A SunCam online continuing education course

Introduction to Wastewater Clarifier Design

by

Nikolay Voutchkov, PE, BCEE

1. Introduction

Clarifiers (also referred to as sedimentation tanks or settlers) are an integral part of every wastewater treatment plant. At these treatment facilities, solids are removed from the wastewater by using gravity sedimentation in quiescent conditions. All clarifiers have two functional zones – a **clarification zone**, where the process of gravity sedimentation occurs, and a **thickening zone**, where the settled solids are accumulated forming a dense layer of sludge (sludge blanket). Clarifier effluent of low solids concentration is collected from the top of the clarification zone over overflow weirs and into collection channels where it is conveyed to the tank outlet. The sludge collected at the bottom of the clarifier is removed for further treatment at the wastewater treatment plant's solids handling facilities. The depth of the clarification zone is commonly referred to as the **clear water zone** (CWZ) depth, while the depth of the zone of sludge accumulation is called the **sludge blanket depth** (SBD). The sum of the CWZ depth and the SBD is defined as the side water depth (SWD).

2. Types of Clarifiers

Depending on their function, clarifiers are categorized as **primary** and **secondary**. Primary clarifiers are located downstream of the wastewater treatment plant headworks. Their main purpose is to remove the settleable suspended solids in the plant influent. Customarily, primary clarifiers are also equipped with devices for removal of the floatable compounds (i.e., scum, oil and grease) in the wastewater influent as they accumulate on the surface of the tanks during the sedimentation process.

Secondary tanks are located downstream of the biological (secondary) treatment facilities of the wastewater treatment plant (such as activated sludge aeration basins or trickling filters) and are used to separate the biomass generated during the secondary treatment process from the treated plant effluent.

Primary and secondary clarifiers are classified in two main categories according to geometrical shape: rectangular and circular. The most suitable shape for a given application depends on several factors and should be selected based on a cost-benefit analysis. Table 1 summarizes the key advantages and disadvantages of rectangular and circular clarifiers.

Table 1 - Comparison of Rectangular and Circular Clarifiers

Item	Rectangular Clarifiers	Circular Clarifiers
Advantages	Less land required for construction of multiple units. Potential construction cost savings due to use of common walls between individual tanks. Longer flow path minimizing short-circuiting. Higher effluent weir loading rates acceptable. Better sludge thickening.	Shorter detention time for settling sludge favoring use as secondary clarifiers. More simple sludge collection system. Easier to accommodate in-tank flocculation chamber – a benefit for activated sludge settling. Overall, lower maintenance requirements. Easier to remove heavy sludge.
Disadvantages	Longer detention time of the settled sludge – not favorable for plants with septic wastewater influent. Less effective for high solids loading conditions.	Higher short-circuiting potential. Higher flow distribution headlosses. Small circular tanks require more yard piping than rectangular tanks of similar size.

3. Rectangular Clarifiers

Rectangular clarifiers are long concrete structures consisting of individual basins (units) having common inner walls with inlet and outlet channels (Figure 1). Each tank basin is equipped with a separate sludge collection mechanism that transports the solids settled in the tank into a hopper for withdrawal. Alternatively, a sludge suction collection mechanism may be used to sweep and remove solids accumulated at the tank's bottom. The length-to-width ratio of the individual tank basin is usually 3:1 to 15:1 (Figure 1). The minimum clarifier length from inlet to outlet is commonly 3 meters (10 ft.). Tank depth is most frequently between 2 to 6 meters (6.6 to 20 ft). Rectangular tank unit width is usually selected based on available standard sizes of sludge collection mechanisms and varies between 2 to 6 meters (6.6 to 20 feet).

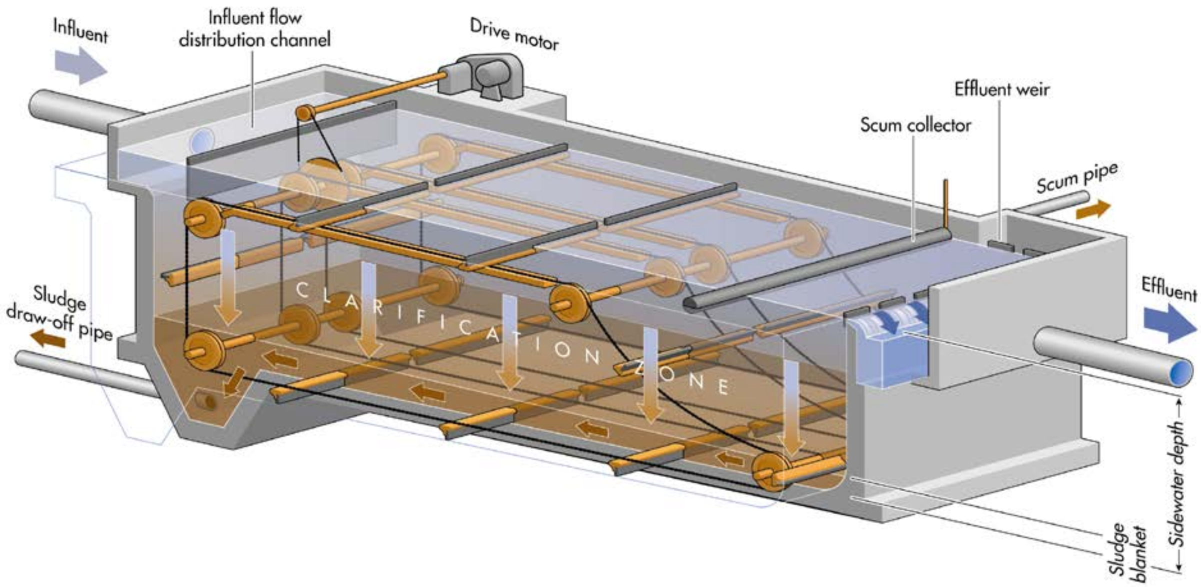


Figure 1 – Rectangular Clarifier

4. Circular Clarifiers

Circular clarifiers are round facilities consisting of an inlet structure, a cylindrical clarification zone, a conical sludge accumulation zone, and effluent weirs (Figure 2). The effluent weirs are placed near the facility perimeter to create a radially directed flow pattern from the tank center towards the walls. The slope of the bottom conical floor is usually 1:10 to 1:12 and depends on the type of the sludge collection mechanism. The tank diameter ranges from 3 meters (10 ft) to over 100 meters (300 ft). Circular clarifiers are typically built in pairs of 2 or 4 to simplify the influent flow distribution between the individual units. Circular tank sidewater depth varies from 2.5 to 5 meters (8 to 16 feet).

Depending on the configuration of the tank inlet, circular clarifiers are classified as either **center feed** or **peripheral feed**. Currently, the most widely used circular tanks are center feed type (see Figure 2). In these tanks, influent flow enters through a feed pipe located in the center of the tank and into a feed well. The purpose of the feed well is to provide uniform radial distribution of the tank influent and to dissipate the energy of the feed stream to a level adequate for efficient quiescent settling and uniform radial flow distribution.

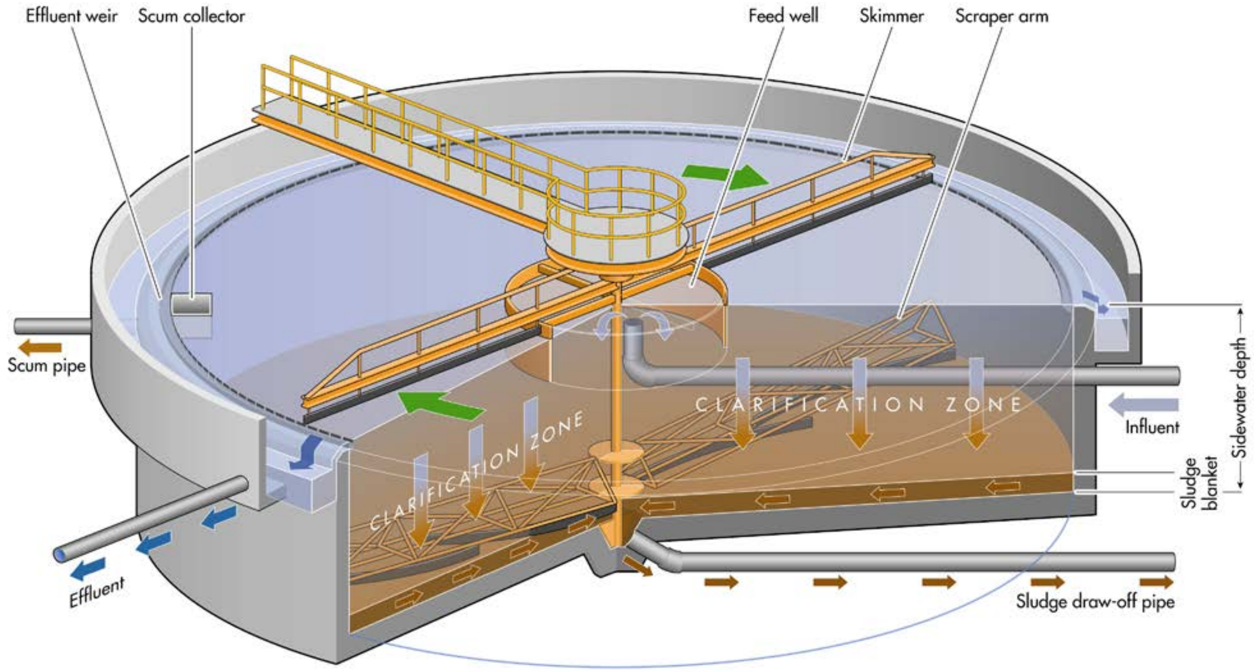


Figure 2 – Circular Clarifier

The conventional feed wells most widely used today are cylindrical metal structures with diameter that is 15 to 25% of the tank diameter and which extend to 30 to 75% of the tank sidewater depth. Typical conventional feed wells are designed for an average downflow velocity of 10 to 13 mm/s (2.0 to 2.5 ft/min) and maximum velocity of 25 to 30 mm/s (5.0 to 6.0 ft/min).

5. Enhanced Clarifiers

Inclined plates and **ballasted flocculation** are used predominantly to enhance the performance of primary rectangular clarifiers. A typical inclined plate (lamella) system consists of bundles of parallel plastic tubes or metal plates inclined at 45 to 60^0, which are installed at the surface of the clarifier to a vertical depth of approximately 2 meters (6 feet). The distance between the individual plates is between 40 and 120 mm (1.6 to 4.8 inches).

Ballasted flocculation combines the addition of coagulant and settling ballast (usually fine sand or sludge) to the tank influent with the installation of inclined plates in the tanks. A portion of the settled sludge or the recovered ballast is recycled to the primary clarifier influent to seed the influent. The addition of ballast increases the density of the influent floc particles by agglomeration. This enhancement typically yields a three to five-fold increase of the allowable clarifier surface overflow rate (SOR). Ordinarily, conventional clarifiers are designed for SOR of 33 to 49 m^3/m^2.day (800 to 1,200 gal/ft^2.day). The use of high-rate ballasted solids separation technology allows increasing design clarifier SOR to at least 160 m^3/m^2.day (4,000 gal/ft^2.day). Because the ballast enhances solids removal, its use in primary clarification reduces the solids and organic loading of the downstream biological treatment processes.

6. Flocculating Center Feed Well For Improved Sedimentation

Flocculating center feed wells are used to enhance the performance of secondary clarifiers used for settling of activated sludge. Compared to a conventional center feed well with a radius that is approximately 10 to 13% of the tank radius, the flocculating feed well's radius extends to 20 to 50% of the tank radius. Because of this, the well size is designed to obtain a detention time of 20 to 30 minutes. The flocculating feed well typically extends down 40 to 50% of the tank depth. Some designs also include installation of mechanical mixers in the feed well to enhance the flocculation process. The enhancement aims at creating optimum conditions for coagulation and flocculation of the incoming solids with the **return activated sludge** (RAS) recycled to the sedimentation tank. In the feed well, the larger recycled RAS particles are given an ample time to attract and flocculate the smaller activated sludge particles conveyed form the aeration basins, thereby creating stronger and heavier solids particles that settle better and faster. More detailed design considerations for circular clarifiers with flocculating feed wells, as well as several other available sedimentation tank process and equipment enhancements, are presented elsewhere (WEF, 2005).

7. Key Design Criteria For Primary Clarifiers

Performance efficiency of t primary clarifiers is affected by the upstream wastewater collection and treatment facilities and has a significant impact on downstream biological treatment and solids handling facilities. Primary clarifier performance is typically measured by the tank's total suspended solids (TSS), biological oxygen demand (BOD), phosphorus removal efficiencies, and by the condition of the primary sludge (sludge septicity, concentration, and volume). Adequately designed and operated conventional primary clarifiers that treat municipal wastewater will typically remove 50 to 65% of the influent TSS, 25 to 35% of the influent BOD and, 5 to 10% of the influent nitrogen and phosphorus. Clarifier TSS, BOD and nutrient removal efficiencies could be improved by chemical coagulation and flocculation of the influent wastewater solids prior to sedimentation.

Table 2 - Key Design Criteria for Primary Sedimentation Tanks

Design Guideline Source	**Surface Overflow Rate (m^3/m^2.day)**	**Hydraulic Detention Time (hrs)**
Metcalf & Eddy (Primary Settling Followed by Secondary Treatment)	32 – 48 (at average flow) 80 – 120 (at peak hourly flow)	1.5 – 2.5
Randall, Barnard & Stensel	For SWD of 1.83 – 3.05 m: $\leq$ 2.184 x SWD^2 (at average flow) $\leq$ 4.368 x SWD^2 (at peak hourly flow). For SWD of 3.05 – 4.57 m: $\leq$ 6.672 x SWD (at average flow) $\leq$ 13.344 x SWD (at peak hourly flow)	NA
Ten State Standards	$\leq$ 40 (at average flow) $\leq$ 60 (at peak hourly flow) Tank surface area is determined based on the larger of the two SORs. Minimum SWD = 2.1 m	NA
Qasim	30 – 50 (at average flow) 40 (typical at average flow) 70 – 130 (at peak hourly flow) 100 (typical at peak hourly flow)	1.0 – 2.0

Note: SWD – Sidewater Depth; 1 m^3/m^2.day = 24.542 gpd/ft^2

Key design criteria for sizing primary clarifiers are **surface overflow rate** and **hydraulic detention time**. Recommended values for these criteria, per various design guideline sources, are presented in Table 2. Normally, primary sedimentation tanks are designed for effluent weir loading rates of less than 190 m^3/day per meter of length of the weir (5,000 gpd/ft).

Proper primary clarifier sludge collection, removal, and withdrawal are of key importance for maintaining consistently high primary effluent quality and efficient and cost-effective solids handling. If primary clarifier sludge is retained for excessively long time in the tanks, the sludge could easily turn septic. Sludge septicity is accompanied by the release of malodorous gases that may disturb the normal sedimentation process as they travel from the tank bottom to the surface. Septic sludge is also more corrosive and more difficult to pump and dewater. Besides creating conditions for sludge septicity, maintaining a relatively deep sludge blanket in the primary clarifiers may also make sludge collection and withdrawal more difficult. In extreme conditions, this may cause damage to the sludge collection and withdrawal equipment (broken sludge collectors, plugged solids lines, and damaged pumps).

A widely-accepted practice to prevent primary sludge septicity and its negative effect on clarifier performance is not to carry a sludge blanket. This is achieved by removing sludge continuously or very frequently from the clarifier's bottom. When not controlled appropriately, continuous sludge removal often results in pumping large quantities of diluted sludge or wastewater to the downstream solids handling facilities, which has a negative effect on their performance. Primary clarifier sludge blanket and concentration have to be maintained at optimum levels in order to avoid over-pumping of diluted sludge to the downstream solids handling facilities and to prevent the negative effects of an excessively deep sludge blanket and associated sludge septicity, The optimum primary sludge concentration is usually 3 to 5%, while the most viable sludge blanket depth is typically between 1 and 3 feet. The optimum sludge blanket depth would vary seasonally and change during dry-weather and wet-weather conditions.

7. Key Design Criteria for Secondary Clarifiers

The performance of the secondary clarifiers has a significant effect on the wastewater plant's effluent water quality, on the operational efficiency of the biological treatment system, and on the solids handling facilities. The secondary clarifiers have two key functions: clarification of the biologically treated wastewater; and thickening and storage of the sludge from the biological treatment process. Main factors that impact secondary clarifier performance are: (1) the amount of solids retained in the tanks, which is determined based on the concentration of the solids removed from these tanks (return activated sludge (RAS)/waste activated sludge (WAS) concentration) and the sludge blanket depth; (2) the amount of solids in the aeration basins, which is established by measuring the MLSS concentration and the RAS flowrate; (3) the activated sludge settleability; and (4) the plant influent flow and waste load, significant fluctuations of which may result in shifting solids between the clarifier and the aeration basin, and ultimately in solids loss with the secondary clarifier effluent. The two key secondary clarifier design criteria are: the SOR; and the solids loading rate (SLR). Table 3 presents recommendations for determining secondary clarifier design SOR and SLR. The tank effluent weir loading rates are typically designed not to exceed 124 m^3/day per meter of length of the weir (10,000 gpd/ft).

Table 3 - Key Design Criteria For Secondary Sedimentation Tanks

Design Guideline Source	**Surface Overflow Rate (m^3/m^2.day)**	**Solids Loading Rate (kg/m^2.h)**
Metcalf & Eddy Settling Following Air Activated Sludge (Excluding Extended Aeration). Settling Following Extended Aeration. Settling Following Trickling Filtration.	16 - 32 (at average flow) 40 – 50 (at peak hourly flow) 8 – 16 (at average flow) 24 – 32 (at peak hourly flow) 16 - 24 (at average flow) 40 – 50 (at peak hourly flow)	4 – 6 (at average flow) ≤ 14 (at peak hourly flow) 1 – 5 (at average flow) ≤ 7 (at peak hourly flow) 3 – 5 (at average flow) ≤ 8 (at peak hourly flow)
Randall, Barnard & Stensel	For CWZ of 1.83 – 3.05 m: ≤ 2.184 x CWZ (at average flow) ≤ 6.672 x CWZ (at peak hourly flow). For CWZ of 3.05 – 4.57 m: ≤ 4.368 x CWZ (at average flow) ≤ 13.344 x CWZ (at peak hourly flow) Minimum SWD = 4.5 m	≤ 5 (at average flow)
Ten State Standards Settling Following Air Activated Sludge (Excluding Extended Aeration). Settling Following Extended Aeration.	≤ 49 (at peak hourly flow) ≤ 41 (at peak hourly flow) Minimum SWD = 3.7 m	≤ 10 (at peak hourly flow) ≤ 10 (at peak hourly flow)
Qasim	≤ 15 (at average flow) ≤ 40 (at peak hourly flow)	≤ 2 (at average flow) ≤ 6 (at peak hourly flow)

Note: CWZ – Clear Water Zone Depth; 1 m^3/m^2.day = 24.542 gpd/ft^2; 1 kg/m^2.h = 0.2048 lb/ft^2.h.

The maximum allowable SLR of clarifiers for clarification of activated sludge could be determined by using solids flux analysis (WEF, 2005). This method is based on the fact that for an activated sludge of given settleability there is a maximum amount of solids that can be processed through the clarifier (limiting solids flux), above which the clarifier will not be able to operate in a steady-state condition in terms of sludge blanket elevation and effluent water quality. One of the main benefits of the solids analysis concept is that it allows a link between the design and operation of the secondary clarifier and the aeration basin to optimize their performance as one system.

The amount of solids retained in the sedimentation basins can be effectively monitored by frequent manual or automated measurements of the clarifier sludge blanket depth and the concentration of the sludge removed from the clarifiers. While keeping track of the sludge blanket and plant influent flow changes provides a general understanding of the clarifier performance, it is also very advantageous to monitor sludge settleability as well.

The Water Environment Research Foundation and the Clarifier Research Committee of the American Society of Civil Engineers have developed protocols for evaluating sludge settleability and analyzing secondary clarifier performance (Wahlberg, 2001). These protocols are suitable for operational assessment of existing secondary clarifiers and for planning of new facilities.

Primary and secondary clarifiers are an inseparable and integral part of every conventional wastewater treatment plant. Their performance efficiency is affected by the upstream wastewater collection and treatment facilities and has a significant impact on downstream biological treatment and solids handling facilities.

8. Clarifiers and the Wastewater Collection System

Effect of Wastewater Collection System Type on Clarifier Design

Wastewater collection system type has a pronounced effect on the wastewater treatment plant influent. Combined sewer systems are subject to wider flow variations, as compared to separate sanitary sewers. With combined sewer system, wet weather plant influent flow could reach several times the average plant dry weather flowrate. The enforcement of more stringent regulations limiting combined sewer overflows (CSOs) and increased requirements for storm water treatment, would ultimately result in elevated plant influent flows and higher potential for negative effect on clarifier performance. Recent developments of CSO regulations induced the wider use of wastewater storage during wet weather events and real-time control of the CSOs to maximize the use of the sewer storage capacity and minimize the overflows. These CSO control measures typically result in plant clarifiers being subjected more frequently to peak wet-weather conditions and for longer periods of time (Ekama et. al., 1997).

Under wet weather conditions, the increased and diluted influent plant flow agitates the clarifier sludge blanket and intensifies the transient currents in the clarifiers. This impacts both clarifier effluent quality and clarifier sludge density and quality.. Transient flows have negative impact on both primary and secondary clarifiers as well as on the overall secondary treatment process. Cooler storm water deteriorates activated sludge settling characteristics and the overall hydraulic performance of the clarifiers. Prolonged wet weather events may also result in significant washout of grit from the sewer and grit chambers to the primary clarifiers. The sludge volume index in the secondary clarifiers may also be reduced to very low levels.

Another industry-wide trend that has a measurable impact on plant capacity and clarifier performance is the implementation of comprehensive wastewater collection system infiltration and inflow (I&I) reduction programs. Infiltration and inflow could contribute significantly to the plant influent quality and quantity, especially in areas with highly permeable soils, high groundwater tables and old wastewater collection systems. As an effective infiltration and inflow reduction program is implemented, plant influent flowrate would typically decrease between 5 and 25%, which in general would have a positive effect on clarifier performance. However, plant influent wastewater strength is also likely to increase significantly, resulting in increased sludge production and sludge blanket depth in the primary and secondary clarifiers.

Wastewater treatment plant hydraulic design flows used to measure the effect of the type of sanitary sewer on clarifier design are: **daily average flow**; **maximum daily flow**; **peak hourly flow**, and **peak instantaneous flow**. Each is important for different reasons. The peak instantaneous flow is used for the design of the plant influent pumping capacity and for determining clarifier design provisions needed for handling sewer system flow surges during wet weather conditions. The peak instantaneous flow is also considered when selecting sludge blanket depth control strategy in secondary clarifiers during transient flows. Average and peak daily flows are used to determine clarifier average and maximum daily hydraulic and solids loading rates and to select the type, size and configuration of the clarifier sludge collection and withdrawal systems. Peak hourly flow is used to estimate the maximum depth of the clarifier sludge blanket. Peak daily and hourly flows are also used to size plant equalization basin and/or other on-site or off-site wastewater storage equipment.

Traditionally, peak daily flow is estimated by applying a peaking factor to the average daily flow. However, when available, actual flows provide more accurate representation of the plant peaking factors and should be used for determining peak design flows. Computer models based on actual wastewater collection system data and existing flow patterns are recommended to be used for large complex sewer systems to establish key design plant flows. These models typically incorporate key sewer system characteristics such as: tributary area served, rainfall duration and intensity and time of concentrations, location and volume of sewer system retention basins (if any) and existing CSOs, which allow the accurate determination of plant peak instantaneous flow and its effect on clarifier design.

Mitigation of Transient Flow Impact on Clarifier Performance

Transient Flow Reduction Measures in the Wastewater Collection System. The effect of transient loads on the plant clarifier performance can be decreased by a few sewer system peak flow-reducing measures such as:

- implementation of a comprehensive I&I flow reduction program
- more frequent sewer system cleanings and repairs aimed at restoring collection storage system capacity and removing flow obstructions that decrease sewer retention volume
- minimizing industrial wastewater discharger peak flows by enforcing construction of discharge flow and load equalization measures
- enlarging key bottlenecked sections of the wastewater collection system
- providing sewer system retention tanks

Reduction of Transient Flow Impact by Equalization. Plant influent equalization is an effective transient flow reduction measure. Use of equalization basins in facilities with wide variations of diurnal plant influent flow (peaking factor higher than 2.5) would allow a significant decrease in size of the plant primary and secondary clarifiers. Another benefit of flow equalization is improved primary clarifier performance due to the influent pre-aeration in the equalization basin. Reduced peak flows would also allow increasing the mixed liquor suspended solids (MLSS) concentration in the aeration system and at the same time maintain acceptable solids loading of the secondary clarifiers.

For systems where achieving complete nitrification is essential, the increased MLSS concentration would allow an increase in activated sludge system solids retention time (SRT) and a decrease in the food-to-organism ratio, which would facilitate nitrification. Shifting treatment from high-peaking factor periods during the day to off-peak periods would also help reduce plant energy costs.

The positive effect of flow equalization on clarifier capacity has been demonstrated at the Lake Buena Vista, Florida 20 MGD nutrient removal plant (Hubbard et al., 2001). Using off-line equalization basins at this facility enabled the activated sludge basins and secondary clarifiers to treat up to 11.5 MGD (44,000 m^3/day), surpassing the clarifier's design capacity of 9.3 MGD (35,000 m^3/day). Flow equalization maintained the flowrate to the activated sludge system to within 15% of the average daily flow.

Transient Flow Handling Using High-Rate Solids Separation. Performance of primary clarifiers is closely related to their **surface overflow rate** (SOR) (WEF, 1998). In wastewater treatment plants with high wet-weather peaking factors, over-sizing primary clarifiers to handle transient flows could be avoided by using various ballasted flocculation processes. These processes combine addition of coagulant and settling ballast (usually micro-sand) to the primary clarifier influent with installation of inclined tubes (lamellas) in the clarifiers. A portion of the settled sludge or the recovered ballast is recycled to the primary clarifier influent to seed the process.

The addition of ballast increases the density of the floc particles by agglomeration. This results in a three to five-fold increase of the design clarifier surface overflow rate. Typically, conventional clarifiers are designed for SOR of 33 to 49 m^3/m^2.day (800 to 1,200 gal/ ft^2.day). The use of high-rate ballasted solids separation technology allows increasing the design clarifier SOR to at least 160 m^3/m^2.day (4,000 gal/ ft^2.day). Because the ballast enhances solids removal, its use in primary clarification reduces the solids and organic loading of the downstream biological treatment processes.

Currently, there are more than 50 plants worldwide using ballasted floc settling, with the largest units treating peak flow of 500 MGD. A high-solids separation facility can be designed with the built-in flexibility to operate as a primary clarifier during wet weather conditions and as an effluent polishing clarifier for enhanced phosphorus removal during dry-weather flows. Lamella settlers have been employed with and without chemical and ballast addition to handle high-magnitude transient flows and achieve enhanced **total suspended solids** (TSS) and **biological oxidation demand** (BOD) removal.

Transient Flow Handling by Increasing Clarifier Depth. Clarifier depth increase can effectively reduce the negative effect of transient flows on facility performance. Deeper clarifiers provide more room for sludge blanket buildup within the clarifier's thickening zone and protect the clarification zone from sludge blanket incursions. A full-scale primary clarifier performance study completed by Albertson in 1992, concluded that the maximum hydraulic overflow rate that can be processed by the primary clarifiers is proportional to the clarifier sidewater depth. Studies on full-scale circular secondary clarifiers, competed by Parker, 1983 and Voutchkov, 1993, indicate that deeper clarifiers are better suited to accommodate hydraulic surges and maintain desired effluent water quality.

Plants with high wet weather peaking factors (typically above 2.5) are more prone to clarifier sludge blanket washouts and are recommended to be designed with a sidewater depth of at least 4.3 to 5 meters (14 to 16 feet).

In conventional activated sludge plants, under daily average dry weather flow conditions, secondary clarifiers should be designed to maintain a 0.3 to 0.6 meters (1 to 2 feet) deep sludge blanket. For biological nutrient removal (BNR) plant clarifiers, the sludge blanket is not recommended to exceed 0.5 meters (1.5 feet) under average conditions. For municipal plants with separate sanitary and storm drain sewer systems, clarifier blanket depth during transient flows should be allowed to temporarily rise to 1.0 meter (3 feet). For combined sewer systems with wet weather peaking factors higher than 2.5, a transient solids blanket depth allowance of up to 1.8 meters (6 feet) is suggested. In any case, a buffer distance of a minimum of 1 meter (3 feet) should be provided between the sludge blanket level and the clarifier surface to maintain consistent effluent water quality.

Depth is not the only clarifier design variable that can be adjusted to accommodate transient wet-weather flows. The design engineer must consider the tradeoffs between higher clarifier depth and lower surface loading rate (Parker, 1983; Tekippe, 1986; Voutchkov, 1993; and Wahlberg, 2001) as well as the potential advantages of activated sludge system process modifications (i.e., contact stabilization and step feed aeration), to determine the optimum aeration basin-secondary clarifier system design for handling transient flows. A hypothetical example of the potential tradeoffs between clarifier depth and surface overflow rate is depicted on Figure 3.

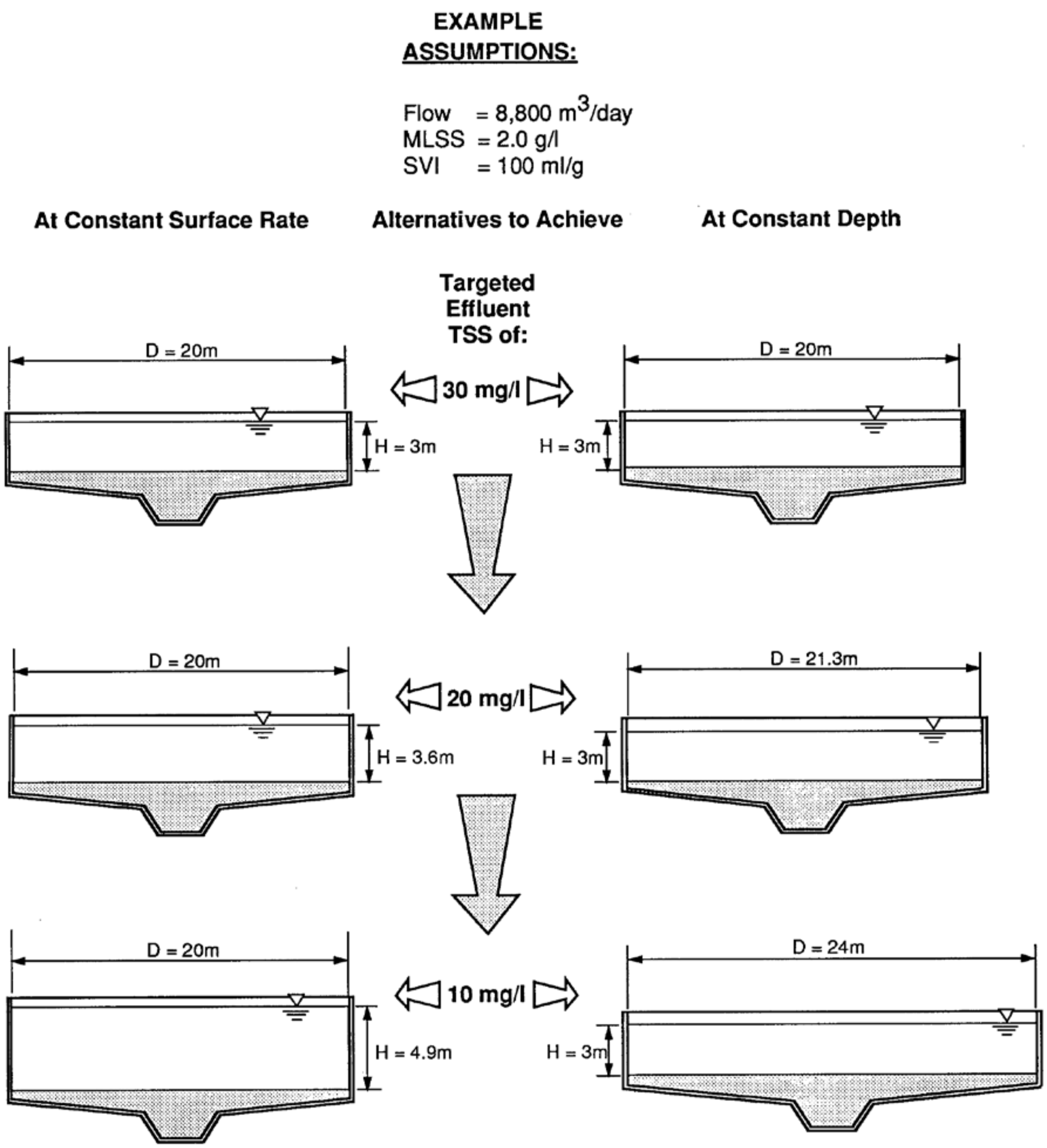


Figure 3 - Example of Tradeoffs Between Clarifier Depth and Surface Overflow Rate

Mitigation of Transient Flow Impact by Reducing Overall Solids Inventory. Transient flows often result in the temporary transfer of significant amounts of activated sludge solids from the aeration basins to the secondary clarifiers. This solids transfer could quickly build a sludge blanket high enough to result in solids carryover and deterioration of clarifier effluent water quality. An alternative to providing deeper clarifiers for handling solids blanket buildup during transient flow events, is to reduce the sludge blanket depth buildup in the clarifiers by decreasing the total amount of solids in the aeration basin – activated sludge system (overall solids inventory). In practical terms, this means designing and operating the aeration basins at lower mixed liquor suspended solids concentration and SRT.

For example, if an activated sludge system operates at 2,500 mg/L and during wet-weather conditions generates a transient sludge blanket of 1.8 meters (6 feet), reduction of MLSS concentration in the aeration basin to 1,500 mg/L and of the overall activated sludge system solids inventory by 40%, would typically result in a reduction of the transient sludge blanket to approximately 1.1 meters (3.6 feet) under similar operational and sludge settleability conditions. In a 2.5 to 3.0-meter (8 to 10-foot) deep clarifier, a transient solids blanket of 1.8 meters (6 feet) is likely to result in deterioration of effluent water quality, while a 1.1-meter (3.6-foot) solids blanket would not significantly affect clarifier effluent quality.

The example above illustrates two alternative approaches for handling transient flows and the associated tradeoff between clarifier depth and reduced solids inventory . Designing activated sludge systems to operate at low solids inventory allows the use of shallower secondary clarifiers to achieve effluent quality that is comparable to that of high solids inventory systems with deeper clarifiers under transient flows.

Operation at lower solids inventory is often the key reason why shallow clarifiers produce effluent quality comparable to deeper clarifiers at similar or sometimes higher surface loading rates. Therefore, when comparing the effect of sidewater depth and surface loading rate on secondary clarifier effluent water quality, activated sludge solids inventory is one of the key parameters that must be taken under consideration. Otherwise, shallow clarifier performance may appear better or sometimes superior to the performance of deeper clarifiers. This may lead to the inaccurate conclusion that higher sidewater depth provides little or no benefit for improving clarifier performance under transient loads.

Another potential benefit of operating an activated sludge system at lower solids inventory (lower MLSS concentration) is an improvement of the clarifier's overall performance. Higher MLSS concentrations typically contribute to formation of density currents in clarifiers and usually result in lower mixed liquor settling velocities (Wahlberg, 1996).

Mitigation of negative transient flow effects on clarifier performance by reducing activated sludge solids inventory is very beneficial when upgrading existing plants with shallow clarifiers and when adequate aeration basin capacity is available to achieve plant secondary treatment goals. This approach, however, may have a limited application for biological nutrient removal plants targeting high levels of nitrogen removal. In this case, maintaining high solids inventory/SRT in the activated sludge system is needed to achieve stable nitrification and consistent effluent water quality.

Transient Flow Control by Increase of Return and Waste Activated Sludge Rates. In secondary clarifiers, the effect on transient flows could also be partially mitigated by increasing the **waste activated sludge** (WAS) removal rate and the **return activated sludge** (RAS) recycle rate. However, increasing the RAS recycle rate is only useful for controlling the effect of relatively short transient events on the clarifiers (4 to 8 hours) and is limited by the capacity of the sludge collection and withdrawal systems. Still this strategy has a limited benefit for long-lasting transient flow conditions. The main reason is that the increased RAS recycle rate only transfers sludge temporarily from the clarifier to the aeration basin and after being retained for a short time in the aeration thanks, the RAS solids return back to the clarifiers. The increased RAS recycle flow will ultimately increase the hydraulic loading of the clarifiers and the sludge blanket will begin to rise again.

RAS flowrate increase has to be gradual and coordinated with the rate of sludge collection. Sudden increases in the RAS recycle rate may result in sludge blanket channeling ("rat-holing"). In addition, abrupt change of the RAS recycle rate may create a hydrodynamic shockwave that may propagate quickly to the clarifier's clear effluent zone and cause excessive turbulence in the clarifiers.

Typically, the recommendation is to limit the RAS recycle rate increase to no more than 50 % of the influent flow during wet weather events, to minimize the potential for disturbing clarifier performance. The optimum design RAS recycle rate and control strategy are most accurately determined using clarifier solids flux state point analysis (Keinath, 1985). Under this state point concept, the design RAS recycle rate is established as the rate at which the clarifier is in a critically loaded condition corresponding to a stable steady state sludge blanket level. The design range of the RAS recycle rate is typically between 25 and 75%. The total RAS pump capacity is recommended to be designed for 120% of the average dry weather flow or 50% of the peak wet weather design flow (whichever is higher).

Installation of variable frequency drives (VFDs) on the RAS pumps may be warranted if the plant is exposed to frequent transient flows of a magnitude exceeding 2.0 to 2.5 times the daily average flow. Separate RAS pumps and flowmeters should be provided for the individual plant clarifiers, rather than using a common suction header and RAS flowmeters for all units. The above-described RAS recycle measure can be combined with the increase in the waste activated sludge withdrawal rate to mitigate transient load effect on the clarifier performance during extended peak flow conditions.

Handling of Transient Flows by Activated Sludge Contact Stabilization. An additional measure for successful control of transient loads is the temporary transfer and storage of some of the activated sludge in the aeration tanks, rather than in the clarifiers, by using a portion of the aeration tank volume as a zone of contact stabilization (sludge reaeration) fed only with RAS. The contact stabilization (sludge reaeration) zone of the aeration tanks is located usually ahead of the main aeration zone. Return activated sludge is added to the tank inlet separately and aerated for a period of time before being blended with the primary effluent, which is introduced directly to the aeration zone.

The solids balance between the aeration zone and contact stabilization zone is controlled by the RAS recycle rate. As the RAS recycle rate is increased, a greater portion of the activated sludge solids is transferred from the clarifier blanket to the contact stabilization zone of the aeration basin. These solids will be retained in the contact stabilization zone for a certain period of time (typically, 4 to 6 hours) effectively allowing the clarifier sludge blanket depth to reduced..

Taking under consideration that the clarifier sludge solids originated in the aeration zone of the aeration basins, increasing the RAS recycle rate will also decrease the amount of MLSS in the aeration zone of the basins, i.e. the higher rate of return will shift solids from the aeration to the contact stabilization zone of the aeration basins as well. Therefore, as the RAS recycle rate is increased, the detention time of the MLSS in the aeration zone is lowered. This solids inventory shift will proportionally increase the food-to-biomass ratio in the aeration zone, which may have a negative effect on aeration basin BOD removal and nitrification, as well as on sludge setleability.
If the RAS recycle rate is increased to such an extent that aerobic zone MLSS and contact time are reduced significantly (to gain a rapid reduction of clarifier sludge blanket depth), the increased food-to-biomass ratio may result in deterioration of sludge settleability, negating the positive effect of this control measure on clarifier performance. Therefore, use of contact

stabilization coupled with increased RAS recycle rate for transient flow control has to be optimized against aeration zone contact time, secondary effluent BOD and nitrogen water quality, and sludge settleability.

The conversion of a conventional activated sludge system to a contact stabilization system has been successfully implemented at the Camp Creek Water Pollution Control Plant in Fulton County, Georgia (Danco et al., 1994). At this facility, which has two aeration tanks and four shallow circular secondary clarifiers, one aeration tank has been converted to an aeration zone and the other to a contact stabilization zone. Conversion to contact stabilization has improved facility nitrogen removal, minimized floating sludge problems in the clarifiers and simplified activated sludge process control. Prior to the conversion, secondary clarifier performance had been affected by frequent solids overloading and sludge blanket denitirification.

Handling of Transient Flows by Step-Feed Aeration. Step feed aeration basin configuration allows influent flow feed at two or more locations along the length of the aeration basin. Under this configuration, the entire RAS flow is recycled to the inlet of the aeration basin. MLSS concentration decreases along the length of the basin as each of the influent entries dilutes the mixed liquor. By directing much of the solids load to the inlet end of the aeration basin and diluting MLSS towards the outlet, the clarifier solids loading is reduced and the sludge blanket level is controlled at transient flow conditions. In effect, the step-feed configuration allows the shifting of solids inventory from the clarifier to the front end of the aeration basin, thereby reducing clarifier solids flux.

Mitigation of Transient Flow Effects by Aeration Basin Adjustable Effluent Weirs. Installing adjustable effluent weirs on the aeration basins, coupled with providing extended aeration basin freeboard, can further reduce the transient flow effect on the secondary clarifiers. When a flow surge occurs, the adjustable weirs are elevated. The additional aeration basin volume retains some of the excessive flow in the aeration tanks and dampens the transient effect on the secondary clarifiers. This approach is typically applied to aeration basins with diffused bubble aeration and may have limited use for those with surface aerators. In addition to the extra costs for constructing deeper aeration tanks, this approach for reducing transient flow impact on the secondary clarifiers may result in excessive activated sludge flock breakup due to elevated effluent weir drop.

A potent measure for mitigating the flock breakup effect in adjustable aeration basin effluent weirs is the addition of a small dosage (0.5 to 1.5 mg/L) of cationic polymer to the secondary clarifier feed. Adding polymer usually strengthens the flock structure and at the same time improves clarifier effluent quality.

Mitigation of Transient Flows by Temporary Shutdown of Aeration. A measure that could be used as a last resort in controlling clarifier blanket depth and preventing solids carryover with the final effluent is to shut off the aeration, the internal recycle, and the mixing equipment in the activated sludge basins. This will immediately prevent additional solids from reaching the clarifiers and will allow the biomass that has been conveyed to the clarifiers to be returned back to the aeration basins (Randall et all, 1992). This measure, however, is typically applicable only to aeration basins equipped with mechanical aerators or coarse-bubble diffusers.

Plants using fine-bubble diffuser systems for activated sludge tank aeration may implement this transient flow mitigation approach only for a very short period of time (typically not more than 30 minutes) without exposing the aeration diffusers to significant fouling. If the aeration system type is not a constraint, this mode of operation can be used for 3 to 4 hours without significant negative impact on plant effluent quality.

Hydrodynamic modeling considers the effect of wide influent water quality and quantity fluctuations during wet weather events and identifies the most efficient and cost-effective combination of design and control measures to handle wet weather conditions, and produce flow of target water quality.

9. Clarifiers and Pretreatment Facilities

Effect of Plant Influent Pumps Station Design on Clarifier Performance

Plant influent pump size, configuration, and type of motor controls have a significant effect on clarifier performance. Wide and sudden changes in plant influent flowrate typically create hydraulic transients that degrade clarifier effluent quality and the overall clarifier performance (Collins and Crosby 1980; Maskell and Lumbers, 1974; and Porta et al.,1980). Therefore, frequent and abrupt starts and stops of large influent pumps as well as direct pumping into the clarifier units must be avoided.

Installation of variable speed drives on the plant influent pumps will mitigate abrupt changes in clarifier influent flowrate and hydraulic loading. Use of screw pumps is recommended, if feasible, due to the intake configuration and the mode of operation that dampen plant influent flow variations.

Effect of Screening Facilities on Clarifier Performance

Plant influent wastewater contains a variety of large suspended or floating materials that must be removed to protect the structural integrity and treatment performance of the downstream treatment facilities. The type and performance of the screening pretreatment facilities have a measurable impact on the performance of the primary clarifiers, and to a lesser extent, on the secondary clarifiers. There are two different types of screens: fine and coarse screens that retain and remove large solid materials from the influent wastewater, and grinders that reduce the size of the influent debris to smaller settleable particles, leaving the ground materials in the influent for further removal in the primary clarifiers.

The most widely used mechanically cleaned screens have bar openings between 6 and 38 mm (0.25 to 1.5 inches). The amount of screenings removed at the mechanically cleaned screens is typically in a range of 3.5 to 80 m^3/million m^3 of treated wastewater (0.5 to 11 ft^3/million gallons) and averages 20 m^3/million m^3 of treated wastewater (2.7 ft^3/million gallons), (Qasim, 1985). The screenings usually contain 10 to 20% solids and weigh between 600 and 1,100 kg/ m^3 (40 to 70 lbs/ ft^3) and typically average 960 kg/m^3 (60 lbs/ft^3).

Comminuting devices (grinders) are sometimes installed in the plant influent channel to screen and reduce material sizes to between 6 t-19 mm (0.25 to 0.75 inches). These devices are intended to reduce odors, flies, and cumbersome operations related to screenings removal, handling and disposal.

If grinders are used and the screenings are left in the plant influent flow, they would contribute an additional 3 mg/L to 77 mg/L (average of 20 mg/L) of total suspended solids to the design plant influent TSS concentration. This would result in an average increase of 5 to 10% in the primary sludge quantity. This percentage could be several times higher during wet weather periods. In addition, peak daily screening quantities may vary considerably from average conditions (as much 20:1 on a hourly basis). The sludge increase resulting from the use of grinders is measurable and has to be reflected in the design of the primary clarifier sludge collection and withdrawal equipment. This sludge increase also has to be taken under consideration in the solids handling facility design.

If left in the influent flow, most of the settleable screenings would be removed in the primary clarifiers. However, some of the screenings could reach the aeration basins where they may aggregate and increase in size due to the vigorous aeration in the basins, and subsequently may clog secondary clarifier sludge collection orifices if the clarifiers are equipped with suction sludge collection systems. Therefore, if grinders are installed as screening facilities, the use of clarifier suction sludge collection systems is not recommended. If grinders are the only viable screening process for a given application, the design of the clarifier sludge suction system and the influent grinding system have to be carefully coordinated to avoid clogging of the suction system orifices and pipes. Screenings left in the plant influent may also pose settling lamella tube clogging problems if lamella blocks are installed for enhanced settling.

Effect of Grit Removal System Type and Design on Clarifier Performance

The main purpose of primary clarifiers is to remove mostly fine organic suspended solids settleable by gravity. Plant influent contains a relatively large amount of coarse inorganic solids such as sand, cinders and gravel that are called grit. Grit must be removed upstream of the primary clarifiers and in to grit chambers to protect treatment plant equipment from excessive wear and abrasion and to prevent obstruction of channels and pipes with heavy deposits that reduce their conveyance capacity. Additionally, this will prevent cementing effects at the bottom of the primary clarifiers and digesters and reduce the amount of inert materials in the solids handling facilities. Grit chambers are typically designed to remove particles of specific gravity of 2.5 and retained over a 65-mesh screen.

The grit quantity and quality are important factors that need to be taken under consideration in designing primary clarifiers. The quantity of grit removed in grit chambers varies significantly, depending on the type and condition of the wastewater collection system, proximity to the sea/beach areas; and type of industrial waste dischargers. Grit amount typically ranges between 5 and 200 m^3/million m^3 of treated wastewater (0.7 to 28 ft^3/million gallons) and averages 30 m^3/million m^3 of treated wastewater (4 ft^3/million gallons) (Qasim, 1985). The grit usually contains 35 to 80 % solids and has a specific weight in the range of 400 to 1,800 kg/m^3 (90 to 110 lbs/ ft^3).

If grit chambers do not operate adequately, the excessive amount of grit left in the primary influent may cause an overload of the clarifier sludge collection equipment, may increase the amount of primary sludge and may have a negative impact on the facilities and equipment for handling primary sludge. This excessive grit carryover may increase primary sludge solids quantity by 10 to 30 %. If the primary sludge contains a large amount of grit, sludge de-gritting prior to conveyance of the primary sludge to the solids handling facilities is warranted.

De-gritting devices (hydro cyclones and centrifuges) separate grit from the organic materials in the primary sludge and provide a beneficial effect on downstream solids handling facilities.

Sludge de-gritting is usually recommended as an improvement measure in existing plants with poorly performing grit chambers. In new plants, the grit chamber design has to be focused on effectively removing grit before it reaches the primary clarifiers rather than on providing equipment for de-gritting of the primary sludge.

Aerated grit chambers have a positive effect on the primary clarification process by reducing the potential for primary clarifier sludge septicity. Uncontrolled sludge septicity usually impacts the overall clarifier performance. In addition, plant influent aeration ahead of the primary clarification reduces hydrogen sulfite concentration of the raw wastewater, and thereby diminishes the rate of corrosion of clarifier equipment and structure.

Ordinarily, the aerated grit chambers are designed for a hydraulic retention time of 2 to 5 minutes. However, if the aerated grit chambers are used for pre-aeration/septicity control or to remove fine grit, the retention time should be increased to 10 to 20 minutes. In addition, installation of a coarse bubble aeration system in the channels connecting the grit chamber and the clarifiers is recommended. All aerated channels have to be covered and ventilated for odor and corrosion control. In case the plant influent contains a significant amount of oil and grease, aerated grit removal reduces the amount of floatables reaching the primary clarifiers. Aerated grit chambers can also be used for chemical addition, mixing and flocculation ahead of the primary clarifiers.

10. Clarifiers and Biological Wastewater Treatment

Effect of Primary Clarification on Nutrient Removal in Activated Sludge Systems

The plant influent organic substrate-to-nutrient ratio is a fundamental factor affecting the performance of the biological wastewater treatment systems. Usually, this ratio is measured as biological oxygen demand-to-nitrogen-to-phosphorus ratio (BOD:N:P). Standard conventional biological removal systems require a BOD:N:P ratio of 100:5:1.

Primary clarification reduces the organic substrate-to-phosphorus ratio in the plant influent, thereby reducing the amount of phosphorus and nitrogen that can potentially be removed in the conventional biological treatment process (WEF, 1998).Normally, primary clarifiers remove a higher percentage of organic materials (BOD and COD) than they do nutrients (nitrogen and phosphorus).

In industrial plants where influent BOD:N:P ratio could be unbalanced; primary clarification may further negatively impact activated sludge system BOD removal efficiency due to an inadequate amount of nutrients in the wastewater. Under such conditions, additional sources of soluble nitrogen and phosphorus may need to be added to the primary effluent to compensate for substrate-to-nutrient reduction in the primary clarifiers. This effect of primary clarifiers on the organic substrate-to-nutrients ratio has to be taken into consideration when designing activated sludge systems.

Because of the negative effect of the primary clarifiers on the substrate-to-phosphorus ratio, some BNR plants have been designed without primary clarification (Randall et al., 1992). Due to the significantly higher secondary sludge production (50 to 70%) without primary clarification, the aeration basins of the BNR systems have to be increased in size. The elimination of the primary clarifiers also produces sludge that, overall, is more difficult to handle. Therefore, primary clarification is a recommended treatment process ahead of biological nutrient removal systems.

Use of Primary Clarifiers for Chemical Phosphorus Removal

Phosphorus removal by addition of chemicals to primary clarifier influent is easy to implement and simple to operate. Chemicals (typically iron or aluminum salts) are added upstream of the clarifier in locations providing conditions for good mixing with the plant influent. The influent phosphorus reacts with the metal salt forming phosphate precipitate, which is removed as sludge in the primary clarifiers.

Chemical addition in primary clarifiers allows removal of up to 90% of the particulate phosphorus in the plant influent. Chemical clarification processes such as contact clarifiers, sludge blanket clarifiers, and clarions have been successfully used for chemical phosphorus removal.

A key disadvantage of chemical phosphorus precipitation is the significant amounts of sludge produced and the resulting increase in solids handling and disposal costs. Typically, 2.9 milligrams of solids are produced per milligram of aluminum and 1.9 milligrams of solids are generated per mg of Fe if iron salts are applied (WEF, 1998).

Aluminum is more efficient than iron in terms of the amount of metal needed to remove one pound of phosphorus. Theoretically, precipitating one milligram of phosphorus requires 1.8 milligram of iron and only 0.87 milligrams of aluminum. Therefore, the total amount of solids generated from removal of one milligram of phosphorus using aluminum salts is only slightly (10 to 15 %) higher than that produced by iron salt precipitation.

In addition, the chemical phosphorus precipitation process consumes a significant amount of plant influent alkalinity (5.8 mg as $CaCO_3$/mg Al and 2.7 mg as $CaCO_3$/mg Fe). Use of plant alkalinity upstream of the biological nutrient removal system typically has a negative effect on the nitrogen removal efficiency of the system, since a significant amount of alkalinity is needed for wastewater nitrification.

Usually, if total phosphorus is reduced below 2 mg/L in the primary clarifiers, the amount of phosphorus in the primary effluent may be insufficient to provide the nutrients needed for adequate biomass growth in the activated sludge system A nitrogen removal study at the Washington, D.C. Blue Plains wastewater treatment plant (Bailey et. al, 1997) where iron salts were added to the primary and secondary treatment processes for enhanced phosphorus removal, indicates that enhanced primary clarifier phosphorus removal can result in inadequate soluble phosphorus concentrations available for the denitrifying microorganisms. This deficiency at the Blue Plains wastewater treatment plant resulted in reduced biological nitrogen removal, erratic denitrification rates, filament growth, increased sludge yields, and inefficient use of methanol.

The type of chemical coagulant used for phosphorus precipitation should be carefully selected since it may have a significant effect on some of the downstream treatment facilities. For example, iron compounds, unlike those containing aluminum and calcium, can also effectively control septic odors. However, if the treatment plant has a UV disinfection system, overdosing iron salts can foul the UV tubes and measurably reduce their disinfection efficiency. Residual iron also interferes with the disinfection process because iron absorbs the UV portion of the spectrum.

Aluminum salts produce precipitates that do not re-dissolve under anaerobic conditions, unlike iron phosphates.. This is a key consideration in wastewater treatment plants with anaerobic digesters. Use of aluminum salts, although producing slightly higher amount of solids, would typically minimize phosphates release in the anaerobic digesters and related solids handling sidestreams.

Use of chemically enhanced primary clarification may also impact the final sludge quality and its disposal options. Along with phosphorus, coagulants also precipitate heavy metals from the plant influent, thereby increasing the metal content in the plant sludge.

Aluminum and iron salts used for precipitation would also contribute to the increased content of heavy metals in the sludge due to trace amounts of metal impurities in their composition. . If the plant sludge is planned to be beneficially used, then the effect of chemical precipitation on the final sludge quality must be carefully assessed for compliance with applicable regulatory requirements.

The costs for chemical and biological phosphorus removal are affected significantly by the plant influent BOD_5:TP ratio and the target level of effluent phosphorus concentration. Chemical phosphorus removal becomes less cost effective as the BOD_5:TP ratio decreases and effluent phosphorus target level decreases. The use of biological phosphorus removal is favored when the incremental sludge handling and disposal costs are relatively high.

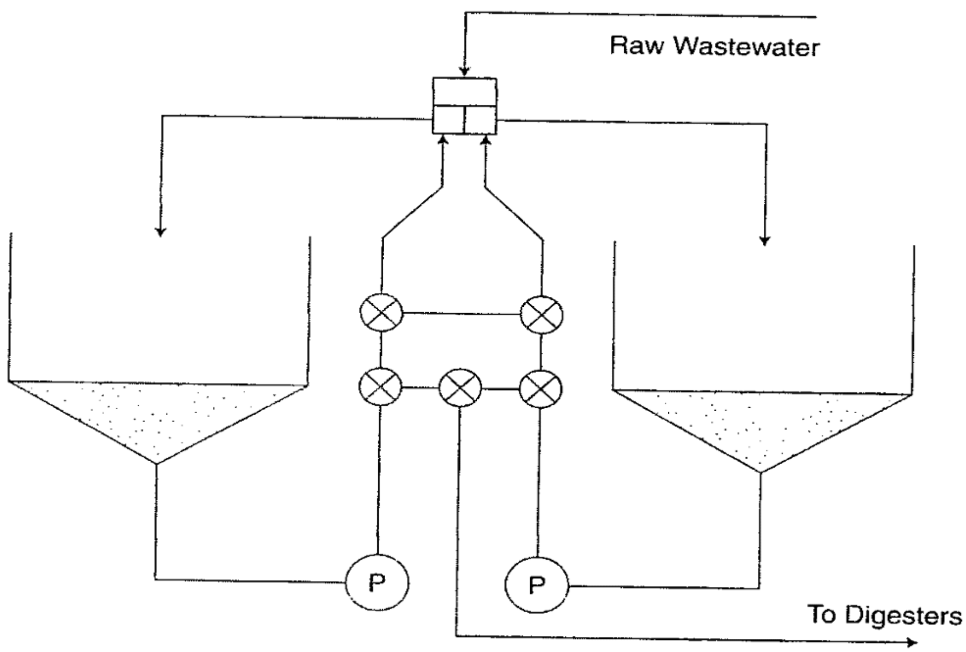
Use of Primary Clarifiers for Solids Pre-fermentation

Volatile fatty acids (VFAs) play a key role in the metabolism of bacteria, such as *Acinetobacter*, that are capable of enhanced phosphorus removal in the biological nutrient removal systems. The accumulation of VFAs gives the phosphorus removal organisms a competitive edge for growth and survival in the activated sludge system.

Typically, VFAs are created during the natural fermentation process occurring in the wastewater collection system upstream of the wastewater treatment plants. This phenomenon has been observed at several plants in the US and is typical for wastewater plants in tidewater areas where the sewers are long and the slopes are small (WEF, 1998). However, VFA generation in the sewers varies with temperature and may be quite low in the winter. Therefore, to provide optimum conditions for enhanced biological phosphorus removal in the downstream anaerobic zones in the BNR system, additional VFA can be generated by pre-fermentation in the primary clarifiers.

The fundamentals of design of primary clarifiers and other pre-fermentation facilities for enhanced biological phosphorus removal have been described in detail by Barnard, 1984 and Randall et al., 1992. The primary clarifiers can be used to ferment organic carbon, available in the plant influent, to generate short-chain VFAs.

Ideally, acid fermentation would provide enough VFAs to remove phosphates biologically to levels below 0.1 mg/L as phosphorus, if the BNR system is followed by tertiary filtration (WEF, 1998). This is achieved by operating the primary clarifier to carry a sludge blanket and by slowly recycling this sludge to the clarifier inlet. Figure 4 illustrates the concept of an "activated primary sedimentation tank" operated to maximize VFAs production.



Source: Barnard, 1984

Figure 4 - Arrangement of Two Activated Primary Tanks

The constant recycling of sludge seeds the incoming clarifier influent with fermenters, elutriates the VFAs from the sludge blanket, and prevents the formation of methane and hydrogen sulfide through the constant exposure to air with every recycle. Because the sludge recycle leads to a slow buildup of methane organisms, the primary sludge has to be completely removed from the clarifiers regularly. The frequency of complete clarifier blanket removal is site-specific and varies seasonally.

There are a few possible operational scenarios for sludge recycle when using two primary clarifiers as shown on Figure 4.. The sludge can be separately recycled back to the influent of each tank, where the sludge withdrawal pumps are connected directly to the underflow of each tank and the sludge lines are interconnected using two-way valves. By providing the operational flexibility indicated on Figure 4, the underflow from one of the tanks can be pumped to the other, while the underflow of the second tank is pumped to the digesters. This configuration maintains a continuous fermentation process in the clarifiers, while completely removing the sludge out of one of them.

The key disadvantage of using primary clarifiers as pre-fermenters is that the recycled primary sludge increases the organic and solids load of the primary clarifiers and thereby reduces the available clarifier capacity. Another problem experienced with primary clarifiers as pre-fermenters is the additional load on the scraper mechanism caused by the high sludge blanket required for this process. Therefore, in existing plants, the size and capacity of the sludge collection mechanisms must be carefully assessed to confirm that modifying a primary clarifier to a pre-fermenter is viable. In new clarifiers, the sludge collection mechanisms must be carefully selected to maintain a 2 to 3 feet of sludge blanket.

Secondary Clarifier Design for Enhanced Nutrient Removal

Secondary clarifier design is paramount to the successful operation of biological nutrient removal systems. The clarifiers must be designed to produce an effluent TSS concentration below 10 mg/L to effectively reduce total phosphorus to below 2.0 mg/L (Morales et al., 1991; Voutchkov, 1992). This usually requires secondary clarifiers to be designed for relatively conservative surface loading rates within the range of 0.5 to 1.0 m^3/m^3.hr (300 to 600 gpd/sq. ft), Sedlak, 1991.

Resolubilization of phosphorus in the sludge blanket and subsequent phosphorus release with the final effluent is a problem that typically occurs in shallow clarifiers that carry relatively deep sludge blankets. The effect of phosphorus resolubilisation can be reduced by increasing clarifier sidewater depth such that the clarifier can be operated with minimal upflow velocity through the sludge blanket.

It is recommended to design the clarifiers with a sidewater depth in a range of 4.3 to 5.5 meters (13 to 16.5 feet) to prevent significant upflow through the sludge blanket. The clarifier upflow and phosphorus elutriation can further be minimized by increasing the RAS recycle rate and sludge waste rate of the clarifier. The need to minimize upflow through the clarifier sludge blanket renders the use of rimflow-type clarifier undesirable when the treatment plant includes BNR facility targeting production of effluent with low phosphorus concentration.

BNR systems are susceptible to induced growth of filamentous and scum-producing organisms (Sedlak, 1991). Therefore, the secondary clarifiers are recommended to be designed with scum collection and removal mechanisms. The collected scum should be conveyed to the solids handling systems.

In addition, the BNR clarifiers should be designed with provisions to handle bulking sludge. Anaerobic and anoxic selectors have a positive effect on the sludge settling characteristics and will effectively control excessive filamentous organism growth.

Biological phosphorus removal can be further enhanced by adding phosphorus precipitating salts (alum, ferric sulfate, ferric chloride, etc.) to the activated sludge. The phosphorus polishing chemical can be added at several locations within the activated sludge system. Typically, coagulant is recommended to be added to the secondary clarifier influent because it reduces interference with the BNR process in terms of alkalinity consumption. At this stage of treatment, phosphorus is predominantly in the form of orthophosphates, which can be precipitated and settled in the clarifiers.

The most suitable points of chemical addition are locations where flash mixing can be achieved effectively. These include clarifier splitter boxes, flocculation wells (if such are provided), aerated distribution channels, etc. Coagulant addition can also partially compensate for activated sludge flock sharing due to excessive mixing and turbulence in the aeration basins.

Metal salts addition increases the non-volatile portion of the activated sludge system. Therefore, higher MLSS concentration must be maintained in the aeration basins to provide the same amount of active biomass. Increased amounts of solids in the activated sludge system would require the clarifiers to be designed to maintain higher sludge blanket depth and would require the installation of larger RAS and WAS pumps.

Biological nutrient removal systems that incorporate anaerobic and anoxic selectors have a positive effect on sludge settling characteristics, thereby allowing an effective control of excessive filamentous organism growth. Availability of these systems results in improved secondary clarifier performance.

The creation of anaerobic conditions in the clarifier blanket causes denitrification in the activated sludge secondary clarifiers and leads to the uncontrolled floatation of solids. This is due to the gaseous nitrogen that is produced during the denitrification process. The nitrogen becomes trapped within the activated sludge flocks and causes solids to float to the clarifier surface. In addition, anaerobic conditions in the secondary clarifier sludge blanket may result in the release of soluble phosphorus from the biomass in the clarifier, which would lead to an increase in the plant's effluent phosphorus concentration. This unwanted condition could be prevented by limiting the amount of nitrates entering the clarifier, by maintaining aerobic conditions in the secondary clarifier, and by minimizing the time the sludge is retained in the clarifier. For plants targeting enhanced phosphorus removal only, the amount of nitrates entering the secondary clarifier can be reduced by designing the activated sludge system to operate at low SRT (typically less than 6 days), thereby minimizing the presence of nitrifiers in the activated sludge biomass and eliminating nitrification. For BNR plants that have to comply with both nitrogen and phosphorus removal requirements, the amount of nitrates entering the clarifier can be achieved by denitrification in the anoxic zones of the activated sludge system.

Maintaining aeration basin effluent dissolved oxygen concentration in a constant range of 2.5 to 3 mg/L, along with operating at RAS recycle rate above 50 % and maintaining a clarifier sludge blanket below 0.5 meters (1.5 feet) are effective measures to prevent denitrification and phosphorus release in the secondary clarifiers.

Typically, high solids blankets tend to deteriorate effluent water quality in BNR plants. Therefore, BNR system secondary clarifiers are recommended to be designed with sludge collection and withdrawal systems that have adequate capacity to remove the sludge in a relatively short time and to maintain the sludge blanket level between 0.35 and 0.5 meters (1.0 and 1.5 feet). The sludge blanket level, however, should not be allowed to drop below 0.20 meters (0.6 feet) during minimum plant flow conditions or channeling may occur in the sludge blanket, resulting in low RAS concentrations. To accommodate the sludge blanket control measures suggested above, secondary clarifiers of BNR plants with significant diurnal flowrate fluctuations are recommended to be provided with two-speed or variable speed controls of the sludge collection mechanism drive.

The solids retention time of the clarifier sludge blanket is another criterion that can be used to determine the following:

- acceptable clarifier sludge blanket depth
- the capacity of the clarifier sludge collection
- withdrawal systems in BNR plants

. This parameter is calculated by dividing the mass of solids in the clarifier blanket by the rate of withdrawal of RAS and WAS solids from the clarifier. In general, sludge blanket retention time in BNR clarifiers, estimated for daily average conditions, should not exceed 2 to 3 hours to avoid potential denitrification in the sludge blanket.

Optimization of Clarifier – Aeration Basin System

It is well established that aeration basin and secondary clarifier size are inter-related and their design can be optimized to achieve a minimum life-cycle cost for the entire activated sludge system. The optimization of the aeration basin secondary clarifier system focuses on selection of the most cost-effective design mixed liquor suspended solids concentration in the aeration basins. Typically, there is an optimum design MLSS that will allow minimized total capital costs for the aeration basins and the clarifiers, when the clarifier capacity is **thickening limited** (van Haandel, 1992). An activated sludge secondary clarifier is thickening limited under the following conditions:

- whenever the MLSS concentration exceeds some minimum value
- when the MLSS concentration is less than the concentration at the minimum of the solids flux curve
- when the RAS concentration is greater than the critical concentration.

It should be noted that the optimum MLSS concentration typically increases with e a gain in the system solids retention time. Since biological nutrient removal systems usually operate at higher SRTs, the optimum design MLSS concentration for these systems is sometimes higher than that of conventional activated sludge systems. Another important factor for determining the optimum MLSS concentration is the activated sludge settling characteristics. As sludge settling improves, the optimum MLSS increases and the overall cost of the activated sludge system is reduced.

11. Interaction with Solids Handling Facilities

Clarifiers and Sludge Thickening

The sludge generated during the sedimentation process is initially thickened in the primary and secondary clarifiers. The extent of sludge thickening that can be achieved in the clarifiers depends on numerous factors, such as:

- plant influent wastewater quality and quantity
- the capability of the clarifiers to carry sludge blanket
- the type and capacity of the sludge collection and withdrawal system
- sludge settleablity
- sludge septicity in the primary clarifiers
- the type of biological treatment process in the activated sludge system

Thickening in Primary Clarifiers. A non-septic plant influent is found in a short sewer collection system with septicity/odor control provisions, operating in an area of cold to moderate climate. Ideally, if the plant influent is not septic the grit removal facilities will operate well. In addition, the primary clarifiers will have adequate depth to carry 2 to 3 feet of sludge blanket and the sludge collection and withdrawal systems will be adequately sized and automated to maximize sludge concentration. Thus, the primary sludge in the clarifiers can be consistently thickened to a level of 3 to 5% solids.

The primary sludge concentration is considered optimum from a sludge collection and conveyance point of view. . Thicker sludge would be difficult to remove from the clarifiers and convey to the solids handling facilities. If the primary clarifiers are designed to perform both sedimentation and thickening functions, then further downstream thickening facilities are not required.

If the plant influent and primary sludge are prone to septicity, the plant experiences frequent transient flows, dry weather daily peaking flow, and/or TSS load factors are consistently above 2. If the primary clarifiers have to be built relatively shallow due to site-specific constraints, the primary sludge will not settle well and the primary clarifiers will have to be designed to only perform sedimentation function and to continuously withdraw sludge. The primary sludge concentration typically varies between 0.5 and 1.5% in cases where the primary clarifiers cannot carry sludge blanket and sludge withdrawal is continuous.. This sludge contains a significant amount of water and has to be thickened further for cost-effective and efficient solids handling.

Successful design of primary clarifiers for maximized thickening has been reported for the City of Memphis, TN 302,800 m^3/day (80-MGD) Maxson wastewater treatment plant (Collins et. al., 1999). The new 60-meter (180-foot) diameter plant primary clarifier raised primary sludge solids concentrations from 4 to 7% solids.

Thickening in Secondary Clarifiers. The level of activated sludge thickening that can be achieved in secondary clarifiers depends on several variables, most of which are related to the type of activated sludge system and the biological treatment processes. These factors have a direct impact on the sludge settleablity, compressibility, and side effects impacting the clarification process. Such side effects include:

- the occurrence of uncontrolled denitirifaction in the sludge blanket
- filamentous growth
- sludge bulking
- pin flock, etc.

In addition, factors related to clarifier configuration, hydraulics and sludge current distribution have a significant effect on the waste activated sludge concentration.

Typically, the TSS concentration of the activated sludge wasted from the secondary clarifiers ranges between 4,000 and 8,000 mg/l (0.4 to 0.8% solids). Under best case-settling and activated sludge system performance conditions, the WAS concentration may reach 10,000 to 14,000 mg/L (1 to 1.4% solids). The WAS, even under best-case clarifier thickening scenario, contains a very large quantity of water that has to be reduced before further processing in the downstream solids handling facilities.

Co-thickening of Primary and Secondary Sludge in Primary Clarifiers. Co-thickening of primary and secondary sludge in primary clarifiers includes conveyance of the secondary sludge to the primary clarifiers, blending with plant influent and co-settling this sludge with the plant influent suspended solids. Several key benefits of the co-thickening in the primary clarifier are: enhanced primary sedimentation caused by the flocculating effect of the secondary sludge on the influent suspended solids; cost reduction due to elimination of secondary sludge thickening facility; and simplified solids handling operations.

Successful co-thickening of primary sludge and trickling filter secondary sludge has been reported at a number of wastewater treatment facilities (Kemp and MacBride, 1990). In these plants, the primary clarifiers were designed for relatively low loading rates, and were equipped with sludge collection and withdrawal systems allowing relatively rapid sludge removal. Rapid sludge removal prevented an increase in primary clarifiers effluent soluble BOD caused by the biological activity in the clarifier sludge blanket. The co-thickened concentration was in a range of 2 to 3.5% solids. At present, co-thickening of primary and WAS in the primary clarifiers is not practiced widely, because past full-scale experience shows that this approach had a detrimental effect on the overall primary clarifier performance (WEF MOP 8, 1998).

Key disadvantages of co-thickening of primary sludge and WAS in the primary clarifiers are: elevated soluble BOD concentration of the primary effluent; reduction of the clarifier treatment capacity by 40 to 50 %; and production of primary sludge of 1 to 3 % lower solids concentration.

Sludge Thickening Facilities. Usually, additional post-clarification sludge thickening is applied to minimize the volume of the solids handling facilities. Thicker sludge requires smaller piping and pumping equipment to convey, and chemicals and digester capacity to stabilize. Commonly used methods for sludge thickening are: gravity thickening, dissolved air flotation, and mechanical thickening (centrifugation and belt filter press thickening).

Gravity thickening is usually accomplished in circular sedimentation basins, similar to those used for primary or secondary clarification. Gravity thickening is used to concentrate low-solids primary sludge, trickling filter sludge, and activated sludge. Thickeners are also used for combined and chemical sludge. The level of thickening achieved by gravity is typically 2 to 5 times the concentration of the feed sludge. The gravity thickeners are most suitable for low-solids primary and trickling filter sludge. Waste activated sludge and chemical sludge are difficult to dewater by gravity thickening. The most cost effective method to concentrate this sludge is by dissolved air filtration and mechanical thickening.

Primary clarifier performance has a significant impact on the downstream thickening facilities. Septic primary sludge usually thickens at a lower rate and requires special provisions for thickener gas release and odor control. Activated sludge age and settleability impact the size of the dissolved air flotation thickeners, the mechanical thickening equipment, and the amount of chemicals needed to condition the sludge prior to thickening. The lower the concentration of the primary and secondary sludge, the proportionally higher the volume of the thickening facility would need to be.

Clarifiers and Anaerobic Sludge Digestion

Effect of Clarifier Performance on Digester Operation. Performance of primary and secondary clarifiers has a significant effect on the plant's anaerobic digestion process. This process is very sensitive to changes in sludge volatile organic content, quantity, and concentration. Therefore, primary and secondary sludge removal frequency, quantity, and quality should be closely monitored and controlled to avoid digester process upsets and failures.

If the treatment plant is prone to frequent transient loads and significant daily variations of influent water quality and quantity, construction of sludge storage tanks ahead of the anaerobic digesters is recommended to dampen daily fluctuations of sludge quantity and quality and to provide a homogenous feed to the anaerobic digesters.

To optimize digester performance, primary sludge concentration is recommended to be maintained in the range of 3 to 4% solids. Lower concentrations would result in conveying an unacceptably high amount of water to the anaerobic digesters and would affect the acid formers to methane formers ratio. Ultimately, this would result in destabilization of the anaerobic digestion process. Therefore, primary sludge concentration of 1% of solids or less requires thickening prior to digestion.

Primary sludge concentrations higher than 6% of solids are achievable. However, sludge at this concentration is difficult to pump and is likely to have a negative impact on clarifier performance due to septicity.

Primary sludge contains more readily biodegradable organic compounds than secondary sludge and therefore, yields a higher volatile suspended solids removal rate and higher digester gas production rate. Digester foaming problems also tend to occur less frequently and are less severe when digesting primary sludge.

Currently, it is a common practice to combine primary and secondary sludge for anaerobic digestion. In this case, it is most desirable to maximize the influent TSS and BOD removal in the primary clarifiers and to minimize the amount of activated sludge production.

Digester Hydrogen Sulfite Control by Chemical Addition to Primary Clarifiers. Adding oxidants such as ferrous chloride, ferric chloride, ferric sulfate or chlorine could be used effectively to control the content of hydrogen sulfide in the digester gas. Typically, hydrogen sulfide emissions from digesters are limited by pertinent air quality management regulations. Without the addition of oxidizing chemical to the primary clarifiers, anaerobic digesters typically generate 1,000 to 4,000 mg/L of hydrogen sulfide.

Chemically enhanced primary clarification typically allows reducing the hydrogen sulfide concentration to below 40 mg/L. If both ferric chloride and chlorine are used for hydrogen sulfide control, chlorine must be added upstream of the point of ferric chloride addition. Usually, the primary sludge would float if ferric chloride and chlorine are added at the same point due to the formation of iron sulfide, which forms black fine particles that are difficult to settle. Ferrous chloride is more effective in hydrogen sulfide control in digesters than ferric chloride. However, ferric chloride more effectively removes suspended solids and phosphorus from the wastewater. Use of aluminum sulfate for hydrogen sulfide control is not as effective as the application of iron salts.

The effectiveness of hydrogen sulfide control by addition of iron salts depend on the point of their addition and how effective the coagulant is mixed with the plant influent. Coagulants may be added prior to the grit chambers to maximize the benefit of grit chamber contact time during mixing. Other potentially appropriate locations are ahead of the plant influent Parshall flumes (if used) or in the grit chamber splitter boxes where wastewater creates adequate turbulence for efficient mixing.

Effect of Enhanced Primary Clarification on Digester Capacity. Chemically enhanced primary clarification will result in enhanced clarifier suspended solids, phosphorus and BOD removal efficiency and therefore, will also increase the amount of sludge generated in the clarifiers. This primary sludge quantity increase has to be taken under consideration in the digester design.

Clarifiers and Aerobic Sludge Digestion

Aerobic sludge digestion is most commonly used in relatively small plants of design capacity of 18,900 m^3/day (5 MGD) or less (WEF MOP 8, 1998). Aerobic digesters usually process sludge from extended aeration activated sludge facilities with or without primary clarifiers. If primary clarifiers are not used, the amount of the secondary sludge increases measurably. This extra sludge must be taken under consideration when sizing the aerobic digesters.

The aerobic digester retention time and oxygenation requirements for stabilization of a mixture of primary and WAS are significantly higher than those needed for WAS stabilization only. Because of the high energy costs associated with aerobic digester aeration for plants larger than 10 to 15 MGD, it is more economical to treat primary sludge separately in anaerobic digesters, while aerobically digesting only the waste activated sludge.

By contrast, in small wastewater treatment plants, overall system simplicity considerations may favor the elimination of the primary clarifiers and aerobically digesting all plant sludge. Aerobic digesters may be more cost-effective when treating WAS from extended aeration facilities operating at very high solids retention time because partial aerobic stabilization is already completed within the aeration basins.

Because aerobic digesters are not as sensitive as anaerobic digesters to fluctuations of plant influent quality and quantity, sludge storage in equalizing day tanks is not typically required. However, providing high-efficiency sludge thickening facilities ahead of the anaerobic digesters is recommended to minimize aerobic digester volume and associated power costs for digester mixing.

While usually the optimum sludge feed concentration to anaerobic digesters is 3 to 4% solids, aerobic digestion favors feed sludge concentrations in a range of 4 to 6% solids. These high levels of thickened WAS can be achieved cost-effectively only by mechanical thickening equipment (centrifuges or filter presses) and sludge conditioning prior to thickening. Since the mechanical thickening process is also energy intensive, the most cost-effective level of thickening must be determined based on the lifecycle cost analysis of expenditures for sludge thickening and stabilization.

Aerobic digesters, like the activated sludge systems, may frequently experience foaming problems caused by excessive growth of filamentous bacteria. If secondary clarifier WAS contains a significant number of filaments, these microorganisms will seed the biomass of the aerobic digester and will cause or contribute to digester foaming problems. Therefore, incorporating provisions for effective control of filamentous growth in the design of the activated sludge system is of even greater importance, when the sludge is stabilized by aerobic digestion.

Effect of Plant Sidestreams on Clarifier Performance

Sidestreams from various solids handling facilities (thickener supernatant, dissolved air flotation subnatant, anaerobic digester supernatant and waste streams from sludge dewatering) are typically returned upstream of the primary clarifiers. The BOD, TSS, COD, ammonia and phosphorus concentration of these sidestreams is several times higher than that of the plant influent.

Sidestreams such as tertiary filter backwash may cause surges in flow. Therefore, the primary clarifiers, the biological treatment system and the secondary clarifiers have to be designed to handle this additional organic load. Sidestream recycle load and flow fluctuations have to be minimized and if possible, sidestreams should be recycled during low influent flow/low influent load periods.

Usually, the waste streams from the solids dewatering facilities (centrifuges, belt filter presses and plate-and-frame presses) contain very fine solid particles that are difficult to settle in conventional primary clarifiers. Contact sludge blanket type clarifiers are more effective in removing these particles.

Depending on the content of the sidestreams, their treatment prior to recycling may warrant consideration. This is especially relevant to sidestreams that contain high levels of toxic compounds (such as cyanide and heavy metals) or elevated nitrogen and phosphorus levels.

A key advantage of aerobic digestion compared to anaerobic sludge stabilization is that it produces significantly lower strength supernatant, which minimizes any additional load on the primary clarifiers and activated sludge system. The organic strength of the aerobic digester supernatant is comparable to that of the plant influent and usually does not exceed 1% of the total plant flow.

12. Case Studies

Use of Primary Clarifiers for Solids Fermentation and Enhanced Phosphorus Removal

The 20 MGD biological nutrient removal plant, located in Lake Buena Vista in Florida and operated by Ready Creek Improvement District has successfully tested recirculation of a portion of the primary clarifier sludge to enhance plant phosphorus removal. The plant activated sludge system applies a five-stage Bardenpho process to achieve discharge permit limits of 5,5,3 and 1 mg/L for carbonaceous biochemical oxygen demand, total suspended solids, total nitrogen and total phosphorus, respectively.

Currently, facility effluent total phosphorus concentration is 0.8 mg/L using biological treatment with minimal enhancement by chemical precipitation. Chemical precipitation was found to be necessary because the plant's wastewater collection system is relatively short and does not provide adequate time for formation of short-chain volatile fatty acids (VFAs). VFAs are needed and used as a carbon source in the anaerobic selectors of the BNR plant to achieve enhanced biological phosphorus removal. To improve phosphorus removal, the plant staff experimented with enhancing the formation of VFAs in the primary clarifiers by re-circulating a portion of the primary sludge to the clarifiers.

Primary sludge recirculation increased the sludge retention time in the clarifiers, induced solids fermentation and thereby, increased the concentration of VFAs in the primary effluent. As expected, the FVA-enriched primary effluent significantly increased the overall phosphorus removal in the BNR process.

However, solids recirculation reduced the treatment capacity of the primary clarifiers and limited plant capacity expansion as influent plant flow increased. Therefore, the staff decided to continue their experiments with primary sludge fermentation in a separate tank. Pilot testing completed in a 160-gallon continuously mixed fermenter with solids retention time of 10 days indicated that VFA concentration could be increased by several times. VFAs in the unfermented solids ranged between 300 to 500 mg/L and VFAs after fermentation reached up to 1,700 mg/L.

Another beneficial effect of the primary sludge fermentation was the significant reduction in the primary sludge volume. Because of the fermentation process, the average primary sludge concentration decreased from 2.3 to 1.1%.

Case Study of Optimization of Clarifier – Aeration Basin System Design

Optimization of the clarifier-aeration basin system at the 16,860 m^3/day (4.5 MGD) Preston wastewater treatment plant (WWTP) in the Regional Municipality of Waterloo, Canada allowed a reduction in overall system tankage requirements by up to 25%, compared to the original conventional design approach (Ross et al, 1997). The clarifier-aeration basin system optimization was part of a plant upgrade effort to accommodate future plant flow increase, achieve year-around nitrification (effluent ammonia winter and summer limits of 10 mg/L and 5 mg/L, respectively), and meet a relatively stringent phosphorus effluent limit of 0.6 mg/L.

The Preston WWTP treats a combination of municipal and industrial wastewater in a conventional activated sludge process. The plant has two major contributors of industrial wastewater: a potato chip factory and an automotive manufacturing facility. The potato chip industrial discharger contributes approximately 25% of the hydraulic load and 75% of the organic load of the wastewater plant. The automotive manufacturer contributes approximately 25 % of the plant hydraulic load and a low portion of the organic load.

At the time of the optimization study, the plant was operating at approximately 60 % of its rated capacity of 18,860 m^3/day (4.5 MGD) and without nitrification requirement. The facility treatment processes included grit removal, primary clarification, secondary treatment in activated sludge process and disinfection using sodium hypochlorite.

The activated sludge treatment system consisted of two parallel aeration tanks equipped with mechanical aerators followed by four parallel circular secondary clarifiers. The treatment plant schematic is presented on Figure 5.

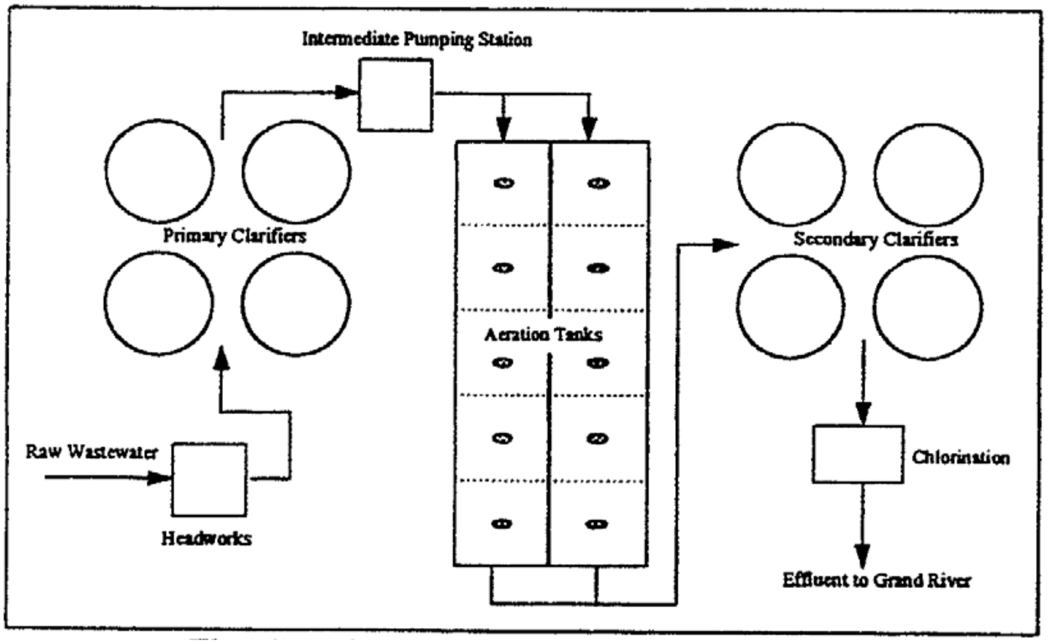


Figure 5 - Schematic of Preston Wastewater Treatment Plant

Table 4 presents plant's key influent wastewater quality characteristics.

A dynamic biological simulation model, coupled with extensive field studies, was used to determine the capacity of the existing clarifiers and aeration basins and to identify plant capacity and process upgrade measures. A commercially available plant clarifier-aeration basin optimization software package that operates in a Windows environment (BIOWIN™) was selected to complete system optimization.

A comprehensive primary effluent and final effluent monitoring program was implemented to generate data needed to calibrate the model. The site-specific values of the biological growth kinetics coefficients in the model were determined by using a bench-scale sequencing batch reactor that was fed with primary effluent from the treatment plant.

Table 4 - Average Raw Sewage and Primary Effluent Quality at the Preston WWTP

Parameter	Raw Sewage		Primary Effluent
	Preston WWTP	Typical Ontario Municipal	
BOD5, mg/L	397	170	222
TSS, mg/L	348	200	126
TKN, mg/L	39	35	35.5
TP, mg/L	5.9	6	4.2

The maximum available capacity of the existing clarifiers (maximum hydraulic and solids loading rates) and aeration basins (hydraulic retention time under maximum MLSS concentration of 3,000 mg/L) was determined by implementing field stress tests. After combining stress testing and dynamic modeling results, the maximum daily capacity of the activated sludge system was determined to be 32,000 m^3/day (8.5 MGD) and peak instantaneous capacity was found to be 47,460 m^3/day (12.5 MGD).

Considering three types of influent sources optimized the clarifier-aeration basin system. :

- influent source representing current flows and loadings to the plant
- influent source representing growth within the WWTP service area
- storm water flow component used to simulate plant operations during transient flows.

To reflect worst-case scenario for nitrification in the aeration basins, the minimum aeration tank wastewater temperature was assumed 10°C (50°F).

The size of aeration tanks and secondary clarifiers is directly related to several process parameters including:

- influent flow
- MLSS
- RAS concentration and rate

- WAS concentration and rate
- SRT
- the clarifier surface overflow rate a
- solids loading rate (SLR)

The following design boundaries were applied during the optimization process for the aeration basin and secondary clarifiers: maximum MLSS concentration of 3,000 mg/L; peak instantaneous clarifier SOR of 190 kg/ m^2.day (39 lbs/ ft^2.day); peak day SLR of 35 kg/ m^2.day (859 gal/ ft^2.day).

Using the boundary conditions defined above, the dynamic clarifier-aeration tank model simulations was applied over a range of aeration tank volumes to identify optimal secondary clarifier surface area requirements as a function of the aeration tank volume. The relationship between the aeration tank volume and secondary clarifier surface area is shown on Figure 6. This plot shows that the plot aeration tank volume must be at least 5,650 m^2 (60,820 ft^2) based on maintaining practical average MLSS concentration of 3,000 mg/L. The secondary clarifier surface area requirement for this case is 1,150 m^2 (12,380 ft^2) and is limited by the peak solids loading rate.

With larger aeration basins, the secondary clarifier surface area can be reduced. However, the clarifier size is limited by the peak solids loading rate. The minimum clarifier size was established at 915 m^2 (9,850 ft^2), at which point clarifier design is limited by the peak surface overflow rate. At the minimum clarifier surface area, the clarifier tank volume that results at both peak secondary clarifier SLR (i.e., 190 kg/ m^2.day (39 lbs/ ft^2.day)) and peak clarifier SOR [i.e., 35 m^3/m^2.day (869 gal/ ft^2.day)] is 7,200 m^3 (1,902,000 gallons).

The optimal design secondary clarifier-aeration basin configuration for upgrading the Preston WWTP lies on a point along the sloped line of Figure 6 where the maximum clarifier surface area is limited by the solids loading rate (SLR). The actual selection of aeration tankage and secondary clarifier sizing will be dependent on a number of other site-specific factors, such as: site requirements and constraints; capital costs and the ultimate site capacity requirements.

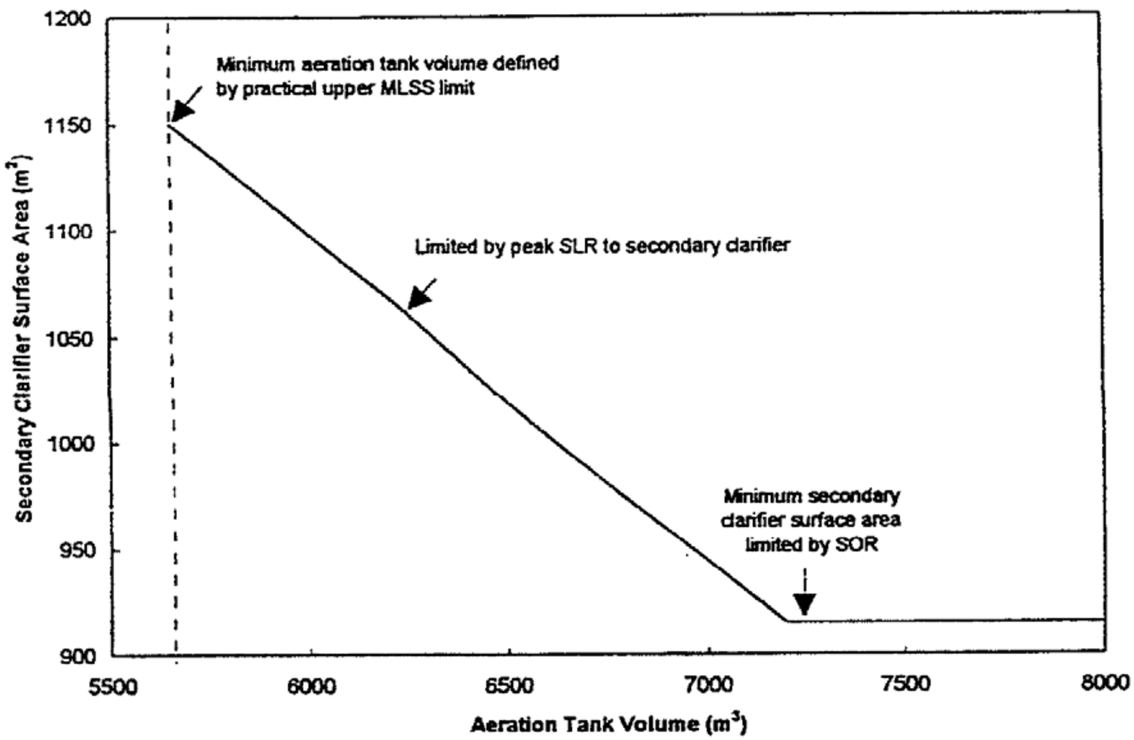


Figure 6 - Tradeoffs Between Aeration Volume and Clarifier Surface Area for Preston WWTP

Table 5 provides a comparison between the clarifier-aeration basin conventional plant design based on general industry guidelines, and optimal design extremes discussed above. Analysis of the results presented in Table 5 indicates that the optimization of the secondary clarifier-aeration basin system achieves a significant reduction of the overall system capacity and cost, as compared to conventional design using general industry guidelines and practices.

Depending on the scenario considered for the site-specific conditions of the Preston WWTP, aeration tank size could be reduced by 6 to 27% and secondary clarifier size could be decreased between 15 and 31% compared to conventional plant design.

Table 5 - Comparison of Conventional and Optimized Activated Sludge System Design

Parameter	Conventional Design	Application of Dynamic Modeling and Stress Testing	
		Minimized Aeration Volume	Minimized Secondary Clarifier Size
Aeration Volume, m^3	7,680	5,650	7,200
SRT, days	12	10	10
MLSS, mg/L	2,500	3,000	3,000
Clarifier Surface Area, m^2	1,340	1,150	915
Peak SLR, kg/ m^2.day	120	190	190
Peak SOR, m^3/m^2.day	24	28	35

Source: Ross et al., 1997

13. References

Albertson, O. (1992) "Clarifier Design" in Design and Retrofit of Wastewater Treatment Plants for Biological Nutrient Removal, Randall, C. et al., editors, Technomic Publishing Co., Lancaster, Pennsylvania.

Barnard, J. L. (1984) Activated Primary Tanks for Phosphate Removal, Water SA, 10, 121.

Bailey W., Tesfaye, A., Dakita, J., McDonald, J., McGrath M., Daigger, G., Sadick T. (1997) Nitrogen Removal Demonstration at the Blue Plains Wastewater Treatment Plant,
Paper presented at the 70-th Annual Conference of the Water Environment Federation, Chicago, Illinois.

Collins, M.A. and R. M. Crosby (1980) Impact of Flow Variation on Secondary Clarifier Performance. Paper presented at the 53-rd Annual Conference of the Water Pollution Control Federation, Las Vegas, Nevada.

Collins, J. R. and E. Jenkins (1999) Design-Build Accelerates WWTP Expansion Project. WaterWorld, April.

Danco, L. M. and C. W. Dickens (1994) Increase Capacity Without Expanding. Converting Existing Facilities Can Provide a Capacity Increase Without a Big Construction project, Operations Forum, September.

Ekama, G. A., J. L. Barnard, F. W. Guntert, P. Krebs, J.A., McCorquodale, D. S. Parker and E. J. Wahlberg (1997). Secondary Settling Tanks: Theory, Modeling, Design and Operation, International Association on Water Quality.

Hubbard, J., Kirkpatrick, B., Lee, G., and M. Langston (2001) Without Limits. Speeding Towards the Future One Facility Gives 110 % to Achieve Effective BNR, Operating at 300 % Design Capacity, Water Environment and Technology, July.

Keinath, T. M. (1985) Operational Dynamics and Control of Secondary Clarifiers, Journal of Water Pollution Control Federation, Volume 57, Number 7, July.

Kemp, F.D. and MacBride, B.D. (1990) Rational Selection of Design Criteria for Winnipeg South End WPCC Primary Clarifiers. Paper presented at the 63-rd Annual Conference of the Water Pollution Control Federation, Washington, D.C.

Maskell, A.D. and J. P. Lumbers (1974) Automatic Flow Balancing, Progress in Water Technology, Volume 6-th.

Metcalf & Eddy (2002) Wastewater Engineering, Treatment and Reuse. Fourth Edition. McGraw-Hill, Inc.

Olsson, G., Stephenson, J., Chapman, D. (1984) Hydraulic Shocks and Their Impact on Secondary Clarifier Performance, Paper presented at the 57-th Annual Conference of the Water Pollution Control Federation, New Orleans.

Parker, D. S. (1983) Assessment of Secondary Clarification Design Concepts, Journal of Water Environment Federation, Vol. 55, (4).

Porta, F. R., Williams, S.E. and Oisthuma, A. R. (1980) Plant Scale Clarifier Studies in Detroit. Paper presented at the 53-rd Annual Conference of the Water Pollution Control Federation, Las Vegas, Nevada.

Quisim, S. R. (1985) Wastewater Treatment Plants. Planning, Design and Operation. CBS College Publishing. Holt, Reinhart and Winston.

Morales L. M., Daigger G. T., Borberg, J. R. (1991) Capability Assessment of Biological Nutrient Removal Facilities, Research Journal of the Water Pollution Control Federation, Volume 63, Number 6, September/October.

Randall, C.W., Barnard J. L. and Stensel H. D. (1992) Design and Retrofit of Wastewater Treatment Plants for Biological Nutrient Removal, Technomic Publishing Co., Lancaster, Pennsylvania.

Ross, D., Briggs, T., Hribljan, M., Cober, K. (1997) Use of Dynamic Biological Simulation Model to Optimize Secondary Treatment Process Sizing at the Preston WWTP, Paper presented at the 70-th Annual Conference of the Water Environment Federation, Chicago, Illinois.

Sedlak, R.I. (Editor) (1991) Phosphorus and Nitrogen Removal From Municipal Wastewater: Principles and Practice. Second Edition. The Soap and Detergent Association.

Ten States Standards (2014), Great Lakes – Upper Mississippi River Board of State Sanitary Engineering Health Education Services, Inc. (2014). Recommended Standards for Wastewater Facilities, Albany, New York.

Tekippe, R. J. (1986) Critical Literature Review and Research Needed on Activated Sludge Secondary Clarifiers. Water Environmental Research Laboratory, Office of Research and Development, US EPA, Cincinnati, Ohio.

Voutchkov, N. S. (1992) Full Scale Phosphorus Removal Study at Two Modified Activated Sludge Plants, Paper presented at the 65-rd Annual Conference of the Water Environment Federation, New Orleans, Louisiana.

Voutchkov, N. S. (1993) Relationship of Clarification Efficiency of Circular Secondary Clarifiers, Fluid/Particle Separation Journal, Volume 6, Number 4.

Van Haandel, A. C. (1992) Activated Sludge Settling Part II: Settling Theory and Application to Design and Optimization. Water SA, 18, 3.

Wahlberg, E. J. (1996) Secondary Clarifiers: The Mechanisms Behind the Myths. Operations Forum, January.

Wahlberg, E. J. (2001) WERF/CRTC Protocols for Evaluating Secondary Clarifier Performance, Water Environment Research Foundation, Project 00-CTS-1.

Water Environment Federation (1998) Biological and Chemical Systems for Nutrient Removal. Alexandria, Virginia.

Water Environment Federation MOP 8 (1998) Design of Municipal Wastewater Treatment Plants, Fourth Edition, Alexandria, Virginia.

Water Pollution Control Federation (1980) Manual of Practice No. FD-1, Sludge Thickening, Alexandria, Virginia.

Water Pollution Control Federation (WEF), Manual of Practice FD-8, Clarifier Design, Second Edition (2005).

