which is adjustable only on one end. Vertical settling tanks integrally connected with the aeration tank and with or without positive sludge return have not proven at all satisfactory. As a matter of fact, vertical settling tanks should be designed on the basis of 500 gpd per square foot. It is difficult to return the sludge from a normal settling tank by the use of a sludge return pump and gravity sludge return is extremely difficult to accomplish. In the case of the gravity return settling tanks, the sludge accumulates until it becomes septic and then floats to the surface because of the release of nitrogen gas. The sludge cannot be effectively skimmed by shovel or scoop from the surface since it is so fragile, and the operators tend to let it accumulate until it becomes septic and very odorous.

Sludge Return Pumps

At all of the existing plants sludge return is accomplished by use of an air lift pump. An air lift pump consists of a 2 ½ or 3 inch pipe extending down into the sludge hopper with air being defused through a footpiece at a point near the lower end of the intake pipe. The air lift pump has been a source of trouble at almost every plant since it frequently clogs. In addition, the capacity of the pump is normally about 50 gpm. At some plants, particularly those with two sludge hoppers, the sludge return rate is 1,000% of the raw sewage flow. The rate of flow of an air lift pump can only be reduced approximately 15%. Reduction of the air supply beyond this point shuts off the pumping action. Operating the pump intermittently seems to correct the clogging at some plants. Where it is possible to place a pump inside a heated building, the use of a diaphragm pump or small pump having a strong suction would have considerable advantages over the air lift pump. The sludge return pump should be designed and/or chosen so that the rate can be varied and frequency of operation can be controlled by a time clock. The capacity of the pump should be approximately 100% of the raw sewage influent rate. It should be noted here that the sludge return rate determines in part the concentration of mixed liquor solids. The sludge pumping rate also affects the flow pattern in the settling tank.

Miscellaneous

All of the treatment plants produce excess sludge. The build up of excess solids is determined in large part by the efficiency of the settling tank. Under equilibrium conditions of food to metabolism, the rate of sludge build up will be approximately 11% of the amount of BOD removed. Therefore, there will be an increase in build up of solids in the treatment unit. At most plants it is felt that removing excess solids by means of a septic tank truck is the best method of wasting. It is also possible to pump the solids to a septic tank or an anaerobic holding tank. One manufacturer provides an aerobic digester. Since the prevention of odors in the vicinity of the plant is important, the use of sludge drying beds is discouraged.

The control of bacteria in the waste is almost always required. Because the unchlorinated effluent contains excessive bacteria, chlorination is necessary. The chlorinator and contact chamber should be capable of providing a 1 mg/1 (flash test) chlorine residual after 15 minutes contact time. The contact chamber should be designed to reduce short circuiting. In the author's opinion the flow pattern through the usual baffled contact chamber does not reduce short circuiting but may actually induce it and therefore the design should be critically reviewed.

Most of these plants are called package plants. The proper design of a plant, however, involves considerably more than merely looking in a catalogue and picking out a package. Each component part must be tailored to the strength, volume and variations of the particular waste. For this reason the author suggests that the plants be called Small Activated Sludge Plants and that the word package be abandoned.

Conclusions

1. The efficiency of the treatment unit is determined by the pollution assimilative capacity of the receiving stream.
2. The treatment plant should be designed to produce an effluent BOD of less than 30 mg/1 and preferably 20 mg/1. The effluent should be low in suspended and visible solids.
3. Provision should be made in the design of influent works for proper operation at the low flow rates which occur at small plants.
4. There is no present efficiency formula for the design of aeration tanks which is extremely accurate for all plants receiving any or all wastes. The Fair-Thomas formula has the proper ingredients and is accurate over a wide range of application. With more data in the high efficiency range the formula should become more accurate.
5. The air supply equipment should be carefully designed to provide sufficient oxygen in solution, sufficient mixing and be variable to provide for fluctuation in operation and raw waste.
6. The settling tank at the small plants in large part determines the effluent concentrations and must be carefully designed. Standard proven tank designs should be used. The surface area loading should be less than 800 gpd/square foot.
7. The sludge return pumps at existing plants have been a source of trouble. The pump should provide variable capacity, at least 100% of raw waste flow and intermittent operation. The use of presently designed air lift pumps should be reviewed.
8. Chlorination is required at almost all plants.