PROCEEDINGS

OF THE

SYMPOSIUM ON

WATER FILTRATION - THE STATE OF THE ART

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May 14, 1969

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May 16, 1969

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SYMPOSIUM ON WATER FILTRATION - THE STATE OF THE ART

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Sidney F. Dommes and Clarence L. Young, Co-Chairmen

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H. J. Ongerth, Chief
Bureau of Sanitary Engineering
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II. Morning Session

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Professor James E. Foxworthy, Ph.D.
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RECENT THEORETICAL CONCEPTS OF FILTRATION — Section B

Professor Warren J. Kaufman, Sc.D.,
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EXPERIMENTAL STUDIES RELATED TO ADVANCED CONCEPTS — Section C

Gordon G. Robeck, Chief
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Chairman (Berkeley)

William R. Seeger, General Manager and Chief Engineer
Marin Municipal Water District
Corte Madera

Chairman (Los Angeles)

Roy E. Dodson, Assistant Director of Utilities
City of San Diego

DESIGN CONSIDERATIONS
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William W. Aultman, Chairman of the Board
James M. Montgomery, Consulting Engineers
Pasadena

OPERATIONAL ASPECTS OF MODERN FILTRATION PLANTS Sections E & F

Gordon L. Laverty, Manager
Water Production and Distribution Division
East Bay Municipal Utility District
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W. L. Harris, Water Production Superintendent
Contra Costa County Water District
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PREFACE

The following material consists of reprints of papers presented at the symposium and a heavily edited record of the discussions which followed the formal presentations.

The participation of the speakers and chairmen representing water utilities, universities, consulting engineering firms, and public health authorities is gratefully acknowledged. Special thanks to the Los Angeles Department of Water & Power for hosting the symposium in Los Angeles.

A roster of participants is appended to these proceedings.

Sidney F. Dommes and
Clarence L. Young, Co-chairmen

SECTION A

Introduction by the General Chairman

H. J. Ongerth, Chief
Bureau of Sanitary Engineering
California State Department of Public Health

INTRODUCTION

H.J. ONGERTH

Water filtration is a part of a water purification process which is intended to produce a highly clarified water and, after disinfection, water which is unquestionably free from pathogenic organisms. As practiced in this country for over 50 years, water filtration has been more of an art than a science and, hence, we are now having troubles dealing with changes in the art of water treatment. In the pre-World War II period, a more-or-less standard design evolved which has been described in textbooks, particularly the 1939 edition of "Manual of Water Treatment Design". Since World War II, water filtration design has been subject to a host of new ideas.

Although a satisfactory water can be produced by the standard rapid sand filter, it has long been recognized that the process could be improved upon. In the past, we've been using empirically developed design criteria. Until recently, most design engineers and the regulatory agencies have been quite comfortable with the traditional design criteria based on long established empirical "standards" including filtration rates of 2 to 3 gallons per square foot per minute; but, things have changed. Engineers are now thinking about filtration rates that are 2 or 3 times higher than they used to be, about the use of polyelectrolytes, about multi-media beds, about changes in every one of the elements that go into making up this total unit process; recognizing the interdependence of pre-treatment and filtration, and the interdependence of design and operation. The prospect for new design concepts for improved performance is good. However, engineers have not kept up with all the possibilities of improved performance by applying knowledge presently available. This applies to the designers, to the operators, to the regulatory authorities, to every group that's involved in the process.

We hope that bringing together the academicians, researchers, designers, and water utility operators in this seminar we will develop answers to some of the problems we are now facing, and to stimulate advancement of the art of filtration.

SECTION B

RECENT THEORETICAL CONCEPTS OF FILTRATION

Professor Warren J. Kaufman, Sc.D.
Department of Civil Engineering
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RECENT THEORETICAL CONCEPTS OF FILTRATION*

Warren J. Kaufman

The problem in devising a general theory of filtration which describes the real systems encountered in water quality engineering practice is one of truly great complexity. It has not been solved in spite of the many attempts over the past 80 years. Nevertheless, we continue to filter water more or less efficiently and economically and occasionally even make a little progress toward better engineered systems. We are on the brink of such a step of progress today even though we have no comprehensive theory to describe that step. The step I have in mind is polyelectrolyte aided, high-rate, depth filtration.

Why do we need a theory? Actually we don't. Yet even in engineering, theories serve several useful purposes. They represent a kind of encapsulated knowledge. A theory allows the beginner to understand a great deal about a phenomenon with relatively little effort and to carry this information about in a small convenient package. Theories allow the advanced student - the practicing engineer - to extend the meaning of his experiences; to anticipate accurately and economically new solutions to new problems. In other words, good theories, well understood, allow a little experience to go a long ways.

What kind of theory would we like to have on the phenomenon of filtration? First, we would like an explicit rational formula describing the separation of particulate matter from suspension during its transport through porous

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*Presented at 4th Annual Symposium, Water Filtration - the State of the Art, Bureau of Sanitary Engineering, California State Department of Public Health, 14 and 16 May 1969.

media. This formula should ideally describe in time and space the separation of the suspended matter and do this as a function of the measurable physical and chemical properties of the suspension and the filter medium and with due regard for the hydrodynamic environment in which the process occurs. Second, we would like a formula that describes the system energy loss in time and space and with appropriate and known dependence on our first equation. Finally, we would like the most impossible of all, an equation giving us the least-cost specifications of a filter that will perform in accordance with the standards proscribed by our health authority and in which we only need plug-in the appropriate water quality parameters and the material and other costs.

Present theories of filtration fall woefully short of meeting any of our three needs for specific formulas and it is unlikely that this situation will change greatly in the near future. Mintz (1) stated the consensus of a majority of the practically oriented theoreticians in 1966:

> "It is apparent that an attempt to work out an exact mathematical description, with theoretical constants, of the filtration process is bound to fail. Obviously, it will be always necessary to determine the parameters of the process experimentally. The task of the theory is to provide a rational experimental procedure and a rational method of working out the experimental data so as to get the results required for engineering practice."

Similarly Camp (2) in 1964 stated:

> "Pilot plant studies similar to those described herein are recommended, prior to the design of water treatment plants, as a means of selecting the coagulating chemicals, coagulant and filter aides, and filter media; and of determining filter rates, wash rates, and size, number, and dimensions of units."

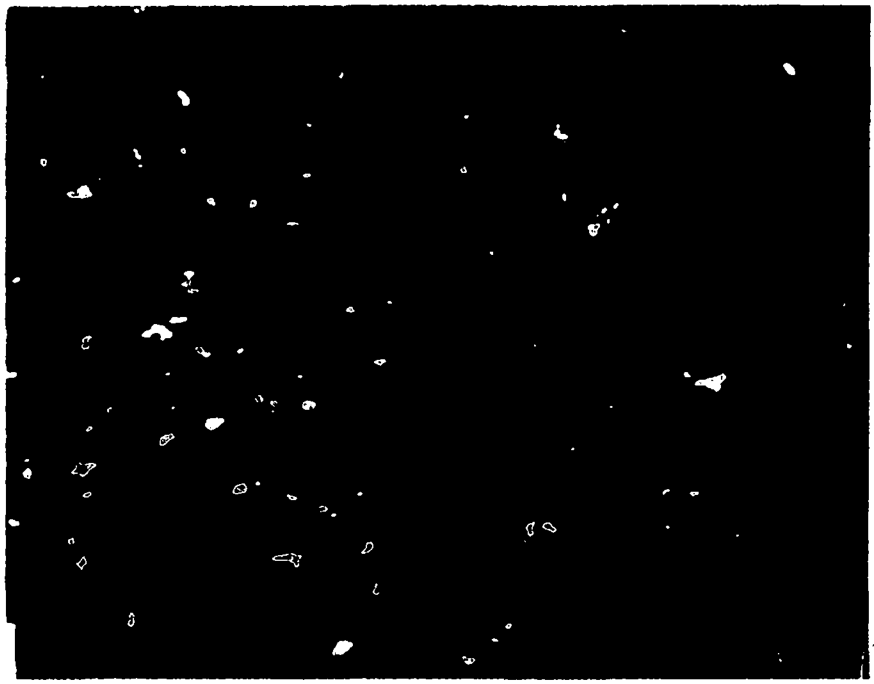
In the conventional water filtration plant the first step has traditionally been to remove as much of the suspended material as possible by coagulation-flocculation-sedimentation and to follow this operation by rapid sand filtration. It was early recognized that a rapid sand filter was not an efficient separator of naturally occuring uncoagulated particulates and that chemical coagulation and sedimentation served to not only reduce the turbidity entering the filter, but to also condition the particulates passing the sedimentation basin such that they were readily removed by the filter. However, in flocculating and settling waters of low turbidity, the *volume* of the suspension leaving the settling basin and entering the filter is often very much greater than that in the raw water. This voluminous material is very efficiently removed, forming a mat at or near the surface of the filter. Unfortunately this process results in a rapid build-up of headloss in the upper portion of the sand bed. The traditional hydraulically graded filter bed through which the flow is downward, from the fine media to the coarse, unfortunately contributes further to the buildup of headloss as clogging occurs in the fine media.

Recent advances in filtration practice have aimed at reducing the capital investment in plant facilities through reducing or eliminating flocculation and sedimentation and by increasing filtration rates and the volume of water filtered per cycle. The principal features of such plants has been coarse-to-fine filtration, either upflow or downflow through media of increasing density, and by the application of polymers that serve to control floc or particle penetration into the filter bed. Ideally, the bed should be designed and operated to function throughout its depth such that the breakthrough of turbidity and the attainment of the limiting headloss are reached simultaneously and after a sufficient time of filter operation as to comprise a least-cost system.

Filter designers are faced with two rather different situations. With waters of high or variable turbidities it is likely that the traditional practice of alum coagulation, flocculation, and sedimentation will continue to be employed with the objective of reducing the particulate load reaching the filters. In this situation the filter influent will contain large alum floc-turbidity aggregates as well as small coagules of hydrolyzed aluminum and raw water particulates that failed to be flocculated and settled.

Thus the filter influent may contain particles ranging in size from microns to millimeters. On the other hand, waters of low turbidity may be filtered directly providing the suspension is properly conditioned with metal coagulants, polymers, or both. Here the particulates are not likely to be flocs in the usual sense, but rather micron-size coagules conditioned to attach to the filter media and to each other. In both of these situations, depth filtration in coarse-to-fine filter beds is to be prefered, but the treatment of the filter influent may be somewhat different.

Figures 1 and 2 serve to illustrate the extremes in particle characteristics as well as show the complexity of the problem of those who seek simple and explicit formulae to describe filter performance. Figure 1 shows photomicrographs of kaolinite-alum flocs formed after 2 and 20 minutes of flocculation at a root-mean-velocity gradient (G) of 60 sec^{-1}. Even after 20 minutes about 20 percent of the kaolinite remained unincorporated in the floc and thus largely non-settleable (and not visible in the photograph). Many of the flocs are nearly one millimeter in size and would tend to be collected near the surface of even the coarser filter media used in practice. It should be added that such flocs do not necessarily settle well and may be expected in filter influents. Figure 2 attempts to show a typical anthracite coal filter media and a one micron kaolinite platelet. It is evident that the pore openings are from two to three orders of magnitude greater than the suspension particles. Bacteria are of similar size and even algae are an order of magnitude or more smaller than the pore openings. Moreover, it is evident that neither the filter media nor the suspension can be properly described as spherical in shape. This aspect of the problem is emphasized because much of the current theory and experimentation is concerned with such idealized geometries.



T = 2 MIN

T = 20 MIN

FIG 1. ALUM-KAOLINITE FLOCS
G = 60 SEC^{-1}

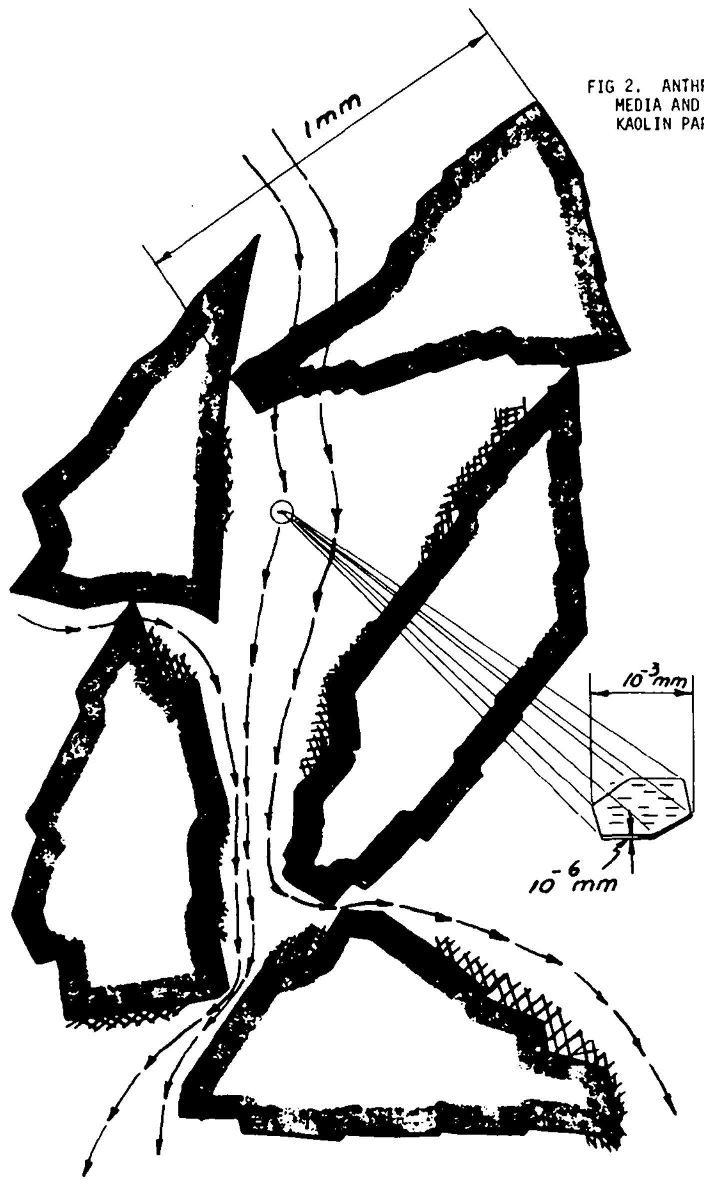


FIG 2. ANTHRACITE MEDIA AND A KAOLIN PARTICLE

A - SEDIMENTATION
B - INTERCEPTION
C - DIFFUSION

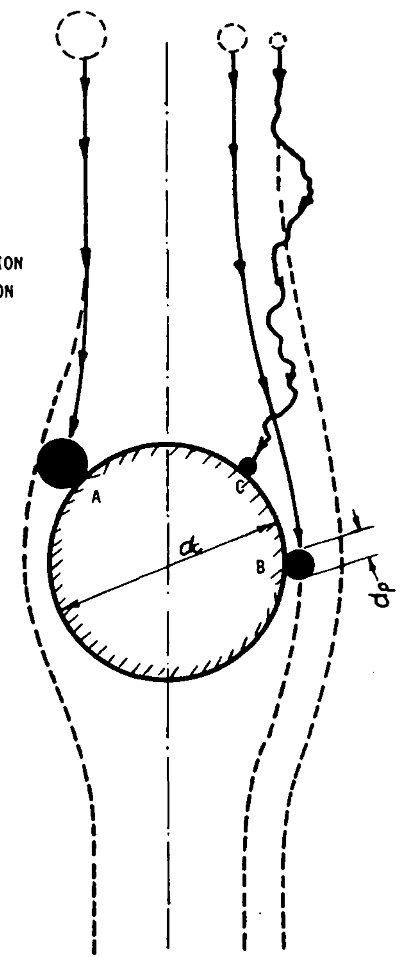


FIG 3. TRANSPORT MECHANISMS

MECHANISMS OF FILTRATION

The mechanism of filtration has intrigued sanitary engineers and scientists for a great many years. The fact that one micron particles can be separated in 500 micron pores is of itself something of a conundrum and one that seems to preclude simple straining as a principal mechanism. It is convenient and theoretically sound to consider the filtration process as two separate but sequential phenomena; 1) transport and 2) attachment. Particles must first move relatively great distances to reach the surface of the filter media and, once having reached this destination, they must become attached with sufficient energy to resist the shearing force of the moving liquid. The separation of transport from attachment is reasonable because the ranges of the forces associated with the latter process are no more than a few Angstroms (1 Å = 10^{-4} microns).

Transport

Particle transport mechanisms have been given considerable attention by the filtration theoreticians. As noted earlier, while the separation of millimeter diameter flocs can be handily explained by straining (especially if we consider the region near the points of contact of sand grains), the transport of the smaller fraction is more difficult to account for. Ives and Gregory (3) list gravity sedimentation, hydrodynamic forces (as characterized by the Reynolds Number), interception, and diffusion as the dominant transport mechanisms. They dismiss straining and inertial impaction as making negligible contributions.

Yao and O'Melia (4) have considered the relative contributions of sedimentation, interception, and diffusion to transport for spherical particles and spherical media (or "collectors"). These three mechanisms are illustrated by Figure 3. These authors conclude that a particle size exists at which filtration efficiency is a minimum and that for particles

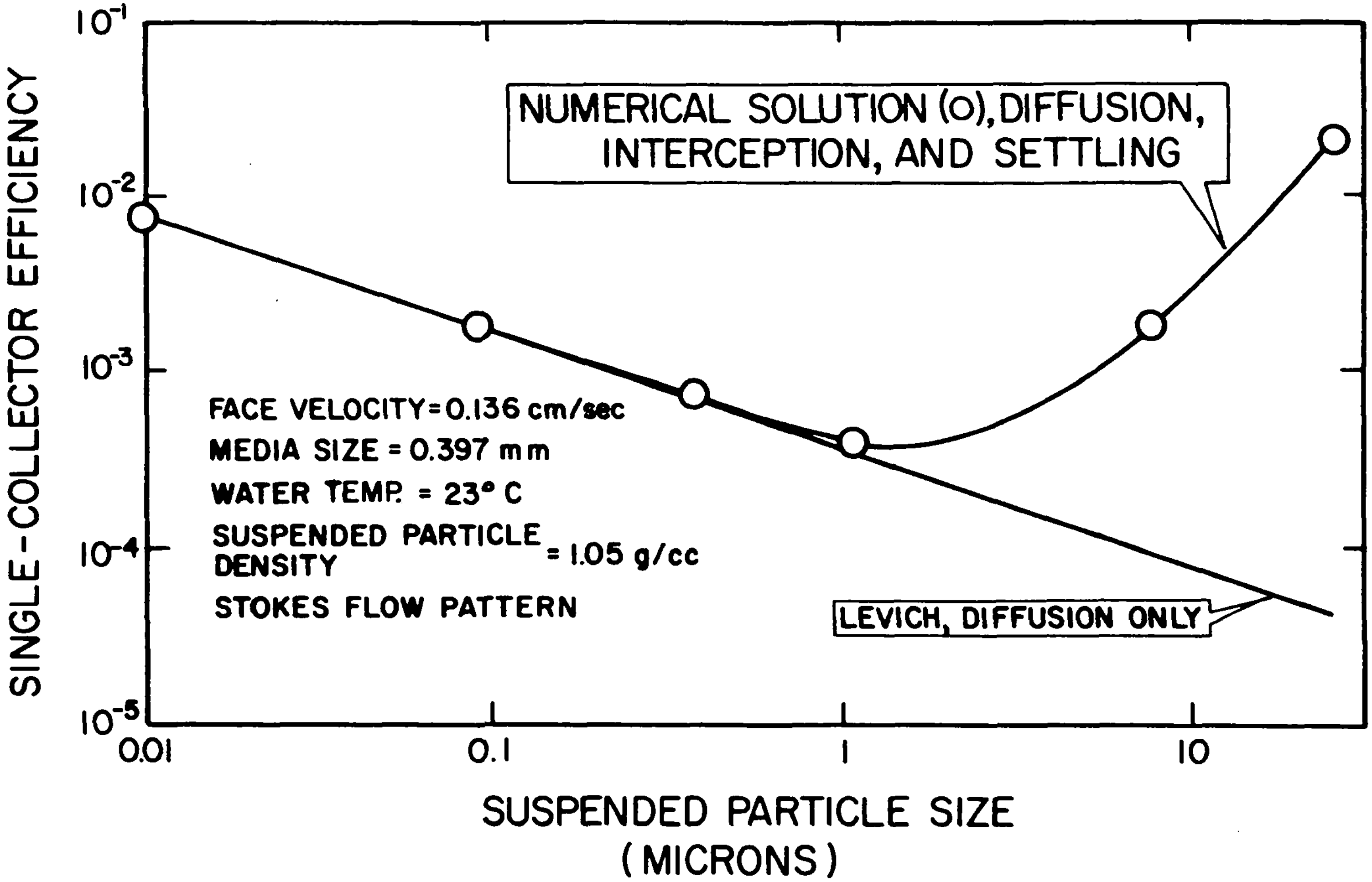


FIG. 4 THEORETICAL PARTICLE TRANSPORT EFFICIENCY IN A POROUS MEDIUM (YAO AND O'MELIA [4])

of a density of 1.05 g/cc in water this is around one to three microns. For smaller particles diffusion is the dominant mechanism, whereas for larger particles, sedimentation and interception control. This relationship is illustrated by Figure 4 taken from the Yao and O'Melia paper. The "single-collector efficiency," a term developed in aerosol filtration, represents the fraction of particulate matter removed from suspension in passing a single sand grain or collector. The conditions of this example computation correspond approximately to those existing in conventional rapid sand filters.

In an earlier and less rigorous treatment of the mass transport process, O'Melia and Stumm (5) developed a similar relationship which included diffusion and interception, but not sedimentation. This formulation of single-collector efficiency (η) and its relationship to the impediment modulus (or filter coefficient) (λ) are given by the following equations:

$$\eta = \left(\frac{1.34\ k^{\frac{2}{3}}\ T^{\frac{2}{3}}\ \rho^{\frac{1}{6}}}{\mu^{\frac{5}{6}}}\right)\left(\frac{1}{d_p^{\frac{2}{3}}\ d_c^{\frac{1}{2}}\ v^{\frac{1}{2}}}\right) + \left(\frac{3\ \rho^{\frac{1}{2}}}{\mu^{\frac{1}{2}}}\right)\left(\frac{d_p^{\ 2}\ v^{\frac{1}{2}}}{d_c^{\ \frac{3}{2}}}\right) \qquad (1)$$

$$\lambda = \frac{4\ (1 - f)}{\pi\ d_c} \cdot \eta \qquad (2)$$

in which k is the Boltzman constant and T, ρ, and μ characterize the liquid (absolute temperature, density, and viscosity) and d_p is the particle diameter, d_c is the collector diameter, v is the velocity of fluid approach, and f is the media porosity. The first term in Equation 1 expresses transport by diffusion while the second accounts for interception, both for spherical particles. Both η and λ have a minimum when these equations are differentiated with respect to particle diameter. In more simplified form, Equations 1 and 2 predict the following dependence of λ on d_c and v,

$$\lambda \propto d_p^{-\frac{2}{3}}\ d_c^{-\frac{3}{2}}\ v^{-\frac{1}{2}} + d_p^{\ 2}\ d_c^{-\frac{5}{2}}\ v^{\frac{1}{2}} \qquad (3)$$

in which the first term describes the influence of diffusion and the second pertains to interception.

Ives and Sholji (6) studied the filtration of 1.3 micron polyvinyl-chloride spheres through sand and concluded that the filter coefficient was not constant, but that initially (i.e., before appreciable deposition had occurred) it conformed to the relationship,

$$\lambda_o \propto \mu^{-2} d_c^{-1} v^{-1} \quad (4)$$

a quite different relationship than that theoretically developed by O'Melia and Stumm. In summarizing the work of others, Ives and Sholji reported the following relationships,

$$\lambda \propto d_c^{-1.7} v^{-0.7} \quad (5)$$

$$\lambda \propto \mu^{\frac{1}{2}} d_c^{-2} v^{-1} \quad (6)$$

and several others and concluded that filtration efficiency was inversely proportional to velocity (or filtration rate), media diameter, and viscosity and that in most instances performance was inversely dependent on grain size to a power greater than unity. These authors correctly conclude that performance depends on the suspension characteristics. They might well have added that the theory provides little specific guidance as to the effect of the two most important design parameters on filter performance, media diameter and filtration rate, except that the larger they are the poorer filtration. These results should also lead to the conclusion that the transport step may in theory be separated from attachment, but that filter performance (which Ives and Sholji and many others have measured) is the sum of the two steps.

Attachment

As with the transport step, the mechanisms proposed to account for the attachment (or adhesion) of the suspended particles to the filter media are also a matter of considerable conjecture. It is generally agreed that clay suspensions applied to normal filter media at rapid sand filtration rates pass through to the effluent with only partial removal, often less than 20 percent. Yet these same suspensions, when conditioned with dosages of Al^{+++} or Fe^{+++} in concentrations sufficient or less than necessary to form flocs, are almost completely removed. It has also been observed that a period of "ripening" is often required and that removal improves over the first few minutes of operation (and sometimes longer) and then reaches a plateau of good performance which may last for many hours.

Attachment may be attributed to two categories of phenomena, 1) the interplay of the electrostatic and van der Waal forces in the so-called "double layer model"* and 2) the chemical bonding of the particle to the sand surface by an intermediate material - the "bridging model." Here it is well to be reminded that the same two concepts are applied to explain the coagulation of liquid suspensions in which fixed surface collectors are absent. It is suggested that filtration is perhaps only a special case of flocculation in which some of the particles are fixed (i.e., those that become attached to the sand) and some are in suspension. There is considerable experimental evidence and a good foundation in theory to support the application of *both* theories concurrently.

Figures 5A, B, and C serve to illustrate the double layer model. The media surface is assumed to be negative, while several states of the clay particle are considered. In Case I the clay platelet is depicted (in Fig. 5A) as untreated and thus negative and opposed in closely approaching the media surface by a potential barrier. However, considering the planar

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*"Double layer" refers to the ions attached to a surface which give it a primary potential and the counter ions of opposite charge, both attached and in solution, which result in the net charge or zeta potential, the latter being the potential at the edge of the attached portion of the counter ions.

FIG 5A. ELECTROSTATIC FORCES
CASE I: WITHOUT METAL COAGULANT

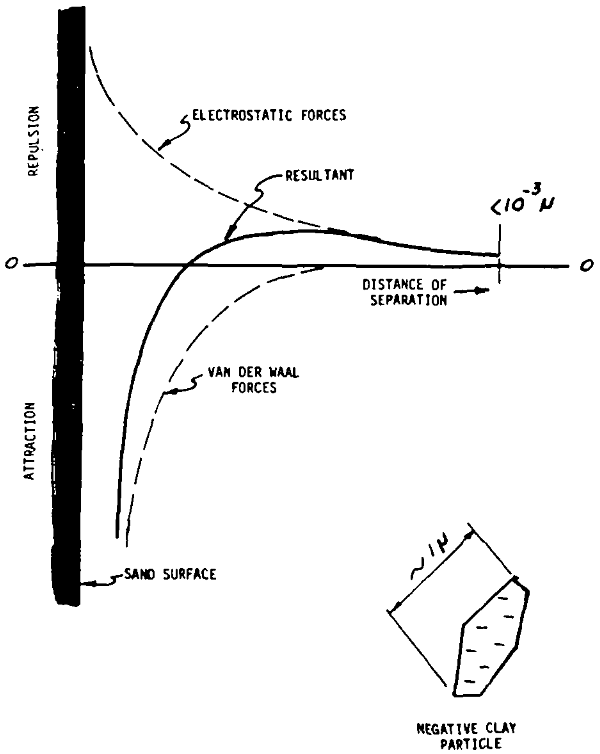


FIG 5B. ELECTROSTATIC FORCES
CASE II: EXCESSIVE METAL COAGULANT REVERSES CHARGE ON CLAY PARTICLE

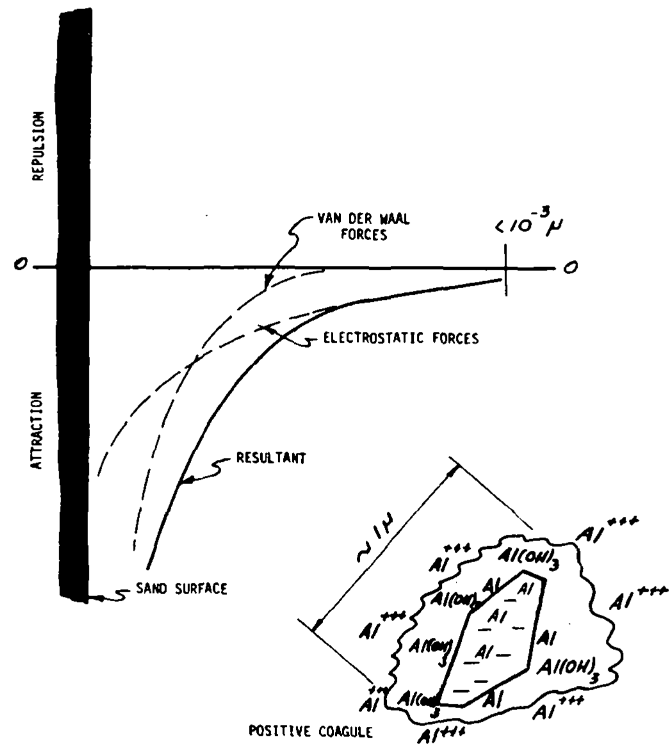
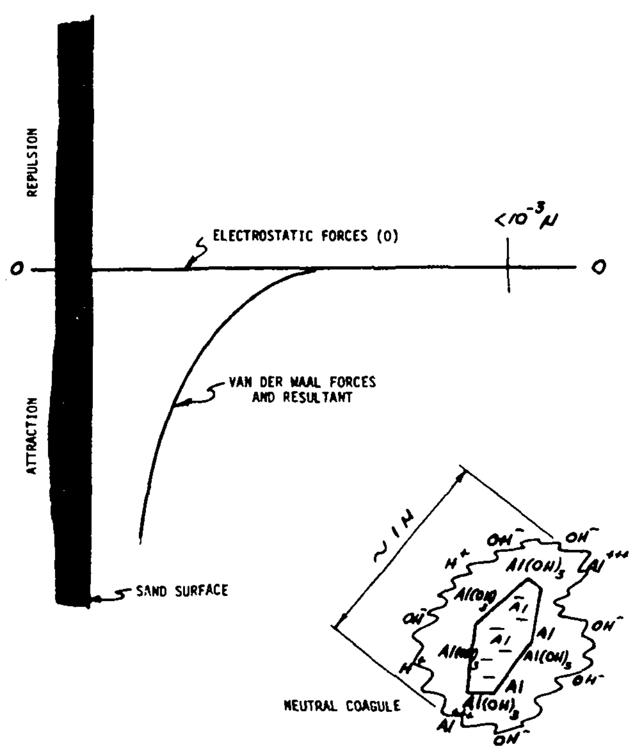


FIG 5C. ELECTROSTATIC FORCES
CASE III: SUSPENSION AT THE ISOELECTRIC POINT



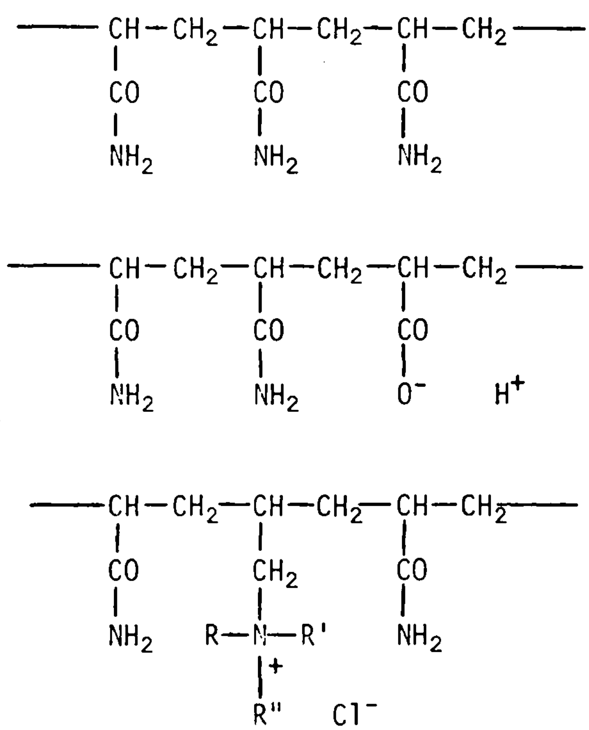
shape of the clay and the fact that some clays (kaolinite for example) may have both positive and negative regions at low pH, it is not unexpected that some penetration of the barrier will occur and some of the particles will be separated from suspension. Note especially the small range of the barrier compared with the size of the particle.

In Case II the clay platelet is depicted as a coagule, i.e., a micron-size floc in which aluminum hydrolysis products are incorporated such that it has a net positive charge. Here the barrier has been replaced by an attracting potential comprized of the additive effects of the van der Waal and electrostatic forces. Transport of the coagule close to the media results in attachment. Filtration is initially highly efficient, but as the surface of the media becomes positive, performance may become less satisfactory. The analogous situation in coagulation is charge reversal or peptization. This state may be limited to soft waters and is less likely to exist in the presence, for example, of concentrations of sulfate ions of 25 mg/l or more.

Case III depicts a third condition in which the particle is at or near its isoelectric point, i.e., neutral. Here the potential barrier has disappeared and contact results in attachment and efficient filtration. Unfortunately, when viewed in practical terms, even this condition may be unsatisfactory as it may result in excessive particle deposition in the upper layers of the filter and a rapid increase in headloss. Thus a somewhat less "sticky" coagule, perhaps one having a slight negative charge, may be the ideal situation. Larger filter media and greater approach velocities also contribute to less efficient filtration (cf., equations 3, 4, 5, and 6), but perhaps better overall performance because of a more uniform in-depth distribution of the deposit.

The bridging model supported by O'Melia and Stumm (5), as proposed earlier by LaMer and Healy (7), postulates the chemical bonding of the coagule to the media surface where the bonds may be of several types (ionic, hydrogen, coordinate as well as van der Waal). The organic polyelectrolytes are excellent examples of bridging materials which increase filtration and flocculation efficiencies by aiding attachment by both electrostatic (i.e., double layer) and chemical bonding. Here, with varying degrees of effectiveness, cationic, non-ionic, and anionic polymers may affect attachment. However, there is considerable experimental evidence to show that the cationic polymers are superior coagulants for clay and silica suspensions in comparison to the other two. The structures of the three types of polymers are illustrated by Figure 6 which shows the polyacrylamide polymer in the pure form, hydrolyzed with alkali to form an anionic polymer, and co-polymerized with quaternary amines to the cationic form.

FIG 6. POLYACRYLAMIDE FLOCCULANTS



polyacrylamide

$COOH^{+}$ substituted anionic chain

amine substituted cationic chain

Figure 7 illustrates the effect on the flocculation-sedimentation of a silica suspension of varying dosages of cationic polyacrylamide (8). An optimum dose brings about rapid flocculation and good settling, whereas an excessive polymer concentration may redisperse the suspension (giving it a positive charge) and thus retard settling. It is readily seen that filtration efficiency may be similarly influenced by varying dosages of polymer. This situation is illustrated in Figures 8A and 8B. "Optimum" coverage leaves open sites on both the media surface and on the clay particles such that particle--particle and particle--media attachments may occur. Excessive polymer coverage reduces both types of attachment and results in the suspension penetrating deep into the filter. Here it should be pointed out that the suspension particles and media are believed to have specific sites on which polymer sorption may occur.

Polymers may be used in combination with aluminum and iron salts as aids to coagulation and filtration. For example, during studies of orthokinetic flocculation at the Sanitary Engineering Research Laboratory (U.C. Berkeley) the addition of Dow C-31 (a cationic polymer) to an alum-kaolinite suspension was shown to both increase the rate of flocculation and the strength of the resulting floc. Similarly, it is recognized that adding a polyelectrolyte to the effluent of sedimentation basins will improve the efficiency of coarse-to-fine filters. These applications can in a general way be explained by the foregoing models, especially if the reader accepts both double layer and bridging concepts.

When filtration is conceived as a combination of transport and attachment it is easily appreciated why even a fine filter media operated at a low loading may perform poorly while a coarse media under heavy loading may be very efficient. It is also evident why various investigators have obtained such a diversity of functional relationships between filter efficiency (λ) and media diameter (d_c) and approach velocity (v). If it were necessary to

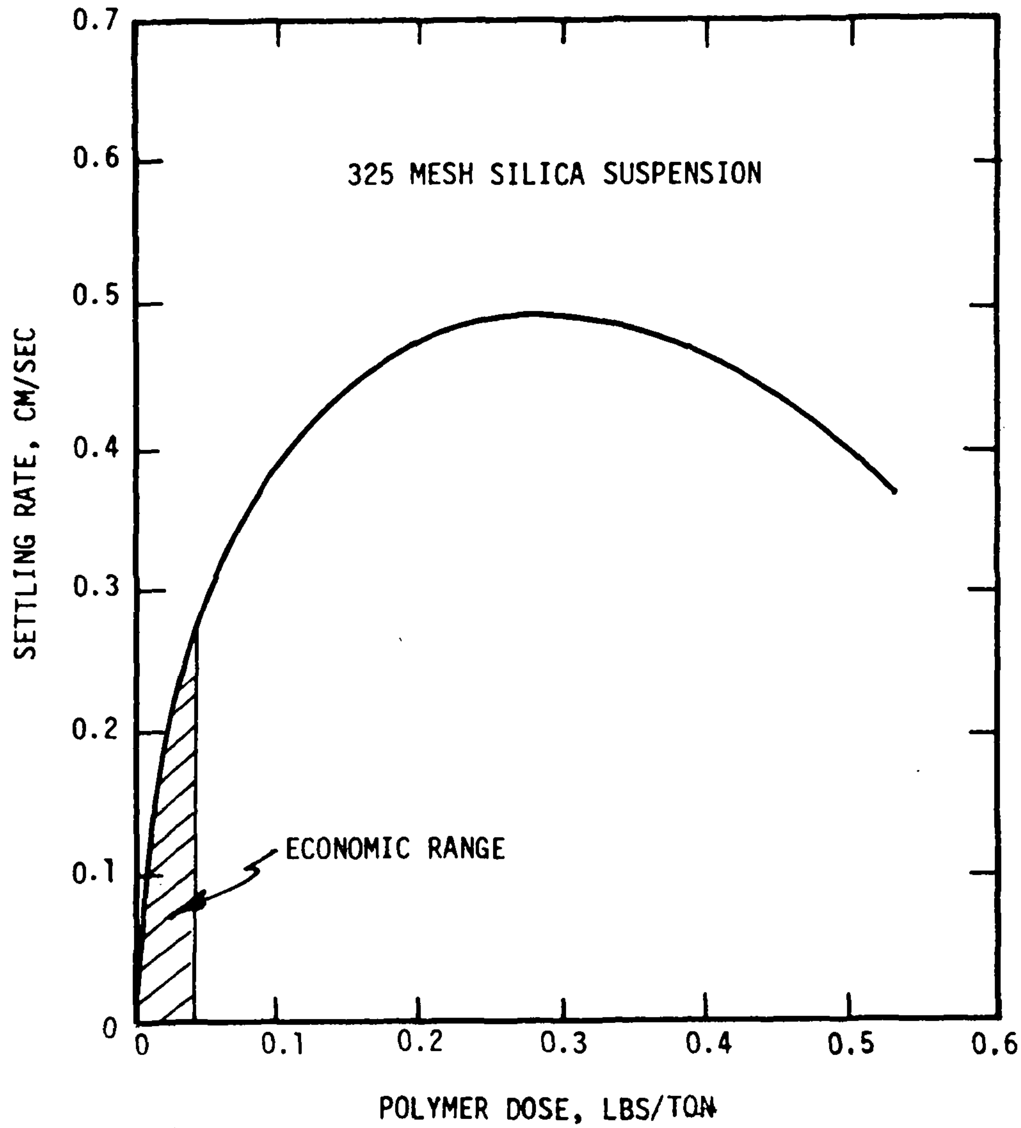


FIG 7. EFFECT OF POLYACRYLAMIDE DOSE ON FLOCCULATION AND SEDIMENTATION

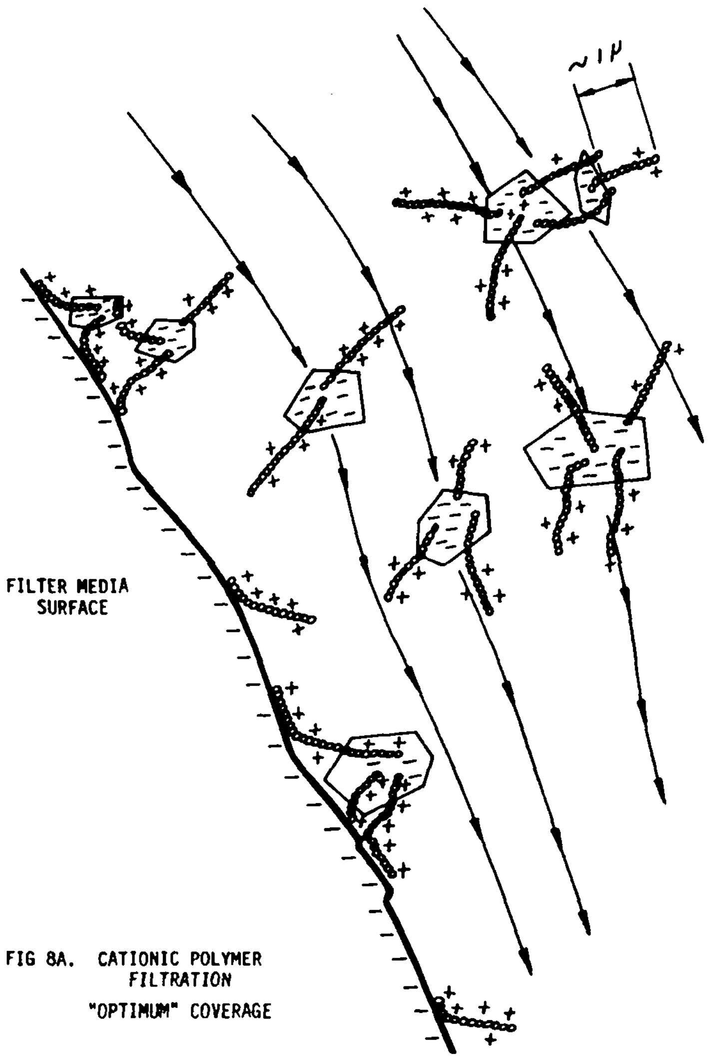


FIG 8A. CATIONIC POLYMER
FILTRATION
"OPTIMUM" COVERAGE

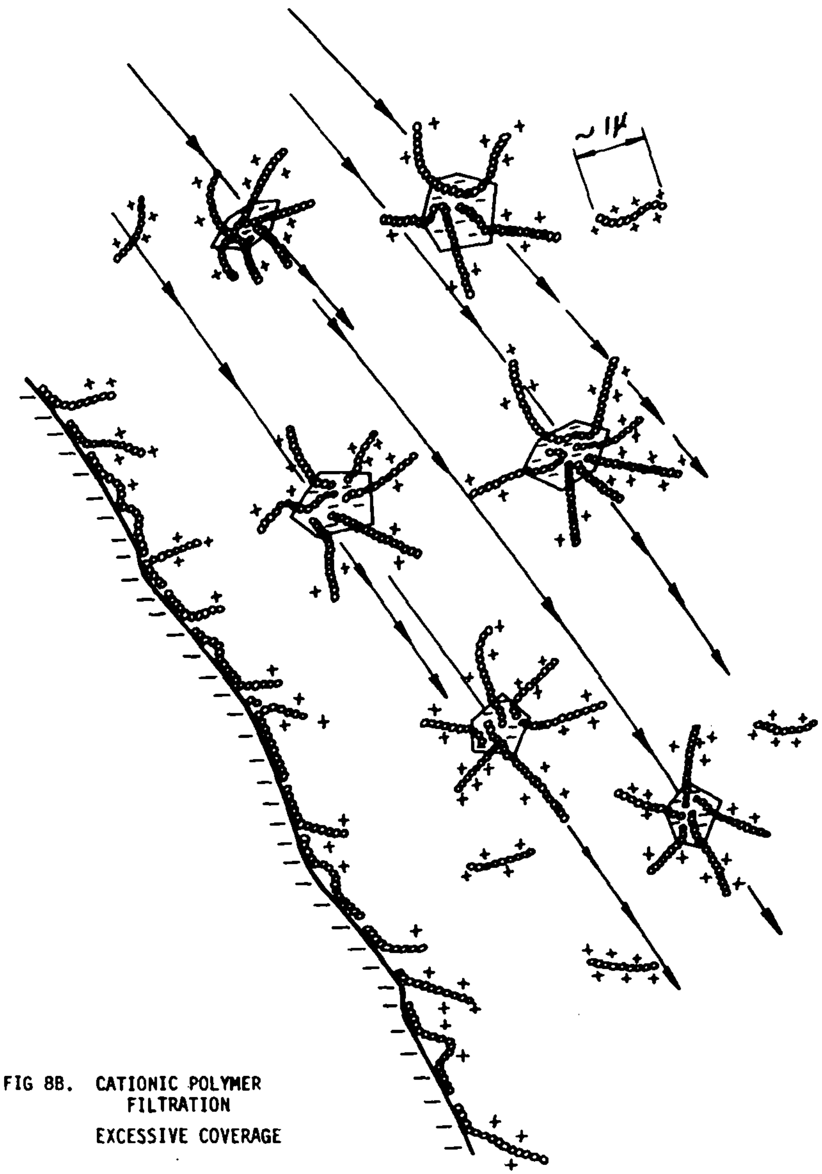


FIG 8B. CATIONIC POLYMER
FILTRATION
EXCESSIVE COVERAGE

make a choice regarding the relative importance of transport versus attachment, it is believed that the latter would rank first. On the other hand, a poorly designed filter does not lead to the best performing filtration system, even when great care is exercised in operation.

FILTER PERFORMANCE FORMULATIONS

No discussion of filtration theory would be complete without consideration of the formulae describing performance of the system as a whole; the consequences of both transport and attachment. These two processes must obviously reduce the concentration of the suspension passing through the filter pores and increase the resistance of the filter to the passage of water. Iwasaki (9) proposed a simple rate equation and a material balance expression in 1937 and these have served as the point of departure of many investigators.

$$\frac{\partial C}{\partial L} = -\lambda \ C \tag{7}$$

$$\frac{\partial \sigma}{\partial t} = -\frac{v}{(1 - f_\sigma)} \frac{\partial C}{\partial L} \tag{8}$$

Equation 7 expresses the rate of suspension removal as a first order function of the suspension concentration, C, with a constant filter efficiency, λ. The logic of this expression is evident for a uniform media and a homogeneous suspension - the probability that a particle will reach the media surface in traveling a distance ∂L is proportional to the number of particles present. Iwasaki recognized that λ was not constant and introduced an expression in which it varied with time. Equation 8 relates the rate of build-up of the deposit, $\partial\sigma/\partial t$, to its rate of removal from suspension, $\partial C/\partial L$. σ is the specific deposit expressed as volume of deposit per unit volume of filter. v is the approach velocity (e.g., gpm/sq ft) and C is the suspension concentration expressed in volumetric units. The deposit porosity, f_σ, must be introduced to account for the void spaces within the deposit. If C is expressed in mass units the density of the suspended particles, ρ_p, must also be introduced. The utility of these expressions is reduced by the fact that λ is not constant with time and space and f_σ and ρ_p are both variable and quite uncertain. It is safe to state that neither of these quantities are determinable *a priori* from knowledge of the filter media or the suspension.

Recognizing that λ was not constant, Ives and his co-workers (6), (10) have introduced the expression,

$$\lambda = \lambda_\circ + c\,\sigma - \frac{\phi\,\sigma^2}{f_\circ - \sigma} \tag{9}$$

in which $\lambda_\circ$ and $f_\circ$ are the initial filtration coefficient and media porosity, respectively. Early in a filter run (i.e., during the ripening period) λ increases with time and deposition, σ, but as deposition increases and decreases the porosity, λ begins to diminish and suspension breakthrough occurs. Equation 9 was based on a single-size media and a homogeneous suspension of relatively dense particles very much smaller than the pore dimensions of the media. Ives and Sholji (6) have experimentally tested Equations 7 through 9 with a suspension of 1.3 micron polyvinylchloride spheres passing through uniform sands of several sizes. For the PVC suspension the authors evaluated the coefficients of Equation 9, and reported them as inversely proportional to v, d_c, and μ. However, as these coefficients are admittedly dependent on the suspension characteristics, serious doubt exists regarding the practical value of these formulations in evaluating real suspensions and graded media.

Fox and Cleasby attempted to verify Ives' filter coefficient equation with a hydrous ferric oxide suspension and concluded that it was not applicable. They did conclude that the linear portion of Equation 9 ($\lambda = \lambda_\circ + c\sigma$) was valid during the initial period. These authors contend that while the Ives' equation was applicable to algae, it cannot be extended to ferric oxide floc and to the circumstances generally found in U.S. water filtration practice.

Headloss

The increase of headloss during filtration is most commonly given by the expression,

$$\frac{\partial h}{\partial L} = \left(\frac{\partial h}{\partial L}\right)_\circ + k\,\sigma \qquad (10)$$

in which $(\partial h/\partial L)_\circ$ represents the initial gradient and σ is the specific deposit. However, as σ varies with depth and time and cannot be determined directly, the relationship is of little practical interest. The Kozeny-Fair-Hatch equation has also been employed to relate the hydraulic gradient ($i = \partial h/\partial L$) to porosity,

$$i = \frac{j\,s^2\,\nu}{g}\,\frac{(1-f)^2}{f^3}\,\frac{v}{d_c^{\,2}} \qquad (11A)$$

or

$$i = i_\circ\,\frac{K\,(1-f)^2}{f^3} = i_\circ\,\frac{K\,[1-(f_\circ-\sigma)]^2}{(f_\circ-\sigma)^3} \qquad (11B)$$

in which js^2 expresses the shape of the media, ν is the kinematic viscosity, $f_\circ$ the clean sand porosity, and d_c is the media diameter. Porosity, f, includes σ which cannot be separately measured and, as a further complication, K undoubtedly varies as deposition occurs and changes the shape of the pore spaces.

Mintz (1) has proposed a somewhat different relationship between hydraulic gradient and clogging;

$$i = i_\circ\left(\frac{f_\circ}{f}\right)^3\left(\frac{\omega}{\omega_\circ}\right)^2 \qquad (12)$$

in which ω is the specific surface of the media. By replacing f with $f_\circ - \sigma$ and ω by $6/d_c$ and noting that the particle diameter must change with deposition by a function of σ, we may write

$$i = i_\circ\left(\frac{f_\circ}{f_\circ-\sigma}\right)^3\left(\frac{d_\circ}{d}\right)^2 = i_\circ\left(\frac{f_\circ}{f_\circ-\sigma}\right)^3\left(\frac{1-f_\circ}{\sigma+1-f_\circ}\right)^{\frac{2}{3}} \qquad (13)$$

Here the deposit has been assumed to be a uniform sheath on each sand grain, clearly an over simplification. Again i is seen to be a function of the indeterminant specific deposit term. Equations 11A-B, 12, and 13 can be used to show the advantages of uniform deposition throughout the filter column as under these circumstances i is a minimum (for uniform media).

Time of Filter Run

The buildup of the headloss through a filter to the limiting value (with constant filter rate operation) or the penetration of the limiting turbidity concentration will cause a filter run to be terminated. Ideally the time of turbidity breakthrough, t_1, and the time to reach the headloss limit, t_2, should coincide and correspond to some least-cost-of-product value. If all of the coefficients were known these quantities could be calculated from the Ives' equations. As this is not the case (nor likely to become so) most investigators have resorted to empirical correlations such as that of Mintz (1).

$$t_1 = k \frac{\sigma_\ell \rho_\sigma}{vC_o} \left[L - \frac{F(C_\ell)}{\lambda_o}\right] \tag{14}$$

where $\lambda_o \propto v^{-0.7} d_c^{-1.7}$, σ_ℓ is the limiting value of the specific deposit, $F(C_\ell)$ is a dimensionless parameter related to the limiting effluent turbidity, ρ_σ is the deposit density, and the remaining terms are as previously defined.

The relationship of t_1 and t_2 to the character of the suspension is illustrated by Figure 9 in which effluent turbidities and headlosses are shown for a polyelectrolyte (Catfloc) treated clay suspension influent to a dual media filter. With a Catfloc dose of 0.5 mg/l the suspension has a relatively low filter coefficient, headloss buildup (H_L) is small, and turbidity removal is less than desired. At a dose of 0.75 mg/l removal is improved, but headloss increases more rapidly with the course of the run. At 1.0 mg/l performance is even better, but headloss becomes excessive in 15 hours. Recalling Figure 8A, presumably 1.0 mg/l of Catfloc is close to

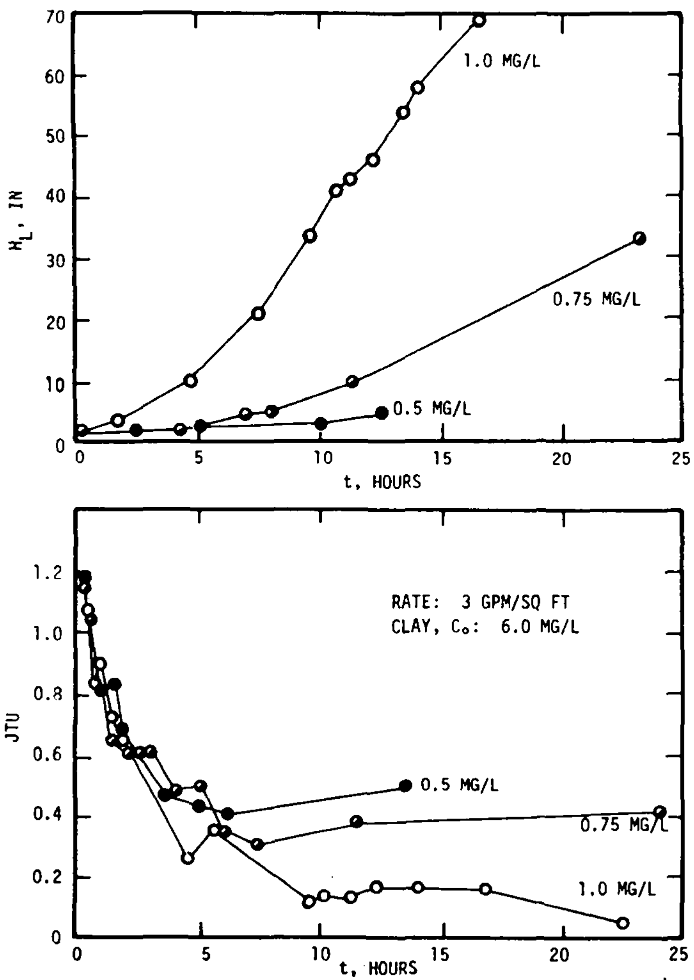


FIG 9. EFFECT OF CATFLOC ON FILTER PERFORMANCE*

ANTHRAFILT: 16 IN, ES - 2.50 MM, UC - 1.32
SAND: 6 IN, ES - 1.10 MM, UC - 1.23

*FLOC STRENGTH AND FILTERABILITY, REPORT BY ENGINEERING-SCIENCE, INC. TO FWPCA, OCTOBER 1968.

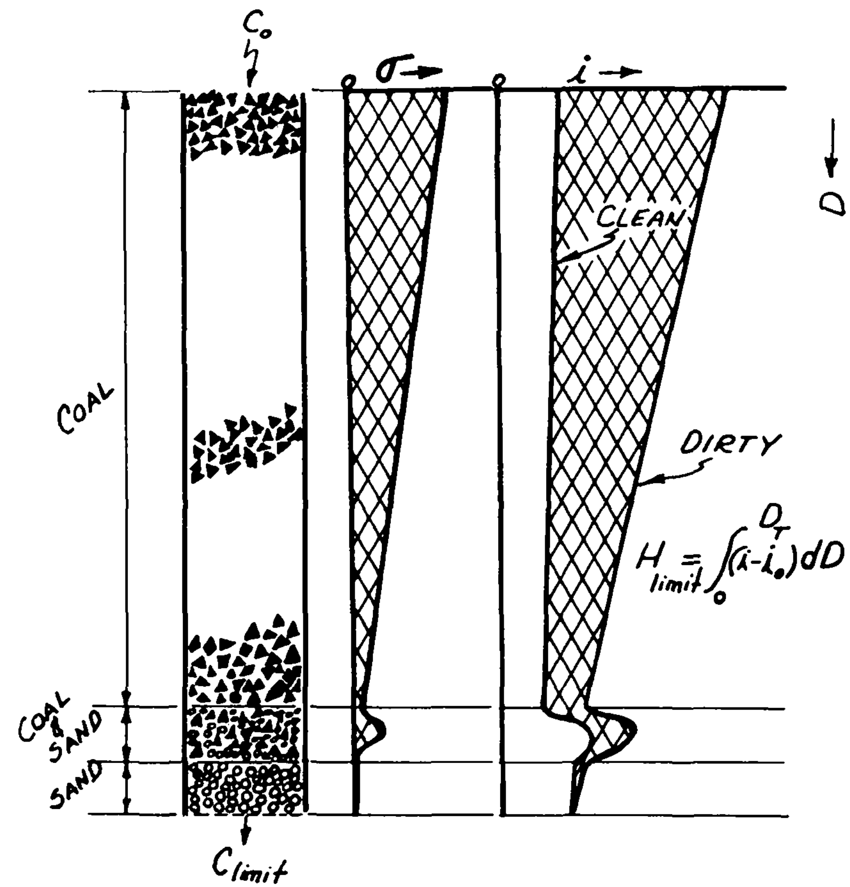


FIG 10. BALANCED FILTER OPERATION $t_1 = t_2$

the "optimum" polymer coverage in terms of removal efficiency, but it is excessive from the standpoint of balanced operation. It is somewhat curious that the three polymer dosages display similar filter ripening characteristics. It is evident that the filter performed poorly until some particle attachment had occurred. Precoating the media with the polyelectrolyte would probably have reduced the early period of poor performance.

POLYELECTROLYTE AIDED COARSE-TO-FINE FILTRATION

It is becoming evident from the literature that the ideal filtration system is one in which the suspension is conditioned to deposit on the media in such a manner that the limiting headloss and effluent turbidity are simultaneously attained and that this is best achieved when the flow is from the coarse to fine media. Clearly, this is a system quite different from that of traditional practice. The various previously cited formulas suggest the rational basis of such a concept, but they fall far short of defining it. For example, Equations 3, 4, 5, and 6 indicate filter efficiency to be inversely proportional to some function of d_c, the media diameter, while Equation 11A indicates that headloss is inversely proportional to d_c^2. It can be shown that for the hydraulic gradient to remain constant in a graded media, the specific deposit must bear a direct relationship to the media size, i.e., more deposition in the larger pores.* Although the larger media particles are less efficient collectors, by collecting from the more concentrated stream they accumulate the greatest specific deposit. By maintaining a constant hydraulic gradient through the depth of the bed the overall headloss is

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*For $i_1 = i_2$; at two points in the filter.

$$\left(\frac{d_2}{d_1}\right)^3 = \frac{f_\circ - \sigma_1}{f_\circ - \sigma_2} \cdot \frac{1 - f_\circ + \sigma_2}{1 - f_\circ + \sigma_1}; \quad d_2 > d_1, \text{ then } \sigma_2 > \sigma_1$$

minimized and higher flow rates are possible. Moreover, by having the finer and more efficient sand at the bottom of the bed, more effective removal of the small residual turbidity nearing the bottom may be expected. These arguments make a general case for coarse-to-fine filtration but they do not tell us the optimum grading. However, here we must again be reminded that filtration efficiency is also very much dependent on the suspension and its characteristics may be conditioned by polyelectrolyte coagulation.

There are as yet several technical questions to be resolved before single media coarse-to-fine upflow filtration will be accepted in the U.S. On the other hand, by using filters of several media, each of a different density, it is possible to approximate coarse-to-fine filtration in a downflow system. Figure 10 illustrates a dual-media filter (coal and sand) and attempts to show "ideal" limiting distributions of σ and i with filter depth. A low-porosity mixed region is shown near the base of the filter. It should be noted that on reaching the limiting effluent turbidity concentration, C_ℓ, the specific deposit at the base must be quite small and thus so must be the increase in hydraulic gradient $(i - i_o)$ over that of the clean sand.

CONCLUSIONS

The theory presented in this report clearly falls far short of providing explicit formulas on which filter design and operation can be based. On the other hand, it does provide conceptual guidelines that should lead to better designs and more economic and efficient filter performance. One point should be made regarding polymer aided filtration in the absence of the pretreatment safeguards of coagulation-flocculation and sedimentation. The filter functioning alone will require much closer monitoring than that functioning with pretreatment and it is likely that automatic control of the polymer feed will be desirable in situations experiencing rapid changes in water quality. As a final point, it is recommended that pilot-scale studies be considered as the initial step in the design of filtration-only clarification systems, especially where the waters have not been established as amenable to conditioning with polyelectrolytes.

BIBLIOGRAPHY

1. Mintz, D. M. Modern Theory of Filtration. Special Report No. 10. International Water Supply Congress, Barcelona, October 3-7, 1966.

2. Camp, T. R. Theory of Water Filtration. Jour. Sanitary Engineering Div., ASCE, 90, 1-30, August 1964.

3. Ives, K. J. and Gregory, J. Basic Concepts of Filtration. Proc. Society for Water Treatment and Examination, 16, Pt. 3, 1967.

4. Yao, K-M. and O'Melia, C. R. Particle Transport in Aqueous Flow through Porous Media, presented Annual Conference of the Hydraulics Div. of ASCE, August 20-22, 1968, at MIT, Cambridge, Mass.

5. O'Melia, C. R. and Stumm, W. Theory of Water Filtration. Jour. AWWA, 59, 1393-1412, November 1967.

6. Ives, K. J. and Sholji, I. Research on Variables Effecting Filtration. Jour. Sanitary Engineering Div., ASCE, 91, 1-18, August 1965.

7. La Mer, V. K. and Healy, T. W. Adsorption-Flocculation Reactions of Macromolecules at the Solid-Liquid Interface, Reviews of Pure and Applied Chemistry, 13, 112-133, 1963.

8. Linke, W. F. and Booth, R. B. Physical Chemical Aspects of Flocculation by Polymers. Amer. Inst. of Mining Engrs., Trans., 217, 1960.

9. Iwasaki, T. Some Notes on Sand Filtration. Jour AWWA, 29, 1591, 1937.

10. Ives, K. J. Rational Design of Filters. Procedings, Institution of Civil Engineers, London, England, 16, 189-193, June 1960.

11. Fox, D. M. and Cleasby, J. L. Experimental Evaluation of Sand Filtration Theory. Jour. Sanitary Engineering Div., ASCE, 92, 61-82, October 1966.

SECTION C

EXPERIMENTAL STUDIES RELATED TO ADVANCED CONCEPTS

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EXPERIMENTAL STUDIES OF WATER FILTRATION*

Gordon G. Robeck**

In the past 70 to 80 years there has been a gradual improvement in the basic processes of filtering surface water. Some of the earliest innovations centered on increasing from slow to rapid filtration rates and introducing various cleaning methods such as back and surface washing. More recently the engineering advances have been concerned with modifications of filter media that would allow greater production of high quality water from a given filter area. This aspect of design change will be discussed today. Specifically, the discussion will be limited to the design and operation of filters with more than one type of porous medium -- which in turn may allow successful use of high rates.

Before presenting experimental data, perhaps it would be appropriate to describe briefly some of the theory and historical development behind these newer systems. In essence, the theoretical basis for a multi-layer filter is simply to have the permeability of a bed decrease with depth, thereby achieving greater unit process efficiency. This is just the opposite of the conventional rapid sand filter which stratifies after backwashing, thus positioning the finest sand on top. This arrangement limits the straining or filtering action to the upper few inches of the bed.

If the stratified sand medium could be turned upside down the situation would be ideal, allowing the larger and more easily removed particles to be trapped at the top and the more difficult ones to be stopped at greater depth in the bed. Because rotation of a filter is not practical, the logical solu-

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tion is to provide a lightweight, coarse material on the top of a finer but heavier medium.

Baylis[1] experimented with dual-media filters in 1935 by using 2 to 3 inches of crushed quartz and anthracite over magnetite and sand, respectively. The results showed that this roughing filter of coarse material greatly increased the length of run. He also reported that several filter plants were already using this thin layer of anthracite on top of sand filters by 1939.

Very little more was written about dual-media systems until 1960 when Conley and Pitman[2] described research and plant testing conducted at the U.S. Atomic Energy Plant, Hanford, Washington. Sand filters at this plant were unable to meet desirable volume and quality requirements, but the dual-media system of coal and sand was found to be much more successful. Conley's design included mixing the two media during backwashing so that no two distinct layers of media existed after cleaning. Camp[3] disagreed with Conley about the merits of operating with the two media somewhat mixed, especially as this arrangement might affect bacterial removal.

Work at this laboratory was conducted with pilot dual-media filters to help resolve this issue. By introducing an overload of virus, coliform, activated carbon and other particles, the efficacy of various treatment processes was determined. Several reported small-scale and other joint field studies have indicated that these particles can be and are usually removed by either media arrangements.[4] A recent report[5] did show, however, that mixing the two media in small filters at Erie, Pennsylvania, extended filter runs and made backwashing easier. Actually, algae and alum floc were both well removed by any media arrangement tried, so operational efficiency rather than improved quality had to serve as a basis for improved design.

In an attempt to achieve more mixing and less permeability toward the bottom of the filter, Conley designed and reported on a bed with anthracite, sand, and garnet. The garnet with high specific gravity was used as a very fine bottom layer. His report[6] showed some data illustrating the advantages of this system over a sand or anthracite-sand filter.

One could go on and on with such refinements by selecting media of different specific gravity and size such as putting large, light-weight plastics on top of coal. The economic breakpoint on size selection may vary with raw water conditions and availability of materials. The economic answer is not yet known for all waters. Therefore, this discussion will be confined to the principles of the authors' experiences with dual and triple media, commonly referred to as multi-media.

PRE-FILTRATION TREATMENT

Undoubtedly, the designer has a degree of control over the final water quality by his selection of media size, type, and depth and filtration rate; and the operator can control rate and floc strength. The last of these, however, is the most flexible factor and can be most readily altered to meet the varying raw water conditions; hence, its development and control will be discussed first.

A strong floc is one that can be readily removed in any type of filter usually producing a rapid increase of head loss, particularly near the surface of the filter. A weak floc, on the other hand, is one that has a tendency to penetrate a filter easily, thus producing very little head loss. Hudson[7,8] has discussed the implications of floc strength and proposed a method of quantifying it from filter performance. No one, however, has perfected a practical system for determining floc strength directly and quickly.

We have tried various approaches, none of which does much more than demonstrate an after-filtration-event that the floc was weak or strong. One indirect method shows some promise, however. It involves stressing a set of small pilot-type filters with a variety of coagulant doses and a hydraulic rate that would create shear stresses on the floc similar to those experienced near the end of a normal, long filter run. In other words, by operating a small replica filter at a 12 gpm/sq ft rate for 30 to 60 minutes, an operator can predict whether he will have an early breakthrough, a rapid head loss increase, or a good long run.

Figure 1 shows the experimental dual-media test filter arrangement and the outlets for measuring head loss and turbidity.

Figure 2 shows the results of operating a small dual-media filter with different coagulant aid doses added to the rapid mix. In the lower set of curves where 20 mg/l of activated silica was used along with 75 mg/l of alum, the accumulation of floc was mainly between points 0 and 2. On the other hand, in the upper set of curves derived from a parallel run using only 10 mg/l of activated silica, the floc accumulation was somewhat at the sand-coal interface as well as throughout the coal layer. Head loss curves such as these tell the operator where the work is being done in the filter and whether he is using an excessive amount of aid, thus making the floc so strong that it is forming a layer on top of the filter instead of being removed somewhat throughout the entire coal layer. As indicated earlier these trends can be noticed very early in a pilot run if all the coagulant is fed to the filter influent and excessive hydraulic rates are used to simulate conditions near the end of a normal filter run. This technique has been reported on in the March 1968 AWWA Journal.

To illustrate the influence of more than two layers of media, it might be well to first show a few figures comparing the results from single layer with those from two layers. Figure 3 indicates that with relatively weak floc the breakthrough that is apparent in the top set of curves for coal and sand were put off 10 to 12 hours by using 18 inches of coarse coal on top of 6 inches of sand, instead of 2 feet of sand or anthracite alone. Many other runs demonstrated this to be a common pattern during winter conditions.

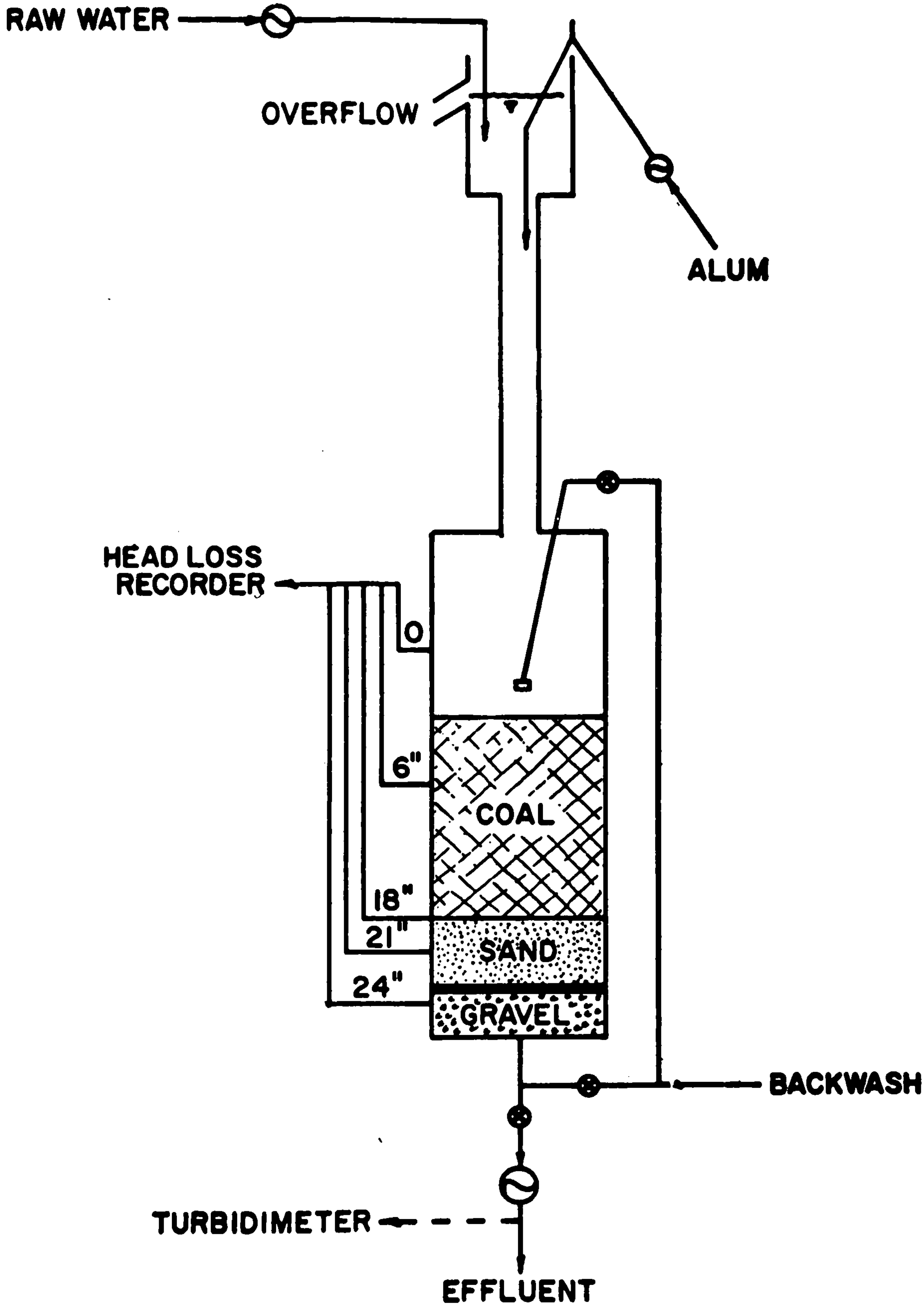

FIGURE 1. Details of Experimental Filter.

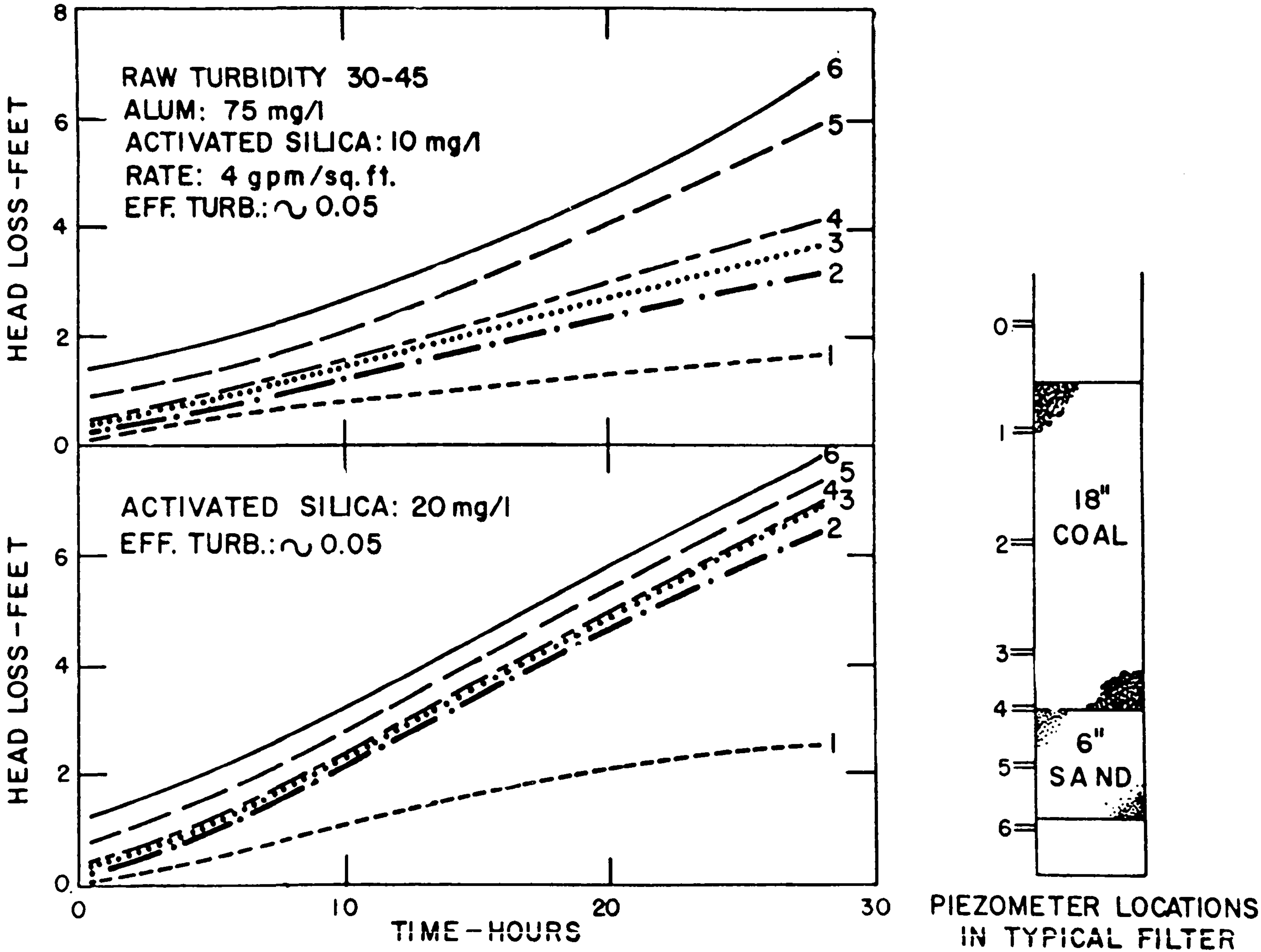

FIGURE 2. Influence of Polyelectrolyte on Floc Penetration.

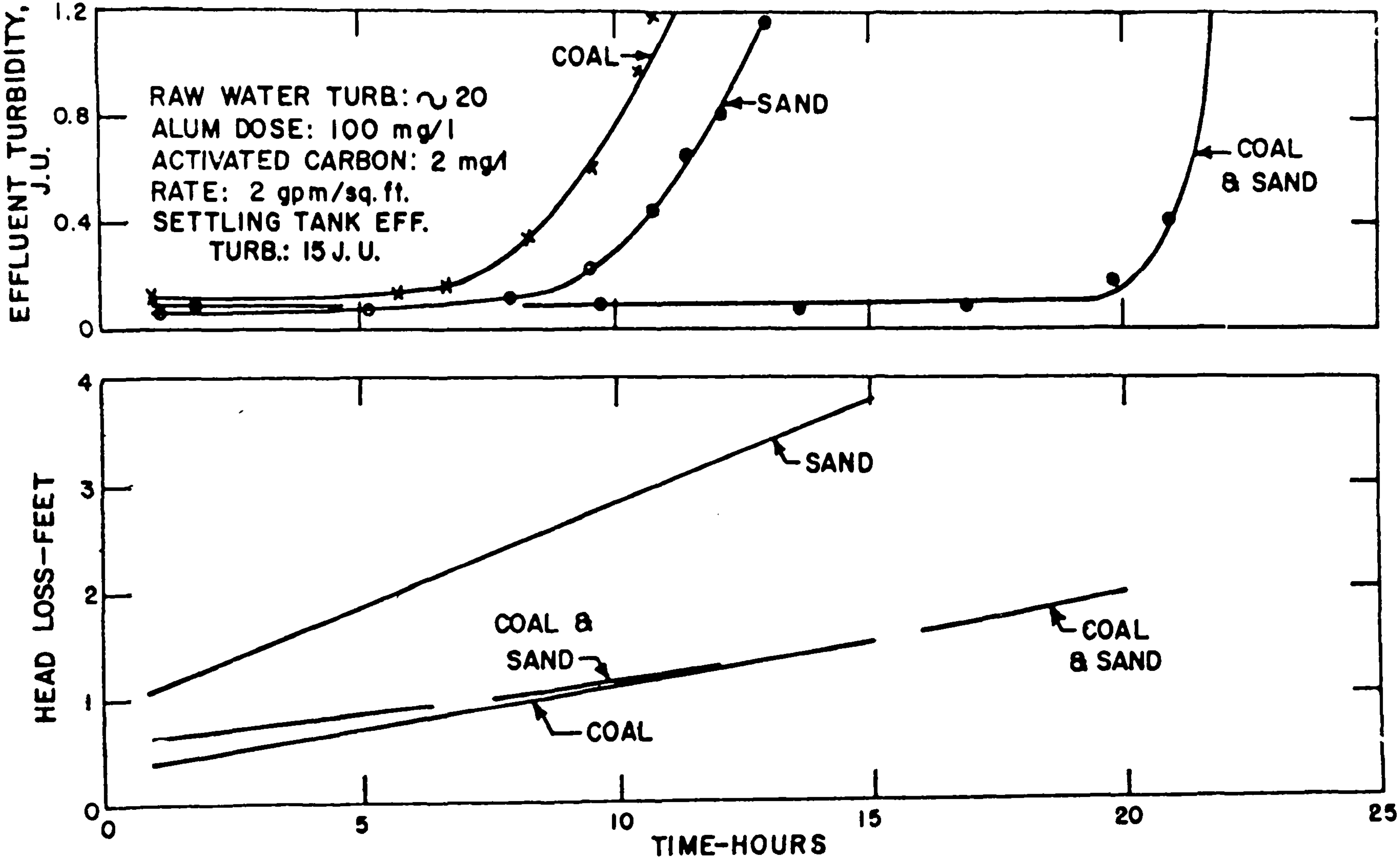

FIGURE 3. Influence of Filter Media on Length of Run with Weak Floc.

Figure 4 shows that floc was strengthened by using activated silica with alum. In this case, there was no breakthrough with any of the three filters. The double-layered filter, however, again proved most efficient because there was not such a rapid overall head loss buildup. Deeper but not complete penetration of the floc through the bed allowed this condition to develop.

Actually, an aid is not always necessary. Several cases were found, especially in the summer, when turbidity was well removed by all three combinations of filter media with alum alone. Incidentally, previous studies indicated that coliform and virus penetration usually increased whenever the filter effluent turbidity increased[4].

The effect of adding another finer but heavier medium is shown in the next figures. Figure 5 indicates that,when dealing with relatively weak floc, the multi-media arrangement (MM-2) with only 1 inch of coarse and 2 inches of fine garnet prevented a breakthrough to 1 J.U. for 130 minutes. The dual media, however, permitted such a breakthrough in 92 minutes and a multi-media arrangement (MM-1) with 3 inches of coarse garnet under 3 inches of fine, permitted a breakthrough in 110 minutes. Thus, it is apparent that both the finer material and the degree of mixing after backwashing can influence the water quality when applying a certain floc. However, to further illustrate the influence of coagulation and floc strength, Figure 6 demonstrates that another run was terminated more quickly with the media arrangement that had the most garnet in it and the dual media extended the run almost twice as long without a breakthrough. Thus it is apparent that no ideal media mixing can be selected for all conditions. The designer must simply strive to build-in all the sensible safety factors he can, but still attempt to economize on capital and operating expenses.

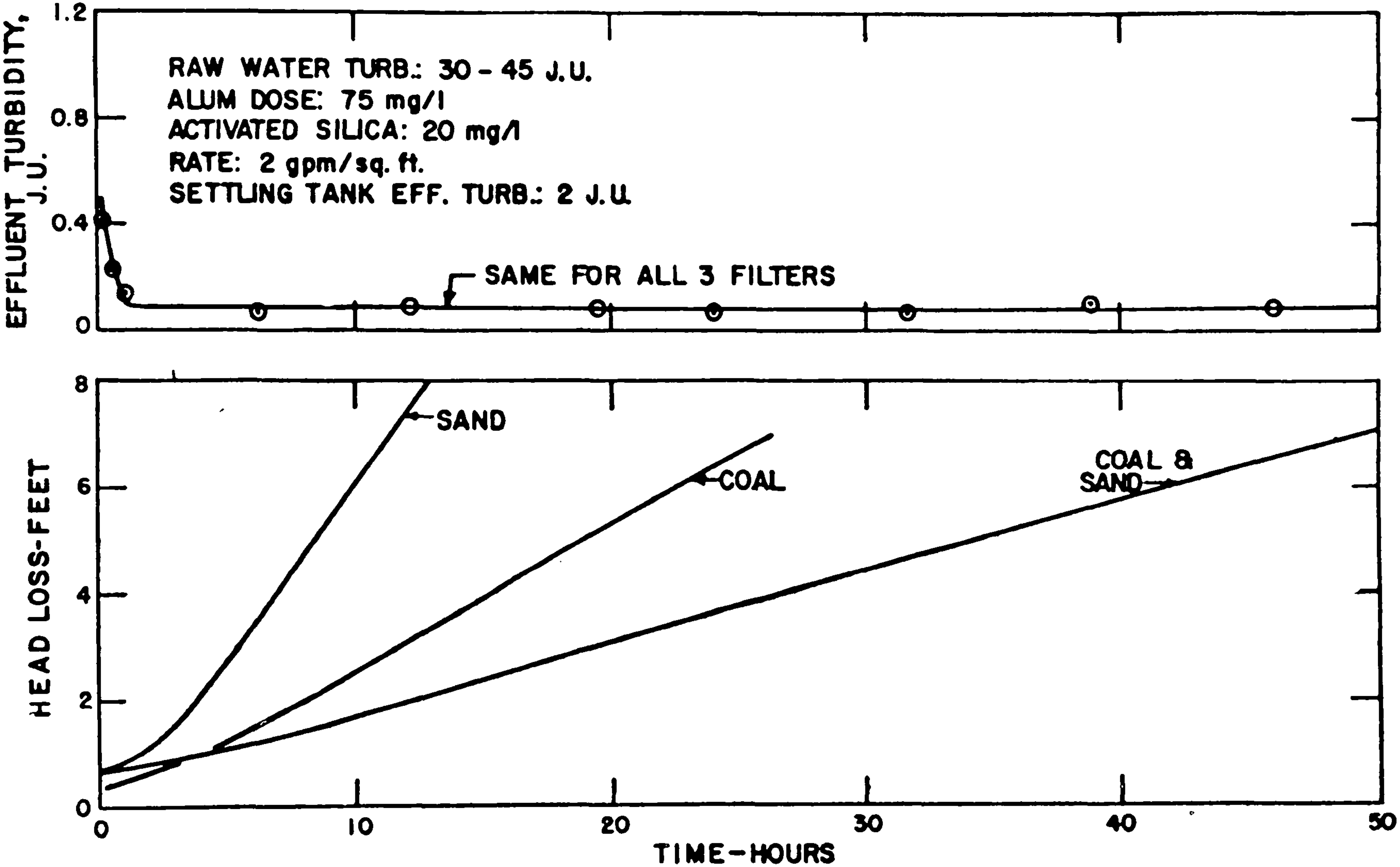

FIGURE 4. Influence of Filter Media on Length of Run with Strong Floc.

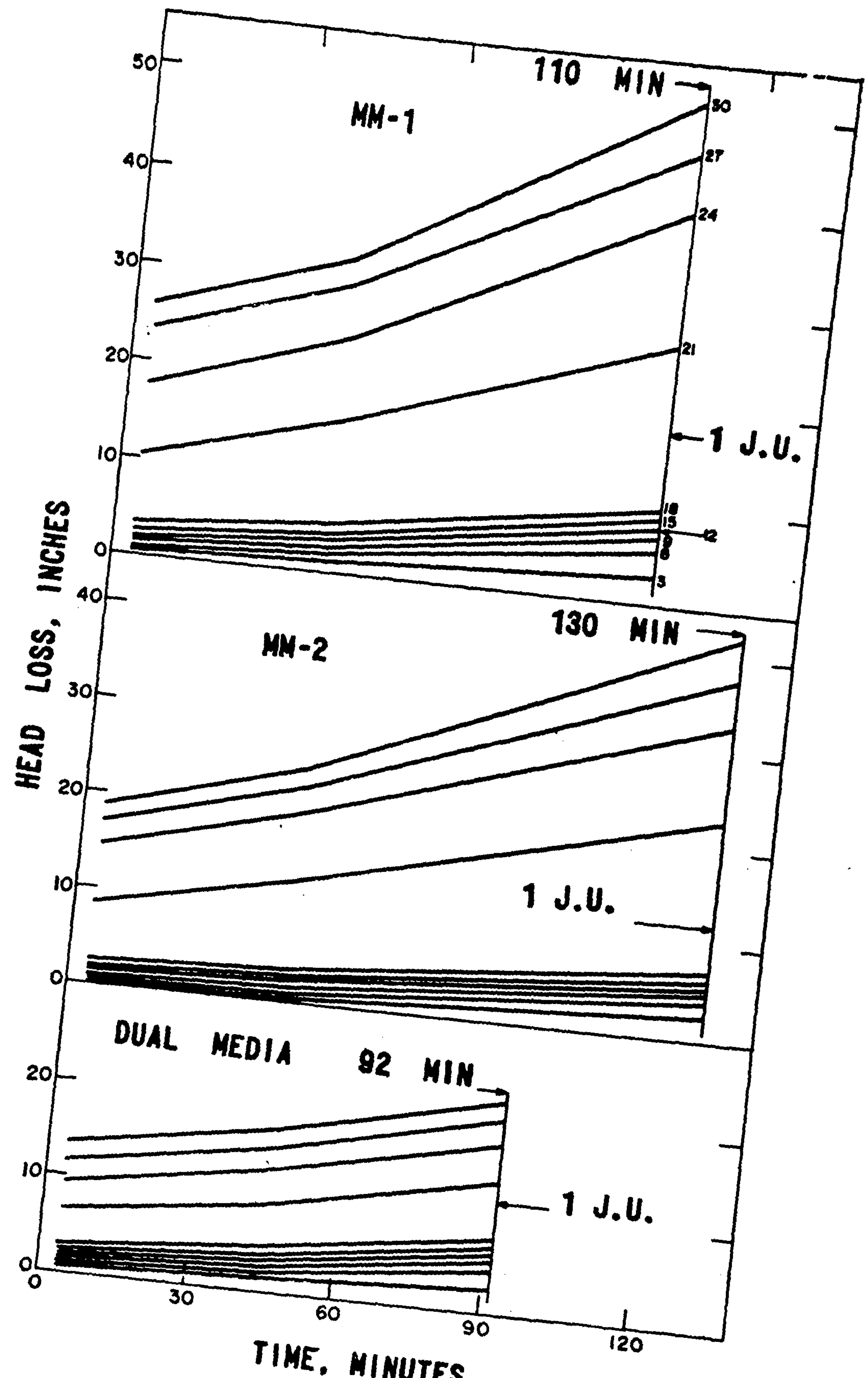

FIGURE 5. Weak Floc Condition, 4 gpm/sq ft.

FILTRATION RATE

Rate of filtration has been considered quite important as far as protecting the final effluent quality. Regulatory agencies have, therefore, usually specified some constant rate for all municipal operations and for many years the 2-gallon rate has been accepted as adequate from a safety standpoint.

Numerous studies have been made over the years on the influence of filtration rate and Baylis[9,10] was one of the first to indicate that higher rates, up to 5 gallons/sq ft, were tolerable with Lake Michigan water. Conley[11], at Hanford, has reported the use of rates as high as 8 when producing industrial water without significant quality reduction. Segall and Okun[12], at North Carolina, have concluded that the filter effluent is degraded by higher rates. Much of this conflict and confusion amongst investigators comes from unspecified test conditions, especially those regarding floc strength. In other words, it is certainly theoretically possible to force more of a given set of particles through a porous medium if the filtration rate is increased. On the other hand, there is a practical way to overcome this influence by the use of chemical coagulants, otherwise rapid filtration rates could not be used at all, be it 1 or 6 gpm/sq ft. Ideally, the regulatory groups could do better by requesting a more careful control of coagulation; or they might even suggest the use of a decreasing hydraulic throughput, because specific velocities within the pores are constantly increasing as the pores are being filled with particulates. This would minimize the increase in hydraulic shear on deposited floc. Hudson[13,14] and Easterday[15] have reported on such variable rate design systems, and even indicated that entire plants could be designed to cope with the changing water production.

Another flow rate variation that is frequently experienced and discussed is the surge phenomenon. Some of the surges are due to a natural hydraulic action, and other are due to willful, sudden changes of rate controllers when

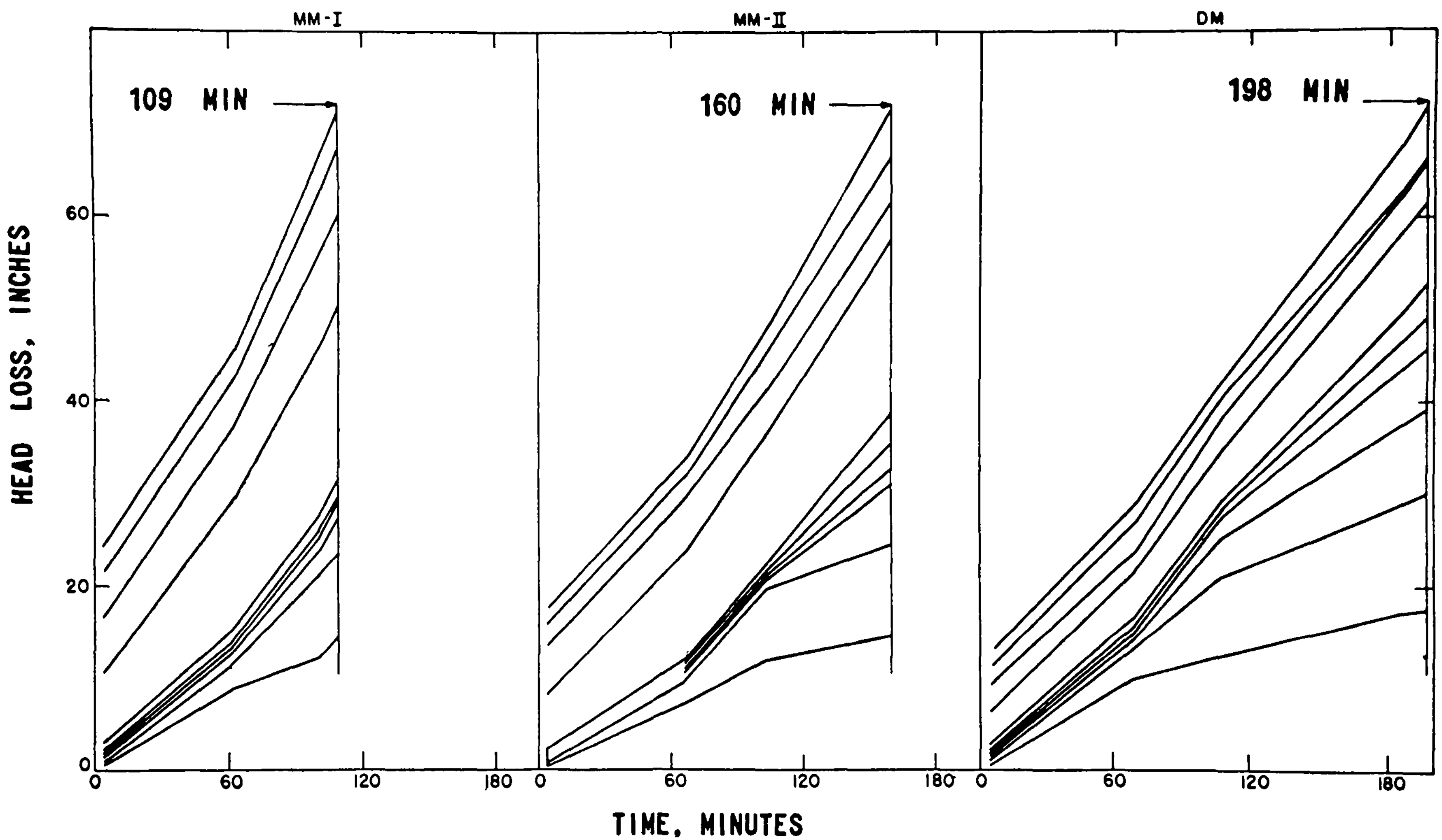

FIGURE 6. Strong Floc Condition, 4 gpm/sq ft.

more water is needed for a few hours to fill a particular reservoir. The size of most surges, as indicated by Baylis[16] in 1958, are less than 1 percent of the total. Many man-induced hydraulic changes, however, can be as high as 100 percent when rates are changed from 1 to 2 to 4 gpm/sq ft. Cleasby[17] has attempted to evaluate the effects of these changes on water quality and has determined that the quicker the rate change is made, the more material passes the filter. His work was done with weak floc and is subject to revision when done with strong floc. Conley[18] has indicated that multi-media filters using fine garnet are less sensitive to breakthrough due to rate changes.

Tests on surges conducted by this laboratory are shown in the next two figures. Figure 7 shows the effect of a 100 percent increase in 55 seconds or 1.8 percent per second. The test conditions involved going from a 2- to a 4-gpm/sq ft rate and using weak floc. In all three cases there was a slight breakthrough but each reached a peak turbidity and then began to decline. The stresses, however, were such that the filters could not completely recover at the higher 4 gpm/sq ft rate. When the same rate of change was used to return the filters to their original 2 gpm/sq ft rate, all three media arrangements quickly allowed a recovery of effluent quality.

Figure 8 illustrates what happens when using strong floc under the same change of rate. The turbidity breakthroughs resulting from this rate increase were insignificant. To stress the system more, a nearly instantaneous surge of 100 percent increase and return in 8 seconds was tried. This, too, produced no breakthrough.

Examples of the influence of filtration rate on length of run under strong and intermediate floc conditions are shown in the next two figures. Figure 9 depicts strong floc being held in the first 6 inches even at 6 gpm/sq ft. Figure 10 shows that this higher rate of 6 with intermediate strength floc permitted a deeper penetration and thus allowed a filter run equal to a 4-gallon rate before reaching an 8-foot terminal head loss.

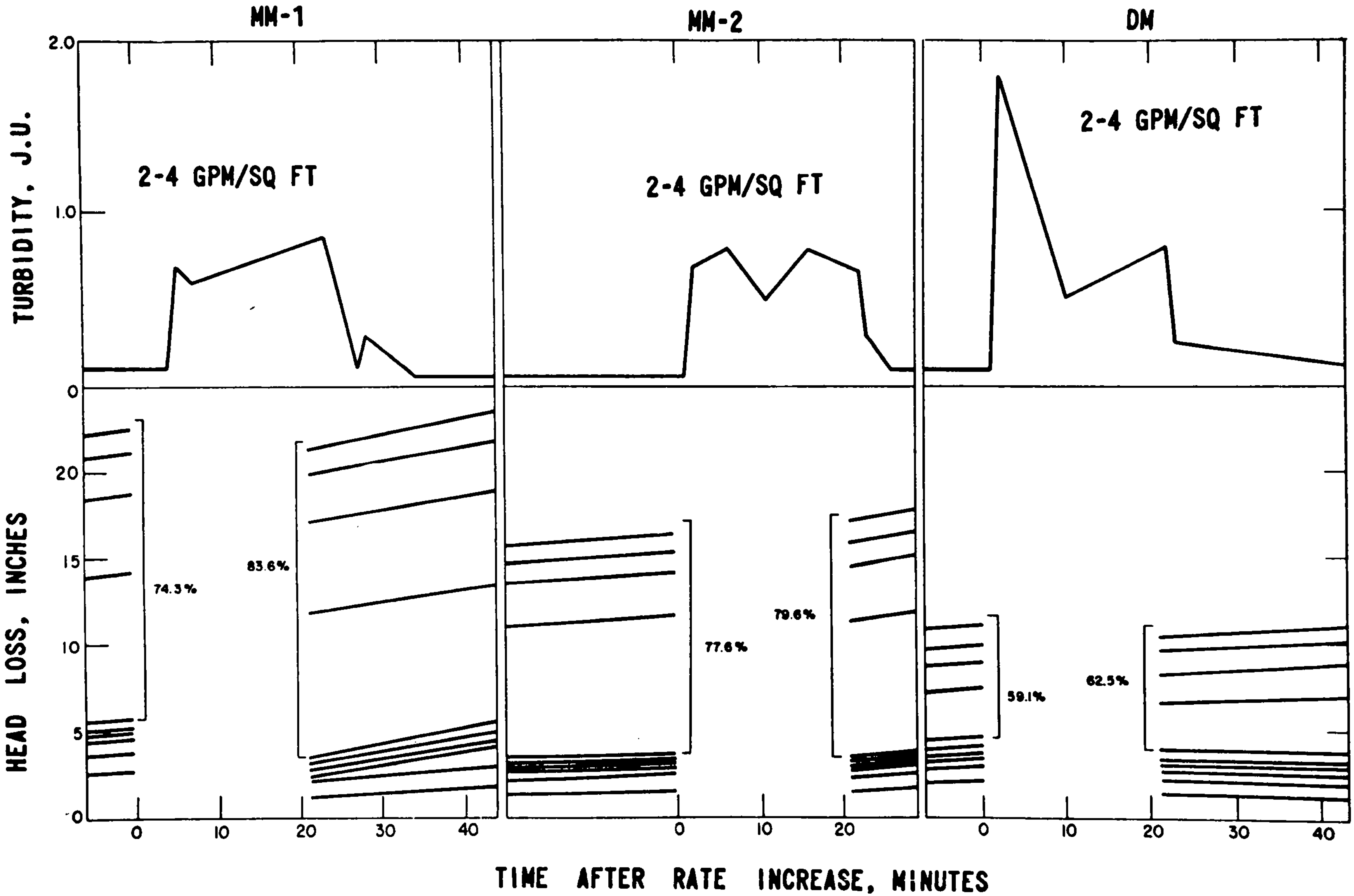

FIGURE 7. Rate Changes with Weak Floc.

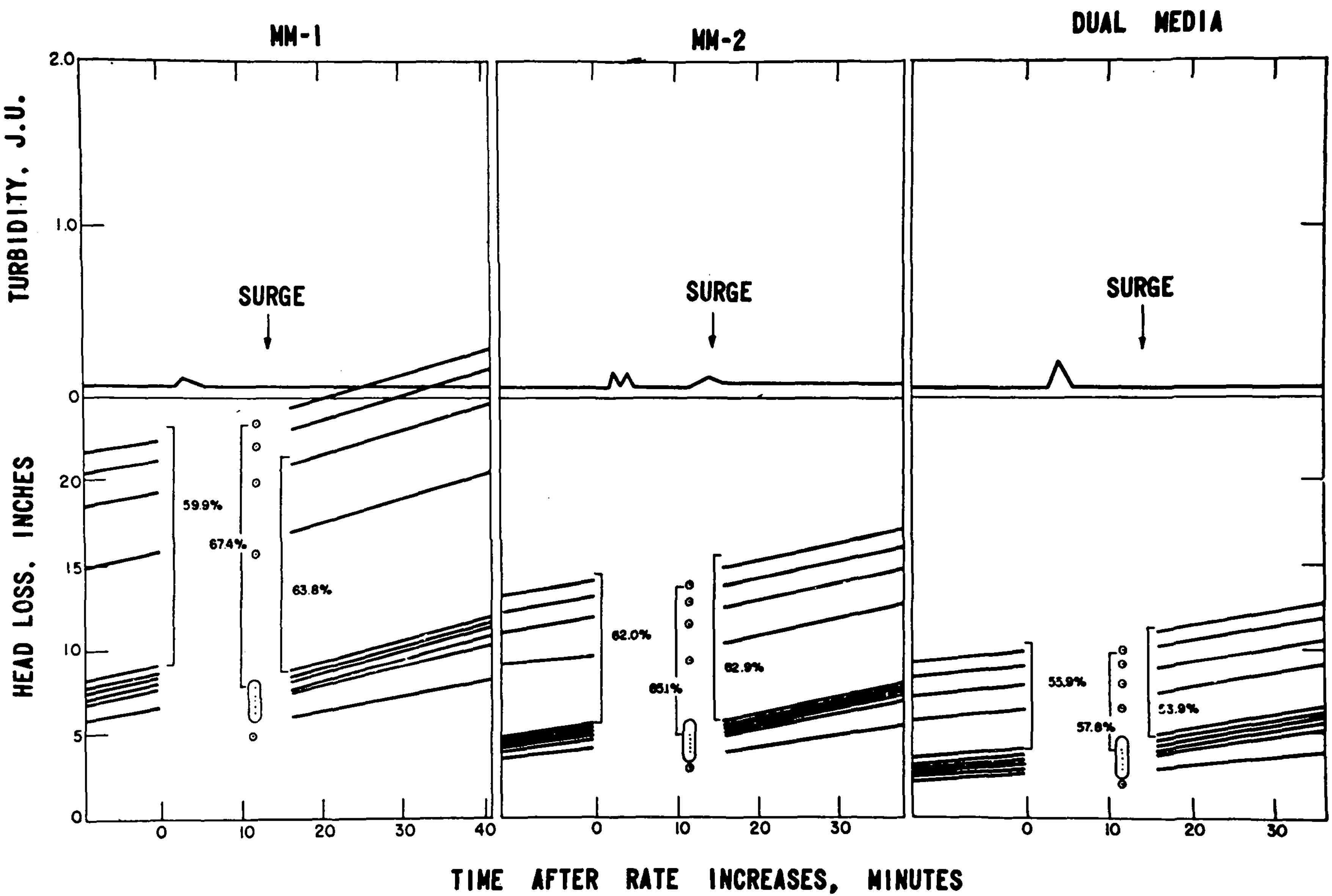

FIGURE 8. Rate Changes with Strong Floc.

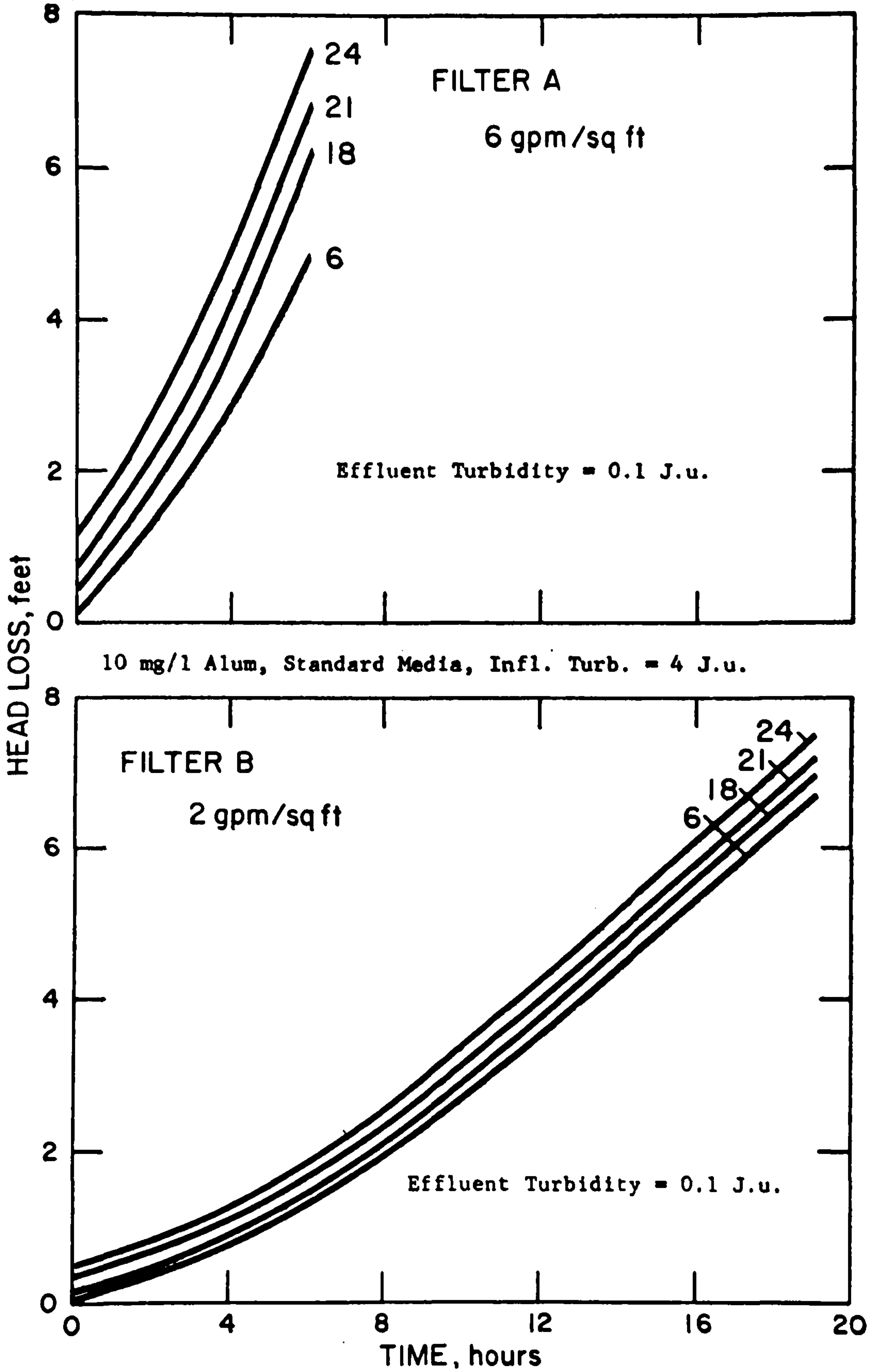

FIGURE 9. Influence of Filtration Rate on Run Length.

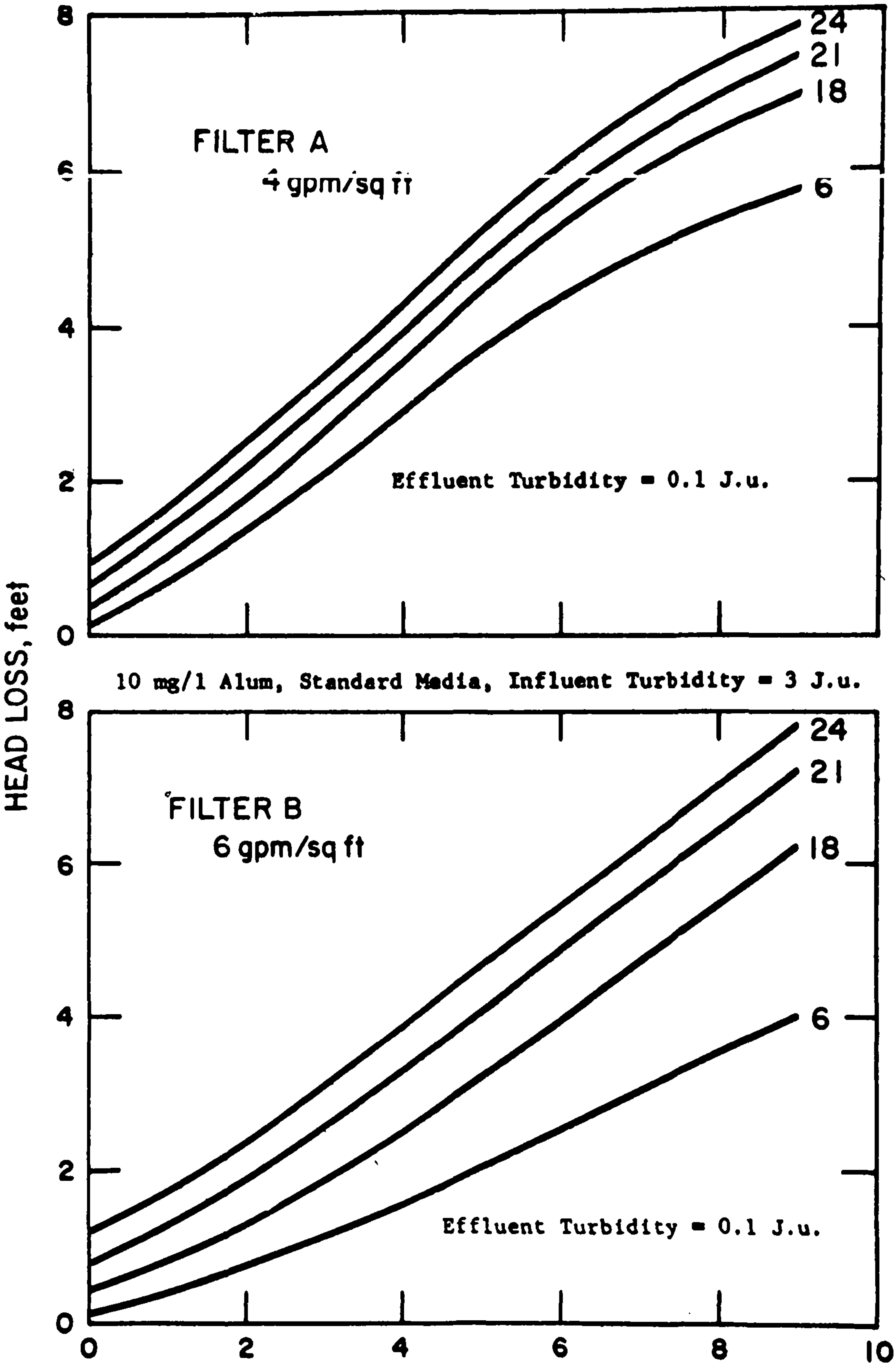

FIGURE 10. Influence of Filtration Rate on Run Length.

The designer, therefore, must take into consideration the potential control of quality the operator has via the coagulation process before he makes an arbitrary decision about rates.

The benefits in net production from higher rates can be noted in Figure 11 which shows that, with dual media, about 8 mgd/1000 sq ft can be filtered at 6 gpm/sq ft, whereas only about 2.8 mgd/1000 sq ft can be put through at 2 gpm/sq ft.

FILTER MEDIA

All of this discussion about floc and rate makes it apparent that media size selection is not the principal consideration in designing or operating a treatment plant. However, some reasonable improvement in quality and economics can be accomplished by intelligent use of various media. The next figures demonstrate some of the alternatives a designer has and why one arrangement may be better than another.

When using a multi-media filter, both the effluent clarity and length of run may be influenced by the following: effective size, uniformity coefficient, specific gravity and depths of each medium. The size of the top layer is particularly important.

The size selection of coal has changed somewhat during recent years. Five years ago, No. 1 anthrafilt, which is a coal with an effective size (e.s.) of 0.7 mm, was placed on top of a sand with an e.s. of 0.5 mm. Now, much coarser top material is used. Figure 12 shows a comparison between the so-called standard media filter, which is 18 inches of coal with an e.s. of 1 mm and a uniform coefficient of 1.11 over 6 inches of sand with an e.s. of 0.49 mm and a uniform coefficient of 1.14, and a No. 1.5 commercial anthrafilt with all particles finer than 1.2 mm removed over a commercial Muscatine sand with an e.s. of 0.43 mm and a uniform coefficient of 1.62. The second filter with the commercial cuts that used coarser media on top gave a much improved

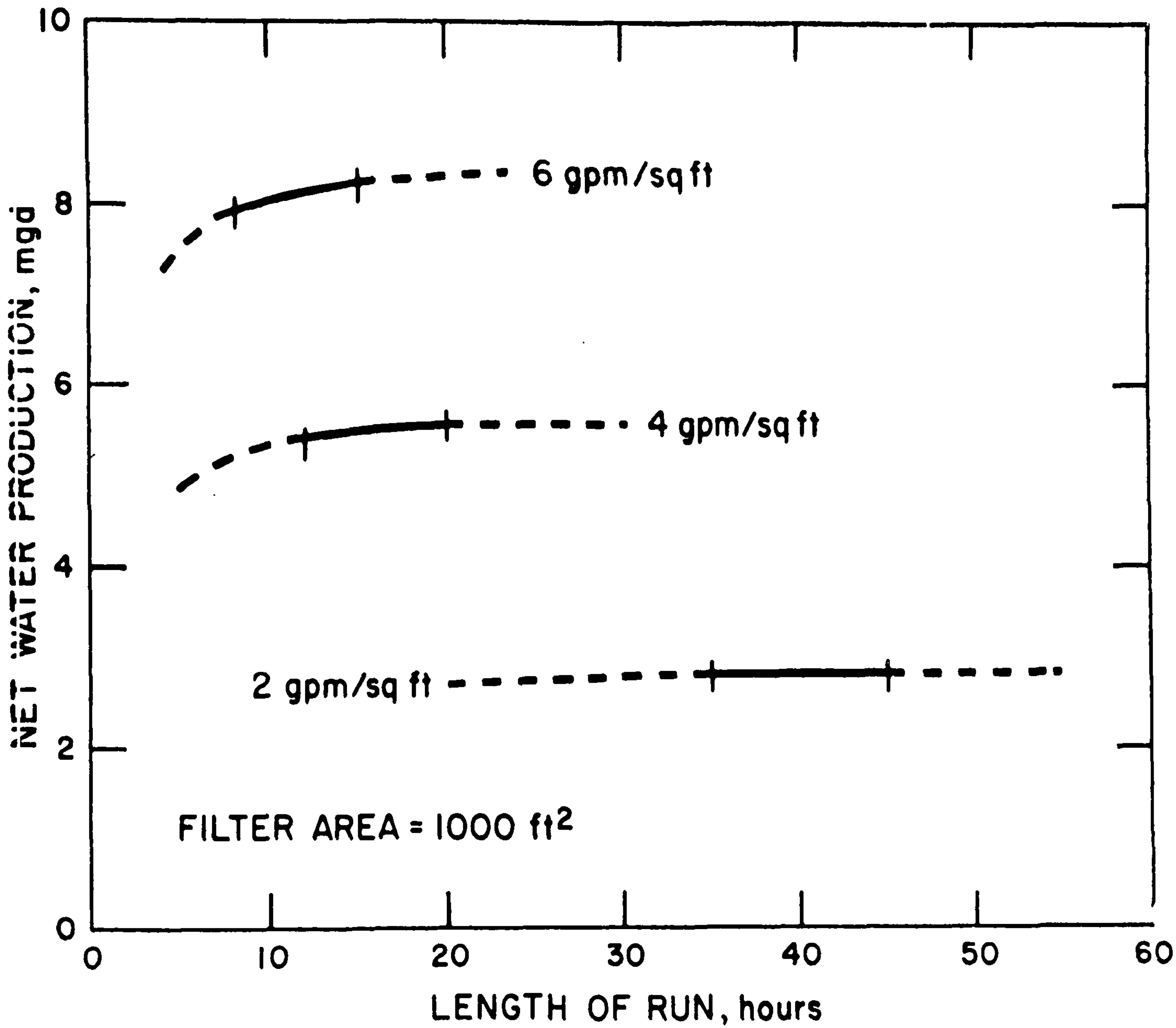

FIGURE 11. Influence of Filtration Rate and Run Length on Net Water Production.

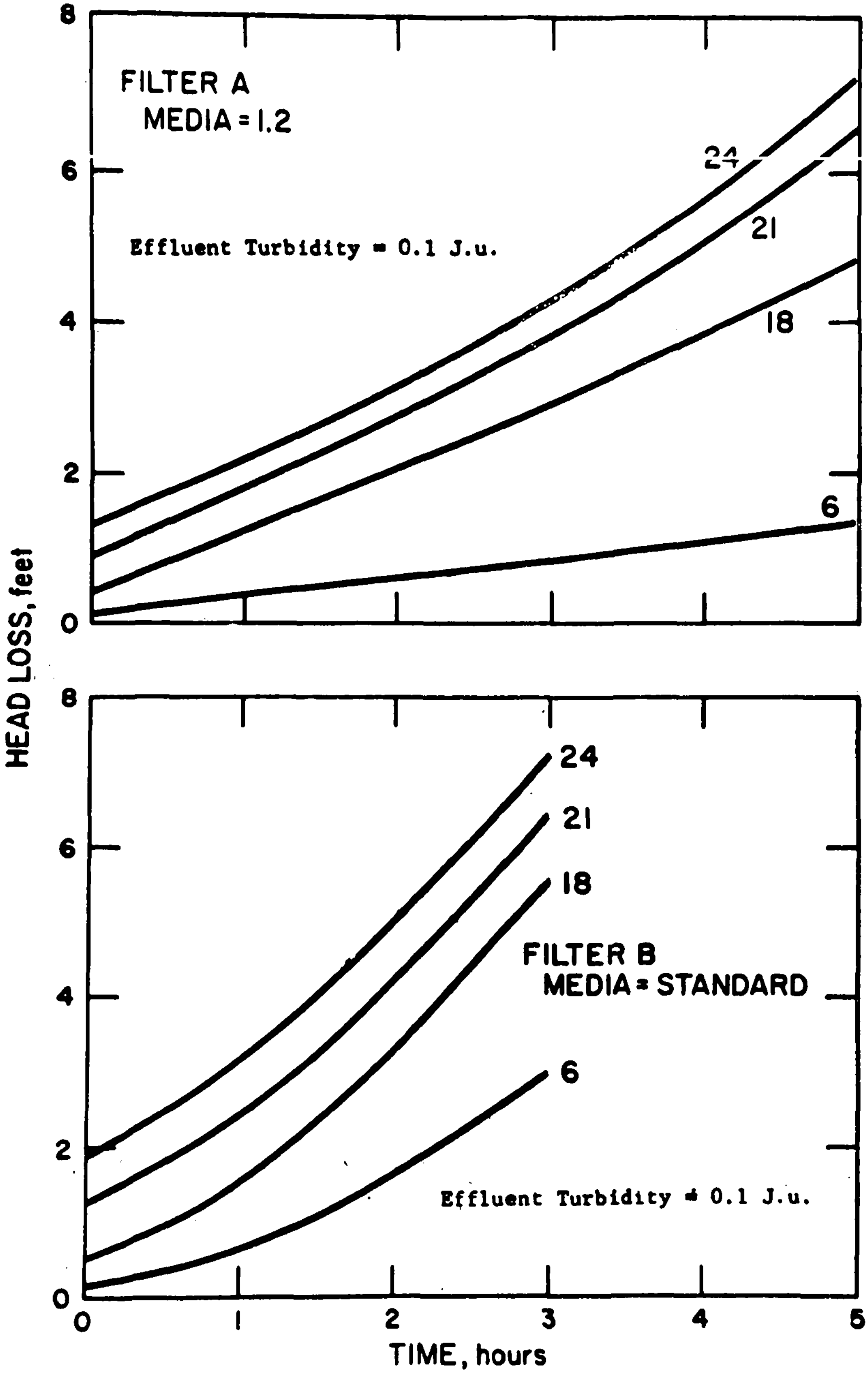

FIGURE 12. Influence of Filter Media on Run Length (6 gpm/sq ft, 10 mg/1 Alum, Influent Turbidity 35 J.u.).

filter performance because of longer runs. Effluents were of the same excellent quality in all cases (<0.1 J.U.).

Test results in Figure 13 show that the standard media was outperformed by an arrangement where all the coal finer than 1.4 mm was removed. At a top size of 1.7, however, early breakthroughs began to occur so there is a practical limit to this practice. This size is also impractical if only No. 1 1/2 anthrafilt is used because 70 percent is finer than 1.7 mm and, other than fuel value, this material would be wasted.

Figure 14, a graphic summary of runs made at Erie, Pennsylvania, shows how the commercial coal and sand dual-media arrangement was superior to the very narrow cuts of the same materials in the so-called standard filter. Generally, the run length increased with increased coal size in the top layer of the dual-media filters under these Lake Erie test conditions. This change in coal size caused the ratio of coal-to-sand size to change and thus the amount of mixing to increase. It is, therefore, somewhat difficult to isolate the true influence of mixing alone. Nonetheless, this mixing is an inherent part of an improved design.

Because this mixing of the coal and sand was considered beneficial to the length of run, a satellite study was performed to determine the actual degree of mixing near the interface by measuring head loss at 3-inch intervals in a small filter operated at a high rate of 14.5 gpm/sq ft. This rate made for a pronounced change of head loss over each 3 inches so that the results would be graphically illustrative.

The example in Figure 15 is for three different filter media; sand, coal and a combination of the two. The third contained 18 inches of 1.14 mm coal and 6 inches of 0.43 mm sand, whereas the other two had 24 inches of each type. Both the sand and coal beds created a high head loss per inch of material in the top 2 inches. When all of the coal finer than 1.0 mm was removed, this head loss was markedly reduced. In fact, after backwashing, it

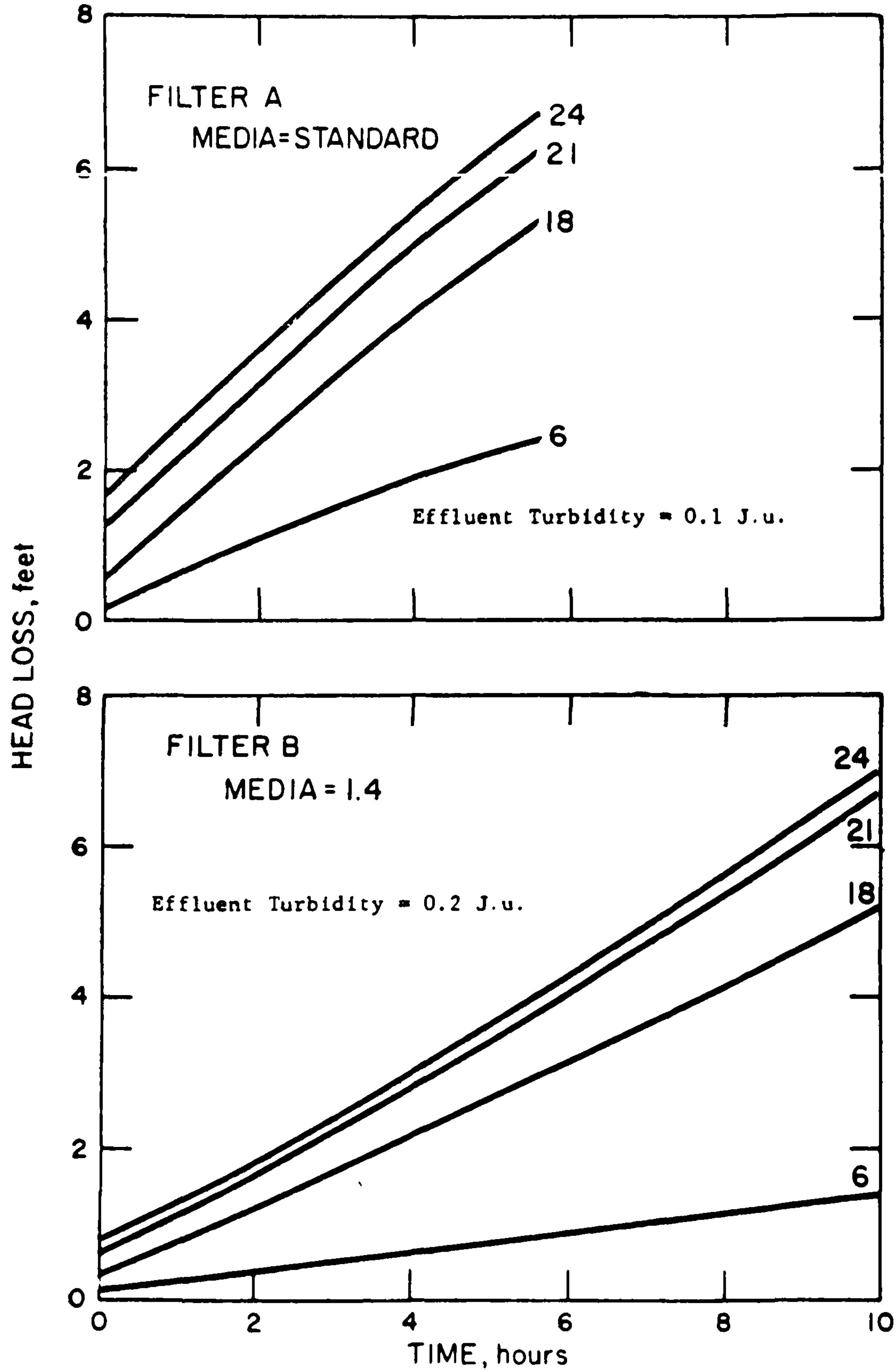

FIGURE 13. Influence of Filter Media on Run Length (4 gpm/sq ft, 10 mg/l Alum, Influent Turbidity=7 J.u.).

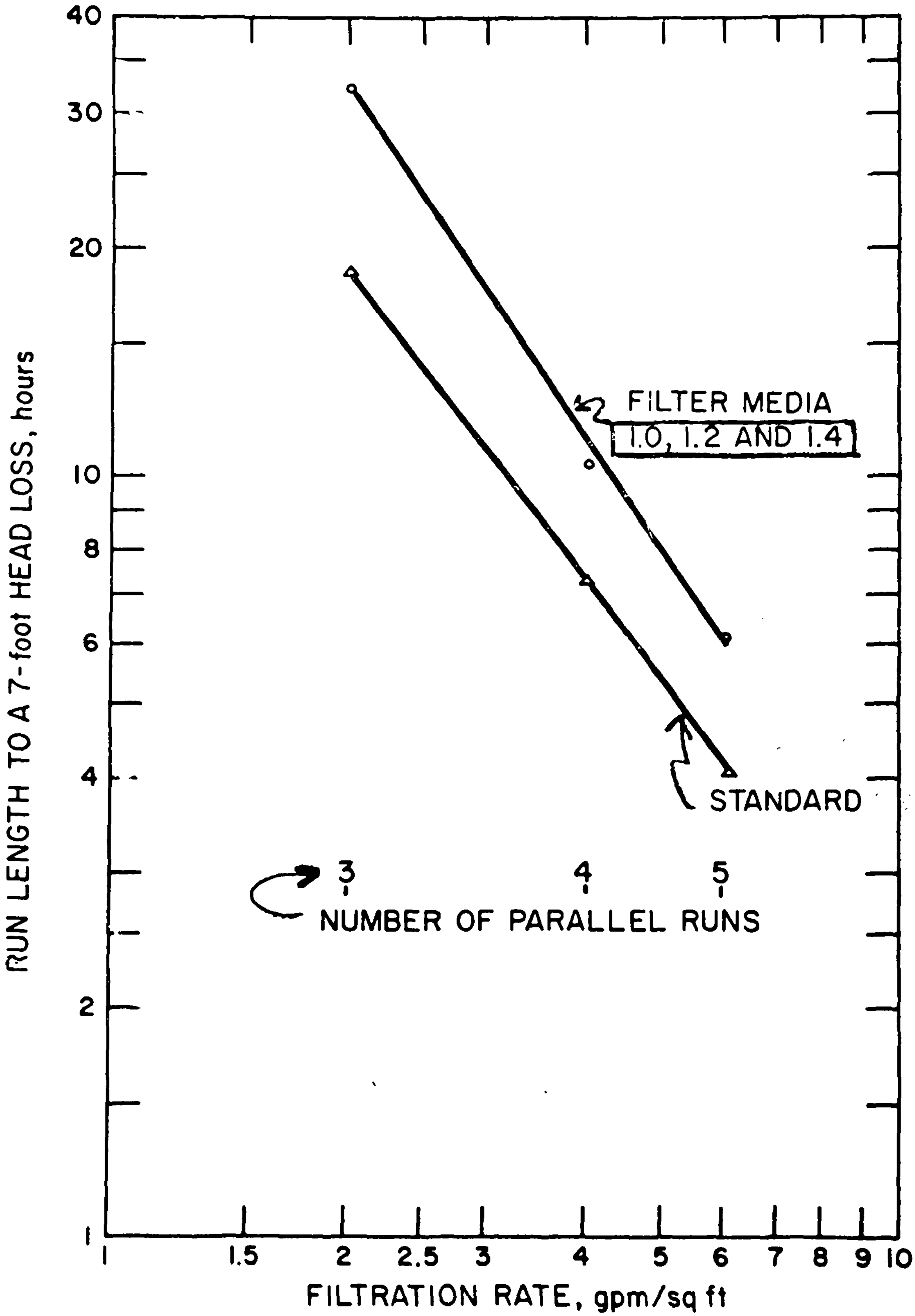

FIGURE 14. Influence of Filtration Rate and Filter Media on Run Length.

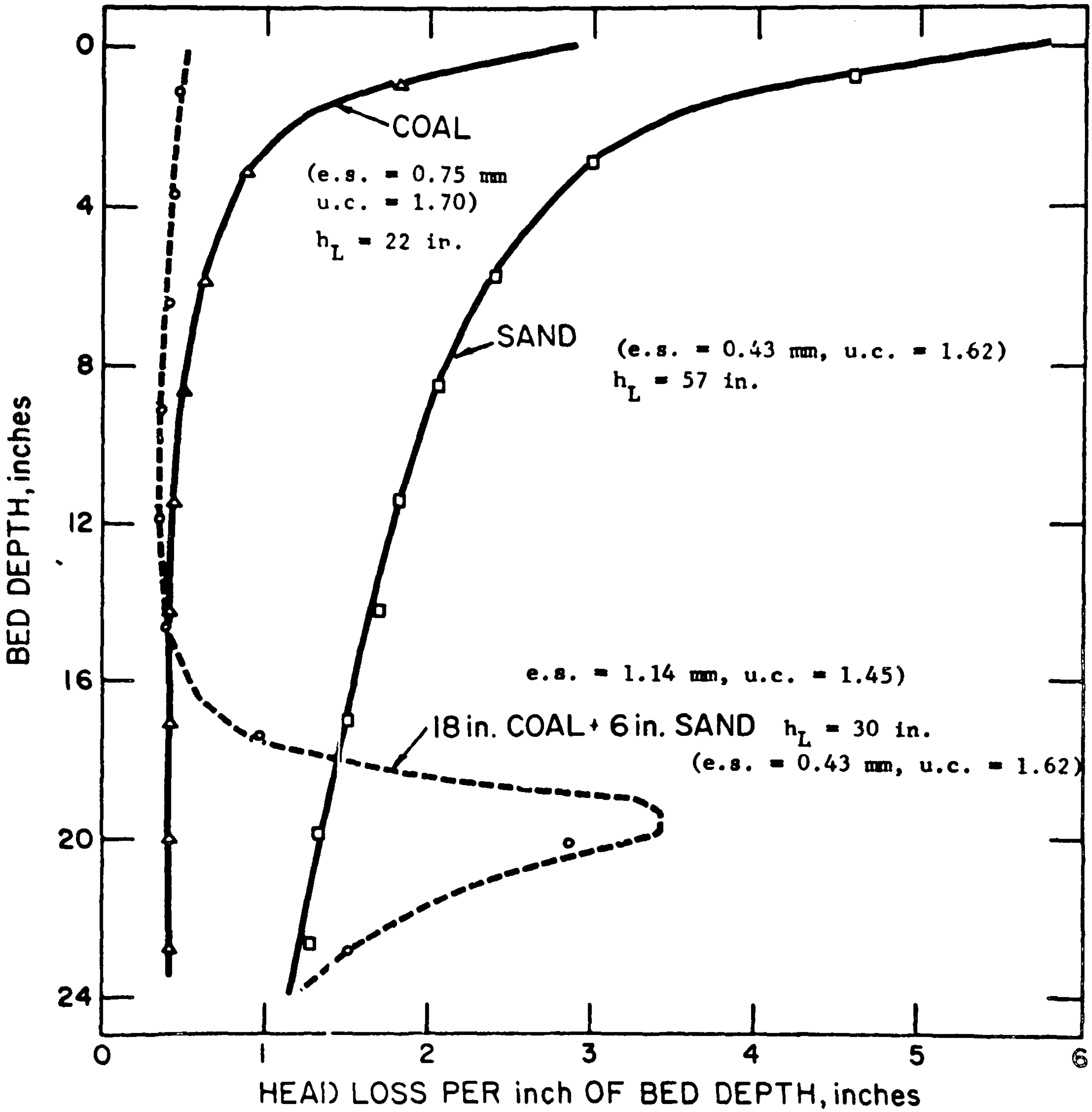

FIGURE 15. Clean Water Head Loss for Three Filter Media at 14.5 gpm/sq ft.

virtually uniform to a depth of 14 inches, thus allowing for an efficient use of the pores for floc storage. The unit head loss gradually increased from 14 to 20 inches as a result of the mixing of sand and coal. An effective barrier is located in this zone. Conley's[16] concept of adding garnet to these two materials is meant to shift the position of the less permeable zone so that the head loss curves are even greater near the bottom than those portrayed in Figure 15. The degree to which a designer wants to extend this principle depends on how much initial head loss he is willing to accept and how short a run he can economically justify. Actually, there is an economic or practical limit to such refinements. If the penetration of relatively weak floc and the influence of surges can be minimized by using inexpensive sand under coarse coal, it may not be necessary to incorporate garnet which is finer, heavier and more expensive.

In spite of vagueness in choosing media, experiments have been conducted at this laboratory to demonstrate the influence of mixing garnet with sand and coal. These experiments were conducted by applying clean water at 2, 4, and 6 gpm/sq ft rate on to 30 inches of media that is described in Table 1.

The next three figures show that the permeability in the lower zone can be greatly altered by simply varying the media, the mixing or the backwashing procedure. For example, in Figure 16 the maximum head loss at a 6 gpm/sq ft rate with dual media was about 3 inches, whereas in Figure 17 the multi-media filter (MM-2) with 2 inches of fine garnet produced a maximum head loss of 6 inches. As shown in Figure 18, this change in permeability (a head loss of 9 inches) was further extended by adding 3 inches of fine garnet (MM-1). To demonstrate that this permeability can be achieved by proper selection of just sand with coal, several tests were made with 12 inches of fine Ottawa sand and 18 inches of coal. This resulted in 11 inches of maximum head loss. (See Figure 19). The relative tightness caused all the runs with this arrangement to be entirely too short, so it was concluded that there

TABLE 1

Filter and Media Characteristics

Filter	Type	Depth, Inches	Top Layer Size, mm	Effective Size, mm	Uniformity Coefficient
Dual Media	Anthracite coal	18	1.00	1.12	1.39
	Muscatine sand	12	0.42	0.48	1.37
Multi-Media I	Anthracite coal	18	1.00	1.12	1.39
	Muscatine sand	6	0.42	0.48	1.37
	Fine Garnet	3	0.18	0.19	1.35
	Coarse Garnet	3	>0.30	>0.30	-
Multi-Media II	Anthracite coal	18	1.00	1.12	1.39
	Muscatine sand	9	0.42	0.48	1.37
	Fine Garnet	2	0.18	0.19	1.35
	Coarse Garnet	1	>0.30	>0.30	-
Dual Media Special	Anthracite coal	18	1.00	1.12	1.39
	Ottawa sand	12	0.18	0.19	1.37

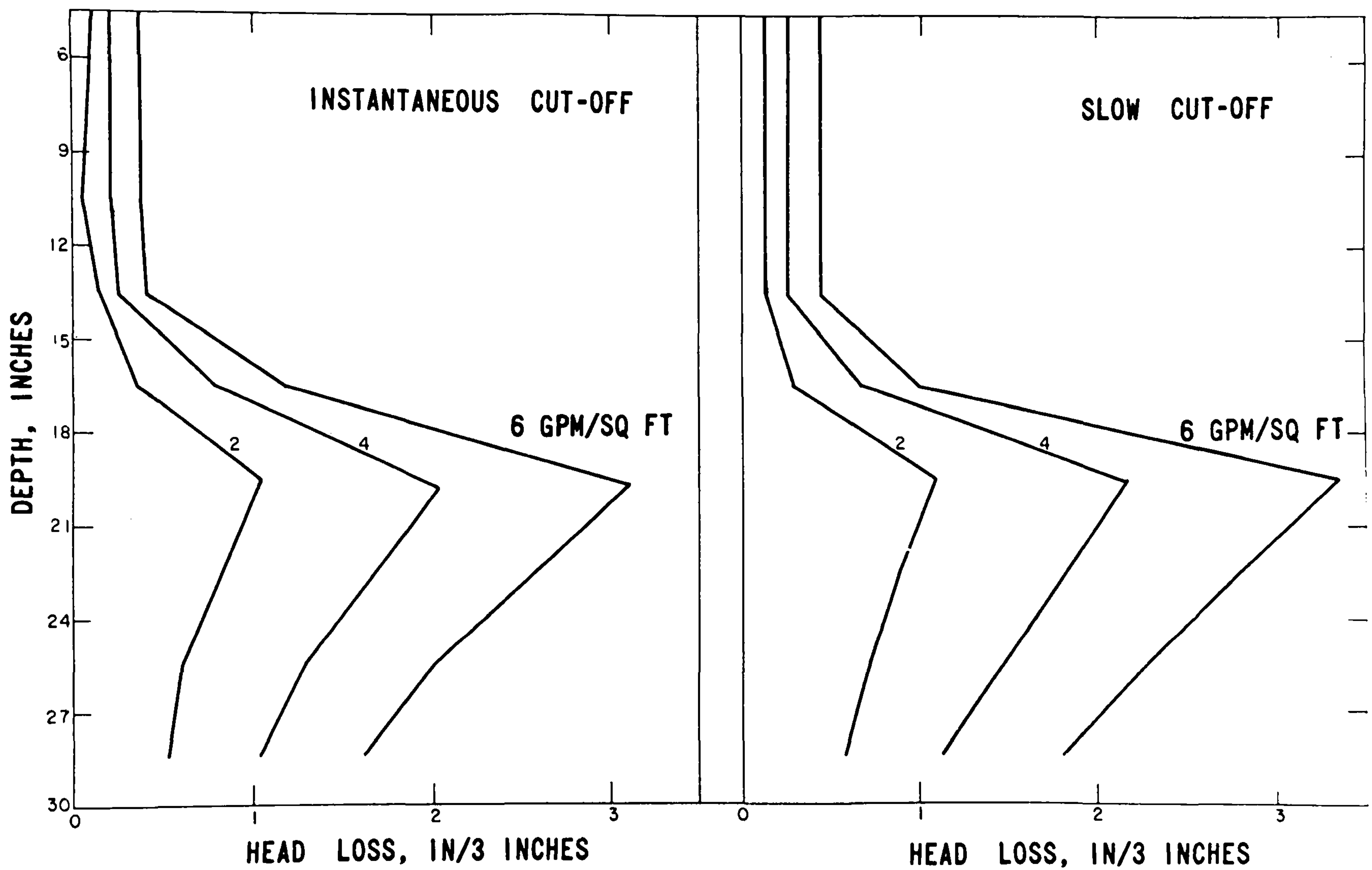

FIGURE 16. Head Loss Gradient, Dual Media.

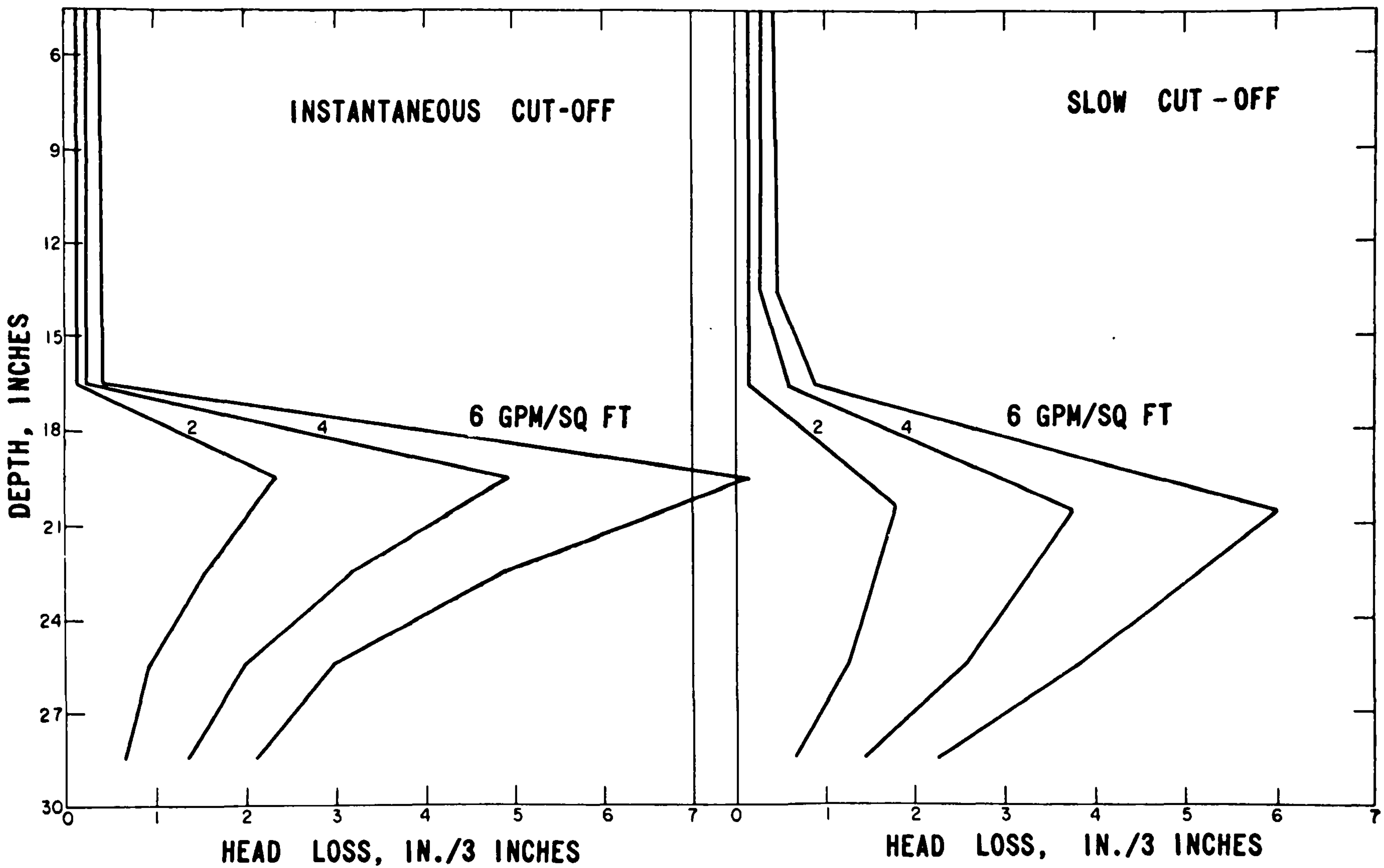

FIGURE 17. Head Loss Gradient, MM-2.

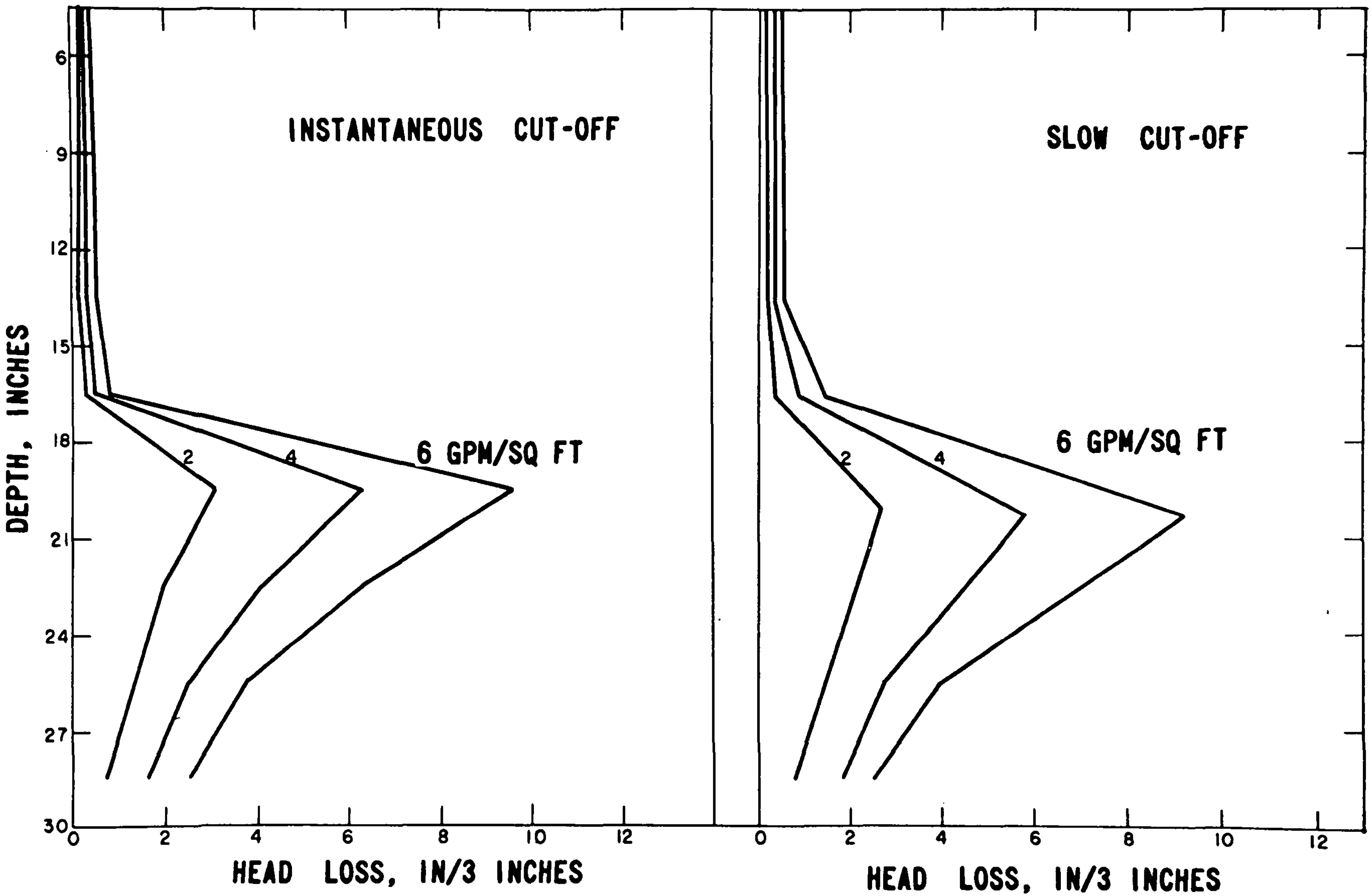

FIGURE 18. Head Loss Gradient, MM-1.

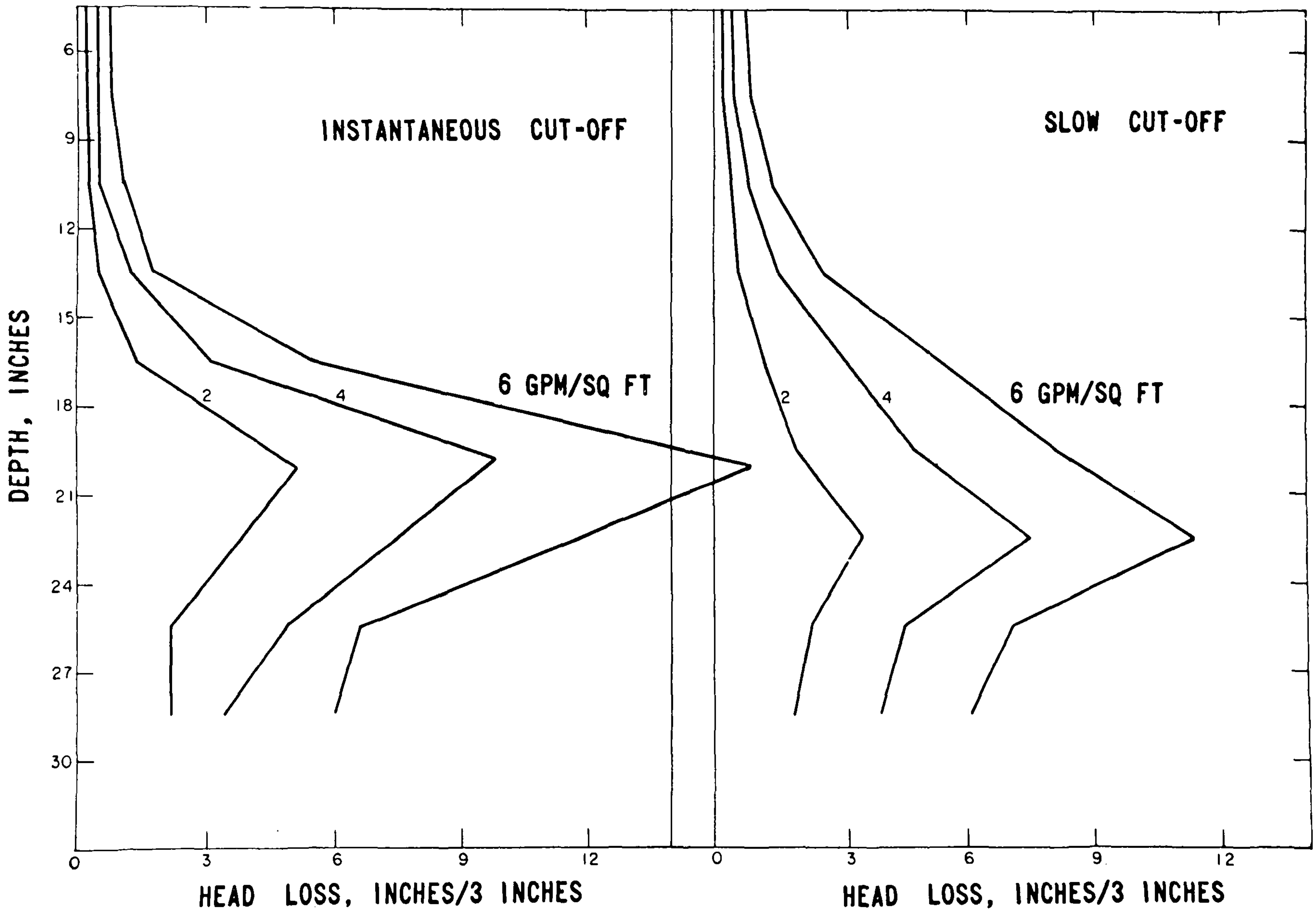

FIGURE 19. Head Loss Gradient, Dual Media Special.

were diminishing returns from using such fine sand. Preliminary results indicated that,of the 30-inch units tested, the one with 2 inches of fine garnet and 1 inch of coarse garnet, along with 6 inches of sand and 18 inches of coal, was probably the best technical arrangement. However, as indicated before, there are no full plant data to indicate that this is the most economic design or the easiest to maintain.

Previous pilot testing and economic analysis of dual systems only, indicated 18 inches of coal to be an optimal depth for the upper layer. When used for sewage plant effluent clarification, some designers extend this layer to 24 inches. Sand depth remains more debatable. This laboratory has frequently used 6 inches, others suggest 8 to 12 inches because of gravel mounding and possibly breaking up through the sand during backwashing.

Backwashing techniques can cause a noticeable shift in the point where the maximum head loss occurs. If, for instance, the backwash water is shut off slowly, the maximum occurs at about a depth of 22 inches for the bed with 12 inches of Ottawa sand; whereas, if the water is shut off quickly, the point is shifted to 20 inches and that layer is far less permeable too. (See Figure 19). Actually, this probably leaves less room for floc storage, and is thus not a preferred positioning of fines.

SUMMARY

The production of high quality water with rapid rate filters at all times, is most greatly influenced by strength of floc applied to the filters. There are probably, therefore, many filter media arrangements and filtration rates that can be considered acceptable from a public health standpoint. However, until there is a good, quick way to predict and achieve proper floc strength, regulatory groups may request one year of pilot experiments with the local water before radical changes are made in design or operation. After all, the quality of the raw water and operators may vary more than the floc strength.

REFERENCES

1. Baylis, J. R., Discussion of Test Program for Filter Evaluation at Hanford. Jour. AWWA, 52:214 (February 1960).

2. Conley, W. R., and Pitman, R. W., Test Program for Filter Evaluation at Hanford. Jour. AWWA, 52:205 (February 1960).

3. Camp, T. R., Discussion of Experience with Anthracite - Sand Filters. Jour. AWWA, 53:1478 (December 1961).

4. Robeck G. G., Dostal, K. A., and Woodward, R. L., Studies of Modifications in Water Filtration. Jour. AWWA, 56:198 (February 1964).

5. Dostal, K. A., and Robeck, G. G., Studies of Modifications in Treatment of Lake Erie Water. Jour. AWWA, 58:1489 (November 1966).

6. Conley, W. R., Integration of the Clarification Process. Jour. AWWA, 57:1333 (October 1965).

7. Hudson, H. E., Factors Affecting Filtration Rates. Jour. AWWA, 48:1138 (September 1956).

8. Hudson, H. E., Factors Affecting Filtration Rates. Jour. AWWA, 50:271 (February 1958).

9. Baylis, J. R., Experiences in Filtration. Jour. AWWA, 29:1010 (July 1937).

10. Baylis, J. R., Seven Years of High-Rate Filtration. Jour. AWWA, 48:585 (May 1956).

11. Conley, W. R., Experience with Anthracite-Sand Filters. Jour. AWWA, 53:1473 (December 1961).

12. Segall, B. A., and Okun, D. A., Effect of Filtration Rate on Filtrate Quality. Jour. AWWA, 58:368 (March 1966).

13. Hudson, H. E., Functional Design of Rapid Sand Filters. Proc. ASCE, San. Eng. Div., 89:SA1-17 (January 1963).

14. Hudson, H. E., Design Criteria for Rapid Sand Filters: Declining-Rate Filtration. Jour. AWWA, 51:1455 (November 1959).

15. Easterday, E. E. New Ideas in Filter Operation Prove Economical at St. Louis. Water Works Eng., 104:973 (October 1951).

16. Baylis, J. R., Surges in the Flow of Water Through Filters. Pure Water, 10:77 (May 1958).

17. Cleasby, J. L., Williamson, M. M. and Baumann, E. R., Effect of Filtration Rate Changes on Quality. Jour. AWWA, 55:869 (July 1963).

18. Conley, W. R., Discussion: Theory of Water Filtration. Proc. ASCE, San. Eng. Div., 91:SA2-72 (April 1965).

SECTION D

DESIGN CONSIDERATIONS REFLECTING RECENT DEVELOPMENTS IN FILTRATION

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DESIGN CONSIDERATIONS REFLECTING-

RECENT DEVELOPMENTS IN FILTRATION

William W. Aultman

The title of this symposium is "Water Filtration - the State of the Art". The questions which the Bureau prepared for me to speak on really pertain more to "Water Filtration - the State of the Science".

Prior to the early 1950's, water treatment could really be considered an art, but since that time a great number of reports in the literature have introduced the scientific approach to water treatment problems. It is a direction in which we should be going. We are developing the theory to support those practices which we have accepted for years. Actually, this symposium is being held several years too early. Later we could report more definitively on the state of the science. At the present time, we can only tell you the progress that has been made.

Research has produced advancements in the chemical and the physical aspects of coagulation and new chemicals. It has revealed the mechanics and the desirability and necessity of high energy in the flocculation process. In clarification, there are indications that deep basins are not necessarily desirable. Generally speaking, shallow basins provide better clarification and such basins are as efficient as the deep basins.

Development of filtration includes different types of filter media, filter aids, the rate, the up-flow and the bi-flow filter. Turbidimeters have been developed which are giving meaningful results. Backwashing is one of the fileds which has not been given the academic study desired such as scientific comparison of the use of water alone or water and air combination for backwashing filters. In the United States, the use of air and water wash has been negligible because, generally speaking, we have obtained satisfactory results with water alone. The European practice has been to use air and water. There is an indication that in this country more consideration is being given to the use of air in the backwashing of filters.

The use of a pilot filter in the control operations of a filter plant is probably essential if the plant contains very little or no pre-treatment capacity. However, if the floc is correctly conditioned, pre-treatment facilities are so designed and chemical feeds controlled so that it is questionable whether a pilot filter is needed. Satisfactory results can be obtained by laboratory studies such as jar tests.

As an indication of what has developed in the last 30 years, the American Society of Civil Engineers' Manual 10 - "Water Treatment Plant Design" was a small 128-page pamphlet when published in 1939. The new edition, just printed, has increased to 353 pages and was edited from about twice that size in order to produce a publication that was not too voluminous. There is more technical information being made known everyday, and it will continue to be, so that we should have fewer problems in both the future design and operation of water treatment plants.

Some of the main design considerations will be discussed briefly. Most of these are well known to all of you in this audience. The basic consideration is water quality. What is the quality of the raw water and what is the desired finished product? Next, what hydraulic capacity is required, not only for immediate needs, but for future demands? This will establish the ultimate design capacity of the plant.

One of the things which occurs often, and for which the engineer should be severely criticized, is to design a plant which cannot be *readily* enlarged. All too frequently very serious problems occur, not only in making physical connections, but also in endeavoring to keep the plant in operation while making changes. With proper pre-planning, you can design a plant so that future additions can be made with a minimum of interference with the existing plant operations and the necessity of only a very short shut-down period.

In contemplating rapid mix, consideration must be given to the energy input, and to provide for good dispersion of the chemicals in the water. It is necessary to determine what chemicals are to be used and where they should be introduced.

In the design of the sedimentation basins, the basin loading, horizontal velocity, the weir overflow rate must be considered,and whether sludge removal equipment is to be provided. In this country, wherever coagulants are used in a plant of any size, sludge removal equipment is provided. Many foreign countries do not consider sludge removal equipment desirable or necessary. Where labor is cheap, engineers and operators frequently prefer to provide two basins where one would do, and consider it no problem to shut down a basin every couple of months to take out the sludge. Each case must be considered independently.

The design of the filters is becoming more and more interesting. What should be used for: 1) maximum filtration rate; 2) type and gradation of filter media; and 3) type of under drain system. All of these factors will influence the basic design of the filters.

How is the wash water from filters and the sludge from the basins to be disposed of? In places where water is valuable, wasting of the filter wash water is undesirable. The questions that always arise are: Can the water be recovered satisfactorily? Can it be pumped directly back into the raw water? Does something else have to be done with it? A large plant seldom requiring chemical coagulation has been operated where the filter water was pumped back into the raw water supply. Some very serious problems were encountered. Whenever high plankton growths, particularly diatoms, developed in the reservoirs, many of the plankton did not settle out in the basins and a pyramiding occurred causing normal 24-hour filter runs to be reduced to two hours. Under such conditions, there was neither enough backwash water nor time to wash the filters properly. Sometimes it is necessary to provide a separate small plant to treat the filter backwash water alone so that,when it is returned to the system, it will not disrupt the treatment process.

A soon to be published report in the AWWA Journal on "The Art of Water Filtration", points out that the designer of a water filtration plant is faced with a rather difficult problem - he is to design the plant to deliver a high quality water in the amounts adequate to meet the demand anticipated some years hence, taking water from a source whose properties he knows imperfectly, and whose future changes in quality he must try to anticipate. The report further states that his design should be such that the total cost of water, including capital, operating and maintenance costs, properly evaluated over the anticipated life of the plant, will be near the minimum. The plant should also be capable of being expanded considering possible future changes in technology and water quality. The report indicates that it is probably safe to say that no treatment plant has yet been so designed. The technological interaction between pre-treatment and filtration is not at all well defined. Research is clarifying some of the relationships between variables in filter design, but considerably better knowledge is needed. There is evidence that this conference is indicative of what is said in the report.

When I was asked to participate at this meeting, somebody in the Health Department had posed a list of 21 questions which I might prepare to discuss and answer. My answers to all of these questions are predicated on the assumption that the engineer designing the water treatment facility is qualified to do so and has the knowledge, not only of hydraulics, structures, mechanics, and chemistry, but also of sanitary engineering and plant operation.

I would like to read from the introduction of the new ASCE-AWWA-CSSE water treatment plant design book, because many people in the past have thought that the old manual contained the answers to all design questions. "It must be recognized that no set of general design criteria will fit every different design problem. The data in this book has not been prepared to be plugged in by a layman to solve an individual problem. They are presented as a guide to a qualified engineer to show the consensus of opinion of recognized authorities of the field regarding the essential elements of water treatment plant." This statement is very well put. The book, as it now appears, can undoubtedly be updated and improved. Now for the questions.

Question 1. HOW CAN THE INTERRELATIONSHIP BETWEEN WATER QUALITY, PRE-TREATMENT AND FILTER DESIGN BE RATIONALIZED BY THE DESIGNER TO PRODUCE PREDICTABLE RESULTS FOR THE FILTER OPERATING ON DESIGN "FILTRATION RATE" EXCEEDING THE TRADITIONAL 2 to 3 GALLONS PER SQUARE FOOT PER MINUTE?

There are enough factual design and operating data available, if they are studied and understood, to show the general interrelationship between water quality, pre-treatment and filter design to permit a designed rate of filtration substantially higher than 2 to 3 gallons per minute per square foot of sand area.

Question 2. SHOULD ANY COMBINATION OF COAGULATION, FILTER DESIGN AND FILTRATION RATES BE ACCEPTED ON THE DESIGNER'S CONDITION OF ANTICIPATED GOOD PERFORMANCE? IF NOT, WHAT SHOULD BE THE BASIS OF YOUR DESIGN ENGINEER AND WHAT ARE SOME OF THE ESSENTIAL DETAILS?

It is not presently possible to establish any general rules regarding the necessary combination of coagulation, filter design and filtration rates to provide good performance. It is essential that the persons responsible for reviewing the design and giving the approval have a thorough knowledge of both design and operation of such plants. As an alternative, the regulatory agency must require that plans be prepared or be reviewed by engineers whose capabilities in the field have been previously proven to the agency.

Professor Kaufman discussed the possibility of using the filter alone. We have done so under conditions of good raw water and when the plant was augmenting the existing water supply. They were not for treatment of the prime sources of water. I don't believe, at the present time, the science of water filtration is sufficiently advanced that we can forecast and design a plant with filtration alone to treat a prime source, when no alternate supply is available, to take care of all conditions that might occur so that the plant would continuously produce quality water.

Question 3. WHAT PRE-TREATMENT IS NEEDED FOR RAW WATER TO BE TREATED AND WHAT IS THE BASIS FOR THIS DETERMINATION?

The basis for determining the pre-treatment needed for a surface supply would include: a sanitary survey of the watershed and a study of the historical records of bacterial, chemical and physical quality of the water source. Also, when operating records from other plants utilizing this same water source were not available, laboratory jar tests and possibly pilot plant tests to determine the satisfactory treatment method would be required.

There has been a lot said about the use of pilot plant tests. There is no question that they are desirable if time and money permit. Much can be learned from them. If pilot plant tests are carried on in cooperation with the operating personnel who will ultimately be in charge of the plant, it would give the operator a wonderful opportunity to see what happens under varying conditions of control.

Question 4. ON WHAT BASIS IS THE SELECTION MADE BETWEEN SINGLE, DUAL OR MIXED MEDIA AND WHAT ARE ACCEPTABLE STEPS?

The filter media being used is somewhat dependent on the maximum unit rate of filtration desired. If high rates are desired, dual media or mixed media filters are essential. If the filter is operating at a low unit flow rate, a single media media alone may be satisfactory. Formulas have been developed and offer a guide to sand depth for pilot plant tests and, ordinarily, that indicated depth is increased about 25 per cent in the plant filter. Sand filters have operated for years with depths of sand of 11 inches in the automatic backwash filters to over 30 inches. Dual media filters that we have designed have had from 8 to 12 inches of sand and 18 to 22 inches of anthracite coal. These depths are probably on the conservative side. We want to get a high degree of protection. The trend at the moment seems to be about six inches of sand and 18 inches of coal. The actual requirements will depend upon what quality of water is going to your filter and how the plant is being operated. It is possible to get by with a very poorly designed filter if only good water goes to it. It is a question whether the quality of water provided by a mixed media filter is sufficiently better than that produced by a dual media filter to justify the additional cost involved.

Question 5. WHEN ARE PILOT PLANT STUDIES WITH RAW WATER NECESSARY?

When the water contains undesirable constituents that may not be subject to removal by conventional treatment methods. These constituents might be such things as color, odor, iron, manganese, toxic substances, and pesticides.

Question 6. IF PILOT PLANT OPERATIONS ARE DESIRABLE, WHAT SHOULD TEST PROTOCOL INCLUDE?

Pilot plant test protocol will depend strictly on the conditions of the test. The end results must be to produce a water to meet the kind of use to which it will be placed.

Question 7. HOW NECESSARY IS IT TO CONTROL FILTRATION RATES?

The work of Hudson and others has shown that uniform flow through a filter is not necessary. A diminishing or tapered rate filter apparently produces as good quality finished water as a uniform rate operation. It may be preferable to have a diminishing rate filter. By so doing, a more uniform flow velocity is probably maintained through the interstices of the filter media as the floc builds up within the media. It has long been known that a sudden increase in the rate of flow through a filter under the present methods of operation may cause an increase in the turbidity of the water leaving the filter. The effect of these radical, rapid changes on the effluent turbidity is directly dependent upon the quality of water going to the filter. It has been noted in actual operation records that on one day, making a change in rate or flow through an individual filter, a substantial increase in turbidity leaving the filter would occur and some time later no such effect would be evident. The results apparently could be related to the quality of floc produced in the coagulating and settling basins. Parameters have not yet been developed with which to determine whether or not good floc is being produced.

Question 8. WHAT FILTER RATE CONTROLLER, IF ANY, SHOULD BE USED AND HOW SHOULD THEY BE SELECTED?

This is generally a matter of opinion. Diminishing rate filters use only an upper limit type of controller so that you won't go beyond a certain rate. Some filters are controlled by venturi or orifice type of rate of flow meter. In others, the outgoing flow is regulated by a level control butterfly valve.

Question 9. WHAT IS A REASONABLE FINISHED WATER TURBIDITY STANDARD EXPRESSED IN WHAT TERMS - AVERAGE VALUE, PROBABILITY LIMITS, ETC.? MAY THE TURBIDITY LIMIT BE VARIED DEPENDING ON RAW WATER QUALITY? IF SO, HOW SHOULD THIS BE DETERMINED?

I am going to leave this to be answered by the operating experts.

Question 10. HOW MUCH ATTENTION HAS TO BE PAID TO FILTER BYPASSING THAT MAY RESULT IN COMMON WALL, RETURN OF FILTER TO USE TO NON-FILTER WASH OPERATION, ETC.? CAN WE AVERAGE GOOD WATER FROM POOR?

The potential problems with cross-connection must always be carefully considered. The main potential cross-connection within the filter at the common floor between the upper and lower gullet has been discussed for years. The potential danger from this design is reduced or eliminated if a chlorine residual is maintained in the water entering the filters. I have never heard of any serious bacteriological contamination traced to this condition.

If the filter is adequately washed using chlorinated filtered water, there is little possibility of contamination due to not filtering to waste at the startup. It is fully realized that any contamination is undesirable, but it is also realized that all filter plant operations tend to average the quality of the finished water. The plant should operate to maintain a definite free chlorine residual in the water entering the distribution system so that the slight and remotely possible contamination that might get through a filter at the startup would be made harmless.

When we talk about achieving filter effluent turbidities of less than one-tenth Jackson Unit, and we consider the fact that the PHS Drinking Water Standards in the past have permitted turbidity of 10, or more recently five Jackson Units, I don't know how you could trace an epidemic of any sort to the filter operation where you have a filter plant operating under the conditions meeting the PHS Standards. We all know that any turbidity is a potential source of bacteriological contamination or a source of virus but, from a practical design and operating standpoint, we must be reasonably realistic. I am not accepting contamination as being permissible, but I think that we have to look at the practical standpoint from the design and operation of the filter plant.

Question 11. HOW LONG SHOULD A FILTER BE RIPENED BEFORE WATER PRODUCED IMMEDIATELY AFTER BACKWASH IS READY TO ENTER THE DISTRIBUTION SYSTEM? MUST THIS FIRST WATER BE WASTED OR CAN IT BE USED FOR FILTER BACKWASHING PURPOSES?

From my experience in operating a plant, when a filter is properly backwashed, no ripening is necessary. The pre-treatment must be proper and the filter must be in good shape.

Question 12. HOW CAN THE ADEQUACIES OF THE BACKWASH SYSTEM OR BACKWASH PROCEDURE BE DETERMINED? WHEN SHOULD A FILTER BE BACKWASHED?

We need more research on this matter. At the present time, adequacy of backwashing is determined visually. Over a long time, you may want to observe the condition of the filter as a result of the backwashing procedure that you are using. If you find an accumulation like mud balls, you are not backwashing properly. There is no direct answer to this question. When should you backwash? This is determined by two criteria: head loss and turbidity. The turbidity should really control. Due to plant design, however, the head loss may control. The ideal situation is to have the filter reach the design head loss and allowable turbidity at the same point.

Question 13. WHAT AUTOMATIC FAIL-SAFE CONTROL AND WATER QUALITY MONITORING EQUIPMENT SHOULD BE PROVIDED?

The extent and type of fail-safe control and quality monitoring equipment required will depend upon the design of the plant and the method of its operation. There is no question but that all plants should be designed to be fail-safe. If something should go wrong in some part of the process that might seriously impair the quality of water produced, controls should be provided to either alert the operator of the condition, or to shut the plant down if there is no operator in attendance. To provide good continuous operation of the plant and provide a record to show that you are putting into the distribution system a safe water, a chlorine residual recorder on the pipeline delivering water to the distribution system is desirable. Since the equipment now being produced is reasonably free of operating difficulty, they should be used more.

Question 14. MUST NEW PLANT DESIGN BE LIMITED TO LARGER PLANTS WHERE SOME SKILLED OPERATION IS AVAILABLE?

No. The new concepts of water treatment plant design are available for any size plant. This is evidenced by the large number of small plants that Micro-floc has installed in recent years.

Question 15. WHAT TRAINING SHOULD THE OPERATORS RECEIVE? SHOULD THE TRAINING BE PROVIDED BY THE DESIGN ENGINEER?

Operators should have basic training provided by the utility or by means of school instruction. The designing engineer should clearly advise the person in charge of the plant about the basis of design so that full utilization will be made of the facilities provided. It is very desirable to have the man who is to be in charge of the plant operation be available as an inspector during the construction period to know what is in the plant and what it should do.

Question 16. WHAT KIND OF A RECORD SHOULD THE PLANT OPERATOR MAINTAIN? WHAT EVALUATION OF THIS DATA SHOULD BE MADE AND HOW OFTEN?

I think that the operator should answer that question.

Question 17. WHAT EVALUATION SHOULD BE MADE OF A NEW FILTER, OR WHAT PERIODIC EVALUATION OF AN EXISTING FILTER, IF ANY, SHOULD A HEALTH DEPARTMENT MAKE?

After the initial shakedown period of a new filtration plant, the health department should observe the plant operation and examine the operating records. A number of states require the submission, monthly, of the pertinent operating records for review.

Question 18. SHOULD THE WATER PURVEYOR OR THE DESIGN ENGINEER BE RESPONSIBLE FOR DETERMINING THE OPERATION REQUIREMENTS OF THE PLANT SUCH AS COAGULANT DOSE, THE USE OF COAGULANT FILTER AIDS, AND BACKWASHING SHOULD BE ACCOMPLISHED AND HOW BACKWASHING SHOULD BE PERFORMED, ETC.?

It is essential that the design engineer establish, in advance, the anticipated method of operation, chemicals to be used, minimum and maximum rate of chemical dosage, and normal filter operating procedures. A plant that is properly designed will provide great flexibility in operating procedures for development of the future cannot be foretold today. An example of this is the Weymouth Plant of the Metropolitan Water District developed in 1938-39 and placed in operation in 1940. This plant has been able to keep up with modern developments without any substantial change in the plant because it was made with great flexibility of operation.

The plant operator is the only one who should determine the operational requirements of the plant after it is put into service, provided that the plant operator is trained and qualified to make such decisions. In small plants, the water purveyor should retain the services of a consultant, well qualified in plant operation, to provide regular supervision of the plant operation. The consultant may or may not be the engineer who designed the plant.

Question 19. SHOULD THESE OPERATION REQUIREMENTS BE DETERMINED BEFORE APPROVAL IS GRANTED BY THE HEALTH DEPARTMENT?

Unless the health department makes a thorough study of the complete project and review of the operation requirements established by the engineer, approval by this agency would be meaningless.

Question 20. SHOULD HEALTH DEPARTMENTS ACCEPT ANY FILTER DESIGN AND FILTRATION RATE AND DEPEND UPON ADJUSTMENTS IN USE OF COAGULANTS, FILTRATION RATES DOWNWARD AND EARLIER CONSTRUCTION OF ADDITIONAL UNITS TO ASSURE ACCEPTABLE PERFORMANCE?

No. Reasonable design parameters have been included in the new book on water filtration plant design. These parameters should be used as a guide by the health department engineer in determining acceptable design standards for a specific plant. The actual operating results obtained at the plant should be determined by a regular review of the plant operating records by health department engineers.

Question 21. SHOULD THE HEALTH DEPARTMENTS ABANDON THEIR TRADITIONAL DETAILED REVIEW OF PLANS AND SUBSTITUTE A STRICT PERFORMANCE STANDARD BASED ON CONTINUOUS MONITORING OR TURBIDITY AND OTHER PARAMETERS?

Review by the health department should consist of a careful study of the general design criteria for the plant, a review of the plans and specifications for a few specific items such as: potential cross-connections, chlorination facilities, etc. It might be reasonable for the health department to set up some requirements regarding continuous monitoring and recording of such things as chlorine residual, fluoride, if added, and turbidity.

In conclusion, I would like to say very clearly that it must be realized fully that these ideas are the opinion of only one consulting engineer. It is entirely possible that we can get as good an argument between two consulting engineers as you frequently do between a consulting engineer and a health department engineer. Both -- or neither -- may be right.

SECTION E

OPERATIONAL ASPECTS OF MODERN FILTRATION PLANTS

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OPERATIONAL ASPECTS OF MODERN FILTRATION PLANTS

Gordon L. Laverty

A short time before the development of the rapid sand filter, Rudyard Kipling wrote as poet laureate of England some words appropriate to a great event of the empire:

> "The tumult and the shouting dies;
> The Captains and the Kings depart;
> Still stands Thine ancient sacrifice,
> An humble and contrite heart."

Rudyard Kipling's admonition to England's rulers should be received kindly by human beings involved in 24-hour-a-day water supply engineering and operations. For these words could describe the birth and later workings of a modern filtration plant. The "Captains" and the "Kings" are of course the developers of theory, the Site Selection Committee, the design engineers, the specification writers and those who take us to the point of contract award.

The "tumult and the shouting" usually come at contract acceptance time and may continue to plant dedication ceremonies. The "ancient sacrifice" and the "humble and contrite heart" are left to those who operate the facilities left to them by the "Kings" and the "Captains" who have gone on to other frays and other monuments.

The "Captains and the Kings" depart to greater developments and other fields, while the operators remain to work 24 hours a day to provide humanity with the water of life under a legal obligation which may strain the capabilities of the monument.

My discussion today of water filtration as a part of the water purification process will be in the context of man in between a facility he is given and the law which requires zero-defect performance.

The operator generally does not get the plant that he would create, were he the "Captains and the Kings". He must start out with the plant the way it exists the day he takes it over and from that point determine what he can do with what he has. Having arrived at this point, he then determines what he could do to improve upon the plant he has been left. Giving attention to the water filtration process at the plant he finds that he is again in an in-between situation. The filter box and filter media, the underdrain system and backwash facilities, are in between the pretreatment works and the disinfection and distribution facilities. Thus the water filtration process and its operator are not free agents but are at the outset the inheritors of conditions often beyond their control. Whatever has gone wrong in the storage reservoir, whatever aeration has not been resolved, whatever chemical treatment has not remedied, and whatever imperfection or disturbance in the sedimentation process, all affect the filtration processes.

The range of performance of the filter, on the other hand, is limited also by what comes after it -- the degree to which disinfection is effective at the plant, and the contact time provided in the distribution system for disinfecting materials prior to consumption. The limitations placed ahead of and behind the filtration system lead to the suggestion that the filtration process must operate within the following framework:

1. *DESIGN.* Design should include applied water entry not disruptive to media and media top-of-bed buffer areas to absorb applied water quality changes. The prime filter area itself involves the media "where the action is" on which reliance is placed to remove materials which one does not wish to pass the filter. A support area supporting the filter media and an appropriate backwash area which permits proper maintenance of the media and some assurance of longevity of the media.

2. *OPERATION.* Operation should include assurance that the design concept and plant limitations are understood by operators. This presupposes

some form of communication between the designers of year "X" and the operators of a later year "Y". It presupposes trained, alert, imaginative operators who are motivated to constantly look for cause and effect as raw water or plant chemistry or physical phenomena change -- while wondering how the fish are biting at Lake Oudago, or where he can park his camper next weekend. It presupposed, in short, that the brain of the designer has somehow been left at the filter console when the mind of the operator is on vacation.

3. *PERFORMANCE STANDARDS* may be the salvation of the day. For only by a periodic, consistent check on performance of man and filter can the ultimately responsible supervisor be sure that water quality standards will be met.

Performance standards for filters are easily arrived at by setting parameter limits after observing filter performance under different operating conditions over a period of time. These limits are based on measurable items such as effluent turbidity, turbidity reduction, rate of head loss buildup, and flow rate versus head loss or plant performance cramping parameters.

At East Bay Water's six plants, for example, we arrived at these standards for our plants:

a. Effluent turbidity should routinely be less than 1 ppm at coagulating plants, and never greater than 5 ppm at any plant.

b. Filter rates should be close to 75 percent of the optimum rate for a filter.

c. The number of filters in service should be such as to permit a 5 MGD rate increase without putting another filter in service.

d. One filter is usually operated at the design or an agreed-upon standard rate for performance comparison with other filters at the plant.

e. Periodic microscopic examination of effluent samples.

Standards numbers have yet to be set.

Filter backwash performance standards are also an "artistic" balance between theory and test experience.

a. Wash filters at a 6-foot loss of head unless one of the following conditions prevails:

   (1) Filter run is in excess of 144 hours.

   (2) Effluent turbidity is noticeably increasing. (This requires taking a sample if a filter is marginal in hours run or head loss but is in competition with another filter for wash effect.)

   (3) Effluent has demonstrated taste or odor.

   (4) Rate controller is wide open, or flow rate is decreasing.

b. Backwash should have a maximum duration of 5 minutes per filter.

c. Wash water percentage may range from 1.5 to 3.0 percent.

Turbidity of plant effluents is based on Hach Recording Turbidimeters installed or presently on order.

Filters are generally washed at night during minimum draft periods. This poses the problem of placing filter "aging" after backwash on one shift and is a major reason we installed or are installing recording turbidimeters on all plant effluents.

In answer to the question raised by the organizers of this symposium, let me say this:

The interrelationships between raw and effluent water quality, pretreatment, and filter design cannot in my judgement be known in the design stage for *any* filter rate unless model studies are used during design. Plant geometry affects mixing of chemicals, short circuiting through basins, and flow disparity at Y's in channels which in turn affects filter performance more than the construction or flow rate of the filter.

For administrative purposes it will be in the future quite difficult to set up arbitrary standards for "design" rates for a given water utility's new or modified plant proposal. I know of few other fields where specifics are so important and localized as in filtration of one water as compared to another. The burden of proof for approval of significant departures from generally accepted practice should rest upon model testing using the water to be treated if at all possible.

What treatment a particular water requires varies from season to season and year to year. The basis for determination should be a system of logic starting with finished product standards and then step by step economic alternative and design considerations tied back to the raw water quality and its fluctuation.

Media selection will depend upon personal education, experience, consultant advice, economical or political considerations. The manager who arrives at an economical and logical conclusion as to single, dual, multiple or mixed media should have tested all options if possible prior to decision. The basis for final selection will relate to manpower economy, raw and treated water quality reliability and instrumentation, among others.

Media depths are important for adequate support, wash water distribution and safety factor. I believe that while grain size is more significant than depth, one will not generally feel secure with less than 24 inches of gross media, whatever the makeup.

Pilot plant studies using raw water are indispensable to good filter design if the raw water supply exists at design time.

Rate controllers are a must if the operator is to feel that he has the plant under control. Controllers offer a rationale to operators and reduction of rate "hunting" problems which otherwise are lacking.

Plants in operation today are averaging water of lesser quality with water of better depending on filter wash cycles. Filter ripening is not of great significance and is generally opposed by busy operators.

Many plants filter to waste enough to pass the clean wash water to waste, believe the filter to be ripened and then put the filter in line. We have run tests that indicate that up to 9 hours can be required to "ripen" a filter.

By all odds, chlorination is the most significant fallible function of a filter plant. Therefore, "fail-safe" indication and alarm of this function's failure should have priority. Turbidity monitor and alarm also rank high in such priority.

Application of "new" plant design should not be limited by plant size, but rather by the initiative, desire-to-do, record keeping, knowledge and training of the utility staff and operators.

Plant operators require instruction and reinstruction to maintain levels of performance. It is rare that the design engineer can do more than get a new plant started. Smoothing out operations can require months, and until data can be collected for review after a period of operation there is often no knowledge upon which to base operations changes.

Plant operators should maintain key parameter records: turbidity; chlorine demand; dosage and residual; chemical dosage and filter rate and loss of head; hot and cold taste and odor. Evaluated by comparison to *plant performance standards*, these can mean something to the on-shift operator.

Filter evaluation, filter by filter, by a health department would not be profitable since the purveyor is primarily responsible for the *as delivered* product. But required reports of filter or media change plans should be discussed with health department engineers prior to going to contract.

The water purveyor should be responsible for determining operations requirements for a plant except in the case of small utilities which retain or hire consultants because they have no staff of their own competent to perform. Operations requirements should be determined before health departments grant approvals. Otherwise needed equipment, personnel or other unpredictables are not included in early planning.

"Acceptable performance" by a proposed filter design should be judged by the health department. After all, the only justification for so-called progressive shortcuts is economics at the usual expense of reliability. Hence, the "sole" of progressive, money-saving new designs which have failure-of-quality risks higher than accepted standards at a given time should require strong documentation.

Health departments should maintain the prerogative to either in detail review plans or await construction and rely on performance based upon the variables in each water plant design case.

The economics of design and operation cannot be ignored. But it is obvious that in the future increasingly stringent water quality standards will require higher levels of expenditure for both chemical treatment and sophisticated plant operations.

SECTION F

OPERATIONAL ASPECTS OF MODERN FILTRATION PLANTS

W. L. Harris, Water Production Superintendent
Contra Costa County Water District
Concord

OPERATIONAL ASPECTS OF MODERN FILTRATION PLANTS

W. Leslie Harris

I appreciate the opportunity to appear on this panel as an operator and to present along with Mr. Laverty some of the aspects of our current interest in providing better filter effluent quality while operating at high rates.

The Treated Water Division of the Contra Costa County Water District has had a continuing interest in advancing the art of water filtration. This interest goes back to the creation of the Division in 1961 to operate properties purchased from the California Water Company and serving a large area of North Central Contra Costa County.

Steps were taken in 1962 to provide greater reliability of a seasonally operated plant and at the same time produce better water at higher filtration rates. A portion of this work involved rebuilding the five filters and substituting a dual media of sand and anthracite. Facilities were installed for the addition of a nonionic polyelectrolyte to the applied water and for the measurement and recording of the plant effluent turbidity. In the same year, an agreement was reached with the MicroFloc Corporation of Corvallis, Oregon for a four month pilot plant test program at our local plants. This project and the plant renovation were both successfully completed in 1963.

The renovated plant operating at 6 gallons per square foot per minute produced better water than had heretofore been possible and at that rate the capacity was ample for many years to come. The MicroFloc process as demonstrated at two of our plants was found satisfactory but did not go beyond this phase. It being determined that the resources of our consultants and our own organization were ample for the upgrading work that was being done.

One of the side benefits which the upgraded plant and distribution system improvements provided was the elimination of any further need for a small pressure filter installation used for peaking. This unit with its raw water supply from an undependable seasonally operated canal lateral and operated by hastily recruited help was far below the standards which we desired to maintain. The plant was never used by the Treated Water Division.

In 1964, the Treated Water Division again struck out in two directions in its drive for improvement and added capacity. It provided a nine square foot experimental filter for a research program and started ahead on doubling the capacity of its main plant using dual media. The added age of the plant along with the scattered approach to past additions made the job more involved than the first. The 100 percent increase in raw water supply had to be brought in from an opposite direction and the added supply when filtered had to be pumped direct from the filters. In renovating the eight filters we used new gravel but found from our studies that the existing sand would be satisfactory. This sand had an effective size of .55 mm as compared to the .41 to .45 mm effective size specified for the first plant. We also varied the respective depths of both sand and anthracite so that upon completion we had five filters at the seasonally operated plant with 6 inches of .43 mm sand and 20 inches of anthracite, 4 filters with 15 inches of .55 mm sand and 18 inches of anthracite. The anthracite in all filters being what is known as Grade 1 1/2 or #1 Special with an effective size of .85 mm. All filters were equipped with rotary surface wash units. At no time over the ensuing years were we able to discern any advantage in one class of filters over the other two and while it was beginning to be evident that our water filtration conditions were less critical than many it was also apparent that the dual media requirements allow considerable latitude in sizing and depth relationships.

The conversion of the existing plant was completed for the summer of 1965 and discussions started almost immediately on the requirements for the first unit of a new plant which would have an ultimate capacity of 250 mgd. The rapid course of events made it essential that a decision be made in a relatively short time on many basic features of design. It was therefore necessary to select a specific area of investigation for our experimental program if we were to develop any new concepts for inclusion. We had already evaluated an air and water backwash method and found it lacking. This in no way was a blanket condemnation of air scour systems, but time did not allow for varied trials. We had used garnet as an added material in a mixed bed filter and our observations gave us some concern. Since its inclusion at a later date, if found desirable, would involve no major design changes, it was logical to move on to other areas of investigation. This investigation along rather specific lines indicated that the major design change in planning for a new plant should be to provide for higher rates of filtration. Our work indicated that, in the season when peak demands are made on our facilities, there would be no problem in meeting AWWA goals on water quality as measured by filter effluent quality when operating at 10 or more gallons per square foot per minute. The adoption of such a rate would move the maximum capacity of the first unit from 50 to 80 mgd and enable the abandonment of the other plants and the remaining wells which at one time numbered 22.

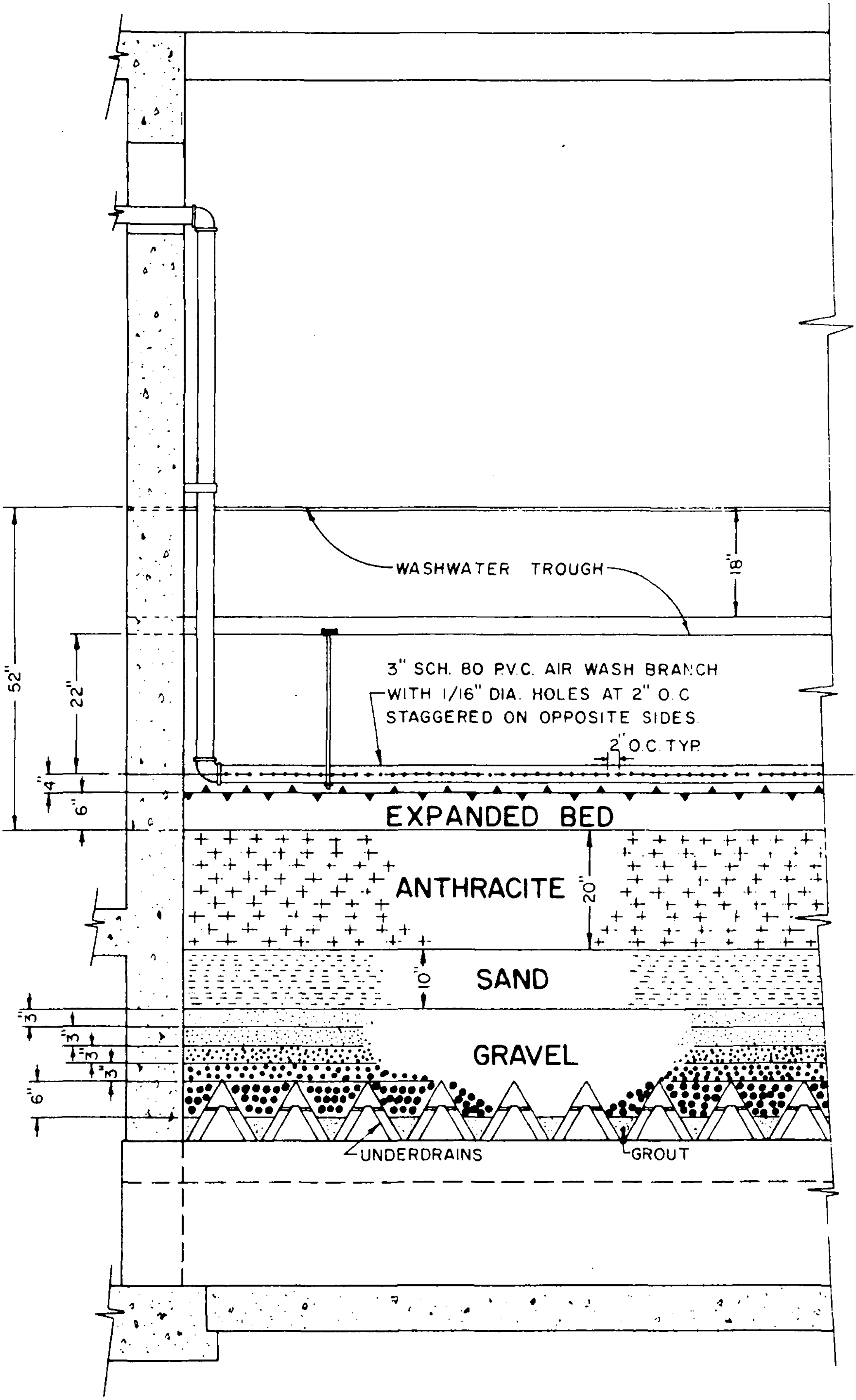
The plant design which was well along in its preliminary phase was changed to provide for this higher rate. Whereas the ultimate number of filters had been 20 the number is now projected at 12. The maximum hydraulic rate per square foot became 11 gpm and the capacity of each filter moved from 12 million to 20 million gallons per day in proportion to the rate change from 6 gpm per square foot to 10. The potential of the new plant was also increased by making possible greater loading of the settling basins. We had established that the filters could readily handle higher turbidity values in the applied water while producing water of the same effluent quality. Hence, the hydraulic capacity of the two settling basins for the initially planned 50 mgd unit was raised to 125 mgd. At this point, we were at variance with widely accepted tenets that effluent turbidity varied directly with the applied turbidity and with the rate of filtration.

In moving to new high ground in the matter of filtering rates it is appropriate to consider the background for the long used standard of 2 gpm per square foot. Cleasby and Baumann[1] have pointed out that George W. Fuller is usually given credit for establishing the standard of 2 gpm per square foot rate and that this early researcher working with proprietary devices had no control over the rates used and that later he was restricted to less than 2 gpm by the physical limitations of the apparatus. Further that he was aware he had not reached the upper limit of desirable filtration rates. It so happens that I started 40 years ago in a George W. Fuller water treatment plant and, while there had been many changes since the plant was designed 20 years previous, the vestiges of the past remained. It is worthwhile in this discussion of water filtration to compare practice at the time of start-up in that plant to that in effect at the new plant which we have operated since May, 1968. Marginal chlorination using manually controlled solution boxes was practiced in the old, whereas the new plant uses much higher doses proportioned automatically to rate. While the old plant depended upon unstable chloride of lime, the new plant has gaseous chlorine immediately available to injectors at the point of application. The old plant had taken on the added task of partial softening with lime oblivious of the reduced effectiveness of chlorine at the higher pH while,on the contrary, the new plant prechlorinates at pH 7.0 for a 40 to 50 fold advantage in germicidal effect. The coagulant feed in the old plant dropped into a large cavernous chamber with no thought given to the efficiency of the reaction whereas, in the new plant, 2 rapid mix units with a total of 30 horse power provide immediate dispersal of the coagulant. Black, Riddick and others have provided operators with a working knowledge of coagulation theory based upon Zeta potential measurements. The new plant is equipped for this work and hence in a position to secure maximum benefits from its coagulant feed. The old plant had no such advantage and felt no concern over the effect of higher pH floc strength and residual alumina. Our plant today keeps up the high energy input as the water moves through the distribution channel by using air from two 5 H.P. blowers. In each of the two flocculation areas, three 10 H.P. variable speed drives to turbine type flocculators and one 3 H.P. variable speed drives to two conventional reel flocculators complete the installation for mixing and flocculation. Thus 106 horse power is available to effectively mix and flocculate as compared to no mechanical installation sixty years ago. Mechanical removal of sludge is now very nearly universal but many years elapsed before there was any start to get away from the lost capacity of basins partially full with sludge or out of service for cleaning. Filters now wash and operate automatically to eliminate many short comings of manual operation. How many years have elapsed since operators with no awareness of hazard "bumped" the manually operated filter to extend its run? There are numerous other improvements in today's plant but the important truth to remember is that the old plant did its job well. The plant to which I referred started up in a community burdened with one of the highest typhoid fever rates in the state and its operation accompanied by an intensive drive to eliminate contaminated wells reduced waterborne typhoid fever rates to zero. It is therefore logical to consider what weight should be given to the improved state of technology in possibly setting a new standard rate of filtration. A serious disadvantage to having a standard is that it tends to polarize our attitude towards what is acceptable in the best interest of public health. Thus, little or no concern may be felt regarding a plant operating at one or two gpm per square foot whereas some plant producing the same quality water but at rates well above the standard may cause unnecessary concern. Perhaps what is needed is a production rather than a design standard. Regardless of what finally evolves, I believe we all agree that the 2 gpm standard is shattered, that there is a restless desire to improve filter effluent quality and that there are compelling financial reasons for maximizing production from any given facility.

In setting and attaining our own high production standard, we were aided tremendously by the experimental filter program. Operators who became involved found themselves giving more attention to the quality of the water produced in the plant proper and to the effects of varying raw water conditions and treatments. Reeser[2] has already noted the requirements for a better qualified operator in today's high rate water treatment plant. In addition to the operator benefits, the program enabled the development of three concepts which are distinctive and exclusive in the new plant.

Surface water scour units have been replaced with an air lift system for greater removal efficiency. If deep penetration of the filter is effected, then the surface caking feature common to sand filters should not be present. It was the surface cake with an attendant cracking and mud ball formation on sand filters which led to the development originally of the surface wash. On the other hand, coal has been known for the better self cleaning properties caused by its angularity. We observed no need for breakup of the material separated from the filter in backwash. There was, however, a need to complete this removal by an effective higher rise rate. Also, there was an advantage in maintaining a water-media interface free of fines. All this we found could be accomplished by an air lift system as shown in Figure 1. In practice, the filter in backwash comes up to full backwash rate of 30 inches rise. Partial closing of the washwater valve to provide the rise dictated by the water temperature follows and,at this time, air is introduced above the expanded bed. Currently the rise is 24 inches and is very nearly matched by the air flow of just over 2 cubic feet per square foot of surface. We have noted the following advantages:

1. Less washwater consumption due to;
    a. lower rise rate,
    b. shorter period of wash,
    c. longer filter runs.



HARRIS AIR ASSIST

FIG. I

2. The cleaner filter contributes to less start up turbidity.

In a second area, we have moved to provide more void space in the upper reaches of the filter bed to augment its more open face. In the development of the mixed bed filter, the industry has directed its attention toward the introduction of materials having higher specific gravity and smaller size in the lower reaches of the filter to prevent passage of fine material and breakthrough. The top layer of coal remained very much the same and was free to grade hydraulically with the fines on top. We moved away from this arrangement by using coal of two distinct specific gravities and sizes. Starting with a #2 size, 3/16" X 3/32", low specific gravity coal we had an upper filter layer with excess void space but an improved water-media interface. We then introduced high specific gravity, size 1 1/2 coal to reduce the void space for the control needed in effecting near uniform removal of turbidity. As constituted at this time, the filters at the new plant have 2/3 light weight #2 size coal and 1/3 high density #1 1/2 size above the sand. Operation to date has not demonstrated any need for materials more heavy than sand to prevent passage of fines and to prevent breakthrough. Credit for this attainment is due to great extent to the third distinctive feature of the plant involving conditioning of the filter media.

When a polyelectrolyte is used in the applied water, it soon becomes apparent that a portion of the benefit derived is in conditioning the filter. The first runs when this technique is employed are not typical of what to expect subsequently and,if the application is terminated, the following run will show some of the effects of the residual material. To improve upon this effect and distribute its action more uniformly, the plant adds a polyelectrolyte to a portion of the washwater. While the total quality of polyelectrolyte used is less than would otherwise be needed, it is not the economy involved which governs. Rather, it is the ability of the resultant filter to:

1. Produce high clarity water from the start.
2. Accept major surges in rate without breakthrough.
3. Go to maximum head loss with no deterioration of quality.
4. Provide longer runs because of the reduced polyelectrolyte in the applied water.

Conley, in an early discussion, noted the residual effects of polyelectrolytes in the applied water. It should also be mentioned that Woodward[3], in his discussion of Hudson's paper on "High Quality Water Production and Viral Disease", states the case for improved initial clarity in the filter effluent even to the detriment of routine operations by filtering to waste at the start of a run. We feel that our technique makes this step backwards unnecessary and that herein lies part of its value. (An application for patent on this process has been made).

Several filter plants were visited in the Bay Area to verify our belief that treatment techniques and not rate determine effluent quality. Table 1 lists the pertinent data and initial effluent turbidity for these plants plus our own Bollman Water Treatment operation. During the visits, effluent turbidities were also checked for a period covering the first 200 gallons filtered per square foot. The choice of quantity rather than time eliminated the bias introduced by varying filter rates for a fixed period. The results are shown in Figure 2 which is based on the data presented in Table 1. The information gathered is limited to a few plants but it does indicate strongly that the effect of rate is negligible or nonexistent when compared to treatment and technique employed.

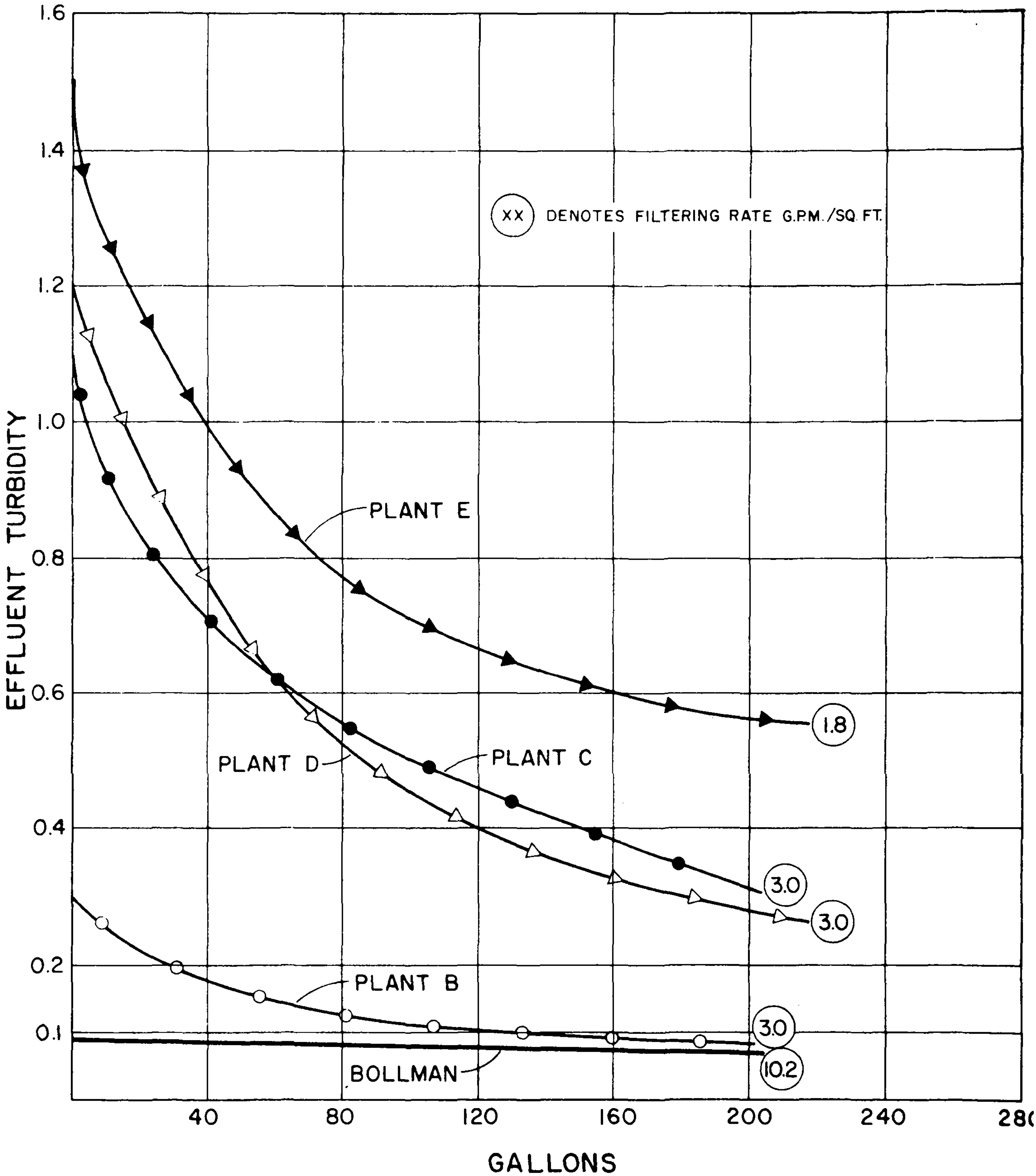
The Harris Filter Technique now enables us to accept major surges in rate without break-through. Figure 3 illustrates the absence of effect on effluent turbidity when moving from a 2.5 gpm/sq ft rate through progressive steps, which are approximately 100 percent of the initial rate up to

10.2 gpm/sq ft. Also included are the results of starting a run at 10.2 gpm/sq ft and then dropping to 5.1 and eventually to 2.5 gpm/sq ft. A set amount of 600 gallons was used for each rate.

TABLE 1

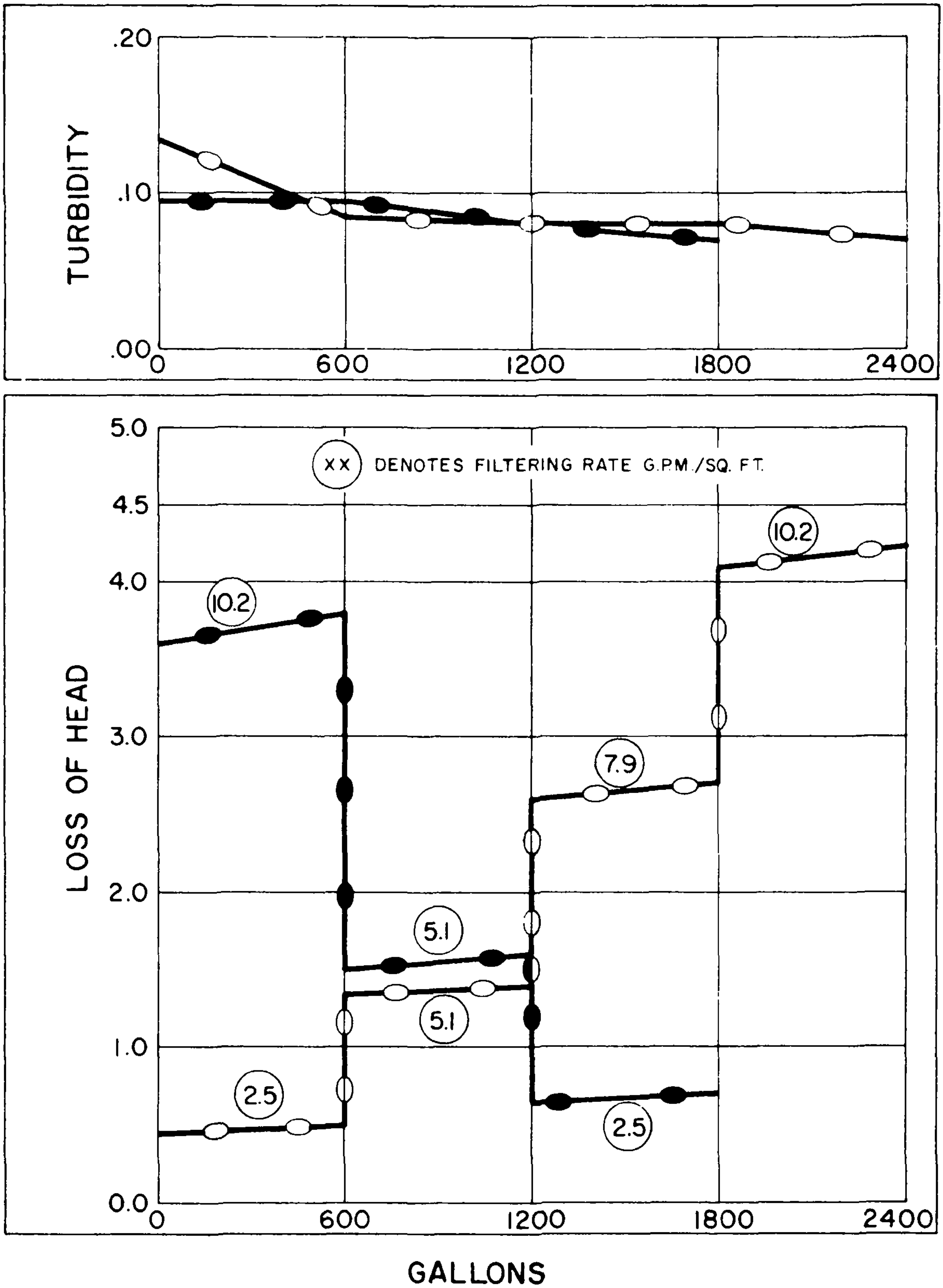
Plant Operating Data and Initial Effluent Turbidity

Plant	Alum PPM	Coag. Aid PPM	Filter Aid PPM	Media	Harris Filter Technique	Rate GPM per Sq.Ft.	Initial Effluent Turbidity J.U.
Bollman	35	0	.008	Sand & Coal	Yes	10.2	.09
B	48	0	.020	Sand & Coal	No	3.0	.29
C	25	0	.000	Sand & Coal	No	3.0	1.10
D	0	0	.020	Sand	No	3.0	1.20
E	60	.5	.000	Sand & Coal	No	1.8	1.50



INITIAL EFFLUENT TURBIDITIES AND FILTERING RATES

FIG. 2



RATE VERSUS LOSS OF HEAD AND EFFLUENT TURBIDITIES

FIG. 3

1538

REFERENCES

1. John L. Cleasby and E. Robert Baumann. Selection of Sand Filtration Rates. Jour., AWWA May, 1962.
2. David M. Reeser. Operational Experiences with Dual Media Filters at San Francisco Water Department's Sunol Valley Filtration Plant. Paper presented October 26, 1967 at California AWWA Annual Conference in Los Angeles.
3. Richard L. Woodward. Discussion of Herbert E. Hudson's Paper "High Quality Water Production and Viral Disease". Jour., AWWA, October, 1962.

SECTION G

DISCUSSION AND QUESTIONS

DISCUSSION AND QUESTIONS

Morning Session

Question: WHY DOES THE 1968 REVISION OF THE 10-STATE STANDARDS MAINTAIN THE FILTRATION RATE OF 2 GALLONS PER MINUTE PER SQUARE FOOT?

Robeck: I cannot fully speak for them, but I imagine that they want to have some control over future changes by starting at that level and putting the burden of proof of any changes upon the utility. For example, we have worked in Pennsylvania and Ohio where we were able to obtain approval for higher rates after six months to a year of tests. At Erie, they were so strict that we had to waste 13 million gallons per day out of the filters because we were propsoing to go at a higher rate even though we were monitoring the water to determine what happened to quality at all times. They had many other filter plants in the state where they did not know the head loss, the flow rate through them and certainly no knowledge of the turbidity, yet they were operating at 2 gallon rate. With a little education, they allowed use of this water which turned out far better.

Question: ARE THERE ANY MINIMUM TIME REQUIREMENTS AFTER YOU ADDED THE COAGULANT BEFORE YOU ADD THE POLYMER?

Kaufman: I have not done studies on adding aids after the coagulant, but I am sure there is a factor of time and energy. In other words, depending on how much stirring and energy is imparted to the flocculating process, it would influence how much detention period to provide for settling. Further, it is going to depend a little on the nature of the floc chamber.

Question: WHAT COMMENT DO YOU HAVE REGARDING THE SO-CALLED HIGH ENERGY MIXING?

Robeck: Up to a point, increasing energy into the flocculation process will improve the performance of the sedimentation basin. The extent to which one can increase energy depends upon the strength of the floc that is produced. In one study, we found that, by adding a small amount of Dow C-31, we could increase rate of flocculation and increase mixing two or three times without getting breakup of the particles. I think, traditionally, we have used much too slow energy and we could probably double it at any plant and get better performance. We have been able to verify some of Hudson's report, but we found the performance was not as remarkable as he had found. So, it would be well for you to check this out yourself.

Question: HAVE YOU EXPERIMENTED USING A POLYMER DIRECTLY AHEAD OF THE FILTER AS A FILTER AID?

Robeck: Most of our work was done adding a polyelectrolyte just ahead of the filter. We call it a filter aid, but the editors in New York are constantly crossing this out and changing it to coagulant aid. We are trying to make a distinction from what is used up at the head of the conventional plant in a rapid mix. This may be questioned because, in a long run, it does aid the coagulation that might take place in a contact medium arrangement as the water passes through. You only need filtration when you have fluffy, weak floc going over from the settling basin to the filter. A filter aid helps coagulation take place in the filter. You should check it out very carefully if you are treating an alkaline water. We have found that you have better floc when you do this with newly formed floc because it sticks in the filter better than the weaker aged floc that comes out of the settling basin.

Kaufman: Where the polymer is being added prior to filtration rather than as an aid to flocculation, it might be of interest to consider putting in a flash mixer at that point following the sedimentation basin with two objectives:

a. to break up any large fluffy floc that might form a cake and reduce the filter run

b. to mix the polymer with the settled water in order to get more uniform coverage of the particle.

Robeck: Professor Kaufman has made an interesting suggestion for mixing the filter influent after you find fluffy unsettled floc coming over. You really have to surround all these particles with your aid if it's going to do any good. We have found that most of the so-called coagulant aids are really floc aids. They merely act by bridging and bringing together the coagulants and are necessary to aid the coagulating process. You have to be careful what you add and to fill you specific need.

Young: Thorough consideration of the raw water quality should be made before this suggested procedure is undertaken. Studies on virus removal by coagulation show that virus is tightly bound to the flocculated matter. By stirring and breaking up this bound, you may permit the passage of virus particles through your filter.

Question: WHAT COMMENTS DO YOU HAVE REGARDING APPLYING WATER OF LOW TURBIDITY DIRECTLY TO THE FILTERS?

Robeck: In the so-called integration of the clarification step in the filter, we want to be careful and not burden the filter with high sludge volumes. I want to emphasize that we want to experiment very thoroughly with this before we advocate it. There is something about the nature of charges and so forth that we may be able to create on the medium itself ahead of time that would permit more sludge to enter and still remove the particulates.

Question: DO YOU GET A BETTER FLOC DISTRIBUTION THROUGHOUT THE FILTER IF YOU ADD ANOTHER MEDIA ON TOP OF YOUR SAND?

Robeck: If the top layer is composed of fines or if you simply put six-tenths or seven-tenths coal on the top, you are really not going far enough to make it very profitable. I think that this is where a lot of people have been disappointed in the length of run they got. They really don't get penetration or filtration with depth. In answer to your question, you get better distribution as a general thing.

Question: WOULD YOU COMMENT ON THE USE OF ACTIVATED CARBON IN LIEU OF COAL?

Robeck: We have experimented with the use of granular activated carbon for removal of taste and odor by putting it over the sand. In these cases, the filtration rate was quite low because we are after a lot of contact time to get optimum use of the surface within the granular carbon. Penetration of floc and virus is minimal compared to other arrangements. Virus is actually a proteinaceous material that is readily removed on activated carbon as contrast to say sand.

The full scale experiments in Nitro, West Virginia had remarkable success in removing particulates. The organic removal has not been as successful as our original small scale experience. The plant has a furnace to regenerate the carbon. Regeneration is done when the chloroform extract is high going through the filter or when the taste and odor gets above a certain level. The main point is it is able to remove the carry over from the settling basins without any breakthrough. The constant monitoring of this turbidity proved to the State Health Department that it works. We feel that the accumulation material on the outer carbon pores, not the internal pores themselves, has not interferred with the absorptive properties of the carbon. A lot of people have contended that it should be so, but we have experimental evidence to show otherwise. It's amazing how open floc is as far as organic molecules in water, since anything dissolved can pass through it. This technique is something that we in public health are vitally interested in so as to guard against contaminants like carcinogens and pesticides that have subtle influences over a long period of time.

Question: WHAT ABOUT REGENERATION?

Robeck: Unfortunately, some people have tried it, but were unsuccessful and simply just left the spent activated carbon in the filter. The economic break point is not known, whether you should throw it away, replace it, or regenerate it. The regeneration costs are a little higher than expected in addition to attrition. Generally speaking, the big problem is when you have a slug of CCE (Carbon Coliform Extractables) going down the river. So, it's a tricky matter to learn how to operate these things to cope with variations in the raw water.

Kaufman: We are working with resins primarily for color removal. We feel certain absorbent resins that do not function as ion-exchanger can be incorporated in the filter media and remove color along with particulates. Some of these resins are as good or better than activated carbon. Furthermore, they can be regenerated in place in the filter boxes instead of being removed.

Question: WOULD YOU COMMENT ON YOUR WORK ON BACTERIAL AND VIRUS REMOVAL?

Robeck: This is to us ancient work, but I guess we should highlight that this is our primary reason for being in existence. We have discovered that virus, though smaller than coliform, can be removed by rapid sand and anthracite filters as well as coliform. Obviously, this is accomplished by adherence to the floc. We feel that there is limitation to this process as far as retaining virus that may be in your raw water which may have escaped pre-chlorination or clarification. We feel also that monitoring of gross turbidity is a very inexpensive and fortunate indicator of relative safety. It shows you that the water is free of particulates that perhaps would interfer with chlorination after filtration. I hasten to say that, just because the raw water is clear and you use it without filtration, this does not necessarily mean that it is free of the viral materials or that chlorination would necessarily suffice in doing away with these hazards. What we say here about virus removal applies to water that receives treatment with a coagulant and putting it through a filtering media.

Question: WHAT DO YOU MEAN BY THE TERM GROSS TURBIDITY?

Robeck: I am saying that you do not have to characterize the effluent turbidity to its constituency whether it's algae debris, silica or virus. It is too expensive to go through that. You can do a good job of characterizing once in a while with a membrane filter and putting it under a microscope to see just what kind of things are going through. As a routine, every minute of the day kind of thing, the turbidimeter is helpful.

Question: CAN YOU IDENTIFY LIMITING TURBIDITIES?

Robeck: As we all know, many people assume that the Five Jackson Unit in the Drinking Water Standards are suppose to be for health purposes and safe. However, if one reads the entire text, it indicates that, with filtration plants, it should be far less than five. Five, in other words, applies mainly to so-called protected watersheds. I personally am becoming to have a lot of doubts about it as a result of our study in the northwest of three different watersheds. In answer to your question, we would prefer to stay well below one Jackson Unit and it is relatively a straight-forward matter to get down to one-tenth most of the time. When averaging out, you should certainly keep it down below five-tenths. You hear a lot of people talk about hundredths, it is possible, but we are constantly pressed to justify this low level and we are reaching some kind of compromise. We cannot necessarily demonstrate any further health implication since we have removed the virus and chlorination is provided. What it boils down to is that we are trying to keep the system clean so that you do not have subsequent chlorine demand in your system, and that you have to have flushing programs and that you do not need to provide excessive chlorination in order to maintain satisfactory bacteriological conditions in the system. It is a practical consideration to a certain extent.

Question: ARE YOU SUGGESTING SETTING THE TURBIDITY LIMITS SOMEPLACE BETWEEN TWO-TENTHS AND FIVE-TENTHS?

Robeck: I think that is realistic. Incidentally, turbidimeters get off calibration very easily and you may get discouraging results. With proper calibration, I think you will see that such results are achieveable.

Question: WOULD YOU CARE TO COMMENT ON HERBERT HUDSON'S HYPOTHESIS OF THE RELATIONSHIP OF TURBIDITY AND THE PRESENCE OF VIRUSES?

Robeck: He tried to show that, if you provide clarification of a poor quality water, you would have less incidents of virus illnesses in the community. By clarification, I mean the whole process of coagulation, sedimentation, filtration, and chlorination. I would like to believe Hudson's hypothesis, but I think that the data that he used to base his hypothesis is limited. Although the hypothesis has not been proven, I don't think it cost very much extra to provide treatment to achieve the desirable quality. I would like to argue that it is to the benefit of the utilities on a long run in hope that everybody gets on the band-wagon and does it when they have the equipment and the chemicals to do so.

Question: WOULD YOU CARE TO COMMENT REGARDING THE NEED FOR FILTRATION OF WATER COMING FROM A LARGE WATERSHED WITH LIMITED INHABITANTS?

Robeck: We have studied three watersheds in the northwest. One is very well protected, and the others have varying degrees of inhabitants and recreational use. We have difficulty in measuring any appreciable differences in the quality of the water coming from any of the watersheds regardless of population. In each watershed, we found fecal coliform, shigella, salmonella, and such pathogens. Comparison of the turbidity of the water from these watersheds makes it apparent to us that it should be well if they clarify the water in addition to chlorination.

Question: IF YOU HAVE A RELATIVELY CLEAR AND UNPOLLUTED WATER, DOES CHLORINATION PROVIDE YOU A FAIRLY HIGH LEVEL OF PROTECTION?

Robeck: Here again, we are talking about a number of different situations. You will get a lot of algae debris and other matter in the distribution system if you don't clarify these waters. Many systems have alternate sources such as wells that they could use during periods when the water is turbid. Others might divert the water into a nearby lake and let it settle out. It seems that, with well protected watersheds, there is an amazing amount of animals per square mile compared to people and we can only attribute the occurrence of fecal coliform in the run-off water to these animals.

Question: ARE YOU INTRODUCING A NEW PRINCIPAL THAT HEALTHY PEOPLE MIGHT BE BETTER ON THE WATERSHEDS THAN UNHEALTHY ANIMALS?

Robeck: I am sure that if only healthy people had been up there, there would not have been any unhealthy animals. This is the reason why I am trying to bring out the practical aspect of keeping a distribution system relatively clean. If you get aftergrowth, you will get taste and odor and you will get into impaired screens and plumbing in your house and your lawn equipment. We had a case in Pittsburg which was written up in the New Yorker Magazine where three babies died because of pseudomonas caught in the strainer at the end of a facet. I personally think that the contaminant did not come from the water system, but from the outside into the tap. Removing particulate matter in water is not that difficult and it is constructive to do so.

Question: THERE IS COMING INTO POPULAR USE TODAY IN SWIMMING POOLS SO-CALLED HIGH RATE FILTRATION AT 15-20 GALLONS PER SQUARE FOOT PER MINUTE WITHOUT THE AID OF COAGULANTS. WHAT IS THE PROBABILITY OF BREAKTHROUGH IN THIS CASE?

Robeck: It is pretty hard to show the change or load going through a swimming pool filter so it is somewhat like diatomacious earth filters - it works fine while you are not putting too much on it. I think it is risky using high rate if you are trying to remove things that people spit or come out of the upper respiratory tract and these are really the culprits in most swimming pools. I think that you would probably lean very heavily on chlorination to be the workhorse.

Question: WILL YOU MAKE SOME COMMENTS REGARDING DIATOMACEOUS FILTERS FOR DOMESTIC WATER SUPPLY APPLICATION?

Ongerth: California has about 25 of these filters and we are satisfied with them. We feel a lot better about them now than we did 10 years ago. At that time, we were quite willing and pleased to have the DE filters used on the cleaner waters where previously they had been considered good enough to use with only chlorination treatment. However, we had reservations in their application to dirty waters in the state. It's probably a matter of the mechanics, the whole arrangement, what factors of safety you have, and economics. As a purely mechanical removal process, we feel that it is as acceptable as sand filtration. There are times when pretreatment is desirable and should be provided along with filtration through DE filters.

Question: HOW DO YOU FEEL ABOUT PRESSURE SYSTEMS VS. VACUUM SYSTEM TYPE DE FILTERS?

Ongerth: We are relaxed about a lot of things here in California. Largely because we are not dealing with large dirty rivers. We have one vacuum filter for filtration of lime softened water. The other filters are pressure type, again treating relatively clean waters. We feel monitoring of the finished water turbidity is essential to give you a kind of safeguard. We have found problems with breakthroughs in DE filters, problems with pressure sand filters. It gets back to assessing what the basic hazards are, what kind of risks you think you can accpet, and what factors of safety you think you need to get your ultimate objective of safe, wholesome and potable water for the public.

Afternoon Session

Question: WHAT IS YOUR OPINION OF MIXING AT THE INTERFACE MEDIA OF A MULTI-MEDIA FILTER PLANT AND MIGHT THAT BE BENEFICIAL?

Aultman: I think it would be very difficult not to get some mixing at that interface. I see no indication that it is harmful, but there is a good possibility that it is beneficial.

Question: THERE WAS A REAL STEEP HEAD LOSS GRADIENT NEAR THE BOTTOM OF YOUR FILTER MEDIA. WOULD THIS BE BENEFICIAL?

Robeck: The theory, and actually the practice, is that, if you have a transitional change in your incrimental head loss where mixing takes place, you get less opportunity for discret accumulation of floc like you would at an abrupt change from coarse coal to fine sand and, therefore, you have better opportunity to wash this free during the backwash than you would if you had a discret definite layer of fine sand starting out in the sand regime. What we are striving for is a gradual change in permeability with depth like a cone. I think that we are after the steep head loss as a factor of safety. However, there will be a lot of trouble with backwashing some things out when they accumulate at the lower level.

Kaufman: I think the effect of mixing is to reduce the porosity in that region. In theory, filter efficiency is proportioned to 1 minus the porosity, so be reducing the porosity, say from 50 percent to 20 percent, one gets a slight improvement in the filter efficiency in this small area and still not suffer excessive head loss.

Robeck: Actually, you are reaching a danger point if that seems to be the work horse. You should be keeping most of your stuff out near the top of the filter. You just add to your backwash problems too and, when you pack your filter regularly, you are going to have trouble in the long run.

Question: THERE HAS BEEN A LOT OF QUESTION ON PRE-TREATMENT BEFORE PUTTING THE INFLUENT INTO THE FILTER. IS THERE A POINT WHERE PRE-TREATMENT IS NOT NECESSARY BEFORE FILTRATION SUCH AS A MOUNTAIN WATER SOURCE WITH LOW TURBIDITY?

Robeck: I may be a little biased here but I feel that, once you have made an investment in a filter, you should add chemicals all the time. It is a relatively small cost and I think it will keep your clear well and distribution system all that much cleaner. You will have less problems five to ten years from now by the fact that you didn't allow a little turbidity to get into your system.

Question: WOULD THE METHOD OF DIMINISHING RATE OF FILTRATION INCREASE THE LENGTH OF FILTER RUN?

Aultman: I don't believe that this has been investigated. It's just a question of the quality of the water produced under diminishing rates. There had been a concern that you did not dare start a filter out to take all the water it would take for fear that it would not properly filter. The work that Hudson has done indicates that such is not the case, that you can start with a high rate.

Robeck: Hudson reported on work he did at Wyandotte, Michigan. Frankly, he set a rather limiting initial so-called "higher" rate. Therefore, he didn't make too much more water in a given time as his production rate proceeded downward with time. The big emphasis that he was trying to make was that this should and did improve the quality of the water. He didn't break up the floc with more and more force or shear. I would not rush to diminishing rate of filtration as a panacea if you want to produce twice as much water. It does save a little power and you can eliminate the rate controller although I've heard some equipment manufacturer carry on about what you do with the rest of your hydraulics. You do get a lot of your water piling up over the weirs and the settling basin if you could not get the valves organized right; but, that can be overcome.

Question: WOULD YOU COMMENT ON THE REPORT BY DAVE REESER OF SAN FRANCISCO THAT THERE WAS A TIME LIMITATION ON A FILTER RUN EVEN THOUGH YOU HAD NO HEAD LOSS USING POLYELECTROLYTES IN AN ANTHRACITE SAND FILTER?

Harris: At certain times of the year, Dave was removing manganese using polyelectrolytes without flocculating and he had very, very long runs. We have no such condition at any time of the year, and I would expect that no run that we will ever have will exceed 72 hours.

Question: HAVE YOU HAD ANY EVIDENCE THAT THE BEDS COMPRESS WHEN YOU WENT TO A HIGHER RATE AND THEN DID NOT EXPAND AGAIN WHEN THE RATE WAS LOWER?

Harris: We had a full inch of additional material for our 30 inch designed bed because we realized that at times our bed would compact as much as 3/4 of an inch. We measured this in our experimental filters. When we reduced the rate, the bed did not go back up. There was no loss of efficiency. In trying to measure for coal loss, we measured the top of the media under the same conditions. Not in one case measure the top of the media right after the wash and the other case after you have completed the run because you should have at least 1/2 inch differences.

Question: AFTER A BACKWASH, WOULD YOUR BED COME UP TO ABOUT THE SAME LEVEL REGARDLESS OF WHAT RATE YOU HAD OPERATED?

Harris: We suspend the bed completely by expanding it about 22 to 23 percent for our wash and the bed goes right back.

Question: ON DETRIMENTAL AFFECTS OF A SUDDEN INCREASE IN FILTER RATE THROUGH A BED, WHAT DO YOU MEAN BY SUDDEN IN TERMS OF TIME?

Aultman: I don't know if there is an exact definition of what you would call a sudden increase. I would call it sudden increase in say 30 seconds, a minute or maybe even longer. We get in the literature information which you might say didn't tell the whole story. Teplser and Busher reported in the December Journal that, at a 10 second rate of change, they still got some but a lot less. At the San Francisco plant,when the rate of flow was increased about 25 percent to 33 percent which might be over half a minute, they got an increase in their turbidities. I think turbidity break through is dependent upon the quality of your floc and the condition of your filter media. I think you should stop, backwash and start over again rather than put up with these big slugs of material.

Question: WHAT IS THE USEFUL LIFE OF ANTHRAFILL AND IS THERE ANY DIFFERENCE IN ATTRITION RATE WHEN YOU ARE USING FILTER BACKWASH VS. WATER AND AIR FOR BACKWASH?

Aultman: There is not enough factual data to give an indication. The quality of the anthracite you get has been variable. Some has been softer than others. Some of it has been striated. As a consequence, the San Francisco plant had poor quality coal. If you have a very good quality dense coal, I would not expect a five percent loss a year. You might have more attrition loss with air than without if you had a softer coal. When using an air wash, there are times when you can get air entrapment underneath the coal particles and that will wash the coal out.

Harris: Due to the courtesy of Candy Manufacturing Company of London, England, we were given units which enabled us to use air and we were disappointed almost immediately. We found that we were not properly disturbing the bed and you can not bring on water with this unit. You have the bed full of water but you will not have any rise in water when you are introducing air. Then, after the period of air scour, supposedly we turned on the water and went into a regular rise. However, the British apparently are accustomed to using a much lower rate of rise so even this was not just what we wanted. In Santa Clara County, there is an entirely different method of air scouring. In this case, both air and water come up at the same time. Now you are getting some real action. You are lifting with water and you are scouring with air bubbles. Just before the expanded and floating coal is ready to go over the lip of the trough, the air shuts off and you are washing with straight water. This, I think, has possibilities.

Question: MR. HARRIS, WHAT IS THE QUALITY OF RAW WATER AT YOUR PLANT?

Harris: We have better water than other plants in our area because of a billion gallon raw water storage reservoir. Turbidities are ranging around 25 to 30 T.U. and seldom go over 100 units. Water is easily treated and, with our high energy input, we seem to have better flocculation conditions than older plants that we abandoned.

Question: MR. HARRIS, PERHAPS YOUR EXPERIENCE DOES NOT INCOMPASS THIS-WHETHER YOUR PLANT WOULD WORK AS WELL IF YOU WERE TRYING TO REMOVE COLOR PRIMARILY RATHER THAN TURBIDITY?

Harris: I have no experience with color removal, but I do feel that, in removing colloidal material, zeta potential measurements could probably determine pH range at which color can be effectively removed.

Question: IS THERE A RELATIONSHIP BETWEEN THE MINIMUM ECONOMICAL RANGE OF TURBIDITY LEVEL IN THE FILTER EFFLUENT AND THE REMOVAL OF BACTERIAL ORGANISM? YOU GET TO A POINT WHERE CHLORINATION IS MORE EFFECTIVE AND MORE ECONOMICAL OR PRACTICAL THAN THE FILTRATION.

Robeck: I do not believe that anyone has made such a precise analysis or come up with such a precise relationship. We are probably well within our factors of safety. I think we are the only ones that have made an attempt to show that, when the turbidity exceed 0.5 T.U. and you had a lot of virus purposely added to the influent, you would get a very noticable increase of the polio virus particles along the alum floc. In our publication, we suggested strongly that you keep it below half and I fully agree. We want to stress the importance of getting the most of your facility that you can and you can do this with very little extra effort.

Question: HOW MUCH PROFESSIONAL RESIDENTS ARE REQUIRED FOR THE HIGHER RATE PLANTS FOR THE MANAGER TO GET HIS NIGHT'S SLEEP AND FEEL SECURE THAT EVERYTHING IS GOING ALRIGHT? WHAT SORT OF STAFF DO YOU HAVE TO HAVE TO BE SURE OR DO YOU LIVE AT THAT PLANT?

Harris: I have a reputation for never showing up from Friday afternoon until Monday morning. No one ever looks for me back at the plant. We have an excellent chief operator, he knows what's right from the start. He worked with me on the experimental filters. Actually, our operators don't have very much work to do. Our filters can be started on Friday and they won't be washed again until Monday and they would be washed automatically.

Robeck: From our limited experience in conducing pilot plant studies by taking the full size filters and altering the operation somewhat to prove a point, we have found a great deal of attention immediately put upon the plant and more specifically upon the operator. They have responded very well to this additional attention from the newspaper and from upper management and have become very much more alerted to what they are really turning out. This is particularly true when you put something that show a needle and you explain to them how it can go wrong. You have all the understanding of what they are really turning out instead of just looking down the clear well once a day. I think that this is particularly true with the smaller plant. The little guy is really taking a rap in a lot of the comments made that these things are primed for gib places with sophisticated staffs, but the little guy is not entitled to make use of this technology. I disagree. I think that he can be made aware of what potential there is with these too. A lot of them are part-time employees and you have to educate them about the limits of what turbidities can do. It can be fully calibrated, it can give false reading, and I have had a lot of people question values that we reported below a tenth. In certain cases, I think it was a justified criticism. But, generally, I think that we have been quite successful in improving the quality because of having higher rates or innovation incorporated.

Aultman: This ability for the small operator to be able to walk away and to sleep nights is directly related to the training program that you have with your operators themselves. Sure you get some fellows who don't want to learn, but I don't think most of our operators are that way.

SECTION H

CLOSING COMMENTS

H. J. Ongerth

CLOSING COMMENTS

Henry J. Ongerth

Instead of trying to summarize the vast amount of data presented, I would like to make a few comments to put into proper perspective regarding what we are trying to do and why we are here.

We are trying to produce a high quality, unquestionablly safe product at a reasonable cost. However, we don't know, with relation to safety, where we can draw lines. We have utterly no idea what our factors of safety are. With relation to aesthetic quality parameters that are evident to the consumer, it's a little easier to deal with and to know where we are. I was especially interested in Gordon Robeck's answer regarding treatment for a relatively clean mountain water. His observation that perhaps there are secondary affects in the distribution system that are not properly recognized from putting unfiltered water into the distribution system and then trying to cover it up with a tremendous amount of chlorine. This is a horrible example of a water purveyor trying to kid himself on why he has water quality problems in his distribution system. I don't expect the challenge that we (the regulatory authority) has to prove that a particular level of performance is absolutely necessary or it won't be complied with. I remind all of us here of the fact that the bacteriological standard, that nobody questions, is a standard of obtainability and convenience and not one that anyone has ever proven as being necessary, nor does anybody know what the factor of safety is.

I am very much impressed with the general consensus among the panel that pilot plant studies are not completely indispensable at least very desirable and helpful in design. This is a lesson that should be taken to heart. I am very pleased at the comments about more instrumentation on performance with particular reference to turbidity.

I was particularly interested in this entire proceeding because I have very great interests in the process of the regulatory authority reviewing proposals or plans for new projects.

Bill Aultman commented that a regulatory authority plans review should not be in great detail in all respects. Disinfection and cross-connection control are things he thought justified very careful and detailed review but as the rest of it, it was far more general. I'm of the opinion that we are forced into this position. I am not sure how to assess the next set of plans that come sailing through.

I have moved very far in the 30 years I've worked in the field, to the point that I am now much more interested in performance standards and much less interested in examining details. It is not a simple matter and we must give much more serious consideration to how far we can go in the direction of performance standards. I don't know how many of my fellow State Sanitary Engineers would agree with me. I am sure that I will be in a minority, and probably a very small minority in this regard. I feel that the California Department of Public Health has been not one of the blacker culprits among the regulatory authorities around the country. I don't take too seriously some of the pleasantries that have been passed here with relation to what regulatory authorities do and how unreasonable they might be. We have been pretty reasonable and my aim is to be even more reasonable. That doesn't mean less interested in what you are doing and the quality of job that should be done. I see our role as being the conscience of the designing engineer and of the operator. I very strongly disagree with looking perhaps more at the dollar than at the end product to prove that it's worth the dollar to do something a little bit better. I reject that philosophy emphatically.

APPENDIX

ROSTER OF PARTICIPANTS

ROSTER OF PARTICIPANTS

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