FIELD STUDY OF PERFORMANCE OF RECTANGULAR CHLORINE CONTACT TANKS

by

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TABLE OF CONTENTS

Page

LIST OF FIGURES

LIST OF TABLES

I. INTRODUCTION

The chlorination of treated and partially treated sewage effluents has played an important part in helping break the cycle of the transmission of waterborne diseases. Most waterborne pathogens have a high resistance to natural disinfectants occurring in watercourses. The artifical disinfection of these pathogens before they reach the rivers and streams reduces the possibility of these organisms entering water supplies and completing the disease cycle.

The treatment of sewage in the United States began around the year 1880[1]. Chlorine was introduced as a disinfectant around 1887, but its use was limited until the early 1900's[2]. With the introduction of compressed chlorine gas in 1907 and propietary chlorinators in 1912-13, chlorination became much more economical and practical[2].

Though chlorine has been the most popular disinfectant for use in wastewater treatment, there are many other disinfecting agents which have been and are still being investigated and employed. Other halogens such as iodine and bromine provide adequate disinfection, but high cost generally prevents their use on a large scale basis. Ozone and chlorine dioxide also provide very good disinfection, but storage and production costs in most cases make them less desirable than chlorine[3, 4]. Though chlorine may not be at the top of the list as the most efficient disinfecting agent, its other properties make it the most desirable to use.

The method and point of application of the disinfectant is as important as the type of disinfectant being used. The theory of sewage disinfection notes the importance of having the disinfecting agent evenly dispersed throughout the fluid and having the fluid stay in contact with the disinfecting agent for a prescribed amount of time. This principle is very important in the inactivation of pathogens, since a large part of the disinfection occurs at the initial contact of the pathogen and disinfectant. Satisfying these two considerations is the major goal of designing an adequate disinfection unit.

In most cases the chlorine contact tanks of wastewater treatment plants are designed on a plug flow basis. Plug flow implies that the fluid particles enter the tank at the same time, travel parallel paths at the same rate through the tank, and exist at the same time. The detention time of the plug flow unit is calculated by dividing the volume of the unit by the average flow rate. This theoretical state of plug flow very seldom exists, mainly due to adverse currents present in most tanks.

It is suspected in many chlorine contact units that there is a high degree of short circuiting. Short circuiting can be defined as occurring when the fluid particles do not remain in the unit for the designated plug flow detention time. This condition is normally due to slip streaming or channeling of the fluid on the surface or at any depth of the tank depending on the tank configuration and the flow. The inlet and outlet locations and configurations can also have an influence on short circuiting.

The point of chlorine application can be very important in determining the chlorine distribution in the tank and also the effectiveness of the initial mixing of the fluid particles and disinfectant. In some cases, the chlorine is applied with diffusers as the fluid enters the tank, but in other cases initial mixing of the fluid and disinfectant occurs before reaching the tank. The initial mixing can be accomplished by mechanical mixing or by fluid motion. As stated before, the initial mixing of the pathogens and the disinfectant is very important in determining the deactivitation of the pathogens.

Chlorine contact tanks are usually of two basic shapes, rectangular and circular, each having several variations. This thesis will be concerned only with rectangular units. All units studied were designed on a plug flow basis, but differ in hydraulic configuration by the addition of various baffles. The three types of units investigated were unbaffled units, end-around baffled tanks and over-and-under baffled units. End-around baffled tanks direct the flow back and forth across the width of the tank while over-and-under baffles direct the flow up and down through the height of the tank. Both types of baffles are for the purpose of using the entire volume of the unit in order to reduce the amount of short circuiting.

A chlorine demand is produced by sewage, thus reducing the amount of measurable chlorine at any time following application. The level of measurable chlorine at any time is termed the chlorine residual. The longer the sewage remains in contact with the chlorine or the higher the concentration of the sewage, the greater the chlorine demand. It may be concluded that in any areas of the units where there are heavy solids accumulations, dead spots, or areas of little flow, the chlorine demand will be the highest. By measuring the chlorine residual at different points throughout each unit, it is thought that areas of short circuiting, showing high chlorine residuals, can be differentiated from dead spots, showing low chlorine residuals.

The two most common methods of measuring chlorine residual are the Amperometric Titration method and the Orthotolidine method. Amperometric titration is a measure of chlorine residual by a oxidation-reduction procedure, while the orthotolidine method uses a colorimetric process[(3)]. There is some discrepancy in the chlorine residual indicated by each of these methods, though the amperometric method is generally considered the more accurate. The amperometric titration method was used for collection of data since it can also be used for indication of free chlorine residuals while the orthotolidine method can not. During this study a comparison was also made between the two analytical methods.

It is therefore the purpose of this thesis to: 1) study and compare the efficiencies of several types of rectangular chlorine contact units by measuring the various chlorine residuals at many points throughout the tanks, 2) compare the distribution of chlorine in the units with tank design and the type of initial mixing being used, and 3) make a comparison of the amperometric titration method and orthotolidine method for measuring chlorine residual.

II. LITERATURE REVIEW

Chlorine Chemistry of Wastewater

In wastewater treatment, chlorine is generally used for disinfection purposes in either the form of free chlorine or as a hypochlorite salt[3]. Both forms are strong oxidizing agents and both react identically as disinfection and odor control agents[3, 6]. At small installations the choice of form of chlorine is usually based on local preference, but at large operations free chlorine is normally found optimal because of its low cost, storage capability and availability in large quantities.

On initial contact of free chlorine and hypochlorite salts with water, different reactions occur, though the same end products result. With free chlorine, the following reaction results:

$$Cl_2 + H_2O \rightleftarrows HOCl + H^+ + Cl^-$$

This is a very rapid reaction, which is slightly reversible, but very seldom does any measurable free elemental chlorine exist after a few seconds of contact with water. Only if the pH is below 3.0 or if the dosage is in the range of 1000 mg/l will any measurable free elemental chlorine remain in solution[7].

Chlorine as hypochlorite is usually used in the form of Sodium hypochlorite (NaOCl), which is normally available in liquid form, or as Calcium hypochlorite ($Ca(OCl)_2$) which usually is sold in a powder form. When a hypochlorite is mixed with water, using calcium hypochlorite as an example, the following solubilization reaction occurs[3, 7]:

$$Ca(OCl)_2 \rightarrow Ca^{++} + 2OCl^-$$

Both forms of chlorine after being introduced to water eventually must satisfy the expression for hypochlorous acid equilibrium. In the case of free elemental chlorine, hypochlorous acid is formed from the reaction with water, and with either of the hypochlorite compounds the hypochlorite ion is released. In either case, the following equilibrium equation is satisfied in accordance with the pH of the solution[3]:

$$HOCl \rightleftarrows H^+ + OCl^-$$

From approximately pH 4.0 - 6.0, 95 to 100 per cent of the chlorine will exist as hypochlorous acid and at pH 9.0 - 11.0, 95 to 100 percent of the chlorine will be in the hypochlorite ion form[3]. It should be noted that the addition of free elemental chlorine to water normally lowers the pH due to the release of hydrogen ions and the addition of hypochlorite compounds increases the pH, though hypochlorites seldom have a great effect on the pH unless they are added in large doses[3].

Another important reaction which normally occurs in wastewater chlorination is the formation of chloramines. Unless nitrification has occurred in previous treatment, ammonia usually is present in concentrations in the range of 12 - 50 mg/l as N in sewage effluents. An average concentration of ammonia in effluents is 25 mg/l as N[8].

The formation of chloramines results from the reaction of hypochlorous acid and ammonia. There are three types of chloramines, monochloramine (NH_2Cl), dichloramine ($NHCl_2$) and trichloramine or nitrogen trichloride (NCl_3). The formation of the chloramines normally follows the following reactions, but pH, ammonia concentration, temperature and contact time effect greatly the rates of these reactions as well as the distribution of end products[9].

$$HOCl + NH_3 \rightarrow NH_2Cl + H_2O$$

$$NH_2Cl + HOCl \rightarrow NHCl_2 + H_2O$$

$$NHCl_2 + HOCl \rightarrow NCl_3 + H_2O$$

The formation of monochloramine occurs at a maximum at pH 8.3 and the rate of reaction drops off with a change in pH in either direction[7, 9]. This rate of reaction is also decreased as temperatures are lowered[7].

The formation of dichloramine is dependent upon pH and the ammonia concentration and is related to the following equilibrium expression[7]:

$$2NH_2Cl + H^+ \rightleftarrows NH_4^+ + NHCl_2$$

From this equation it can be seen that a low pH and high ammonia concentration will favor the formation of dichloramines over monochloramines.

After formation of mono and dichloramines, there exists an equilibrium condition which is pH dependent. At a pH above 8.5 only monochloramines will exist and in the pH range of 4.5 - 5.0 only dichloramines will exist[9].

The formation of nitrogen trichloride generally occurs only at a pH of less than 5.0, though it has been noted to occur at higher pH levels. Nitrogen trichloride has been found at a pH of 7.0 - 8.0 when the chlorine to ammonia ratio exceeds 15:1 by weight[9].

Chlorine also reacts with many other compounds such as ferrous iron, reduced manganese, nitrite, hydrogen sulfide, elemental carbon, cyanide, and organic compounds such as proteinaceous material. When chlorine reacts with the inorganic compounds, chlorides of these compounds result and a reduction in disinfection efficiency occurs since chlorides are of no use as a disinfecting agent. Very little disinfection power also results from products of the reaction of chlorine with proteinaceous material[7,10,11].

Disinfection

As stated by Chang[12], there are five general groups into which disinfecting agents can be classified: 1) oxidizing agents, 2) cations of heavy metals, 3) quaternary ammonium and pyridinium compounds, 4) gaseous agents, and 5) physical agents. Ozone and the halogen group fall under the heading of oxidizing agents.

The killing mechanism of oxidizing agents is accredited to the oxidation and the subsequent disruption of the normal functions of enzyme groups[9, 12, 13]. Most strong oxidizing agents have this effect on pathogens, but the real power of a disinfectant is its ability to penetrate the outer shell of the pathogen, whether it be a cell wall, capsid, or cyst wall. The outer shell of the pathogen is its shield which protects its vulnerable life systems from natural and artifical disinfectants and which makes it able to exist for long periods of time in the environment.

Not all pathogens have the same resistance to disinfecting agents due to the varied ability of the outer shell to withstand penetration. Chang[12] also classified the major pathogens into Cyto Structural groups according to their resistance to disinfectants: 1) bacterial spores, 2) protozoan cysts, 3) viruses, and 4) vegetative bacteria. With the exception of a few viruses, the bacterial spores are generally the most resistant mircoorganisms to inactivation even at high temperatures. This resistance is accredited to the strong spore wall and its formation of D.P.A. (dipicolinic acid) and storage of basic ions. The partial dehydration of proteins in the protoplasm, due to heat, also aids in the resistance to inactivation[12].

Next in the order of resistance are the protozoan cysts, such as Endamoeba histoytica, the cause of amoebic dysentery. These cysts are very resistant to pH changes in the range of 1.0 to 13.0, but are very susceptible to heat and must be surrounded by water to survive. As will be discussed later, the cysts are much more resistant to compounds of chlorine than to free chlorine because of better diffusion of the cyst wall by free chlorine[12].

In the case of enteroviruses, a protein capsid is their protection against oxidizing agents. Enterovirus' higher resistance to chlorine over enterobacteria is generally thought to be due to the lack of enzyme systems of the viruses, though their nucleic acid core is very susceptible to chlorine. In some cases, it has been shown that the denaturation of the protein capsid without destruction of the nucleic acid core has caused deactivation of the virus[12].

The vegetative bacteria are very susceptible to the disinfecting power of chlorine. This lack of resistance is due to the fact that 1) the cell wall is not very strong, 2) respiration takes place on the outside of the cell, and 3) the susceptible enzyme systems are located near to the cell wall. Chlorine first attacks the respiratory system which in turn effects the metabolism of the cell[12].

In trying to make a correlation between the contact time and the number of organisms inactivated in that time period, Chick[8], developed the following familiar first order expression:

$$\frac{N}{N_o} = e^{-Kt}$$

where N_o = original numbers of organisms present,

N = number of organisms after a detention time, t,

and K = constant.

From this equation and experimentation, the constant, K, can be developed for a particular organism, disinfectant and dosage. Having determined the constant, the time required to inactivate a particular number of organisms can be calculated for a particular species and chlorine dosage assuming that Chick's law holds true.

Using bacteria as the organism, Chick's equation gives good results which are reproducable; however, with other pathogens the results vary. Viruses, for example, deviate from the normal linear plot of time versus ln N/N_o which bacteria produce. Even though there is a variation from Chick's equation, it is still often used to give an indication of the nature of the disinfection kinetics for a wide variety of organisms.

An equation similar to Chick's law has been developed which correlates the concentration of a particular disinfectant with contact time as follows(3, 8):

$$C^n t \approx \text{per cent Kill}$$

where C = concentration of disinfection,

t = time, and

N = constant for a particular disinfectant.

Again with experimentation, where a particular per cent kill is chosen, the constant, N, can be determined. With the equation and the constant, N, either time or concentration of chlorine can be determined when the value of the other variables is known.

Both of the above equations are approximations, but they can give an estimate of the relationship of the per cent inactivation with disinfectant concentration and contact time for a particular pathogen.

As mentioned before chlorine is found in one of four states in wastewater 1) free chlorine, 2) monochloramines, 3) dichloramines, and 4) nitrogen trichloride, though the fourth state very seldom exists unless high chlorine and ammonia concentrations are present. Of the four states, free chlorine is the most effective as a disinfecting agent[9, 14]. As stated previously, hypochlorous acid and the hypochlorite ion exist in an equilibrium which is pH dependent. Of these two, hypochlorous acid is much more effective as a disinfectant, mainly because of the neutral charge and low molecular weight enabling it to be easily diffused through the cell wall[19]. The negative charge of the hypochlorite ion causes it to be a less effective disinfectant due to the difficulty in cell wall diffusion[9, 14]. It follows then that the pH of wastewater will have an effect on the disinfecting power of the free chlorine. In the pH range of less than 7.5, the greatest amount of chlorine is present as hypochlorous acid, and since wastewater is normally in the pH range of 6.5 - 7.5, the free chlorine present will give a relatively high efficiency of disinfection[11].

Butterfield and Associates[15], studied the relative disinfecting power of free chlorine and the effect of various pH and temperature levels. Their report concluded that with a pH increase the chlorine required for adequate kill also increased. Higher temperatures increased the efficiency of free chlorine, and time of contact was an important factor in the effectiveness of free chlorine.

Though free chlorine has a very high disinfecting power, it very seldom exists in chlorine contact tanks in any high concentration for long periods of time because of its high oxidation power and the amount of ammonia present in treated sewage effluents. Free chlorine has been shown to exist up to 60 minutes[3], though this normally indicates breakpoint chlorination has occurred.

Chlorine in the form of monochloramines and dichloramines comprise most of the chlorine present in wastewater chlorine contact tanks. As mentioned previously, mono and dichloramines exist in an equilibrium which is pH dependent. At a pH of approximately 6.7 mono and dichloramines exist at equal concentrations which would indicate that for normal wastewater they would be present in almost equal amounts with a slightly higher concentration of monochloramines[9].

In another study in 1946, Butterfield and Wattie[16] investigated the efficiency of chloramines as a disinfectant and the effects of pH and temperature variations. Again their first basic conclusion was that contact time is the single most important factor in determining efficiency. Variations in pH indicated that at higher pH levels a decrease in disinfection efficiency resulted. This result would seem to indicate that dichloramines provide better disinfection than monochloramines. The effect of temperature was the same as for free chlorine in that a decrease in temperature meant a decrease in disinfection efficiency.

In a later report, Butterfield[17], combined his previous studies and made a comparison between the efficiencies of free and combined available chlorine. As in both previous reports, high temperatures and low pH levels are the most favorable system conditions. For wastewater at pH 7.0 and 20 - 25°C, Butterfield's studies showed combined chlorine required was 1.2 ppm at 20 minutes contact while only .04 ppm of free chlorine for one minute was needed to provide a 100 per cent kill of bacteria. This work shows that 25 times as much combined chlorine as free chlorine is required to produce a 100 per cent kill at the same detention period, and for equal dosages, combined chlorine required a 100 times longer contact period than free chlorine. It should be noted that Butterfield used the orthotolidine method for measuring residuals.

In a similar report, Morris[4], stated that for equal virus inactivation in 30 minutes 25 mg/l chloramines was required, while approximately 100 mg/l hypochlorite ion and 0.5 - 1.0 mg/l hypochlorous acid were needed.

The type of treatment preceding the chlorine contact tank can have a great effect on the amount of chlorine required to provide adequate disinfection. The higher the degree of treatment the smaller the required dosage of chlorine due to reduction of solids and organic matter which consume large quantities of chlorine.

Rhines[10] reported that the clumping of solids presented a serious problem in providing adequate disinfection. With clumping of solids only the outer layers of the clumps may be reached by chlorine in normal detention periods, and any pathogens inside of the clumps are protected against inactivation.

Lothrop and Sproul[18] conducted tests to determine chlorine dosages required for primary and secondary effluents to provide 99.9 per cent and 99.99 per cent inactivation of T2 bacteriophage and Type 1 poliovirus. Their data was collected at system conditions optimum for wastewater disinfection, including pH 7.0 and a temperature of $20^{o}C \pm 5^{o}C$. Amperometric titration was used to determine chlorine residuals and dosages. In their first test the T2 bacteriophage was used as the indicator organism. In primary sewage effluent (2 hours settling) chlorine dosages of 10 - 50 mg/l were used, but combined chlorine residuals of 18 - 24 mg/l for 30 minutes were required for 99.9 per cent inactivation and 28 mg/l for 30 minutes for 99.99 per cent inactivation. With secondary effluent and chlorine dosages of 4 to 6 mg/l, free chlorine residuals of 0.2 - 0.5 mg/l were measured and 100 per cent kill resulted in 30 minutes. The Type 1 poliovirus appeared to be much more resistant to chlorine than the T2 bacteriophage. In primary sewage effluent, at dosages of 10 - 70 mg/l, 30 - 37 mg/l combined chlorine residuals were required for 99.9 per cent inactivation and 33 - 43 mg/l for 99.99 per cent inactivation after 30 minutes. Higher dosages of chlorine, 11 - 16 mg/l, were required to produce free chlorine residuals in the secondary effluent because of the increased ammonia concentrations. In the first test for bacteriophage the ammonia was in the range of 0.97 mg/l, but in the second test for poliovirus the ammonia ranged from 1 - 2 mg/l. With a free residual produced in the second test, a 100 per cent inactivation was found after 30 minutes.

Heukelekian and Smith(19) also conducted tests using primary and secondary effluents, but instead of viruses they used coliforms as an indicator organism. Their results were very similar to Lothrop and Sproul's except much lower dosages of chlorine were required. For coliform counts of 1 mg/l at 30 minutes detention time, a 3.2 mg/l dosage for primary effluent was required and a 1.53 mg/l dosage was needed for secondary effluent. Amperometric titration was also used to measure chlorine dosages.

From both of the above studies it was concluded that the type of effluent determines the required chlorine dosage. It can also be seen that viruses require much higher dosages than coliforms for inactivation. Huekelekian and Smith also concluded that initial coliform counts could not be used to predict a chlorine dosage that would provide a certain effluent coliform count.

Hydraulics of Chlorine Contact Tanks

In most investigations of the hydraulic efficiency of chlorine contact tanks, tracer studies have been conducted to determine the minimum detention time of the unit as measured by the first appearance of the dye or tracer salt in the effluent. The mean detention time is taken to be indicated by the centroid of the area located beneath the curve of the graph of time versus tracer concentration[19]. The Morril Index is also used to indicate the hydraulic efficiency of chlorine contact units. This index is based on the following equation:

$$\text{Morril Index} = \frac{t_{90}}{t_{10}}$$

where t_{90} = the time for 90 per cent of the tracer to pass the sampling point, and

t_{10} = the time for 10 per cent of the tracer to pass the sampling point[20].

If theoretical plug flow conditions exist, the Morril Index is equal to 1.0. As the Morril Index number increases the amount of short circuiting increases[20].

In a study conducted by Marske and Boyle[21], seven types of chlorine contact tanks were investigated using dye studies. The units consisted of circular and baffled and unbaffled rectangular tanks. The results of their study showed that wind had a great influence in increasing short circuiting in shallow units. Of the various outlet weir configurations, these investigators found that a sharp crested weir which extended across the entire width of the tank allowed the least amount of short circuiting. In a comparison of baffled units, narrow longitudinal baffles provided better efficiency than end-around baffling since the length to width ratio was much higher and there were fewer corners for the flow to turn. The authors noted that the greater the number of turns the flow must make, the greater the number of dead spaces created. It was also found that a length to width ratio of 40:1 was required to produce maximum plug flow conditions. After testing the seven types of units, the investigators concluded that a longitudinal baffle unit preceded by a secondary clarifier with an annular ring gave the best efficiency.

Louie and Fohrman[19] studied an over-and-under baffled mixing unit followed by a modified maze-type baffled chlorine contact unit. The first unit which provided very good mixing consisted of a chlorine diffuser followed by an underflow baffle and then an overflow baffle. The modifications to the tank included the addition of fillets in the corners, hammerheads placed at the end of each baffle and flow vanes placed in the stream of flow near the end of each baffle. Using the plain baffled tank as a control, several combinations of the modifications were tried. Results showed that the unit having only the directional vanes with 90^{o} extentions at the end of each baffle provided the best efficiency. The addition of the hammerheads helped reduce the amount of back flow and dead spaces and the fillets reduced solids accumulation in the corners. However, the directional vanes provided a unit with the least amount of short circuiting.

Solids accumulation has always been a problem in baffled units since mechanical sludge removal equipment can not be installed. In a study by Kothandaraman and Evans[22], it was found the air agitation in the unit prevented any solids problem. The addition of the air agitation also improved the flow characteristics of the baffled units without any loss of chlorine in a normal 30 minutes detention period. Better pathogen-chlorine contact and superior mixing were also provided.

In a comparative study of circular and rectangular chlorine contact units, Sawyer[5] found that circular units provided better mixing characteristics, but had a high degree of short circuiting. With the rectangular baffled units, short circuiting was reduced compared to the circular unit, but the amount of dead spaces increased.

Chlorine Analysis by Amperometric Titration And Orthotolidine Methods

As stated previously, the orthotolidine procedure of measuring chlorine residuals is a colorimetric method while amperometric titration is an oxidation-reduction procedure. With the orthotolidine method, an aromatic organic compound, orthotolidine, is oxidized by chlorine and its various compounds to produce a yellow-colored solution. This yellow color is compared to color standards to determine the chlorine residual. Though this is a very convenient and easy test to run, there are many interferences which give false readings. The amperometric titration method uses an electronic meter with a rotating platinum electrode to measure the extent of the oxidation-reduction reaction. A reducing agent, Phenylarsenoxide, is used as the titrant to reduce the available chlorine. This method also measures the amount of each chloramine present as well as the free chlorine, while the orthotolidine method only measures total chlorine. The amperometric method is also susceptible to interference, but not to the extent noted for the orthotolidine method[(3)].

In a study by Huekelekian and Day[(23)], a comparison of orthotolidine and amperometric titration was completed. Results showed that a residual of 1.2 - 1.6 mg/1 of chlorine was required to register a positive reading with orthotolidine. In most cases these investigators found that 2 mg/1 of chlorine measured by the amperometric method equaled 1 mg/l by the orthotolidine method. In a comparison based on MPN's, they found that a MPN of 2.5 could be produced from an othotolidine residual of 0.2 - 0.5 mg/l chlorine at 15 minutes as compared to an amperometric residual of 2.0 mg/l required to produce the same MPN level.

Burns and Sproul[24] in their work found that residuals measured by the orthotolidine method could not be related to the inactivation of viruses while the residuals determined by amperometric titration could be correlated with the extent of virus inactivation.

Summary

From this review of the literature it may be concluded that there are many factors which influence the efficiency of chlorine disinfection. From the chemical stand point, the form of the chlorine is important, as the disinfection power varys greatly for the different species of active chlorine. Dosage and contact time are also critical factors. An adequate concentration of chlorine must stay in contact with the pathogen for a sufficient period of time to provide inactivation. When hydraulic short circuiting occurs, the detention period is reduced, and this reduction requires higher chlorine concentrations to bring about adequate inactivation of pathogens. The initial mixing of the chlorine and liquid is also very important in providing an even distribution of chlorine throughout the liquid. Therefore, hydraulic detention time, the dosage and form of the chlorine, and initial mixing of the chlorine and liquid are all very important factors in providing adequate disinfection.

The purpose of this study is to evaluate several operating systems containing rectangular chlorine contact tanks, to gain an understanding of their actual performance with respect to the factors previously noted, and to postulate the disinfection efficiency of the units under study.

III. METHODS AND MATERIALS

Analytical Procedures

In the collection of data, two methods of measuring chlorine residuals were used. The orthotolidine method was used for measurement of total chlorine and the amperometric titration method was used to measure both free and total chlorine.

For measuring the total chlorine with the orthotolidine method, a Wallace and Tiernan, Incorporated color comparator was used. A standard 15 ml sample was mixed with approximately 1.0 ml of orthotolidine solution. The comparator disc had a range of 0.15 mg/l to 2.0 mg/l and therefore any readings greater than 2.0 mg/l were recorded as + 2.0 mg/l and any readings less than 0.15 mg/l were recorded as a trace. Readings were taken approximately 5 minutes after the orthotolidine was added to the sample.

A Wallace and Tiernan Amperometric Titrator was used for measurement of free and total chlorine. In both tests, a standard 200 ml sample was used and .00564N phenylarseneoxide was used as the titrant. For free chlorine residuals, the samples were buffered to pH 7.0 with 1.0 ml of pH 7.0 buffer. The total chlorine samples were buffered to pH 4.0 with 1.0 ml of pH 4.0 buffer and then 1.0 ml of 0.301 N potassium iodide was added. In both tests 1.0 ml of titrant is equivalent to 1.0 mg/l of chlorine. For free chlorine residuals, the titrant was added until either an end or null point was reached by the indicator needle or until each addition of titrant, the indicator needle returned to its original position. Titrant was added in the total chlorine test until the end or null point was reached by the indicator needle. All reagents were prepared according to Standard Methods for Examination of Water and Wastewater[25].

Experimental Procedures

At each treatment plant, chlorine contact tank sampling points were defined by use of a grid system. The number of samples per tank varied according to the size and configuration of the tank. The vertical distribution of samples was the same for each tank. Samples were taken at 6 inches below the water surface at approximately half the depth of the tank and at the bottom of the tank. In several cases where sludge blankets at the bottom of the tank prevented reaching the bottom of the tank with the sampler, samples were taken just above the sludge blanket.

In the unbaffled units, a row of samples was taken approximately one foot from each sidewall and another row of samples was taken down the middle of the tank. Three to four sample points were chosen on each of these rows depending on the length of the tank. Normally samples were taken approximately one foot from the influent wall and one foot from the effluent weir with one sample near the middle of the tank or samples at the one-third and two-third points along the length of the tank.

For the end-around baffled tanks samples were taken at the three depths as mentioned before, with two sampling points in each bay. One set of samples was taken at the entrance to each bay and a second set was taken at the end of the bay where there appeared to be a dead spot due to back flow in the corner. The sampling point at the beginning of each bay was selected at the position where the flow appeared the greatest. This point would then give the chlorine residual leaving the previous bay and entering the next bay. The second sample was taken near the corner, at the end of the bay in the back flow area. This position would indicate if dead spots or sludge deposits were present.

The sampling procedure for the over-and-under baffled tank was similar to the unbaffled tank except that samples were taken before and after each baffle. Since the one tank of this type sampled was narrow, only two rows of samples were taken, each approximately 3.33 feet from the sidewall with approximately 3.33 feet between the rows.

The samples were taken in a 500 ml polyethylene flask attached to a ten foot metal rod which was marked off at one foot increments. A stopper arrangement was made for the flask so that a sample could be taken at any depth without allowing liquid to enter the collection device while lowering or removing the sampler.

One set of samples was taken for each selected installation. All analyses were run immediately in the field and all tests from a selected sampling point were run on the same sample. The completion of a run required approximately four to six hours.

Analysis of Data

Considerable data was collected for the selected installations concerning hydraulics of the tanks, type of treatment provided, and effects of initial mixing and sludge deposits. Calculations were completed to determine the dosages used and the applied dosage and the residual throughout the tank were compared. The various types of initial mixing were evaluated by comparing the maximum and minimum residuals to the average residual at the inlet of each tank.

To determine the hydraulic efficiency of each tank, the maximum, minimum, and average residuals, for free and total chlorine, were compared with their location in the tank. These residuals indicated the areas of mixing, short circuiting, and sludge deposits. The standard deviation for free and total chlorine residual in each tank was also calculated to aid in determining tank efficiency.

To show the distribution of free and total chlorine in the tanks, diagrams of the tanks at the three sampling depths were drawn. At each sampling point on the diagrams, the free and total chlorine residual were indicated.

IV. DESCRIPTION OF TREATMENT PLANTS SELECTED FOR STUDY

Five sewage treatment plants, each containing rectangular chlorine contacts, were selected for study. The following paragraphs summarize briefly the important data regarding the nature and operation of each of these plants.

Plant No. 1, Stroubles Creek Plant, Blacksburg, Virginia. The Stroubles Creek plant provides secondary treatment and is designed for an average flow of 2 mgd. Treatment includes grit collection and grease removal followed by primary settling, a trickling filter and secondary settling. Dual chlorine contact tanks each with a volume of 2786 cubic feet and a detention time of 15 minutes are provided. They are unbaffled tanks with sludge removal equipment included. Chlorine is applied at a manhole preceding the splitter box which divides the flow between the dual chlorine contact tanks. A distribution wall is also provided at the influent to aid in distribution of the fluid.

For the month in which the sampling was conducted, the following average influent values were recorded: BOD = 271 mg/1, suspended solids = 389 mg/1 and pH = 7.4. Effluent characteristics were as follows: BOD = 28 mg/1, suspended solids = 17 mg/1 and pH = 7.2. The flow for the same month averaged 2.86 mgd.

Plant No. 2, Old Plant, Christiansburg, Virginia. The Christiansburg Treatment Plant No. 1 was one of the older plants studied. In this facility secondary treatment is provided.

Christiansburg has two plants with a large portion of the flow going to the new plant and the flow to the No. 1 plant averaging 0.197 mgd. The design flow for this plant was not available. Treatment facilities consist of an Imhoff tank followed by a trickling filter with a rectangular secondary clarifier also serving as a chlorine contact tank. The clarifier-chlorine contact tank has an approximate volume of 1690 cubic feet which would provide a detention time of 91 minutes at the average flow of .197 mgd. Sludge collection equipment is provided in the tank. The chlorine is applied in the downstream area of line between the trickling filter and the clarifier-contact tank.

The average influent characteristics for the plant during the month in which the tank was sampled were as follows: BOD = 266 mg/1, suspended solids = 115 mg/1 and pH = 7.1. The effluent characteristics were BOD = 10.5 mg/1, suspended solids = 9.6 mg/1 and pH = 6.9.

<u>Plant No. 3, Narrows, Virginia</u>. The treatment plant at the Town of Narrows is another one of the older plants investigated. The Narrows plant employs primary treatment with dual primary clarifiers followed by an over-and-under baffled chlorine contact tank. Chlorine is applied at the influent of the tank approximately 5 feet from where the flow enters the tank. The tank has a volume of approximately 1184 cubic feet and provides 21.2 minutes detention at the design flow of 0.6 mgd.

During the month in which the samples were taken, the flow averaged 1.4 mgd, which would allow only 9 minutes contact time. The averaged settleable solids of the influent for the month was 6.0 mg/l and the pH was 6.9. Settleable solids in the effluent averaged 1.0 mg/l with a pH of 7.5.

Plant No. 4, Harrisonburg, Virginia. The Harrisonburg treatment plant is designed to handle a flow of 2.25 mgd. Treatment consists of grit removal followed by dual primary clarifiers, two stage trickling filter, dual secondary clarifiers and dual end-around baffled chlorine contact tanks. Chlorine is applied to the splitter box preceding the contact tanks. Each contact tank has an approximate volume of 3200 cubic feet which provides 30 minutes detention time at design flow.

Average flow for the month in which the data was collected was 2.7 mgd. The average influent BOD was 182 mg/l with a pH of 7.1 and suspended solids equal to 109 mg/l. The effluent had a pH of 7.6 with an average BOD of 45 mg/l and a suspended solid concentration of 30 mg/l.

Plant No. 5, Lowmoor, Virginia. The Lowmoor treatment plant is a small plant designed for a flow of 40,000 gpd. Treatment consists of dual extended aeration and settling tanks followed by an end-around baffled chlorine contact tank. Chlorine is applied to the influent line of the contact tank. The contact tank has an approximate volume of 153 cubic feet and provides 41.2 minutes detention time at design flow. There is no continuous flow measurement equipment at the plant and only a V-notched weir on the contact chamber effluent for flow measurement. During the month in which sampling was conducted, an average influent pH of 7.3 and settleable solids equal to 3.0 mg/l were measured. The average effluent data showed a pH of 6.9 and settleable solids of 0.1 mg/l.

V. EXPERIMENTAL DATA AND RESULTS

The data collected at the five selected treatment plants are presented in Figures 1 through 15. Each figure shows a plan view of the tank configuration at a particular depth. The dimensions for each of the five chlorine contact tanks sampled is given in the first figure of each tank. At each point where samples were taken, the free and total chlorine residuals as determined by the Amperometric titration method are indicated with the free chlorine being the upper figure. All residuals are recorded in milligrams per liter.

Table No. I includes several important results calculated from the data collected. For each plant the table indicates the type treatment, design and average flows, applied chlorine dosage, the average, maximum, minimum and standard deviation from the average for the free and total residuals, and the average to minimum and maximum residuals at the inlet.

Figures 1 through 3 show the distribution of forms of chlorine in one of the unbaffled chlorine contact tanks at the Stroubles Creek treatment plant. The chlorine appears to be well mixed with the influent flow and well distributed throughout the tank. Free chlorine was found in all but five samples. The average free and total chlorine residuals

TABLE I

SUMMARY OF CALCULATION AND RESULTS OF DATA COLLECTION

Type Treatment	Stroubles Creek	Christians-burg	Narrows	Harrison-burg	Lowmoor
Type Treatment	secondary	secondary	primary	secondary	secondary
Design Flow (mgd)	2.0	0.197	0.60	2.25	0.04
Average Flow (mgd)	2.86	0.197	1.4	2.7	--
Dosage (mg/1*)	9.56	12.2	1.71	12.2	4.49
Maximum Free Residual	0.38	0.12	0.10	0.15	0.12
Minimum Free Residual	0.0	0.0	0.0	0.0	0.0
Average Free Residual	0.0583	0.0711	0.0037	0.0188	0.0517
Standard Deviation For Free Residual	0.547	0.0292	0.0165	0.0345	0.0315
Maximum Total Residual	3.07	1.68	3.91	2.56	3.13
Minimum Total Residual	1.15	1.29	1.10	0.10	0.0
Average Total Residual	2.4175	1.415	1.555	1.117	2.12

*All dosages and residual in mg/1 of chlorine.

TABLE I (Continued)

Type Treatment	Stroubles Creek secondary	Christians-burg secondary	Narrows primary	Harrison-burg secondary	Lowmoor secondary
Standard Deviation For Total Residual	0.4445	0.0868	0.5402	0.5758	0.6578
Minimum to Average Free Residual at Inlet	0.0	0.0	0.0	0.72	0.66
Maximum to Average Free Residual at Inlet	2.4	2.0	3.33	1.20	1.57
Minimum to Average Total Residual at Inlet	0.74	0.94	0.49	0.68	0.92
Maximum to Average Total Residual at Inlet	1.17	1.19	1.41	1.39	1.09

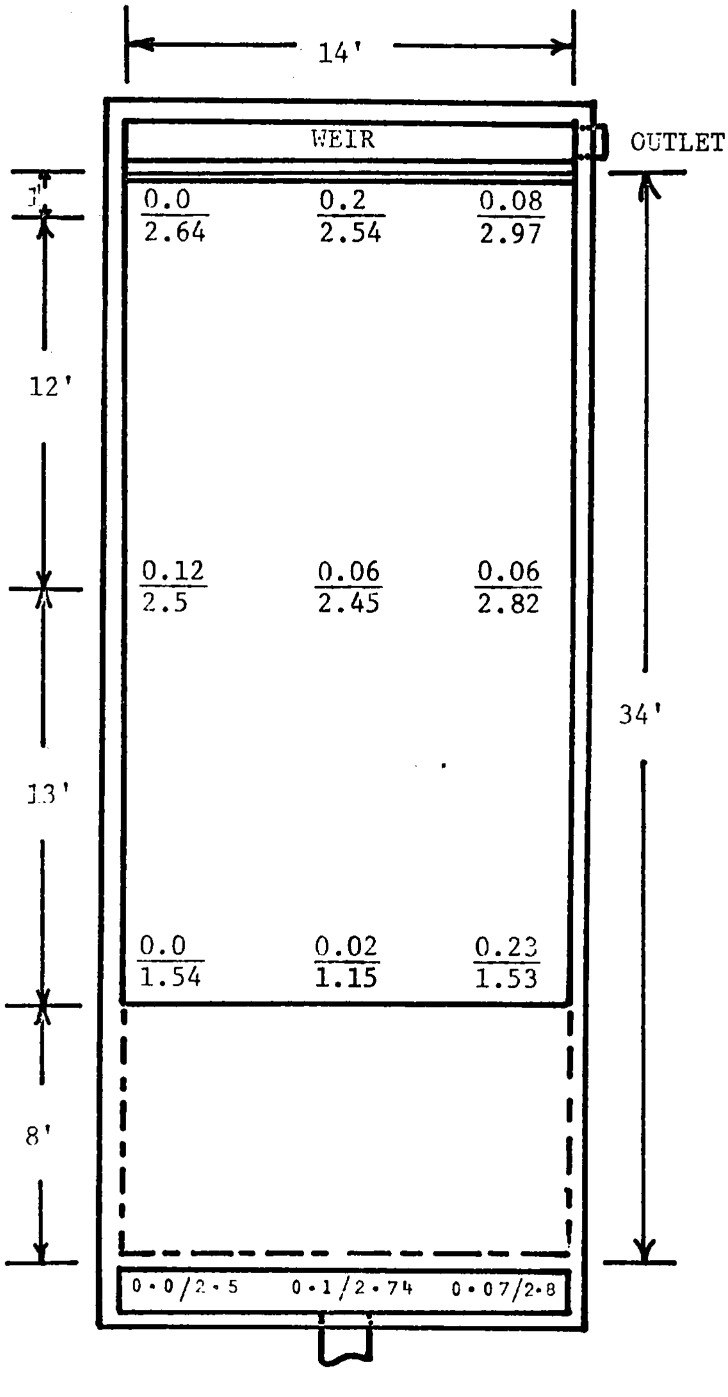

Figure 1 Distribution of Free and Total Chlorine at the Surface in the Unbaffled Stroubles Creek Chlorine Contact Tank

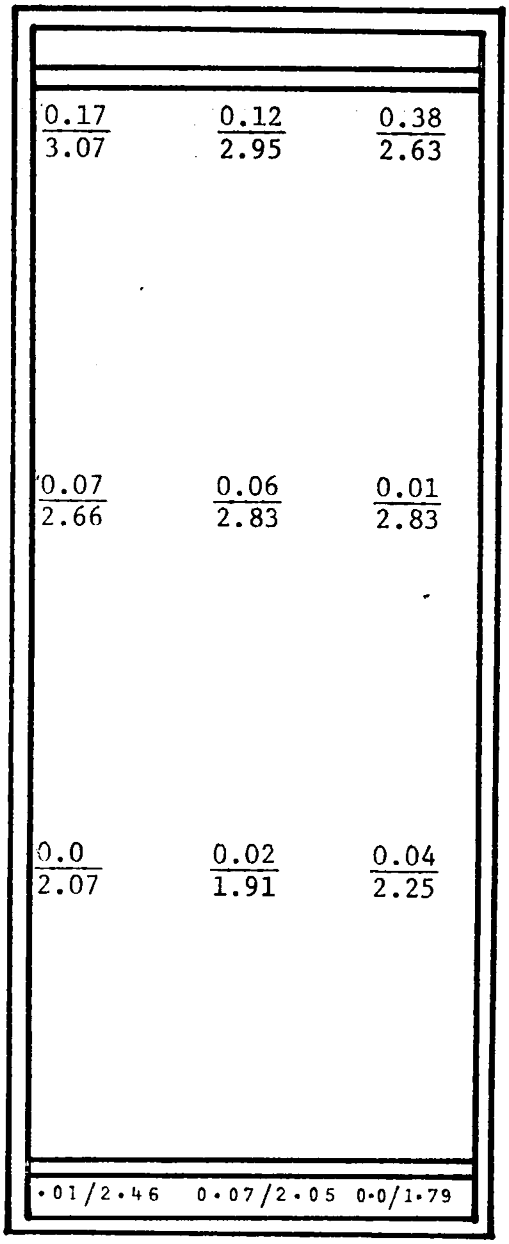

Figure 2 Distribution of Free and Total Chlorine at the Three Foot Depth in the Unbaffled Stroubles Creek Chlorine Contact Tank

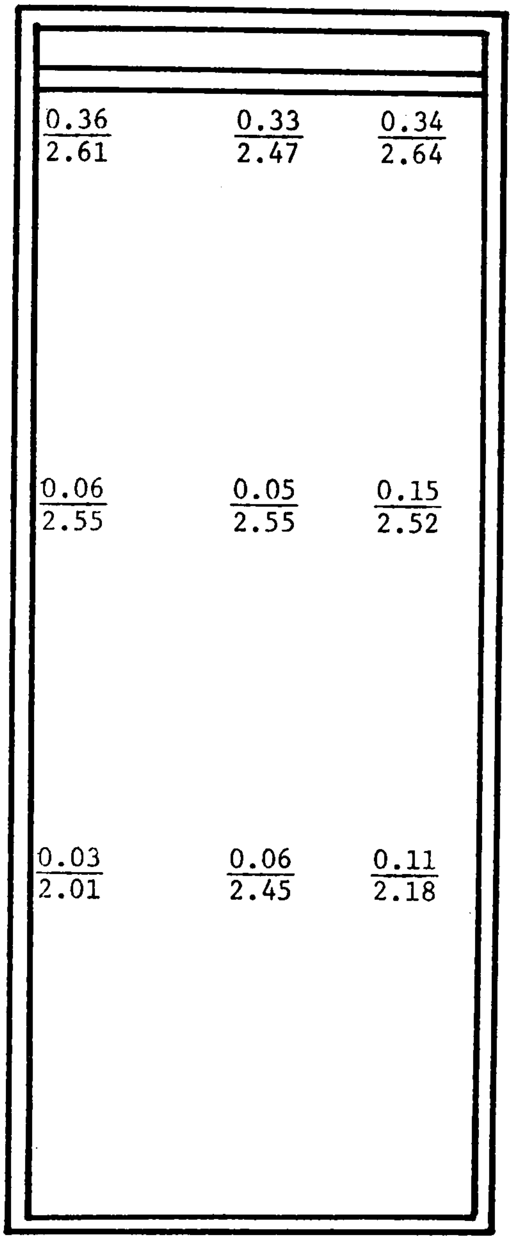

Figure 3 Distribution of Free and Total Chlorine at the 5.5 Foot Depth in the Unbaffled Stroubles Creek Chlorine Contact Tank

were 0.0583 mg/l and 2.4175 mg/l respectively. The standard deviation from the average for the free residuals was 0.0583 mg/l and for the total residuals it was 0.4445 mg/l.

Figures 4 through 6 illustrate the chlorine distribution in the chlorine contact tank at the Old Christiansburg treatment plant. This tank is also unbaffled and in addition serves as a secondary clarifier. The upstream application of chlorine provides good initial mixing. The chlorine is well distributed throughout the tank with free chlorine being absent in only one sample. The variation in free chlorine was from 0.0 to 0.12 mg/l with only a small variation in total chlorine of 1.29 to 1.68 mg/l. This small variation in total chlorine produced a standard deviation of 0.0868 mg/l from the average total residual of 1.415 mg/l. The average free residual was 0.0711 mg/l with a standard deviation of 0.0292 mg/l. The fact that this tank is also used as a clarifier appears to have a possible effect on the chlorine residual since the average total residual is much lower at this plant as compared to the Stroubles Creek plant. However, the lower levels could also be due to a lower dosing rate.

The chlorine residuals at the Narrows treatment plant are also effected by the characteristics of the flow. As can be seen in Figures 7 through 9, a majority of the free residuals

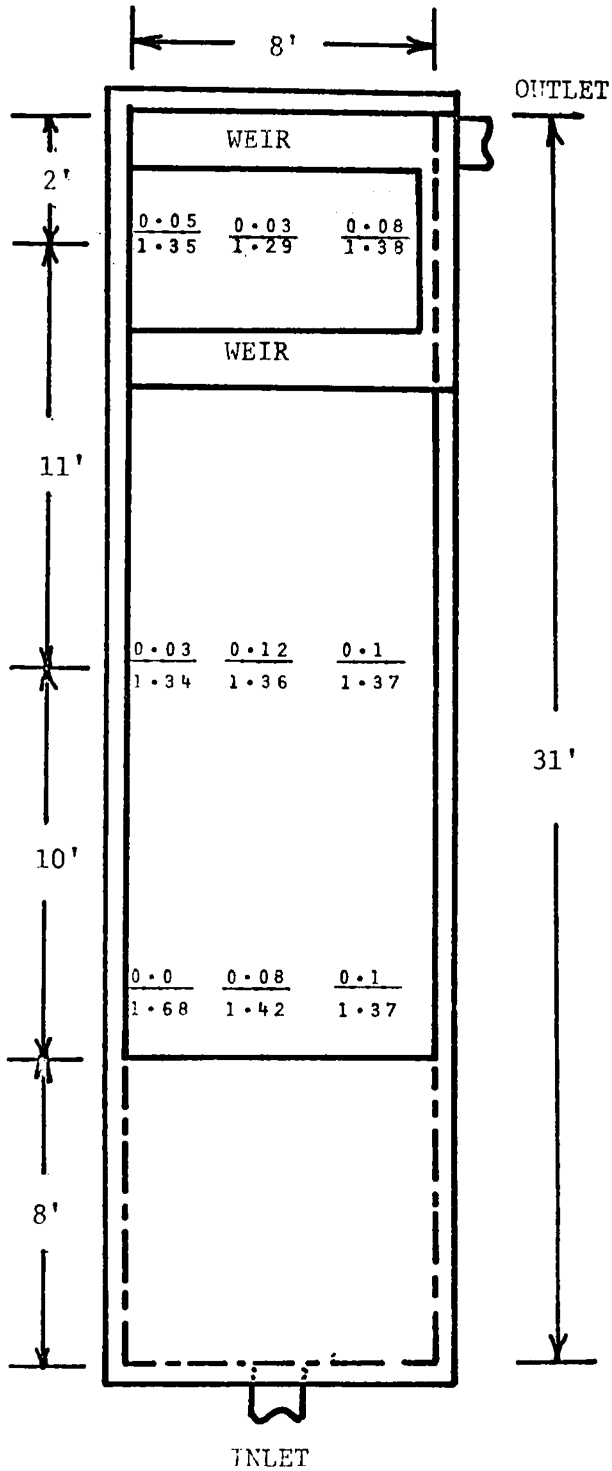

Figure 4 Distribution of Free and Total Chlorine at the Surface in the Old Christiansburg Secondary Clarifier-Chlorine Contact Tank

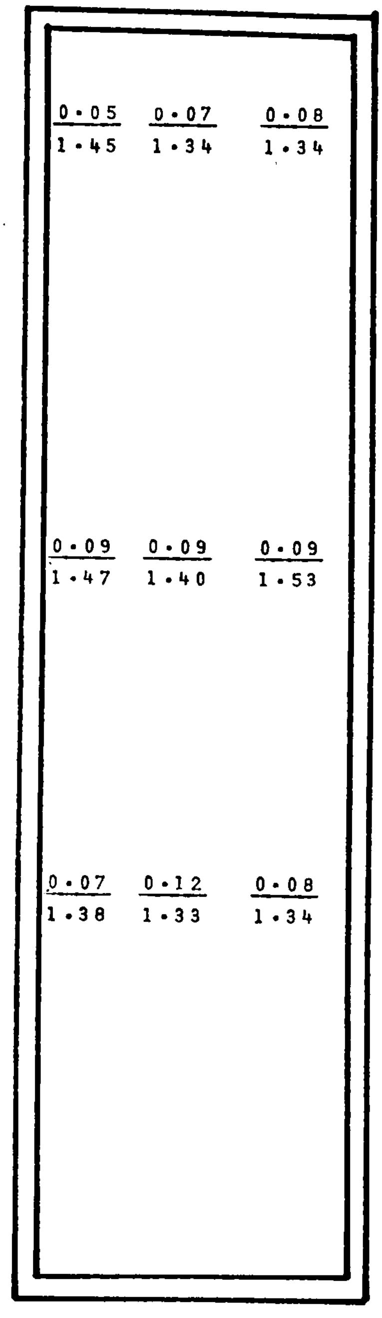

Figure 5 Distribution of Free and Total Chlorine at the Three Foot Depth in the Old Christiansburg Secondary Clarifier-Chlorine Contact Tank

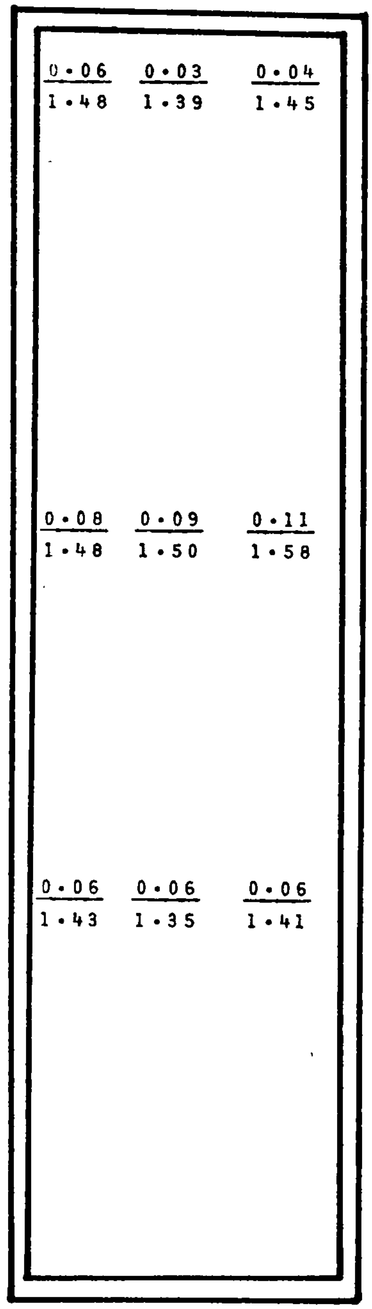

Figure 6 Distribution of Free and Total Chlorine at the 6.5 Foot Depth in the Old Christiansburg Secondary Clarifier-Chlorine Contact Tank

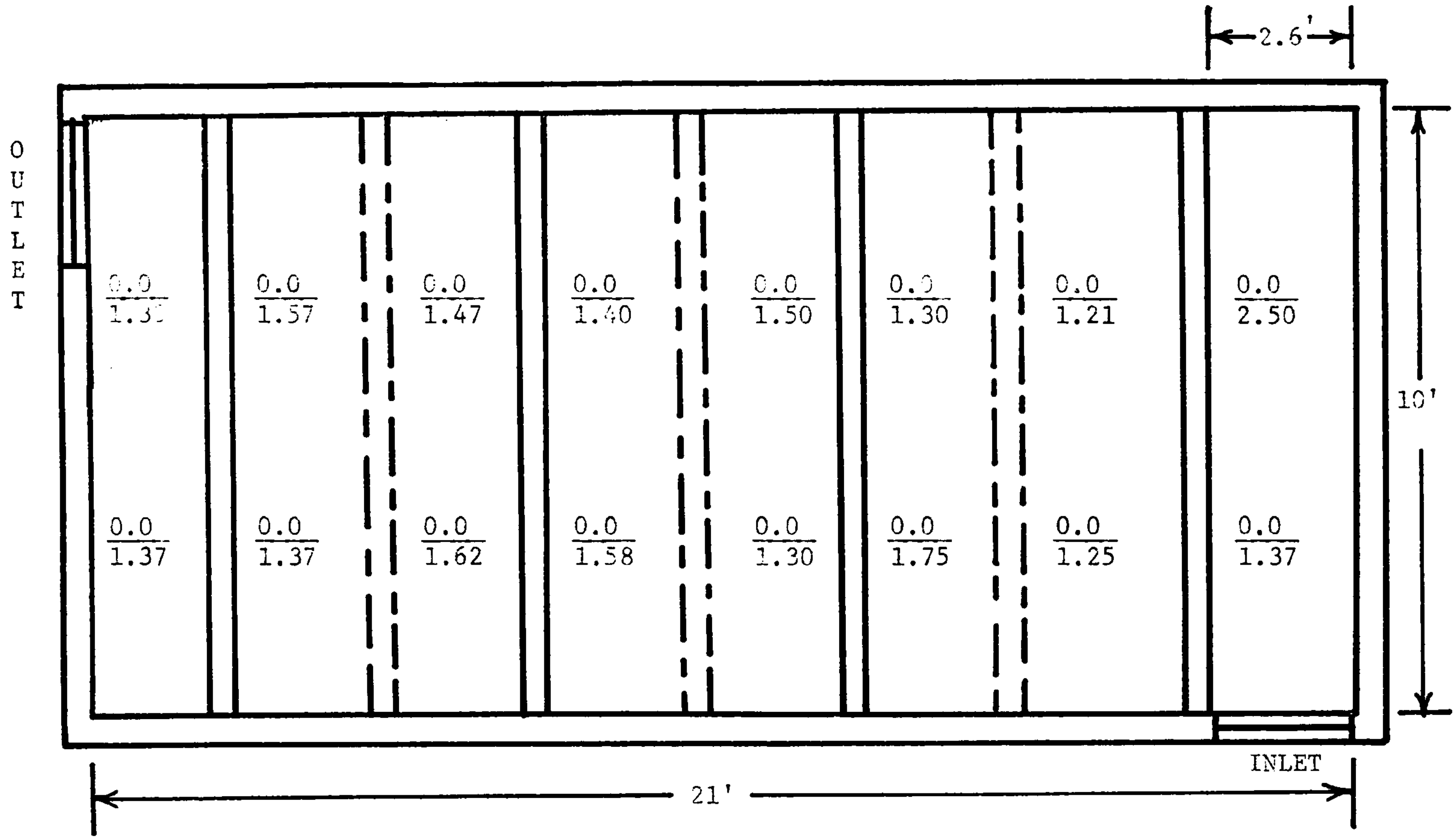

Figure 7 Distribution of Free and Total Chlorine at the Surface in the Over-and-Under Baffled Chlorine Contact Tank at the Narrows Treatment Plant

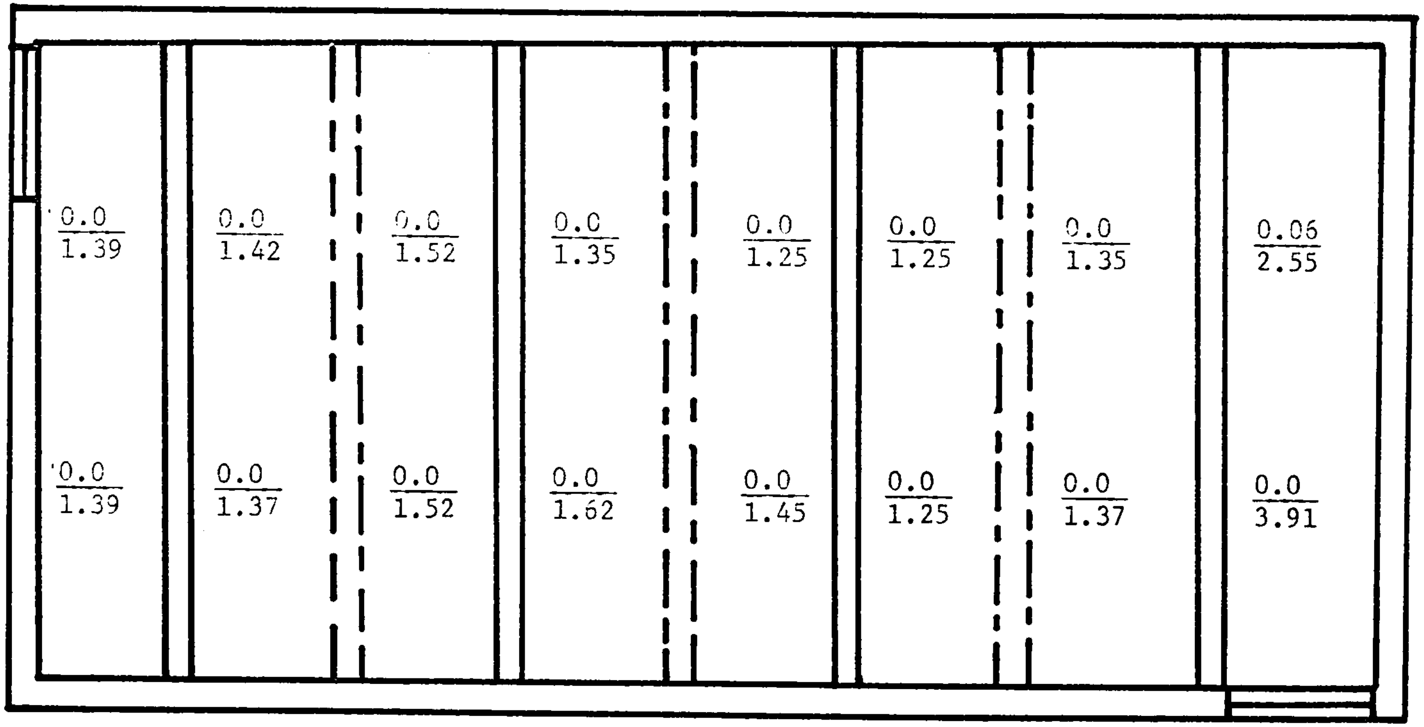

Figure 8 Distribution of Free and Total Chlorine at the 2.5 Foot Depth in the Over-and-Under Baffled Chlorine Contact Tank at the Narrows Treatment Plant

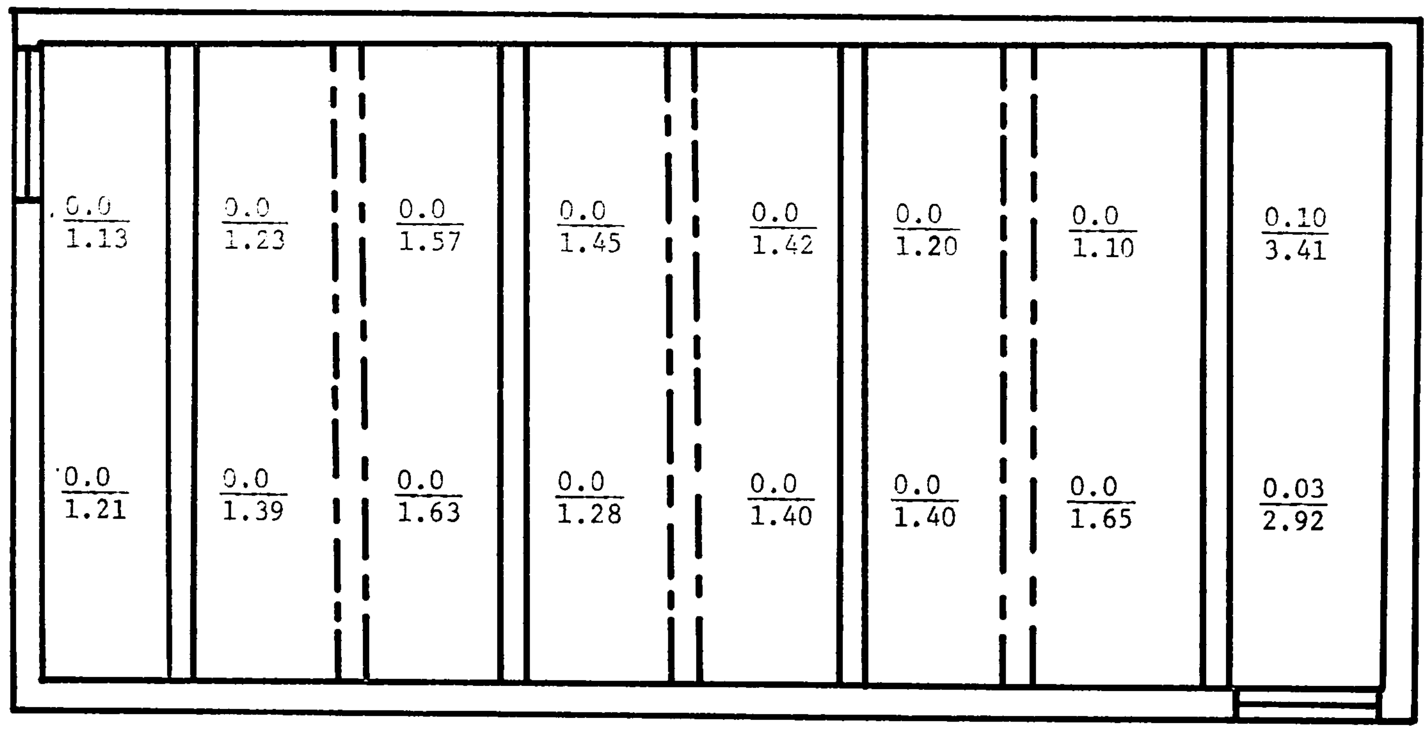

Figure 9 Distribution of Free and Total Chlorine at the Five Foot Depth in the Over-and-Under Baffled Chlorine Contact Tank at the Narrows Treatment Plant

were 0.0 mg/l at this plant where only primary treatment is provided, whereas all other plants investigated were secondary plants. Though free chlorine was not present, the total chlorine was well distributed throughout the over-and-under baffled tank. High total residuals are noted near the point of chlorine application, but drop off sharply as the flow moves through the tank. There also appears to be several low total residuals near the bottom of the tank. The average total residual was 1.555 mg/l with a standard deviation of 0.5402 mg/l. For the free chlorine the average residual was only 0.0037 mg/l with a standard deviation of 0.0165 mg/l.

A large portion of the samples at the Harrisonburg plant also had free residuals of 0.0 mg/l. The location of the end-around baffles and the distribution of chlorine is shown in Figures 10 through 12. Higher total residuals seem to indicate a definite channeling of the flow along the baffles with the formation of dead spots and areas of back flow in the corners of the baffles opposite the main flow stream. The chlorine appears to be well mixed in the main flow stream, but there are several low residuals in the back flow areas. With an average free chlorine residual of 0.0188 mg/l, the standard deviation was only 0.0345 mg/l. A large variation in total residuals produced a standard deviation of 0.5758 mg/l from the average of 1.117 mg/l.

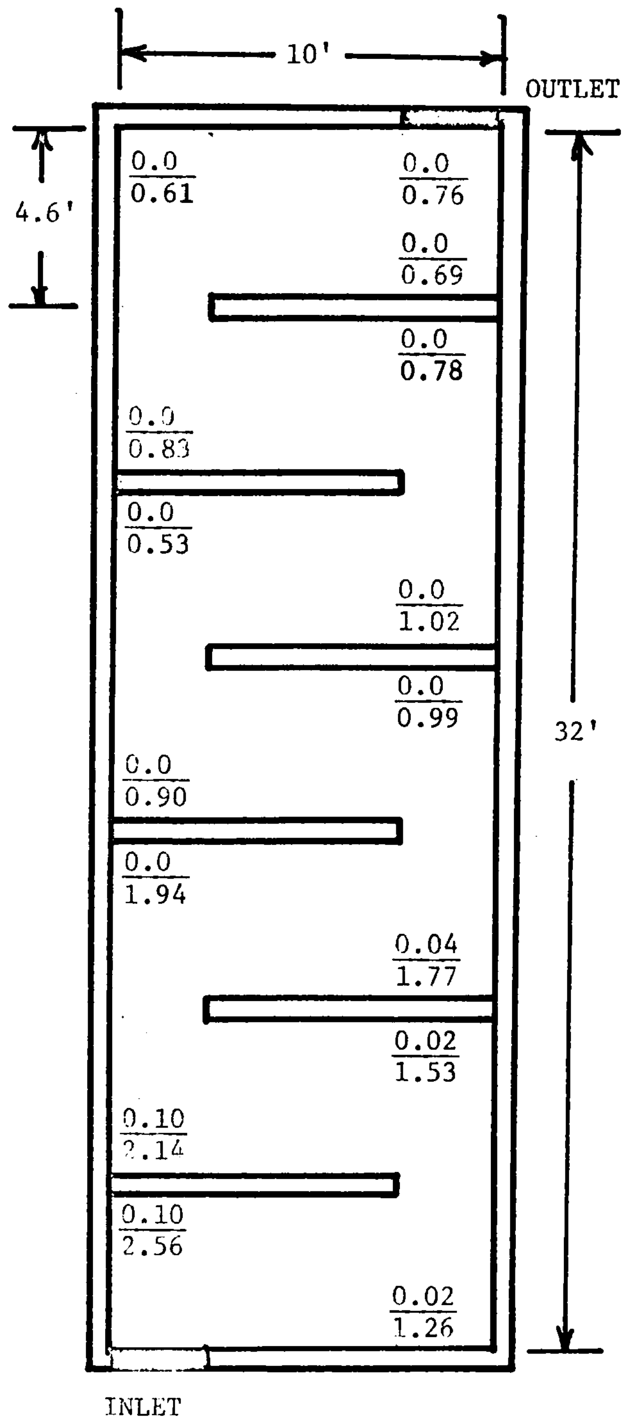

Figure 10 Distribution of Free and Total Chlorine at the Surface in the End-Around Baffled Chlorine Contact Tank at the Harrisonburg Treatment Plant

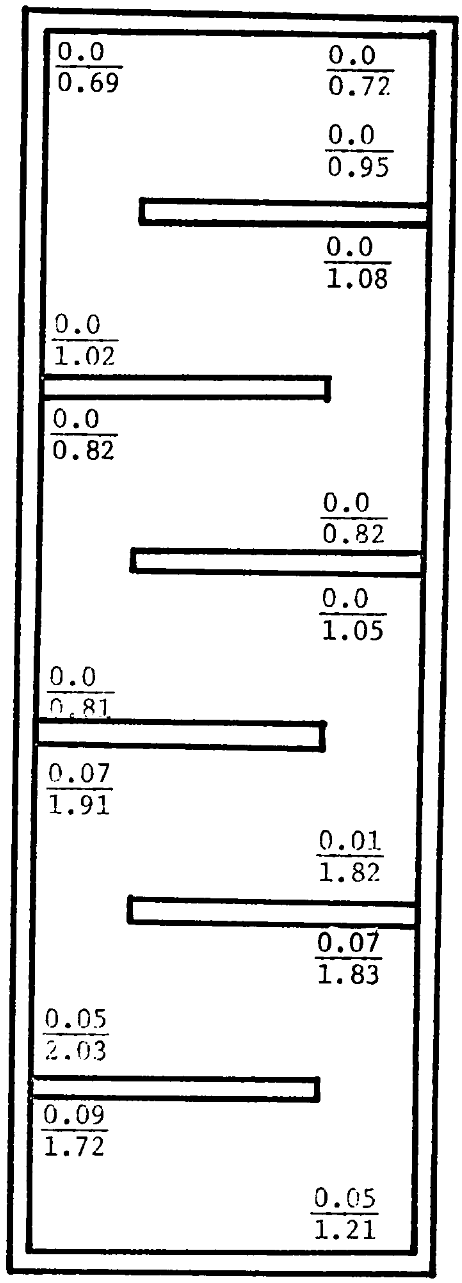

Figure 11 Distribution of Free and Total Chlorine at 4.5 Foot Depth in the End-Around Baffled Chlorine Contact Tank at the Harrisonburg Treatment Plant

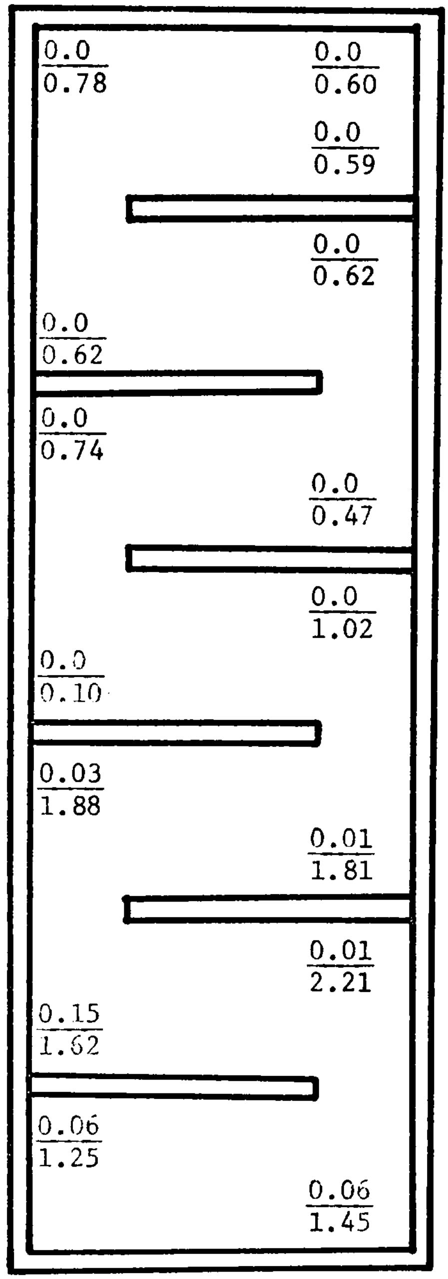

Figure 12 Distribution of Free and Total Chlorine at Nine Foot Depth in the End-Around Baffled Chlorine Contact Tank at the Harrisonburg Treatment Plant

The end-around baffled tank at the Lowmoor treatment plant showed results very similar to the tank at Harrisonburg. There appears to be good mixing in the main flow stream, but poor mixing in the back flow and dead spot areas. Only two samples had no free chlorine present, but several low total chlorine residuals were noted throughout the tank. Free chlorine was found in a majority of the samples and produced a standard deviation of 0.0315 mg/l from an average free residual of 0.0517 mg/l. The standard deviation for the total residuals was much higher at 0.6578 mg/l. The average total chlorine residual was 2.12 mg/l. The distribution of chlorine is shown in Figures 13 through 15.

At each of the treatment plants where the chlorine contact tanks were sampled, a comparison was made between the total chlorine residual recorded by the Orthotolidine method and by the Amperometric titration method. Figure No. 16 is a graph of the Orthotolidine versus the Amperometric titration residuals recorded at the five plants. The graph shows the distribution of the points with the average for each plant indicated by the line drawn through the points.

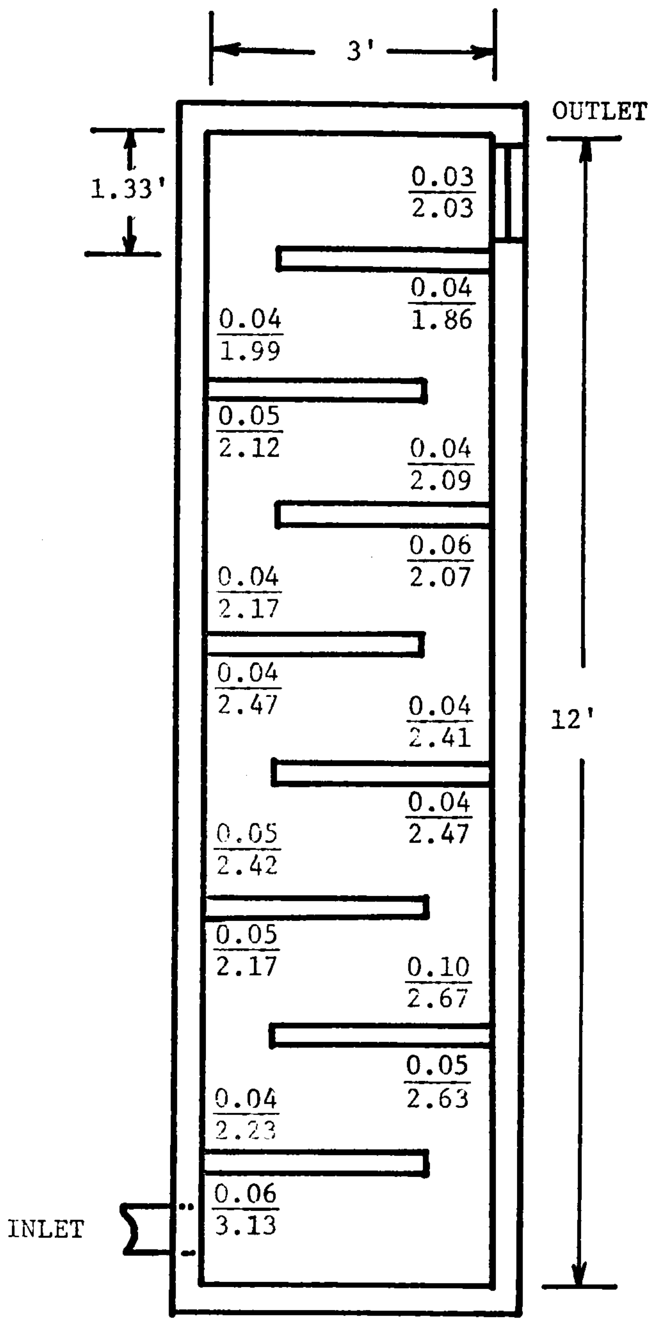

Figure 13 Distribution of Free and Total Chlorine at the Surface in the Lowmoor End-Around Baffled Chlorine Contact Tank

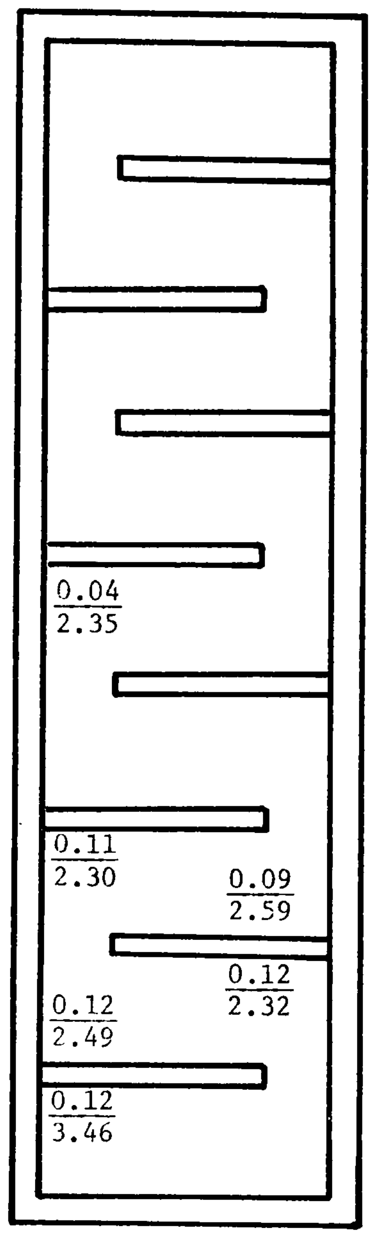

Figure 14 Distribution of Free and Total Chlorine at the Two Foot Depth in the Lowmoor End-Around Baffled Chlorine Contact Tank

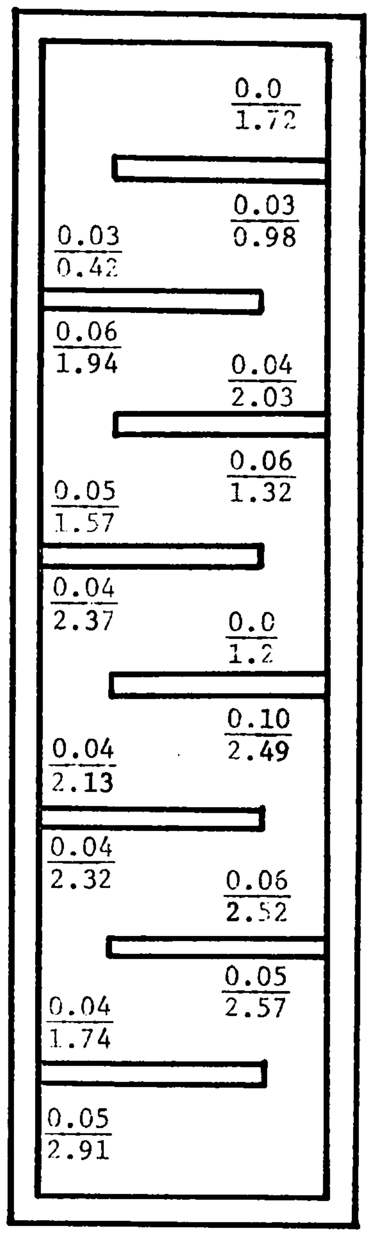

Figure 15 Distribution of Free and Total Chlorine at the Three Foot Depth in the Lowmoor End-Around Baffled Chlorine Contact Tank

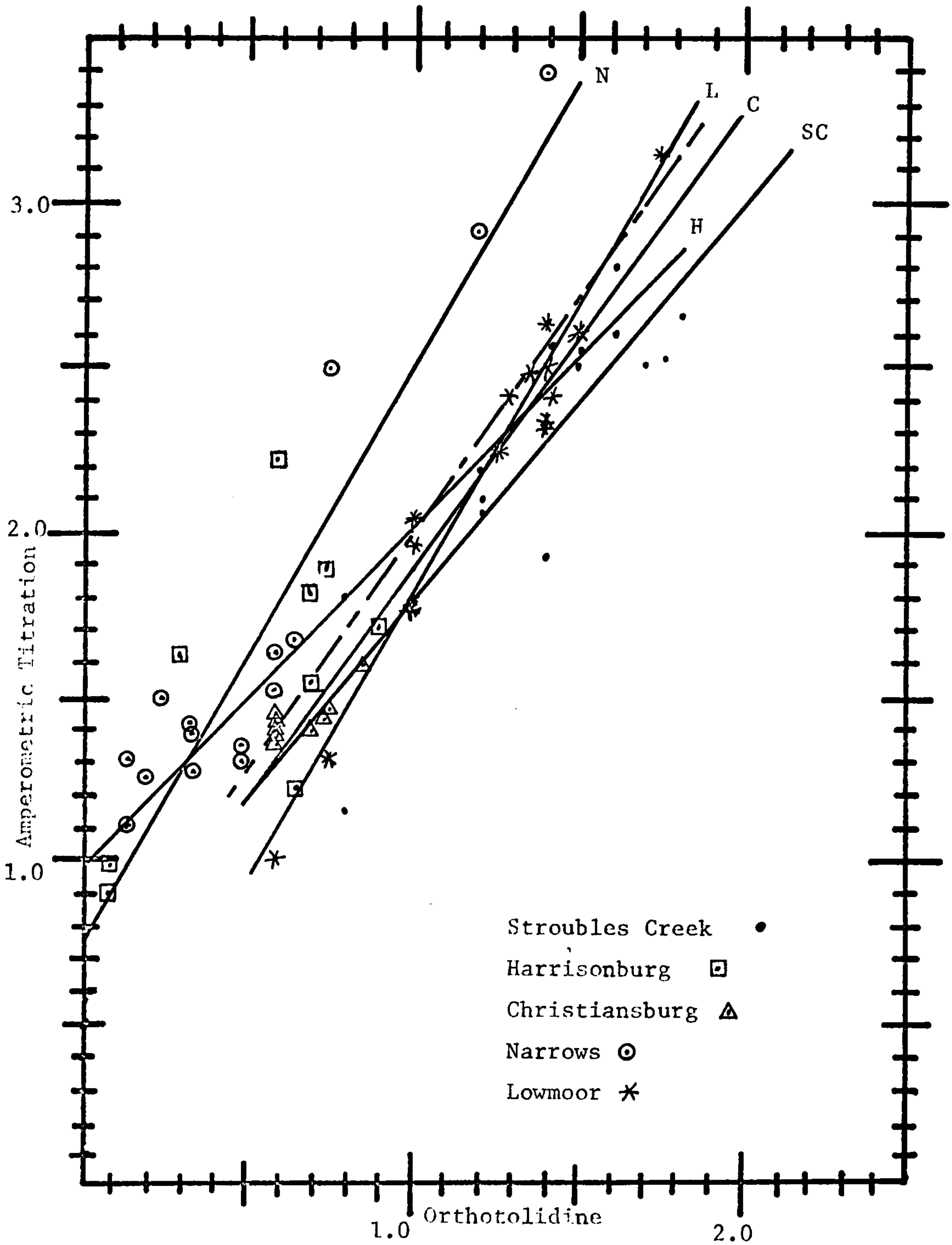

Figure 16 Amperometric Titration Versus Orthotolidine Methods for Total Chlorine Residuals

VI. DISCUSSION OF RESULTS

Analysis of Units Studied

At all of the treatment plants in which sampling was conducted, the chlorine feed rate was preset and thus was not modified in proportion to the flow. Generally, four hours were required to complete the sampling of each contact tank. Due to fluctuations in flow, the chlorine dosage varied during the sampling period. At the Stroubles Creek treatment plant, a reduction in flow increased the chlorine dosage after testing began. This reduction caused higher chlorine residuals at the effluent end of the tank. At the Harrisonburg treatment plant an increase in flow reduced the chlorine dosage, causing a large drop in chlorine residual downstream from the third baffle from the inlet. The Narrows treatment plant receives its influent flow from a pump station which caused a constant fluctuation in flow during sampling. Only minor flow fluctuations at the other two plants caused occasional erratic residuals.

As mentioned previously, chlorine at the Stroubles Creek plant is applied in a manhole preceding the chlorine contact tanks in order to improve mixing. A distribution wall is also provided to improve the distribution of the flow across the width of the tank. According to the chlorine residuals found at the influent end of the tank, the chlorine and fluid appear to be well mixed before entering the tank. Upon reaching the distribution wall there is some solids separation near the corners of the tank indicated by the lower total chlorine residuals and depletion of free chlorine. The residuals in Figure No. 1 indicate that the chlorine is well distributed throughout the tank though it appears that a portion of the flow may be skimming the bottom of the tank and rising to go over the effluent weir. This effect is possibly brought about by density currents common in tanks of this type.

The Christiansburg contact tank is also an unbaffled tank and produced results similar to the Stroubles Creek tank. At this plant, the chlorine is also applied upstream from the tank, and good mixing at the influent end of the tank was found. The low standard deviations for both free and total chlorine indicate that the tank is well mixed, but the total residuals increase with depth. This would again seem to indicate that the flow was skimming the bottom of the tank and rising to go over the effluent weirs. Sludge removal equipment is also provided in this tank since it serves as both a secondary clarifier and a chlorine contact tank.

The Narrows contact tank was the only over-and-under baffled tank sampled. This plant provides only primary treatment which would account for the depletion of free chlorine in a majority of the samples. Free chlorine was found only in the samples near the point of chlorine application.

As mentioned previously, the influent is pumped to the plant which causes continuous flow fluctuations. During sampling the flow fluctuated as much as 1.8 mgd in a few minutes. This caused a surging and a rise and fall of fluid depth in the tank which appeared to aid in the mixing of the chlorine and fluid. The chlorine is applied by a plastic hose placed below the water surface in the middle of the first bay. The point of application can be seen from the high residuals in the first bay in Figures 7 through 9. The wastewater appears to be well distributed throughout the tank except near the effluent weir. As the flow approaches the weir, which is located at one corner of the tank, the higher residuals indicate the flow is channeling towards the weir.

Since there is no sludge removal equipment provided at Narrows, there is a slight sludge build up in several places. This effect is especially evident where the flow is forced to the bottom of the tank by the under flow baffle. The flow appears to scour the solids from beneath the under flow baffle and to cause a build up of the solids along the bottom of the over flow baffles. Low residuals near the bottom of the tank indicate the areas of solids build up.

The Harrisonburg contact tank was the first of the end-around baffled units investigated. Chlorine is applied in a splitter box used to divide the flow between the dual contact tanks.

This system appears to mix the chlorine and water fairly effectively. Additional mixing is evident from the high turbulence at the inlet of the tank.

As indicated by the higher residuals in Figures 10 through 12, the flow appears to be entering at the surface of the first bay and channeling towards the bottom of the tank at the outlet of the first bay. Upon entering the second bay, the main stream of the flow still appears to be near the bottom, but the distribution of chlorine shows the flow becomes well mixed before leaving the second bay. As mentioned previously, there was a large drop in residual following the third baffle due to an increase in flow.

The flow begins to smooth out upon leaving the second bay and a channeling of the flow begins forming along the side of the baffles. The channeling of the flow along one side of the bay forms dead spaces in the corners on the opposite side of the bay. Low residuals near the bottom of the tank indicate that there is a solids build up in the dead spaces. This pattern, with the channeling of the flow and formation of dead spaces, is followed until the fluid enters the next to last bay. Upon entering the next to last bay, the flow again begins channeling towards the bottom of the tank. In the last bay the higher residuals indicate the main stream of the flow is skimming the bottom and rising at the end of the bay to go over the effluent weir.

The contact tank at the Lowmoor treatment plant is also an end-around baffled tank. Since this is a small plant designed for 0.4 mgd, the contact tank is very small and only has a depth of 4.25 feet. At first, three sampling depths were chosen, surface, 2 feet and 4 feet, but because of solids build up on the bottom, the lowest depth was changed to 3 feet. Any samples taken below 3 feet showed a depletion of almost all chlorine due to the sludge. After sampling three bays, it was found that the 2 foot and 3 foot samples were producing the same results, therefore the 2 foot samples were eliminated.

Chlorine is applied in the influent line to the tank which along with the turbulence in the first bay, produce good mixing. As the flow proceeds through the second bay, the main stream seems to be at the 2 to 3 foot depth. Channeling of the flow along one side of the bays is also evident in this tank. It is not as pronounced as at the Harrisonburg plant because of the size of the tank. In the first three bays, the flow appears to follow one side of the bay at about the 2 or 3 foot depth, but after the fourth bay it appears to rise back to the surface though still channeling and forming dead spots through the rest of the tank.

In comparing the five tanks investigated, the over-and-under baffled tank provides the least amount of short circuiting. Though some short circuiting is evident in all of the tanks, the baffled tank appears to provide a more efficient use of the entire volume of the tank as compared to the unbaffled tanks. The end-around baffled tanks have the greatest degree of channeling which produces a substantial volume of dead spaces.

In dye and tracer studies conducted by Sawyer[5], similar results were found for the end-around baffled tanks. As a result of testing at the Lowmoor plant, Sawyer found plug flow conditions were approached with a minimum short circuiting, but the formation of dead spaces was evident.

The unbaffled tanks have an advantage over the baffled tanks in that they are able to be equipped with sludge removal facilities. To remove the sludge from the baffled units, the tank must be drained and the sludge removed by hand or a pump may be used to draw the sludge off the bottom with the tank still in operation.

With the build up of sludge and the resulting short circuiting, the disinfection efficiency of the chlorine contact tank is lowered considerably. The sludge places an extremely high demand on the chlorine while short circuiting due to lower effective volumes reduces the contact time, a very important parameter in determining disinfection efficiency. To provide adequate inactivation of pathogens, the chlorine must be applied at a sufficient concentration and be in contact with the pathogen long enough to inactivate its life systems.

The application of chlorine to the fluid ahead the contact tank appears to provide good mixing. The turbulence provided by the fluid motion in a pipe, splitter box or manhole seem to allow adequate mixing ahead of the contact tank without the expense of mechanical mixing equipment.

The location and configuration of the inlet and outlet of the tank is important in providing good distribution of the fluid. This is much more critical in the unbaffled and over-and-under baffled tanks than with the end-around baffled units. The inlet should be located at the center of the influent wall and the effluent weir should extend across the entire width of the effluent wall. It is also very important for this type of weir to be level to allow the effluent to overflow evenly across the width of the tank. The use of a distribution wall at the influent of the tank aids in distributing the fluid, but the tendency toward sedimentation of solids at the wall provides a possible disadvantage.

The degree of treatment preceding chlorination appears to have an effect on the forms of chlorine present and on the total amount of chlorine in the chlorine contact tank. In the case of Narrows, where only primary treatment is provided, the free residual is depleted almost immediately upon contact. Where secondary treatment is provided, a majority of the samples contained free chlorine residuals, except at Harrisonburg where a sharp increase in flow reduced the chlorine dosage severely. The total chlorine residual is also affected by the sewage characteristic as can be seen by the low average total residuals at both the Old Christiansburg and Narrows plants. Since the secondary clarifier at the Old Christiansburg plant doubles as a chlorine contact tank, the high solids concentration presents a high chlorine demand. The large organic solids concentration resulting from only primary treatment at Narrows places a high demand on the chlorine. As mentioned previously, fluctuations in flow can also affect chlorine residuals where the chlorine feed rate is not proportioned to flow.

Comparison of Amperometric Titration and Orthotolidine Methods of Analysis

In comparing these two methods of measuring total chlorine residual, it was found that there was a considerable difference in results. Approximately 10 to 15 samples were taken at each plant to make the comparison.

Figure No. 16 shows the distribution of data points resulting from the comparison of the Amperometric titration and Orthotolidine methods of measuring total chlorine residuals. A simple linear regression by the Least Squares Method was used to fit a line to the data points from each treatment plant. The dotted line was fitted to all of the data points to show the relationship of the two methods based on data collected from all five plants. From this line, the following equation was developed:

$$AT = 0.55 + 1.47(OT)$$

where AT = Amperometric titration residual, and

OT = Orthotolidine residual.

Based on this equation, a residual of .55 mg/l by the Amperometric titration method is required to produce a residual by the Orthotolidine method.

Most treatment plants in this area use the Orthotolidine method for measuring chlorine residuals. If the Amperometric titration method were used, the apparent chlorine dosage could be lowered considerably and the 2.0 mg/l effluent residual requirements of the Virginia Department of Health could be met. Even though the effluent requirement is being met, there is strong opposition to whether a 2.0 mg/l residual by the Amperometric titration method will provide adequate inactivation of virus.

VII. CONCLUSIONS

1. The initial mixing of the chlorine and fluid prior to the fluid entering the contact tank is essential to good operation.

2. The location and configuration of the inlet and outlet structures of the chlorine contact tank influence the degree of short circuiting and distribution of flow in the tank.

3. Baffled rectangular tanks appear to have less short circuiting than unbaffled rectangular units due to the baffles directing the fluid through the tank.

4. Sludge build up is a problem with baffled tanks since they are not suited to continuous sludge removal equipment.

5. Sludge build up produces a high demand on the chlorine.

6. The Amperometric titration method yields chlorine residuals substantially higher than the Orthotolidine method and 0.1 to 0.98 mg/l residuals were measured by the Amperometric titration method before any residuals were shown by the Orthotolidine method.

7. The type of treatment provided affects the free to total chlorine ratio. The higher the degree of treatment the more free chlorine present and the higher the ratio evident.

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FIELD STUDY OF PERFORMANCE OF RECTANGULAR

CHLORINE CONTACT TANKS

by

GLENN WADE CUSTIS

(ABSTRACT)

The purpose of this study was to make a field evaluation of the efficiency of three types of rectangular chlorine contact tanks. Efficiency of the tanks was evaluated in terms of the type of initial mixing of the chlorine and fluid, location and configuration of the inlet and outlet structures, the influence of baffles and the effect of sludge deposits. Two unbaffled tanks, two end-around baffled tanks and one over-and-under baffled tank were sampled.

The sampling procedure consisted of measuring free and total chlorine residuals throughout each tank by the Amperometric Titration method. Each tank was divided into a grid system, and samples were taken at various depths across the width and length of the tank.

List results showed that the baffled tanks provided less short circuiting, especially the over-and-under baffled units, as compared to the unbaffled tank. Premixing of the chlorine and fluid upstream from the tank produced a good distribution of

chlorine at the influent of the tank. The need for sludge removal was shown by the high chlorine demand of sludge deposits in the baffled tanks.

A second investigation was conducted to make a comparison between the Amperometric titration and Orthotolidine methods of measuring total chlorine residuals. Results showed that the Amperometric titration method measured much higher residuals. An Amperometric titration residual of 0.1 to 0.98 mg/1 was required to produce any Orthotolidine reading.