CHLORINATION OF WASTE STABILIZATION LAGOON EFFLUENT

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4.3.7 DISINFECTION

4.3.7.1 RATIONALE

Municipal wastewaters contain a variety of infectious microorganisms such as salmonellae, shigellae, enteropathogenic Escherichia coli, Pseudomonas aeruginosa, and enteric viruses. Outbreaks of gastroenteritis, typhoid, shigellosis, salmonellosis, ear infections, and infectious hepatitis have been reported for people drinking or swimming in waters mixed with municipal wastewaters. Outbreaks also have occurred when people eat raw shellfish harvested from waters contaminated with municipal wastewaters.

All discharges do not necessarily contain all or part of the pathogens mentioned above, and it may be possible to escape contact with these organisms most of the time. However, practical public health practices dictate that constant protection be provided, because it is impossible to detect the presence of pathogens before they are discharged in a wastewater. Consequently disinfection of wastewater discharges must be practiced continuously.

Experience and judgment have shown that reducing the fecal coliform concentration to 14 per 100 ml or the total coliform concentration to 70 per 100 ml will prevent disease outbreaks caused by shellfish (EPA, 1976). Limited epidemiological data indicates that concentrations of fecal coliform of approximately 200 per 100 ml in recreational waters reduces the probability of contact to an acceptable level (EPA, 1976). Although the U.S. Environmental Protection Agency Secondary Effluent Standards no longer contain a standard for fecal coliform, the logic for its inclusion initially was based on the limited epidemiological data referred to above.

4.3.7.2 DISINFECTANTS

Many chemicals and physical agents are good disinfectants. Heat, sunlight, chlorine, bromine, iodine, potassium permanganate, chlorine dioxide, ozone, and ultraviolet light are effective disinfectants, but experience with most of these materials as a wastewater disinfectant is limited. Because of the extensive experience with chlorine, it is and will likely continue to be the most widely used disinfectant. The principal disadvantage of chlorination is the production of toxic substances and its effect on aquatic life.

4.3.7.3 TOXIC EFFECTS

The production of halogenated organic compounds suspected of being toxic to man has produced concern by public health officials, and efforts are being directed toward developing other methods of disinfection. The implementation of dechlorination to reduce the toxicity of wastewater discharges to natural environments has resulted in concern about the compounds produced by the reactions between the forms of residual chlorine and dechlorinating agents such as sulfur dioxide, sodium bisulfite, sodium sulfite or activated carbon. This effort will eventually result in a reduction in the use of chlorine, but immediate changes are not likely to occur.

Many of the proposed substitutes may have the same disadvantages associated with chlorination. Ozone is used extensively in Europe to disinfect drinking water, but little is known about its interaction with organic matter in wastewaters. All of the disinfectants listed above have disadvantages, and most suffer from high costs, inefficiency in wastewaters with solids, toxic side effects, and no residual. The other halogens will probably have disadvantages similar to those of chlorine.

4.3.7.4 BASIC PRINCIPLES OF CHLORINATION

To understand the effects of chlorinating stabilization pond effluents, it is necessary to review the basic principles of chlorination. When chlorine gas is used the gas reacts with water to form hypochlorous acid (HOCl). In a pure water system, the reaction is as follows:

$$Cl_2 + H_2O \longrightarrow HOCl + H^+ + Cl^- \quad . . . . . (1)$$

The hypochlorous acid then disassociates to form OCl^- and H^+

$$HOCl \rightleftharpoons H^+ + OCl^- \quad . . . . . (2)$$

When $Ca(OCl)_2$, for example, is used to chlorinate, OCl^- is formed by the following reaction:

$$Ca(OCl)_2 \longrightarrow Ca^{++} + 2OCl^- \quad . . . . . (3)$$

The OCl^- is then free to form hypochlorous acid in contact with hydrogen ions. Chlorine in the form of HOCl or OCl^- is referred to as free chlorine. Both forms of free chlorine are powerful disinfectants and react quickly to destroy bacteria and most viruses.

In wastewater, such as stabilization pond effluents, various chemical components react with free chlorine to form compounds which are ineffective as disinfectants. That is, the rates of reactions between chlorine and these components are faster than the rate at which chlorine attacks and kills bacteria and viruses. Fe^{++}, Mn^{++}, $NO_2^=$ and $S^=$ are common reducing agents which combine readily with chlorine to prevent it from disinfecting. A typical reaction is as follows:

$$H_2S + 4Cl_2 + 4H_2O \longrightarrow H_2SO_4 + 8HCl \quad . . . . . (4)$$

Free chlorine also reacts with ammonia found in wastewater to form a series of compounds known as chloramines. Although chloramines are less than 5 percent as efficient as free chlorine in destroying bacteria and

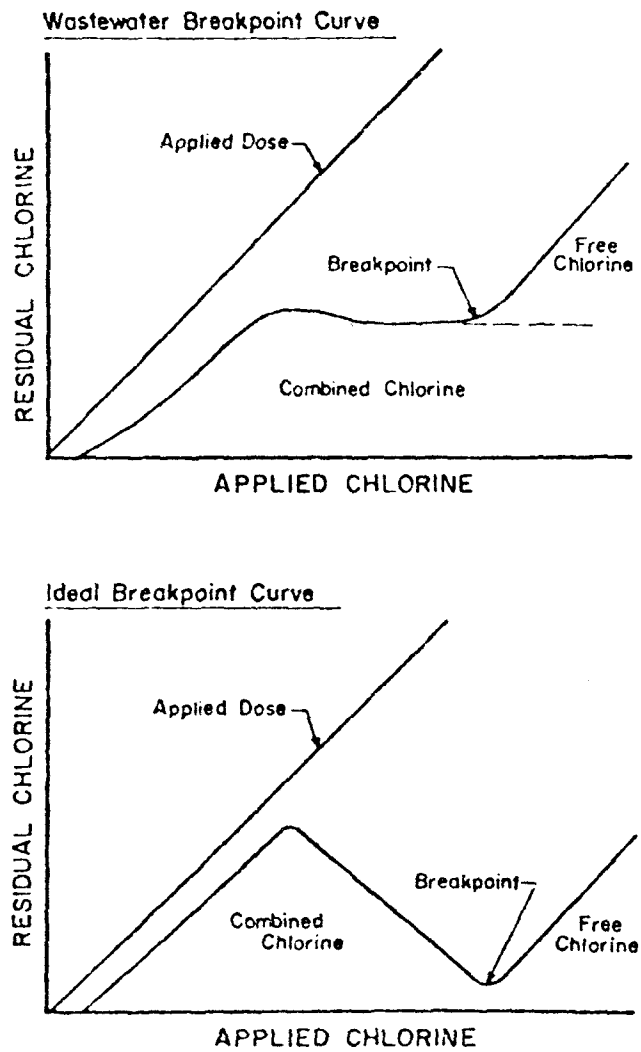


Figure 4.1. Comparison of ideal and wastewater chlorination curves.

viruses, they do play an important role in disinfection because they are fairly stable and can continue to provide disinfection for some time after application. The common forms of chloramines, or combined chlorine, as they are referred to, are monochloramine, dichloramine, and nitrogen trichloride. The reactions for their formation are as follows:

$$NH_3 + HOCl \longrightarrow NH_2Cl + H_2O \quad (5)$$

$$NH_2Cl + HOCl \longrightarrow NHCl_2 + H_2O \quad (6)$$

$$NHCl_2 + HOCl \rightleftharpoons HCl_3 + H_2O \quad (7)$$

In some cases, chlorination is used as a treatment step to drive off undesirable ammonia. This is known as breakpoint chlorination. Basically, chlorine is added until all the chlorine has reacted to form chloramines. With the addition of more chlorine, the ammonia is converted to nitrogen gas and driven off. Any additional chlorine added beyond that point is maintained in solution as free chlorine residual. The mechanisms involved are complex, but the overall reaction may be represented as follows:

$$2NH_3 + 3HOCl \longrightarrow N_2\uparrow + 3HCl + 3H_2O \quad (8)$$

A comparison of ideal breakpoint chlorination and wastewater breakpoint chlorination is presented in Figure 4.1. Because the chlorine dose necessary to reach the breakpoint in wastewater is much higher than the dose necessary to achieve adequate disinfection, breakpoint chlorination is seldom used in the treatment of wastewater.

4.3.7.5 EFFECTS OF CHLORINATING LAGOON EFFLUENTS

Since chlorine, at present, is less expensive and offers more flexibility than other means of disinfection, chlorination, is the most practical method of reducing bacterial populations. However, there is evidence that chlorination of wastewater high in organic nitrogen content, such as stabilization pond effluent, may be accompanied by adverse effects.

White (1973) has suggested that chlorine demand is increased by high concentrations of algae commonly found in pond effluents. It was found that to satisfy chlorine demand and to produce enough residual to effectively disinfect algae laden wastewater within 30-45 minutes, a chlorine dose of 20-30 mg/l was required. Kott (1971) also reported increases in chlorine demand as a result of algae, but found that a chlorine dose of 8 mg/l was sufficient to produce adequate disinfection within 30 minutes and that if contact times are kept relatively short, no serious chlorine demand by algae cells is encountered. Of course, the amount of chlorine demand exerted is highly variable. Dinges and Rust (1969) found that for pond effluents, a chlorine demand of only 2.65 to 3.0 mg/l was exerted after 20 minutes of contact. Brinkhead and O'Brien (1973) found that at low doses of chlorine, very little increase in chlorine demand is attributable to algae, but at higher doses, the destruction of algae cells greatly increases demand. This is because dissolved organic compounds released from destroyed algae cells, as explained by Echelberger et al. (1971), are oxidized by chlorine and thus increase chlorine demand.

Another concern regarding the chlorination of pond effluents is the effects on biochemical oxygen demand (BOD_5) and chemical oxygen demand (COD). Brinkhead and O'Brien (1973) and Echelberger et al. (1971) found that for higher chlorine doses, increases in BOD_5 due to destruction of algae cells were observed. Echelberger et al. (1971) also reported increases in soluble COD. Hom (1972) found that when 2.0 mg/l chlorine was applied to pond effluent, the BOD_5 measured was 20 mg/l. However, when 64 mg/l chlorine was applied, the BOD_5 increased to 129 mg/l. However, Zaloum and Murphy (1974) observed a 40 percent reduction of BOD_5 resulting from chlorination. Dinges and Rust (1969) also reported reductions of BOD_5. Kott (1971) has suggested

that increases in BOD_5 may be controlled by using low chlorine doses coupled with long contact periods.

The formation of toxic chloramines is also of concern in chlorinating pond effluents. These compounds are found in waters high in ammonia concentration and are extremely toxic to aquatic life found in receiving water. For example, Zillich (1972) has determined that a chloramine concentration of 0.06 mg/l is lethal to trout.

Not all of the side effects of chlorinating pond effluents are detrimental. Kott (1973) observed reductions of suspended solids (SS) as a result of chlorination. Dinges and Rust (1969) reported reductions of volatile suspended solids (VSS) by as much as 52.3 percent and improved water clarity (turbidity) by 31.8 percent following chlorination. Echelberger et al. (1971) reported that chlorine enhances the flocculation of algae masses by causing algae cells to clump together.

Four systems of identically designed chlorine mixing and contact tanks, each capable of treating 50,000 gallons per day, were used by Johnson et al. (1978) to study the chlorination of lagoon effluents. Three of the four chlorination systems were used for directly treating pond effluent. The effluent treated in the fourth system was filtered through an intermittent sand filter to remove algae prior to chlorination. The filtered effluent was also used as the solution water for all four chlorination systems.

Following recommendations by Collins, Selleck, and White (1971), Kothandaraman and Evans (1972 and 1974), and Marske and Boyle (1973), the chlorination systems were constructed to provide rapid initial mixing followed by chlorine contact in plug flow reactors. A serpentine flow configuration having a length to width ratio of 25:1, coupled with inlet and outlet baffles, was used to enhance plug flow hydraulic performance. The maximum

theoretical detention time for each tank was 60 minutes, while the maximum actual detention time averaged about 50 minutes.

The pond effluent was chlorinated at doses ranging from 0.25 to 30.0 mg/l under a variety of contact times, temperatures, and seasonal effluent conditions from August 1975 to August 1976. A variety of chemical, physical, and bacteriological parameters were monitored during this period in evaluating the chlorination of pond effluents. A series of laboratory experiments were also conducted to compliment the field study. Some of the major findings of this study are summarized below.

1. Sulfide, produced as a result of anaerobic conditions in the ponds during winter months when the ponds are frozen over, exerts a significant chlorine demand (Figure 4.2). For sulfide concentrations of 1.0 to 1.8 mg/l, a chlorine dose of 6 to 7 mg/l was required to produce the same residual as a chlorine dose of about 1 mg/l for conditions of no sulfide.

2. For all concentrations of ammonia encountered, it was found that adequate disinfection could be obtained with combined chlorine residual in 50 minutes or less of contact time. Therefore, breakpoint chlorination, and the subsequent production of free chlorine residual, was found to be rarely, if ever, necessary in disinfecting pond effluent.

3. It was found that total COD is virtually unaffected by chlorination. Soluble COD was found to increase with increasing concentrations of free chlorine only. This increase was attributed to the oxidation of suspended solids by free chlorine. Increases in soluble COD versus free chlorine residual are shown in Figure 4.3.

4. Some reduction in suspended solids, due to the break down and oxidation of suspended particulates, and resulting increases in turbidity were attributed to chlorination. However, this reduction was found to be of

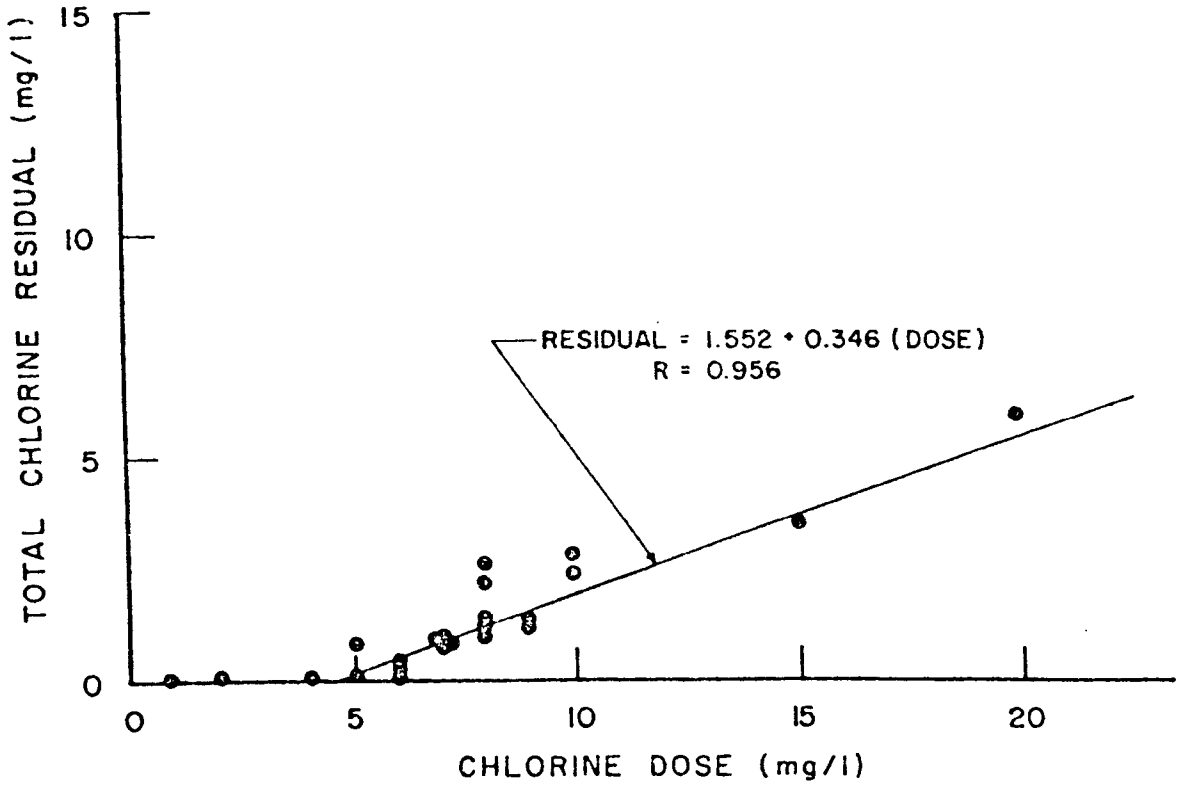


Figure 4.2. Chlorine dose vs. residual for initial sulfide concentrations of 1.0 - 1.8 mg/l.

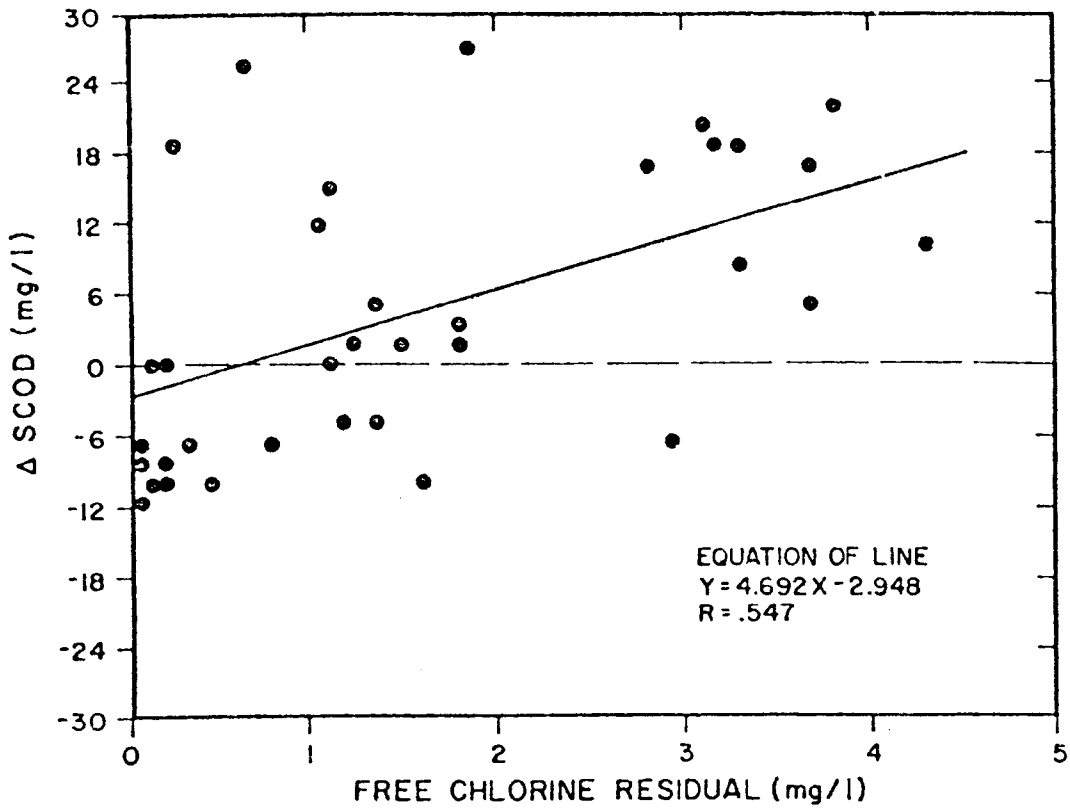


Figure 4.3. Changes in soluble COD vs. free chlorine residual--unfiltered lagoon effluent.

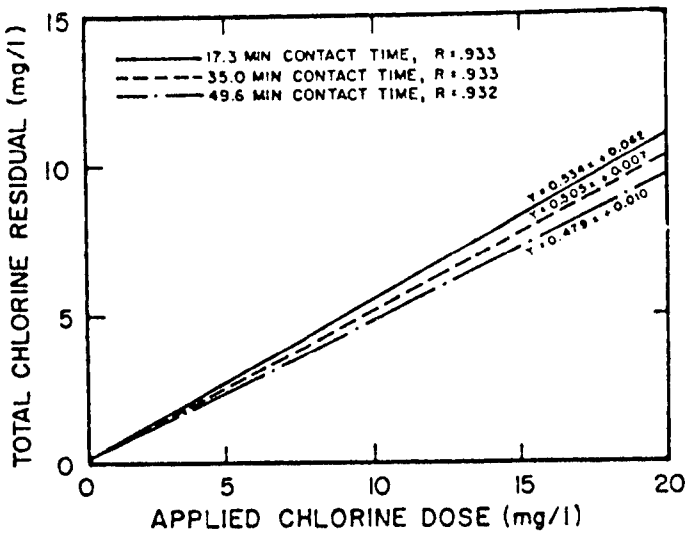
limited importance when compared with reductions of suspended solids resulting from settling. Suspended solids were reduced by 10 to 50 percent by settling in the contact tanks.

5. Filtered pond effluent exerted a lower chlorine demand than unfiltered pond effluent, due to the removal of algae (Figure 4.4). The rate of exertion of chlorine demand was determined to be directly related to chlorine dose and total chemical oxygen demand.

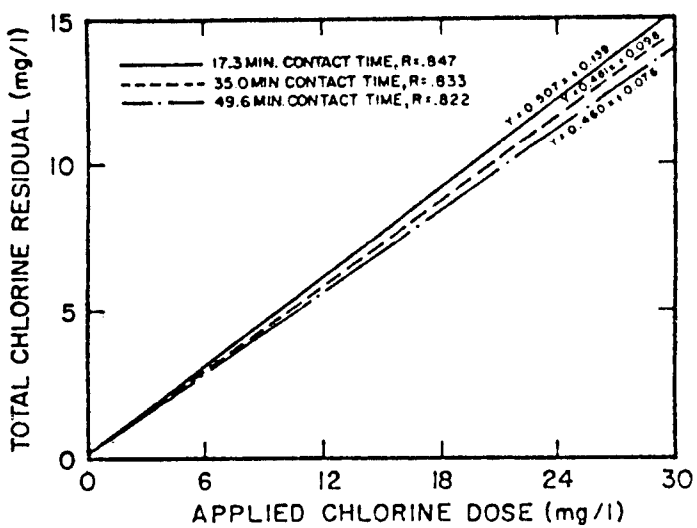
6. A summary of coliform removal efficiencies as a function of total chlorine residual for filtered and unfiltered effluent is illustrated in Figure 4.5. The rate of disinfection was a function of the chlorine dose and bacterial concentration. Generally, the chlorine demand was found to be about 50 percent of the applied chlorine dose except during periods of sulfide production.

7. Disinfection efficiency was temperature dependent. At colder temperatures, the reduction in the rate of disinfection was partially offset by reductions in the exertion of chlorine demand; however, the net effect was a reduction in the chlorine residual necessary to achieve adequate disinfection with increasing temperature for a specific contact period.

8. In almost all cases, adequate disinfection was obtained with combined chlorine residuals of between 0.5 and 1.0 mg/l after a contact period of approximately 50 minutes. This indicated that disinfection can be achieved without discharging excessive concentrations of toxic chlorine residuals into receiving waters. Also, it was found that adequate bacterial removal can be achieved with relatively low doses of applied chlorine during most of the year.

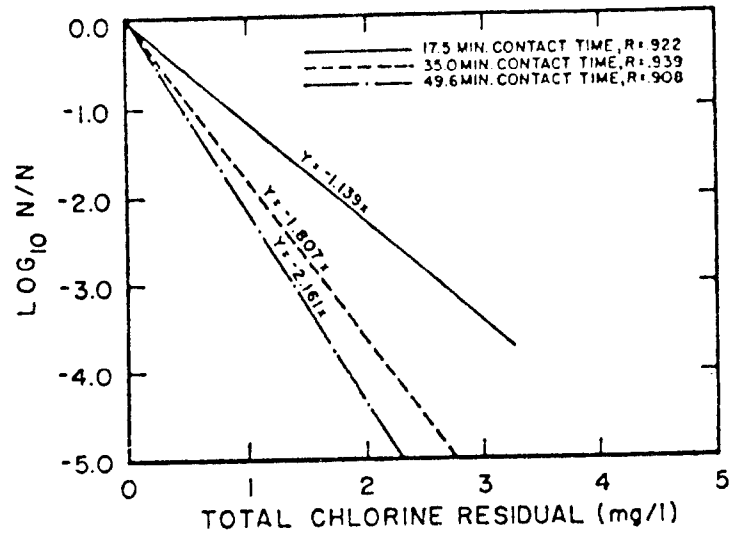


(a) Filtered Effluent

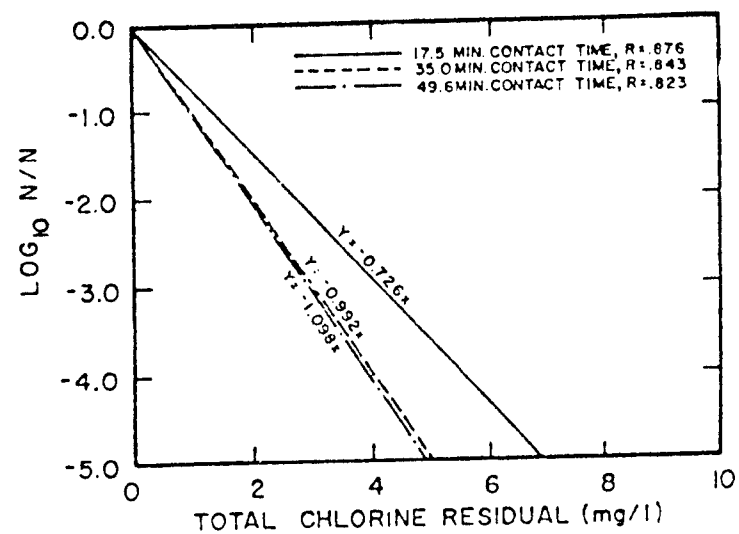


(b) Unfiltered Effluent

Figure 4.4. Chlorine dose vs. total residual--filtered and unfiltered effluent.



(a) Filtered Effluent



(b) Unfiltered Effluent

Figure 4.5. Coliform removal efficiencies--filtered and unfiltered effluent.

4.3.7.6 PREDICTING REQUIRED RESIDUALS

Using the data from the study summarized in section 4.3.7.5, Johnson et al. (1978) developed a model to predict the chlorine residual required to obtain a specified bacterial kill.

The model was used to construct a series of design curves for selecting chlorine doses and contact times for achieving desired levels of disinfection. An example may best illustrate how these design curves are applied. Assume that a particular lagoon effluent is characterized as having a fecal coliform (FC) concentration of 10,000 per 100 ml, 0 mg/l sulfide, 20 mg/l TCOD, and a temperature of 5°C. If it is necessary to reduce the FC counts to 100 per 100 ml, a combined chlorine residual (CCL) sufficient to produce a 99 percent bacterial reduction must be obtained. If an existing chlorine contact chamber has an average residence time of 30 min, the required chlorine residual is obtained from Figure 4.6. A 99 percent bacterial reduction corresponds to log (N_0/N) equal to 2.0. For a contact period of 30 min, a combined chlorine residual of between 1.0 and 1.5 mg/l is required to produce that level of FC reduction. Upon interpolation, the actual chlorine residual is determined to be 1.3 mg/l. This is indicated by Point ① in Figure 4.6.

Going to Figure 4.7, it is determined that if a chlorine dose produces a residual of 1.30 mg/l at 5°C, the same dose would produce a residual of 0.95 mg/l at 20°C. This is because of the faster rate of reaction between total chemical oxygen demand (TCOD) and chlorine at the higher temperature. This is indicated by Point ② in Figure 4.7. For an equivalent chlorine residual of 0.95 mg/l at 20°C and 20 mg/l TCOD, it is determined from Figure 4.8 that the same chlorine dose would produce a residual of 0.80 mg/l if the TCOD were 60 mg/l. This is because higher concentrations of TCOD

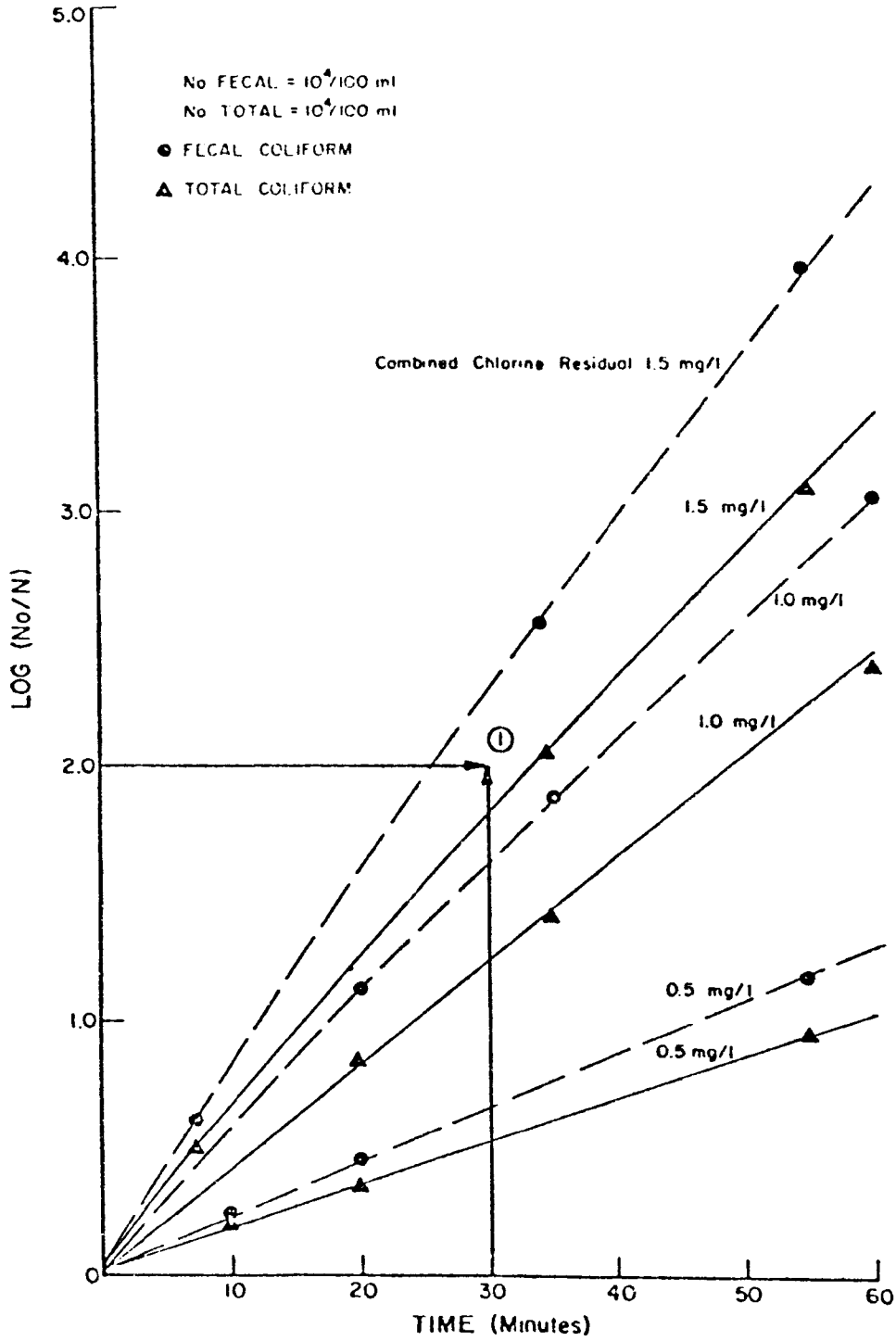


Figure 4.6. Combined chlorine residual at 5°C for coliform = 10^4/100 ml.

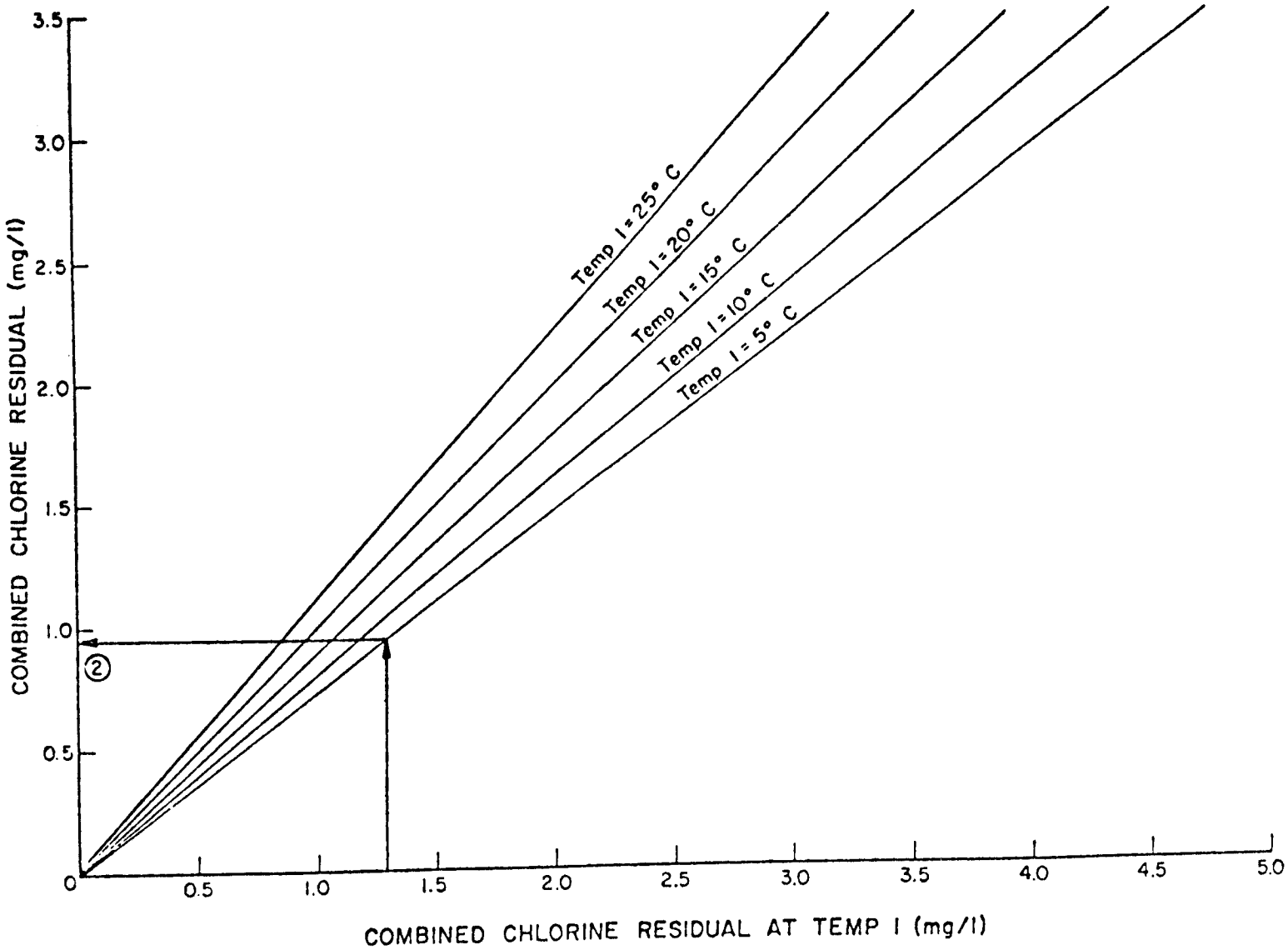


Figure 4.7. Conversion of combined chlorine residual at Temp 1 to equivalent residual at 20°C.

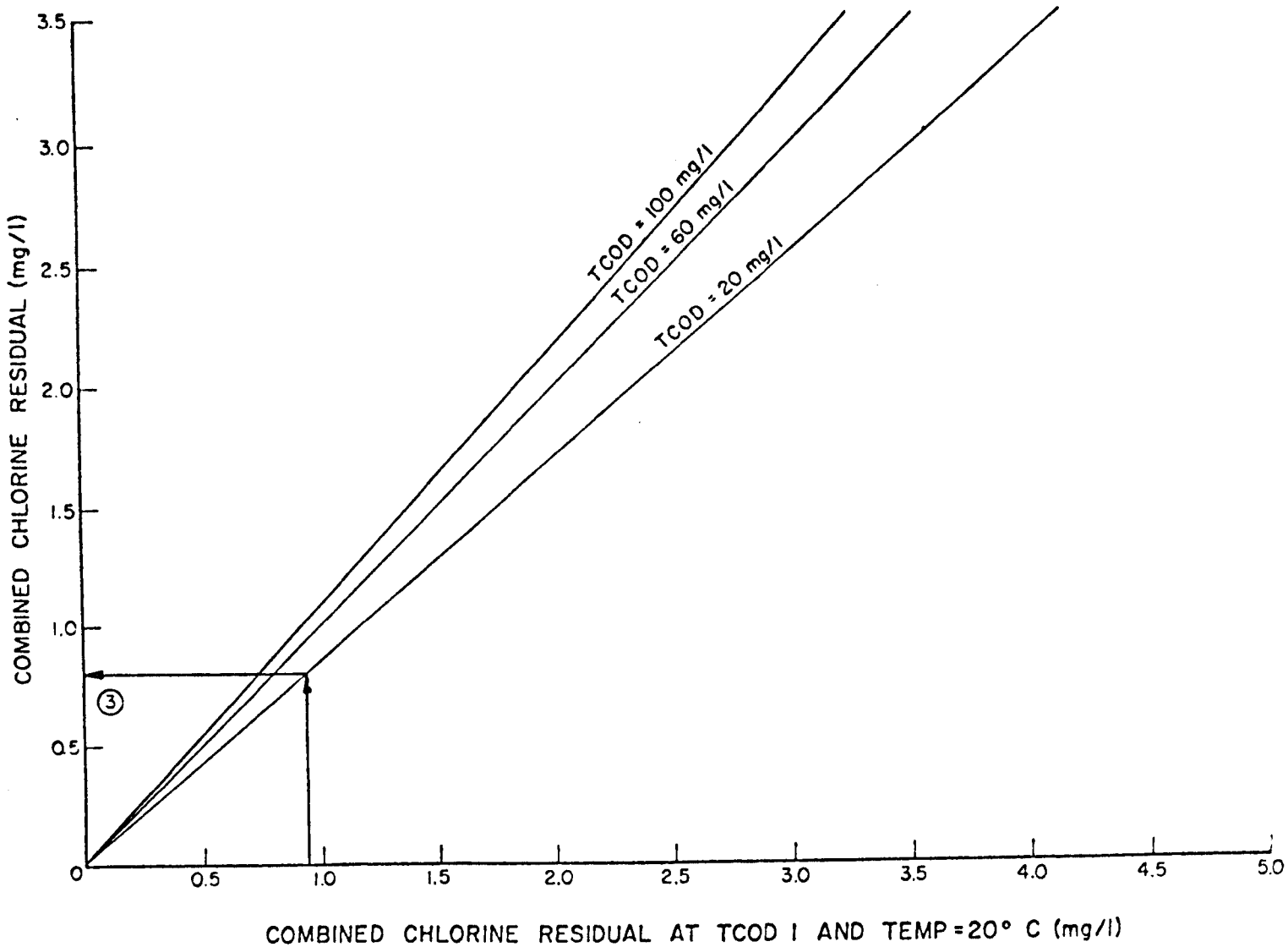


Figure 4.8. Conversion of combined chlorine residual at TCOD1 and 20°C to equivalent residual at 20°C and TCOD = 60 mg/l.

increase the rate of chlorine demand. Point ③ on Figure 4.8 corresponds to this residual. The chlorine dose required to produce an equivalent residual of 0.80 mg/l to 20°C and 60 mg/l TCOD is determined from Figure 4.9. For a chlorine contact period of 30 min, a chlorine dose of 2.15 mg/l is necessary to produce the desired combined residual as indicated by Point ④ on Figure 4.9. This dose will produce a reduction of FC from 10,000 per 100 ml to 100 per 100 ml within 30 min at 5°C and with 20 mg/l TCOD.

If, in the previous example, the initial sulfide concentration was 1.0 mg/l instead of 0 mg/l, it would be necessary to go directly from Figure 4.6 to Figure 4.10. Here, chlorine residual of 1.30 mg/l at the TCOD of 20 mg/l and a temperature of 5°C is converted to an equivalent chlorine residual of 1.10 mg/l for a TCOD of 60 mg/l. This is represented by Point ⑤ on Figure 4.10. Going to Figure 4.11, which corresponds to an initial sulfide concentration of 1.0 mg/l, it is determined that a chlorine dose of 6.65 mg/l is necessary to produce an equivalent chlorine residual of 1.1 mg/l after a contact period of 30 min. Point ⑥ on Figure 4.11 corresponds to this dose. The sulfide remaining after chlorination is determined to be 0.44 mg/l from Figure 4.12 as indicated by Point ⑦.

4.3.7.7 DESIGN OF CHLORINE CONTACT TANKS

Although the degree of bacterial kill is proportional to the concentration of chlorine dose times the contact time, disinfection of wastewater does not necessarily follow Chick's Law (Collins et al., 1971). Chick's Law states that,

$$\ln \frac{N}{N_0} = - kt$$

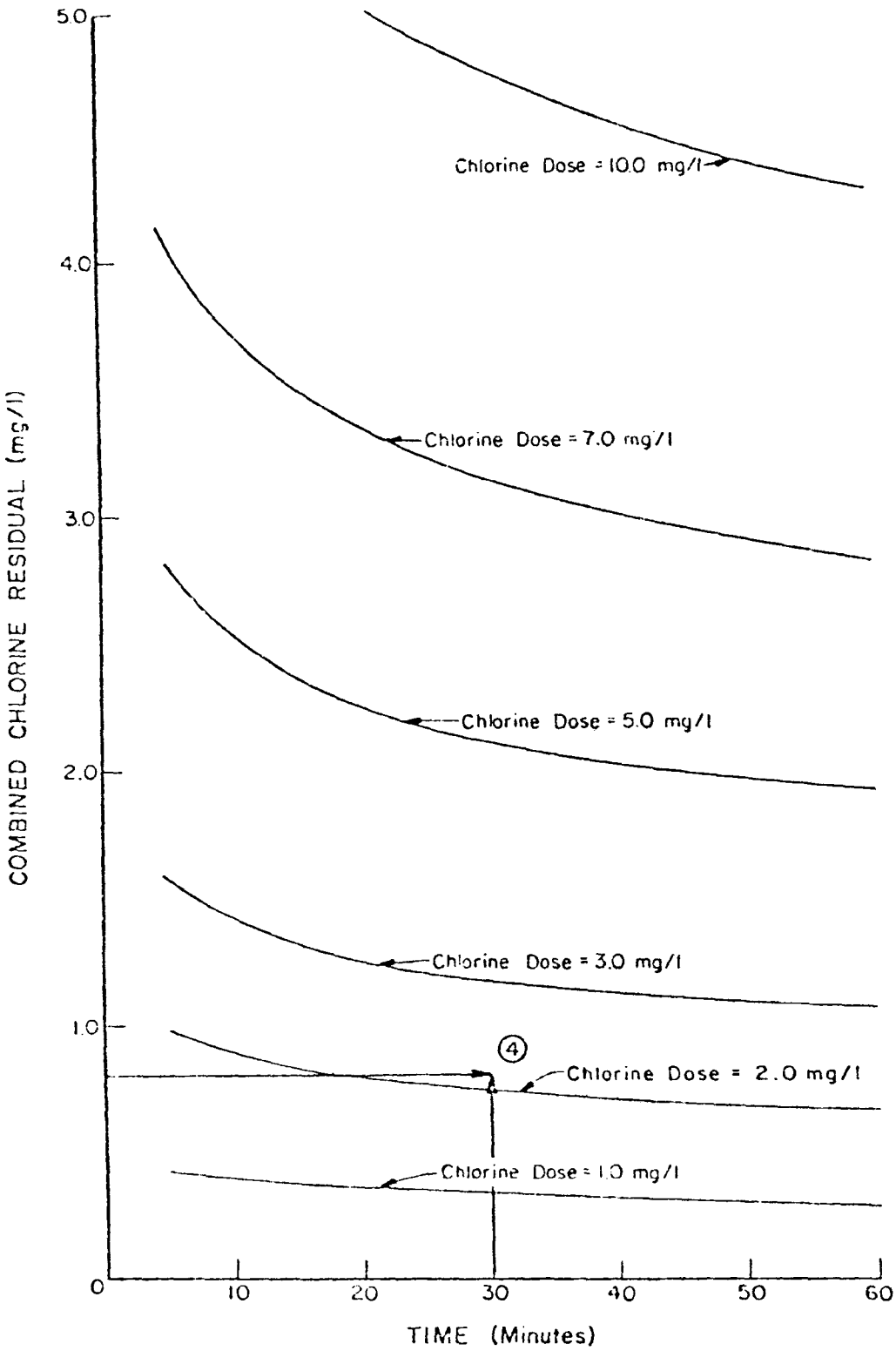


Figure 4.9. Determination of chlorine dose required for equivalent combined residuals at TCOD = 60 mg/l and Temp. = 20°C.

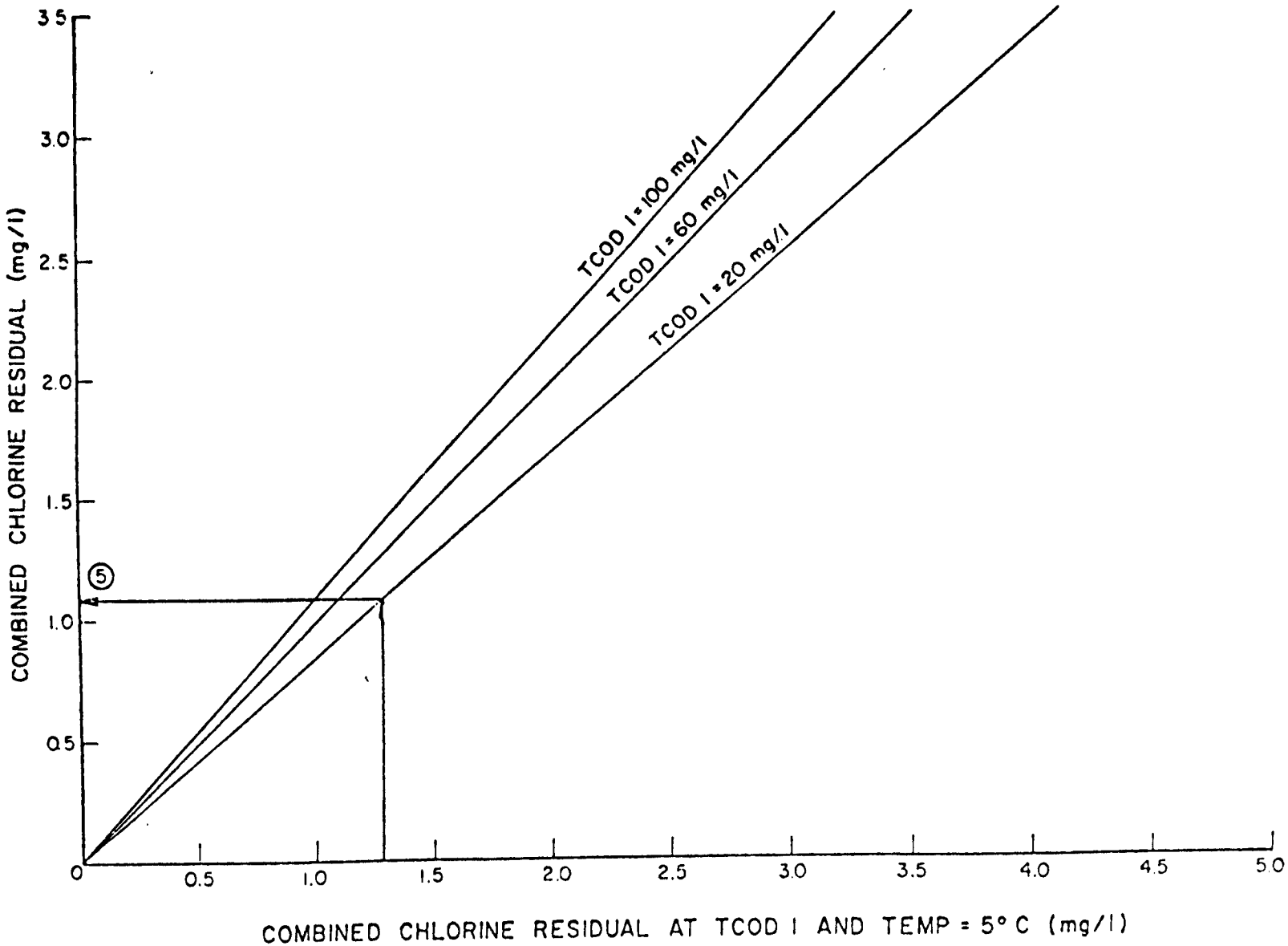


Figure 4.10. Conversion of combined residual chlorine at 5°C and TCOD1 to equivalent residual at 5°C and TCOD = 60 mg/l.

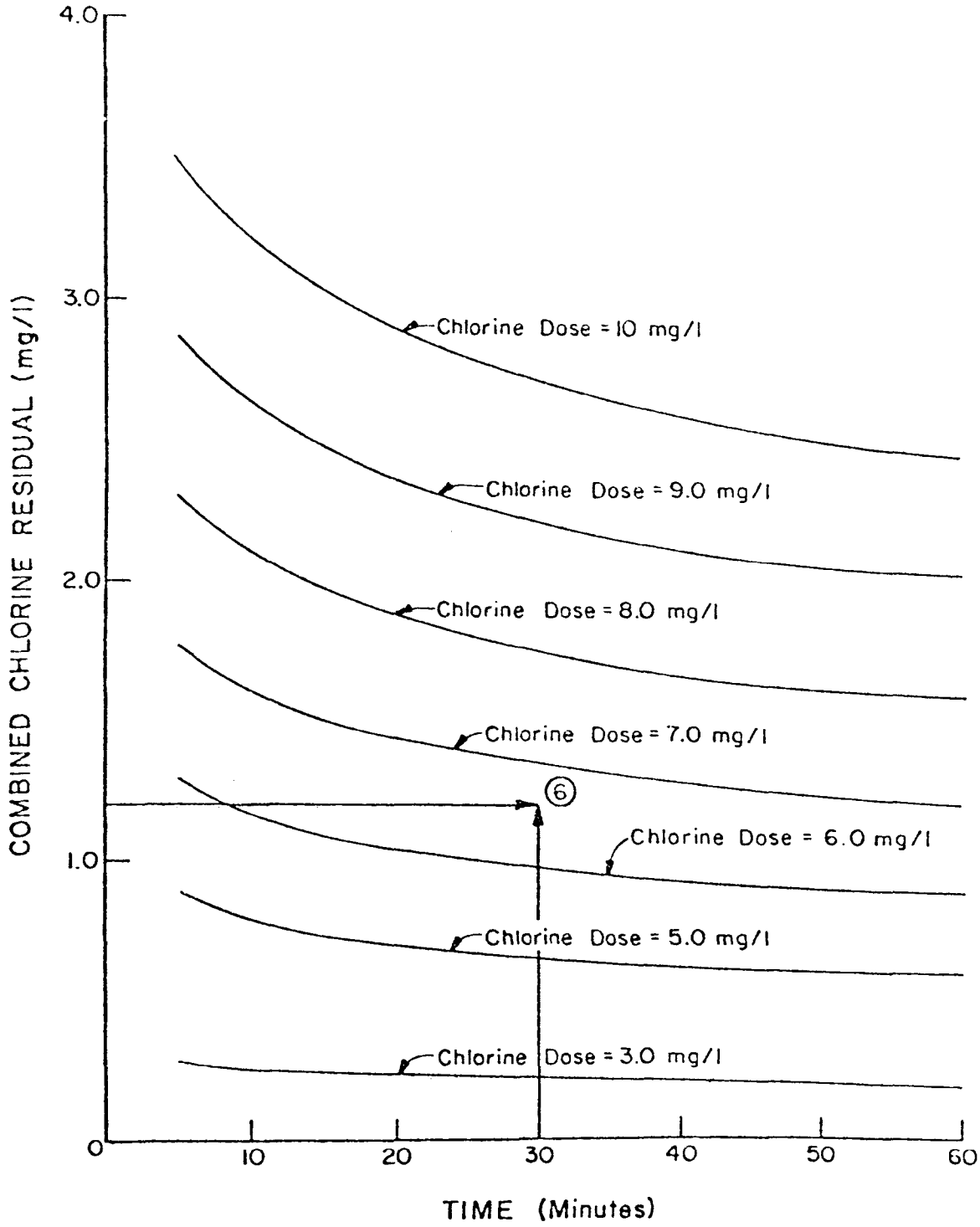


Figure 4.11. Determination of chlorine dose required when $S^{=}$ = 1.0 mg/l, TCOD = 60 mg/l, and Temp. = 5°C.

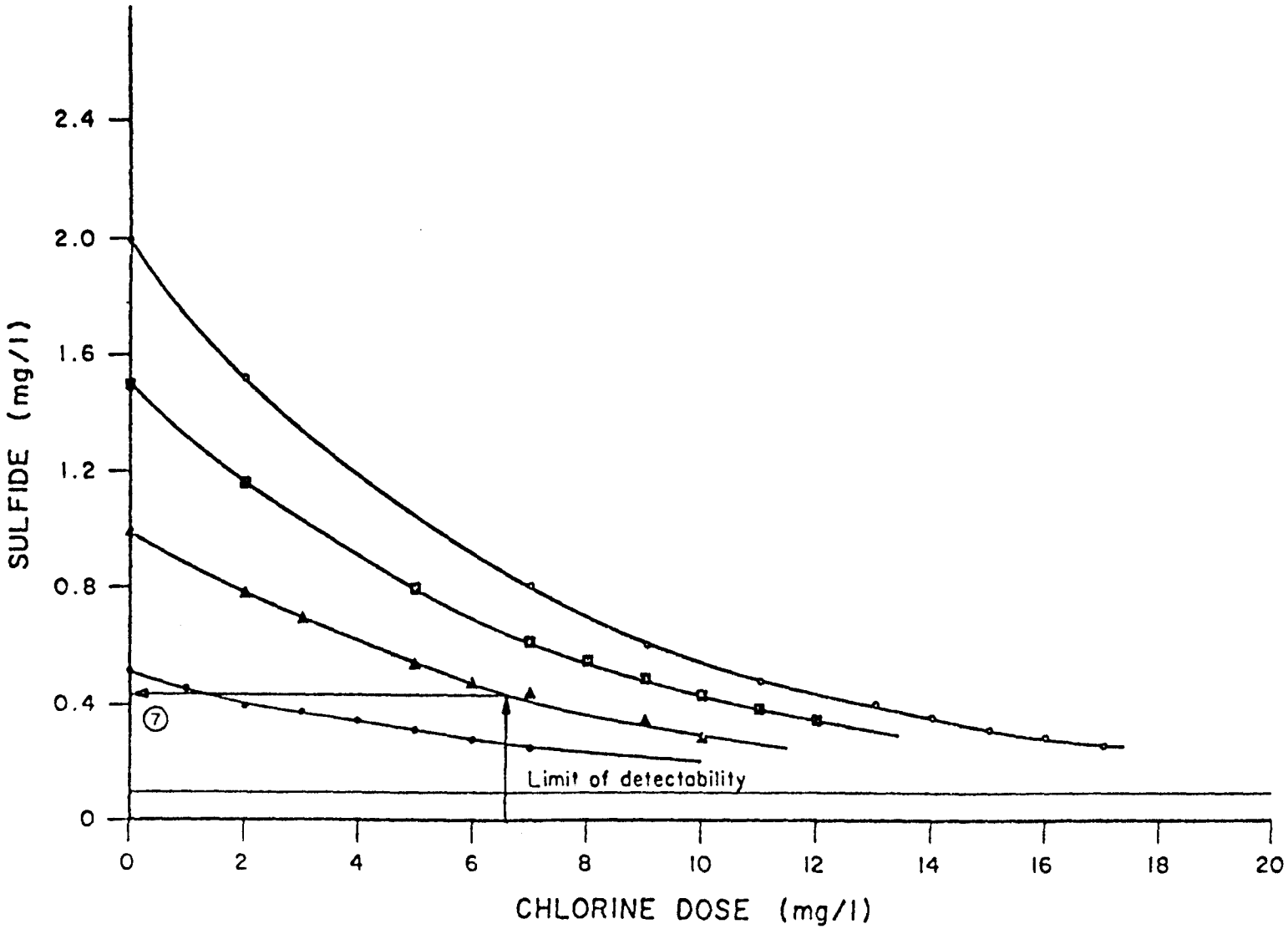


Figure 4.12. Sulfide reduction as a function of chlorine dose.

where N is the number of organisms remaining at time t, N_0 is the initial number of organisms, and k is a constant. The deviation from Chick's Law can largely be attributed to the fact that the disinfectant forms of chlorine in wastewater are mostly chloramines, rather than free chlorine. Chloramines not only decrease the disinfectant properties of the chlorine residual, but may also result in differences in susceptability of organisms exposed to chloramines. Differences in degree of exposure and increases in resistance triggered by the exposure of organisms to the disinfectant also affect the way in which chlorine acts to destroy microorganisms. As a result of deviations from Chick's Law, either the time of exposure or the chlorine dose must be increased to produce the same bacterial kill in wastewater as in water.

Problems associated with the design of contact tanks stem from the fact that most designs are based on a theoretical detention time determined by dividing the tank volume by the flowrate. In practice, actual detention times may vary between 30 and 80 percent of the theoretical detention times (Deaner). Shorter residence times are caused by dead spaces and short-circuiting and result in decreases in chlorination efficiencies and increases in solids accumulations (Kothandaram and Evans, 1974). With shorter contact times and extra chlorine demands exerted by the build-up of solids, chlorine concentrations must be increased to produce desired degrees of disinfection. Increasing the chlorine dose often has serious drawbacks. As well as being an inefficient way to utilize the disinfectant properties of chlorine, it also increases operational costs. This approach also increases the concentration of undesirable compounds discharged into the environment (Hart et al., 1975). Increasing the chlorine dose is also hard

on equipment, because of corrosion resulting from the contact of equipment with high chlorine concentrations.

Short-circuiting has another effect on adequate operations of chlorine contact tanks. With short-circuiting, residence times may be continually changing. This causes difficulty in maintaining prescribed levels of chlorine residuals. The frequent attention of an operator is required to alter chlorine doses in maintaining constant chlorine residuals (Stephenson and Lauderbaugh, 1971).

To provide adequate disinfection of wastewater, the basic approach to good contact tank design should include a thorough investigation of hydraulic characteristics of various designs and then the selection of design features which will optimize hydraulic performance. Some important design considerations include optimization of mixing, contact time, and chlorine dose.

Evaluation of hydraulic characteristics. The hydraulic characteristics of a chlorine contact tank are generally determined by conducting tracer studies on flow patterns through the tank. Several possible tracers are available. Salt is a common tracer and has been used to determine detention times in contact tanks (Louie and Fohrman, 1968). However, it is often difficult to handle the large amounts of salt generally required for such studies. Radioactive tracers are another possibility; however, these are almost never used because of the hazard and regulations controlling their release.

Perhaps the most useful tracers are fluorescent dyes. Most of these are inexpensive and easy to obtain. Two of the dyes commonly used in contact tank tracer studies are Rhodamine WT (Hart et al., 1975) and Rhodamine B (Deaner; Kothandaram and Evans, 1974). Other fluorescent dyes are also available and the choice of which dye to use is a matter of personal judgment. The Rhodamine dyes, however, offer the advantages of being detectable at very

low concentrations and having low sorption tendencies. Also, turbidity has very little effect on the response of the dye. Fluorescence of the dyes at concentrations as low as 0.01 ppb can be detected with a fluorometer (Deaner, 1970).

In conducting tracer studies, the dye or other tracer should be injected into the contact tank at the same point the chlorine solution would enter the tank. If possible, the tracer should also be injected below the water surface to avoid scattering of the tracer by wind on the surface. The most desirable method of conducting tracer studies is to obtain a continuous record of the tracer concentrations at the tank outlet. If fluorescent dyes are used, this may be done by using a continuous flow fluorometer connected to a recorder. This type of approach is more reliable than the collection of grab samples.

The flow characteristics of the contact tank may be determined by evaluating the data obtained from tracer studies in one of several ways. The methods include conventional, statistical, and dynamic analyses (Sawyer, 1967). Conventional and statistical analyses are the most commonly used. The dynamic approach is basically a mathematical modeling technique and will not be discussed.

The conventional method of analysis consists of selecting specific points from the dispersion flow curve as indices to describe the performance characteristics of a tank. The points and indices commonly used are described as follows (Hart et al., 1975; Marske and Boyle, 1973).

T = V/Q (theoretical detention time)

t_i = time for tracer to initially appear at the tank outlet

t_p = time for tracer at outlet to reach peak concentration

t_{10}, t_{50}, t_{90} = time for 10, 50, and 90% of the tracer to pass at the outlet of the tank

t_g = time to reach the centroid of the effluent curve

t_i/T = index of short-circuiting

t_p/T = index of modal detention time

t_{50}/T = index of mean detention time

t_g/T = index of average detention time

t_{90}/t_{10} = Morrill Dispersion Index - indication of degree of mixing

In constructing dispersion flow curves, it is common practice to use dimensionless expressions for tracer concentrations and times. This is done to facilitate comparisons of hydraulic performance between tanks where different tracer concentrations and detention times are involved. The dimensionless dispersion flow curve is obtained by plotting C/C_0 against t/T where C is the tracer concentration at any time t, C_0 is the initial tracer concentration and T is the theoretical detention time (Q/V). A typical dispersion flow plot is presented in Figure 4.13.

The parameter which is probably the most useful in accurately describing hydraulic performance is the Morrill Index (MI) (Marske and Boyle, 1973). As the MI approaches 1.0, the flow through the tank approaches ideal plug flow. The larger the MI, the more closely the flow in the tank approaches backmix (complete mixed) reaction conditions. The two extreme flow conditions are displayed in Figure 4.14.

There are several different statistical approaches used to evaluate hydraulic performance. One approach, which has gained widespread acceptance, describes the flow regime of a basin in terms of plug flow and perfect mixing (Marske and Boyle, 1973; Wolf and Resnick, 1963). It also uses descriptive parameters to define effective space and dead space. A variation of this approach uses the entire tracer curve to describe hydraulic efficiency in

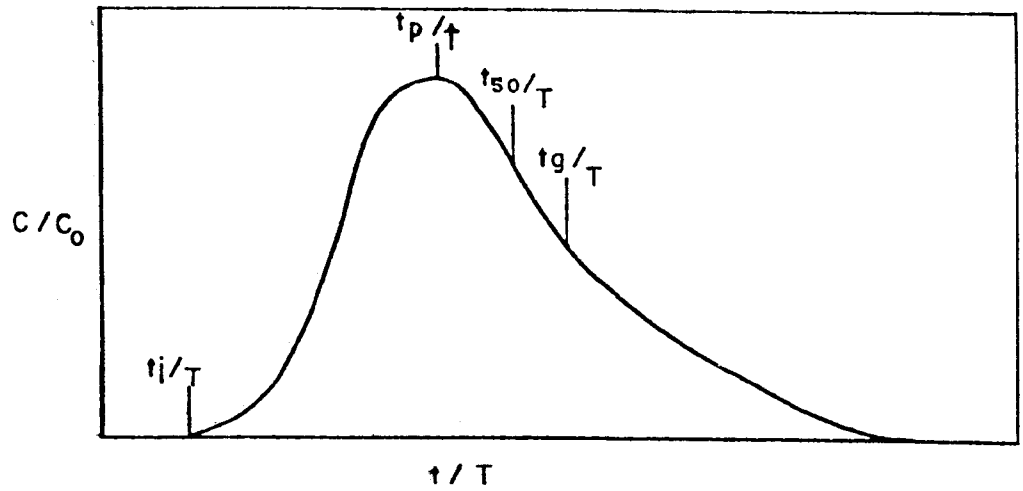


Figure 4.13. Typical dispersion flow curve.

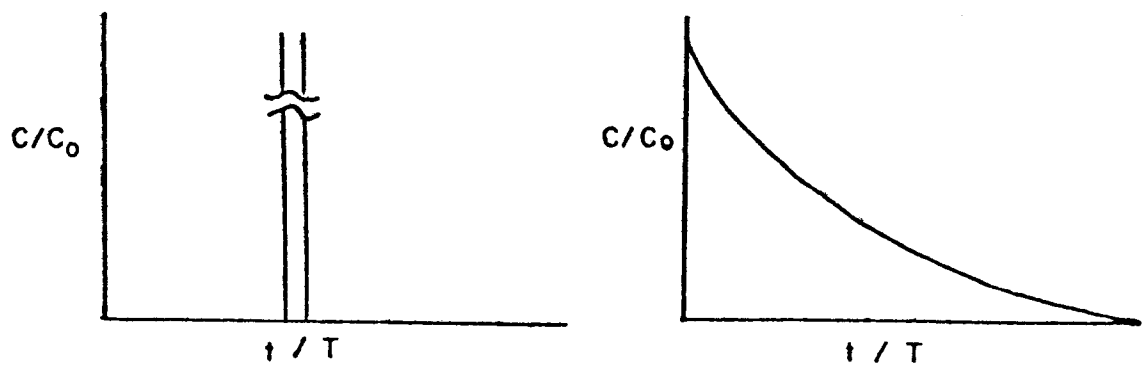


Figure 4.14. Comparison of plug and backmix flow.

terms of a function of time, F(t) (Rebhun and Argaman, 1965). This function is calculated from the following equation:

$$\text{Log}\,[1 - F(t)] = [-\,\text{Log}\,e/(1 - p)(1 - m)]\,[t/T - p(1 - m)]$$

m = dead space fraction

1-m = effective fraction

p = plug flow fraction

1-p = perfect mixing fraction

t = any time corresponding to time used to get F(t)

T = theoretical detention time

Probably the most widely used statistical approach is the chemical engineering dispersion index. It is considered to be reliable, since it is calculated using the entire dispersion flow curve. The dispersion index, δ, is calculated from the following equations (Marske and Boyle, 1973).

$$\delta = \sigma^2 = \frac{\delta t^2}{t_{\bar{a}}^2}$$

$$\sigma t^2 = \left(\frac{\Sigma t^2 c}{\Sigma c}\right) - \left(\frac{\Sigma tc}{\Sigma c}\right)^2$$

$$t_{\bar{a}} = \frac{\Sigma tc}{\Sigma c}$$

In these equations, c is the tracer concentration at any time t, $\delta = \sigma^2$ is equal to the variance of the flow-through curve.

The dispersion index has the strongest statistical probability of correctly describing the hydraulic performance because it includes all points on the dispersion flow curve. Conventional parameters only use one point, or at the best, only a portion of the curve. In comparing the dispersion index with conventional parameters, it has been found that the Morrill Index is closely correlated with the dispersion index and can be considered as the most reliable conventional parameter in accurately describing the hydraulic performance of

a tank. The least reliable indicators of flow characteristics are considered to be the percent of effective space, t_{50}/T, and t_i/T (Marske and Boyle, 1973).

Elements of contact tank design. The primary objective of good chlorine contact tank design is to design for hydraulic performance which will allow for a minimum usage of chlorine with a maximum exposure of microorganisms to the chlorine. An evaluation of a number of wastewater chlorine contact tanks indicates that mixing, detention time, and chlorine dosage are the critical factors to consider in providing adequate disinfection. Good design not only optimizes disinfection efficiency, but should also minimize the concentration of undesirable compounds being discharged to the environment and reduce the accumulation of solids in the tank by keeping the flow-through velocity high enough to prevent solids from settling (Hart et al., 1975).

Initial mixing of the chlorine solution with wastewater is necessary for providing uniform contact of chlorine with microorganisms and for preventing chlorine stratification in the contact tank. This is especially important because most of the disinfection takes place within the first few minutes of contact. It is also important to note that most of the chlorine demand is exerted during this same period. Since the formation of chloramines in wastewater is extremely rapid, it must be remembered that free chlorine is much more effective as a disinfectant than chloramines. Chloramines are ineffective in killing viruses in comparison with free chlorine. Although the reaction rates involved in the formation of chloramines are more rapid than the rate at which free chlorine reacts with microorganisms, it is important to provide as much exposure as possible of free chlorine to the microorganisms for efficient disinfection. Rapid mixing provides this exposure if there is any

free chlorine remaining in solution by the time the chlorine solution is mixed with the wastewater.

The rapid formation of chloramines indicates that there is a possible problem in the manner in which chlorine is put into solution. The common practice is to use a portion of the wastewater stream for the solution water. When this is done, most of the chlorine is in the form of chloramines before the solution line is ever mixed with the mainstream of wastewater. However, studies indicate that this practice does not appreciably affect the efficiency of the wastewater chlorination process (Collins et al., 1971).

Mixing can be accomplished by applying the chlorine solution to the wastewater either in a pressure conduit under turbulent conditions or with a mechanical mixer. A turbulent reactor is generally considered to be the most effective in producing maximum bacterial kill in the shortest contact time. It has been found that a contact time of 0.1 to 0.3 minutes is generally sufficient in a turbulent reactor. Slightly longer might be required when a mechanical mixer is used (Collins et al., 1971). If a mechanical mixer is used, the chlorine solution should be added to the wastewater immediately upstream from the mixer. Another form of mixing, which has been found to be effective, is the use of a hydraulic jump in combination with over and under baffles (Louis and Fohrman, 1968). Both the turbulent reactor and the baffle system of mixing offer the advantage of reducing operation and maintenance costs over those for the mechanical mixer.

Rapid mixing is followed by flow of the chlorinated wastewater into the contact tank. Most approaches to good contact tank design are based on the idea that plug flow is the most desirable hydraulic performance characteristic to achieve in producing efficient disinfection. Plug flow decreases short-circuiting, dead spaces, spiraling, and eddy currents and also closes the gap

between theoretical and actual detention times. However, not all designs are based on the objective of achieving plug flow. At least one design suggests the use of a series of backmix reactors to improve chlorination efficiency (Kokoropoulos, 1973). In this approach, the tank shapes are not important as long as stratification and short-circuiting are eliminated. One advantage to series reactors is the ease with which treatment capacity could be increased by simply adding another reactor. However, high initial and operational costs could offset this advantage.

For the design of tanks in which plug flow is the objective, tank shape is an important consideration. Ideally, plug flow conditions could best be achieved by using a long, narrow, straight contact chamber. A pipe, for example, would be a good contact chamber. However, because of cost and space limitations, this approach is generally not practicable. Circular contact tanks have been used, but generally they do not perform efficiently with respect to hydraulic characteristics (Warwick, 1968). Most tanks are based on a rectangular shape, which is the most practial design.

Conventional design practices can be enhanced by paying particular attention to inflow and outflow structures. They should be designed in such a fashion as to distribute wasteflow uniformly across the tank cross-section. One of the most effective designs is that of a sharp-crested weir covering the width of the contact tank at the inlet and outlet (Marske and Boyle, 1973). This design minimizes the weir overflow rate and greatly enhances hydraulic characteristics through the tank.

A common practice for improving plug flow conditions in a contact tank involves the use of baffles. Longitudinal baffles are generally more effective than cross baffles. In a study of seven different types of chlorine contact tank configurations, it was found that the longitudinally baffled

serpentine flow and flow resulting in an annular ring around a secondary clarifier were the best configurations for approaching ideal plug flow. Both have the effect of increasing the ratio of the length to the width (L/W) of the contact tank. The L/W ratio is often considered to be the most important design consideration for chlorine contact tanks. It has been recommended that a minimum L/W ratio of 40:1 be used in order to obtain maximum plug flow performance (Marske and Boyle, 1973); however, Johnson et al. (1978) obtained excellent hydraulic characteristics using a L/W ratio of 25:1. Baffles have also been used effectively across the width of contact tanks. Hydraulic performance has been improved by placing baffles near the inlet end of tanks to suppress the kinetic energy of incoming jets.

Often, baffles by themselves are not sufficient to produce desired hydraulic characteristics. Hammerhead shapes at baffle tips have been demonstrated to reduce short-circuiting and flow separation. Corner fillers have been used to eliminate dead spaces and to decrease the build-up of solids in corners. These fillers, however, seem to have little effect on flow characteristics. In some cases, directional vanes around the ends of baffles have been found to produce lower head losses and to produce more uniform flow through the contact tank (Louie and Fohrman, 1968).

Another approach to improving the effectiveness of chlorine contact tanks has involved aeration. It has been found that mild agitation with compressed air improves hydraulic characteristics and may improve bacterial kill by providing closer contact of microorganisms with residual chlorine (Kothandaraman and Evans, 1974). This method also reduces solids accumulation and thus decreases the chlorine demand caused by putrefaction of solids. Using this approach in a field evaluation, it has been found that adequate bacterial kill can be obtained in secondary sewage with a dose of 2 to 3

mg/l chlorine and a contact time of only 15 minutes. Fifteen minutes should be considered as the minimum actual hydraulic residence time for chlorine contact tanks. If the accumulation of solids is not adequately prevented by aeration, it is recommended that they be removed at least once a day by some mechanical or other means in order to keep chlorine demand as low as possible.

One final design consideration is that of depth. In shallow contact tanks, it is possible for wind to cause short-circuiting. However, this is generally not a problem in tanks designed with standard design depths.

For existing chlorine contact tanks, it is generally not possible to completely redesign the tank. However, improvements can be made in flow characteristics with practical alterations. Gates added to screen and sludge notches have been found to reduce short-circuiting. Spiraling flow patterns have been eliminated by circular baffle plates placed at tank inlets. Additional improvements can be made by using directional vanes to direct flow in a more uniform fashion and by using stop baffles with curved vanes to reduce eddying. In one example, the improvements reduced short-circuiting by 80 percent in an existing contact tank (Hart et al., 1975). Another way to improve hydraulic performance in existing tanks is to use pre-cast baffles. These can be installed with minimum down time. Although it is more efficient to use longitudinal baffles, it may be more economical to use cross baffles. It has been demonstrated that baffles installed in a maze configuration improved performance sufficiently to make economical factors more important in choosing a design than efficiency considerations (Stephenson and Lauderbaugh, 1971).

In conclusion, the most important design considerations for efficient disinfection appear to be rapid and complete initial mixing, an adequate L/W ratio to produce near plug flow conditions, and a sufficienty long residence

time to produce an optimal amount of disinfection for the chlorine dose applied with a minimum amount of chlorine residual remaining in the effluent. Design considerations are summarized in Table 4.1.

4.3.7.8 LITERATURE CITED

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Table 4.1. Summary of chlorination design criteria.

Mixing

I. Rapid initial mixing should be accomplished within 5 seconds and before liquid enters contact tank. Design hydraulic residence time $\geq$ 30 seconds for tanks using mechanical mixers.

II. Methods available
   1. Hydraulic jump in open channels.
   2. Mechanical mixers located immediately below point of chlorine application.
   3. Turbulent flow in a restricted reactor.
   4. Pipe flowing full. Least efficient and should not be used in pipes with diameter > 30 inches.

Contact Chamber

I. Hydraulic residence time
   1. $\lesseqgtr$ 60 minutes at average flow rate.
   2. $\lesseqgtr$ 30 minutes at peak hourly flow rate.

II. Hydraulic performance
   1. Modal value obtained in dye studies $\lesseqgtr$ 0.6, $t_p/T \lesseqgtr 0.6$ (Figure 4.13).
   2. Efficiency of disinfection increases as t_p/T increases.
   3. Design for maximum economical t_p/T.

III. Length to width ratio
   1. L/W $\lesseqgtr$ 25:1.
   2. Cross-baffles used to eliminate short circuiting caused by wind.

IV. Solids removal
   1. Baffles arranged to remove floating solids.
   2. Provide drain to remove solids and liquid for maintenance.
   3. Provide duplicate contact chambers.
   4. Width between channels should be adequate for easy access to clean and maintain chamber.

V. Storage
   1. Provide a minimum of one filled chlorine cylinder for each one in service.
   2. Maintain storage area at a temperature $\lesseqgtr$ 55°F.
   3. Never locate cylinders in direct sunlight or apply direct heat.
   4. Limit maximum withdrawal rate from 100 and 150 pound cylinders to 40 pounds per day.
   5. Limit maximum withdrawal rate from 2,000 pound cylinders to 400 pounds per day.
   6. Provide scales to weight cylinders.
   7. Provide cylinder handling equipment.
   8. Install automatic switch-over system.

Table 4.1. Continued.

VI. Piping and valves
1. Use Chlorine Institute approved piping and valves.
2. Supply piping between cylinder and chlorinator should be Sc. 80 black seamless steel pipe with 2000 pound forged steel fitting. Unions should be ammonia type with lead gaskets.
3. Chlorine solution lines should be Sc. 80 PVC, rubber-lined steel, saran-lined steel, or fiber cast pipe approved for moist chlorine use. Valves should be PVC, rubber-lined, or PVC lined.
4. Injector line between chlorinator and injector should be Sc. 80 PVC or fiber cast approved for moist chlorine use.

VII. Chlorinators
1. Should be sized to provide dosage $\leq$ 10 mg/l.
2. Maximum feed rate should be determined from knowledge of local conditions.
3. Direct feed gas chlorinators should be used only in small installations. Check state regulations. Prohibited in certain states.
4. Vacuum feed gas chlorinators are most widely used and are much safer.
5. Hypochlorite solutions should be considered in small installations where safety is major concern.

VIII. Safety equipment and training
1. Install an exhaust fan near floor level with switch actuated when door is opened.
2. Exhaust fan should be capable of one air exchange per minute.
3. Gas mask located outside chlorination room.
4. Emergency chlorine container repair kits.
5. Chlorine leak detector.
6. Alarms should be installed to alert operator when deficiencies or hazardous conditions exist.
7. Operator should receive detailed hands-on training with all emergency equipment.

IX. Diffusers
1. Minimum velocity through diffuser holes $\leq$ 10-12 ft per sec.
2. Diffusers should be installed for convenient removal, cleaning and replacement.

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