Decay of Total Residual Chlorine and Reduction of Fecal Coliform in the Plum Island Outfall Pipe Prior to Discharge to the Charleston Harbor

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Prepared By:

John B. Cook, PE
Director of Engineering
Charleston CPW

Andrew W. Fairey
Director of Water Resources
Charleston CPW

INTRODUCTION

The Charleston Commissioners of Public Works has successfully used chlorine as a disinfectant during the history of the operations of its Plum Island Wastewater Treatment Plant (WWTP). However, because of chlorine's potential toxicity, there is currently an NPDES permit limit for total residual chlorine (TRC) of 0.5 mg/L and 1.0 mg/L for a monthly average and daily maximum, respectively. The Plum Island WWTP is currently designed to achieve secondary treatment levels at a design average daily flow of 36 MGD. A review of historical data from the period of 1991 through 1995 reveals an average BOD_5 value of 4.0 mg/L and an average suspended solids value of 6.0 mg/L for the period. Disinfection limits required by the NPDES permit (No. SC0021229) are equivalent to the Charleston Harbor stream classification of SB, namely, 200 counts fecal coliform per 100 mL as a monthly geometric mean with no more than ten percent of the total samples during any thirty-day period exceeding 400 counts. As the rate of flow increases, the Plum Island WWTP will have less detention time available in its chlorine contact chamber to inactivate pathogens while continuing to meet the total residual chlorine limit, without employing either dechlorination or changing the disinfection process.

Even though an alternate disinfectant such as ultraviolet radiation or ozone could be used, or dechlorination of the existing chlorine feed could be employed, each option has major drawbacks. Ultraviolet radiation (UV) has been shown to be an effective disinfectant of wastewater effluents but an oxidant would have to be provided for secondary uses, namely, for the control of filamentous organisms and for the control of odors using air scrubbers. Ozone's drawback is that it is both energy and capital intensive although it could be effective as a control agent for filamentous growth. However, an additional oxidant or other appropriate chemical would be needed for the air scrubbers. Finally, dechlorination using SO_2 would allow higher chlorine concentrations but would also introduce another chemical subject to stringent controls under the Process Safety Management (PSM) regulations (Occupational, Safety, and Health Act) and the Risk Management Plan (RMP) regulations (Clean Air Act). Hence, the ideal solution would be to continue to chlorinate with an optimum quantity of chlorine necessary to achieve both fecal coliform inactivation and to meet the total residual chlorine limits. The purpose of this report is to present the results of field and bench-scale testing which demonstrates significant TRC and fecal coliform decay in the Plum Island outfall pipe. This report further recommends that the outfall pipe be taken into consideration when determining actual TRC and fecal coliform effluent concentrations. In addition, this report recommends a procedure for accomplishing this task.

The current sampling point to determine NPDES permit compliance for TRC and coliform is the effluent flume which is the last accessible sampling point after the chlorine contact chamber but before the outfall pipe. However, this sampling point doesn't take into consideration the detention time in the outfall pipe which consists of 1219 meters (approximately 4000') of 1.52 meter (60 inch) diameter concrete pipe on pile supports (Figure 1). The effluent from the outfall pipe is discharged from seven (7) 51 cm (20-inch) diffuser risers into a mean water depth of over six (6) meters (20 feet). The diffusers yield an effective far-field dilution ratio in the Charleston Harbor of approximately 50:1 (Hazen & Sawyer, 1995). The outfall pipe was constructed in 1969 and placed into operation in 1970.

At the design flow of 36 MGD, the contact time in the outfall is 23.5 minutes (Table 1). This is in addition to the contact time in the effluent flume and chlorine contact chamber. The reduction of fecal coliform counts and the decay of chlorine over time are well documented in the literature and have been further documented in the field and bench-scale testing conducted for this study. A brief review of current literature is in order.

THEORETICAL FRAMEWORK

Decay of TRC

The decay of TRC in drinking water distribution systems has been well documented (Biswas et al. 1993; Rossman et al. 1994; Sharp et al. 1991; Wable et al. 1991). For example, Sharp et al. (1991) showed how chlorine residuals can vary throughout the day at different locations in a water distribution system depending on the flow path and residence time of the water. Studies on chlorine decay rates in single lengths of pipe reveal that the decay rate in the pipe is several times greater than the decay rate of the same water in a flask (Wable et al. 1991), suggesting the pipe wall can contribute to overall chlorine demand.

However, due to relatively low concentrations of organics in drinking water, results from these studies cannot be compared directly with wastewater effluent, though the total organic carbon (TOC) levels in the Plum Island plant's effluent are comparable to some potable waters. Nonetheless, field and bench tests are necessary to calibrate and confirm model applications. In addition, chlorine decay is a function of a number of site-specific properties such as pipe diameter, pipe material and flow rate (Sharp et al. 1991).

Disinfection Theory for Fecal Coliform Reduction

In addition to TRC decay, there is a significant reduction in fecal coliform as a result of additional disinfectant contact time in the outfall pipe. In fact, Metcalf and Eddy, Inc. (1991, page 504) suggests, subject to regulatory limits, the use of the outfall pipe for disinfection in lieu of using a contact chamber. The idea of achieving disinfection in the outfall pipe can be understood in light of disinfection theory and the result of case studies, and is further confirmed by the bench-scale testing associated with this study. The principal disinfection theory currently used by the scientific community to explain the inactivation of bacteria and viruses is the Chick-Watson model which forms the basis for the disinfection requirements of the Surface Water Treatment Rule (U.S. EPA, 1989). Chick's law expresses the rate of destruction of microorganisms by the relationship of a first-order chemical reaction as follows:

$$\ln \frac{N}{N_o} = -kt \qquad \text{(Equation 1)}$$

in which N is the number of organisms present at time t; N_o is the number of initial organisms present; k is a rate constant which is characteristic of the type of disinfectant, microorganism, and water quality. Watson refined Chick's equation to produce an empirical relationship that includes changes in the disinfectant concentration:

$$\ln\left(\frac{N}{N_o}\right) = -AC^{n}t \qquad \text{(Equation 2)}$$

in which C is the concentration of disinfectant; A is the coefficient of lethality; n is the coefficient of dilution; and the other parameters are as defined above. The value of n in the Chick-Watson equation is a function of the disinfectant type and pH value.

From Metcalf and Eddy, Inc. (1991), it is seen that when chlorine (in the form of gaseous chlorine for Plum Island) is added to wastewater, two reactions occur, namely, hydrolysis and ionization. The hydrolysis reaction may be defined as:

$$Cl_2 + H_2O \langle=\rangle HOCl + H^+ + Cl^- \qquad \text{(Equation 3)}$$

and the ionization reaction is defined as:

$$HOCl \langle=\rangle H^+ + OCl^- \qquad \text{(Equation 4)}$$

in which the stability and ionization constants and variation of the species is subject to pH and temperature.

In addition to these products, a portion of the free available chlorine reacts with nitrogen in the form of ammonia and combined organic forms to produce combined available chlorine. Therefore, TRC consists of both free and combined chlorine species and each species acts as a disinfectant, though at differing rates.

EVALUATION OF MODELING APPROACHES AND SELECTION OF MODEL

A number of approaches to modeling chlorine decay have been employed and a brief review of key ideas is appropriate. Haas and Karra (1984) described the decay of chlorine below the break point in wastewater. This rate expression was applied to water distribution systems by Vasconcelos et al. (1995) as:

$$\frac{dC}{dt} = -kC \qquad \text{(Equation 5)}$$

which is the first-order kinetic expression for the decay of residual chlorine due to reactions with materials in the aqueous phase at different residence times. In this equation, C is the chlorine concentration (mg/L), k is the first-order decay constant (min^{-1}), and t is residence time (min). Integration of this equation gives:

$$C(t) = C_o e^{-kt} \qquad \text{(Equation 6)}$$

in which C(t) is the chlorine concentration (mg/L) at time t, C_o is the initial chlorine concentration (mg/L), and t is the pipe residence time (min.).

From the work of Haas and Kara (1984), other researchers noted that multiple factors appeared to affect the chlorine decay reaction. Kroon and Hunt (1988) observed that apparent chlorine decay rates were significantly higher in small diameter pipes than in large diameter pipes. Sharp et al. (1991) described k values as a function of pipe diameter, pipe material, and flow rate. Kiene et al. (1993) found that the decay rate can be correlated directly with the surface to volume ratio (S/V), initial chlorine concentration, and flow regime. The Kiene et al. model was expressed as:

$$\frac{dC}{dt} = -k_b C - k_w\left(\frac{S}{V}\right)C \qquad \text{(Equation 7)}$$

in which k_b is a first-order decay coefficient of chlorine in bulk liquid, k_w is a wall decay coefficient which is a function of pipe material, pipe surface condition and liquid-pipe interaction, to be determined empirically.

Researchers then began proposing models which considered mass transfer effects. Biswas et al. (1993) developed a two-dimensional model that considered first-order decay in the bulk flow, chlorine transport in the axial direction by convection, radial transport of chlorine to the pipe wall by diffusion, and a first-order reaction of chlorine at the pipe wall. Rossman et al. (1994) simplified this approach by replacing the radial diffusion term with a mass transfer coefficient. The resulting equation was expressed as:

$$\frac{dc}{dt} = -k_b C - \left[\frac{k_w k_f}{R_h(k_w + k_f)}\right] C \qquad \text{(Equation 8)}$$

in which k_b and k_w are bulk and wall coefficients, respectively, as previously defined, R_h is the pipe hydraulic radius (the inverse of surface to volume ratio), and k_f is a mass transfer coefficient which is a function of the Reynold's number, the molecular diffusivity of chlorine, and for laminar flow, the pipe length.

The Water Treatment Plant model (U.S. EPA, 1992) describes chlorine demand by dividing the decay curve into three discrete components. These include an initial "instantaneous" reaction ($t< 5$ minutes), a second-order reaction (5 minutes $< t <5$ hours) and a first-order reaction ($t>5$ hours). These three phases are described by the following equations:

For $t <5$ minutes ("instantaneous"):

$$\ln (C_0 - C_1 - 7.6\, NH_3) = -0.62 + 0.522 \ln (C_0/TOC) + 0.302 \ln (UV254) + 0.842 \ln (TOC)$$

(Equation 9)

For 5 minutes < t<5 hours:

$$1/C_t = 1/C_1 + k_1 t \qquad \text{(Equation 10)}$$

For t > 5 hours:

$$C_t = Ae^{-k2t} \qquad \text{(Equation 11)}$$

in which C_1 is the chlorine residual at 5 minutes, k_1, and k_2 are second and first order rate constants, respectively, and A is a constant obtained by setting the second and third equations equal to each other at t = 5 hours.

The prediction of chlorine decay with respect to time is complicated by the change in reaction rates that occur immediately after chlorination. For example, the Vasconcelos et al. model implicitly ignores this difference by being used to model water in the distribution system following the chlorination period at the water treatment plant. Likewise, the Vasconcelos et al. approach would model chlorination at Plum Island only after the initial "instantaneous" period which would occur completely in the chlorine contact chamber.

Rossman et al. (1994) developed a one-dimensional conservation of mass equation for a dilute concentration of total free chlorine in water flowing through a section of pipe as:

$$\frac{\partial C}{\partial t} = -U\frac{\partial C}{\partial x} - k_b C - \frac{k_f}{R_h}(C - C_w) \qquad \text{(Equation 12)}$$

in which C is chlorine concentration in the bulk flow; t is residence time; U is flow velocity in the pipe; x is distance along the pipe; k_b is the bulk decay coefficient; k_f is the mass transfer coefficient; R_h is the hydraulic radius of the pipe; and C_w is the chlorine concentration at the pipe wall. Under steady state conditions (i.e., $\partial C/\partial t=0$), this equation was reduced by Huang et al. (1997) to:

$$U\frac{\partial C}{\partial x} = -KC \qquad \text{(Equation 13)}$$

in which U, C, and x are defined as above and K is expressed by Rossman et al. (1994) and defined previously as:

$$K = k_b + \frac{(k_w k_f)}{R_h(k_w + k_f)} \qquad \text{(Equation 14)}$$

The solution to the Huang/Rossman equation with the boundary condition $C=C_0$ at $x = 0$ is:

$$C=C_0e^{-Kt} \qquad \text{(Equation 15)}$$

in which $t = x/U$, the detention time. The assumptions behind the development of this model are as follows from Huang et al. (1997): (1) the pipe flow is steady state and plug flow; (2) chlorine decay is governed by first order kinetics following the “instantaneous” demand; (3) the decay is due to reactions both within the bulk flow and within the pipe wall biofilm; and (4) the rate of wall reaction is affected by the rate at which chlorine can be transported from the bulk flow to the pipe wall and is represented by a film-resistance model of mass transfer.

Based upon the success of applying the Rossman et al. (1994) model by Huang et al. (1997) to the Broward County ocean outfall pipe in their 1995 study, the Huang/Rossman model was used to model the Plum Island outfall pipe.

FIELD AND BENCH SCALE TESTING

A total of twelve (12) field tests were conducted in a three week period during August and September 1997 to determine Plum Island effluent TRC decay in the outfall pipe. Two sampling stations were used, namely, the effluent flume at which current NPDES permit data is collected; and, a second (temporary) sample station within the first riser of the outfall diffuser, approximately 1219 meters between sample stations. The temporary sampling station was installed by using underwater divers to attach a 3/16" steel cable and 1/2" polybutalene sample tube inside the first riser pipe. Pairs of samples were collected from a boat once or twice daily depending upon weather and flow conditions. Each pair of samples were taken under the following procedure. The first sample was taken at the effluent flume to include the following parameters:

-TRC

-Temperature

-pH

-Flow

-BOD_5

-Suspended Solids

With the collecting of the first sample at the effluent flume, the flow was noted. The second sample of the sample pair was collected at the temporary sampling site at a time equal to the theoretical detention time in the outfall for the flow observed for the first sample. This assumes that the time for chlorine decay is equal to the theoretical pipe detention time which is reasonable given laminar, plug-flow conditions in the outfall pipe, and the constant flow rate over relatively short durations of time. In other words, steady-state and plug-flow conditions prevailed. The samples collected at the temporary station (the diffuser) included the following parameters:

-TRC

-Temperature

-pH

Samples at the temporary station were collected from the 1/2" sampling tube using a diaphragm pump powered by battery. Each field test involved purging the sampling tube, collecting the sample, and immediately measuring TRC concentrations in the boat. Samples collected at the effluent flume were likewise measured for TRC concentrations. All analyses were conducted in accordance with EPA approved methods for NPDES permit sampling. Flow readings were obtained from the effluent Parshall flume. During the study, flows ranged from 16.5 MGD to 22.5 MGD during the sampling events and temperatures of the effluent ranged from 27.3°C to 29.3°C. TRC levels for the receiving water were also taken at a distance outside of the diffuser plume and were always <0.05 mg/L. A summary of the field data is shown in Table 2.

Bench-scale testing was conducted on November 3 and 10, 1997, to determine chlorine decay in bulk samples as a function of time. This data was subsequently used to determine the rate coefficient, k_b. In addition, fecal coliform testing was conducted concurrently to determine the reduction in fecal coliform as a function of time. The results of the bench-scale testing for TRC decay is shown in Table 3, and the results of the concurrent bench-scale testing for fecal coliform is shown in Table 4.

ANALYSIS OF DATA

With the selection of the approach to modeling as developed by Rossman et al. (1994), Huang et al. (1997) and Vasconcelos et al. (1995), there are three (3) decay coefficients to be input into the equation to predict TRC as a function of C_o (the initial chlorine concentration) and time. The bulk coefficient, k_b, can be determined using bench-scale tests for various initial chlorine concentrations versus holding times. Using the data from Table 3 and normalizing the final residual chlorine concentration, TRC, by dividing by C_o, and plotting these data points versus time, results in Figure 2. For a semi-log plot, the relationship can be assumed to be a straight line, with the slope representing the decay rate, namely, k_b. Performing linear regression using the method of least squares yields a bulk decay coefficient (k_b) of 0.0013 min^{-1} for the data. Interestingly, this bulk coefficient compares very favorably with the bench-scale test results of Huang et al. (1997), of 0.00139 min^{-1} for the Broward Co. outfall.

The value of 0.0013 min^{-1} for k_b will be used in the chlorine decay model as follows:

$$C(t) = C_o e^{-Kt} \qquad \text{(Equation 16)}$$

in which $K = k_b + \dfrac{k_w k_f}{R_h(k_w + k_f)}$

The two remaining coefficients, namely, k_w and k_f, can be determined using the least squares approach by optimizing a best fit equation using the *<u>Solver</u>* macro program of Microsoft Excel, and $R_h = 0.380$m. Using a least squares optimization of the field data for TRC, C_o, and time, and using the known values for k_b and R_h, produces the following results:

$$k_w = 0.0034 \text{ min}^{-1}$$

$$k_f = 0.102 \text{ min}^{-1}$$

Simplifying, the model equation becomes:

$$TRC = C_o e^{-0.00935t} \qquad \text{(Equation 17)}$$

This equation is plotted in Figure 3 against the field data and yields a robust correlation coefficient of R^2=0.96. An examination of the residuals shows a good fit between the actual and predicted data sets.

From Equation 17, the mean overall decay coefficient K, is 0.00935 min^{-1}. By comparison, the bulk decay coefficient, k_b, is 0.0013 min^{-1}, suggesting that the greatest level of TRC decay occurs in the outfall as a result of the combination of wall decay and mass transfer. Once again, it is interesting to observe that Huang et al. (1997) reached a similar conclusion with respect to the Broward Co. outfall.

Knowing the relationship between TRC, C_o, and time, allows one to use Equation 17 to calculate an allowable C_o for any given detention time, i.e., for any given effluent flow rate. Using TRC permit limits of 0.5 mg/L and 1.0 mg/L, enables Table 5 to be constructed for various rates of effluent flow. The data points from Table 5 have also been plotted in Figure 4, which shows graphically the allowable initial chlorine residuals at the effluent flume (C_o) as a function of effluent flow rates, for TRC permit limits of 0.5 mg/L and 1.0 mg/L.

From Equation 17, one can determine the allowable concentration in the effluent flume (C_0) for any flow rate that would ensure that TRC discharge permits limits are met. For example, using the design flow of 36 MGD, and the permitted monthly average discharge TRC limit of 0.5 mg/L, yields an allowable C_0 of 0.623 mg/L. Likewise, for the permitted daily TRC maximum limit of 1.0 mg/L, and a design flow of 36 MGD, the allowable C_0 would be 1.25 mg/L.

As previously indicated, the results for bench-scale testing of November 3 and 10, 1997, for fecal coliform inactivation decay are shown in Table 4. The plot of CT (C_0 x time) versus the natural log of the ratio of final fecal coliform counts to initial fecal coliform concentration is shown in Figure 5. The initial fecal coliform count is >120,000 and is conservatively taken to be 1,000,000. With increasing C_0 concentrations and detention times, there is an observed exponential decay of fecal coliform. Therefore, the bench-scale tests confirm the general Chick-Watson relationship indicating the benefits of additional detention time in reducing fecal coliform counts.

CONCLUSIONS AND RECOMMENDATIONS

In addition to the disinfection contact time provided by the chlorine contact chamber at the Plum Island WWTP, considerable detention time is provided in the 1219 m outfall pipe prior to discharge of the effluent through the diffusers into Charleston Harbor. Results from bench scale and field tests demonstrate that effluent TRC decay can be described using first order kinetics based upon a model developed and applied by Rossman et al. (1994), Vasconcelos et al. (1995), and Huang et al. (1997). The overall decay coefficient determined from the field tests (0.00935 min^{-1}), is significantly greater than the bulk decay coefficient (0.0013 min^{-1}), suggesting that TRC decay is dominated by consumption by the pipe wall bio-film and mass transfer within the outfall. In addition, results from bench-scale testing indicate significant fecal coliform reduction with additional contact time. The reduction of both TRC and fecal coliform in the outfall pipe is very significant and therefore should not be ignored when evaluating the actual impact on the receiving waters.

With this information, the following recommendations can be made without changing the NPDES permit discharge limits for either TRC or fecal coliform:

1. The current NPDES permit sampling point for fecal coliform is the effluent flume which ignores the demonstrated and significant fecal coliform reduction in the outfall pipe. In order to compensate for the additional contact time provided by the outfall pipe, a holding time for the sample from the effluent flume should be provided which is equivalent to the

theoretical detention time provided in the outfall pipe, which is a function of the plant flow rate.

2. The current NPDES permit limits for TRC are based upon sampling at the effluent flume which ignores the significant TRC decay in the outfall pipe. To compensate, one of two approaches is recommended. In the first approach, the allowable chlorine concentration in the effluent flume (C_0) can be predicted, based upon the chlorine decay model (Equation 17), such that the discharged TRC level would never theoretically exceed the permitted TRC limit. For example, at a design flow of 36 MGD, in order to meet a TRC permit limit of 0.5 mg/L, the allowable C_0 would be 0.623 mg/L. In the second approach, the TRC level could be reported for NPDES permit compliance after the holding time referenced above, although the second approach would be extremely conservative.

APPENDIX I.

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APPENDIX II

NOTATION

The following symbols are used in this paper:

C = TRC concentration (mg/L);

C_o = TRC concentration at x=o or t=o (mg/L);

K = overall decay coefficient (min^{-1});

k_b = decay coefficient for the bulk solution (min^{-1});

k_w = decay coefficient associated with the pipe wall surface (min^{-1});

k_f = mass transfer coefficient (min^{-1});

R_h = hydraulic radius (m);

x = distance along pipe with x_o being at the effluent flume (m);

t = detention time in outfall pipe;

U = mean velocity of pipe flow

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Table 1. OutFall Detention Time as a Function of Plant Flow

Flow MGD	Travel Time (Min)	Flow MGD	Travel Time (Min)	Flow MGD	Travel Time (Min)	Flow MGD	Travel Time (Min)
10.0	84.6	20.5	41.3	31.0	27.3	41.5	20.4
10.5	80.6	21.0	40.3	31.5	26.9	42.0	20.1
11.0	76.9	21.5	39.3	32.0	26.4	42.5	19.9
11.5	73.6	22.0	38.5	32.5	26.0	43.0	19.7
12.0	70.5	22.5	37.6	33.0	25.6	43.5	19.4
12.5	67.7	23.0	36.8	33.5	25.3	44.0	19.2
13.0	65.1	23.5	36.0	34.0	24.9	44.5	19.0
13.5	62.7	24.0	35.2	34.5	24.5	45.0	18.8
14.0	60.4	24.5	34.5	35.0	24.2	45.5	18.6
14.5	58.3	25.0	33.8	35.5	23.8	46.0	18.4
15.0	56.4	25.5	33.2	36.0	23.5	46.5	18.2
15.5	54.6	26.0	32.5	36.5	23.2	47.0	18.0
16.0	52.9	26.5	31.9	37.0	22.9	47.5	17.8
16.5	51.3	27.0	31.3	37.5	22.6	48.0	17.6
17.0	49.8	27.5	30.8	38.0	22.3	48.5	17.4
17.5	48.3	28.0	30.2	38.5	22.0	49.0	17.3
18.0	47.0	28.5	29.7	39.0	21.7	49.5	17.1
18.5	45.7	29.0	29.2	39.5	21.4	50.0	16.9
19.0	44.5	29.5	28.7	40.0	21.1	50.5	16.8
19.5	43.4	30.0	28.2	40.5	20.9	51.0	16.6
20.0	42.3	30.5	27.7	41.0	20.6	51.5	16.4

Table 2. Summary of Field Data

Date	Flow MGD	TRC (mg/l) Plant Effluent	Plant Effluent pH	Plant Effluent Temp C	TRC (mg/l) Outfall Pipe	Detention Time (minutes)	BOD (mg/l)	SS (mg/l)
29-Aug	17	0.53	6.75	29.3	0.28	47	3.4	7.1
3-Sep	17	0.37	6.7	26.8	0.16	50	3.9	3.6
5-Sep	21.6	0.89	6.67	27.3	0.73	38	3.5	2.5
8-Sep	21	0.51	6.75	28.2	0.22	40	3.5	4
8-Sep	20.5	1.1	6.72	28.6	0.78	45		
9-Sep	22.5	1.45	6.7	28.3	0.97	37	3	2.7
9-Sep	17	0.44	6.76	28.4	0.38	47		
10-Sep	16.5	1.17	6.76	28.3	0.77	51	2.8	3.8
10-Sep	19	0.73	6.76	28.8	0.44	44		
11-Sep	35.6	0.3	6.67	28.2	0.25	23	3.5	3.5
11-Sep	35.5	0.45	6.89	28.6	0.24	22		
15-Sep	20.5	0.4	6.8	28.3	0.3	41	4.2	5.8

Table 3. Bench Scale TRC Decay Tests
for Nov. 3 and 10, 1997

Co(mg/L)	TRC(mg/L)	Time(min)	TRC/Co
0.27	0.22	85	0.81
0.33	0.20	42	0.61
0.36	0.32	24	0.89
0.35	0.34	19	0.97
0.37	0.28	85	0.76
0.40	0.26	42	0.65
0.49	0.44	24	0.90
0.48	0.39	19	0.81
0.64	0.50	85	0.78
0.68	0.48	42	0.71
0.75	0.71	19	0.95
0.36	0.27	85	0.75
0.39	0.32	42	0.82
0.47	0.43	19	0.91
0.56	0.50	85	0.89
0.65	0.56	42	0.86
0.66	0.65	24	0.98
0.71	0.59	19	0.83
0.77	0.72	85	0.94
0.82	0.77	42	0.94
0.89	0.88	24	0.99
0.95	0.84	19	0.88

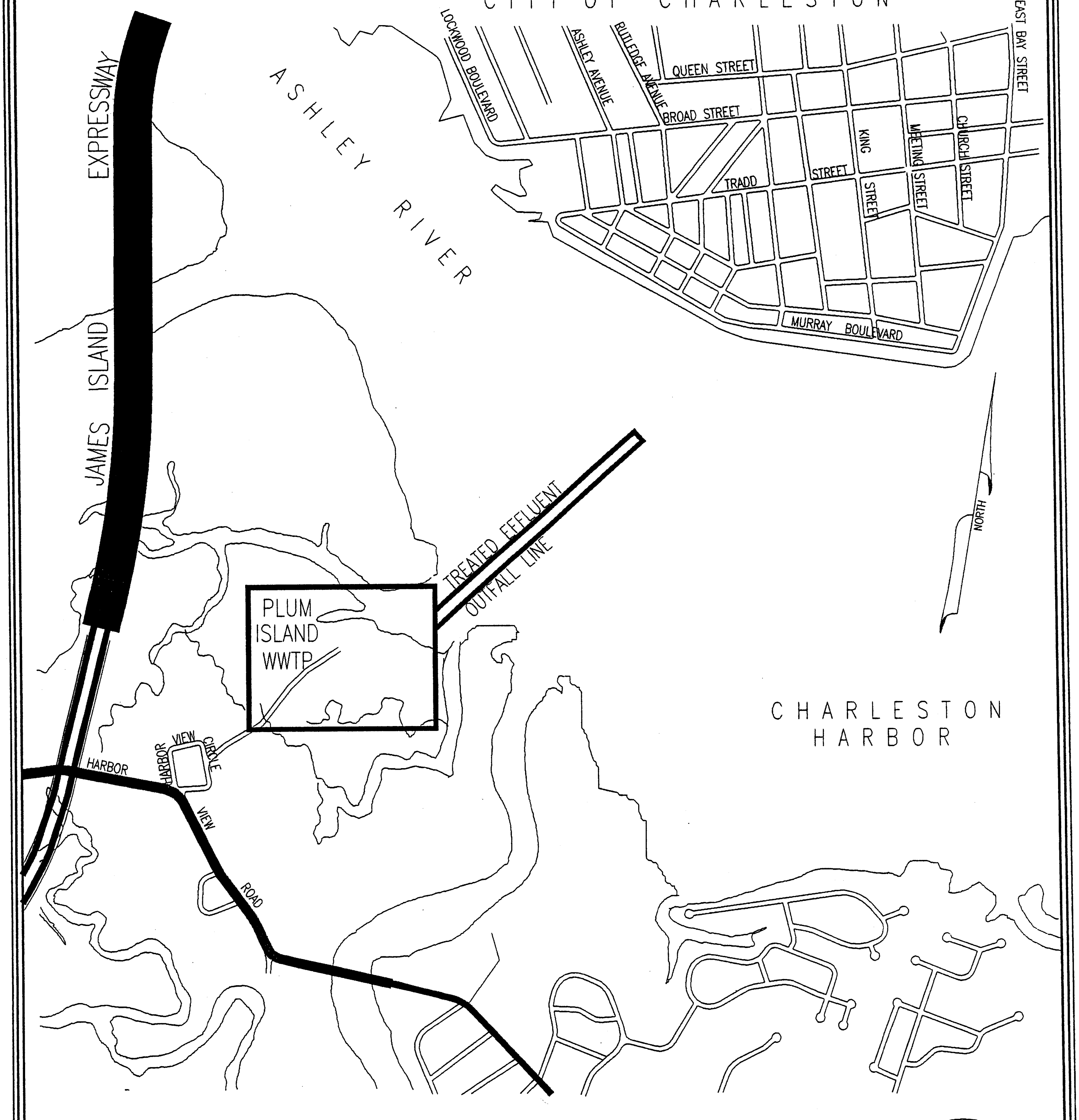
Table 4. Bench Scale Test
for Fecal Coliform -v- CT (TRC*Detention time)

Fecal Coliform # / 100ml	CT (min-mg/L)
120000	9.9
76000	19.8
2000	29.7
88	39.6
68	49.5
120000	4.9
37000	9.8
16000	14.7
300	19.6
200	24.5
120000	2.8
63000	5.6
38000	8.4
6000	11.2
400	14
112000	2.2
70000	4.4
57500	6.6
20000	8.8
3800	11
120000	18.4
88000	36.8
6000	55.2
300	73.6
15	92
15000	9.1
17000	18.2
6000	27.3
32	36.4
28	45.5
100	5.2
30500	10.4
36000	15.6
200	20.8
100	26
120000	4.1
53500	8.2
20000	12.3
2300	16.4
100	20.5

Table 5. Calculated Co as a Function of Flow and TRC = 0.5 mg/l and 1.0 mg/l

(from Equation 17)

		Co	
Flow (MGD)	TIME(min)	TRC = 0.5	TRC = 1.0
5	169.20	2.43	4.86
6	141.00	1.87	3.74
8	105.75	1.34	2.69
10	84.60	1.10	2.21
12	70.50	0.97	1.93
16	52.88	0.82	1.64
18	47.00	0.78	1.55
20	42.30	0.74	1.49
22	38.45	0.72	1.43
24	35.25	0.70	1.39
30	28.20	0.65	1.30
36	23.50	0.62	1.25
42	20.14	0.60	1.21
48	17.63	0.59	1.18
54	15.67	0.58	1.16
60	14.10	0.57	1.14
66	12.82	0.56	1.13
72	11.75	0.56	1.12
78	10.85	0.55	1.11



SITE MAP OF STUDY AREA

FIGURE 1

COMMISSIONERS OF PUBLIC WORKS OF THE CITY OF CHARLESTON S.C.

Figure 2. Bench - Scale Test for
Ratio of TRC to C_o v. Time
November 3 & 10, 1997

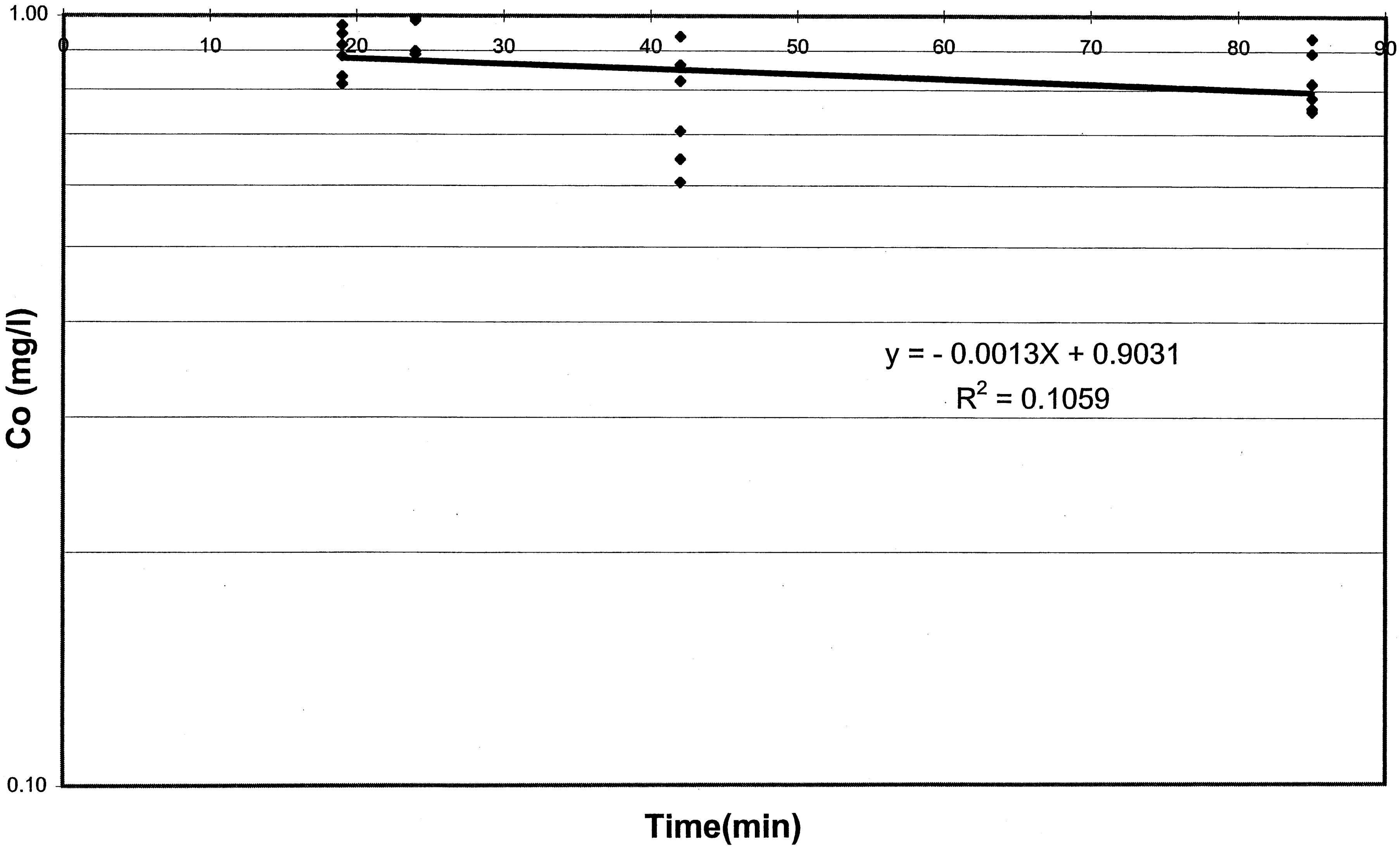


Figure 5. CT -v- the Natural Log of the Ratio of Final to Initial Fecal Coliform Counts of Bench-Scale Test