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CONTENTS

PAPERS

Flow Approaching Filter Washwater Troughs <i>Jonathan A. French</i>	139
Ozone Disinfection of Secondary Effluents <i>Carl Nebel, Ronald D. Gottschling, Paul C. Unangst H.J. O'Neill and George V. Zintel</i>	161

MEMORIAL

Ernest Lincoln Spencer	188
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PROCEEDINGS

Meetings Held	190
---------------------	-----

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FLOW APPROACHING FILTER WASHWATER TROUGHS*

By
Jonathan A. French**

Abstract

A potential flow model is used to study the flow of backwash water, laden with filtered solids, in its progress from the top of the filter bed to the washwater troughs above the filter. For backwashing effectiveness and economy, it is of interest to minimize the region of nonuniform flow between troughs. The best practical dimensions for promoting uniform flow beneath and between the troughs are determined. For this optimal trough design and two alternative designs, the paths and travel times of fluid elements and suspended particles are studied. Factors affecting trough spacing are also discussed.

Introduction

Granular-bed filtration is a popular process for preparing water for human consumption and for treating wastes, particularly industrial wastes, before discharge. Usually, the water is allowed to flow downward through a bed of sand, granulated coal, or a combination of these or other media, until the bed is sufficiently clogged with removed solids. Filtration is then temporarily halted while the filter is cleaned by backwashing, in which a relatively brief but intense upward flow of clean water scrubs the solids from the media grains and flushes them out of the filter. Backwash of sufficiently great intensity will cause the bed to fluidize, wherein the overall bed volume expands so that the filter grains do not rest packed against each other, but are held individually suspended in the upflow. Backwash water bearing the removed solids is removed from the filter unit either by gutters in the sidewall (Fig. 1a) or by a set of troughs spanning the filter (Fig. 1b). Troughs are common in North America only (1); elsewhere the side weir design finds favor.

Such filters have operated with great success in many parts of the world

*Paper submitted June 27, 1975 for consideration for the John R. Freeman Hydraulics Prize, and awarded Honorary Mention citation on January 21, 1976.

**Senior Engineer, Camp Dresser & McKee, Inc., Boston, Mass.

for more than a century. Design has evolved slowly, guided by a few rules of thumb, except for a few early thinkers such as Hazen, and for a recent surge in experimental and theoretical research. However, design changes since the turn of the century cannot be called profound; the 1909 Cleveland, Ohio, water filters as described by Ellms and Gettrust (2) bear a jolting resemblance to today's designs, the differences being principally in a few structural materials.

Most of the recent research has been on the filtration process itself (3), with some attention given to backwashing (4). Yet there is also a need for more reasoned study of some of the concomitant problems, such as the means by which the underdrains support the bed, collect the filtrate, and distribute the backwash; and the means by which the backwash water emerging from the top of the filter bed is collected and removed.

This paper will treat the upflow of backwash water from the bed to the troughs or sidewall weirs. After a brief discussion of fluidization and introduction of the potential flow model, the importance of uniformity in the upflow will be discussed. The potential flow model will be used to show (a) which sizes and shapes of troughs provide the most uniform flow; and (b) how suspended floc particles behave in a nonuniform upflow.

Fluidization

Fluidization is employed not only in water treatment but also in widespread applications in industry as a contact process. The phenomenon is described in general terms by Kunii and Levenspiel (5):

"Pass a fluid upward through a bed of fine particles At a low flow rate, fluid merely percolates through the void spaces between stationary particles. This is a *fixed bed*.

"With an increase in flow rate, particles move apart and a few are seen to vibrate and move about in restricted regions

"At a still higher velocity, a point is reached when the particles are all just suspended in the upward flowing gas or liquid. At this point, the frictional force between a particle and fluid counterbalances the weight of the particle, the vertical component of the compressive force between adjacent particles disappears, and the pressure drop through any section about equals the weight [per unit area] of the particles in that section. The bed is considered to be just fluidized and is referred to as an *incipiently fluidized bed* or a bed at *minimum fluidization*.

"In liquid-solid systems [such as in a water filter] an increase in flow rate above minimum fluidization usually results in a smooth, progressive expansion of the bed. Gross flow instabilities are damped and remain small, and large-scale bubbling or heterogeneity is not observed under normal condition. A bed such as this [may be] called a *smoothly fluidized bed*

"Gas-solid systems generally behave in quite a different manner. With an increase in flow rate beyond minimum fluidization, large instabilities with bubbling and channeling of gas are observed. At higher flow rates agitation becomes more violent and the movement of solids becomes more vigorous. In addition, the bed does not expand much beyond its volume at minimum fluidization

"Both gas and liquid fluidized beds are considered to be *dense phase fluidized beds* as long as there is a fairly clearly defined upper limit or surface to the bed. However, at a

sufficiently high fluid flow rate the terminal velocity of the solids is exceeded, entrainment becomes appreciable, and [filter media] solids are carried out of the bed with the fluid stream. In this state we have a *disperse-, dilute- or lean-phase fluidized bed . . .*"

In this paper we shall be concerned largely with a *smoothly fluidized* bed of solids in water, with a fairly clearly defined upper surface to the bed.

The Potential Flow Model

It is assumed that the reader is familiar with the technique of modelling ideal, two-dimensional flows with potential flow theory,* describing flow boundaries by conformal mapping. In this case, the usual assumptions will be made: that viscous effects and molecular and turbulent diffusion effects are negligible, and that the flow is incompressible and irrotational. Furthermore, it is assumed that neither the bed of fluidized filter particles nor the suspended floc particles being washed from the filter affects the flow pattern appreciably.

Consider a filter structure of infinite depth and without filter media. A section of such a structure, taken in a plane normal to the weirs, is shown in Fig. 1a for sidewall weirs and in Fig. 1b for troughs. In each figure, the total flow pattern is seen to be merely the pattern within a single vertical strip of width s , reflected and/or replicated one or more times.

Let us examine first the pattern in Fig. 1a, bounded by the x and y axes (note the unconventional downward orientation of the y axis) and by the vertical line $x = s$. If the top water surface is assumed plane despite the slight drawdown due to flow over the weir, and if the water depth over the weir is neglected, the flow in the region ($0 \leq x \leq s$, $y \geq 0$) may be modelled by a sink of strength $v_r s$ at the coordinate origin. The potential streamline function for this is

$$\psi = \frac{2s v_r}{\pi} \tan^{-1} \left[\frac{\tan \frac{\pi}{2} \frac{x}{s}}{\tanh \frac{\pi}{2} \frac{y}{s}} \right] \quad (1)$$

where
$$-\frac{\partial \psi}{\partial x} = \frac{dy}{dt} = -v \quad (2)$$

and
$$\frac{\partial \psi}{\partial y} = \frac{dx}{dt} = u \quad (3)$$

and v_r is the upward velocity far down at great y , where the velocity may be expected to be uniform.

*Potential flow theory is discussed in many texts on hydrodynamics, applied mechanics, and hydraulic engineering. The treatment that is perhaps most appropriate to this paper is that of Streeter (6,7), who not only provides a rather complete theoretical exposition, but includes among his examples a flow configuration essentially equal to that used herein; compare Fig. 2 of this paper with one quadrant of either Fig. 4.24 in Reference (6) or Fig. 56 in Reference (7) — after rotating the Streeter figure through 90 degrees. A derivation of this paper's Equation 1 accompanies Fig. 56 of Reference (7).

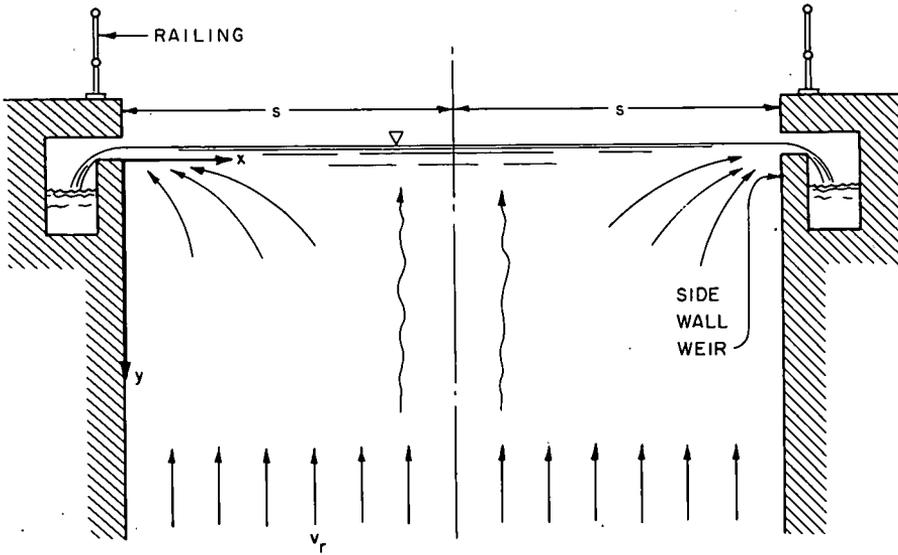


Figure 1a. Upflow to side wall weirs.

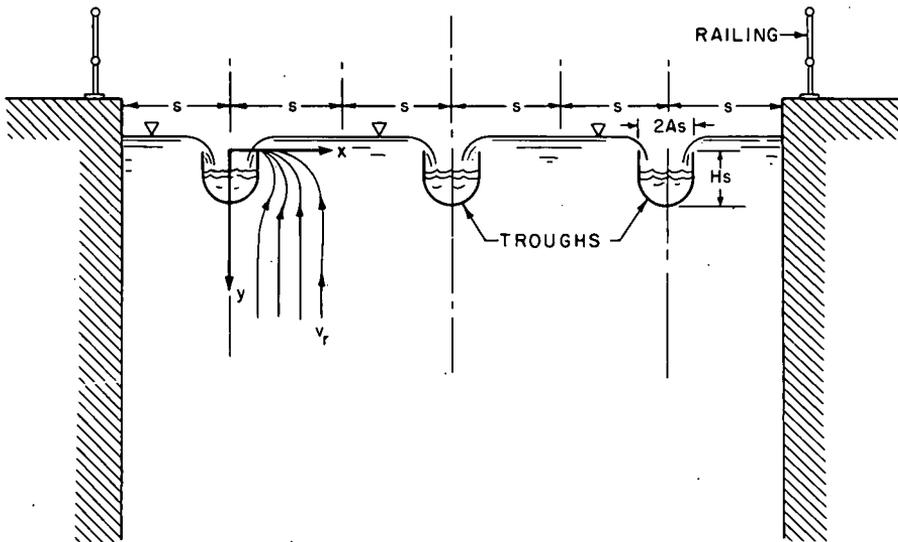


Figure 1b. Upflow to troughs.

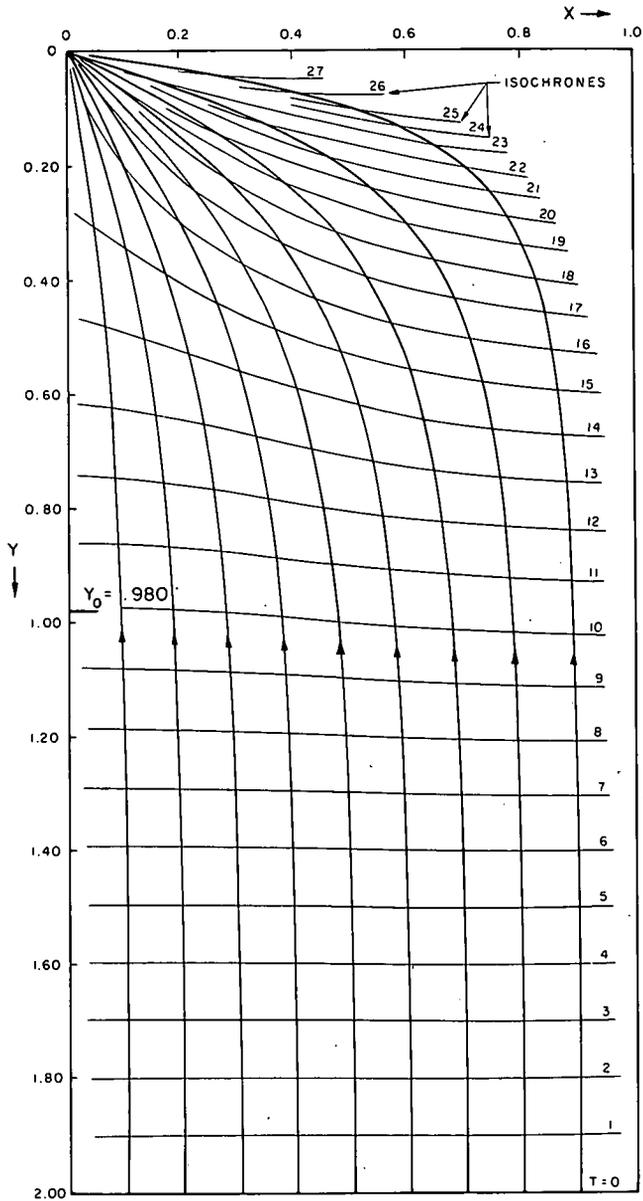


Figure 2. Pattern of flow approaching side wall weir (A=0, B=0).

It is convenient to introduce now the normalized variables $X = x/s$, $Y = y/s$, $T = v_r t/s$, $\Psi = \psi / v_r s$, and $V = v/v_r$, so that

$$\Psi = \frac{2}{\pi} \tan^{-1} \left[\frac{\tan \frac{\pi}{2} X}{\tanh \frac{\pi}{2} Y} \right] \quad (4)$$

$$-\frac{\partial \Psi}{\partial X} = \frac{dY}{dT} = - \frac{(\tanh \frac{\pi}{2} Y) (\sec \frac{\pi}{2} X)^2}{(\tanh \frac{\pi}{2} Y)^2 + (\tan \frac{\pi}{2} X)^2} \quad (5)$$

$$\frac{\partial \Psi}{\partial Y} = \frac{dX}{dT} = - \frac{(\tan \frac{\pi}{2} X) (\operatorname{sech} \frac{\pi}{2} Y)^2}{(\tanh \frac{\pi}{2} Y)^2 + (\tan \frac{\pi}{2} X)^2} \quad (6)$$

The normalized source strength is now unity, as is the upflow velocity at great Y . The flow pattern in terms of the normalized variables is shown in Fig. 2. Note the stagnation region near $(X = 1, Y = 0)$.

The presence of round-bottom troughs is modelled by not one but two sinks of unit strength, located at $(X \pm A, Y = 0)$, and two sources of half strength, located at $(X = 0, Y = \pm B)$. The equations corresponding to Equations 4, 5, and 6 for the stream function and the two velocity components are:

$$\Psi = \frac{2}{\pi} \tan^{-1} \left[\frac{\tan \frac{\pi}{2} (X + A)}{\tanh \frac{\pi}{2} Y} \right] + \frac{2}{\pi} \tan^{-1} \left[\frac{\tan \frac{\pi}{2} (X - A)}{\tanh \frac{\pi}{2} Y} \right] \\ - \frac{1}{\pi} \tan^{-1} \left[\frac{\tan \frac{\pi}{2} X}{\tanh \frac{\pi}{2} (Y + B)} \right] - \frac{1}{\pi} \tan^{-1} \left[\frac{\tan \frac{\pi}{2} X}{\tanh \frac{\pi}{2} (Y - B)} \right] \quad (7)$$

$$\frac{dY}{dT} = - \frac{\tanh \frac{\pi}{2} Y \sec^2 \frac{\pi}{2} (X + A)}{\tanh^2 \frac{\pi}{2} Y + \tan^2 \frac{\pi}{2} (X + A)} - \frac{\tanh \frac{\pi}{2} Y \sec^2 \frac{\pi}{2} (X - A)}{\tanh^2 \frac{\pi}{2} Y + \tan^2 \frac{\pi}{2} (X - A)} + \frac{1}{2} \cdot \frac{\tanh \frac{\pi}{2} (Y + B) \sec^2 \frac{\pi}{2} X}{\tanh^2 \frac{\pi}{2} (Y + B) + \tan^2 \frac{\pi}{2} X} + \frac{1}{2} \cdot \frac{\tanh \frac{\pi}{2} (Y - B) \sec^2 \frac{\pi}{2} X}{\tanh^2 \frac{\pi}{2} (Y - B) + \tan^2 \frac{\pi}{2} X} \quad (8)$$

$$\frac{dX}{dT} = - \frac{\tan \frac{\pi}{2} (X + A) \operatorname{sech}^2 \frac{\pi}{2} Y}{\tanh^2 \frac{\pi}{2} Y + \tan^2 \frac{\pi}{2} (X + A)} - \frac{\tan \frac{\pi}{2} (X - A) \operatorname{sech}^2 \frac{\pi}{2} Y}{\tanh^2 \frac{\pi}{2} Y + \tan^2 \frac{\pi}{2} (X - A)} + \frac{1}{2} \frac{\tan \frac{\pi}{2} X \operatorname{sech}^2 \frac{\pi}{2} (Y + B)}{\tanh^2 \frac{\pi}{2} (Y + B) + \tan^2 \frac{\pi}{2} X} + \frac{1}{2} \frac{\tan \frac{\pi}{2} X \operatorname{sech}^2 \frac{\pi}{2} (Y - B)}{\tanh^2 \frac{\pi}{2} (Y - B) + \tan^2 \frac{\pi}{2} X} \quad (9)$$

The flow pattern among these sources and sinks, plotted for one quadrant in Fig. 3, consists of one zone inside a roughly elliptical shape (half of which is shown) centered on the coordinate origin, and another zone outside. The inside flow consists of flow from the two sources. The streamline separating the flows when $Y > 0$ represents the exterior cross-sectional shape of a round-bottom trough of normalized width $2A$ and normalized depth H ; where H is the positive value of Y , for $X = 0$, at which the right-hand side of Equation 8 equals zero. Note the stagnation region near $(X = 0, Y = H)$, as well as at $(X = 1, Y = 0)$. This profile may be considered a reasonable approximation of a typical trough shape; further refinement of the round-bottom shape, or the modelling of other shapes, could be attained with a more complex array of sources and sinks, and with other conformal mappings.

The flow nets shown in Figs. 2, 3, and 6 were plotted largely by computer. The program consisted of Equations 8 and 9 for the two velocity components, values of A and B , initial positions for each streamline, instructions to increment X and Y according to the local velocity components, and instructions to plot X and Y at all times and to tick the resulting plotted streamlines at values of $T = .1, .2, .3 \dots$. The initial positions are $(X = \text{small positive value}, Y = H)$ for the trough boundary streamline, and $(X = 0.1, 0.2, \dots 0.9, Y = 2.0)$ for the other streamlines; $Y = 2.0$ is chosen as a starting point because it is found to be in the zone of essentially uniform flow.

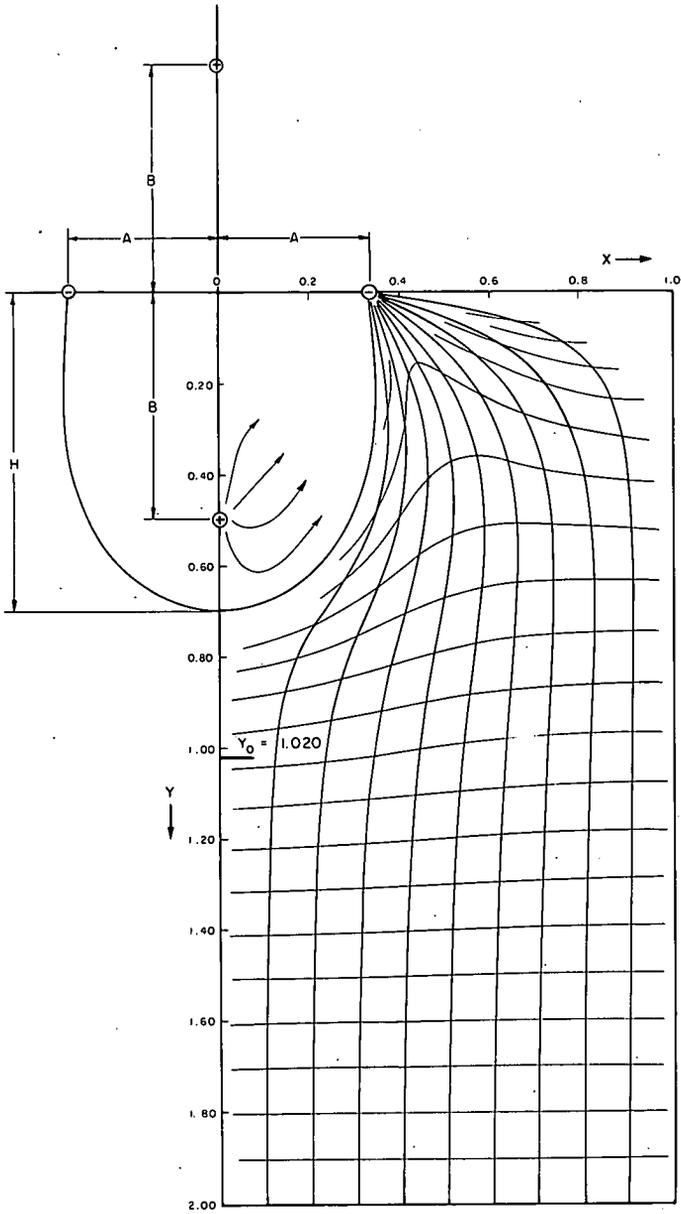


Figure 3. Sources and sinks for trough model ($A=.33, B=.50$).

Isochrones

In Figs. 2, 3, 6, and 7, curves connecting ticks representing equal elapsed time were faired in by hand to obtain what may be called isochrones or isochronal curves. In Fig. 2, consider the fluid elements along the lowest, horizontal isochronal curve ($T = 0$), perhaps tagged with a streak of dye. At a time 0. In later, the dye streak will be congruent with the n th isochronal curve above the first.

Obviously, isochrones are not to be confused with the equipotential curves conventionally found in a flow net. In a conventional net, the spacing of the streamlines equals the spacing of the equipotential curves at any point, and both are inversely proportional to the local speed. In this flow net, however, the spacing between isochronal curves is directly proportional to local speed, a fact which aids in the visualization of velocity profiles.

Degree of Uniformity in the Upflow

A uniform flow has everywhere the same speed and direction. It has been noted that for sufficiently large Y , the upflow towards a trough is uniform, but that near the trough the flow is unavoidably not uniform.

Hirsch (8) has stressed the importance of flow uniformity in the fluidized bed during backwash:

"Operating difficulties which in many cases stem from insufficient or poorly distributed backwash include: mounding of gravel, jetting of sand, formation of mud balls, early turbidity breakthrough, high washwater consumption, and short filter runs. So important is an evenly spread backwash that equipment manufacturers offer this feature as their most appealing sales argument

". . . Filter plant operators are aware of the unevenness of a backwash, their usual observations being the early appearance of clear areas and the persistence of muddy spots in corners

". . . Similarly, toward the end of a wash a band of clear water extends the length of the trough lip before clearing is discernible in the intertrough areas

". . . Even when uniformity of washwater introduction from the underdrains is assured, the troughs themselves, by their discrete linear positions, are the cause of the nonuniformity."

Clearly, the troughs themselves by their discrete linear positions can indeed be the cause of the nonuniformity if the bed is allowed to expand into the nonuniform region near the trough. Furthermore, if the bed expands into the nonuniform region near the trough, the nonuniformity may be exacerbated, rather than damped. The student of flow in porous media can see that the presence of a fixed bed in Figs. 2 or 3, with a plane, horizontal top surface at, say, $Y = .8$, would tend to make more uniform the flow within the bed at $Y > .8$, because flow emerging from a porous medium tends to do so in a direction normal to the exit surface. However, if the bed is not fixed but is fluidized, the bed will expand more in high velocity zones than in low

velocity zones, thus causing the top surface to mound up where the isochronal curves are seen to mound up.

The filter grains on the high-velocity mound will tend to sidle downslope, i.e., to restore the horizontal surface of the top of the bed. Such removal of grains from the high-velocity area will reduce the resistance to upflow there, inviting a greater upflow there than elsewhere, thus amplifying the nonuniformity caused by the rigid boundaries of the flow region. This argument, while qualitative, is presented here merely to underscore the importance of establishing a sufficiently uniform flow within the bed.

In this paper, nonuniformity is conveniently defined as V_{\max}/V_{\min} , a function of Y and the trough dimensions, where V_{\max} is the greatest normalized upward velocity at a given value of Y , and V_{\min} is the smallest velocity at that Y . It is, of course, desirable to have V_{\max}/V_{\min} as close to unity as possible; in this paper an arbitrary value of $V_{\max}/V_{\min} = 1.2$ is chosen as an acceptable tolerance. (The choice of tolerance deserves more study.)

In Fig. 2, for the sidewall weir (or for a very small trough), notice that for Y less than about 1.0, V_{\max} is found at $X = 0$ and V_{\min} is found at $X = 1$, with nonuniformity increasing as Y decreases, reaching the value 1.2 at $Y_0 = .980$. The isochrones tilt upward to the left.

In Fig. 3, for a trough relatively large with respect to its spacing, the isochrones tilt upward not to the left but to the right; and V_{\min} is now found at $X = 0$. The V_{\max} is found at $X < 1$, at the "hump in the middle" of the isochrones. The nonuniformity of 1.2 is attained at $Y_0 = 1.020$; the pattern in Fig. 3 may, therefore, be considered slightly "worse" than that in Fig. 2, because a greater depth, Y_0 , below the weirs is required to establish acceptable uniformity.

Contemplation of Figs. 2 and 3, with their rather opposite patterns of nonuniformity for very small and rather large troughs, respectively, suggests, a la Goldilocks, that a trough of intermediate size would engender a flow pattern with isochrones tilting neither to the left nor to the right, and with $V_{\max}/V_{\min} = 1.2$ occurring at a Y_0 that is less than in either of the first two patterns. A systematic search for an optimal trough size and shape (i.e., with a minimal Y_0) was thus undertaken.

In this search, B/A ranged from 0.6 to 1.4. For each B/A , the value of A was varied between .18 and .33. For each value of A and B studied, the vertical velocity, using Equation 8, was computed for $X = 0, .1, .2, \dots, 1.0$, and for $Y = .6, .7, \dots, 1.2$. (The range of Y was chosen and amended as necessary to include Y_0 , when found.) The value of H was also computed. For each Y , the values of V were scanned, V_{\max} and V_{\min} noted, and their ratio computed.

For each B/A , the nonuniformity V_{\max}/V_{\min} was plotted as a function of H for the several values of Y , as shown by an example in Fig. 4. For each Y , there is clearly a value of H for which nonuniformity is least, for a given B/A . The curve for each Y has two limbs; conditions on the left limb

represent conditions shown in Fig. 2, or for small troughs, where V_{\min} is found at $X = 1$. The right limb represents conditions shown in Fig. 3, where V_{\min} is found at $X = 0$. Where the limbs cross, V_{\min} is both at $X = 0$ and $X = 1$, and nonuniformity is minimized. Note that on Fig. 4 the value of H for least V_{\max}/V_{\min} changes little with Y ; see also that $V_{\max}/V_{\min} = 1.2$ can be attained for Y less than 0.7, compared with Y_0 values of .980 and 1.020 for Figs. 2 and 3.

For B/A other than 1.4, plots were made that were qualitatively similar to Fig. 4. Interest centered on conditions at point P, where for a given B/A the optimum H provides the lowest V_{\max}/V_{\min} , at $Y = 0.7$.

To specify B/A and H also is to specify A ; conditions at P are plotted in Fig. 5 as a curve relating A to H , with values of V_{\max}/V_{\min} noted at points along the curve (the value $V_{\max}/V_{\min} = 1.168$ in Fig. 5 is the one for $B/A = 1.4$, shown in Fig. 4). The A-curve in Fig. 5 indicates, for any given H , the value of A that will give the least nonuniformity in the upflow at $Y = 0.7$.

While for a given H we now can find the A for best uniformity, we still have not specified an optimum H . There is a problem here; note that V_{\max}/V_{\min} varies along the curve, becoming least at $H = 0$, $A = 0.5$. While one would normally search for the point on the curve with the least V_{\max}/V_{\min} , it is obviously not practical to have a trough of zero depth H ;^{*} what then shall we call a practical optimum?

The trough's internal capacity is roughly proportional to $Ah^{1.5}$, a parameter which has also been plotted on Fig. 5. There is a maximum in this parameter at $H = .41$, $A = .25$; the corresponding value of B is .25. These values, therefore, define what may be called a practical optimum.

The flow pattern for $A = B = .25$, $H = .41$ is shown on Fig. 6. In this case, the depth to $V_{\max}/V_{\min} = 1.2$ is $Y_0 = .675$, considerably less than for the cases shown in Figs. 2 and 3. Depths to other values of V_{\max}/V_{\min} are shown on the right-hand margin.

In more real-world terms, the basic dimensions of this practical - optimum trough are: total external width ($2As$) equal to .25 times the center-to-center trough spacing ($2s$) and external drafts (Hs) equal to $.41/2 = .20$ times the center-to-center spacing ($2s$). The filter may be designed so that when the bed is fluidized during backwash, the top of the bed comes as high as $Y_0 = .675$ (or $y_0 = .34$ times the center-to-center spacing, $2s$), or another value if a tolerance of nonuniformity other than 1.2 is adopted.

^{*}It may be of interest to note that the pattern for $A = .5$, $H = 0$ is the same as for $A = 0$, $H = 0$, but to one-half the scale.

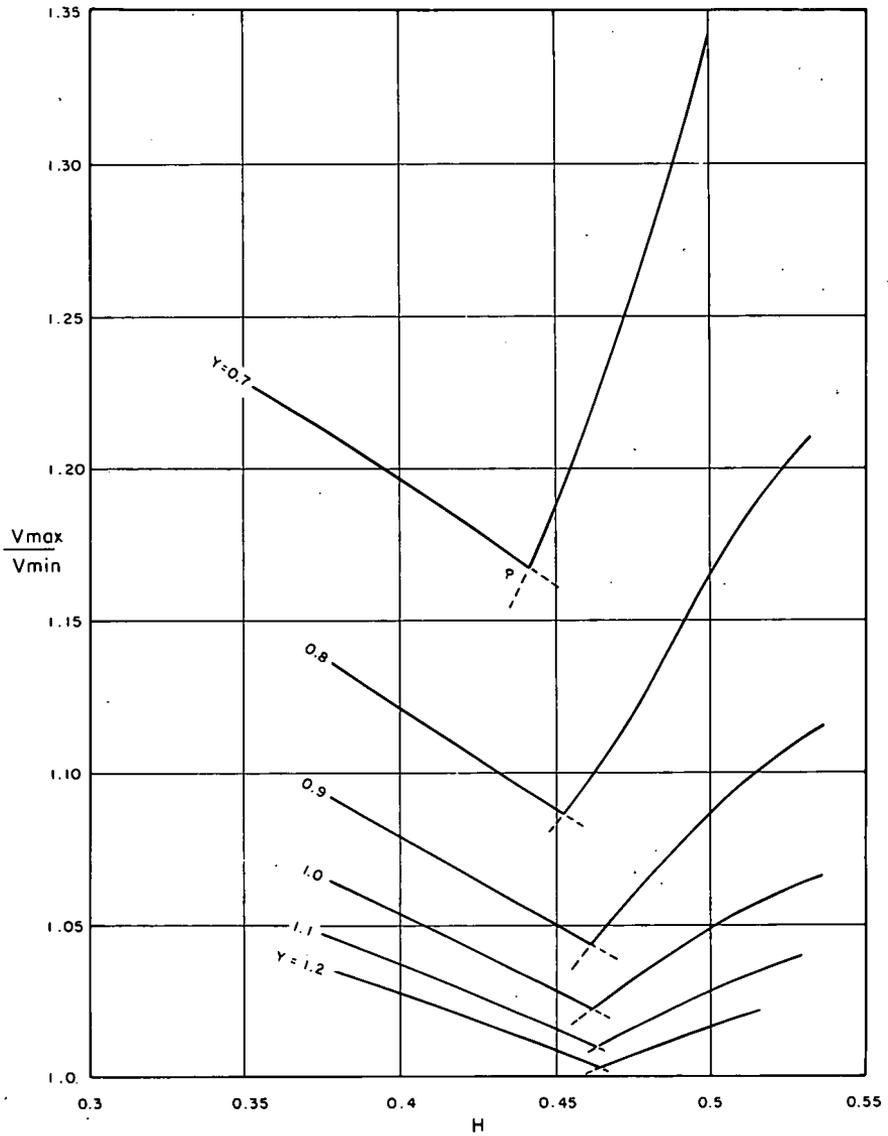


Figure 4. Uniformity of upflow as a function of trough size, H, and water depth, Y, for B/A = 1.4.

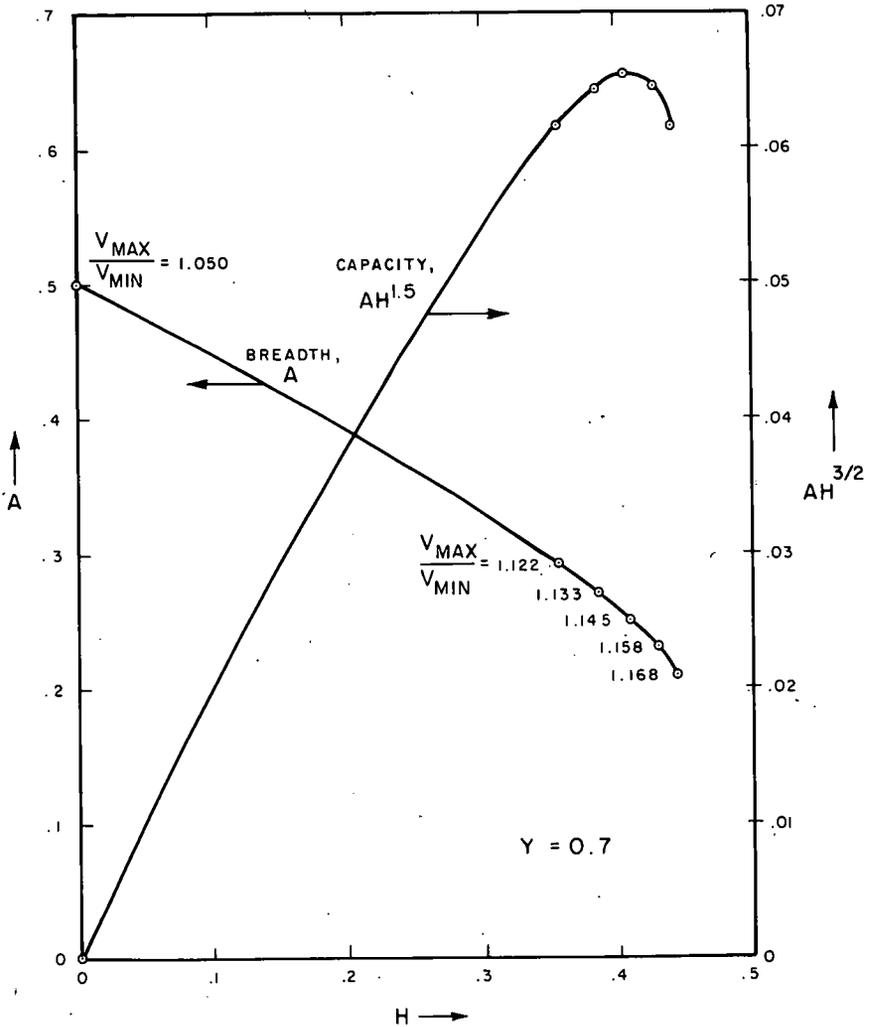


Figure 5. Breadth and capacity functions of H for optimum uniformity.

Suspended Solids Flushing Histograms

Not only is it important to keep the expanded bed below the region of nonuniform flow; it is also important to have a trough design that engenders a flow pattern that efficiently removes the filtered floc particles from the filter. Baylis et al (9) state that except for problems caused by uneven distribution of washwater and improper grading of filter materials, nearly all

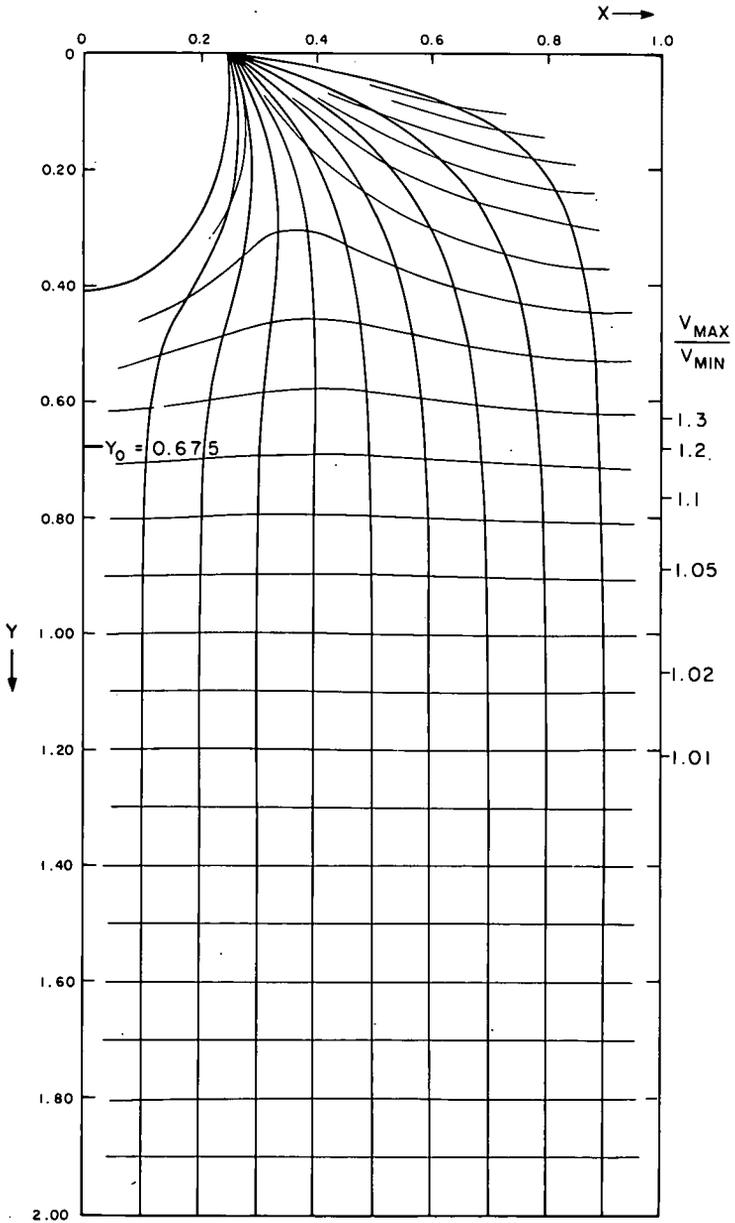


Figure 6. "Optimal" flow pattern (A=B=.25, H=.41).

filter bed trouble is attributable to failure of the washing process to remove from the filter the material which has been filtered from the water. Part of the washing process is to dislodge floc from the filter grains and bring it to the surface of the bed. A second and important part is to lift the floc clear of the bed and up to the trough weirs, not difficult for floc that has a small settling velocity, but increasingly costly and time consuming as the settling velocity of the floc increases. Since it has been estimated that 92 percent of the cost of backwashing is merely the cost of the pumped water used (10), it would appear worthwhile to seek a design that would minimize the time, hence the volume, of water used per backwash.

At several filter plants, Hirsch (8) performed backwash tests in which suspended solids concentration in the washwater was measured at frequent time intervals during the wash. Composite water samples were taken at the point of trough discharge to the sewer; when possible, samples were also taken at troughside bottom (in terms of our coordinates, at about $X = A$, $Y = H$) and midway between the troughs ($X = 1$).

Curves of suspended solids concentration plotted against time typically showed an early peak followed by a gradually decreasing tail. Generally, the troughside samples showed higher peaks and shorter tails than did the composite samples, or the samples from midway between troughs. In such plots, one may define a "good wash" as that for which the composite sample shows an early, high peak and a short tail.

In the present study, the paths of floc particles are computed as follows: Consider a particle of unhindered settling velocity, W_{v_r} , or in normalized terms, W . Assume that the particle settles at this velocity with respect to the surrounding fluid, so that with respect to fixed coordinates its velocity is the vectorial sum of W and the local fluid velocity:

$$\frac{dY_p}{dT} = \frac{dY}{dT} + W \quad (10)$$

$$\frac{dX_p}{dT} = \frac{dX}{dT} \quad (11)$$

where the subscript (p) denotes the particle, and dY/dT and dX/dT are as given by Equations 8 and 9. The path of the particle may be computed and traced, with time recorded, in the same manner as for streamlines. Since the depth, Y_0 , to a sufficiently small nonuniformity v_{\max}/v_{\min} can be determined, computations are begun at $Y = Y_0$ instead of the more arbitrary $Y = 2$, it being assumed that the top of the expanded bed is at $Y = Y_0$.

Consider first the "practical optimum" trough configuration, $A = B = .25$, $H = .41$. For $W = 0$, the particle paths are identical to the streamlines in Fig. 6, for $Y \leq Y_0$. For $W = 0$, the paths may be quite different, as shown for $W = 0.9$ in Fig. 7. The values T_f are the normalized times required to flush the particles from $Y = Y_0$ to the weir at $X = A$, $Y = 0$. Comparison of the

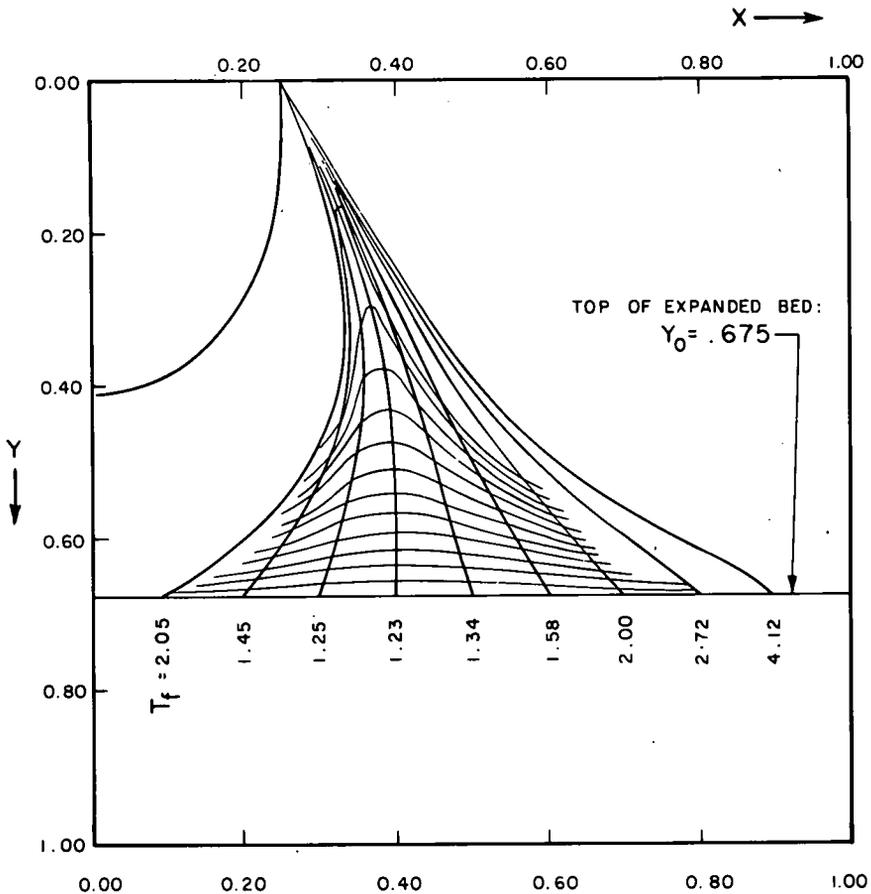


Figure 7. Particle paths for $W = 0.9$, with $A=B=.25$, $H=.41$.

particle paths in Fig. 7 with the streamlines in Fig. 6 shows that a heavy particle lifted towards a stagnant region makes little progress vertically until it can drift laterally, crossing streamlines, sufficiently far into a high velocity zone to be carried up and out.

The flushing times T_f were computed not only for $W = 0.9$, as shown in Fig. 7, but also for $W = 0, .1, .2, \dots, .8$, for floc particles initially located at $Y = Y_0$, $X = .1, .3, .5, .7, .9$. Values of T_f are presented in Table 1. As one would expect, the lowest T_f at any W occurs at about $X = .3$, where the stream velocity is greatest. As W increases, T_f also increases for any given X , but T_f for $X = .3$ for a given W is usually less than T_f at $X = .1$ for the next lesser W . Thus, a heavy particle initially in a high velocity zone may be flushed out

more quickly than a lighter particle initially in a low velocity zone.

The data in Table I are plotted as a frequency histogram in Fig. 8a. This histogram may be considered comparable to Hirsch's plots of suspended solids concentration vs. time reported earlier, provided (a) we can assume that all floc particles begin the flush-out process at the same time from $Y = Y_0$ (whereas, in fact, some may still be deep within the filter when the first particles begin to rise from $Y = Y_0$); (b) the distribution of floc settling velocities is uniformly graded from $W = 0$ to $W = .9$. The computation process may be quite easily amended to represent more realistic conditions than these two provisions, should one wish; but the procedure as given is quite adequate for the expository purpose of the paper.

TABLE I
FLUSHING TIMES T_f FOR $A = B = .25$, $H = .41$, $Y_0 = .675$

$W \setminus X$.1	.3	.5	.7	.9
0	.479	.374	.420	.584	1.024
.1	.479	.439	.459	.639	1.119
.2	.579	.479	.509	.689	1.189
.3	.579	.519	.539	.759	1.299
.4	.669	.539	.589	.849	1.429
.5	.749	.599	.649	.909	1.569
.6	.949	.689	.739	1.029	1.799
.7	1.069	.779	.859	1.199	2.169
.8	1.399	.949	1.049	1.479	2.749
.9	2.049	1.249	1.339	1.999	4.119

In Fig. 8a, the histogram shows an early peak and a slowly decreasing tail, quite like Hirsch's graphs as described. (The subsidiary peaks, valleys, and gaps are due to computational coarseness and have no physical meaning; inclusion of more paths would smooth out the histogram.) The peak occurs at $T_f = .5$ to $.6$; 50 percent of the T_f values are less than $.8$, and 80 percent are less than 1.3 .

Histograms for $A = .33$, $H = .70$ (the trough of Fig. 3) and for $A = B = H = 0$ (the sidewall weir of Fig. 2) are shown in Figs. 8b and 8c, respectively, for comparison. For these the peaks and 50 and 80 percentile marks occur much later than in Fig. 8a. In large part, but not entirely, this is due to the differing values of Y_0 ; it is also due in part to the less uniform flow patterns for the large trough and the sidewall weir. Certainly the fact that the peak is less well

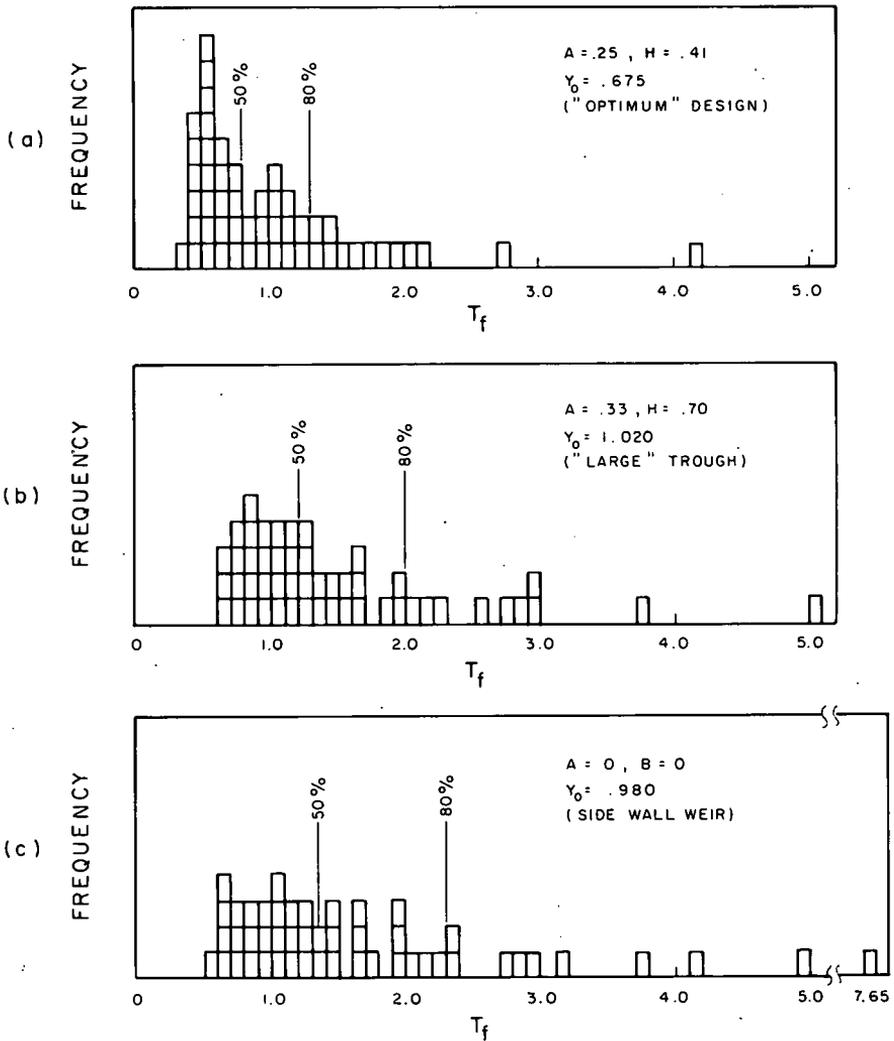


Figure 8. Flushing histograms for three trough designs.

defined and the tail longer in Figs. 8b and 8c is due to less uniformity in the respective flow patterns; and points out an advantage of the "optimum" design beyond the fact that its Y_0 is least: it also removes the floc more quickly. A wash of duration $T_f = 2.2$ would theoretically be sufficient for the conditions of Fig. 8a; the volume per unit area of washwater required (perhaps a more important factor than the time) is theoretically $T_f s = 2.2s$.

Trough Spacing

To minimize the volume of washwater required, $T_f s$, it appears that one should select the "optimal" trough dimensions as determined herein to minimize T_f and then minimize s . What are the practical lower limits on the center-to-center trough spacing, $2s$?

There are at least two important hydraulic considerations. The first is that the troughs must be large enough to carry the flow entering them, yet they should not be larger than necessary. When discharge out of the trough is free (unhindered by any conditions in the collector channel) and flow into the trough is uniformly distributed along its length, the discharge Q is related to the *internal* breadth $2a$ as

$$Q^2 = g (2a)^5 \text{ times a constant} \quad (12)$$

where the constant is a function of the trough's cross-sectional shape, and the ratio of flow depth to trough breadth.

For U-shaped troughs with flow depth about equal to breadth $2a$, the constant is about equal to 0.1 (11). The discharge Q is also equal to $2 sLv_r$, where L is the trough length and $2 sL$ is the filter area served by the trough, so that

$$(2 sLv_r)^2 = 0.1 g (2a)^5 \quad (13)$$

If the trough wall is relatively thin, $2a$ is nearly equal to $2As$, the trough external breadth.

If A is the optimal value of .25,

$$(2sLv_r)^2 \cong 0.1 g (2 \times .25 s)^5 \quad (14)$$

or

$$L \cong .03 g^{0.5} v_r^{-1} s^{1.5} \quad (15)$$

The trough length L is thus quite sensitive to the half-spacing, s ; to reduce s , while maintaining constant A and H , would entail a concomitant reduction in trough length, hence filter width, L .

The second hydraulic consideration is that reasonable uniformity of discharge *per unit length of weir* should be assured, in addition to flow uniformity in a plane normal to the weir as considered previously.

Although the troughs should be installed with the greatest care given to the elevation of the weirs (1/32 of an inch is sometimes specified as a

tolerated maximum variation in weir elevation in one filter), a certain variation is unavoidable, and design should, therefore, render the flow acceptably insensitive to such variation. Insensitivity is increased by increasing the depth of flow over the weir: If the water surface elevation midway between troughs is h_0 and the weir elevation is h_1 , the head on the weir is $h_0 - h_1 = h$, varying slightly from point to point. The discharge per unit length of weir is

$$q = Ch^{3/2} \quad (16)$$

where C is a coefficient and $q = v_r s$.

A variation in head, dh , will result in a variation in discharge

$$dq = \frac{3}{2} Ch^{1/2} dh \quad (17)$$

The fractional variation in discharge is

$$\frac{dq}{q} = \frac{3}{2} \frac{dh}{h} = \frac{3}{2} \left(\frac{C}{v_r s} \right)^{2/3} dh \quad (18)$$

For a given tolerance dh , and given v_r and C , an acceptably small dq/q depends, therefore, on a sufficiently large value of s .

Besides these two hydraulic considerations, one may note simply that a low value of s implies a greater number of troughs, and a greater construction cost. Furthermore, troughs too close together would prevent access to the bed for maintenance or inspection.

If the filter is to be provided with facilities for simultaneous air and water backwash, there is yet another design factor: the rising air bubbles lift considerable amounts of filter media to the water surface. These grains would be lost from the filter if water were passing over weirs as conventionally designed. One solution, practiced in some European designs, is to design the weirs with a quiescent zone sheltered from air bubbles, to permit filter grains to settle out before the water reaches the weir crest; the settled grains fall back to the bed (12). The other solution is to set the troughs high enough so that the water surface, initially drawn down to the top of the bed, never reaches the weirs during the relatively brief air-water wash. Weir elevations set by this consideration will thus be some given distance y_0 above the top of the expanded bed at the greatest backwash rate. If $Y_0 = y_0/s$ is to be the optimum value of 0.675, s is then established at $y_0/0.675$; there is no point in making it less.

Under what conditions are sidewall weirs feasible, according to this discussion, and how are they able to operate with relatively little vertical clearance between the top of the bed and the weirs? Filters with sidewall weirs and relatively small Y_0 , common in many European designs, can only be, and obviously are, backwashed with only very slight fluidization; any intention to expand the bed in backwash must be accompanied by a design

to permit removal of water at numerous points across the top of the filter, as by troughs, and not just by sidewall weirs, unless one is prepared to keep the top of the bed a depth y_0 below the weirs equal to about half the filter width, $2s$.

Conclusions

For a given trough spacing, the round-bottom trough that minimizes the region of nonuniform flow above a filter bed, consistent with an efficient shape for internal hydraulic capacity, is one with external total width equal to .25 times the center-to-center spacing, and with a total external draft equal to .20 times the center-to-center spacing. This design also promotes more efficient flushing of floc particles from the filter than do either the much larger or much smaller trough sizes studied.

Diminution of the nonuniform flow region, and more efficient flushing, can also be promoted by decreasing the trough spacing; but there are practical lower limits to trough spacing.

The computational techniques described have not been employed exhaustively herein. Rather, it is hoped that they may be of use to those who may care to refine and extend them, to model other shapes of trough, or perhaps more realistic frequency distributions of settling velocity of the floc particles.

Acknowledgments

The author wishes to thank his employer, Camp Dresser & McKee Inc., for permission to prepare this paper for the Freeman Hydraulics competition and for publication; for making available the CDM computer; and for the programming skills of Mrs. Nancy LeSaffre, the drafting skills of Mrs. Erika Kalam, and the typing skills of Mrs. Esther MacLaughlin.

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OZONE DISINFECTION OF SECONDARY EFFLUENTS⁽¹⁾

By

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George V. Zintel⁽²⁾

Introduction

The current demand for water quality improvement has resulted in a search for new, improved and economically feasible treatment methods. As standards are raised and water re-use becomes more likely, tertiary treatment procedures become necessary. Recent investigations^{1,2} have indicated that secondary effluents disinfected with chlorine are toxic to aquatic life. A viable alternative to the use of chlorine is ozone for disinfection and concurrent tertiary treatment.

Ozone, O₃, is an unstable gas naturally occurring in the upper atmosphere. It is responsible for the characteristic sweet odor in the air immediately following an electrical storm. Because ozone is unstable it cannot be effectively stored or shipped and must be generated on-site. Commercial ozonators mimic nature by generating ozone by electrical discharge through air or oxygen. The process produces ozone as a mixture with either air or oxygen, and the exit gas from the ozonator typically contains 1 percent ozone from air feed or 1.7 percent ozone using oxygen feed. Higher concentrations are possible but less economically efficient. The ozonized gas can then be dispersed into the water to be treated either by various injection methods or by forcing the pressurized gas through a porous disperser at the bottom of a treatment tank. Various methods of ozone mass transfer have been recorded recently.³

A variety of laboratory studies^{4,5,6,7,8,9,10,11} initially demonstrated the effectiveness of ozone in secondary effluent disinfection and contaminant removal; however, the lack of uniformity in experimental techniques and methods has made comparison of data difficult. The goal of this work was to establish the degree of treatment possible with various ozone dosage levels. Additional information has been obtained in pilot scale studies in the cities of London, England,¹² Chicago, Illinois¹³ and Washington, D.C.,¹⁴ where water of potable quality has been produced by ozonation of a variety of effluents. As an extension of this work, a pilot study at the Louisville, Ky., Fort Southworth Sewage Treatment Plant was reported recently.¹⁵

Previous work has established that ozone treatment of sewage can be considered at several dosage levels: (1) as a treatment for primary sewage and storm water overflow (ozone dosages of 10 to 100 mg/l required), (2) as

(1) Presented in part by Mr. Carl Nebel at a meeting of the Environmental Group, February 26, 1975.

(2) Welsbach Ozone Systems Corporation, Philadelphia, Pennsylvania.

a tertiary treatment to convert secondary effluent to water of potable quality (ozone dosages greater than 50 mg/l required), and (3) as a replacement for chlorine for disinfection whereby a certain degree of tertiary treatment is concurrently observed (ozone dosages of 5 to 15 mg/l required). In this study we have attempted to elucidate more accurately the ozone dosages required for disinfection and to determine the effectiveness of removal of specific contaminants in the effluent which are removed simultaneously.

Experimental Methods

Secondary effluents obtained from sewage treatment plants in Millville, New Jersey (contact stabilization), Hatboro, Pennsylvania, (standard activated sludge), and Northeast Philadelphia, Pennsylvania, (pilot plant oxygen enriched activated sludge), were ozonized in a 6.5 in. (16.5 cm) I.D. Plexiglas ozone contact column of approximately 15 gal. (56.7 lit.) capacity (Figure 1). Ozone was generated from either air or oxygen feed with a Welsbach T-408 or T-816 ozonator. The column off-gases were passed through 2 per cent unbuffered potassium iodide solution for excess ozone analysis. Ozone analyses were performed via the iodine-standard thiosulfate titration procedure.¹⁶ Samples were removed from the column at specified times (i.e., ozone dosage levels) and were analyzed for contaminant concentrations. The bacteriological, suspended solids, and BOD analyses were performed by Dalare Associates, Inc., Philadelphia, Penna. COD analyses were determined by the dichromate method and dissolved oxygen via a modified Winkler procedure. Other contaminant analyses were performed on a Hach AC-DR colorimeter using standard procedures.

Disinfection By Ozonization

The potent germicidal properties of ozone have been attributed to its high oxidation potential, approximately twice that of chlorine in water. Both disinfectants destroy bacteria via chemical oxidation, possibly via inactivation of the essential enzymes of the cells. An important step in the action of disinfection is the penetration of the cellular membrane. In applying the criterion of free oxidant to ozone, it is assumed that the effectiveness of the bactericidal action is a function of the ozone remaining. This reasoning has been extended in the ozone literature^{13, 17} to the concept of "all or nothing" disinfection, implying that a certain threshold of ozone dosage must be reached before any disinfection takes place. Data developed within the scope of this work have shown that this effect is not valid, i.e., oxidation of pollutants takes place simultaneously with the destruction of microorganisms. Leiguarda's¹⁸ investigations support the finding that the bactericidal action of ozone proceeds in parallel with the oxidation of organic matter. It would be presumptuous to assume that a powerful oxidant such as ozone would selectively attack organic contaminants and not simultaneously destroy microorganisms that are also composed of organic compounds.

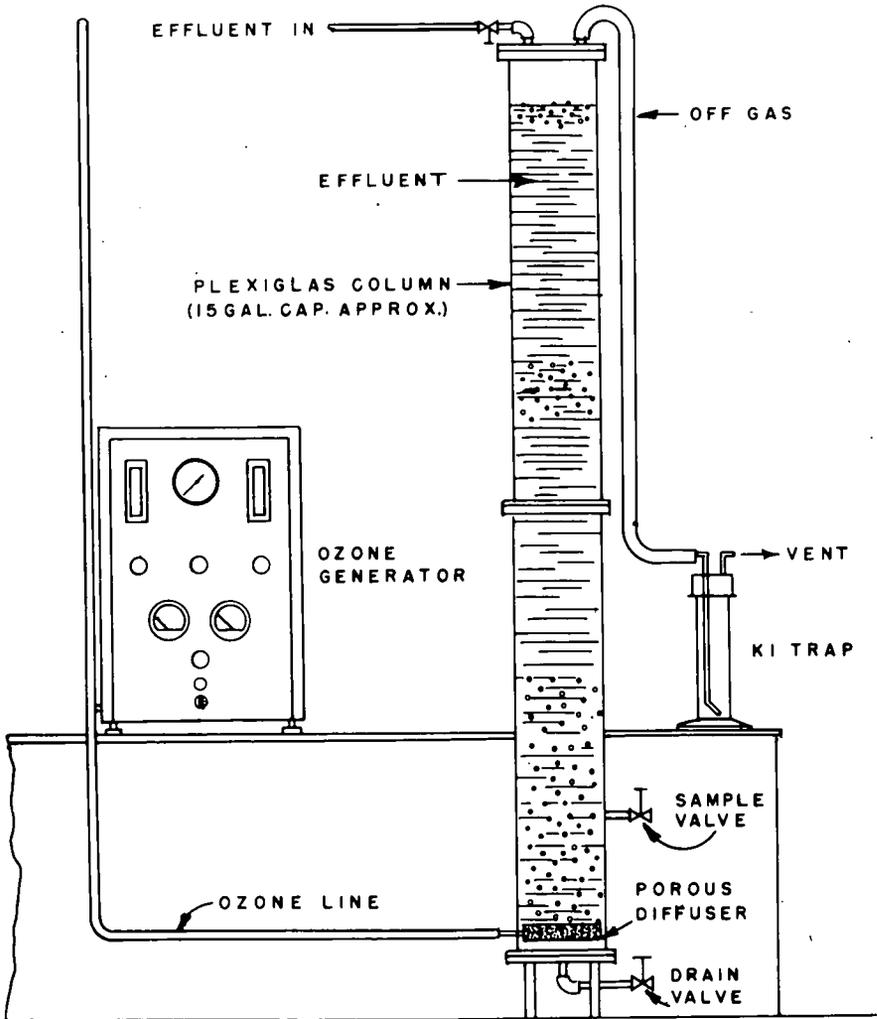
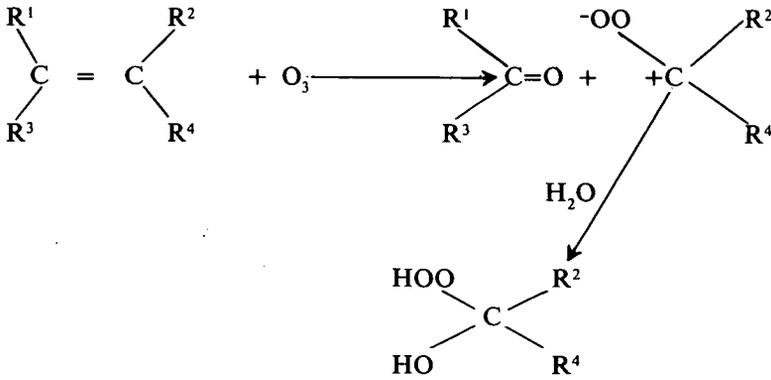


Figure 1. Ozonization apparatus

Leiguarda's findings imply that some particles produced by the ozonization of sewage are also effective in reducing the concentration of microorganisms. Modern organic ozone chemistry would predict that oxidized organic compounds containing carbon-carbon double bonds give rise to hydroperoxy alcohols by the following reaction route:



where R = alkyl, aryl, or hydrogen.

Indications exist that organic peroxides exert a stronger bactericidal action than hydrogen peroxide since they are more prone to decomposition.¹⁹ Thus the disinfection action of ozone can proceed by oxidation with ozone directly and by intermediate hydroperoxy compounds that can interact with the cellular cytoplasm. There is no indication in the literature of any microorganism resistant to the action of ozone.

The amount of ozone required to produce sterilization or any level of disinfection is dependent upon the exact nature of the effluent. The characteristics of a secondary effluent vary from one treatment plant to another and are also a function of time within a given plant. Figure 2 shows that destruction of total coliform in an ozonized effluent varies slightly from one treatment plant to another. Figure 3 indicates that the total coliform count is reduced at approximately the same rate as the fecal coliform count. The standard plate count, an indicator for a large number of microorganisms, was rapidly reduced from approximately one million to one hundred upon application of ten mg/1 (ppm) of ozone in a typical example, Figure 4.

Although coliform standards among different regulatory agencies vary widely and are often ambiguous, requirements of 2.2 to 1000 M.P.N. per 100 ml are common. Most states require that secondary effluents be disinfected to a plate count of 200/100 ml of fecal coliform. Ozone dosages of five to ten mg/1 are adequate for this level of disinfection with the particular effluents studied. The limited data available in the literature are in general agreement with our results,^{12,13,20} however, higher dosage levels may be required when significant amounts of ozone susceptible organic materials from industrial loading or other sources are present.^{15,21,22}

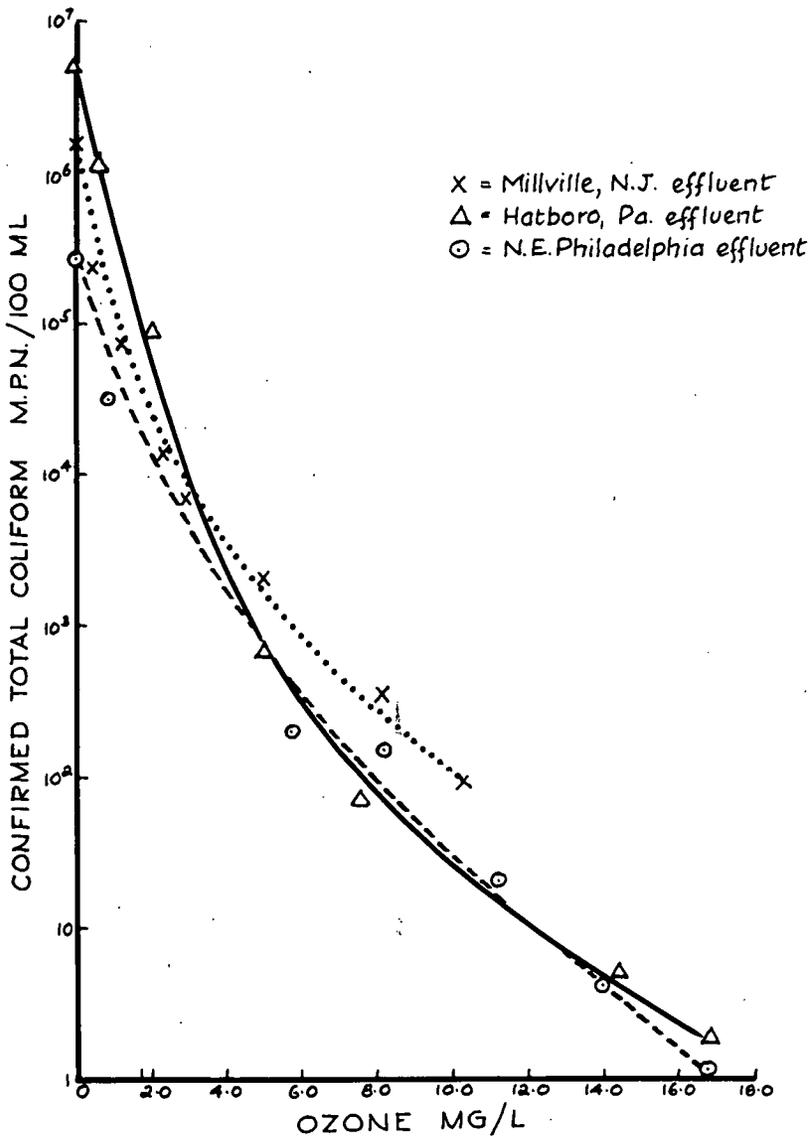


Figure 2. Total coliform reduction for different secondary effluents.

The rate of ozone disinfection has been shown to be 3125 times faster with ozone than with chlorine.²³ Secondary effluents have been disinfected within a 2.5 second contact time in a static mixer.²⁴ Hence it can be concluded that the rate of ozone disinfection is mass transfer controlled and not contact time controlled as is the case with chlorine.

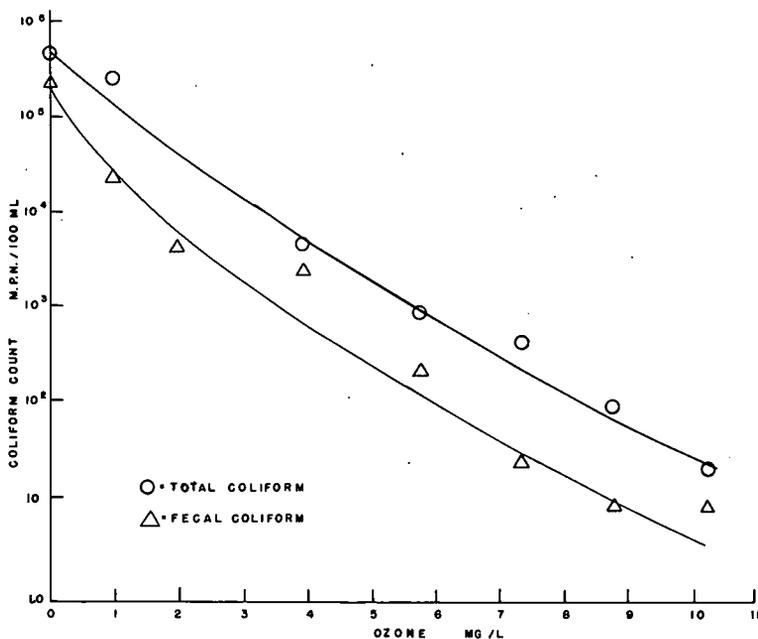


Figure 3. Total and fecal coliform reduction by ozonation.

Recent literature has shown that ozone is more effective than chlorine in removing virus from secondary effluents.^{15,25,26} Recent work has shown that approximately one ppm of ozone is required to remove polio virus from a secondary effluent.²⁷

Ozone Residual

Ozone is often considered as the choice of disinfectant because it is not harmful to the biota.² It is safe because no residual or harmful reaction products reach the receiving stream. Analysis of secondary effluents after treatment with up to 20 ppm of ozone do not show any true ozone residual. It appears that the rate of reaction of ozone in this concentration range (0-20 ppm) with carbonaceous materials is faster than the mass transfer step. Of course, true ozone residuals can be observed in secondary effluents when very high dosages (greater than 40 ppm) are applied.²⁸ In the high concentration ranges, the reaction is kinetically controlled rather than mass transfer controlled.

Without the presence of an ozone residual it is impossible to automatically pace ozone disinfection systems operating in the dosage range of 0-20 ppm. The present work has shown the existence of ozone reaction products which may be correlated with bacteria kill. In the previously shown ozone reaction scheme it was indicated that interaction of ozone with the carbon-carbon

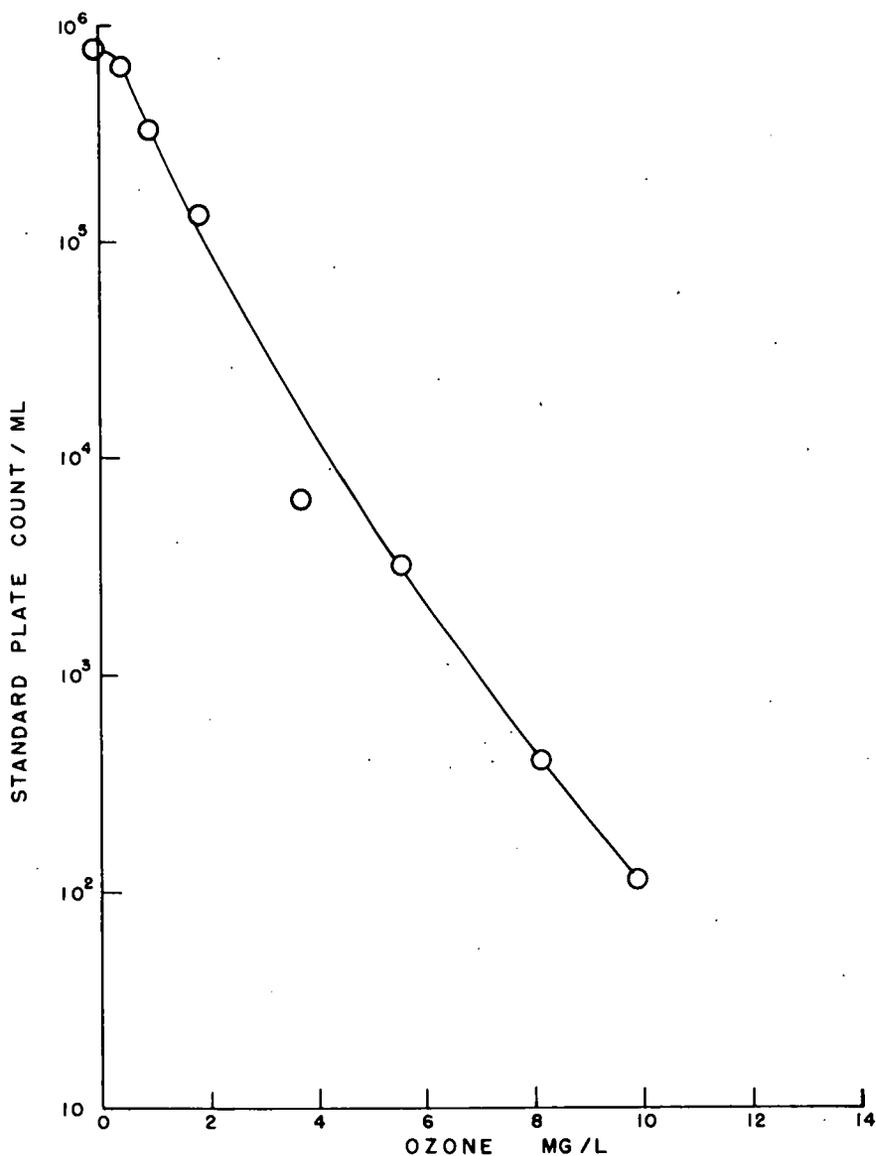


Figure 4. Standard plate count reduction by ozonization.

double bonds produces hydroperoxy alcohols. Compounds possessing carbon-carbon double bonds exist in all effluents after secondary biological treatment. The fact that peroxides are formed by the ozone reaction can be demonstrated by their free radical initiation of the polymerization of added styrene.

The exact nature of the peroxides formed by the ozone reaction depends on the structure of the molecule which contains the carbon-carbon double bond. Many of the different types of peroxides which can be formed include hydrogen peroxide, peracids, hydroperoxy alcohols and hydroperoxy ethers. Such peroxy compounds all react with potassium iodide to liberate iodine. The quantity of iodine generated can be readily determined by wet chemical, light absorption or amperometric techniques. Since it is impossible to identify which molecule in the secondary effluent has been attacked by ozone, it is impossible to determine the exact structure of the peroxide formed. It is convenient, therefore, to calculate the peroxide residual as a quasi ozone residual. Figure 5 shows that the quasi ozone residual increases as a function of the quantity of ozone absorbed by the secondary effluent. Increasing ozone residuals can also be correlated with decreasing total coliform counts. See Figure 6.

It is readily conceivable that peroxidic compounds might be harmful to the biota but toxicity studies show the contrary.^{2,38} Detailed studies of such compounds have shown that these peroxides are chemically very unstable and tend to decompose rapidly.^{29,30} They are also removed from the secondary effluent by oxidizing other molecules. The rate of removal to a given level depends on the initial concentration of these peroxides. See Figure 7.

Utilization of the above data shows that an ozone disinfection system can be automatically controlled by the analysis of the quasi ozone residual in a manner similar to the chlorine residual method.

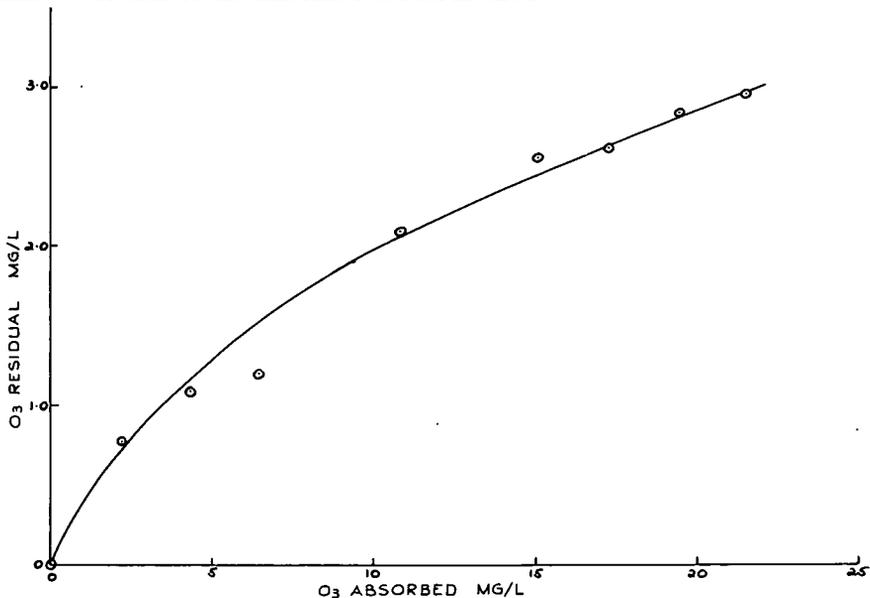


Figure 5. Increase of ozone residuals as a function of dosage levels, Millville secondary effluent.

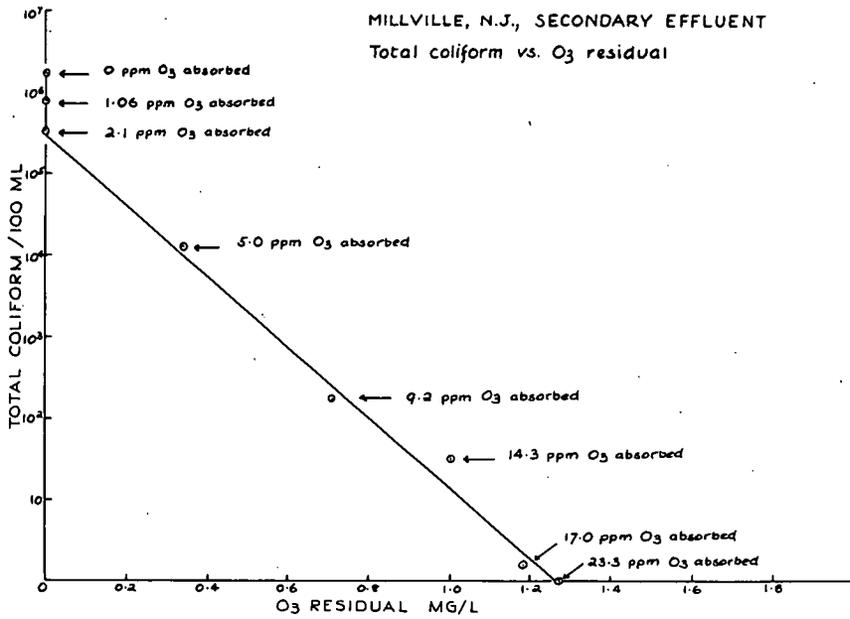


Figure 6. Total coliform as a function of ozone residual.

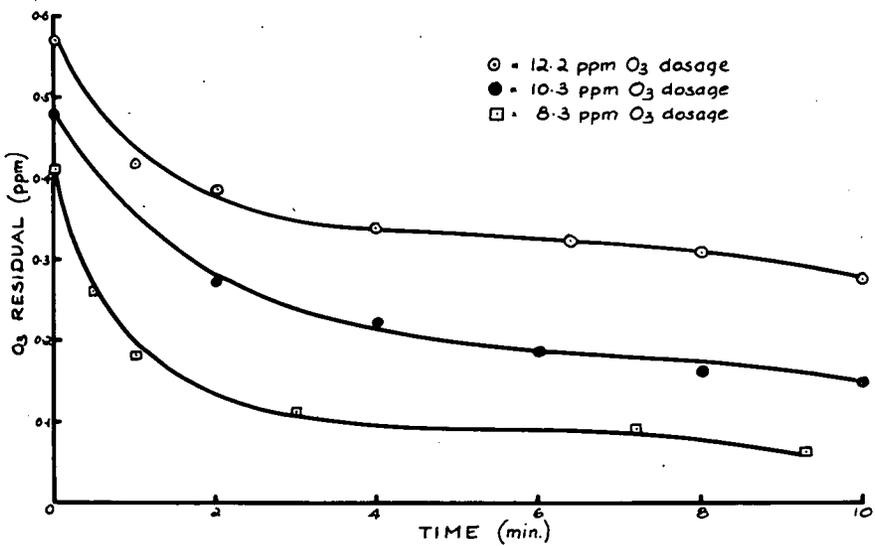


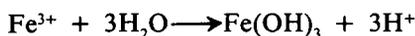
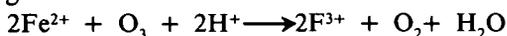
Figure 7. Rate of ozone residual decay.

Frothing Process and Contaminant Removal

Contaminant removal from secondary effluents proceeds by a two-fold process when ozone is properly applied. The classical method of pollution control with ozone is via chemical oxidation. With proper application, ozone can be used to produce a frothing operation that will also remove the pollutants by a physical flotation process. This process relies on the large volumes of inert gas introduced into the contact chamber with the ozone. (Recall that ozone is produced in one per cent weight concentration from air or 1.7 per cent from oxygen).

The utilization of this process does not require any additional ozone above that necessary for disinfection, but does require that the contact chamber be designed to remove the collapsed froth. Thus it can be envisioned that this secondary mode of pollution control would not require any additional capital or operating expense for ozone generating equipment.

Foulds, Wilson, and Clark³¹ have also observed frothing upon the ozonization of secondary effluents and theorize that a froth formed on the surface of the liquid in the ozone contact chamber may be stabilized by finely dispersed iron and manganese hydroxides formed by ozone oxidation of the naturally occurring metal ions in solution.³² These insoluble metallic hydroxide products act as coagulants for the organic material that is then floated to the surface of the liquid (rather than settling as would be the case in a clarifier) by the large volumes of gas flowing upward in the ozone contact chamber. Iron hydroxide is chemically produced with ozone by the following reactions:



In an effort to test the metal oxidation hypothesis, additional iron as ferrous ammonium sulfate was added to a secondary effluent, and the amount of froth formed on ozonization was measured. The results recorded in Table I clearly show that the addition of iron aids the process, yielding a greater amount of collapsed froth. In practice no additional iron salts need be added to the secondary effluent since there is an adequate concentration available as a natural background.^{33,34}

An alternate explanation may be envisioned where the frothing phenomenon is attributed to foaming induced by residual detergents present in secondary effluents. However, Ryckman^{5,6} has shown that ozone destroys the surfactant properties of anionic detergents by oxidative cleavage of the detergent molecule. The white detergent foam was replaced during ozonation by a thick, dark brown froth containing significant quantities of suspended solids that had been floated to the top of the contact chamber. This frothing was not observed in the absence of ozone when only air or oxygen was passed through the effluent. Only the white detergent foam was then

TABLE I
EFFECT OF ADDED $\text{Fe}(\text{NH}_4)_2(\text{SO}_4)_2 \cdot 6\text{H}_2\text{O}$ UPON
FROTH FORMATION

Amount of Iron Present	Weight of Collapsed Froth
Natural Background (0.29 ppm Fe)	11.6 g
Natural Background +5 ppm Fe added	21.8 g

observed, and no suspended solids flotation was evident.

The froth formed by this process is readily removed by a stationary skimmer installed in the top of the contact chamber. The stability of this gelatinous froth depends on the existence of moisture and large volumes of gas. When the froth flows into a skimming trough, the moisture and gas are no longer present and the froth collapses to a material that is analogous to secondary sludge. This process creates a super-clarifier that negates the use of sand filters or micro-straining apparatus. Studies have shown that ozone contact times in the range of ten minutes are adequate for the incorporation of the flotation process.

Visual Characteristics of Ozonated Effluent

Suspended Solids

The suspended solids not captured by the plant's secondary clarifier are readily removed from the ozone contact chamber by the flotation process previously described. Figure 8 shows that major portions of suspended solids can be separated by flotation and froth removal within the dosage levels of ozone required for disinfection. Field work with dynamic experiments has indicated that the suspended solids of an ozonized effluent are in the range of one to two ppm when the ozone contact chamber is properly designed.¹⁵

Turbidity

The turbidity of a secondary effluent is a composite of suspended solids and the dispersed colloidal matter. Removal of the suspended solids by flotation contributes to the turbidity decrease. Colloidal matter is material within a negatively charged spherical envelope. The negative charges of the sphere are neutralized by oxidation with ozone since this allotrope of oxygen often reacts as if it has a positive charge. Therefore, colloids are destroyed by a combination of charge neutralization and chemical oxidation. The complex nature of these reactions is composited in Figure 9, showing that ozone rapidly destroys the turbidity.

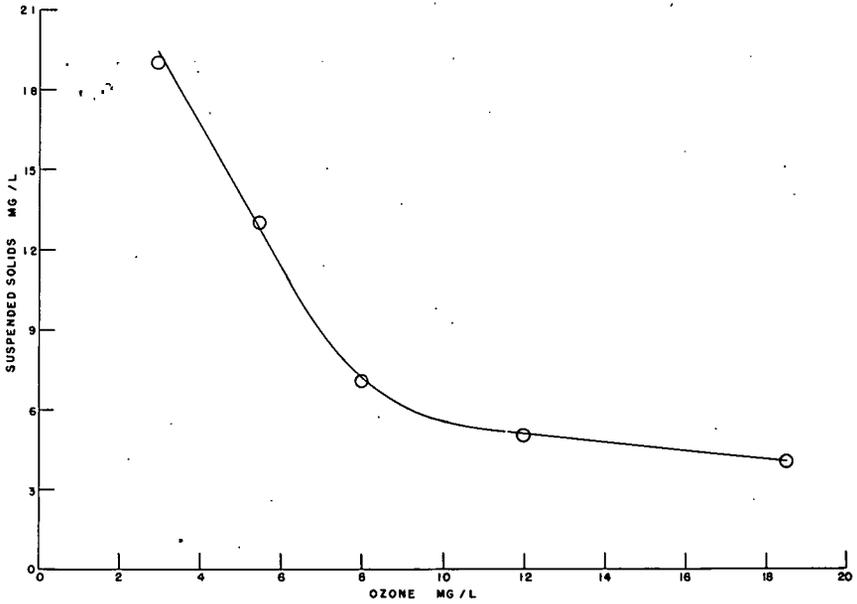


Figure 8. Suspended solids reduction by ozonation.

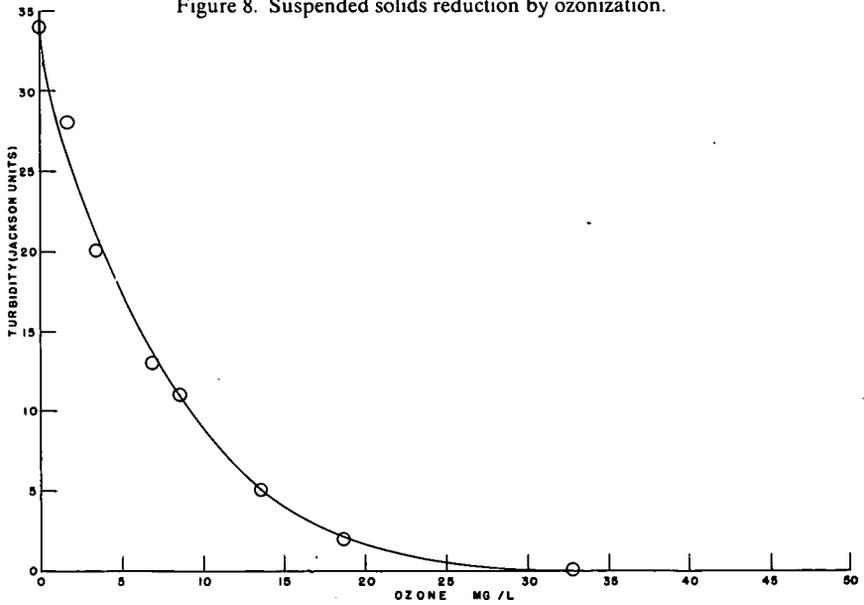


Figure 9. Turbidity reduction by ozonation.

Color

Color is most often caused by organic molecules having long chains of carbon-carbon double bonds that are conjugated with other functional groupings. Molecules of this type absorb light in the visible and ultraviolet region and reflect light in the visible spectrum. Breakage of the double bonds destroys such chromophoric groupings. Ozone is known as a double bond reagent, i.e. it is very effective in breaking double bonds. Large reductions in color are found when a secondary effluent is treated with ozone (Figure 10). Other workers^{12,13} have reported color reduction as high as 90 percent resulting from ozone treatment. Only small ozone dosages are required to remove substantial amounts of colored material. The combination of suspended solids, turbidity and color removal with ozone produces a secondary effluent that cannot be distinguished visually from potable water. Secondary pulp and paper mill effluents can also be decolorized with ozone,^{22,35} also effluents from synthetic dye operations.³⁶

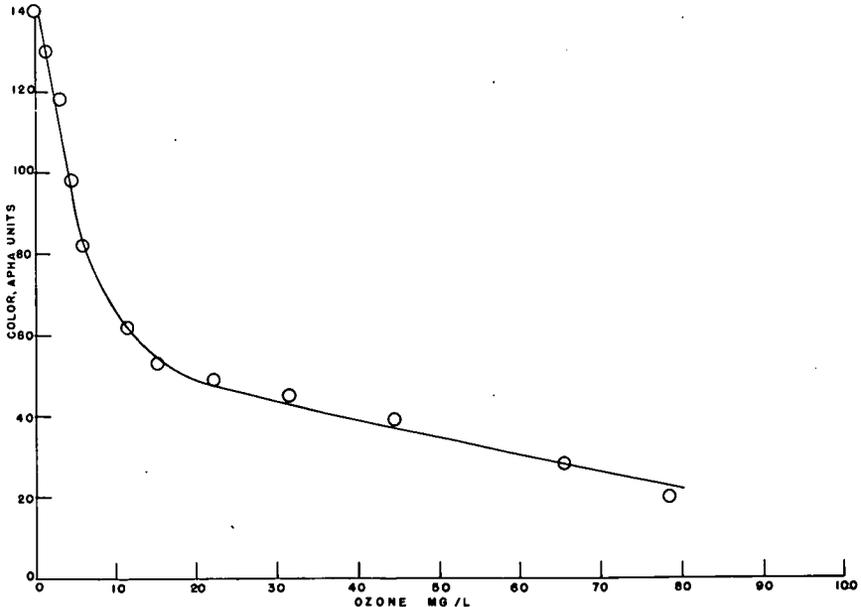


Figure 10. Color reduction by ozonization.

Removal of Carbonaceous Materials

Chemical Oxygen Demand

Reduction of COD levels of a secondary effluent via ozone oxidation and physical removal by frothing varied; however, an approximate correlation

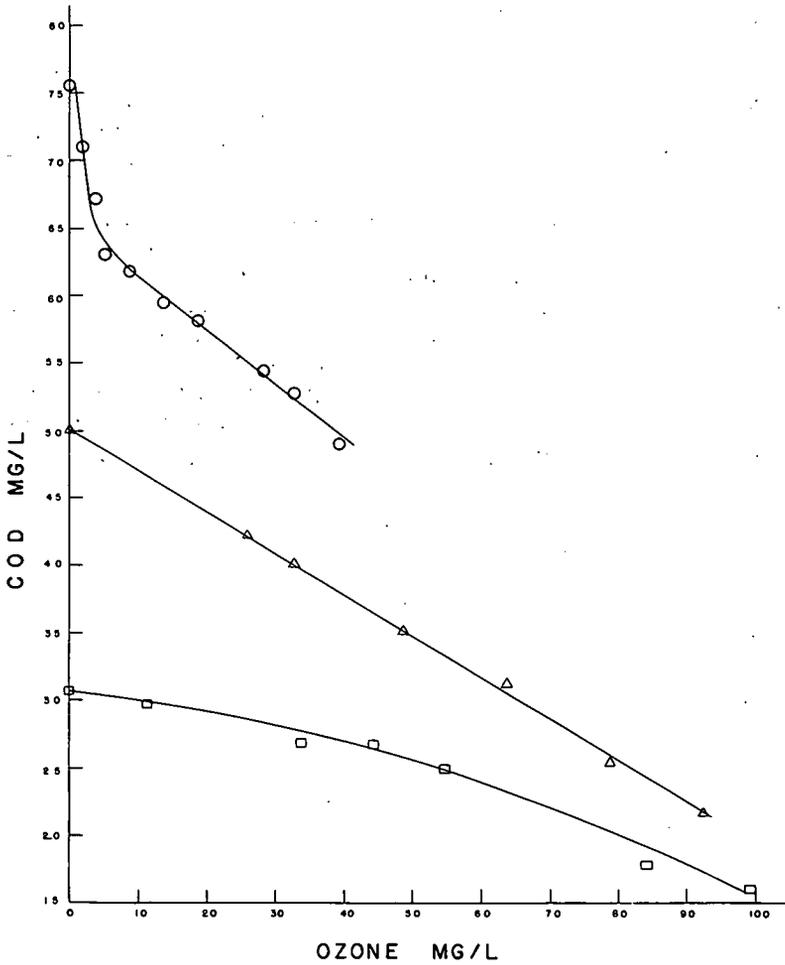


Figure 11. COD reduction of different secondary effluents.

could be made between the rate of reduction and the initial COD level of the effluent. Figure 11 depicts the decrease in COD upon ozonization of three samples of Millville, N.J., effluent collected at different times of the year. (Effluent collected during the winter had the highest initial COD, as might be predicted). The rate of COD lowering is generally proportional to the initial COD level. The higher COD effluents may contain a greater proportion of readily oxidizable organic materials, whose rates of reaction with ozone are often more rapid than those of the materials remaining after extensive secondary oxidation. These data show that it is virtually impossible to predict the exact amount of COD reduction to be expected from a given

ozone dosage level when grab sample techniques are employed; it depends on the inherent differences between various effluents, the nature of a given effluent, and the nature of its prior treatment. To obtain data that will allow COD predictions, a statistically significant number of experiments must be performed at various ozone dosage levels and effluent parameters.

Biological Oxygen Demand

The reduction of BOD_5 level of the ozonized effluent was less pronounced than the observed COD reductions. In some cases the soluble BOD_5 level of the ozonized effluent remained constant or actually increased slightly before being reduced (Figure 12). Initial increases in BOD_5 upon ozonation have been observed by previous workers.^{6,20} The COD decrease shows that oxidation is taking place in this secondary effluent. Non-biodegradable (refractory) organic compounds are made biodegradable by oxidation with ozone. Ozonation will introduce oxygenated functional groups into the refractory compounds, thus producing a site on the molecule where biodegradation can commence. The increase in the total amount of biodegradable compounds will, of course, exert an additional oxygen demand in the BOD_5 test. It should be noted that the refractory materials that are made biodegradable by ozonation are most often those materials that would have been otherwise toxic to the receiving stream.² The effect of mildly increasing BOD_5 values is generally observed in mixed industrial-municipal effluents and not in purely municipal effluents.¹⁵

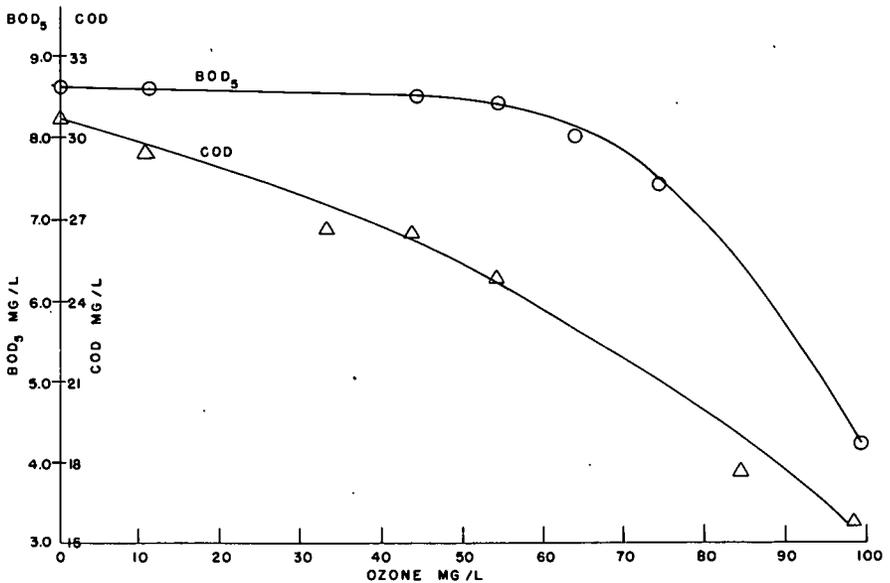


Figure 12. Concurrent COD and BOD_5 reduction by ozonation.

Oxidation of Nitrogenous Materials

Theoretically, all nitrogen-containing compounds should be oxidized by ozone to the nitrate (NO_3^-) anion. This prediction would be true if large enough dosages of ozone were applied. However, in ozone treatment, as in any other treatment process, practical limits are specified. The question then arises as to how much oxidation of nitrogenous materials can be expected if a practical limit (i.e., ozone dosage level) is specified.

Oxidation of Ammonia

Thermodynamic predictions indicate that ammonia should be readily oxidized to nitrate by ozone:



The literature^{8,10,11,12,13,20} records many inconclusive studies in this area. Laboratory results have shown only a mild reduction of ammonia despite high ozone dosage levels (Figure 13). These data indicate that the removal of ammonia from a secondary effluent is influenced by poor kinetics, utilizing ozone to oxidize carbonaceous materials rather than the ammonia. Although mild reduction of ammonia is seen, ozonolysis cannot be considered a nitrification process.

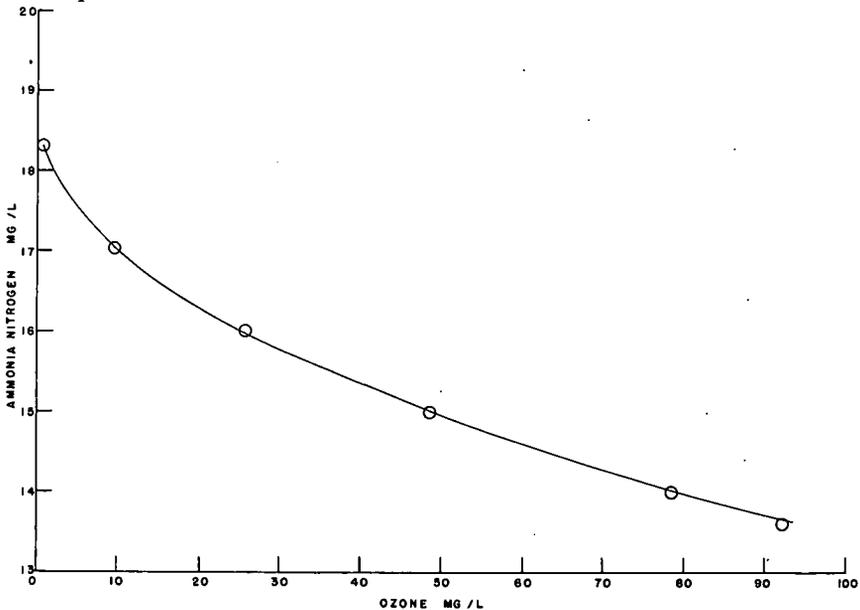


Figure 13. Ammonia nitrogen reduction by ozonization.

The efficiency of ozone disinfection is not greatly affected by the presence of ammonia, whereas the effectiveness of chlorine as a disinfectant is dependent on the amount of ammonia present. Ammonia is oxidized to the toxic chloramines by chlorine before the disinfection step occurs appreciably. Ozone disinfection can thus be considered as a process independent of the ammonia concentration.

Oxidation of Nitrite

Weingarden¹³ reported that small amounts of nitrite existing in secondary effluents are readily oxidized to nitrate by ozone. Since nitrite is so readily oxidized, it is not considered a severe pollutant in the small concentration in which it is normally found. Its ease of oxidation by ozone does not permit it to exert a BOD demand on the receiving stream. Figure 14 shows that nitrite was rapidly destroyed in the early stages of the laboratory ozone treatment. This oxidation is represented by the equation:

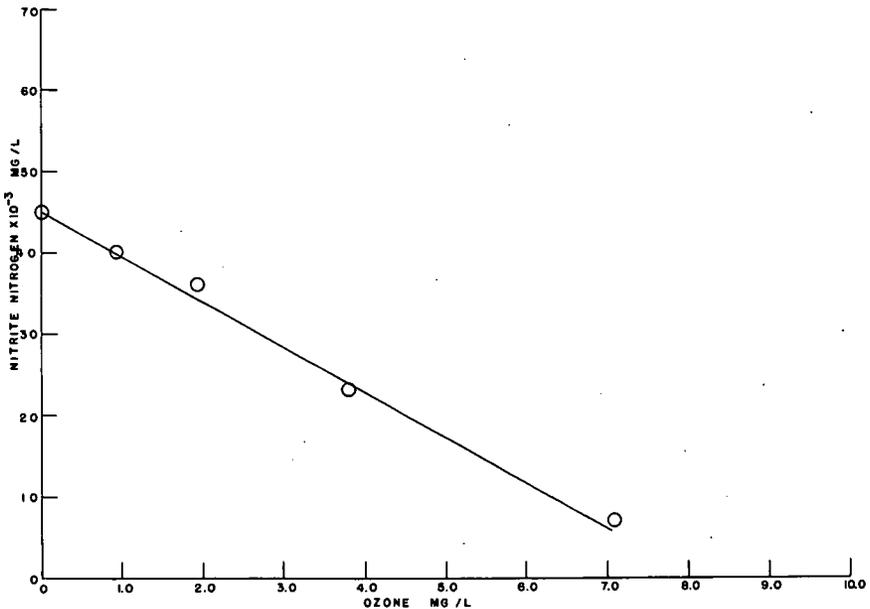
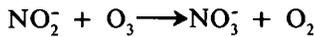


Figure 14. Nitrite nitrogen reduction by ozonization.

Increase in Nitrate Concentration

The oxidation of nitrogenous materials ultimately produces the nitrate anion. We have shown two processes that produce nitrate — the oxidation of nitrite and of ammonia. Certainly other paths of nitrate formation also influence the ozonolysis of secondary effluents. Figure 15 shows that only mild increases in nitrate concentration were experimentally observed despite the high levels of ozone applied.

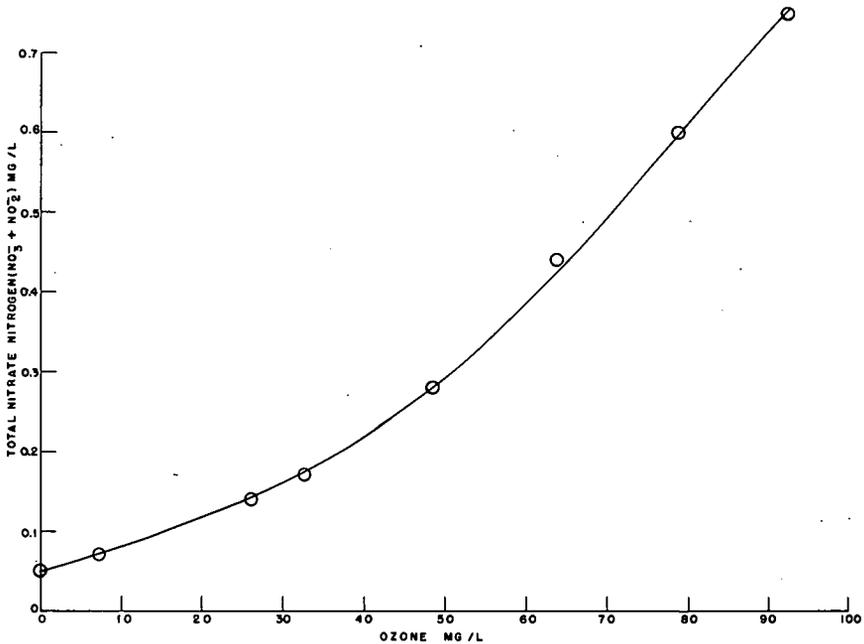


Figure 15. Total nitrate nitrogen increase by ozonization.

Effect of Ozonated Secondary Effluent on The Environment

Dissolved Oxygen Levels

Consideration of BOD₅ changes on ozonization must take into account the complete oxygen saturation of the effluent as an inherent part of the ozone treatment process. Table II depicts the amount of dissolved oxygen observed in the effluent as a function of the ozonized gas applied.

TABLE II
DISSOLVED OXYGEN CONTENT
OF OZONIZED EFFLUENT

Feed Gas	Ozone in Feed Gas (Wt. %)	Ozone Dosage to Effluent (mg/l)	Dissolved Oxygen in Effluent (mg/l)
Air	1.2	0	5.2
"	"	3.3	8.9
"	"	10.0	9.3
Oxygen	2.2	0	5.2
"	"	3.4	25.7
"	"	10.7	40.0

Note that use of oxygen feed gas at moderate ozone dosage levels results in a dissolved oxygen content of the effluent of 40 mg/l. Proper discharge of the ozonized effluent (containing the high D.O. level) below the surface of the final receiving stream will allow utilization of the available oxygen for the normal biological purification processes. In this manner the dissolved oxygen content of the receiving stream can be increased before the oxygen is lost to the atmosphere by equilibration. Because many discharge requirements are written with respect to a maximum BOD₅ content of the effluent, a credit can sometimes be obtained for the oxygen content of the discharged effluent above that of the receiving body of water. For example, an ozonized effluent having a BOD₅ of six mg/l and a D.O. level of 40 mg/l in effect has a negative BOD₅ of 34 mg/l. In all cases the need for a post aeration step is eliminated by the ozone disinfection step.

Phenol Removal

Phenol, C₆H₆O, is considered to be a stringent water pollutant. Annually, 1.5 billion pounds are produced domestically. This fundamental building block of the chemical industry finds uses in almost every manufacturing facility. Phenol is a water soluble material often used as a disinfectant, suggesting that it is toxic in nature and not very biodegradable. Shock loads of phenol have been known to kill the bacteria in an activated sludge process.

The severity of phenol pollution is felt in potable water. Since portions of the phenolic compounds are not completely biodegraded in the secondary process, they are expelled intact to the receiving stream. These materials are subsequently treated with chlorine in a potable water processing plant and are transformed into chlorophenols that are related to commercially produced insecticides and have been shown to taint the taste of fish. Chlorophe-

nols are more toxic and less biodegradable than the parent compound, phenol. Furthermore, chlorophenols have odor and taste thresholds approximately two orders of magnitude lower than phenol itself. Consequently most of the potable water in the large cities of our nation tastes and smells of "chlorine". Detailed analysis of these waters indicates that the taste and odor of chlorine does not come from chlorine itself but rather from products of chlorination, the chlorophenols.

The most practical way of eliminating this taste and odor problem is to eradicate its source, the phenolic materials in our effluents. Phenol has been removed from industrial effluents with ozone for over a decade.³⁷ An investigation of a purely municipal effluent has shown that trace amounts of naturally occurring phenol escape oxidation in an activated sludge system. Weingarden¹³ has shown that trace amounts of phenol are completely destroyed by ozone during the disinfection process. The ozone oxidation products of phenol are oxygenated materials that are nontoxic and biodegradable.³⁸ One of the first stable ozone oxidation products of phenol is the biodegradable cis-cis muconic acid. Large ozone dosages will oxidize this phenolic material to water and carbon dioxide.³⁹

A small amount of pure phenol was added to a secondary effluent sample before ozonization. Samples were taken as ozone addition continued and were immediately analyzed for residual phenol. Table III shows that phenol reduction is quite rapid at relatively low dosages of ozone. Although the lower limit of phenol detection was 0.05 ppm for the present study, Weingarden has shown that ozonization can remove phenol in the sub-ppm region.¹³

TABLE III

REDUCTION OF ADDED PHENOL BY OZONE

OZONE DOSAGE ppm	PHENOL CONCENTRATION ppm
0	0.73
1.2	0.56
3.4	0.05
6.8	<0.05
10.8	<0.05

pH Changes

A shift in the pH can be expected when ozone is employed as an oxidant for a secondary effluent. The formation of weak carboxylic acids from hydrocarbons would tend to reduce the pH. The carbon dioxide produced in the activated sludge process is removed by sweeping large volumes of air or oxygen through the sewage in the ozone contact chamber; this process tends

to increase the pH. These changes in pH are undoubtedly masked to a large extent by the large buffering capacity of domestic sewage.

Recently, workers at the Washington, D.C. Blue Plains treatment plant have reported¹⁴ that ozonolysis always shifts the pH towards neutrality, regardless of the nature of the secondary effluent to which ozone was applied. This observation was confirmed in a static experiment with large ozone dosages (Figure 16). Notice that large amounts of ozone must be applied before minor shifts in the pH occur.

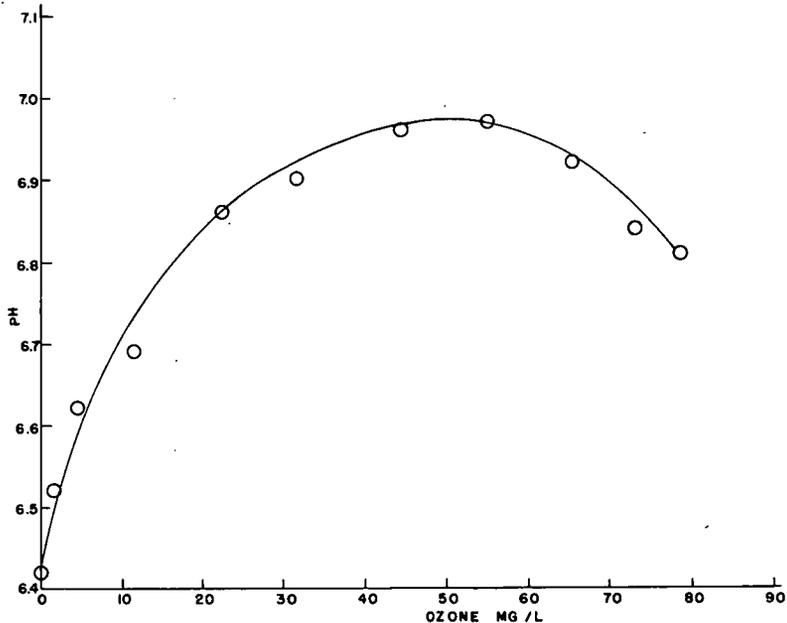


Figure 16. pH change by ozonization.

Cyanide Removal

Small amounts of cyanide are normally found in secondary effluents. These cyanide concentrations are usually not large enough to pose any health hazard to humans. Concentrations of cyanide are sub-ppm but it is nevertheless interesting to note that ozone will effectively destroy cyanide. Figure 17 shows that 6 ppm of ozone will destroy all the naturally occurring cyanide in a secondary effluent. The product of cyanide oxidation is cyanate anion (CNO^-). Cyanate is a nontoxic anion which is readily biodegradable and is also destroyed in the receiving stream by hydrolysis to ammonia and carbon dioxide. The use of ozone for cyanide oxidation has been practiced on a commercial basis for many years.⁴⁰ Thus, disinfection with ozone becomes important in municipal treatment plants where the possibility of cyanide influx from industry is prevalent.

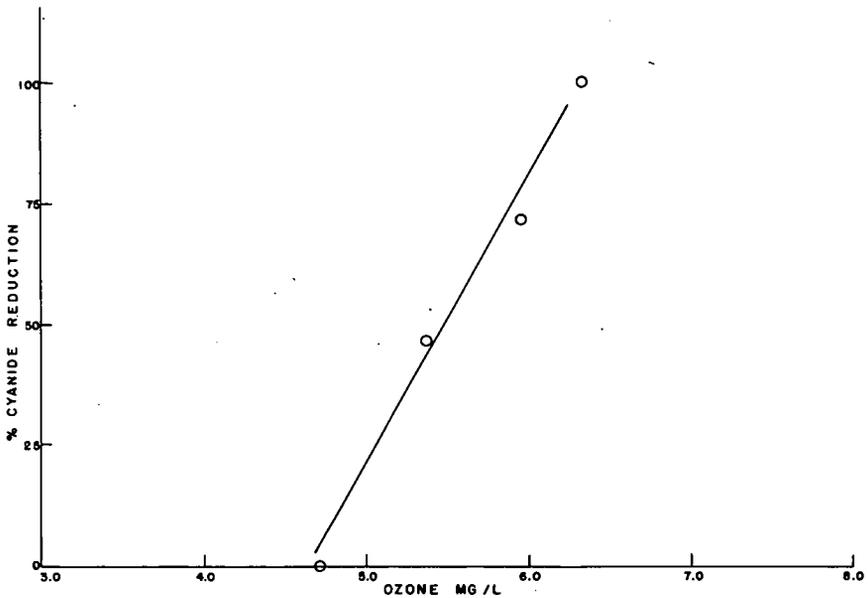


Figure 17. Cyanide reduction by ozonization.

Detergent Removal

Detergent foaming in treatment plant receiving streams was a severe problem prior to the introduction of biodegradable surfactants. The destruction of detergents by ozone oxidation is therefore well documented in the literature,^{5,10,12} since ozonolysis was considered as a solution to this problem.

Although problems associated with detergent foaming have been greatly reduced by the introduction of linear carbon chain detergents that are biodegradable, small amounts of detergents nevertheless can escape secondary treatment and cause problems. In addition, small fractions of the detergents may consist of branched-chains that are not amenable to biological oxidation. Detergent levels in the range of 1 to 4 mg/l are frequently seen in secondary effluents and are capable of generating foam.

Figure 18 illustrates the destruction of ABS and LAS detergents observed in laboratory ozonization of secondary effluent. While disinfection of a secondary effluent is being accomplished with ozone, substantial reductions in detergent levels are realized. In no case was any detergent foam observed in a secondary effluent that had been disinfected with ozone.

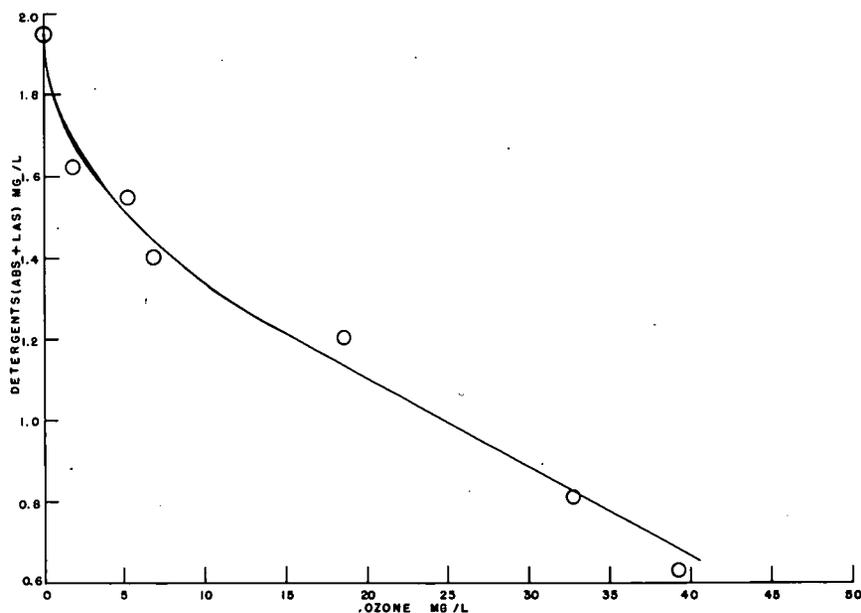


Figure 18. Detergent reduction by ozonization.

Pesticide Removal

The removal of toxic pesticides from surface waters remains an important ecological problem. It is obvious that a total ban on pesticides is not a practicable solution; we must therefore seek other solutions that will have a smaller impact on our way of life. Most pesticides are of the chlorinated hydrocarbon type whose composition makes them resistant to biological degradation; their removal from surface water must be accomplished either by physical or by chemical means.

Several workers have shown that ozone can be employed to reduce or eliminate organic pesticides.^{10,12} Ozone will react rapidly with most pesticides because these materials often contain carbon-carbon double bonds. Gardiner and Montgomery¹⁰ observed substantial lowering of pesticide concentrations when effluents containing chlorinated hydrocarbon pesticides were treated with ozone. In this study γ -BHC, Dieldrin, DDT and TDE were added to the sewage effluent. Dieldrin and γ -BHC were removed at approximately the same rate, about 50 per cent of each disappearing with the absorption of 20 ppm O_3 . A more rapid removal was observed with DDT and TDE.

Safety Considerations

Large-scale unit operations such as those under discussion should always be viewed in light of safety precautions. Ozone, being a very powerful oxidant, is also a toxic compound. When ozone is used for water pollution control it does not come in contact with man nor is it allowed to escape into the atmosphere; i.e., it is used under controlled conditions. Note that ozone is generated on-site and used immediately; there is no shipping, handling or storage of toxic materials. If all the ozone at a plant site treating 100 MGD of effluent with ten ppm ozone escaped at any one instant, only a few pounds of ozone would be released to the atmosphere; this certainly would not be sufficient to cause a fatality. The human nose can detect less than 0.01 to 0.02 ppm ozone in the atmosphere, and it is not possible to become insensitive to the odor of ozone. In the seventy years of large scale ozone usage for disinfection of potable water throughout the world, no known death has been attributed to ozone, yet fatalities are attributed to the use of chlorine almost monthly.

The human tolerance for ozone depends on ozone concentration and time of continuous exposure. The maximum allowable concentration of ozone in an enclosed area for continuous exposure is 0.1 ppm as determined by the American Council of Governmental Industrial Hygienists. The human tolerance for ozone is portrayed in Figure 19.⁴¹ Note that a deleterious condition can only exist when a combination of high ozone concentration and long exposure time occurs.

Conclusions

The above data have demonstrated that ozonization of secondary effluent is an effective method of disinfection, and effects a degree of tertiary treatment at no additional cost. Ozone dosages of five to ten mg/l can disinfect to a level below most legal requirements. Higher ozone dosages will probably be required if the effluent contains a significant amount of industrial loading that contains ozone oxidizable materials.

The concurrent decrease of COD and coliform levels with ozonization argues against the necessity of satisfying an "ozone demand" due to dissolved organics before any disinfection can occur. Indeed, since the disinfecting ability of ozone is ascribed to its high oxidation potential, it seems logical that disinfection and contaminant removal would occur simultaneously, although not all materials in solution will react at the same rate.

An added benefit of ozone disinfection is an ozone-induced frothing process capable of removing significant amounts of suspended and dissolved material via oxidation and physical separation by flotation. Exact reductions cannot always be predicted due to the variable nature of the effluent. Installation of suitable froth skimming devices in an ozone contact chamber would allow tertiary treatment as an adduct to disinfection.

Ozone should be given serious consideration as an alternate to chlorination for disinfection of secondary effluents, especially in view of the toxic nature of chlorinated effluents. In addition to coliform and other bacteria, ozone has also been shown to be effective against viruses.^{26,32} On-site generation of ozone avoids the storage of large quantities of hazardous gas. Small treatment plants could employ an air-feed ozone generator, while large requirements would be met by an oxygen-feed gas system.

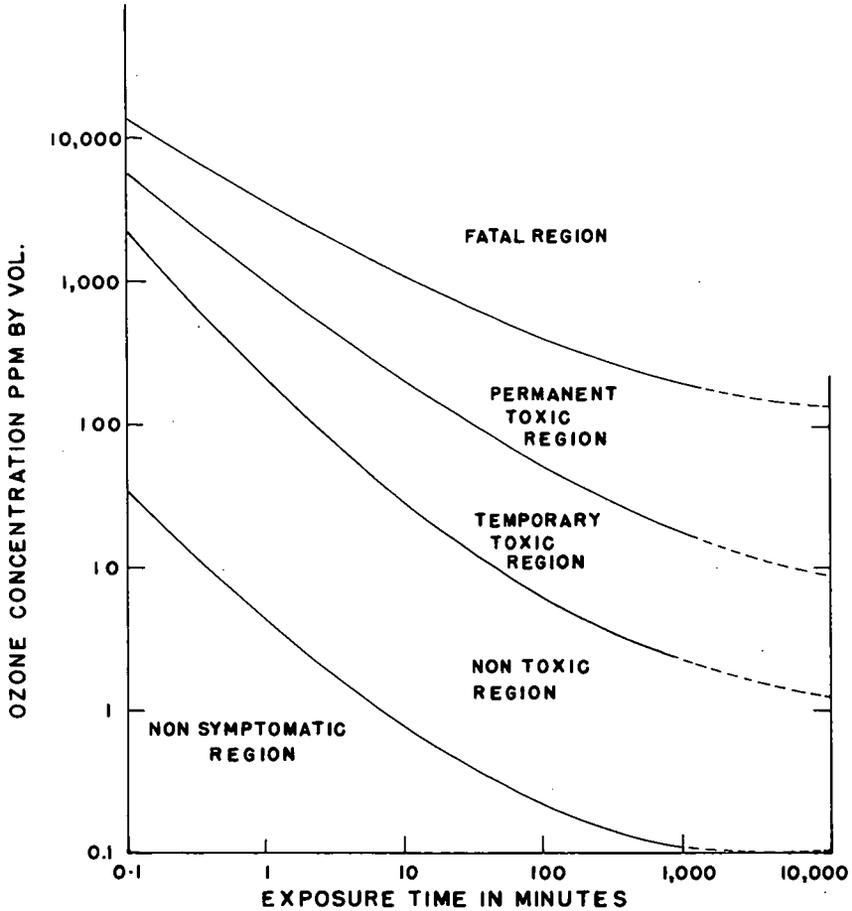


Figure 19. Human tolerance for ozone.

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ERNEST LINCOLN SPENCER**1913 - 1975**

Ernest L. Spencer, Professor of Civil Engineering and Chairman of the Department at Northeastern University, died suddenly on October 18, 1975, at the age of 62. Long active in professional societies, he was President of the Boston Society of Civil Engineers in 1970. He had sustained a severe heart attack in April, 1974, and subsequently underwent open-heart surgery for the implantation of artificial valves. His decision to undergo this serious operation made it possible for him to return to work full time from September, 1974, to the time of his death. He would not have wanted it otherwise.

Ernie was friend to literally hundreds of students, faculty, alumni, and staff, and although he gave civil engineering matters top priority, he was a kind of good will emissary to all parts of the University. A graduate of Norwood High School, he received a Bachelor of Science in Civil Engineering from Northeastern in 1936, and spent the next three years in engineering practice, as chief of survey party for E. M. Brooks, as engineering aide for the Massachusetts Department of Public Works, and as assistant engineer at Jackson & Moreland. He joined the Northeastern faculty in 1939, received an MSCE from Harvard in 1943, and followed Emil Gramstorff and Charles O. Baird as department chairman, assuming that position in 1961.

He oversaw the growth of the civil engineering department from six full time faculty members to 22, and was instrumental in developing and strengthening programs at associate, baccalaureate, and graduate degree levels. His own expertise was in concrete technology, in which he was a consultant throughout his career. He was, for four years, Chairman of the American Society for Testing and Materials, New England District, and was a member of the Prestressed Concrete Institute. A Fellow of ASCE, he was for six years Secretary-Treasurer of the Massachusetts Section. He was later Vice President, and in 1959 President of that Section. He was a member also of the American Society for Engineering Education, Tau Beta Pi, and Sigma Xi.

Ernest Lincoln Spencer was born in Norwood, Mass. on May 17, 1913, and in recent years made his home in Medfield. He was a member of the Medfield Planning Board for 12 years. Devoted to Northeastern University, he was generous in his support financially and in time and effort. He was Chairman of the Northeastern Diamond Anniversary Celebration Committee, and served on many academic groups, including the Committee on Graduate Study in Engineering and the College Tenure Committee. From 1961 to 1974 he was a member of the University Cabinet.

Students and faculty could depend on his office door being open at all times; his valued counsel was always available. Ernie's warmth, good humor, and dedication markedly influenced those who came within his large sphere of activity. Through his initiative and guidance, a chapter of Chi Epsilon, the national civil engineering honor society, was brought to Northeastern in 1965, and he was made a charter member. He continually gave his encouragement and assistance to the N. U. Student Chapter of ASCE.

Lighthearted in his approach to academic matters, he seldom underestimated the importance of problems to students and staff, and was always ready to lend his assistance and advice. Professor Spencer enjoyed maintaining communication with alumni, and visited many in cross-country trips in his beloved Saab. He took great pride in his mechanical ability, and was not above plunging into laboratory and maintenance problems to keep things in working order in the Botolph Building. In 1975 he helped initiate formation of a Visiting Committee of professional engineers to review programs and facilities in his department, in the interest of continuing improvement and upgrading of the learning experience made available to civil engineering students. He did not live to see the final report of that committee; it was dedicated to his memory. His presence will be sorely missed and long remembered by friends and associates.

Professor Spencer is survived by his wife, Doris E. (Howlett), by his son, Kennard A. of Littleton, by his mother, Mrs. Alice (Pettee) Spencer of Foxboro; by two brothers, Robert G. of Burbank, California, and Arthur L. of Boston, and by two grandchildren.

(Memoir by Robert L. Meserve)

PROCEEDINGS
BOSTON SOCIETY OF CIVIL ENGINEERS SECTION
AMERICAN SOCIETY OF CIVIL ENGINEERS

Meetings Held
TECHNICAL GROUP MEETINGS

Computer Group

November 19, 1975. Evening meeting at Ralph M. Parsons Laboratory for Water Resources, MIT, Chairman Vittands presiding. Speakers, Mr. Kenneth A. Guff and Mr. Steve Lyons, both of Stone & Webster, on management techniques in planning projects using computers. This was also an official meeting of the BSCE Section, ASCE. Secretary Kinner spoke of the two newly designed National Historical Civil Engineering Landmarks, Middlesex Canal and Lawrence Experiment Station. Attendance, 25.

Construction Group

November 5, 1975. Luncheon meeting at Red Coach Grill, Chairman Rieksts presiding. Mr. James Roop of Franki Foundation Company spoke on the slurry wall foundation for the 60 State Street office building. Attendance, 35.

January 14, 1976. Luncheon meeting at Red Coach Grill, Boston, Chairman Rieksts presiding. Speaker, Mr. Bruce Boleyn, of the company that furnished equipment for the incinerator in Saugus. Subject, operation of a plant for extracting energy from solid waste. Attendance, 45.

Environmental Group

November 12, 1975. Evening dinner meeting at the Playboy Club, Boston. Mr. Gerald Conklin of Dufresne and Henry Engineering Corporation spoke on current procedures in sewer system evaluation surveys. Attendance, 48.

Geotechnical Group

December 10, 1975. Dinner meeting at Pound Hall, Harvard, Chairman Stevens presiding. Speaker, Mr. Kalman Szalay, Assistant Chief, Geotechnical Department, Tippets-Abbott-McCarthy-Stratton. Subject - design, construction and failure of Tarbela Dam in West Pakistan. Attendance, 80.

January 21, 1976. Dinner at Pound Hall, then lecture at Pierce Hall, Harvard. Presiding, Mr. Zoino, chairman of the Group's program committee. Speaker, Mr. William L. Shannon of Shannon and Wilson, Seattle, Washington. Subject, loss prevention programs for consulting engineers. Followed by a discussion led by Messrs. Charles A. Parthum, William McTigue and Woodrow Wilson. Attendance, 75.

Hydraulics Group

November 25, 1975. Evening meeting at the Ralph M. Parsons Water Resources Laboratory at MIT, presided over by new Chairman Oscar L. Donati, the previous chairman having withdrawn. It was noted that there would be no Freeman Lecture this year, and that the Group would be presenting the Thomas R. Camp lecture series. Speakers at the November 25 meeting were Mr. Richard Reardon and Mr. Stephen Parker of the Corps of Engineers, on the Dickie-Lincoln School Lakes Hydroelectric Project, Attendance, 45.

Structural Group

December 17, 1975. Evening meeting at MIT, Chairman Hagen presiding. Speaker, Marvin Mass, consulting mechanical engineer and partner, Consentini Associates, Cambridge. Mr. Mass spoke on the influence of electrical, mechanical and plumbing requirements on design of structures. This was designated as an official BSCES meeting. Attendance, 17.

Transportation Group

December 9, 1975. Luncheon meeting at Red Coach Grill, Boston, Chairman Freedman presiding. Speaker, Mr. Francis R. Shollock, Location and Survey Engineer, Massachusetts Department of Public Works. Subject, Boston Central Artery, its history and the potential for depressing it. Attendance, 145.

January 28, 1976. Dinner meeting at SS Peter Stuyvesant, Pier 4, Boston, Chairman Freedman presiding. Speaker, Thomas P. O'Neill, III, Lieutenant-Governor of Massachusetts, on the future of transportation in the Commonwealth.

Officers of the Transportation Group for 1976-1977 were elected at this meeting and took office immediately:

Chairman, Marvin W. Miller

Vice Chairman, Robert J. McDonagh

Clerk, A. Russell Barnes, Jr.

Members Executive Committee, Leo DeMarsh, Donald Graham and Maurice Freedman.

This was designated as an official meeting of BSCES. Attendance, 70.

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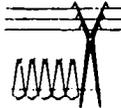
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