Folklore In Design of Final Settling Tanks

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INTRODUCTION

Conventional waste treatment practices developed as an art out of necessity. The need for pollution abatement techniques predated knowledge of basic concepts controlling performance of the unit operations and processes which could be used for wastewater treatment. Under this conditions, pioneers in the field of waste treatment did the best they could. They developed sophisticated waste treatment techniques by trial and error and by continually introducing appropriate refinements.

To permit designs and operational procedures to be predicated in the successes of the past, early workers in wastewater management tried to identify parameters which might reasonably characterize process performance. Parameters were selected which intuitively or empirically could be expected to influence the behavior of treatment processes. An additional requirement was that the parameters should be readily measurable. By making the magnitude of these parameters the same as used in successfully performing treatment processes, it was intended that reasonable process design could be accomplished in the absence of basic understanding of factors controlling process performance.

Examples of such parameters developed on an empirical or intuitive basis are widespread in the wastewater treatment field. For example, the organic loading intensity per unit volume of an aeration tank in the activated sludge process is a fundamentally unsound, but historically useful parameter. Similarly, the hydraulic loading per unit area of trickling filters and the volatile suspended solids loading per unit volume of anaerobic digester capacity are related only circuitously to process performance. Many other examples of such parameters are included in common waste treatment plant design criteria.

These empirical parameters have served for many decades as bases for design and as guidelines for operational control of waste treatment facilities. They stand as tributes to pioneers who got a job done. However, because the parameters don't reflect fundamental relationships between process loading and process performance, their use can lead to a lack of cost effectiveness in design and to a lack of precise operational control.

Much of the research in water pollution control in past decades has been directed toward acquiring fundamental understanding of processes developed previously out of need. Such improved understanding has enabled new designs to be more cost effective, has allowed increases in the capacity of existing treatment facilities at nominal cost, and has permitted improved control of the performance of waste treatment facilities.

At this point, when basic understanding of process performance becomes available, the "rules-of-thumb" developed over the years as valuable aids to design and operation become sort of "technical old wives tales." A significant body of such "tales" or "folklore" has accumulated in the annals of water pollution control technology.

Regrettably, there is a tendency to become comfortable with the familiar parameters of the past. There is a reluctance on the part of professors, design engineers, equipment manufacturers, regulatory agencies, and operating personnel to abandon comfortable and well-tried parameters for less familiar, less well-tried (but potentially more useful) rational approaches to analysis of performance of waste treatment facilities.

The purpose of this paper is to examine the folklore which has accumulated concerning design and operation of the final settling tank in the activated sludge process. The intent is not to criticize empirical parameters which have been developed and proven useful in the past, but to evaluate such parameters in light of current knowledge of factors influencing performance of final sedimentation tanks. Such a critical assessment of the parameters seems necessary in order to evaluate the merit of their continued use.

ROLE OF FINAL SETTLING TANKS

It is appropriate to select the final settling tank of the activated sludge process for a discussion of folklore, for an appreciable amount of folklore has developed concerning design and operation of that facility. Additionally, it is a very important part of a treatment plant — proper performance of the final settling tank in the activated sludge process is crucial to overall effective performance of the activated sludge process.

The clarification performance of the final settling tank in the activated sludge process has a direct effect on effluent quality. The dissolved BOD in effluent from properly designed and operated activated sludge plants often is less than 5 mg/l. It is the suspended solids which escape separation in the final settling tank which contribute most of the effluent BOD.

Another important effect of the performance of the final settling tank on overall treatment plant efficiency concerns the tank's influence on the biological phase of the process. With a given aeration tank volume, the capacity of an activated sludge process is determined by the mixed liquor suspended solids concentration of the sludge recycled from the final sedimentation tank. Thus, the thickening performance of the final settling tank also is crucial to proper overall treatment process performance.

Another effect of the performance of final settling tanks is on the cost of treatment and disposal of excess sludge produced in the activated sludge process. Inasmuch as the yield of dewatering equipment, the cost of transportation and the effectiveness of other processes used in sludge treatment and disposal depend on the concentration of waste sludge, the thickening which occurs in the final sedimentation tank also influences sludge handling costs.

Finally, a basic understanding of the performance of the final settling tank in the activated sludge process is needed to permit optimization in treatment plant design and precise control of activated sludge treatment facilities. This is because of the intimate interaction between the biological and solids separation phases of the activated sludge process. This interaction is illustrated by Figure 1 which shows the effect of the mixed liquor suspended solids concentration in an aeration tank on the overall capital cost of activated sludge treatment of a particular industrial waste. It is seen that if a low mixed liquor suspended solids concentration is maintained, capital costs are high because of the excessive cost of the large biological reactor required. If an extremely high mixed liquor suspended solids concentration is maintained, costs likewise are high, but this time because of the high cost of the solids separation facility required to maintain the high mixed liquor concentration. It is seen that, in this case, the optimal mixed liquor volatile suspended solids concentration was about 4,000 mg/l. It is emphasized that this optimization is valid only for the particular waste for which the data for Figure 1 were gathered. The important aspect of Figure 1 is that the capital costs of activated sludge treatment can vary several fold depending on the relationship established by the designer between the biological oxidation and solids separation phases.

The simulation models required to effect optimal design such as illustrated in Figure 1 must reliably predict performance. The potential cost savings and improved operational performance offered by application of systems concepts to wastewater treatment plant design cannot be realized if the mathematical models of process performance are founded on folklore which does not adequately describe process performance.

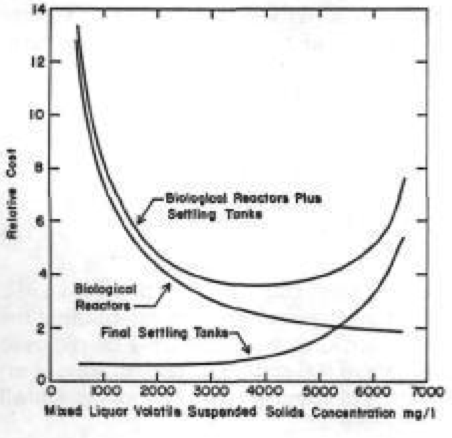

Figure 1 — Use of mathematical models of final settling tank and biological reactor to optimize process design.

FOLKLORE

The original paper on treatment of wastes by the activated sludge process by Arden and Lockett (1) gave no clues as to how solids separation facilities for the process should be designed. They allowed only that "despite its low specific gravity, (activated sludge) separates from water or sewage at a rapid rate."

It is clear that the workers in water quality control engineering approached design of the solids separation facility for the new activated sludge process in somewhat the same manner as they viewed design of other clarification facilities in use at the time. In the 1925 edition of Babbitt's "Sewerage and Sewage Treatment," (2) design criteria which had been used for the activated sludge treatment plants constructed by that time were reviewed. Babbitt indicated that retention times of 30 to 90 minutes had been used and that surface settling rates of 1,600 gpd/sq ft (65 cu m/day/sq m) based on maximum daily flow were common.

Over the years, these two parameters, retention time, and surface loading have been widely adopted. To illustrate, the Ten State Standards (3) (and, hence, the design standards of many states) currently contain criteria for final sedimentation tank design based on overflow rate and detention time. Many other "rules-of-thumb" also have been developed to serve as guidelines in operation or design of the final settling tanks. The nature of these contributions to final settling tank folklore is examined briefly in the sections which follow.

"Surface Settling Rate Should Not Exceed 800 gpd/sq ft"

A limitation on the hydraulic loading per unit of surface area commonly has served as the sole basis for determining the area of final sedimentation tanks. The Ten State Standards (3) prescribe surface settling rates of from 300 to 800 gpd/sq ft (12 to 33 cu m/day/sq m) depending on the type and size of the activated sludge process. No other basis for determining the necessary area of the tank is continued in these or other typical design standards.

The fundamental relationship between the surface settling rate of a final settling tank and the removal of settleable solids is well documented by the work of Hazen (4), Camp (5), and others. The theoretical hydraulic retention time, t_c, in the clarification portion of a final settling tank with effluent flow rate, Q_c, is

$$t_c = \frac{h_c A}{Q_c} \tag{1}$$

where A and h_c are the area and depth of the clarification portion of the tank respectively. If the direction of flow in the clarification portion of a final settling tank is considered to be primarily horizontal, then, during the retention time, t_c, a particle must settle a maximum distance of h_c to be removed. If the settling velocity of the slowest settling particle completely removed is v_c, then

$$v_c t_c = h_c \tag{2}$$

Combining Equations 1 and 2 gives

$$v_c = \frac{Q_c}{A} \tag{3}$$

The right side of Equation 3 is the surface settling rate. Thus, it is seen that design of a final settling tank on the basis of a surface settling rate is equivalent to establishing the settling velocity of the slowest settling particle which theoretically should be removed completely by the final settling tank. An overflow rate of 800 gpd/sq ft (12 cu m/day/sq m) for example, is equivalent to a particle settling velocity of about 4.5 ft/hr (1.4 m/hr), and all particles which settle this fast or faster should be removed.

If flow in the clarification portion of a final settling tank is considered to be primarily vertical, the same conclusion as stated by Equation 3 obviously obtains. That is, a particle will be removed if it settles as fast or faster than the vertical fluid velocity. (In the case of vertical flow, however, partial removal of particles which settle slower than v_c does not occur as it does in horizontal flow clarifiers (5).)

It seems, then, that adoption of the surface settling rate as a parameter for final settling tank design was a good choice *for the clarification function.* However, use of this parameter as a means of designing a final sedimentation tank does not assure that the tank will satisfactorily accomplish its thickening function. That is, the overflow rate on a final sedimentation tank is not directly related to the expected degree of concentration of return sludge solids. For this reason, the surface settling rate alone is not an adequate basis for establishing final settling tank area. The author previously has illustrated how selection of final settling tank area solely on the basis of the surface settling rate could lead to deterioration of the biological phase of the process and of the settling characteristics of the sludge solids because of the inability of the tank to sufficiently concentrate solids for recycle (6). In such cases, thickening requirements — not surface settling rate considerations — govern selection of the proper final settling tank area.

"A Minimum Detention of Two Hours Must Be Provided"

The second stipulation of final sedimentation tank design criteria commonly adopted by regulatory agencies is that the hydraulic retention time (final sedimentation tank volume divided by the throughput rate) should not be less than some stipulated value — typically in the order of 2 hr. It is unclear whether this standard is intended to assure some minimum retention time for the clarification function of the tank or for the thickening function. In view of the preoccupation with clarification and the normal exclusion of recycle in calculation of retention time, it is assumed that proper clarification is the goal of the minimum retention time requirement.

Theoretically, volume (or depth) for the clarification function of sedimentation basins is unimportant unless the particles being removed are flocculant (7). This lack of importance of depth for clarification of nonflocculant particles is illustrated by Equation 2. If the depth, h_c, of the clarification portion of a settling tank is doubled, a particle which settles at velocity v_c has twice as far to settle in order to be removed. However, this effect is exactly compensated by the fact that the particle now has twice as much time to reach the bottom (because t_c also doubles). Thus, nothing theoretically is accomplished by providing added depth for clarification, and Equation 3 is independent of depth when particles are not flocculant. When particles are flocculant, v_c may increase as a function of time because of particle growth. Then, retention time does become a reasonable parameter in design of clarifiers.

Activated sludge particles, of course, do fall into the domain of flocculant solids. Data, such as presented by Fischerström, et al, (8), Parker, et al, (9), and others show this to be the case.

In view of this flocculant nature of activated sludge solids, specification of a retention time requirement for final sedimentation tanks seems possibly to be an appropriate type of standard for design of the clarification function, and laboratory evaluation of the flocculant nature of activated sludge particles (10) may be warranted. However, as commonly used, the retention time design standard would seem to have two basic limitations. One is that the entire tank volume is used in calculating the theoretical retention time when, in fact, a substantial volume may be occupied by sludge. The second limitation is that while retention time calculated on the basis of the waste flow rate normally is the sole basis for establishing the volume of a final settling tank it deals only with the clarification function of the tank. Thus, in the case of the surface settling rate criterion, the retention time requirement ignores totally the thickening function of final settling tanks.

The recognition here of retention time as a rational parameter for conceptual consideration of the performance of activated sludge final sedimentation tanks does not infer that the retention time required by conventional standards (3) necessarily are warranted. Flocculation of activated sludge, as it occurs by provision of residence time in final sedimentation tanks, must be very inefficient. If, as some investigators [for example, Bradley and Krone (11) and Parker, et al (9)] suggest, activated sludge clarification is inhibited due to floc particle deterioration in the aeration tank, then better floc conditioning than is afforded by provision of retention time in the settling basin may be justified. Parker, et al (9) have suggested that a mildly stirred flocculation step is warranted between the aeration tank and the final tank. It would seem that, if this advice were followed, justification of common final settling tank retention time requirements would become even more tenuous.

"The Return Sludge Concentration Will Be 10^6/SVI"

Another common piece of folklore is that the underflow concentration, c_u, from a final sedimentation tank should equal 10^6 divided by the sludge volume index (SVI). This approach has been advocated for at least three decades (12), is recommended in prominent textbooks (13, 14) and design manuals (15), and forms a part of some current attempts to simulate mathematically the performance of the activated sludge process (16, 17). Some, like Vosloo (17), have modified the more usual approach to calculating the final settling tank underflow concentration from SVI measurements by taking the underflow concentration to be some constant times 10^6/SVI.

Inasmuch as the SVI is the volume in ml occupied per gm of solids following 30 min of sedimentation in a 1 l cylinder, it follows that 10^6/SVI is the average concentration in mg/l, of sludge at the bottom of a graduated cylinder at the completion of the SVI test. This bit of folklore, then, involves the assumption that thickening, as it occurs in a final settling tank, is equivalent to the thickening accomplished by 30 min of batch sedimentation.

A brief evaluation of this approach previously has been presented by Dick and Vesilind (18). They indicated that sedimentation, as it occurs in the small, unstirred cylinder used in the SVI determination, might be appreciably different from that which occurs in large full-scale tanks. It also was noted that the SVI is determined only by one point on a sedimentation curve and that sludges with a variety of different sedimentation characteristics could arrive at the same sediment volume after 30 min of sedimentation.

But the major limitation in using the sludge volume index to approximate final settling tank underflow concentration is that it denies the operator or designer of sedimentation tanks the opportunity to control the concentration of sludge recycled to the activated sludge process. That is, the 10^6/SVI rule suggests that underflow concentration is fixed by the physical characteristics of activated sludge and is totally independent of the design of the final settling tank and of the manner in which it is operated. As will be described later, both of these factors in addition to the inherent settling characteristics of the sludge, influence underflow concentration.

It is concluded that, while SVI is related to settling characteristics of activated sludge, it is not a reasonable basis for estimating anticipated final settling tank performance because it does not take into account the loading and mode of operation of the final settling tank. A more direct and meaningful measure of sludge settling properties than given by the SVI is needed to permit reliable prediction of settling tank performance. It must be noted, however, that Schaffner, et al (18) recently evaluated the 10^6/SVI rule and concluded that it still was the best available operating procedure because its conservative characteristics compensated for remaining unknowns in behavior of sedimentation basins.

"Activated Sludge Solids Must Be Recycled Quickly"

An additional element of folklore which has developed concerning performance of final sedimentation tanks is that it is essential that sludge be quickly recycled back to the aeration tank. The tacit assumption is that it is harmful to hold activated sludge under anoxic conditions for long periods of time.

Data gathered by Whurmann (20) indicated that activated sludge was not influenced by exposure to long periods of anaerobiosis and, in some British experiments (21), retention of activated sludge under anaerobic conditions for periods as long as 24 hrs did not adversely effect the ability of sludge to remove substrate when returned to the aeration tank. It should be noted, however, that neither of these experiments involved investigation of the effect of anaerobic conditions on selection of the type of organisms which prevailed in the activated sludge. In this regard, Okun (22) suggested that long periods of anaerobiosis might be used beneficially as means for avoiding growth of undesirable organisms in activated sludge.

Thus, while it may seem obvious that exposure of activated sludge to extended periods of anoxic conditions should be deleterious to the biological phase of the activated sludge process, little evidence exists to support this view. In contrast, although the effects of temporary anaerobiosis on population dynamics are not well documented, available information indicates that retention of activated sludge solids in final settling tanks does not cause adverse effects on the rate of removal of carbonaceous material. However, it must be emphasized that, under certain circumstances, extremely long periods of sludge retention in the final settling tank could be expected to be detrimental. Flotation of nitrified sludges due to biological denitrification in final tanks is to be avoided (23), and if biological mechanisms are to be relied upon for phosphorus removal, extended periods of sludge storage in the final settling tank cannot be allowed (24).

Attention is given here to folklore concerning the need for rapid recycle of activated sludge solids because it would seem that preoccupation with that undocumented requirement often leads to operation of final settling tanks at far less than their capacity. The urge to quickly recycle solids frequently results in maintenance of a sludge depth too shallow to permit proper thickening. Whereas activated sludge sedimentation velocities may be reduced by use of shallow sludge depths (25), a far more significant effect is that use of shallow sludge depths in final settling tanks maximizes the opportunity for withdrawing overlying clarified liquid along with thickened sludge. A comparison of actual and expected final settling tank underflow concentrations serves to illustrate this point. At a plant which maintained a minimal sludge blanket so as to reduce solids retention time, average return sludge concentration during three daily study periods was 4,500 mg/l (26). Examination of settling characteristics during the same three periods and application of the solids flux analysis (described later) showed that the maximum achievable underflow concentration was 16,000 mg/l, and the 10^6/SVI rule indicated a recycle concentration of about 9,500 mg/l. Thus, it would seem that at this plant, the desire to avoid long retention of solids in the final tank resulted in an appreciable decrease in the suspended solids concentration of the return sludge and, hence, in the mixed liquor suspended solids concentration and the capacity of the treatment plant.

"The Solids Loading on Final Tanks Should Not Exceed 20 lb/sq ft/day"

A recent addition to the folklore of final settling tank analysis is the inclusion of consideration of the solids loading on the tank. Use of the solids loading parameter is, of course, appropriate because it relates to the thickening function of final sedimentation basins (6).

In cases in which maximum solids loadings are suggested, values in the order of 20 lb/sq ft/day (98 kg/sq m/day) are typical. The ASCE-WPCF Manual of Practice on Sewage Treatment Plant Design (15) indicates that "available data show plants operating successfully with loadings of 12 to 18 psf/day with sludge volume indices under 100." In the same manual, it is concluded that "in general, with mixed liquor concentrations of 3,000 mg/l or less, with a sludge index of 100 or less, and a tank underflow of not more than 1 percent solids, the area determined by the overflow rate is adequate for the solids." This condition 'including recycle' corresponds to a solids loading of about 28 lb/sq ft/day (137 kg/sq m/day).

Whereas adoption of solids loading as a parameter related to the thickening performance of final settling tanks is logical, the danger of the approach would seem to be in specifying the permissable magnitude of the solids loading. As illustrated in the following paragraph, the solids load which successfully can be handled by a final sedimentation tank is a function both of the settling characteristics of the sludge and of the mode of operation of the final sedimentation tank, and succsssful performance may be achieved over a wide range of solids loading values.

The reason for restricting the applied solids load on final settling tanks is that each layer of a sludge which might exist in the tank has some definite capacity for transmitting solids to the bottom of the tank (27). This capacity is determined both by the settling characteristics of the sludge and by the rate of removal of return sludge. It is essential that the capacity of any layer which might exist in the tank for transmitting solids not be exceeded or else solids in excess of those which can pass through the limiting layer will accumulate and, in time, move into the clarification portion of the tank (and "bulking" might be said to occur). The capacity, G_i, of a sludge layer of suspended solids concentration, c_i, for transmitting solids to the bottom of a final sedimentation tank is:

$$G_i = c_i v_i + c_i \frac{rQ}{A} \tag{4}$$

where v_i is the gravity settling velocity of the sludge at concentration c_i, Q is the waste throughput rate, r is the recycle fraction and A is the cross-sectional area of the final sedimentation tank. The influence of sludge wastage on the solids transport due to sludge removal has been omitted from the second term on the right side of Equation 4, but readily could be added.

As described in a later section, rational design of final sedimentation tanks requires that the minimum value of G_i for all possible concentrations which could occur in the final settling be identified. Then, the average applied solids flux, G_a, must not be allowed to exceed this limiting value for G_i. The applied solids flux may be computed from:

$$G_a = \frac{(1 + r)Q\, c_{MLSS}}{A} \tag{5}$$

where c_{MLSS} is the mixed liquor suspended solids concentration.

It is noted that for sludge of some concentration, c_i, the magnitude of the first term on the right side of Equation 4 depends only on the settling characteristics of the sludge, whereas the magnitude of the second term depends only on the way in which the final settling tank is operated; that is, on the rate of recycle, r. It is for this reason that it is inappropriate to attempt to designate a proper solids loading on final sedimentation tanks. The magnitude of the first term in Equation 4 varies widely depending on the particular nature of the waste being treated and the mode of operation of the biological phase of the process. The magnitude of the second term is controlled by the way in which the final settling tank is operated, and can be varied over wide limits. There are activated sludge plants operating at solids loadings far less than 20 lb/sq ft/day (98 kg/sq m/day) which have difficulty in thickening solids and, conversely, plants are in operation with solids loadings of 80 lb/sq ft/day (390 kg/sq m/day) or more with good performance. Thus, it seems undesirable to attempt to specify a loading to represent a "typical" design value.

"The Overflow Rate Equals the Settling Velocity of the Mixed Liquor"

In some final settling tank folklore which currently seems to be gaining in popularity is the idea that the overflow rate should be selected so as to equal the settling velocity of the sludge at the mixed liquor suspended solids concentration. The concept is at least as old as Camp's 1946 paper (7), frequently appears in current literature, and has been advocated in recent texts (28, 29). On casual analysis, the approach may appear to have merit because it might intuitively seem that if solids entering a settling tank are to avoid being carried into the overflow their rate of sedimentation should equal or exceed the upflow velocity in the tank. Furthermore, the settling velocity of the mixed liquor can be readily determined so as to permit convenient use of this piece of folklore to establish the design overflow rate. However, critical analysis of this approach to final sedimentation tank design indicates that it lacks fundamental foundation, and it is, perhaps, one of the most misleading bits of final settling tank folklore. This is illustrated in the following paragraph.

Consider a final settling tank, which receives solids from the aeration tank at a rate equal to $(1 + r)Q\,c_{MLSS}$, and consider that solids at the top of the sludge blanket in the tank are at concentration c_t. At steady state, the rate of solids entry into the layer of concentration c_t must equal the rate of removal from that layer, or:

$$(1 + r)Q\,c_{MLSS} = c_t\left(v_t + \frac{rQ}{A}\right)A \tag{6}$$

where v_t represents the rate at which solids at concentration c_t settle under the influence of gravity. The left side of Equation 6 represents the total applied solids load on the settling tank, and the right side is the flux of solids through the layer of concentration c_t as given by multiplying Equation 4 times the cross-sectional area of the tank. Now, folklore has it that the concentration c_t is the same as the mixed liquor concentration, c_{MLSS}. That is, in equating the overflow rate to the settling velocity of the mixed liquor it is assumed that the mixed liquor concentration exists in the final settling tank. To see if this is the case, an expression for c_t can be obtained from Equation 6.

$$c_t = \left[\frac{(1 + r)Q}{v_tA + rA}\right] c_{MLSS} \tag{7}$$

It is seen that only when the quantity in brackets has the value of unity does c_t equal c_{MLSS}. Whereas it is possible that the expression could equal one, there is no particular reason that it should. With a tank of given area, the magnitude of the term depends on the settling characteristics of the sludge (v_t) and the mode of the operation of the sedimentation tank (r and Q). Under normal circumstances, the quantity does not have a value of unity, the mixed liquor would become diluted upon entry to the tank, the concentration c_{MLSS} would not reappear, and this element of final sedimentation tank folklore would be extremely misleading. The concentration which should appear at the upper level of the blanket in well designed final settling tanks corresponds to the intersection of the operating line with the rising portion of a batch flux plot (27). The same value can be computed from

Equation 7. Experimental results from bench scale sedimentation basins agree closely with this expected result (30).

RATIONAL DESIGN OF FINAL SETTLING TANKS

Whereas aspects of some of the folklore in design and operation of final sedimentation tanks have been shown to be compatible with fundamental analyses of sedimentation tank performance, no combination of the many rules-of-thumb affords a totally rational analysis of final sedimentation tank performance. In this section, an approach to final settling tank design based on fundamental principles of sedimentation theory is reviewed. The approach is deficient because it does not take into account conditions at the inlet and outlet related to velocity distribution, density currents, and related factors which detract from the theoretical performance of settling tanks. It is suggested, however, that fundamental understanding of sedimentation tank performance has advanced to the point that there are advantages in beginning to abandon traditional rules-of-thumb as an approach to design and operation of final settling tanks. In this way, the actual effect of remaining unknowns can be observed (as the difference between actual and theoretical predicted performance) and taken into account.

Inasmuch as final settling tanks are expected to accomplish both clarification of final effluent and thickening of return sludge, it seems reasonable that both thickening and clarification should be considered in design. Furthermore, it has been seen that both area and volume (or depth) influence the degree of both clarification and thickening. It would seem, then, that rational analysis of final sedimentation tank design would involve selection of the larger of the two area requirements and the establishment of a depth sufficient to give a total volume adequate for both functions.

The area required for clarification is that corresponding to an overflow rate equivalent to the sedimentation velocity of the smallest particle to be completely removed. Traditional design approaches are adequate in this respect (although the magnitude of required overflow rates deserves evaluation). If flocculation of "stray" activated sludge particles in the upper part of the sedimentation tank is to be provided for, then retention time must be allowed in the clarification portion of the tank. Again, traditional design requirements take this need into account (although the standards do not distinguish between the volume provided in the clarification zone and the volume provided in the thickening zone, and the validity of the detention time commonly required has not been well documented).

The area required for the thickening function is that which assures that the applied flux (Equation 5) does not exceed the solids transmitting capacity of any layer of sludge which can exist in the final settling tank (as given by Equation 4). The value of Equation 4 for all concentrations of sludge can be obtained by determining experimentally the relationship between settling velocity and concentration and selecting an underflow concentration (and, hence, a recycle rate). Figure 2 (from Dick (6)) illustrates the total possible flux for various concentrations of an activated sludge. In the Figure, u represents the downward velocity, r Q/A, caused by sludge removal. The lower, linear curve represents the flux due to underflow ($c_i \frac{rQ}{A}$) while the difference between that curve and the total flux is the additional flux, $c_i v_i$, due to gravity subsidence. It is seen that, in range of solids concentrations which might occur in the thickener, one concentration, c_L, has less capacity for transmitting solids than all others, and the limiting flux, G_L, corresponding to this concentration establishes the maximum permissible value for G_a in Equation 5. This approach has been described in more detail elsewhere (6), and a more convenient procedure for using the method by making use of a geometric construction procedure with a batch flux curve such as illustrated in Figure 3 has been described (17). Essentially, this procedure involves identifying the desired underflow concentration, striking a tangent to the batch flux curve and obtaining, as the intercept, the corresponding maximum applied flux. Alternatively, the procedure may be used to identify the maximum underflow concentration which can be expected to be produced when solids are applied at some predetermined rate.

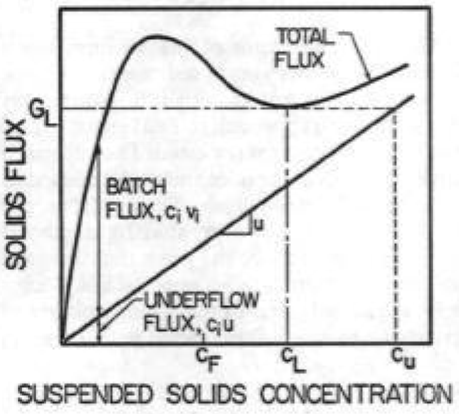

Figure 2 — Typical total flux curve illustrating the contribution of sludge removal to solids handling capacity of final sedimentation tank.

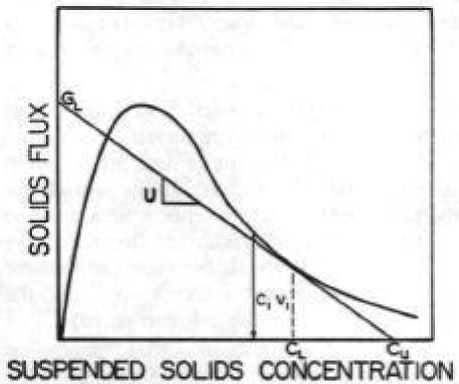

Figure 3 — Identification of operating conditions using flux plot.

The volume (or depth) in the thickening portion of the tank theoretically is unimportant unless the solids are flocculant and, thus, compressible. Such is the case with activated sludge (25) although with sludges of good quality the effect of depth on settling velocity may not be adequate to warrant consideration (30). Should it be desired to evaluate the approximate effect of alternative sludge depths, this can be done by constructing batch flux curves from a series of sedimentation tests conducted at different initial depths. Then the possible effect of sludge depth on performance can be evaluated in a gross way by comparing various combinations of area and volume which give the same performance. Probably of more concern with regard to depth for thickening is the necessity that an adequate sludge blanket be maintained so that overlying clarified liquid is not withdrawn along with thickened sludge. Additionally, it is necessary to provide sufficient depth to store solids during temporary periods of overloading. That is, when the applied solids loading (as given by Equation 5) exceeds the capacity of the suspension, under the operating conditions, for transmitting solids to the tank bottom, (as given by the limiting value of Equation 4), the excess of the applied loading over the solids handling capacity must be allowed to accumulate in the thickening portion of the tank or else, in time it will enter the clarification portion of the tank. This situation, such as might be caused by a temporary increase in waste flow rate, can be controlled without deterioration of performance if adequate depth is available to accumulate the excess solids at the concentration of the limiting layer.

Additionally, inasmuch as a single tank is expected to accomplish two functions, it is appropriate that tradeoffs be made so that overall design is an optimum combination of thickening and clarification requirements. Application of rational approaches to final settling tank design should permit this optimization to be carried out.

SUMMARY

Because final settling tank design and operation criteria have developed as an art, it is appropriate to examine the approaches which have been used. Stipulation of some maximum overflow rate and minimum retention time (as is customary in regulatory agency standards) seem reasonable — but only for description of the clarification function of the tank. These parameters completely disregard the thickening function of the tank.

The sludge volume index frequently has been used (with the equation $c_u = 10^6/SVI$) as a basis for estimating the concentration of return sludge to be expected from a final settling tank. This approach seems questionable for several reasons — most importantly because it does not take into account the designer and operator's ability to alter the performance of final settling tanks to achieve a range of return sludge concentrations.

The concept that solids retention time must be minimized in final sedimentation tanks does not seem to be well-founded except as it relates to possible flotation of sludge due to denitrification and to release of phosphorus from sludges. Unnecessary restriction of sludge residence time in final sedimentation tanks because of fear of deterioration of the sludge's ability to remove carbonaceous material may reduce the effectiveness of final tanks by causing unintentional withdrawal of overlying clarified water with thickened sludge.

The idea that overflow rates in final sedimentation tanks should be selected to be equivalent to the settling velocity of the mixed liquor is misleading, theoretically incorrect, and can result in seriously inaccurate sedimentation tank design. The fallacy of this approach is the tacit assumption that the mixed liquor concentration exists in the final settling tank. It is shown here that sludge should be expected to be present in the final settling tank at the mixed liquor concentration only under very unusual circumstances.

The rational approaches to design of final settling tanks which briefly are described here define final settling tank performance only under optimum conditions because they do not take imperfect distribution of solids and liquid within the tank into account. This "limitation" of the approach serves as a valuable means for evaluating final settling design and operational modes. Comparison of real and predicted performance indicates the extent to which optimal conditions are not being achieved. Final settling tank folklore fails to offer such a means for evaluating performance because it normally is not based on rigorous relationships between tank loading and performance.

NOTATION

c_f = suspended solids concentration of flow entering the final settling tank (c_{MLSS}), ML^{-3}.

c_i = suspended solids concentration of sludge solids in a horizontal layer in a final settling tank, ML^{-3}.

c_L = suspended solids concentration of layer in final settling tank which limits thickening capacity, ML^{-3}.

c_{MLSS} = mixed liquor suspended solids concentration, ML^{-3}.

c_t = suspended solids concentration of sludge at the top of the sludge blanket in final sedimentation tanks, ML^{-3}.

c_u = suspended solids concentration of sludge continuously withdrawn from bottom of sedimentation tank for recycle and wastage, ML^{-3}.

A = horizontal cross-sectional area of final settling tank, L^2.

G_a = applied solids load in final settling tank, $MT^{-1}\,L^{-2}$.

G_i = solids handling capacity of sludge layer with concentration c_i in a final settling tank from which solids continuously are recycled, $MT^{-1}\,L^{-2}$.

G_L = limiting solids handling capacity, $MT^{-1}\,L^{-2}$.

h_c = depth of clarification portion of final settling tank, L.

r = ratio of rate of sludge recycle to Q, $L^3\,T^{-1}$.

t_c = theoretical hydraulic retention time in clarification portion of final settling tank, T.

Q = waste flow rate, $L^3\,T^{-1}$.

Q_c = rate of flow of clarified effluent from final settling tank, $L^3\,T^{-1}$.

Q_u = rate of removal of thickened sludge, $L^3\,T^{-1}$.

u = downward velocity in thickening portion of final settling tank caused by sludge removal (Q_u/A), LT^{-1}.

v_c = settling velocity of slowest settling particle completely removed from clarification portion of final settling tank, LT^{-1}.

v_i = settling velocity of sludge at concentration c_i, LT^{-1}.

REFERENCES

1. Ardern, E. and Lockett, W.T., "Experiments on the Oxidation of Sewage without Aid of Filters," *Journal Society of the Chemical Industry,* 33, 10, 1122, (1914).

2. Babbitt, H.E., *Sewerage and Sewage Treatment,* Second Edition, John Wiley and Sons, Inc., New York, 503 pp., (1925).

3. Great Lakes-Upper Mississippi River Board of State Sanitary Engineers, "Recommended Standards for Sewage Works," Health Education Service, Albany, New York, (1971).

4. Hazen, A., "On Sedimentation," *Transactions American Society of Civil Engineers,* 53, 45, (1904).

5. Camp, T.R., "A Study of the Rational Design of Settling Tanks," *Sewage Works Journal,* 8, 742, (1936).

6. Dick, R.I., "Role of Activated Sludge Final Settling Tanks," *Journal Sanitary Engineering Division American Society of Civil Engineers,* 96, SA2, 423, (1970).

7. Camp, T.R., "Sedimentation and the Design of Settling Tanks," *Transactions American Society of Civil Engineers,* 111, 2285, 895, (1946).

8. Fischerstöm, C.N.H., Isgård, E., and Larsen, I., "Settling of Activated Sludge in Horizontal Tanks," *Journal Sanitary Engineering Division American Society of Civil Engineers,* 93, SA3, 73, (1967).

9. Parker, D.S., Kaufman, W.J., and Jenkins, D., "Physical Conditioning of Activated Sludge Floc," *Journal Water Pollution Control Federation, 43, 9, 1817, (1971).*

10. O'Connor, D.J., and Eckenfelder, W.W., Jr., "Evaluation of Laboratory Setting Data for Process Design," Biological Treatment of Sewage and Industrial Wastes, Vol. II, B.J. McCabe and W.W. Eckenfelder, Jr., eds., Reinhold Publishing Corp., New York, 171, (1957).

11. Bradley, R.A. and Krone, R.B., "Shearing Effects on Settling of Activated Sludge," *Journal Sanitary Engineering Division American Society of Civil Engineers,* 97, SA1, 59, (1971).

12. Bloodgood, D.E., "Application of Sludge Index (Mohlman) Test to Plant Operation," *Waterworks and Sewerage,* 91, 222, (1944).

13. Fair, G.M., Geyer, J.C., and Okun, D.A., *Water and Wastewater Engineering, Volume 2, Water Purification and Wastewater Treatment and Disposal,* John Wiley and Sons, New York, (1968).

14. Metcalf and Eddy, Incorporated, *Wastewater Engineering,* McGraw Hill Book Co., New York, 782 pp., (1972).

15. "Sewage Treatment Plant Design," *ASCE WPCF Manual of Engineering Practice,* Water Pollution Control Federation, Washington, D.C., 375 pp., (1959).

16. Lawrence, A.W., and McCarty, P.L., "Unified Basis for Biological Treatment Design and Operation," *Journal Sanitary Engineering Division American Society of Civil Engineers,* 96, SA3, 757, (1970).

17. Vosloo, P.B.B., "Some Factors Relating to the Design of Activated Sludge Plants," *Water Pollution Control, 69, 5, 486, (1970).*

18. Dick, R.I., and Vesilind, P.A., "The Sludge Index — What Is It?" *Journal Water Pollution Control Federation, 41, 7, 1285, (1969).*

19. Schaffer, M.W., Tilghman, R.H., and Pipes, W.O., "The Use of Batch Settling Tests For the Control of an Activated Sludge Process," Paper Presented at Annual Meet-

ing of the Central States Water Pollution Control Association, Rockford, Illinois, June 13-15, 1973.

20. Wuhrmann, K., "Effect of Oxygen Tension in Biochemical Reactions in Sewage Purification Plants," Advances in Biological Waste Treatment, W.W. Eckenfelder, Jr., and J. McCabe, eds., Pergamon Press, New York, 27, (1963).

21. Water Pollution Research Board, Water Pollution Research 1972, Department of Scientific and Industrial Research, London, England, (1963).

22. Okun, D.A., Discussion of "Effect of Oxygen Tension in Biochemical Reactions in Sewage Purification Plants," by K. Wuhrmann, Advances in Biological Waste Treatment, W.W. Eckenfelder, Jr., and J. McCabe, eds., Pergamon Press, New York, 38 (1963).

23. Sawyer, C.N., and Bradney, L., "Rising of Activated Sludge in Final Settling Tanks," *Sewage Works Journal, 17, 6, 1191, (1945).*

24. Shapiro, J., Levin, G.V., and Huberto, Z.G., "Anoxically Induced Release of Phosphate in Waste Water Treatment," *Journal Water Pollution Control Federation*, 39, 11, 1810, (1967).

25. Dick, R.I. and Ewing, B.B., "Evaluation of Activated Sludge Thickening Theories," *Journal Sanitary Engineering Division American Society of Civil Engineers*, 93, SA4, 9-29, (1967).

26. Rose, W.D., "Misuse of the SVI to Predict Activated Sludge Solids Settleability," Special Problem to fulfill Requirements of Master's Degree, Department of Civil Engineering, University of Illinois, Urbana, 68 pp., (1973).

27. Dick, R.I., "Gravity Thickening of Sewage Sludges," *Water Pollution Control*, 71, 4, 368, (1972).

28. Eckenfelder, W.W. and Ford, D.L., *Water Pollution Control*, Pemberton Press, New York, 269 pp., (1970).

29. Javaheri, A.R. and Dick, R.I., "Continuous Settling and Thickening of Activated Sludge," paper in preparation.