

HIGH LOAD FIELD TEST OF A SECONDARY CLARIFIER

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ABSTRACT

Secondary clarifiers were field tested for hydraulic characteristics and performance and checked with the Clarifier Research Technical Committee's draft protocol at a Wisconsin and North Carolina WWTP. The test clarifiers were 48.78 m (160 ft.) and 42.68 m (140 ft.) diameter peripheral-feed, peripheral-overflow clarifiers with a 4.57 m (15 ft.) and 3.8 m (12.5 ft.) SWD. Settled solids are collected by a rotating hydraulic header.

The WWTP wanted to determine the capacity of the existing secondary clarifier system for use in planning of plant upgrades and permitting. Also, to determine if any upgrades or changes should be considered for the units.

Multiple tests were conducted at average and max flow conditions approximating 800 and 1,300 GPD/sq. ft. (1.36 and 2.21 m/h). Influent mixed liquor concentrations RAS concentrations, effluent TSS, sludge blanket profiles, and flow rates were recorded to document the performance of the clarifier. Stirred sludge volume index (SSVI) and in-situ dispersed solids (IDS) tests were conducted to quantify the settling and flocculation characteristics of the influent mixed liquor. Hydraulic characteristics of the clarifier were assessed by conducting slug dye and flow pattern/solids distribution and drough tests.

To run the tests, it was necessary to take basins off-line. As plant flow increased during the day, the nontest basin was used to maintain a fairly steady flow to the test clarifier. It was possible to maintain an overflow rate of up to 1,300 GPD/sq. ft. (2.21m/h) with a solids loading of up to 50 lbs./sq. ft./day (243 kgs/sq. m/day).

Results of the high load test show a hydraulic solids loading efficiency well above the conventional design parameters. Solids studies show effluent suspended solids (ESS) levels can be maintained below 20 ppm at hydraulic loading up to 1,300 GPD/sq. ft./day (2.21m/h). Effluent SS levels below 15 ppm were achieved at levels of 800 GPD/sq. ft. (1.36m/h).

It appears the existing clarifiers have a capacity of at least 2 times more than the design limits.

KEYWORDS: Clarifier, testing, TSS, dye patterns, overflow rate, PFPO (peripheral-feed, peripheral-overflow), activated sludge secondary clarifier, SLR (solids loading rate), DSS (dispersed suspended solids), flow pattern/solids distribution, plant capacity evaluation.

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INTRODUCTION

In the activated sludge process, the secondary (final) clarifier follows the aeration unit. The primary purpose of the final clarifier is to separate, by gravity, the mixed liquor suspended solids (MLSS) from the activated sludge process mixed liquor. The separated clear water from the clarifier can then be discharged to the receiving water. As required, disinfection or other treatment may be necessary prior to discharge. A portion of the settled sludge in the clarifier is returned to the aeration tank to maintain the desired mixed liquor suspended solids concentration. This is referred to as "return sludge." The balance of the settled sludge is wasted to a sludge treatment system.

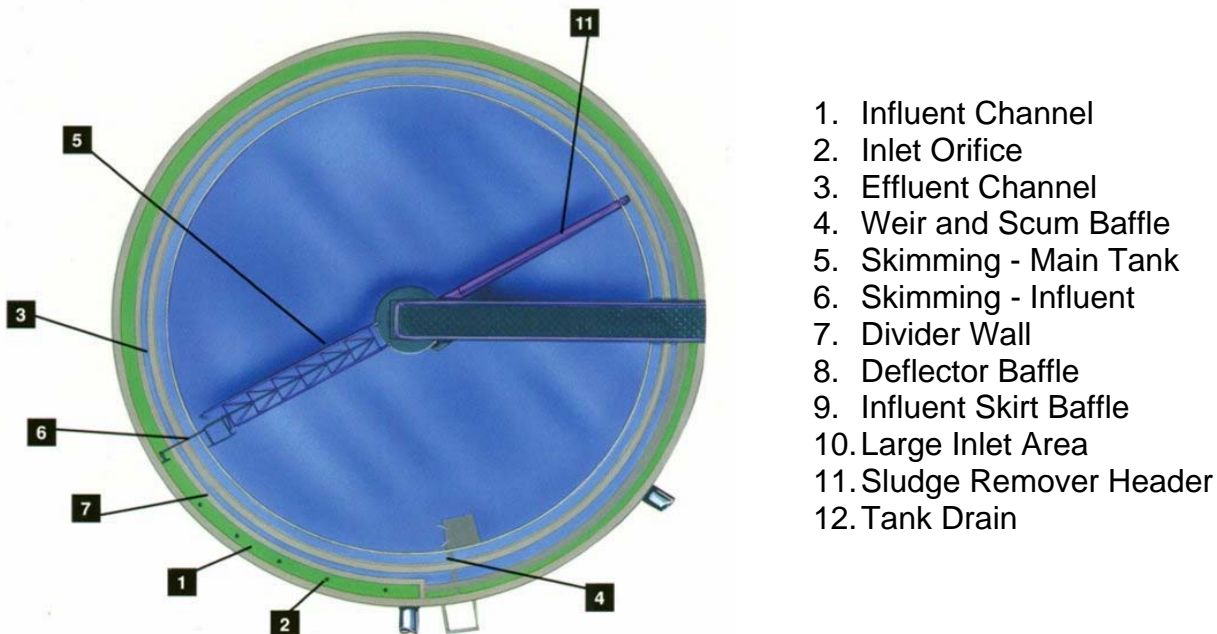
The secondary clarifier must not only produce an effluent of acceptable quality, but must also produce an underflow sludge of sufficiently high solids concentration to maintain the desired MLSS concentration. Highly concentrated waste sludges are also desirable from a sludge dewatering and treatment economics standpoint. The final clarifier, therefore, is a very important treatment unit. If it does not fulfill its function properly, the desired effluent quality may not be achieved.

Although their clarifiers were operating very well, both the North Carolina and the Wisconsin Waste Water Plants wanted to determine the capacity of their existing secondary clarifier system for the use in planning of any future upgrades and permitting. Also, they wanted to know if any upgrades or changes should be considered for the units.

This paper will deal primarily with the results of the high loading test and how and why these results were achievable. For the purpose of brevity, not all of the tables, graphs, and figures will be included in this paper.

BACKGROUND

The clarifiers at these plants are peripheral-feed/peripheral-overflow (PF/PO) clarifiers (Fig. 1) designed for optimum activated sludge secondary clarifier performance. The clarifier is made up of three basic hydraulic components, the inlet channel raceway, the effluent channel and the settled sludge withdrawal header.



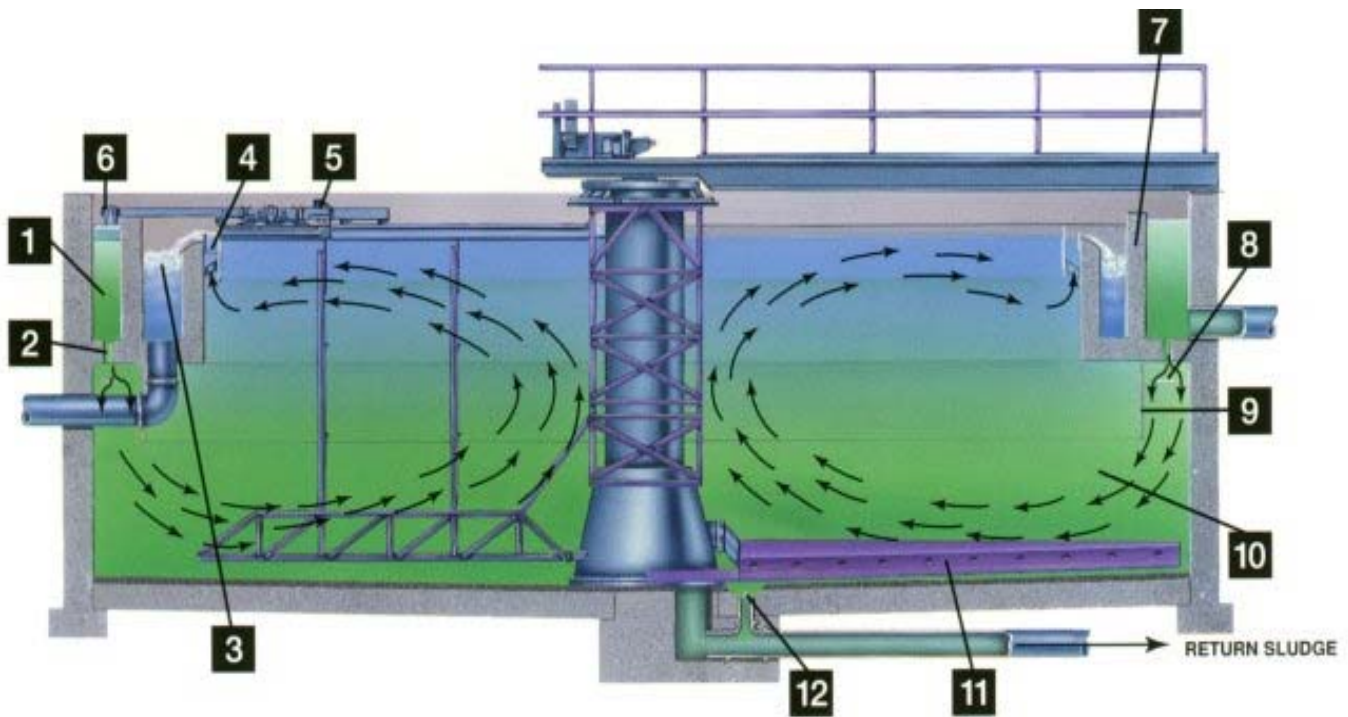


Figure 1

This clarifier evolved from extensive clarifier research studies conducted in the mid 1950's. Before this, circular clarifiers used in activated sludge plants were principally of the center feed design. Studies of center feed basins showed inefficiencies resulting from under utilization of the tank volume and short-circuiting of the incoming flow to both the effluent weirs and the sludge removal hopper located directly below the inlet. Investigations by Anderson had shown that flow enters the center feedwell at a high velocity creating considerable turbulence. This high velocity flow travels in a relatively narrow band over the top of the settled solids layer, turns up at the sidewall and carries solids to the influence of the effluent weirs. Repositioning the effluent weirs out of the way of the long eddy currents did improve efficiencies at high cost, but not beyond levels of a poorly performing rectangular tank.

Recognizing the inherent inefficiency of center feed, radial flow basins, it was decided to evaluate a peripheral feed design by Benham. It was found when comparing hydraulic characteristics of the peripheral-feed basin to that of the center-feed basin that short-circuiting could be reduced 300 to 475%. Less overall short-circuiting was also observed and hydraulic efficiency improved at peak overflow rates. Improved performance was attributed, in part, to a 7 to 9 times reduction of inlet velocities into the clarification zone.

Early developmental bench scale studies comparing peripheral feed tanks to center feed were conducted utilizing wax spheres of known diameter and specific gravity. Pilot studies were also conducted with activated sludge mixed liquor. A summary of results appears in the paper "A Comparative Study of the Hydraulic Characteristics of Two Types of Circular Solids Separation Basins" by Wm. J. Katz and Anthony Geinopolos. Solids separation tests indicated removal efficiency as good as the most efficient rectangular API oil separator. Bench scale studies using sewage and radioactive tracers conducted in South Africa and presented by Bergman, et al., corroborated the higher efficiency of the peripheral-feed tank over center feed models.

The first full-scale installation of the clarifier was at Sioux Falls, South Dakota in 1958. One of two existing 24.4 m (80 ft.) side inlet final clarifier tanks was converted allowing side-by-side comparison of the two designs. Despite a large meat packing plant contribution, the clarifier produced effluent suspended solids concentration levels of 21 ppm at sustained overflow rates of 2,000 GPD/sq ft (3.4 m/h) compared to 60 ppm at 960 GPD/sq ft (1.6m/h) for the center feed tank. Further tests were conducted to find the break point of each clarifier and the clarifier was shown to have a hydraulic capacity 4 to 5 times that of the center feed tank.

Further side-by-side field studies were conducted at Ann Arbor, Michigan in 1960 and 1961 with 21.3 m (70 ft) diameter x 2.97 m (9'-9") SWD tanks. Improved effluent suspended solids concentrations were achieved with a 50% increase in basin overflow rates. Effluent SS levels under 15 mg/l were maintained in these shallow tanks at overflow rates of 1,400 to 1,600 GPD/sq ft. (2.38 to 2.72 m/h).

Early peripheral clarifier basins were designed with effluent troughs suspended at the center of the tank. This interfered with full surface skimming and made the weirs inaccessible to the plant operators for balancing and cleaning. In the late 1960's, the effluent weir location was relocated to the peripheral wall adjacent to the inlet channel, thus the peripheral-feed/peripheral-overflow (PF/PO) clarifier started. Field studies at Kenosha, WI, P.H. Glatfelter Co. and San Jose/Santa Clara WWTP, CA demonstrated that the peripheral overflow weirs performed as well or better than center take-off overflow. At Kenosha, average effluent SS from both arrangements were below 10 ppm with overflow rates of 650 to 1,000 GPD/sq. ft. (1.1 to 1.7 m/h), mixed liquor suspended solids over 4,000 mg/l and mass loadings over 30#/day/sq. ft. During EPA Storm Overflow Study (Grant #11023EKC), this 42.7 m (140') diameter basin was able to limit average SS levels to 24 ppm during storm events when hydraulic and solids loading rates were doubled. Additional operating data is presented in the article by Boyle. Similar results were attained under stress test conditions in an in house paper "High Rate Performance Test at the Kenosha, WI WWTP," by Gross and Rouse. This test also demonstrated compliance to equations predicting effl. ss levels reported in the paper, "A Mathematical Model of a Final Clarifier," EPA Project #17090FJW, 1972.

Dye dispersion studies by Crosby of a 42.67 m (140 ft) diameter x 4.27 m (14 ft) SWD PF/PO clarifier tank at Oakland, CA - East Bay MUD oxygen activated sludge plant gave further insight into the PF/PO clarifier hydraulics. Observations of dye movement revealed that, as in center feed tanks, incoming flow travels along the top of the settled sludge layer. But, unlike center feed tanks, there is no sharp upturn of flow. Rather, a gradual and nearly ideal uniform upflow over the entire surface of the tank was observed. Crosby notes that no other tank studied even approximated the theoretical uniform upflow model. Hydraulic efficiency was an excellent 69% at 650 GPD/sq. ft. (1.1 m/h). Dye studies at Madison, WI showed that efficiency of a 32 m (105 ft) diameter PF/PO clarifier tank was equivalent to that of a much more expensive floc-clarifier tank. Reference "Secondary Clarifier Operating Strategies," by Shin-Tai Yung, Master Thesis-University of Wisconsin-Madison.

Further studies by Crosby and others have shown that density currents in secondary settling tanks are real, will develop outside an energy distribution device and are detrimental to effluent quality. The ASCE Clarifier Research Technical Committee (CRTC) investigated hydraulic efficiency and concluded "that high inlet MLSS concentrations, and therefore high solids loading rate, will increase the degree of short-circuiting and nonideal flow conditions and strongly suggest this will increase effluent SS in the secondary clarifier. Field investigations of highly loaded clarifiers are lacking from the literature and warrant a study."

PF/PO HYDRAULICS (Fig. 1)

The PF/PO clarifier design introduces flow into the tank by means of the inlet raceway which carries the flow around the entire periphery. The raceway floor contains orifices which are sized and spaced in proper relation to channel friction losses for uniform distribution. The raceway cross section narrows in proportion to the flow to maintain a nearly constant velocity. Flow through the orifices is dispersed by means of target baffles into an annular ring beneath the raceway. The inlet distribution zone is defined by the inlet skirt which is attached to the bottom of the raceway. Flow enters the clarification zone near the mid-depth of the tank and diffuses inward and upward.

Uniform distribution of influent MLSS around the basin is achieved by judiciously maintaining an inlet orifice headloss greater than the frictional losses in the channel. Headlosses for the full range of design influent and effluent flows must be computed with orifice headlosses varying from as low as 38 mm (1.5-inch) at average inlet flow to as high as 250 mm (10-inches) at peak capacity. Such a special design is best left to an experienced designer with the aid of an established computer model.

PERFORMANCE BACKGROUND OF THIS PAPER

Despite a wealth of published information on secondary clarifier performance, designers are frequently at odds as to the optimal design of both basin and equipment. Designers often rely on antiquated design standards, folklore and popular fads without an understanding of what the key design parameters are or how varying characteristics influence performance. Design selections are often based on undocumented hydraulic and solids loading data, unconfirmed sludge and settling characteristics and incomplete historical background. A contributing factor to the wide variance in preferred designs has been a lack of an agreed testing protocol and standardized reporting method. This investigation was conducted to document performance of the peripheral-feed design which developed out of research conducted by Katz and Geinopolis during the mid-1950's.

PHYSICAL DESCRIPTION - RALEIGH, NC TEST FACILITY

The Neuse River WWTP , is a 227,100 m³/d (60 mgd) facility which serves the city of Raleigh, NC and surrounding areas. The Neuse River activated sludge plant provides tertiary treatment with single stage nitrification in the extended aeration mode followed by dual media filters. Waste activated sludge is thickened by gravity belt thickeners or dissolved air flotation and stabilized by aerobic digestion then pumped to trucks for land application or stored in solids storage tanks for later land application. Primary sludge is dewatered on belt filter presses and lime stabilized before being discharged for application to farmland. Supernatant and filtrate from solids treatment is returned to the plant influent.

Wastewater arriving at the Neuse River Plant is lifted 9 m (29 ft) by spiral screw pumps to mechanically cleaned bar screens with 12 mm (1/2-inch) clear openings. Flow passes through Ditrator type grit collectors to ten (10) 6.1 m (20 ft) wide by 76.2 m (250 ft.) long primary collectors. Primary effluent enters six (6) 24,100 m³ (6.37 mg) dual pass extended aeration basins equipped with jet aerators in the first pass and fine bubble aerators in the second pass. Mixed liquor is distributed to ten (10) 30.5 m (100 ft.) diameter by 3.65 m (12 ft.) SWD and two (2) 48.75 m (160 ft.) diameter x 4.57 m (15 ft.) SWD PF/PO secondary clarifiers. Return sludge is collected by dual hydraulic removal unitube sludge collectors and passes through a parshall flume before being elevated by spiral screw pumps to the ML/RAS channels. Three (3) sludge pits are provided for the twelve (12) clarifiers with three (3) screw pumps per pit to lift RAS back up into a common RAS channel which runs along the influent mixed liquor channels. Clarifier effluent is polished by twelve (12) effluent filters before disinfection by ultraviolet light. Filter backwash water is collected in a reclaim tank and returned to the plant influent.

PHYSICAL DESCRIPTION - RALEIGH, NC TEST CLARIFIER

The test clarifier is a 48.75 m (160 ft) diameter x 4.57 m (15 ft) SWD split influent flow, peripheral-feed, peripheral-overflow secondary clarifier (Tank #1) with dual unitube sludge collection headers. Mixed liquor is fed from an aerated mixed liquor channel common to all clarifiers. An isolation gate is provided at each clarifier and each of the 30.5 m (100 ft) diameter clarifiers is provided with a parshall flume allowing test flows to be deduced by monitoring primary effluent flow, RAS flows and head on weirs. From the isolation gate, mixed liquor splits in the test clarifier and travels around each half of the tank periphery. Inlet control orifices are located in the floor of the inlet raceway. Each orifice is provided with a target baffle beneath it to dissipate energy and disperse flow behind the inlet skirt. Raceway width decreases and orifice spacing varies as flow travels around the tank to account for frictional channel losses and achieve a uniform inlet distribution into the basin while maintaining a fairly constant channel velocity. A single half-span bridge is positioned along side the mixed liquor feed point and is in line with the effluent collection drop box located 180 degrees from the inlet.

Settled solids are collected in two (2) truss supported rectangular shaped unitube headers and conveyed to a closed manifold which rotates around the 600 mm (24-inch) diameter center support pier. Return sludge drops through a slotted opening beneath the manifold into a 610 mm (24-inch) RAS pipe. The sludge pipe is provided with a control valve and a separate parshall flume to allow for individual tank control and flow monitoring. From the parshall flume sludge flows to a screw pump lift station.

Clarifier Design Data:

Tank Dia.	48.75 m	(160 ft.)
Surface Area	1,865 m ²	(20,105 ft. ²)
SWD	4.56 m	(14.97 ft.)
Tank Volume	8,520 m ³	(300,900 ft ³)
Weir Length	142.3 m	(467 ft.)

PHYSICAL DESCRIPTION - KENOSHA, WI TEST FACILITY

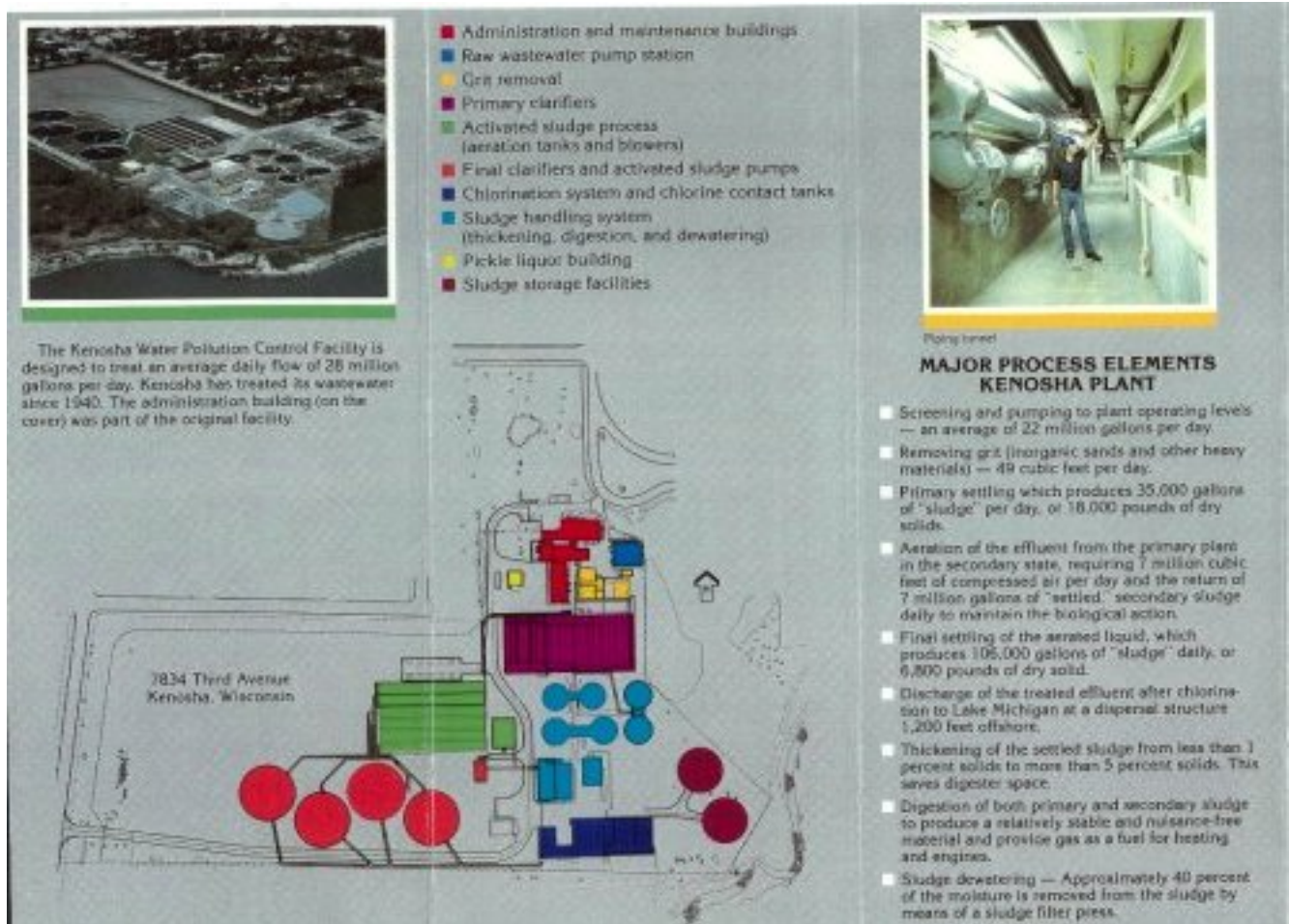


Figure 2

The Kenosha WWTP (Fig. 2) is a 28 mgd ($1.23 \text{ m}^3/\text{s}$) at design facility which serves the city and surrounding areas. The activated sludge plant also provides chemical P removal. Waste activated sludge is thickened by dissolved air flotation and mixed with sludge from the primary tank then stabilized by anaerobic digestion that is heated and has gas mixing. The digested sludge is dewatered by plate and frame Filter Presses and land disposed. Supernatant and filtrate from solids treatment is returned to the plant influent.

Wastewater arriving at the plant goes through two (2) mechanically cleaned bar screens with 18 mm (3/4-inch) clear openings, then pumped to two (2) 15.2 m (50 ft.) x 15.2 m (50 ft.) square Ditrator-type grit collectors. The waste flow then goes to sixteen (16) 4.4 m (16 ft.) wide by 41 m (135 ft.) long by 2.7 m (9 ft.) SWD primary clarifiers. Primary effluent enters three (3) 36.8 m (120 ft.) wide by 62 m (203 ft.) long by 4.6 m (15 ft.) water depth aeration basins with fine bubble diffused aeration. Mixed liquor is distributed to four (4) 42.7 m (140 ft.) by 3.66 m (12 ft.) SWD peripheral-feed/peripheral overflow final clarifiers.

PHYSICAL DESCRIPTION - KENOSHA, WI TEST CLARIFIER



Figure 3

The test clarifier (Fig. 3) is a 42.7 m (140 ft) diameter x 3.66 m (12 ft) SWD unidirectional flow, peripheral-feed, peripheral-overflow secondary clarifier (Tank #4) with one unitube sludge collection header. Mixed liquor is fed from an aerated mixed liquor channel common to all clarifiers. An inlet gate is provided at each clarifier. From the inlet, mixed liquor travels 360° around the tank periphery. Inlet control orifices are located in the floor of the inlet raceway. Each orifice is provided with a target baffle beneath it to disperse flow behind the inlet skirt. Raceway width decrease and orifice spacing varies as flow travels around the tank to account for frictional channel losses and achieve a uniform inlet distribution into the basin while maintaining a fairly constant channel velocity. A single half-span bridge is positioned 90° from the mixed liquor feed point and is 97° from the effluent collection drop box located 187° from the inlet.

Settled solids are collected in one (1) truss supported rectangular shaped unitube headers and conveyed to a closed manifold which rotates around the center support pier. Return sludge drops through a slotted opening beneath the manifold into a 458 mm (18-inch) RAS pipe. Each clarifier sludge pipe is provided with a magmeter and control valve to allow for individual tank control and flow monitoring. The sludge piping is connected directly to pumps.

Clarifier Design Data:

Tank Dia.	42.7 m	(140 ft.)
Surface Area	1431 m ²	(15,386 ft ²)
SWD:	3.66 m	(12 ft.)
Tank Volume:	5,240 m ³	(184,635 ft ³)
Weir Length	123.5 m	(405 ft.)

PLANT CONDITIONS DURING THE TEST PERIOD

During these tests, the activated sludge basins were operated in their normal service mode. The following average characteristics were present for the activated sludge system at the influent to the test clarifier and in the effluent.

For these flow curve tests and dye tracer tests, the majority of the flow was directed to the test clarifier. The desired test flow rate was maintained as constant as possible by diverting all excess flow to one or more of the other plant clarifiers.

Raleigh

	Nov 30	Dec 1	Dec 2	Dec 3
Ave. MLSS (mg/l)	4180	2990	3030	3030
RAS SS (mg/l)	7590	8390	7600	6260
ETSS (mg/l)	8	13	16	9
Stirred SVI ₃₀	75	95	75	75

Raleigh

	Nov 30	Dec 1	Dec 2	Dec 3
Q. Flow Rate (mgd)	16.1	25.6	26.1	11.3
R, RAS Rate (mgd)	10.5	13.5	13.5	13.5
Q + R (mgd)	26.6	39.1	39.6	24.8
Weir Loading (g/lf/d)	34,400	54,700	55,700	25,600
Surface O'flow Rate (g/sf/d)	800	1270	1300	600
Solids Loading (lbs./sf/d)	46	49	50	31

Kenosha

	Aug 7	Aug 8	Aug 9	Aug 10
Ave. MLSS (mg/l)	1880	2091	2330	2318
RAS SS (mg/l)	6497	7708	9203	8200
ETSS (mg/l)	19	17	10	12
Stirred SVI ₃₀	83		72	

Kenosha

	Aug 7	Aug 8	Aug 9	Aug 10
Q. Flow Rate (mgd)	17	17	12	12
R, RAS Rate (mgd)	6.3	5.2	2.9	2.9
Q + R (mgd)	23.3	22.2	14.9	14.9
Weir Loading (g/lf/d)	42,000	42,000	29,400	29,400
Surface O'flow Rate (g/sf/d)	1105	1105	790	790
Solids Loading (lbs./sf/d)	26	26	18	18

The limiting factor that stopped the test to go to a higher value was:

Raleigh - The plant return sludge pumps were not designed for the capacity to remove the necessary volume of RAS from the highly loaded test clarifier.

Kenosha - The effluent structure downstream of the clarifier outlet was not designed to handle the higher flow, thus backing the effluent up into the tank.

SOLIDS/TEMPERATURE PROFILES

The distribution of solids in the basin was monitored throughout the tests by the use of two hand held Royce electronic suspended solids analyzers. The Royce meters were calibrated each day based on MLSS readings taken the previous day and checked against the plant's lab results. Solids profiles from Royce data are tabulated in Tables I and II. Solids concentrations heavier than 2,000 mg/l are shaded to define a generally accepted top of blanket level. Boyle reports, "It may be concluded from the logic, field observations and sludge concentration variance that the sludge blanket is at a level elevation and the concentration (although fairly uniform) is located in a random pattern" (throughout the floor area). Temperature difference from inlet mixed liquor channel to effluent drop box was monitored with a YSI meter during cold and warm periods. Random checks showed a consistent 0.1 degree c drop from inlet to effluent drop box with no more than 0.2 degree C drop from inlet mixed liquor feed channel to farside effluent weir.

DROGUE CURRENT TESTS

The drogue test was first devised by Anderson in 1945 to define density currents within a clarifier. The drogue is suspended from a float, Fig. 4, at increasing depths to catch the current. The movement of drogues were monitored Fig. 5, starting from three locations along the bridge at normal and high flow rates beginning 0.6 m (2 ft) below the surface and progressing downward until movement was judged to be insignificant indicating the drogue is below the plane of the dominant eddy current. The results of these tests are shown in Figs. 6 and 7.



Figure 4



Figure 5

**NEUSE RIVER WWTP
DROGUE DATA @ HIGH FLOW (DEC. 2)**

Plan View in Clarifier 1 @ 18ft., 35 ft., and 52 ft. from Influent Channel

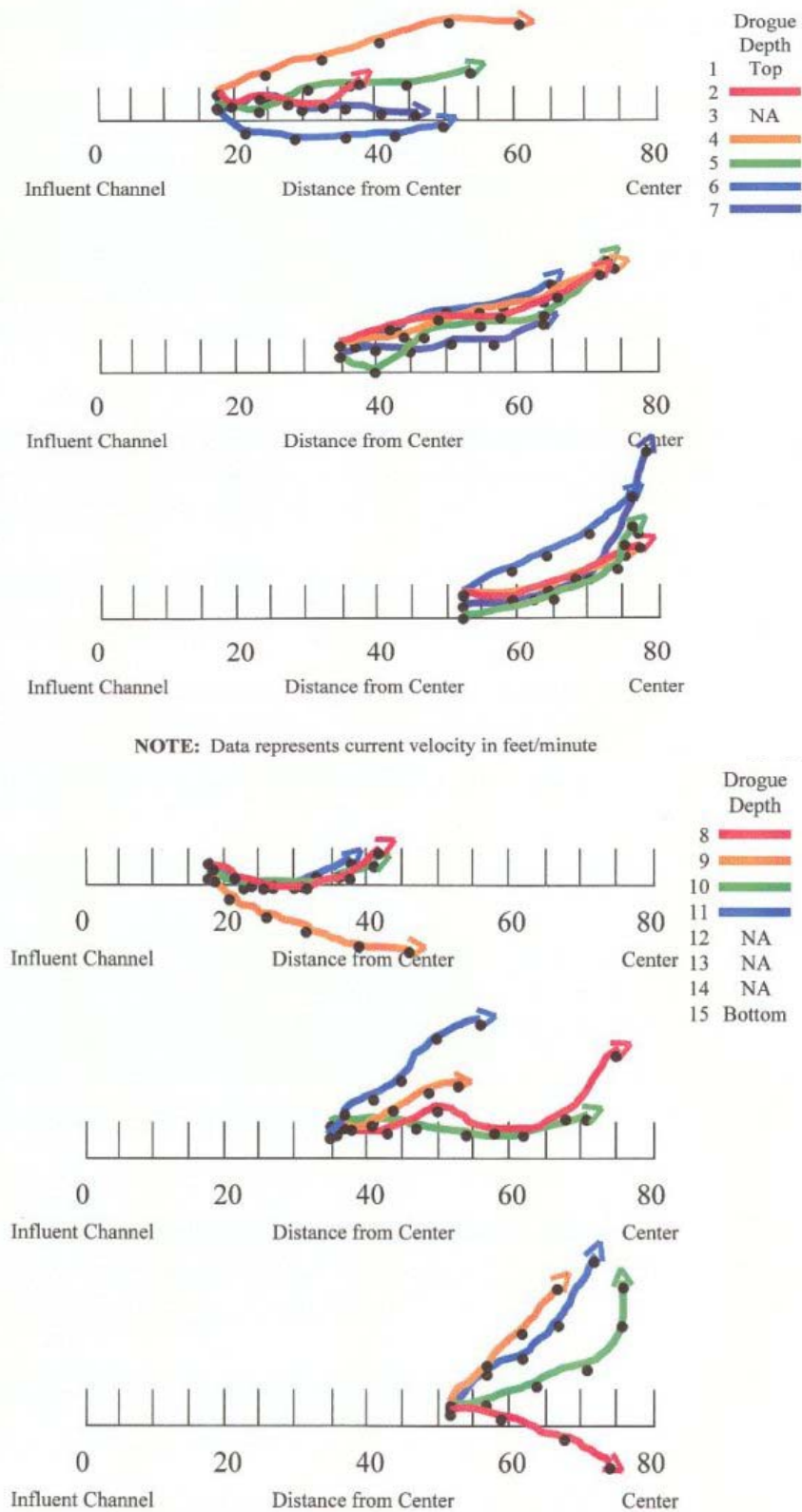
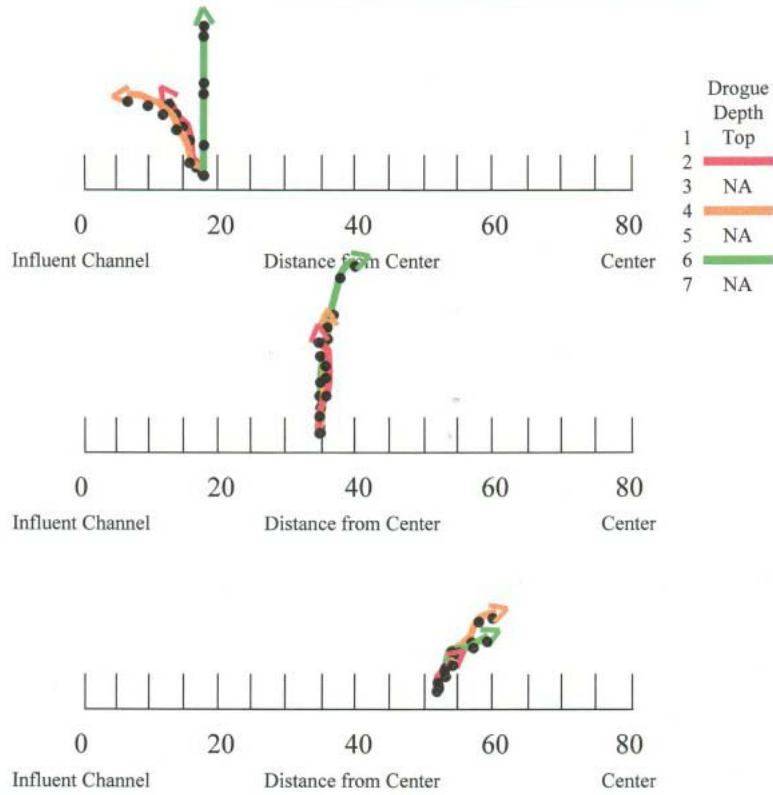


Figure 6

**NEUSE RIVER WWTP
DROGUE DATA @ AVERAGE FLOW (DEC. 3)**

Plan View in Clarifier 1 @ 18ft., 35 ft., and 52 ft. from Influent Channel



NOTE: Data represents current velocity in feet/minute

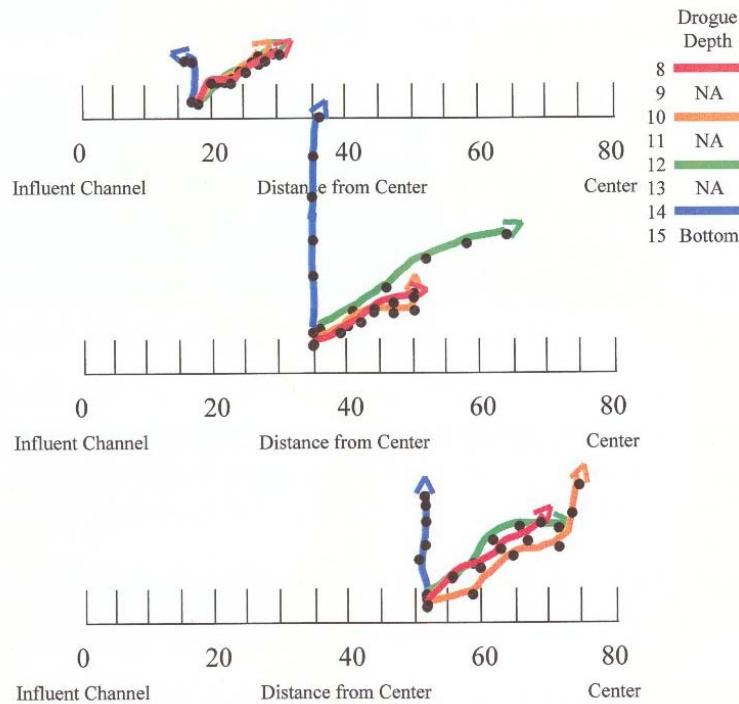


Figure 7

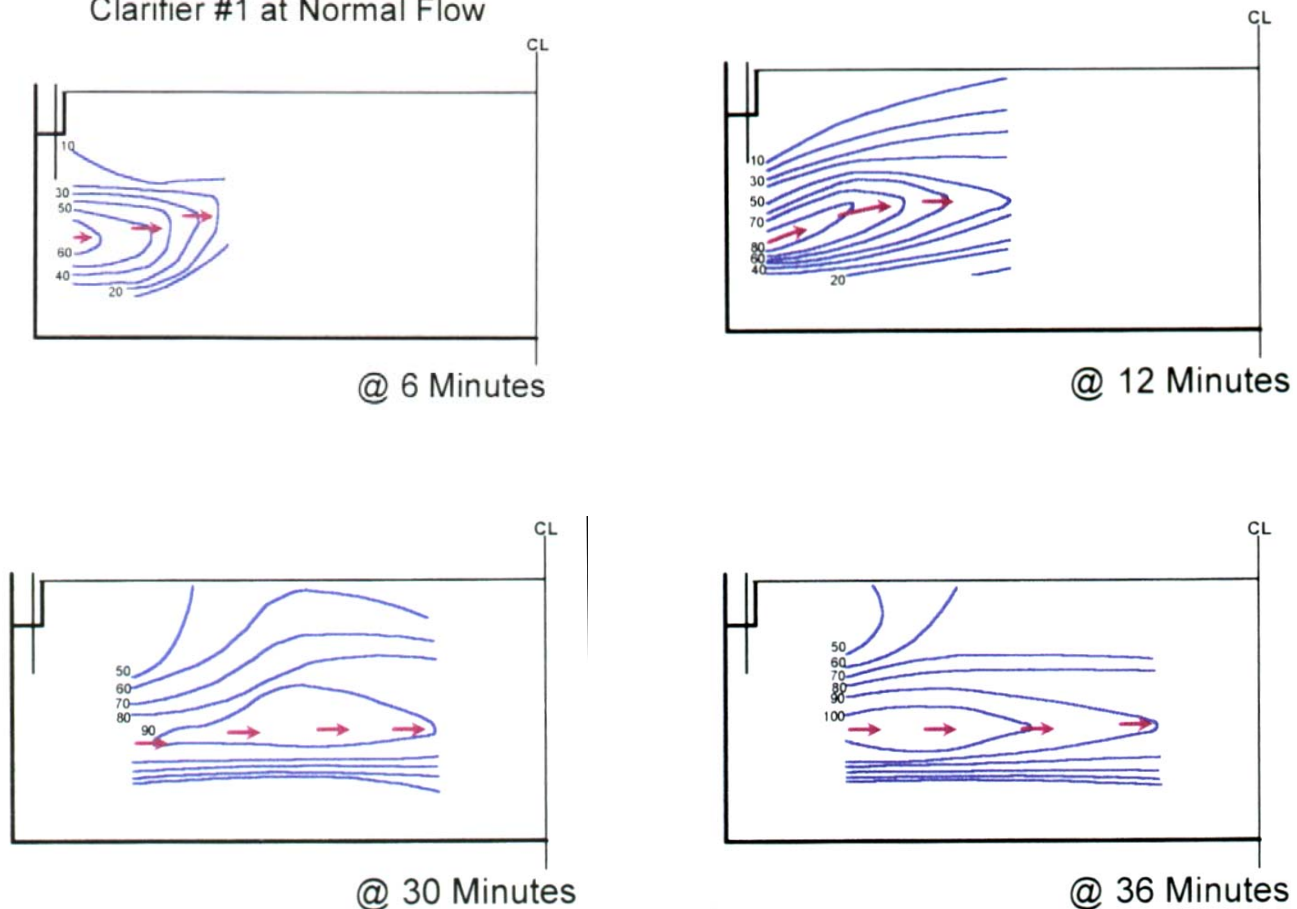
DYE/SLUG/TRACER TESTS

Hydraulic efficiency as measured by the dye slug test, also known as the hydraulic residence time (HRT) distribution test or flow curve test, is one method of rating the performance of a clarifier. Experience has shown that clarifier efficiency can be influenced by a number of energy fluxes among which are inlet kinetic energy, density, temperature and eddy currents. These currents cause a nonuniform distribution of solids and, therefore, short circuiting through the tank. The net result is a shortening of the detention time in the tank and reduction of its solids removal efficiency. The dye slug test tries to quantify this efficiency.

The continuous dye tracer test was developed by Crosby as a substitute for direct velocity measurements and reveals flow characteristics in the tank that cannot be deduced from the slug test alone. A series of "snapshots" of the progression of dye in the tank is included in Fig. 8. Figs. 9 and 10 show the sampling procedures used.

DYE FLOW PATTERN TESTS

Neuse River WWTP
Clarifier #1 at Normal Flow



"The current progressed uniformly toward the center, with the exception that the subsequent motion back toward the perimeter weir would be without the suspended solids that would be carried inward with the bottom current."

Quoted from report on a 140 ft. (42.7m) x 12 ft. (3.7m) SWD unit.

Note the up flow pattern of the clear liquid.

Figure 8



Figure 9



Figure 10

DISCUSSION OF HYDRAULIC PERFORMANCE TESTING

In evaluating a clarifier's hydraulic performance, it is useful to compare it to an ideal clarifier. In an ideal peripheral-feed/peripheral-effluent clarifier, the theoretical detention time (T_t) can be determined by dividing the total clarifier volume by the total flow rate entering the clarifier. This value represents the theoretical time of travel through the reactor. Although no clarifier can be expected to have this actual detention time, the more closely it approaches this value, the more efficient it may be as a settling tank. One of the tests that is used to determine the actual detention time is the "flow curve" or "dye slug" test.

THE FLOW CURVE TEST

The hydraulic efficiency of a clarifier is a measure of its performance as compared to that of an ideal settling tank. Through the use of dye tracer studies, the performance of a clarifier can be described by several methods. For example, if a "slug" of dye is introduced into a clarifier at a point where it will be well mixed with the contents entering the clarifier, the rate at which the dye exits the clarifier can be used to describe its hydraulic efficiency. The graphical representation of this information is called the "flow curve" -Fig.11.

An ideal settling tank is defined as one that has the characteristics of a plug flow reactor. That is to say, the residence time of an element of flow in that tank is equal to the theoretical detention time at that flow rate. Accordingly, if a slug of dye was injected at the inlet of an ideal clarifier, that slug would travel through the clarifier as a compact mass and exit from the clarifier at the theoretical detention time. The plot of this flow curve is shown in Figure 11A.

At the opposite end of the reactor spectrum, a tank with a high degree of stirring can be likened to a completely mixed reactor. If a slug of dye was injected at the inlet of this type of reactor, it would immediately be dispersed throughout the entire tank contents. A sample of the effluent at the beginning of the test would be at the maximum concentration since the slug of dye would have been diluted by the total volume of the tank. Then, as the flow continued to pass into the tank, the dye concentration would be continuously diminished. The plot of the flow curve for a completely mixed reactor is shown in Figure 11B.

An actual clarifier, however, is a non-ideal plug flow reactor. Because of the various inlet and exit conditions and the dispersion due to mixing, the flow patterns are such that the actual clarifier always displays characteristics that are less efficient than those of the ideal clarifier. The actual shape of each clarifier curve, of course, depends on the influence of these conditions and on the clarifier configuration, and is generally similar to that shown in Figure 11C.

Reactor Configurations and Flow Curves

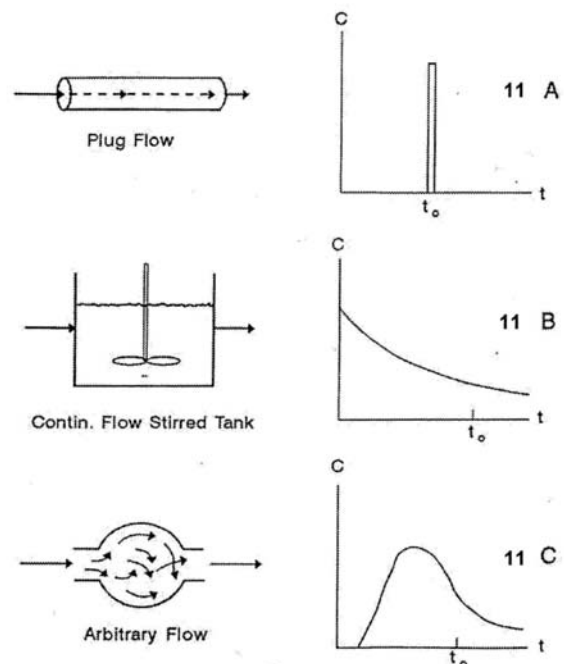


Fig. 11

FLOW CURVE CHARACTERISTICS

With the advances in fluorescent dye tracer technology, the examination of flow curves has become a more useful technique in determining the actual field performance of a clarifier. Graphical plotting and computer-based analytical methods enable the data to be analyzed quickly and completely.

In examining a clarifier's flow curve, there are several features that can be identified for comparison with other flow curves. One of the most important of these is the time of the initial appearance of the dye in the effluent (T_i). This is the first good indication of the degree of departure of the clarifier's performance from the ideal. The ratio of the time of the initial appearance of the dye to the theoretical detention time is one indication of the amount of short-circuiting taking place. A low ratio of T_i/T_t indicates a high degree of short-circuiting, while a high ratio indicates that the clarifier is approaching a plug flow condition.

Another important feature of the flow curve is the rate at which it rises toward the peak value. A fast-rising curve indicates the presence of a strong short-circuiting current while a curve that rises more slowly indicates either less short-circuiting or a broader band of influence of the current. Short-circuiting currents are those that find more concentrated and direct paths to the effluent weirs. The velocities of these currents are often such that they tend to transport to the effluent, the lighter floc particles that would have settled in a more ideal clarifier.

The time that it takes for the exiting dye to reach its peak/modal value T_m has often traveled backwards in the upper layers of the clarifier towards the influent. The dye then repeats this backward-and-forward rolling path until it finally exits the clarifier. This secondary mass of dye, although it may constitute a major part of the flow curve, is generally not indicative of useful settling time, but simply the detention time for the soluble dye in the clarifier. One of the major measures of the efficiency of a clarifier, then, would be a comparison of the T_m to the T_t . This is called the "modal efficiency" (T_m/T_t).

The full development of the flow curve can be evaluated to determine the actual detention time of the clarifier and the degree of dispersion taking place. Although the time elapsed to reach the center of gravity of the area under the flow curve (as determined by the sum of the moments of the individual areas about the y-axis) is the center of mass of this curve, its location is unduly affected by the increasingly smaller concentrations of dye that exit the clarifier towards the end of the dye test. Accordingly, we use the time to the center of the area under the curve as a more representative indication of the actual operating detention time (T_a). This time is generally about 10 percent less than the time to the center of mass (the centroid). The ratio of this time to the theoretical detention time (T_a/T_t) is called the "hydraulic efficiency" of the clarifier.

The degree of dispersion taking place in a clarifier can be quantified by relating the time at which 10% of the dye has left the clarifier to the time at which 90% of the dye has left. This is called the Morrill Index; a low number indicates a condition approaching plug flow, while a high number indicates a greater degree of mixing.

DETENTION TIME TESTS: (APPENDIX B)

The flow curve/detention time tests were conducted at both normal and high flow conditions. For the theoretical detention time based on the total flow to the clarifier, the results of these tests are summarized in the following table:

Raleigh

Flow Rate	Blanket Thickness	Tt (min.)	Ti (min.)	Ti/Tt Effic. (%)	Tm (min.)	Modal Effic. (%)	Ta (min.)	Hydraulic Effic. (%)	Morrill Index
Normal	3' - 4'	122	15	12	45	37	79	65	4.9
High	11' - 12'	83	10	12	30	36	55	66	5.5

Kenosha

Flow Rate	Blanket Thickness	Tt (min.)	Ti (min.)	Ti/Tt Effic. (%)	Tm (min.)	Modal Effic. (%)	Ta (min.)	Hydraulic Effic. (%)	Morrill Index
Normal	1'	132	30	23	80	61	105	80	
High	1' - 2'	85	15	18	40	47	84	98	4.3

Figs. 12 and 13 show the graphical presentation of the above.

From tests on a 140 ft. (42.7m) dia. x 12 ft. (3.66m)

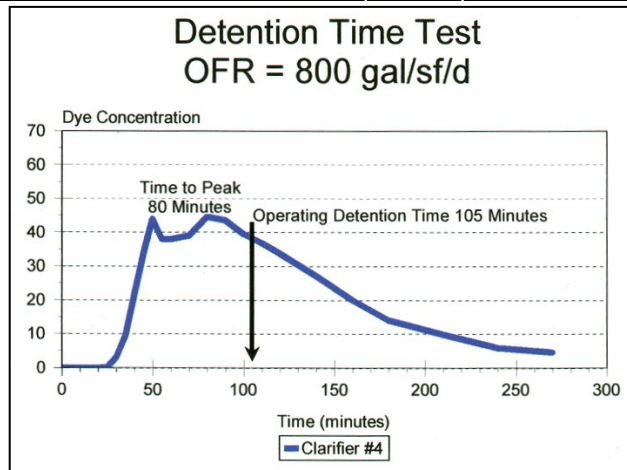


Figure 12

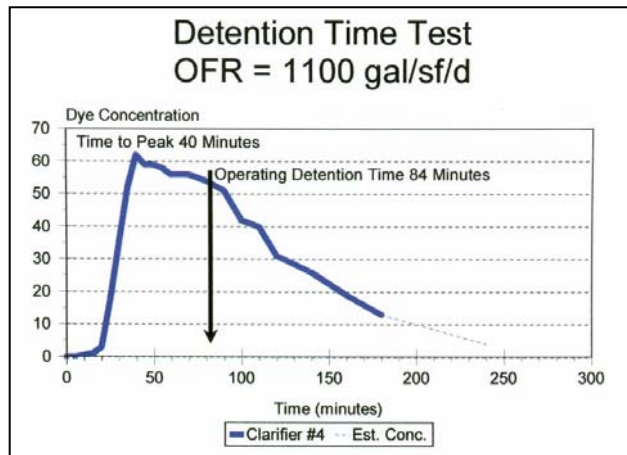


Figure 13

$\text{gal/sf/d} / 588 = \text{m/h}$

The difference in hydraulic efficiency % between Raleigh (65 to 66%) to Kenosha's (80 to 98%) can probably be explained by the extremely higher sludge blanket level in the Raleigh test.

RESULTS AND DISCUSSION

The two (2) studies reported in this paper verifies the original work done by Crosby in the later 1970's that the PF/P clarifier acts as an upflow clarifier and "needs little or no modifications to its design."

The inlet/outlet arrangement, as shown by the drogue test, dye tracer test, and detention time test, allows the influent flow to move away from the effluent weir and impinge upon its self, thus negating the typical "Wall Effect" found in other activated sludge secondary clarifiers. Short-circuiting, inherit on all clarifiers, is greatly reduced as a result of the flow pattern, thus allowing for a high overflow rate, by utilizing the total basin volume and floor area. In fact, a higher overflow rate reduces the short-circuiting factor by driving the density current further and faster into the basin and away from the weir.

The other important aspect of activated sludge final clarifier is the handling of the settled sludge, this has also been addressed in these two studies. Boyle reports, "If the sludge inventory is controlled so that it does not cause gross carry-over of solids into the effluent, and there is a reasonable solid separation zone in the final clarifier, then overflow rates should be used to determine the size of the final clarifier." In these studies, the North Carolina plant was not designed to physically remove the settled solids at a mass loading of more than 50 lbs./D / ft² due to the return pumps limitations. The Wisconsin plant had no problem keeping a low sludge blanket level at all the tested flows. The removal device (Fig. 14) picked up the settled sludge in a clean uniform bite (Table II) as it rotated in the basin with out any adverse effect on the effluent quality. The removal device also allowed for total utilization of the floor area by removing the sludge uniformly across the basin radius.

SLUDGE HEADER

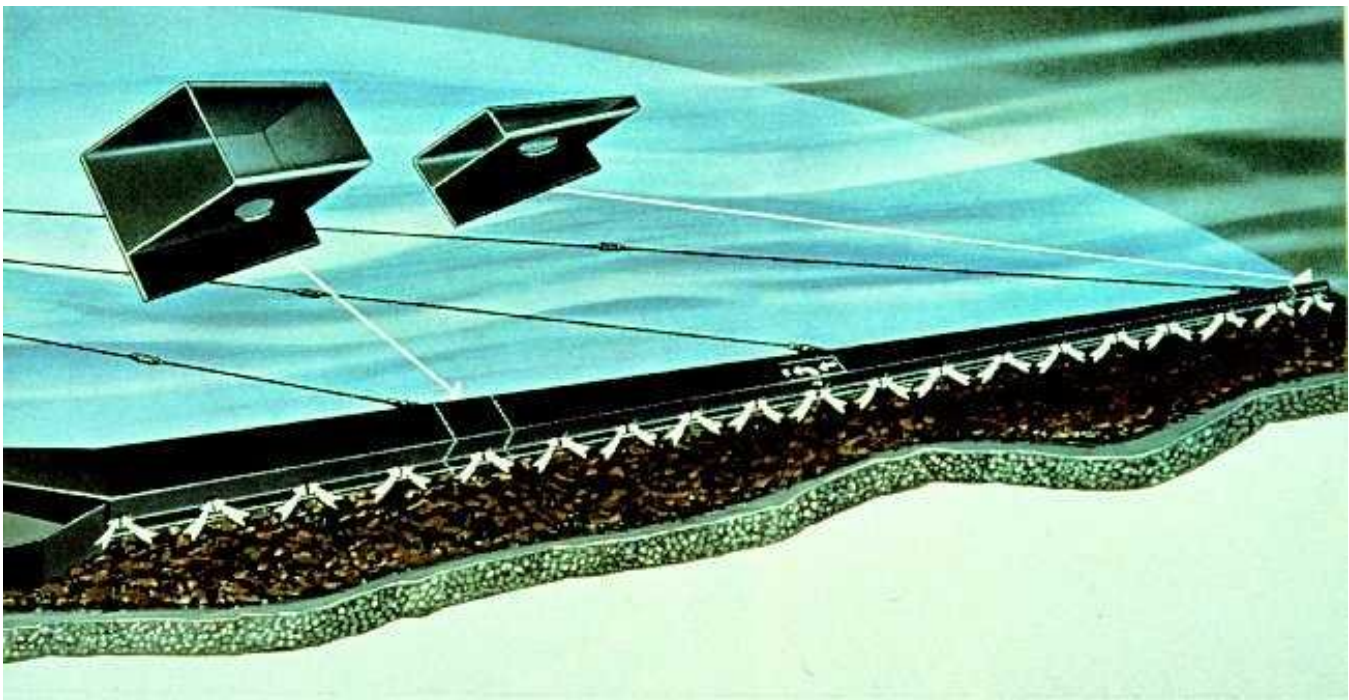


Figure 14

The two (2) “new” studies also reinforce the work reported by the ASCE task committee on Final Clarification that stated the peripheral-feed clarifier “based on the superior hydraulic efficiency of peripheral-feed tanks, it is feasible to allow higher surface overflow rates. (Up to 2,000 gal/day/sq. ft. (80 m³/day/m²).

CONCLUSION

It was possible to maintain, over an eight (8) hour period, an overflow rate of up to 1,300 GPD/sq. ft. (2.21 m/h) with a solids loading of up to 50 lbs./sq. ft./day (243 kgs/sq. m/day).

Results of the high load test show a hydraulic solids loading efficiency well above the conventional design parameters. Solids studies show effluent suspended solids (ESS) levels can be maintained below 20 ppm at hydraulic loading up to 1,300 GPD/sq. ft./day (2.21 m/h). Effluent SS levels below 15 ppm were achieved at levels of 800 GPD/sq. ft. (1.36 m/h).

It appears the existing clarifiers have a capacity of at least 2 times more than the design limits.

ACKNOWLEDGEMENTS

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SLUDGE BLANKET

The location of the sample stations is reported as distance from the center of the clarifier in feet.

8/8 (1100 gal/sf/d) (1.87 m/h)

@ 10:15					@ 10:30					@ 12:00					@ 15:25				
Depth	9'	27'	45'	63'	Depth	9'	27'	45'	63'	Depth	9'	27'	45'	63'	Depth	9'	27'	45'	63'
-1'				0.05	-1'	0.25	0.39	0.21	0.35	-1'	0.24	0.19	0.20	0.15	-1'	0.15	0.20	0.23	0.33
-2'				0.07	-2'	0.82	0.40	0.28	0.46	-2'	0.30	0.19	0.24	0.15	-2'	0.26	0.24	0.40	0.37
-3'				0.09	-3'	0.29	0.45	0.30	0.48	-3'	0.24	0.21	0.26	0.27	-3'	0.23	0.35	0.47	0.45
-4'				0.11	-4'	0.33	0.46	0.34	0.51	-4'	0.38	0.21	0.38	0.32	-4'	0.29	0.33	0.48	0.49
-5'				0.14	-5'	0.34	0.49	0.36	0.52	-5'	0.41	0.30	0.41	0.39	-5'	0.29	0.37	0.53	0.51
-6'				0.12	-6'	0.37	0.51	0.39	0.54	-6'	0.52	0.35	0.47	0.40	-6'	0.38	0.37	0.57	0.52
-7'				0.15	-7'	0.40	0.52	0.44	0.57	-7'	0.57	0.48	0.51	0.49	-7'	0.39	0.40	0.65	0.55
-8'				0.32	-8'	0.55	0.72	0.72	1.20	-8'	0.63	0.81	0.69	1.02	-8'	0.43	0.50	0.82	1.24
-9'				0.45	-9'	0.53	0.72	1.11	1.20	-9'	1.20	0.70	0.88	1.30	-9'	0.43	0.52	0.90	1.20
-10'				0.45	-10'	1.70	1.20	1.33	1.40	-10'	1.40	1.00	1.39	1.24	-10'	0.55	1.00	0.90	1.00
-11'				2.50	-11'	2.40	4.80	2.50	4.00	-11'	2.75	4.10	4.20	1.70	-11'	5.40	5.20	3.50	6.00
-12'				E	-12'	6.50	E	9.00	E	-12'	7.00	E	6.80	E	-12'	E	8.90	9.00	E
-13'					-13'					-13'					-13'				

8/9 (800 gal/sf/d) (1.36 m/h)

@ 9:15					@ 10:30					@ 14:30				
Depth	9'	27'	45'	63'	Depth	9'	27'	45'	63'	Depth	9'	27'	45'	63'
-1'	0.32	0.21	0.29	0.22	-1'	0.15	0.18	0.00	0.11	-1'	0.34	0.14	0.16	0.13
-2'	0.27	0.25	0.15	0.24	-2'	0.29	0.25	0.08	0.14	-2'	0.33	0.18	0.11	0.22
-3'	0.37	0.27	0.31	0.26	-3'	0.30	0.27	0.10	0.17	-3'	0.35	0.19	0.11	0.22
-4'	0.40	0.30	0.37	0.27	-4'	0.34	0.33	0.14	0.23	-4'	0.37	0.22	0.24	0.27
-5'	0.41	0.30	0.39	0.29	-5'	0.39	0.37	0.21	0.24	-5'	0.39	0.23	0.32	0.28
-6'	0.42	0.32	0.42	0.31	-6'	0.43	0.45	0.25	0.26	-6'	0.40	0.26	0.38	0.29
-7'	0.44	0.34	0.47	0.35	-7'	0.45	0.56	0.30	0.29	-7'	0.43	0.35	0.41	0.31
-8'	0.46	0.38	0.59	0.61	-8'	0.58	0.69	0.46	0.70	-8'	0.74	0.45	0.63	0.77
-9'	0.56	0.39	0.74	0.91	-9'	0.61	0.75	0.70	0.79	-9'	0.86	0.50	0.82	0.90
-10'	0.52	0.53	0.85	0.82	-10'	0.63	0.84	0.92	0.84	-10'	0.80	1.40	1.17	0.96
-11'	0.85	0.38	0.91	0.80	-11'	3.00	2.36	0.81	0.81	-11'	2.42	3.80	3.43	4.95
-12'	3.80	4.00	8.00	7.00	-12'	6.50	6.15	8.80	7.20	-12'	7.07	5.80	8.22	E
-13'					-13'					-13'				

Ft. x 0.3048 = m

Table I

b. Solids Profiles Before and After Sludge Header

8/8 @ 1100 gal/sf/d: 45' from Center		Depth	63' from Center	
Before	After		Before	After
0.55	0.60	-8'	1.08	1.14
1.00	0.60	-9'	1.15	1.13
1.10	1.00	-10'	1.25	1.12
1.90	1.00	-11'	1.20	1.00
7.50	3.40	-12'	9.99	1.25

Ft x 0.3048 = m

Gal/sf/d ÷ 588 m/h = m/h

8 @ 1100 gal/sf/d: 45' from Center		Depth	63' from Center	
Before	After		Before	After
1.13	1.08	-8'	1.38	1.45
1.24	1.35	-9'	1.40	1.60
1.50	1.40	-10'	1.50	1.65
6.00	1.40	-11'	3.50	1.60
9.99	2.30	-12'	9.99	1.60

8/10 @ 800 gal/sf/d: 45' from Center		Depth	63' from Center	
Before	After		Before	After
0.80	0.27	-8'	0.46	0.60
0.81	0.41	-9'	0.96	0.55
0.90	0.54	-10'	0.90	0.80
4.17	0.62	-11'	3.63	0.90
8.03	0.63	-12'	9.99	2.50

Table II