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SELECTION OF OPTIMUM FILTRATION RATES FOR SAND FILTERS

by

John LeRoy Cleasby

A Dissertation Submitted to the

Graduate Faculty in Partial Fulfillment of

The Requirements for the Degree of

DOCTOR OF PHILOSOPHY

Major Subject: Sanitary Engineering

Approved:

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1960

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LIST OF ABBREVIATIONS

centimeter	cm
feet	ft
feet per day	ft/day
gallons per minute per square foot	gpm/sq ft
gallons per square foot per day	gal/sq ft/day
hour	hr
horse power	hp
inch	in.
inside diameter	ID
meter	m
micron	μ
milligram per liter	mg/l
millimeter	mm
million gallons	mil gal
million gallons per acre per day	mgad
million gallons per day	mgd
negative log of the hydrogen ion concentration	pH
parts per million by weight	ppm
square feet	sq ft
temperature, degrees Fahrenheit	^{o}F
versus	vs.

I. INTRODUCTION

The most common processes of surface and ground water treatment include filtration of the water through a layer of sand as the final clarification step in the treatment. The modern sand filter used in municipal practice usually consists of an open watertight rectangular tank generally greater than eight ft deep, containing a layer of sand 24 to 36 in. thick supported on a layer of graded gravel 6 to 12 in. thick. The tanks are generally made of reinforced concrete. The gravel is underlain by an underdrainage system which leads to a common point of outlet. During normal operation, water to be filtered is maintained at a nearly constant level 4 to 6 ft above the sand surface and flows through the filter by gravity. A constant rate of filtration is maintained by a rate of flow controller in the outlet pipe.

As filtration proceeds, the sediment removed from the water builds up in the sand layer resulting in an increasing pressure drop or head loss through the sand layer. The rate of flow controller gradually opens the outlet valve to offset the reduced pressure in the outlet pipe. When unacceptable water begins to pass the filter, or when the head loss becomes excessive, the filter is cleaned or backwashed by reversing the flow of water. Water is admitted under pressure into the underdrain system at such a rate that the upward flow of water will expand the sand bed about 50 percent. The rising water, carrying with it the sediment removed from the sand, flows into washwater gutters which conduct the water to waste. The clean filter is then placed back in service. The period of operation from one backwash to the next is known as a "filter run", and varies in

length for different conditions from a few hours to several days.

Since the historic work of Fuller (26) in 1897, rates of filtration for sand filters have been commonly standardized at 2 gpm/sq ft. With the ever increasing population of the United States, particularly in urban areas, and the ever increasing demand for water per capita, many cities have found their filtration capacity at this standard filtration rate inadequate during peak demand seasons. If these towns could produce an acceptable water at higher filtration rates, they might be able to delay the large capital expenditure required for increased filtration plant capacity.

During the past twenty years, considerable work has been done with higher filtration rates and many plants are now operated at rates above 2 gpm/sq ft during peak water demand seasons. In those plants considering the use of higher rates, the plant operator or consulting engineer should consider two questions:

a. Can acceptable water be produced at higher filtration rates?

b. Within the range of rates where acceptable water can be produced, what is the optimum rate from the standpoint of maximum production per filter run?

High rate filtration experiences reported to date generally contain no reference to the latter question, indicating that the researchers may be unaware of its possible existence, or its importance. The evaluation of this question is the objective of this study; namely, under what conditions, if any, can an optimum rate of filtration be expected, and how can it be identified?

If an increase in production of water per filter run is possible by

operation at an optimum rate, two economies may be realized. A lower plant investment will be required for a given capacity if the optimum rate is above the standard rate. Operating costs will be lower due to the smaller physical plant and smaller percentage of water used in backwashing.

II. WORK OF OTHER INVESTIGATORS

A. Early Development of Rapid Sand Filters

The history of water treatment has been well covered by Baker (5) and only those portions pertinent to this study will be summarized here. The sand filtration of water without pretreatment at rates of approximately three mgad began in 1829 in England and the use of such filters was fairly widespread in Europe in the late nineteenth century. Successful use of these English or slow sand filters was also made in the United States on raw water supplies that did not carry a heavy colloidal sediment load. The use of slow sand filters on unpretreated water containing a heavy colloidal sediment load was generally unsuccessful because of the short filter runs, which resulted from the rapid formation of a surface sediment layer, and deep penetration of the sediment into the sand. Since the slow sand filter was cleaned manually by periodic removal of the dirty surface sand layer, deep penetration and short filter runs could not be permitted.

This difficulty led to the development of the American or rapid sand filter system in which aluminum or iron salts were used to coagulate the colloidal matter prior to filtration. The early rapid sand filter systems were proprietary devices, the first one patented by I. W. Hyatt in 1884. A number of others were on the market in the mid 1890's when G. W. Fuller was retained by the Louisville Water Company to test the suitability of rapid sand filters for the treatment of Ohio River water at Louisville, Kentucky. Various companies installed their filter equipment at Louisville for test under Fuller's direction. The equipment installed had the following general characteristics:

a. All plants had a capacity of 0.25 mgd. Sand filtering area, sand size, sand depth and method of applying coagulant were fixed by the company supplying the equipment.

b. Most of the devices added coagulant to the water as it entered the filter box above the sand. No separate mixing and settling was practiced prior to filtration.

c. Most of the devices were cleaned by an upward flow of water through the sand which flushed out the sediment so the sand could be reused.

These proprietary devices were operated at a rate of 0.25 mgd which was equivalent to rates of from 35 to 213 mgad. Most of the experimental runs were made at rates of from 80 to 154 mgad. Fuller (26) concluded that none of these proprietary devices were continually adequate due to the lack of separate coagulation and settling. He also concluded that the lower rates of filtration with these devices did not give significantly better effluent than the higher rates, and that the maximum safe rates were not reached in this work. He recommended that rates be not less than 100 mgad and inferred that higher rates should be considered since the rate of filtration was a predominant factor in the cost of treatment.

Fuller continued his work using separate tanks for pretreatment consisting of plain sedimentation, coagulation and sedimentation of the coagulated water prior to filtration. In these tests, he was limited to maximum filtration rates of 94 mgad by the physical nature of the equipment in use. He concluded that with inadequate coagulation, good filtered water could not be obtained even at rates as low as 50 mgad. With adequate coagulation, he felt rates could be increased materially above those he

studied. He recommended 100 mgad for Louisville with the knowledge that the rates would be increased to at least 150 mgad as the water demand increased.

These recommendations led to the widespread use of filtration rates of about 125 mgad from the time of Fuller's work to the present. Successful filtration at this rate over so long a period has led to the conclusion by many regulatory agency personnel, that 125 mgad (2 gpm/sq ft) is the maximum acceptable rate of filtration, and often cite Fuller's studies as their basis. It is apparent from his original study that Fuller believed considerably higher rates were possible if adequate pretreatment was provided. On the other hand, without adequate pretreatment, even lower rates may not produce acceptable water.

B. Studies of High Rate Filtration

The tremendous increase in filtration rates from the old slow sand filters to the rapid sand filters must have been such an improvement that for many years no one considered the possibility that improvement was still possible. At the time of Fuller's work chlorination of water supplies had not yet commenced and the filter was the final safeguard against the passage of pathogens to the consumer. Perhaps it was only natural that once relatively successful treatment had been obtained at 2 gpm/sq ft, nobody would consider risking any detriment to quality by venturing to use higher filtration rates. With the establishment of chlorination for water disinfection during the 1900-1910 decade, the filter had an invaluable ally in the control of disease. With the improvement of pretreatment, the filter gradually assumed the role of a polishing step to assure low

finished-water turbidity, and chlorination was depended on for the final bacterial control step. Filtration rates of 2 gpm/sq ft remained unchallenged until the work of Baylis which began in 1928 at the Chicago Experimental Filtration Plant. Baylis (9,10,11) and Hudson (33) reported on the effect of rate of filtration on length of filter run for pilot scale studies at the Chicago Experimental Filtration Plant. These studies were made using sand with an effective size of approximately 0.50 mm in steel circular filters with 10 sq ft surface area, and glass tube filters with 0.017 sq ft surface area. Coagulated settled water was filtered in all tests. Tests were conducted during periods of "strong coagulation" when good filter effluent quality was obtained.

On the 10 sq ft filters, studies were conducted with filtration rates from 1.6 to 3.5 gpm/sq ft. Influent turbidity averaged 7.68 units and effluent turbidity ranged from 0.06 units at the lower rate to 0.09 units at the higher rate. Effluent bacteria averaged 60 per ml at 37° C and did not increase at the higher rates. Effluent E. Coli per hundred ml increased from 3.5 at the lower rate to 5.6 at the higher rate. One plot indicating a linear relation of head loss vs. time at 2 gpm/sq ft was reported (9). On the 0.017 sq ft filters, studies were conducted at rates from 1 to 6 gpm/sq ft.

On the basis of the test results Baylis and Hudson both reported that the length of filter run in hours was inversely proportional to the 1.5 power of the filtration rate. An analysis of the original data, however, reveals some interesting facts about the relative production at different rates. Table 1 is an analysis of the data presented by Hudson (33) for relative filtrate production per ft head loss increase and per filter run.

Table 1. Effect of filtration rate on relative water production[a]

Filter size (sq ft)	Period of study	Filtration rate (gpm/sq ft)	Relative production (per run)[b]	Relative production (per ft)[c]	Average run length (hours)
10	1928-29	1.6	1.02	0.99	25.8
10	1928-29	2.0	1.00	1.00	20.1
10	1928-29	2.4	0.92	0.87	15.4
10	1928-29	2.8	0.79	0.84	11.3
10	1932[d]	2.0	1.00	1.00	33.58
10	1932	2.5	0.77	0.80	20.75
10	1932	3.0	0.77	0.81	17.25
10	1932	3.5	0.72	0.80	13.86
0.017	1931-32[e]	1	0.92	0.85	48.2
0.017	1931-32	2	1.00	1.00	26.1
0.017	1931-32	3	0.77	0.82	13.4
0.017	1931-32	4	0.76	0.88	9.90
0.017	1931-32	5	0.58	0.75	6.13
0.017	1931-32	6	0.48	0.68	4.26

[a]Analysis of data presented by Hudson (33).

[b]Per filter run to a terminal total head loss of 8 ft with 2 gpm/sq ft considered unity.

[c]Per ft of head loss increase with 2 gpm/sq ft considered unity.

[d]Jan. 5, 1932 to June 16, 1932.

[e]Dec. 4, 1931 to Feb. 21, 1932.

A study of Table 1 indicates that although the run length may be greatly reduced, the production per run or per ft head loss increase is not so greatly affected. In fact, a slight optimum is evident at 2 gpm/sq ft on the small filters.

Many surface water plants experience periods of "weak coagulation" when the floc will readily pass through the filters at low head loss, even at filtration rates of 2 gpm/sq ft. During such periods, runs are ter-

minated on observation of decreased filtered water clarity rather than on the development of maximum head loss. The practical use of higher filtration rates at such plants hinged upon the solution of this problem. In 1937, Baylis (12) reported the use of colloidal hydrous silicon dioxide (now called activated silica) as an aid to coagulation during such periods. He observed that the passage of floc and turbidity through a filter at 2.5 and 3 gpm/sq ft was prevented by addition of activated silica during a period of "weak coagulation", when the floc would readily pass without the additive. This development opened the way for increased opportunity in the use of high filtration rates.

On the basis of the high rate experience with the experimental plant and the development of a floc strengthening agent, the Chicago South District Filtration Plant was designed and constructed for operation at 3 gpm/sq ft during the winter low demand season, and up to 4.5 gpm/sq ft during peak summer periods. From time to time, Baylis has reported on plant scale experience at rates up to 5 gpm/sq ft (13,14,15,18). Filters were operated continuously at rates of 2, 4, 4.5, and 5 gpm/sq ft. The 10-year results of the Chicago experience are summarized in Table 2. All tests were made while filtering alum coagulated Lake Michigan water which was settled prior to filtration. Activated silica was used to strengthen the floc during periods of weak coagulation. Sand effective size in the filters was 0.65 mm.

The short runs obtained at high rates are due partly to high initial losses in sand, gravel, underdrains and outlet control piping at high rates, which results in a relatively small available head loss increase.

In each reference, Baylis suggests deeper penetration as the reason

Table 2. Summary of Chicago plant-scale experience with high filtration rates[a]

Rate (gpm/sq ft)	Year reported	Run length (hr)	Performance[b]	Time in service (percent)	Effluent quality[c]
4	1949 (13)	15.4			
4.5	1949 (13)	10.4			
5	1949 (13)	8.1			
2	1950 (14)		0.60-0.74		
4	1950 (14)	13.8	0.89-1.03		
4.5	1950 (14)	11.3	0.96-1.03		
5	1950 (14)	9.3	1.07-1.10		
2	1956 (15)	28.7	0.71	98.9	0.044
4	1956 (15)	15.4	1.02	97.9	0.059
4.5	1956 (15)	13.5	1.07	97.6	0.072
5	1956 (15)	10.2	1.10	96.7	0.077
2	1959 (18)	26.5	0.66		
4	1959 (18)	14.0	0.92		
4.5	1959 (18)	11.7	0.92		
5	1959 (18)	9.8	0.99		

[a]From reports of Baylis (13,14,15,18).

[b]Million gallons per ft of head loss increase.

[c]Average coagulated matter in the effluent by cotton plug filter tests (ppm).

for the higher performance at high rates but does not support this hypothesis with experimental evidence. He also notes (15) that in some cases the head loss increase per hour at the end of the run is slightly greater than at the beginning. At higher rates, the total increase in head available is not as great due to higher initial head loss and thus, the run may not enter the steeper range at the higher rates.

If the relative production per run is calculated based on average run

length and filtration rate, the results shown in Table 3 are obtained.

Table 3. Relative water production per run at various filtration rates at Chicago[a]

Data reported	Filtration rate (gpm/sq ft)			
	2	4	4.5	5
1949 (13)		1.00	0.76	0.66
1950 (14)		1.00	0.92	0.84
1956 (15)	0.94	1.00	0.99	0.83
1959 (18)	0.93	1.00	0.92	0.85

[a]From reports of Baylis (13,14,15,18) with 4 gpm/sq ft considered unity.

The data in Table 3 clearly indicates a slight optimum filtration rate at approximately 4 gpm/sq ft. The results summarized here cover several characteristic periods of microorganism content in the raw water. Heavy microorganism populations causes short runs and head loss may develop at an increased rate towards the end of the run. Very light populations may yield long runs with nearly linear total head loss vs. time curves. Intermediate populations yield moderate runs of about 20 hours with slightly increased rate of head loss development as the run progresses (16).

After ten years of experience with high rate filtration at Chicago, Baylis (18) concludes that sand filters will produce satisfactory quality water at 5 gpm/sq ft but recommends that such rates only be used during peak summer demand periods due to the short runs caused by high initial head loss. In these extensive studies no measureable detriment to turbidity or bacterial content was observed at high rates. A slight increase in coagulated matter passing the filter was observed with the cotton plug

filter.

Geyer and Machis (27) in an extensive series of studies of filtration at rates of 2 to 10 gpm/sq ft found that the volume of water produced per run increased as the rate increased. They reported relative production volumes of 1:1.75:2.65 for rates of 2:3.6:6 gpm/sq ft respectively. The filter sand had a mean geometric size of 0.27 mm and runs were terminated at a terminal head loss of 30 ft of water. Alum coagulated Baltimore city tap water was filtered in these studies. The floc was kept in suspension by agitation and the water was not settled prior to filtration. Effluent turbidity was less than 0.3 units in all tests, being slightly poorer at high rates. Head loss was found to be proportional to the second power of the volume of water produced. These results are contradictory to the early work of Baylis (10,11) and Hudson (33) which indicated reduced water production per run as filtration rates were increased. When coarse 1.1 mm sand was used in further experiments, Geyer and Machis noted that the trend toward increased production per run at high rates was reversed, 6 percent less water being produced at 6 gpm/sq ft than at 2 gpm/sq ft.

Hazen (30) emphasized the importance of adequate pretreatment to the use of higher filtration rates. He also cautions against the use of rates greater than 2 gpm/sq ft if activated carbon is being used as a taste control.

Hulbert and Feben (43) reported on plant scale studies at the Water Works Park filtration plant at Detroit, Michigan at rates of from 2.2 to 3.2 gpm/sq ft. They found the rate of head loss increase (ft/day) to be directly proportional to the filtration rate. Thus, the production of water per ft of head loss increase would be the same regardless of rate.

Brown (19) reported on 3 years of plant scale high rate filtration studies at Durham, N. Carolina. Prechlorinated water which had been coagulated with alum and lime was settled prior to filtration. The filter sand had an effective size of 0.55 mm. His data summary is reproduced below in Table 4. An analysis of these data for relative production per run clearly indicates a slight optimum rate at 3 gpm/sq ft. The relative production per run is 1:1.29:1.20 at 2, 3, and 4 gpm/sq ft respectively.

Table 4. High rate filtration data at Durham, N. Carolina[a]

	Filter number		
	12	13	14
Rate (gpm/sq ft)	2	3	4
Length of run (hr)	135.2	116.7	81.3
Turbidity (ppm)	0.34	0.38	0.43
Bacteria (colonies/ml)	0.32	0.42	0.36
Coliforms	neg.	neg.	neg.
Wash water (percent)	1.21	0.89	0.99

[a]Reproduced from the report of Brown (19).

Brown concluded that high filtration rates were acceptable if competent supervision was available. Total head loss vs. time data were obtained from Brown[1]. When plotted with total head loss as ordinate, the curves were linear to slightly concave upward. The curves were highly erratic since the data were taken from circular chart recorders which he considered rather crude devices.

[1]Brown, W. G., Municipal water plant, Durham, N. C. Water filtration plant filter head loss experience. Private communication. Feb. 17, 1960.

Jackson (46) reported successful plant scale high rate operation using both sand and anthracite filtering media. Potomac River water was treated by prechlorination, liquid alum coagulation and settling prior to filtration. Fifteen in. of sand with an effective size of 0.56 mm was used in two filters under study while anthracite was used in other filters. One series of studies was made at constant rates of from 1.5 to 4.5 gpm/sq ft. Comparative production data were presented on two anthracite filters. Runs were terminated at low head losses of considerably different magnitude; therefore, the data on production per ft of head loss is the best means of comparison. These data are reproduced in Table 5.

Table 5. High rate filtration data at Washington, D. C.[a]

Date & run no.	Filter no.	Rate (gpm/ sq ft)	Run length (hr)	Initial loss (ft)	Final loss (ft)	Production (mil gal/ft increase)
June '53, Part 2						
Run 2	22	3.0	72	0.89	3.83	6.34
	24	3.0	66	0.66	3.66	5.63
Run 3	22	4.0	26	1.21	2.53	6.60
	24	4.0	22	1.19	2.67	5.26
Run 4	22	2.0	75	0.53	3.83	3.52
	24	2.0	75	0.50	4.91	2.76
Run 5	22	1.5	58	0.33	2.18	4.20
	24	2.0	42	0.52	1.95	4.57
October '53, Part 3						
Run 1	22	4.5	24	1.76	3.43	5.38
	24	4.0	24	1.23	3.10	4.55

[a]From the report of Jackson (46).

Production per foot of head loss increase was approximately the same from 3 to 4.5 gpm/sq ft averaging about 5.6 mil gal which is slightly better than the production at lower rates. One graph was presented showing a linear relation between total head loss and time at 4 gpm/sq ft, and nearly linear at 3 gpm/sq ft. Effluent turbidity was not recorded during these studies. Residual aluminum in the effluent was observed and found to average about 30 percent of the influent aluminum which averaged about 0.20 ppm. Mud and floc were readily observed in the anthracite to within 2 in. of the bottom at rates of 4.5 gpm/sq ft. Floc was observed in the bottom 2 in. with the aid of a microscope. This author also presented some experimental data on constant pressure (declining rate) filtration.

Tso-Ti Ling (53) studied rates of filtration from 1 to 5.5 gpm/sq ft using a small laboratory scale plant consisting of a mixing tank, flocculation tank, settling tank and 2-1/2 in. ID plexiglass filters. Tap water with added diatomaceous earth for turbidity was coagulated with ferric chloride and slaked lime. Two series of tests were conducted using uniform sized sand in all filters in each series. The effective size of the sand was 0.458 mm in the first and 0.383 mm in the second series. The head loss vs. time curves in all cases were linear for all incremental portions of the bed except the top one in. layer. The rate of head loss increase in the top 1 in. layer dropped off slightly towards the end of the run. Run length for a 24 in. graded sand filter was found to be inversely proportional to the 1.23 power of the filtration rate. With uniform sand, the exponent was 1.48. Total head losses were measured from the top to the bottom of the sand layer. Runs were made with terminal losses of four,

five, and seven feet, but the relation between head loss and run length was the same, regardless of the terminal loss. This relationship was also independent of the sand size. These results are in reasonably close agreement with those of Hudson (33) and Baylis (9,10,11).

Stanley (51) studied filtration of homogenized iron floc labeled with a radioactive iodine tracer. Filter containers were 1-1/2 in. ID lucite tubes. The penetration rate of the labeled iron floc was observed with a Geiger tube. The penetration rate was directly proportional to the filtration rate for a given sand size. Head loss vs. time curves for these studies were linear. The effect of filtration rate on the penetration rate was observed for 20-30 mesh sand (0.84-0.59 mm) and 40-50 mesh sand (0.42-0.29 mm). No data were reported on actual iron concentration in the effluent, but the penetration depth was arbitrarily described as that depth in the bed where the radioactive count was equal to the background count plus one standard deviation. He observed that the run length to an 8 ft head loss was inversely proportional to the 0.6 power of the filtration rate indicating improved production at higher rates. Full penetration, as defined, did not occur within the rates and sand sizes studied. Therefore, since the penetration rate was directly proportional to the filtration rate, one would expect improved production volume at the higher filtration rates due to the greater utilization of the sand bed.

Hudson (36) states that higher rates result in greater penetration of material into the filter and thus greater water production per unit increase in head loss. The water quality can be safeguarded by using greater sand depths or reducing the terminal head loss. Filtration rates of 10 gpm/sq ft are possible without deterioration of quality if grain

size and depth of filter sand are proper. However, in a later article (38) he states that high rates may result in less production per run.

Conley and Pitman (20) studied rates of filtration from 2 to 35 gpm/sq ft on sand and sand-anthracite pilot filters. They found that the use of polyelectrolyte coagulant aid in the filter influent water resulted in equal effluent quality at all rates. However, an upper practical limit of 8 to 10 gpm/sq ft was suggested due to rapid head loss build up at higher rates resulting in short filter runs. An upper limit of 0.01 ppm of turbidity was set as the maximum acceptable effluent level. A light scattering microphotometer was used in turbidity measurement, however, the means of calibration leaves considerable doubt as to the true value of effluent turbidity. They observed the distribution of head loss through the filter bed and concluded that acceptable run length can be obtained by permitting deep penetration of material into the bed. They stated that the penetration depth could be controlled by the use of the coagulant aid.

Holluta and Eberhardt (32) studied the filtration of iron bearing water at rates between 2 and 16 gpm/sq ft on a pilot pressure sand filter to terminal head losses of 30 to 45 ft. Hard tap water was aerated and passed through an activated carbon filter to remove all iron and chlorine. Ferrous sulphate was then added to yield the desired iron content entering the filter and little if any precipitation of hydrated ferric oxide occurred above the sand. The filter which had a diameter of 6 in. and sand depth of 47 in. was equipped with sampling and piezometer connections at 11.8 in. intervals. The penetration and precipitation of iron within the filter was studied as the run progressed, with variable sand size, iron

content and filtration rate. Even deep within the bed only soluble ferrous iron was present in the filtrate. The precipitated iron was immediately removed on the sand and no break through of the precipitated iron occurred even at the high rates. The rate limiting reaction was therefore found to be the oxidation of ferrous iron to hydrated ferric oxide floc. The natural log of the effluent iron content was found to be proportional to the time of iron contact within the filter which in turn is inversely proportional to the filtration rate. High effluent iron content was observed at the higher rates. This was due, however, to inadequate time for the oxidation reaction to take place. On the basis of the observations, an empirical formula was developed for the depth of filter required to reduce the iron content in the finished water to 0.1 mg/l for various conditions of sand size, raw water iron content, and filtration rate. While this work has rather limited application due to the conditions of the experiment, it led Eberhardt to a subsequent study of economic considerations in rapid sand filtration (22).

In this article Eberhardt observed that, in their research studies, as rates of filtration were increased, greater water production occurred. However, due to the conditions of their work, effluent quality decreased as the rates increased. He studied the relative economy of pumping costs for pressure filters to various high terminal head losses. Disregarding the decreased water quality, he observed that pumping costs per unit production went down as rates were increased. When provision was made in the calculations to produce comparable water by using greater sand depth, however, he concluded that the economic advantage was at the lower rates. The nature of his experimental work, with soluble ferrous iron entering

the filter is contrary to common American practice and precludes the direct application of these observations to practical selection of optimum filtration rates. In addition, these observations were made in a pressure filter with high terminal head losses possible. The economics of various rates on open filters is entirely different since the terminal loss is generally fixed at a fairly low value.

C. Summary of Status of High Rate Filtration

It is evident from the literature cited above that considerable work has been done on high rate filtration. The data in some reports indicates less water production or no increase in production per run as rates are increased (9,10,11,33,37,43,46,53). Head loss vs. time curves in most of these studies were linear or nearly linear. The studies indicate that acceptable water can be produced at rates up to 5 gpm/sq ft but special care must be exercised in pretreatment with coagulation aids during periods of weak coagulation. The data in some reports indicates better water production per run or per ft increase in head loss as rates increased (13,14, 15,18,19,27,36,51). With one exception (51) head loss vs. time curves in these studies were not linear; head loss increased at a more rapid rate as the filter run progressed.

It is evident that the relative economy of high rate filtration from the standpoint of water produced per run has not been fully investigated under controlled conditions. Some waters are such that higher rates yield deeper penetration and greater production for a given head loss increase. Other waters do not demonstrate this improved production. The studies reported represent a variety of water characteristics and filter details.

Some were pilot scale or laboratory scale studies with good control and high terminal head loss; some are plant scale studies in which control of conditions and terminal head loss are limited by the physical arrangement of the plant. Some of the reports indicate production per run while others indicate production per ft of head loss increase, which are not comparable. The data thus far available would certainly verify the proposition suggested by Fuller (26) that much higher rates than those which he studied are possible.

D. Studies of the Functioning of Sand Filters

1. General

For approximately thirty years following Fuller's work in 1896-7, little information was reported in the literature on studies of rapid sand filtration. During this period filters were designed more or less along Fuller's recommendations for Louisville, namely, using sand with an effective size of 0.35 mm and filtration rates of about 2 gpm/sq ft. The sand filters were frequently inadequately cleaned during backwashing and the belief was common that the filter would function better with slightly dirty sand. This concept was a carryover from the old slow sand filters which did not function properly until the "schmutzdecke" had been developed. It was not until the late 1920's and the 1930's that much attention was given to the mechanisms involved in filtration. In this section, the more noteworthy contributions to filtration theory will be briefly enumerated.

2. The hydraulics of the clean filter bed

It is well established that flow of water through a filter at rates

common to rapid sand filters is in the laminar or stream line flow range and obeys Darcy's law for laminar flow through granular material:

$$v = KS \qquad \text{Eq 1}$$

Where:

v = velocity of flow (fpd)
K = the coefficient of permeability (gpd/sq ft at unit hydraulic gradient)
S = the slope ratio of the hydraulic gradient.

This fact has been observed by many workers including Hulbert and Feben (41) who observed that the head loss through clean uniform sand filters was proportional to rate of flow and to depth of filter. They developed one of the first comprehensive empirical formulas for the flow of water through clean sand:

$$h = \frac{27 \ lq(73-p)}{10^5 \ d^{1.89}(t+20.6)} \qquad \text{Eq 2}$$

Where:

h = loss of head (ft)
l = depth of bed (in.)
q = rate of filtration (mgad)
d = 50% sand size (mm)
t = water temperature (°F)
p = porosity (percent)

This formula was developed from results of tests using a number of glass tube filters with uniform sand ranging in size from 0.28 to 1.20 mm. Temperatures from 32° to 80° F and rates of filtration up to 300 mgad were investigated. A new method of measuring the sand porosity was suggested for use in this formula (42). Since the formula was developed for uniform sands (those between successive sieve separations) it is necessary to make

successive applications of the formula to the separations found in a typical sieve analysis to find the total loss for a given sand bed.

Filtering clean water through 3-1/2 in. ID filters with a media of uniform glass ballotini (beads), Ghosh (28) observed that the head loss did not increase linearly with depth, but exhibited decreased head loss per unit depth in the deeper lamina. He did not develop positive proof as to the cause of this phenomenon, but hypothesized that it might be the result of the streaming potential developed across the sand. He presented evidence that the phenomenon was not due entirely to stratification, unequal packing, or to the release of dissolved gases in the lower depths. This work would contradict that of Hulbert and Feben who observed head loss proportional to depth for uniform sand filters.

The most rational formulations for the calculation of head loss through a clean filter were developed by Fair and Hatch (24,25).

$$\frac{h}{l} = \frac{1.067\ C_D\ v^2}{g p^4 d} = \frac{0.178\ C_D\ v^2\ A}{g p^4 V'} \qquad \text{Eq 3}$$

Where:

h = head loss in depth l
C_D = coefficient of drag = $\frac{24}{R} + \frac{3}{R} + 0.34$
R = Reynolds number $d\ v/\nu$
v = Face velocity or velocity of water moving down upon the sand bed
g = acceleration due to gravity
ν = kinematic viscosity
p = porosity ratio of the filter bed
d = characteristic diameter of the sand grains defined as $6\ \frac{V'}{A}$
V' = volume of sand particles
A = surface area of sand particles

With the exception of the diameter term ($d = \frac{6V'}{A}$), the evaluation of

the factors in this equation is fairly straight forward. Experimental observations which permit evaluation of this term are presented (25). For a stratified bed it is necessary to evaluate the diameter term incrementally for each sieve size separation assuming that the particles between adjacent sieves are substantially uniform in size.

Application of these formulae to the calculation of head loss in a partially clogged bed as a filter run progresses is not yet possible. Such a calculation will await the development of some universal measure of water filtrability which can be related to head loss development.

3. Observations of filtration action

The first report of any detailed observations of filtration action was made by Baylis (7) in 1926. He observed through a pilot filter with a glass side the predominant surface removal. He noted that as the surface layer became clogged, small channels would break open to permit sediment to pass to a slightly lower depth where the water would fan out in the cleaner sand and suspended matter would again be removed.

Similar observations were reported at about the same time by Tyler, et al. (54). They observed that the principal removal occurred in the upper 6 in. of sand. The large head loss development in the layer results in negative head first appearing at about 6 in. below the sand surface and gradually moving upward and downward from that depth. They suggested that larger positive heads above the sand might reduce problems associated with negative head. They also concluded that effective size, surface modulus, and specific surface were of about equal value in describing the sand characteristics.

In a later article, Baylis (11) elaborated on his observations noting

the upper layer may remove practically nothing but the head loss in that layer will generally continue to increase. Since the head loss is inversely proportional to the fourth power of the porosity (equation 3), a minute deposition of solid matter in an already clogged region of the bed will result in a significant relative decrease in porosity and increase in head loss. It was also observed that the size of the particles reaching the lower layers was smaller than those reaching a somewhat shallower layer.

Hudson (34,35) observed that seasonal variations in water quality often lead to periods when floc passes the filter at low head loss even at normal filtration rates. At such times, which usually occur during the winter, the head loss vs. time curves are apt to be linear. He describes such periods as weak coagulation periods and the floc as weak floc. He also observed that under normal coagulation, the floc did not readily pass the filter and describes such floc as strong floc for which head loss develops at an increasing rate as the run progresses. Intermediate degrees of floc strength were also recognized. He suggested (38) that a linear head loss vs. time curve may be indicative of potential passage of floc through the filter but did not support this hypothesis with evidence. He emphasized the need of the consultant to design for the most critical seasonal conditions that may be expected. As an empirical measure of the floc strength, the "floc strength index" (35) was proposed.

$$\text{Floc strength index} = \frac{hd^3}{l} \qquad \text{Eq 4}$$

Where:

h is the head loss at the time when floc passage is noted through a bed of depth l and effective size d.

that the suspended material which passed through the surface layer was removed primarily at the contact spots of the sand grains. Stein (52) also observed that removal was predominantely at the contact points of the sand in a small filter which he observed with a microscope. He noted that the penetrating material was primarily that in the water being filtered at the moment and not old material being forced deeper into the bed. This latter fact was verified by Stanley (51) using radiotracers.

Since the mid 1930's there has been a gradual tendency toward the use of coarser sands, a tendency supported by several workers (1,2,3,27) who observed that coarse sands will yield good filtered water if they are of sufficient depth. Coarser sands will permit deeper penetration and a heavy surface mat is less likely to form. The deeper penetration results in longer runs. Fine sands result in a dense surface mat which is not readily removed in washing. Pieces of the compacted mat adhere to the sand grains, settle to the bottom during backwashing and become the source of mud ball formation. While coarse sands require considerably higher backwash rates to yield 50 percent expansion, lower expansion may yield better cleaning due to greater opportunity for abrasion between the sand grains. These studies have led to the adoption of sands with an effective size of approximately 0.60 mm. The 1936 ASCE report (2) recommended sands with a top size between 0.60 to 1.00 mm.

Eliassen (23) and Tso-Ti Ling (53) using filters equipped with piezometer and sampling connections at intervals through the depth of the filter, have observed that the burden of filter removal moves progressively downward as the filter run progresses. As the upper layers get clogged they remove less and less of the applied sediment load. As the run progresses,

He later modified the index (36) and renamed it the "filtrability index".

$$\text{Filtrability index for laminar flow}^2 = \frac{qd^3h}{l} \qquad \text{Eq 5}$$

Where:

q is the rate of filtration and all other nomenclature as in the floc strength index.

Hudson suggested that turbulence may develop in the clogged portions of the bed towards the end of the filter run and that breakthrough of floc may be associated with the development of turbulence. In accord with this hypothesis, he developed an alternate "filtrability index" for turbulent flow.

$$\text{Filtrability index for turbulent flow} = \frac{qd^2h^{1/2}}{l} \qquad \text{Eq 6}$$

Experimental evidence will be presented in a later section to question this hypothesis concerning the development of turbulence. Since most rapid sand filters continually pass a small amount of turbidity, the use and value of the "breakthrough index" hinge on some arbitrary turbidity level considered a breakthrough.

Hudson urges better control of filtration with a desired turbidity level of less than 0.2 units. He cautions against unsteady flow to the filters, on-off operation, and surges in filtration rates due to the detrimental effect on effluent water quality (37).

Iwasaki (45) studied in minute detail the penetration of colloidal material and bacteria into slow sand filters. Beginning with rational differential equations for the time rate of removal of suspended matter in

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[2]Now called the "breakthrough index" (39).

a filter lamina, he developed the necessary empirical constants for the use of the equations with three types of particulate matter.

Stein (52) elaborated on the mathematical theory of Iwasaki and developed equations for the time rate of removal of suspended matter in a rapid sand filter. The equations were applied to the actual experimental data of Eliassen (23) and gave reasonable agreement with the experimental observations.

4. Present status of the theory of filtration

Several mechanisms for the removal of particulate matter in the rapid sand filter bed have been proposed, but the experimental evidence supporting a predominant mechanism is still a subject for argument. Those mechanisms which have received most attention are straining, sedimentation, and electrokinetics.

The straining mechanism would result in the removal of large particles at the surface and smaller particles at the contact points between the sand grains. Since only the small unsettleable particles will reach the filter of a well designed treatment plant, one might discount surface straining as a major mechanism. However, as the surface gets partially clogged with the few larger particles remaining in suspension, the openings will be reduced to strain out smaller and smaller particles. The heavy surface mat observed on many filters operated at standard filtration rates testifies to the importance of surface straining as one important mechanism of removal. The geometry of the crevices at the contact points of the sand grains would support the possibility that even the smallest particles could be strained out at these crevices if they occupied a flow line close enough to the contact point. Hall (29) has mathematically analyzed the removal

expected from such a mechanism and finds the predicted curves of percent of accumulated sediment vs. depth for particles from 10 to 25 μ closely follow actual curves (2). The observation by Stein (52) that removal was predominant at the contact points of the filter media would also lend support to this interstitial straining mechanism.

The data which supports the straining mechanism would detract from the second mechanism, sedimentation. Stein (52) discounts this theory due to the fact that particles removed had little preference for horizontal surfaces as would be expected if sedimentation were a factor. However, an analysis of the void spaces between the sand grains would lead one to expect their individual action to be similar to small settling basins. Due to the tremendous horizontal sand surface area within the bed and the short vertical settling path, it should be possible to settle a much smaller particle in a filter than in an equal sized settling tank. Fair and Geyer (25, p. 657) show that a filter one m deep composed of 0.55 mm sand would be expected to remove, by settling, particles 1/20th of the diameter of particles removed in a settling basin with equal hydraulic loading. In arriving at this figure, only 1/18th of the total sand grain surface area was assumed to be available for sedimentation. One must consider however that the hydraulic loading of filters and settling tanks is not equal. Settling tanks are commonly designed at 1000 gal/sq ft/day. Filters can operate successfully up to at least 5 gpm/sq ft (7200 gal/sq ft/day). An average porosity of 45 percent for the filter sand would increase the effective filter loading to 16,000 gal/sq ft/day. Applying this difference in hydraulic loading to Fair's example, the filter would then be expected to remove particles only 1/5th as large as the settling tank. Since

filters seem able to remove even the smallest particles, the importance of sedimentation would seem minor as suggested by Stein (52).

If sedimentation is considered unimportant, and if straining is the only active mechanism, the effluent water quality would be expected to improve continually during the run since surface straining should become more effective as the surface becomes clogged. This is contrary to experimental observations which generally indicate a slight degradation of water quality as the run progresses and in some cases a rather sudden decrease in water quality which necessitates termination of the filter run. These observations lead one to the conclusion that some additional mechanism must be active.

Stein (52) suggested "contact-action" as the principal mechanism of removal. He suggested that particles which came in contact with the sand or with previously deposited matter within the bed will adhere until as the filter becomes clogged, the viscous shear forces gradually increase to a magnitude which prevents further deposition, or even tear away already deposited matter. He developed equations for removal of material resulting from contact of particles due to convergence of the stream lines passing through the interstices and found these equations to be in accord with experimental observations.

Stanley (51, p. 6) studied the effect of pH on filtration of iron floc and found the lowest rate of penetration at the iso-electric point of the floc. He explains the importance of pH on filtration as follows:

> An H ion is a specific counter ion which neutralizes hydroxyl groups on hydrous ferric oxide floc. This high affinity for the floc produces positively charged particles when sufficient H ions are present. When particles are broken up in the presence of a large number of H ions, they will not coagulate again as readily,

because the resulting high positive charge causes an increase in the electrokinetic repelling forces.

This reasoning may also be used in explaining why a large percentage of such floc passes through a sand filter. Clean Ottowa sand grains have negatively charged surfaces. Thus, it would seem that positively charged particles in suspension would be readily removed. This is probably true so long as the surface of the filter medium remains negatively charged. However, the small particles in suspension coat the sand grain surfaces very rapidly, producing charge essentially the same as that on the floc particles in the suspension. The particles would then have the best chance of adhering to one of these surfaces if the electrokinetic repelling forces were at a minimum. This occurs at pH values close to the iso-electric point. Thus, the best coagulating floc would also be the best filtering floc.

Thus it would seem that the removal on contact proposed by Stein was probably being aided by the important mechanism of electrokinetic forces as suggested by Stanley. Since these forces would be greatest in the clean filter and gradually decrease as portions of the bed become clogged to a point approaching zero removal, the effluent quality should decrease gradually during the run. This is in agreement with experimental observation.

In summary, two mechanisms working cooperatively seem most feasible.

1. Straining both at the surface and at the interstitial crevices within the bed.
2. Contact adherence of particles coming in close proximity to sand or previously deposited floc facilitated by electrokinetic forces.

III. OBJECTIVES AND SCOPE OF THIS STUDY

The objectives of this study are as follows:

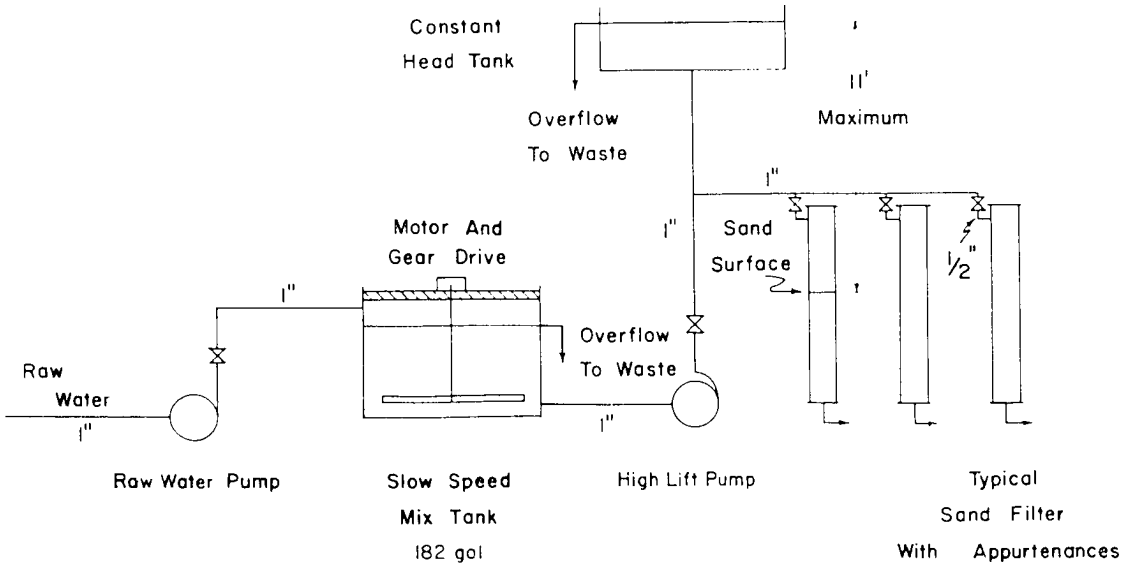
a. to determine whether there is an optimum rate of filtration for a given water supply and for a given filter, from the standpoint of the volume of acceptable water produced per filter run.

b. to establish simple criteria by which an operator can determine whether an optimum rate can be expected for his particular water quality, and, if so; how the optimum rate can be identified.

c. to explain the factors which result in the presence or absence of an optimum rate of filtration on any water supply.

To accomplish the objectives, alternative approaches were available. The first would be to operate one filter at various rates in succeeding time intervals. The second alternative would be to operate several filters each at a different rate simultaneously. In view of the difficulty of maintaining a constant filter influent water quality for an extended period, the latter alternative was selected. A pilot plant was constructed consisting of a raw water pump, mix tank, high lift pump, constant head tank, and three plexiglass filters. Each filter was equipped with a turbidimeter and rate of flow controller. The general schematic arrangement of the pilot plant is shown in Figure 1. Parallel operation of the three identical filters at different rates would permit evaluation of the first objective. Three sources of filter influent water were investigated:

Figure 1. Schematic arrangement of pilot sand filters and auxiliary equipment


Constant
Head Tank
11'
Maximum
Overflow
To Waste
1"
Motor And
Gear Drive
1"
Sand
Surface
1/2"
Overflow
To Waste
Raw
Water
1"
1"
Raw Water Pump
Slow Speed
Mix Tank
182 gal
High Lift Pump
Typical
Sand Filter
With Appurtenances

a. Raw aerated well water from the Ames municipal wells which contains between 8 and 9 mg/l total iron. One-half mg/l of copper was added as copper sulphate and the water was mixed in a slow speed mix tank to assure complete precipitation of the hydrous ferric oxide floc.

b. Ames city tap water to which ferric chloride or ferrous sulphate solution was added through a constant head capillary tube feeder to yield the desired iron concentration. The mixture was mixed in a slow speed mix tank to form the hydrous ferric oxide floc.

c. Filter influent water of the Ames municipal water treatment plant. This water was diluted with tap water in some filter runs to adjust to a desirable turbidity level and to aid in its stabilization prior to use in the pilot filters. The Ames municipal water treatment plant uses a typical split treatment, lime-soda ash softening process. The treatment steps include aeration, chemical addition, mixing, settling, recarbonation, sludge addition, mixing, settling, recarbonation, filtration, chlorination, flouridation, and metaphosphate stabilization.

IV. PILOT PLANT APPARATUS

All components of the pilot plant shown schematically in Figure 1 were designed to enable operation of the three filters at any desired rate between 1 and 10 gpm/sq ft. Each component in the pilot plant is described in the following paragraphs.

A. Pumps and Tanks

1. Raw water pump to the mixing tank

A 1-1/2 in. x 1-1/2 in., 1/8 hp, 1750 rpm close coupled centrifugal pump was used to pump the water from the aerator, or from the filter influent of the Ames municipal plant, to the mixing tank.

2. Slow speed mix tank

A mixing tank 3 ft high and 3.5 ft in diameter equipped with a slow speed paddle was used to provide the necessary detention time to complete the iron precipitation reaction. Allowing 6 in. of freeboard, this 182 gal tank provided a 30 minute theoretical detention time at a pumping rate of 6 gpm.

The slow speed mix tank was also used when filtering the lime-soda ash softened water from the Ames municipal plant. Water was pumped to the mixing tank from the recarbonation tank effluent. The mixing time aided in completing the recarbonation reaction thus providing a more stable water to the pilot plant filters. In some filter runs, tap water was added to the Ames filter influent water to regulate turbidity and aid in stabilization.

3. High lift pump to the constant head tank

A 3/4 in. x 1/2 in., 1/3 hp, 3450 rpm close coupled centrifugal pump was used to pump water from the slow speed mix tank to the constant head tank.

4. Constant head tank

All water to be filtered was pumped to a constant head tank 12 in. in diameter by 18 in. deep equipped with a 10 in. overflow weir. The tank was located to provide 10 ft of head above the filter sand surface. The water flowed by gravity to the three filters from the constant head tank. Head on the three filters varied slightly due to the difference in head loss from the constant head tank to the filters at different filtration rates. The head variation at different rates did not exceed 0.4 ft (4%).

B. Filters and Appurtenances

1. Pilot sand filters

Three sand filters shown in Figures 2 and 3 were constructed of 6 in. ID plexiglass tubes 1/2 in. thick and 53 in. long. This diameter provided a sand surface area of 0.196 sq ft in each filter. Combination piezometer and sampling connections of 1/4 in. ID plexiglass tubing were installed at close intervals through the sand depth. The inside ends of the piezometer connections were 2, 3, or 4 in. from the inner face of the filter shell, alternating to prevent two adjacent sampling points from lying directly in a vertical line. The inner end of the 1/4 in. plastic tube was molded to permit the easy entrance of water but to prevent the passage of sand.

The underdrain system for each filter consisted of 9 in. of graded

Figure 2. Pilot sand filter and control equipment

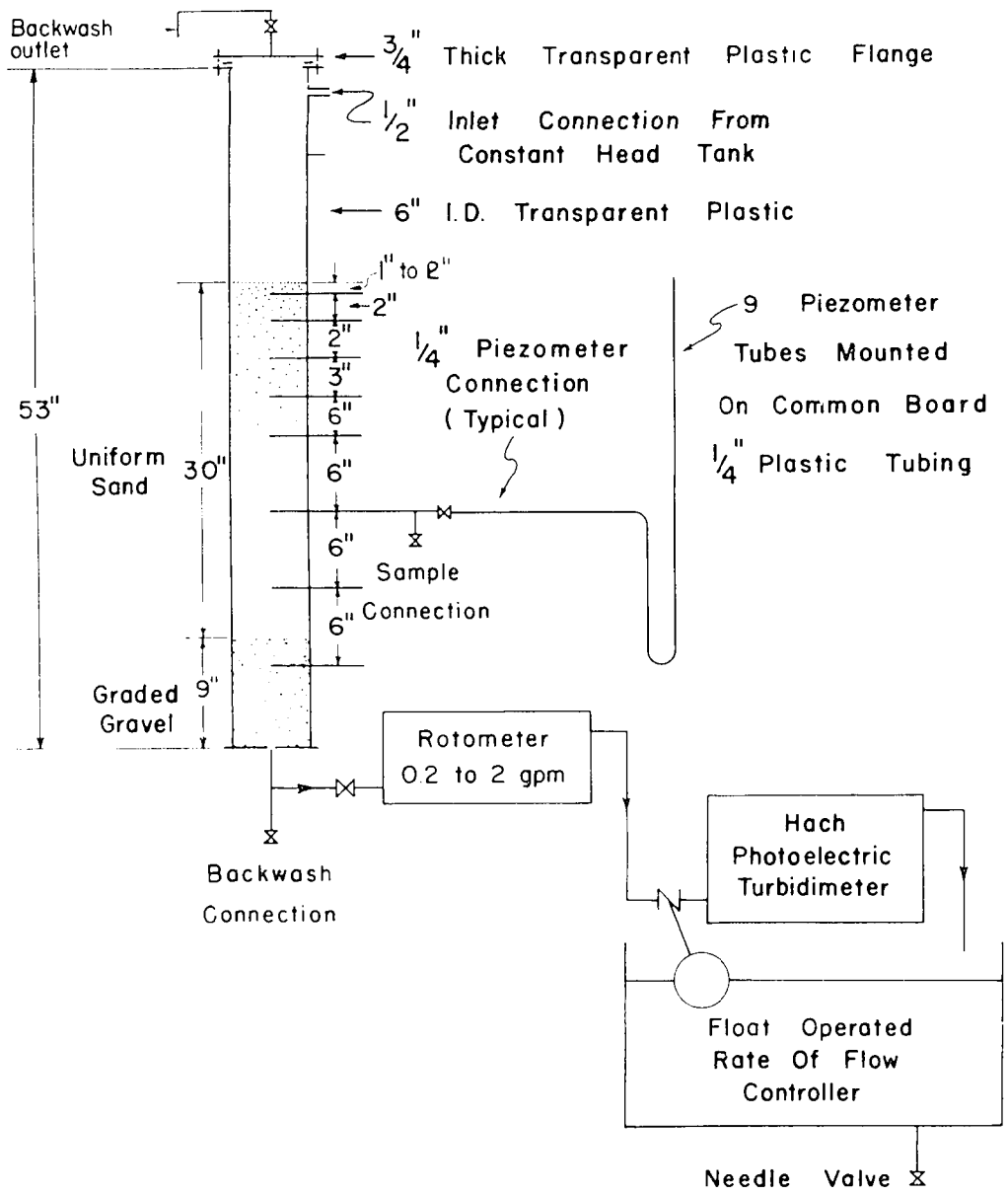
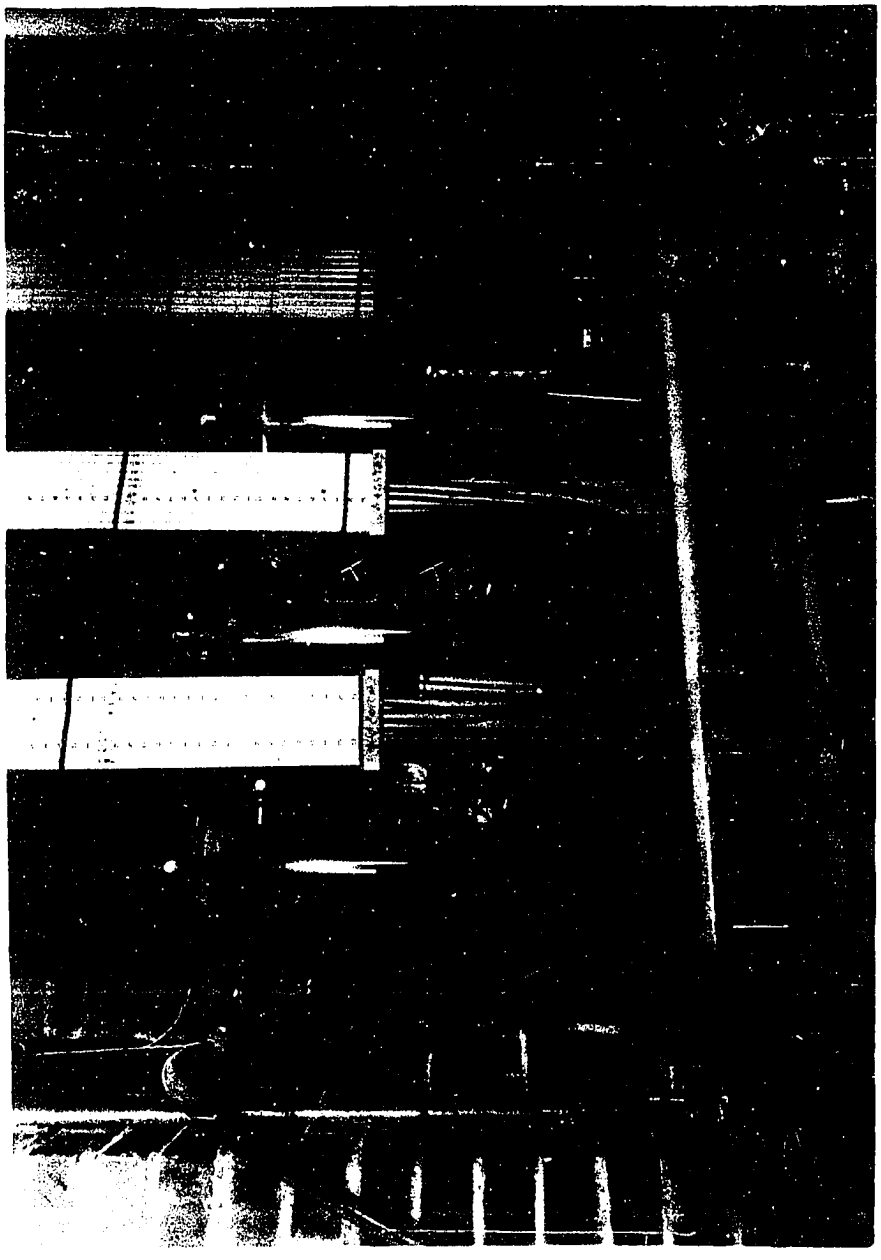

Backwash outlet
3/4" Thick Transparent Plastic Flange
1/2" Inlet Connection From Constant Head Tank
6" I.D. Transparent Plastic
1" to 2"
2"
2"
3"
6"
6"
6"
6"
1/4" Piezometer Connection (Typical)
9 Piezometer Tubes Mounted On Common Board
1/4" Plastic Tubing
53"
Uniform Sand
30"
Graded Gravel
9"
Sample Connection
Rotometer 0.2 to 2 gpm
Hach Photoelectric Turbidimeter
Backwash Connection
Float Operated Rate Of Flow Controller
Needle Valve

Figure 3. Pilot sand filters and bottom of piezometer boards



gravel placed over a perforated aluminum cup which in turn was centered over the 1/2 in. filter effluent and backwash connection. Thirty in. of sand was provided above the gravel. Filter influent water entered approximately 13 in. above the sand surface through a 3/8 in. pipe connection. Backwash water leaves through a 3/4 in. pipe connection in the top flange of the filter.

Immediately outside the filter, each piezometer connection was provided with a glass "T". One connection led to a piezometer board and the other connection was provided with a rubber tube and screw clamp for sampling purposes.

The piezometer boards were 10 ft long and equipped with 9 - 4 mm ID glass tubes. The upper ends of the piezometer tubes were connected to a manifold header to permit the application of a small constant pressure on all tubes. The pressure was exerted by a rubber squeeze bulb at the beginning of each run to depress the water level in the tubes to approximately the middle of the board to facilitate reading.

2. Filter sand

Filter sand was obtained from the Northern Gravel Company of Muscatine, Iowa. Several graded sands were obtained and analyzed using U.S. Standard sieves. The size characteristics of the various grades ar. shown in Table 6. Sands used in the various experimental runs are referred to by the designation in the table.

3. Flow meters

Effluent from each filter passed through a variable area glass tube, float type, flow meter suitable for water flow measurement from 0.2 to

Table 6. Filter sand characteristics

Sand designation	Effective size, mm	Uniformity coefficient
A	0.50	1.46
B	1.34	1.38
C	1.96	1.33
D	1.55	1.89
E	1.05	1.38

2.0 gpm[1] (Figure 4). The flow meters were calibrated by determining the time to fill a two liter volumetric flask. The calibrations gave somewhat different results at different times in the experimental work, apparently due to the deposition of a small amount of material on the inside of the glass tube or the float. The experimental data for the various filter runs was evaluated in terms of the actual flow as determined by the calibration nearest the time of the particular run.

4. Turbidimeters

Each filter effluent is monitored by a photoelectric turbidimeter (Figure 5) which uses a nephelometric method to measure turbidity[2]. The precision and accuracy of these instruments is discussed in a later section on laboratory techniques.

5. Rate of flow controllers

The effluent from each turbidimeter passed into a float operated rate of flow controller shown in Figure 6. The controller maintained a constant rate of filtration by holding a constant head on a needle valve outlet.

[1]Fischer and Porter "Flowrator".

[2]Hach Chemical Company, Ames, Iowa, CR Low Range Turbidimeter.

Figure 4. Flow meter

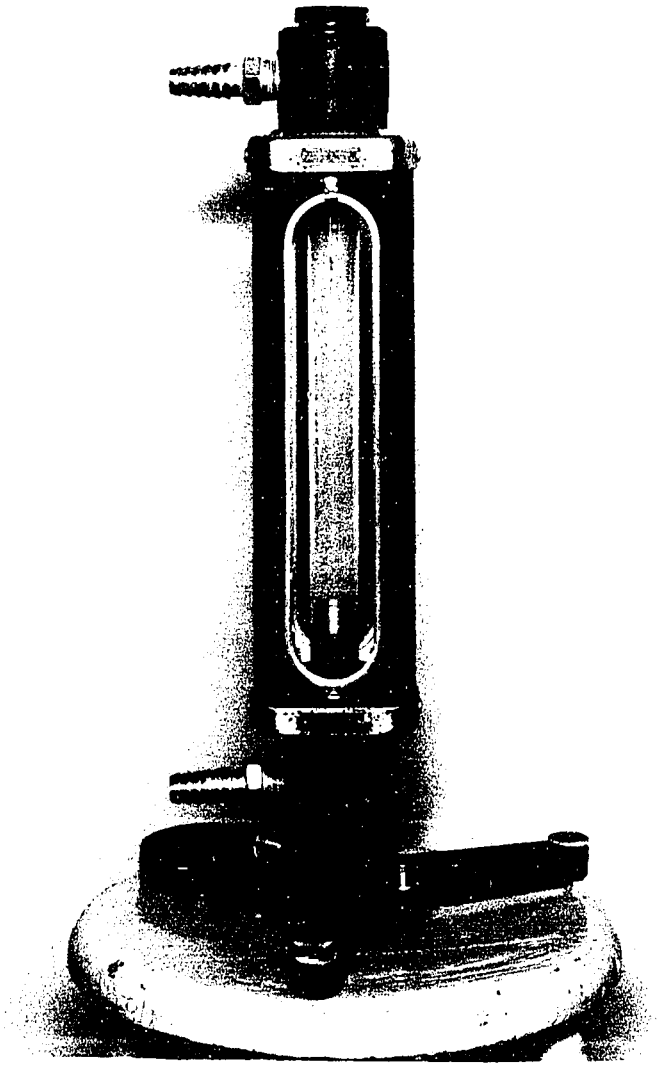
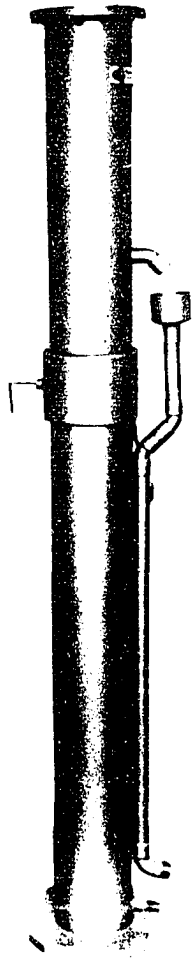


Figure 5. Photoelectric turbidimeter



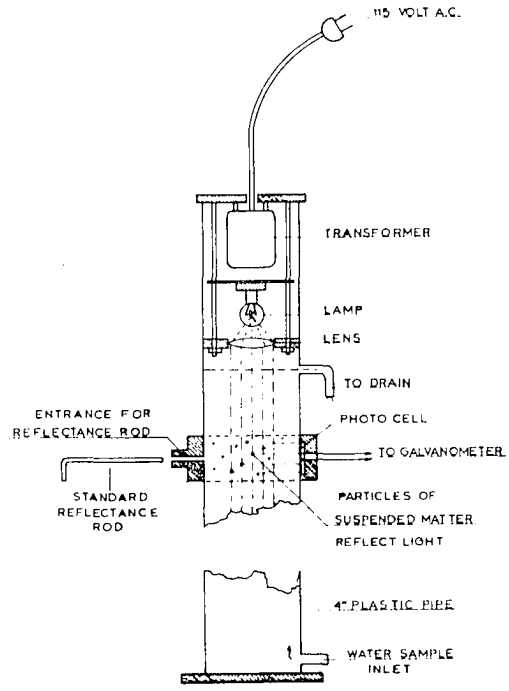
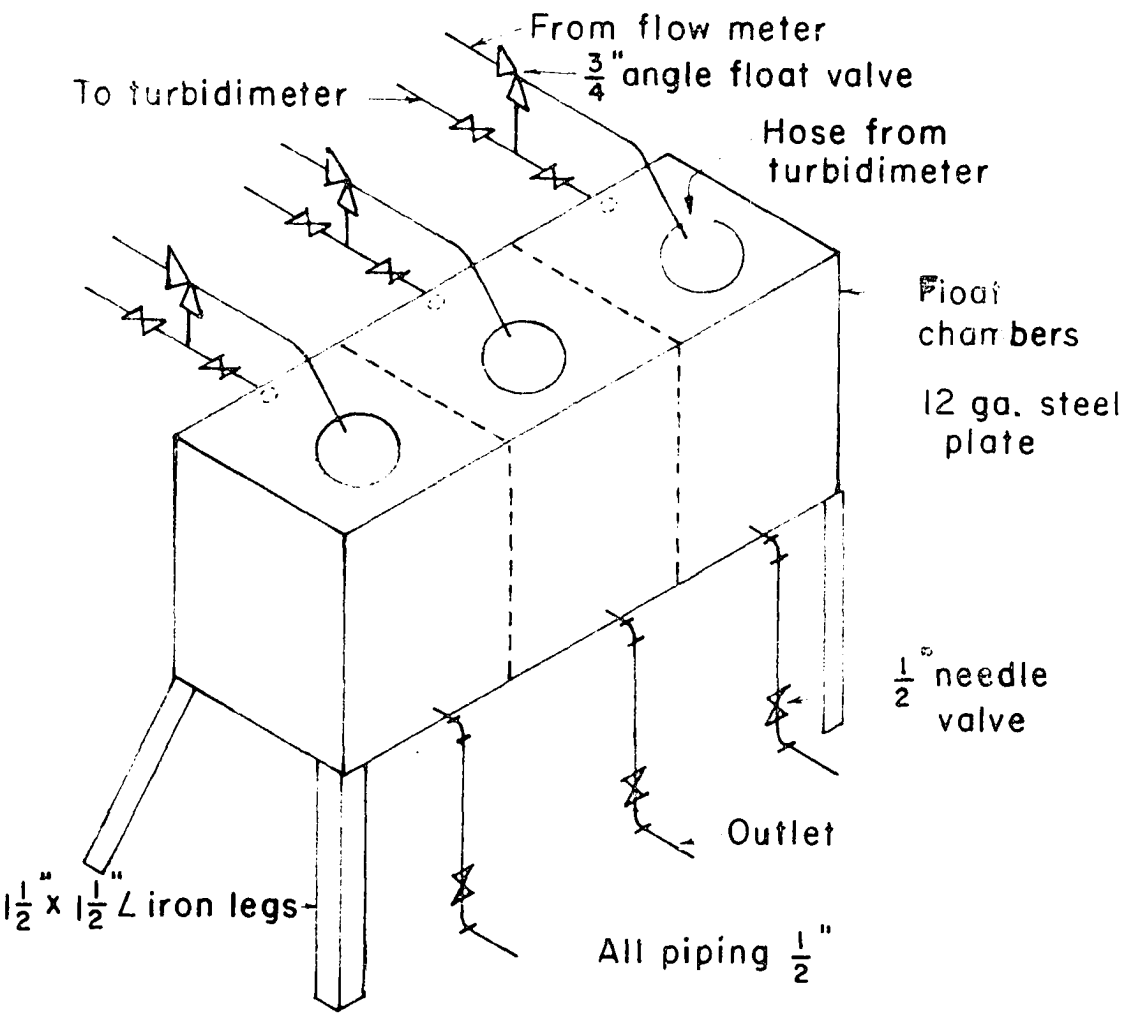

115 VOLT A.C.
TRANSFORMER
LAMP
LENS
TO DRAIN
PHOTO CELL
TO GALVANOMETER
ENTRANCE FOR REFLECTANCE ROD
STANDARD REFLECTANCE ROD
PARTICLES OF SUSPENDED MATTER REFLECT LIGHT
4" PLASTIC PIPE
WATER SAMPLE INLET

Figure 6. Filter rate of flow controllers


From flow meter
3/4" angle float valve
To turbidimeter
Hose from turbidimeter
Float chambers
12 ga. steel plate
1/2" needle valve
Outlet
1 1/2" x 1 1/2" ∠ iron legs
All piping 1/2"

The float chamber, constructed of 12 gauge steel plate, was 9 in. x 9 in. x 12 in. deep. One-half in. needle valves, 3/4 in. float valves, and 4 in. diameter copper floats were provided. Piping was arranged to pass all or any portion of the filter effluent through the turbidimeter prior to dischange to the float chamber.

The rate of flow controllers functioned remarkably well. Occasionally, the float valve might temporarily stick and, when it suddenly released itself, a surge of high flow rate might pass through the filter. When this occurred, some of the previously deposited sediment would be flushed through the filter, resulting in a drop in head loss. The further results for such a run were invalid and the run was terminated. Fortunately, this rarely occurred.

As the head loss through the filter increased, during a run the float valve would gradually open to maintain a constant filtration rate. As the float valve opened, the head on the needle valve was decreased slightly and the rate of filtration would decrease approximately in proportion to the square root of the ratio of the head change. To maintain constant rate, the needle valve was opened very slightly to compensate for this head change.

V. EXPERIMENTAL OBSERVATIONS

A. General

The initial phase of the study was directed at the first objective, namely; to determine whether there is an optimum rate of filtration from the standpoint of the volume of acceptable water produced on a given filter per filter run. When the first objective had been accomplished, additional runs were conducted with the pilot plant to shed light on the second and third objectives, namely: how can the operator identify the optimum rate and what factors lead to the presence or absence of an optimum rate.

B. Laboratory Techniques

1. Turbidity

During the experimental runs using Ames filter influent water as the source of influent water for the pilot filters, the pilot filter effluent quality was evaluated on the basis of its turbidity content. Turbidity of water is that quality which interferes with the passage of light through the water, or which restricts visual depth. It is caused by a wide variety of suspended material ranging in size from colloidal to coarse dispersions. The accurate determination of this optical property of a water presents many problems. The recent development of a new type of photoelectric turbidimeter[1], which is particularly adapted to low levels of turbidity such as those encountered in filter effluent water, was a great help in

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[1]Hach Chemical Company, Ames, Iowa. CR Low Range Turbidimeter.

this research.

Each filter was equipped with a photoelectric turbidimeter. Each turbidimeter has a 4 in. ID plastic tube approximately 4 ft long. The water enters the bottom and overflows through a pipe connection approximately 1 ft from the top. At the top of the tube above the overflow are a light source and two lenses. The light source and lens system sends a 2 in. diameter light beam axially down through the water column. A number of selenium photoelectric cells are mounted around the circumference of the tube at about mid depth. The interior of the tube is dull black to prevent reflection of light from the bottom or sides of the tube. As the light beam passes through the water column, light is scattered by the turbidity particles due to the Tyndall effect and the scattered light received by the photoelectric cells results in the generation of a small electric potential which is measured on a highly sensitive lamp and scale galvanometer. The intensity of the scattered light is in proportion to the surface area of the particles in suspension. A potentiometer in parallel with the galvanometer can be adjusted to impress any desired portion of the photoelectric cell output through the galvanometer. In these experiments, the potentiometer was generally set to yield one mm scale deflection of the galvanometer for 0.02 units of turbidity. With this potentiometer setting, turbidity was easily read with a fineness of 0.02 unit. However, it is a well recognized fact that turbidity is a difficult quality to measure. Many problems prevent the accurate determination of an absolute turbidity level for any water sample.

The standard method (50) for determination of turbidity is the Jackson candle turbidimeter which can be used only above 25 units of

turbidity. Other instruments may be used in place of the Jackson meter, but they must be calibrated against the Jackson candle turbidimeter by making up a suspension above 25 units turbidity and diluting to the level required in the instrument to be used. This requirement is the reason for inaccuracies in calibration.

a. Human inconsistancies are involved in the visual observations required with the Jackson candle turbidimeter in preparation of initial standards.

b. The type of suspended matter used in standardizing the instrument. The standard method suggests using suspended matter in calibration of the same type that will be observed in subsequent use of the instrument.

c. The possibility that the suspension used in calibration may partially dissolve when diluted to less than 1 unit.

d. The changes in particle size or other characteristics which may occur during dilution and handling as a result of natural coagulation.

Further, in the accurate measurement of turbidities less than one unit, zero turbidity water must normally be used in preparing the desired dilutions for calibration. An instrument such as the Baylis Turbidimeter requires an absolute zero turbidity dilution water to be used in preparation of the standards. Any turbidity in the dilution water will result in inaccuracy.

An advantage of the CR meter is that the meter calibration does not depend on zero turbidity water for the preparation of the standards. The photocell output can be observed for the dilution water and the increase

in output can then be observed for any desired level of added turbidity.

The photocell output of the meter is essentially zero for zero turbidity water. Various methods were used in an attempt to prepare a zero turbidity water including low rate sand filtration, recycled diatomaceous earth filtration, membrane filtration of distilled water, and recycled activated carbon filtration. The last method produced a water which most nearly approached zero turbidity. The filter was a perforated copper pipe septum wound with felt cloth 1/2 in. thick. Activated carbon was the filtering medium, and water was recycled through the turbidimeter and the filter. The galvanometer soon reached a constant reading equivalent to approximately 0.05 units turbidity. This photocell output may be due partially to reflected light reaching the photocells within the meter.

Various other problems involved in low level turbidity monitoring are solved with this meter. Some photoelectric turbidimeters measure both transmitted light and scattered light. Though this is a desirable method of observing the optical properties of medium to high turbidity suspensions, the change in transmitted light is insignificant when dealing with very low water turbidities. One unit of turbidity reduces transmitted light by about one percent, and it is difficult to measure such a small difference between two relatively large values accurately. In contrast, an increase of from 0.1 to 1.0 unit of turbidity increases the scattered light approximately tenfold. Such a change can be measured accurately. For this reason, the meter attempts only to measure scattered light.

The measurement of transmitted light presents some additional problems. Color in water from other causes than turbidity will also reduce the intensity of the transmitted beam and will be measured as turbidity.

The manner in which the light beam enters the water may also cause false turbidity readings. In some photoelectric turbidimeters, the light beam passes through a window into the water, and the scattered and transmitted light pass through other windows to the photocells. Submerged windows of this type tend to collect air bubbles or to become fouled with slime. This fouling will scatter the light of the incoming beam and will give false high measurements of scattered light. Fouling will also reduce the intensity of the transmitted beam. In this meter, the light beam enters directly through the surface of the water which is constantly overflowing and is therefore self cleansing.

The windows in this meter through which the scattered light pass may become fouled, and this will result in slightly low readings, but it cannot result in false high readings. The meter is furnished with a glass reflectance rod which can be used to check the potentiometer adjustment and proper functioning of the meter. The potentiometer adjustment can be measured by the galvanometer deflection (in millimeters) caused by the light reflected by the reflectance rod. This value will subsequently be called the sensitivity. If the sensitivity decreases due to fouling, the window can be cleaned, or the potentiometer can be adjusted to increase the sensitivity to the desired level.

The manufacturer of the CR Low Range turbidimeter anticipated a possible correlation between the light reflected by the reflectance rod and the light reflected by some standard unit turbidity suspension. This relationship will vary for different waters due to the variations in particle characteristic and size distribution. This relation, therefore, must be observed for the particular suspension in question.

During the studies with Ames filter influent water, the relationships between galvanometer scale increase, turbidity, and sensitivity were studied. Backwash water from the filter was used in preparing standard solutions of 1 unit, 0.5 unit, and 0.25 unit turbidity which were passed through the turbidimeters. The sensitivity of the turbidimeters was observed. The galvanometer scale increase for the various turbidities was noted at several different sensitivities.

The galvanometer scale reading increased in direct proportion to the added turbidity at these low levels, and the galvanometer scale reading for a given turbidity increased in direct proportion to the sensitivity. Since both turbidity and sensitivity have a linear effect on the galvanometer reading, the calibration for a particular water may be stated in terms of the mm of galvanometer deflection per unit turbidity per mm sensitivity. For the Ames filter influent water, this value was found to be 2.95.

In view of the foregoing discussion, it can be concluded that absolute values of turbidity will remain in question as long as the Jackson candle turbidimeter remains as the standard. When carefully calibrated for a particular water, the CR Low Range turbidimeter will give consistent relative results to a fineness of 0.02 units.

Operational instructions of importance in the use of the turbidimeters are summarized below:

a. Clean the photocell windows periodically. Deposition of material in the windows will result in reduced sensitivity.

b. Clean lens system carefully before each experimental filter run.

c. Check sensitivity at beginning and end of each experimental run;

and in long runs, at intervals during the run. Adjust sensitivity carefully.

d. Several hours are required for the sensitivity to stabilize when the meter power has been turned on after a period of idleness.

e. Limit flow thru the meter to prevent submergence of the outlet pipe. Submergence prevents the water surface from being self cleaning and may cause the lens system to get wet.

f. Voltage regulators on the light source power are necessary to stabilize sensitivity and thus the turbidity readings.

2. Iron

During the experimental filter runs using water containing precipitated hydrous ferric oxide particles, the effluent quality was evaluated on the basis of the iron content. Since several thousand analyses of total iron were required during the experimental work; a simple, accurate method of iron analysis was desireable.

During the first year of the research, the 1,10 phenanthroline method was selected (50) since it is the standard method for the water supply field. This method is not adequately sensitive when measuring iron at the low levels encountered in typical filter effluents, generally less than 0.3 mg/l. For this reason, in the last year of the research, a new reagent was used which has greater sensitivity for low levels of iron. The properties and use of this reagent, 2,4,6,-tripyridyl-s-triazine, commonly abbreviated TPTZ, are discussed by Diehl (21).

1,10-Phenanthroline reacts with the ferrous ion to yield an orange-red color which obeys Beer's law and is therefore suited to colorimetric

determination. Three molecules of 1,10-phenanthroline chelate each ferrous ion.

TPTZ reacts with the ferrous ion to yield an intense violet color which obeys Beer's law and is also suited to colorimetric determination. Two molecules of TPTZ are required to chelate each ferrous ion.

Both reagents are specific only for the soluble ferrous ion. Therefore, the normal method for total iron involves dissolving and reducing any precipitated iron with acid and hydroxyl ammonium chloride before analysis. In some cases, heat is required to complete the solution of the precipitated iron. The reagents can be purchased in a patented powder form which will dissolve and reduce the iron in a single step[2] without heating. To speed up the analyses, the patented reagents were used. No interfering ions were present in the Ames water in sufficient quantity to affect the accuracy of the results. Color was observed on either a photoelectric filter photometer[3] or a photoelectric spectrophotometer[4].

C. Effluent Water Quality Requirements

In evaluating the first objective, it is necessary to set a standard of acceptable water quality. Since the water in question is essentially sterile as it is pumped from the wells, a bacterial evaluation would be of little value.

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[2]Hach Chemical Company - FerroVer powder
TeeVer powder.

[3]Hach Chemical Company - DR Colorimeter.

[4]Baush and Lomb - Spectronic 20.

In those runs where water containing precipitated iron was filtered, an iron content in the effluent of greater than 0.3 mg/l was considered unsatisfactory since this is the standard set by the U.S. Public Health Service (48).

In those runs where the filter influent water of the Ames municipal plant was filtered, the effluent quality was evaluated on the basis of its turbidity content. U. S. Public Health Service drinking water standards limit turbidity to 10 units; however, many plants strive for a much better water quality. Baylis (8) and Babbitt (4, p. 384) state that "turbidity greater than about 0.5 unit may be noticeable to the consumer when the water is held in a white enamel container".

Baylis (8) and Hudson (38,39) have indicated that less than 0.2 unit of turbidity is a desireable goal. However, it is doubtful that the techniques used in their studies would yield valid results at this level. Turbidity in their studies was measured by the Baylis Turbidimeter calibrated against suspensions observed on the candle turbidimeter and diluted to the desired level. Since it is unlikely that they had absolute zero turbidity water in making the dilutions, the absolute level of their turbidity is subject to question.

In this study, a maximum level of 0.5 unit of effluent turbidity was arbitrarily set as a desireable goal.

D. Brief Chronology of Experimental Runs

The first three months of operation of the pilot plant involved some wasted effort as operating techniques were refined to eliminate erroneous influences. While many of the early runs have invalid results, important

operational lessons were learned. Table 7 summarizes the various runs performed. A brief chronological discussion of them follows.

Sand E[5] which is a very coarse sand with an effective size of 1.05 mm was chosen for the initial runs to enable appreciable penetration to occur. It was felt that an optimum rate would be observed only if significant changes in penetration could occur at different rates.

Runs 1, 2, and 3 are invalid due to disturbances in the rate of filtration which resulted in passage of iron through the filter and decrease in head loss. Excessive iron was passed at rates above 2 gpm/sq ft with the coarse sand in use.

Runs 4, 5, and 6 reflect efforts to prepare a larger floc particle size with ferric chloride and city tap water in the hope that the larger floc would not pass through the coarse sand in use. Even though a larger floc was obtained, excessive iron was passed and all three runs are invalid for this reason.

Runs 7 and 8, in which highly turbid water from the second stage mixing tank of the Ames municipal plant was filtered, are invalid due to passage of excessive material through the filter.

It was evident from runs 1 through 8 that a finer sand would be necessary to permit filtration within the desired rates of 1 to 10 gpm/sq ft for which the pilot plant was designed. Sand D with an effective size of 0.55 mm was therefore installed in all three filters.

Runs 9 through 15 with the finer sand indicated that acceptable iron removal could be accomplished at rates up to 6 gpm/sq ft. However, these

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[5]See Table 6.

Table 7. Summary of experimental runs on pilot plant filters

Run no.	Sand[b]	Filter influent water type			Filtration rates[a]		
		Source[c]	Treatment[d]	Additives[e]	Filt. 1	Filt. 2	Filt. 3
1	E	1	1,2		2.5	4	9.5
2	E	1	1,2	1	1	2	4
3	E	1	2	1	0.6	1	2
4	E	5	1	3	1	2	3.5
5	E	5	1	3,5	1	2	3.5
6	E	5	1	3,4	1	2	3.5
7	E	2	1				2
8	E	2	1		1	3	5

[a] gpm/sq ft.

[b] See Table 6.

[c] Water sources designated as follows:
1. Raw Ames city well water
2. Ames treatment plant, no. 2 mix tank water
3. Ames filter influent water, 1 well in operation
4. Ames filter influent water, 2 wells in operation
5. Ames city tap water.

[d] Treatment of source as follows:
1. Mixed in slow mix tank
2. Aerated in tray type aerator
3. Diluted with tap water to control turbidity and stability.

[e] Additives to the source as follows:
1. 1 mg/l copper as copper sulphate solution
2. 1/2 mg/l copper as copper sulphate solution
3. Ferric chloride to yield 10 mg/l iron content
4. NaOH solution added to hold pH at 10
5. "Nalco 600" coagulant aid added at 7 mg/l.

Table 7. (Continued)

Run no.	Sand[b]	Filter influent water type			Filtration rates[a]		
		Source[c]	Treatment[d]	Additives[e]	Filt. 1	Filt. 2	Filt. 3
9	D	1	1,2	1	1	2	3
10	D	1	1,2	1	3	2	4
11	D	1	1,2	1	3	2	4
12	D	1	1,2	1	5	6	4
13	D	1	1,2	1	6	2	8[f]
14	D	1	1,2	1	0.7	3	6
15	D	1	1,2	1	3	3	3
16	D	1	1,2	1	0.7	3 4,5	6[g] 2
17	D	1	1,2	1	3	3	3
18	D	3			1 6	3 7	5 2
19	D	4			6 3	5	2
20	D	4	1		6	4	2
21	D	1	1,2	2			6
22	D	1	1,2	2	2	8 6	4
23	D	1	1,2	2	6 6[f],1	4	2
24	D	4	1,3		1 4	6 3	2
25	D	4	1,3		2	3	4 6

[f]Constant pressure operation, rate indicated is the starting rate on the clean filter.

[g]During and after run 16, the higher rate filters were frequently backwashed and operated at more than one rate. This was possible since the run length at low rates was considerably longer than at high rates.

runs cannot be used to evaluate the first objective for a number of reasons. Rate disturbances occurred during run 9. Inconsistent results during runs 10 through 15 led to the suspicion that the sand was not identical in all three filters at the beginning of the runs.

After refinements in technique, and slight changes in the sand to provide uniform head losses through the clean filter bed, several valid runs were obtained which permitted evaluation of the first objective (runs 16,18,19,20,22,23,24, and 25).

E. Refinements in Operational Technique

Several operational refinements were found necessary during runs 1 through 15. The observations and refinements are discussed below.

1. Incomplete precipitation of iron

Complete precipitation of the iron did not occur due to mixing along in run 1. To facilitate precipitation, 1/2 to 1 mg/l of copper was added in the form of copper sulphate solution in all subsequent runs on raw aerated water. The copper catalyst consistently gave soluble iron levels entering the filter of less than 0.10 mg/l.

2. Rate disturbances

Sediment which had been removed in the filter was very sensitive to rate changes. Any sudden increase in rate would flush a large amount of sediment out of the filter with a resultant decrease in head loss. The effluent quality would quickly recover but such rate increases would invalidate the results for a given run. Therefore, it was necessary to eliminate, as nearly as possible, all sources of rate change. The precautions taken are discussed below.

a. Sample withdrawal To prevent the rate change that would occur if a sample were periodically taken from any of the piezometer sampling connections, such samples had to be withdrawn continuously through a 0.5 mm ID capillary tube of sufficient length to provide a sampling rate of approximately 5 ml/minute.

b. Float valve difficulties Float valves on the controller were adjusted to give minimum linkage friction to try to prevent the float from sticking and suddenly releasing with a resulting rate change.

c. Air-binding Release of dissolved gases within the filter media (air binding) towards the end of a filter run would result in reduction in available filter area and thus increase the effective filtration rate. Such occurrences were observed to have the same effect as the other causes of rate change. It was necessary to terminate the filter runs if such air-binding was observed.

3. Initial start up

The procedure followed at the beginning of a filter run was found to be important in obtaining uniform conditions. It was necessary to adjust the rate of flow controller to the desired rates prior to commencement of the actual run. Several operational procedures were attempted. The method finally selected consisted of opening the backwash supply valve slightly so that water flowed through the turbidimeter and controller and a small amount of excess water flowed upward through the filter and out the backwash outlet. The controller was then set at the desired rate while passing the backwash water. Using this procedure the clean filter was not affected during the time that was required for adjustment of the rate of flow controller. When the rates were properly adjusted on each filter,

the run was commenced simultaneously on all three filters.

4. Backwash

It was observed during runs 9 to 15 that the manner used in closing the backwash valve affected the porosity of the sand and the resulting head loss development in a subsequent run. In all subsequent runs, the backwash needle valve was turned off 1/6 revolution at a time. The sand was permitted to reach equilibrium before the valve was closed further.

When this precaution was practiced, it was noted that some difference in sand depth was present in the three filters following backwashing. City tap water was filtered to observe the loss of head through the three clean filters at the same rate. Filter 1 had considerably more loss in the upper layers than the other two filters. Portions of sand were removed from the desired layers through the top piezometer connection while backwashing the filter. After several trials at sand adjustment, the clean bed head loss was found to be nearly identical in all filters at the same rate. Filter 1 had somewhat smaller head loss in the deeper layers but this had little affect on the head loss development since most removal takes place in the upper sand layers.

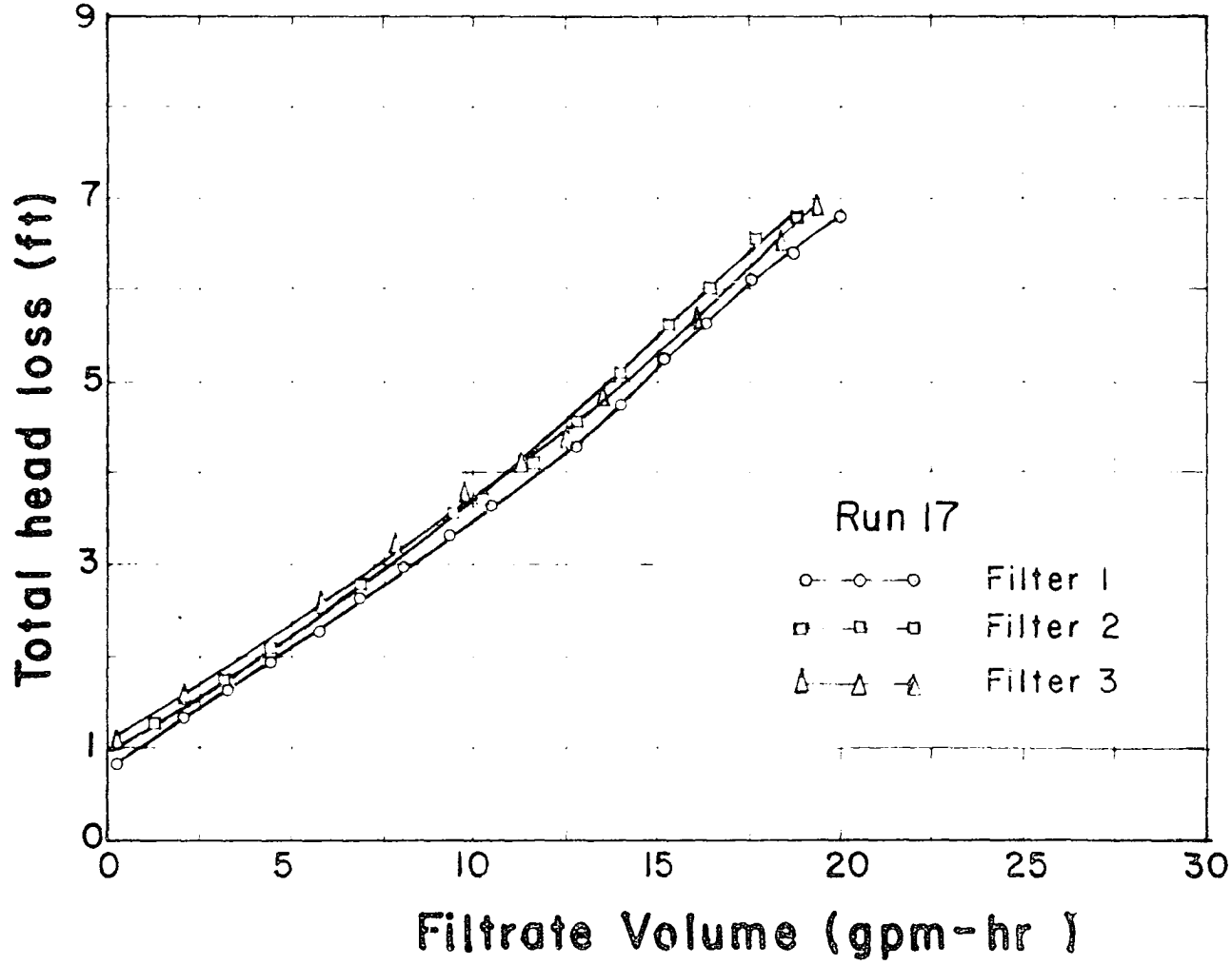
All three filters were held at the same rate during run 17 to check the comparability of the head loss development. The total head loss vs. volume of filtrate curve for the run is shown in Figure 7. It is evident that the sands are nearly identical in hydraulic characteristics.

The plotting of total head loss vs. filtrate volume as in Figure 7 is one of many such plots presented in this paper. Thus, some explanation of the units used may aid in proper interpretation of the curves.

The ordinate, total head loss, represents the total drop in pressure

Figure 7. Run 17, total head loss vs. filtrate volume

All filters were operated at 3 gpm/sq ft for the purpose of comparing sand characteristics.


Total head loss (ft)
Filtrate Volume (gpm-hr)
Run 17
Filter 1
Filter 2
Filter 3
0
1
3
5
7
9
0
5
10
15
20
25
30

(ft of water) from the top piezometer tube above the sand to the bottom piezometer tube in the gravel layer, including the clean bed head loss. The head loss is given at the actual water temperature of the filter run as noted in the tabulations of filter influent water quality data. No attempt was made to correct for viscosity changes to any standard temperature since temperature changes result in other more important head loss influences which cannot be evaluated. Thus, to correct for one variable and not for the others might tend to obscure the true importance of temperature change on head loss development. The temperature change during any single filter run never exceeded 5° F, and was generally less than 3° F.

The abscica, filtrate volume, is expressed in the volume unit, gpm-hr. This volume unit was selected since it is readily determined as the product of the experimental observations made, rate (gpm) and time (hr). This unit of volume can also readily be converted to run length (hr) or to volume expressed in gal or gal per sq ft of filter area. One gpm-hr is equal to 60 gal. Volume (gpm-hr) divided by rate (gpm) will yield the corresponding run length (hr) to produce the indicated volume. Since the sand filter area in the pilot filters is 0.196 sq ft, it is a simple matter to convert to volume/sq ft if so desired. Consider a simple example with a given filtration rate of 4 gpm/sq ft and a filtrate volume of 10 gpm-hr. This is equivalent to a volume of (10 gpm-hr)(60 min/hr) = 600 gal. The filtration rate is (4 gpm/sq ft)(0.196 sq ft) = 0.784 gpm. The run length to produce the indicated volume is (10 gpm-hr)/(0.784 gpm) = 12.8 hr.

F. Experimental Runs without a Pronounced Optimum Rate

After the various refinements in technique had been developed to eliminate erroneous influences, it was possible to make a series of valid experimental runs to evaluate the first objective of this study, namely: is there an optimum rate of filtration from the standpoint of maximum volume of acceptable water produced per filter run.

When filtering water containing precipitated hydrous ferric oxide particles, no pronounced optimum rate was apparent. Figures 8, 9, and 10 for run 16, 22, and 23 are typical total head loss vs. volume of filtrate curves for this type of filter influent water. Filter influent water in runs 16, 22 and 23 had the following average characteristics:

Total Iron: 7-9 ———— mg/l

Ferrous Iron: 0.00-0.07 mg/l

Temperature: 57-58° F Run 16 & 23
57-62° F Run 22

pH: 7.5 to 7.7

Certain general characteristics are evident in Figures 8, 9, and 10. The curves are nearly linear, particularly at the higher filtration rates. The curves tend to be parallel. No pronounced optimum rate is present with nearly linear curves and the lowest filtration rate will generally produce the largest volume of water to a given terminal head loss. An exception to this is the very slight optimum obtained at 3 gpm/sq ft in run 16.

Effluent water quality at a terminal loss of 6 ft was acceptable except at rates of 6 gpm/sq ft or above. Effluent water quality was the poorest near the beginning of each filter run, reaching a peak in tur-

Figure 8. Run 16, total head loss vs. filtrate volume

Typical filter run which shows no strong optimum rate tendency. Filtering water containing precipitated hydrous ferric oxide. Curve for 6 gpm/sq ft not shown due to erratic initial behavior.

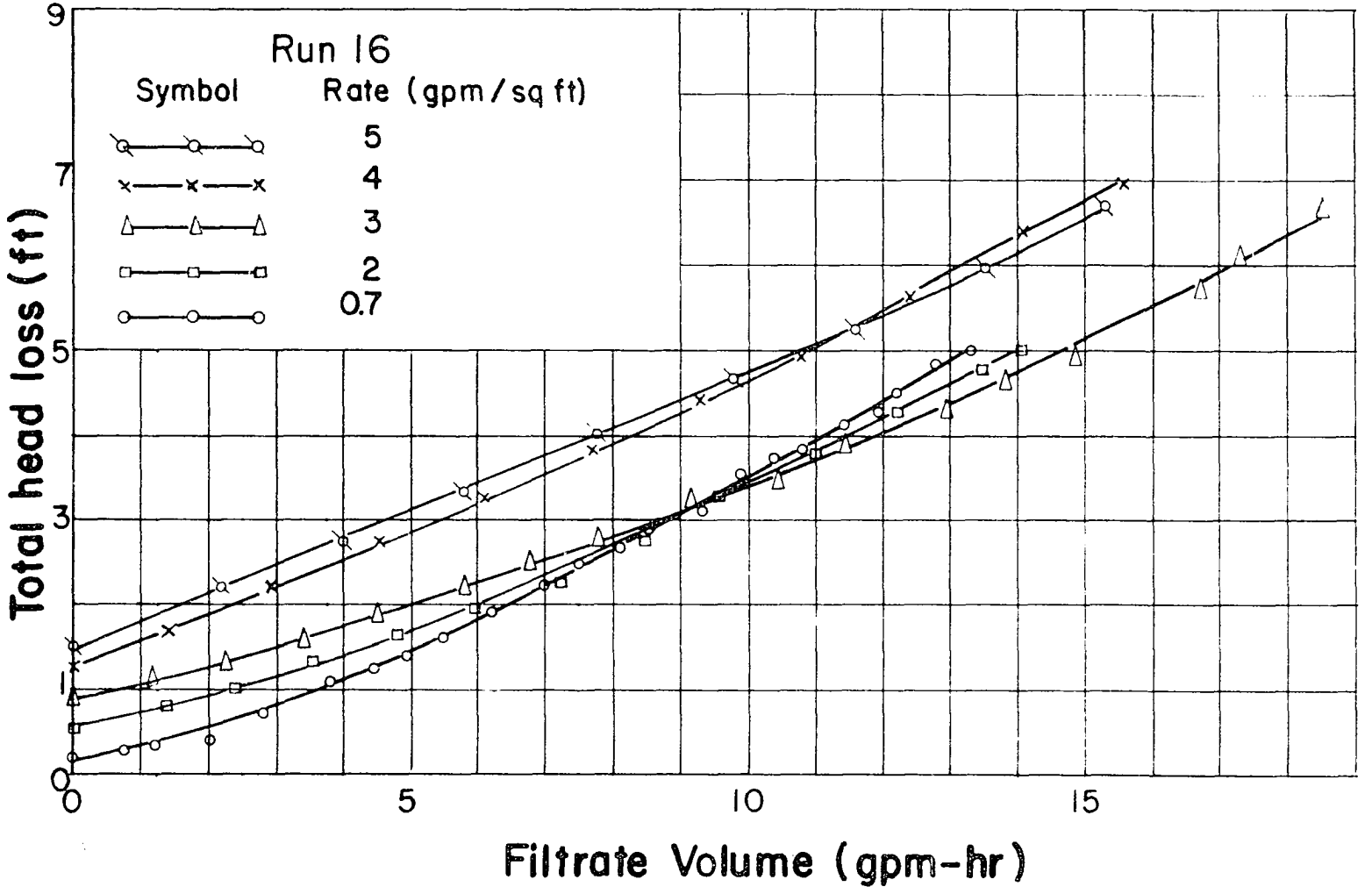

Run 16
Symbol
Rate (gpm / sq ft)
5
4
3
2
0.7
Total head loss (ft)
Filtrate Volume (gpm-hr)
0
1
3
5
7
9
0
5
10
15

Figure 9. Run 22, total head loss vs. filtrate volume

Typical filter run which shows no optimum rate tendency. Filtering water containing precipitated hydrous ferric oxide. Temporary drop in head loss at 2 gpm/sq ft resulted when run was recommenced after 1 day plant shut down.

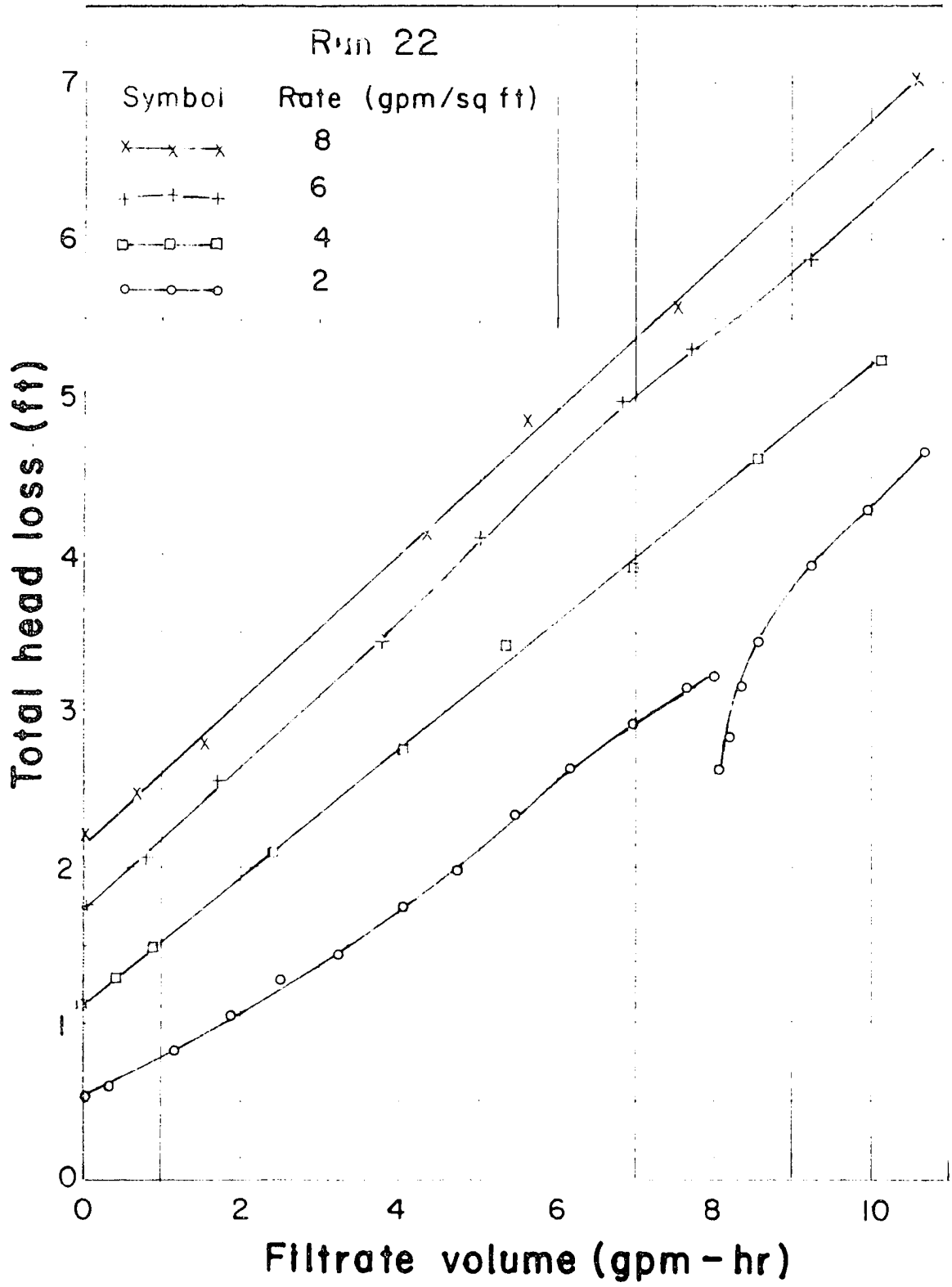
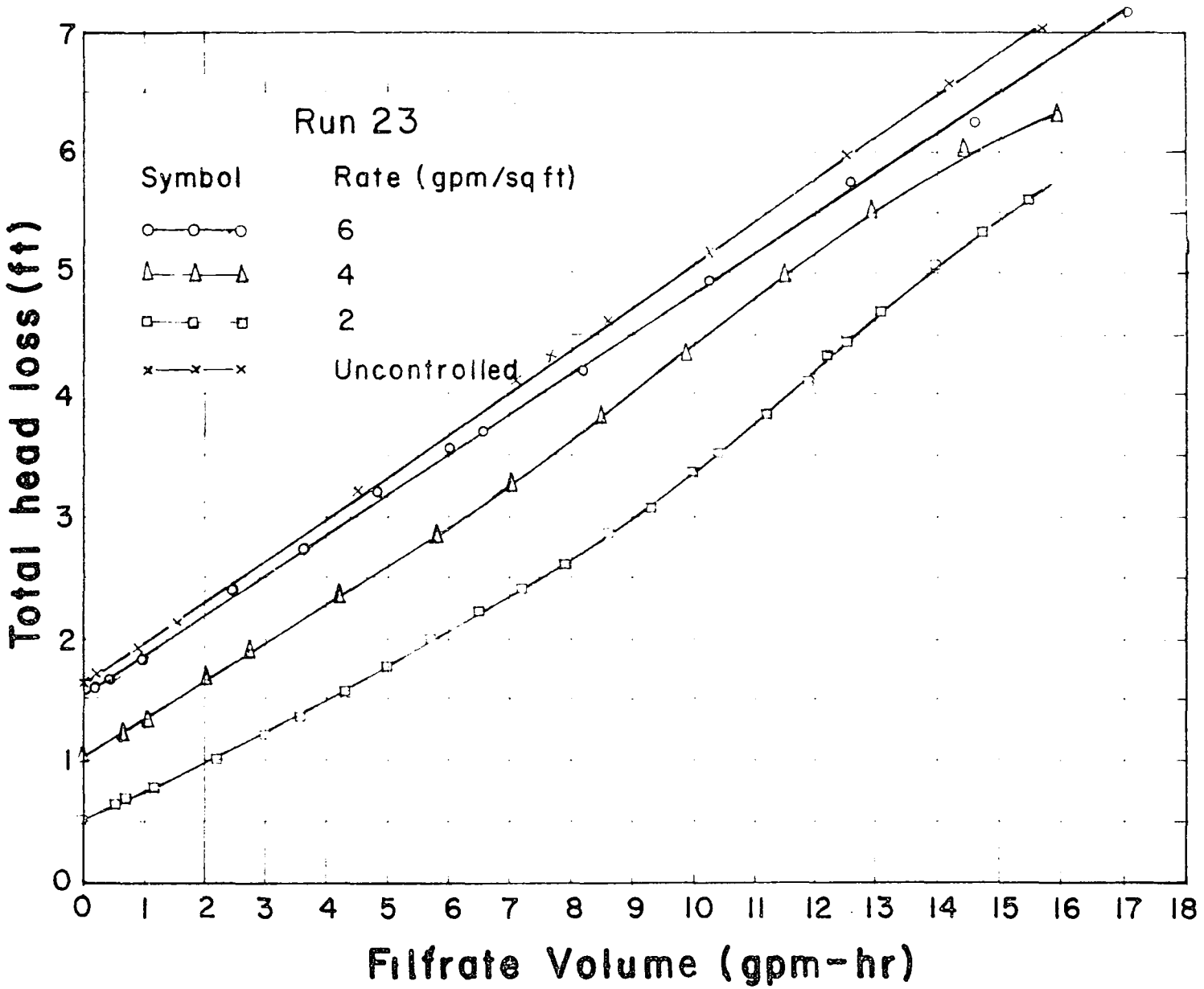

Run 22
Symbol
Rate (gpm/sq ft)
8
6
4
2
Total head loss (ft)
Filtrate volume (gpm-hr)
0
1
2
3
4
5
6
7
0
2
4
6
8
10

Figure 10. Run 23, total head loss vs. filtrate volume

Typical filter run which shows no optimum rate tendency. Filtering water containing precipitated hydrous ferric oxide.

Uncontrolled run started at 6 gpm/sq ft and permitted to decline in rate as the head loss increased.

Curve for 1 gpm/sq ft not shown due to insufficient run length.


Run 23
Symbol
Rate (gpm/sq ft)
6
4
2
Uncontrolled
Total head loss (ft)
Filtrate Volume (gpm-hr)
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
18

bidity or iron content at approximately the theoretical time required to displace the water from the filter and the turbidimeter. This is in agreement with the results of Tso-Ti Ling (53). The effluent water quality then quickly improved to some minimum level of iron content. At the lower rates of filtration, this level was maintained throughout the filter run. At the higher filtration rates, the effluent quality might gradually degrade during the filter run. Figure 11 demonstrates the typical initial improvement in effluent iron content at the various filtration rates in run 23. Figure 12 shows typical effluent iron content as observed in run 23 following the initial improvement period.

From these figures and Table 8, several observations concerning effluent quality can be made. Effluent water quality decreases slightly as filtration rates are increased. However, even at very low rates, some iron passes through the filter. Apparently some portion of the particles are of such size or charge characteristic that they cannot be removed by filtration even at unusually low rates.

The importance of iron passage during the initial improvement period on average water quality increases as the rate increases. However, the effect of the initial improvement period on average water quality is not excessive even at the highest rate of 6 gpm/sq ft.

Even though the full filter bed is apparently active in filtration as evidenced by the presence of effluent iron at all rates, effluent quality does not necessarily degrade as the filter run progresses. Only at 6 gpm/sq ft was gradual effluent degradation apparent in run 23.

Figure 11. Run 23, initial effluent improvement

Effluent iron content vs. the time after commencement of the filter run.

Data for uncontrolled run and 1 gpm/sq ft run not observed.

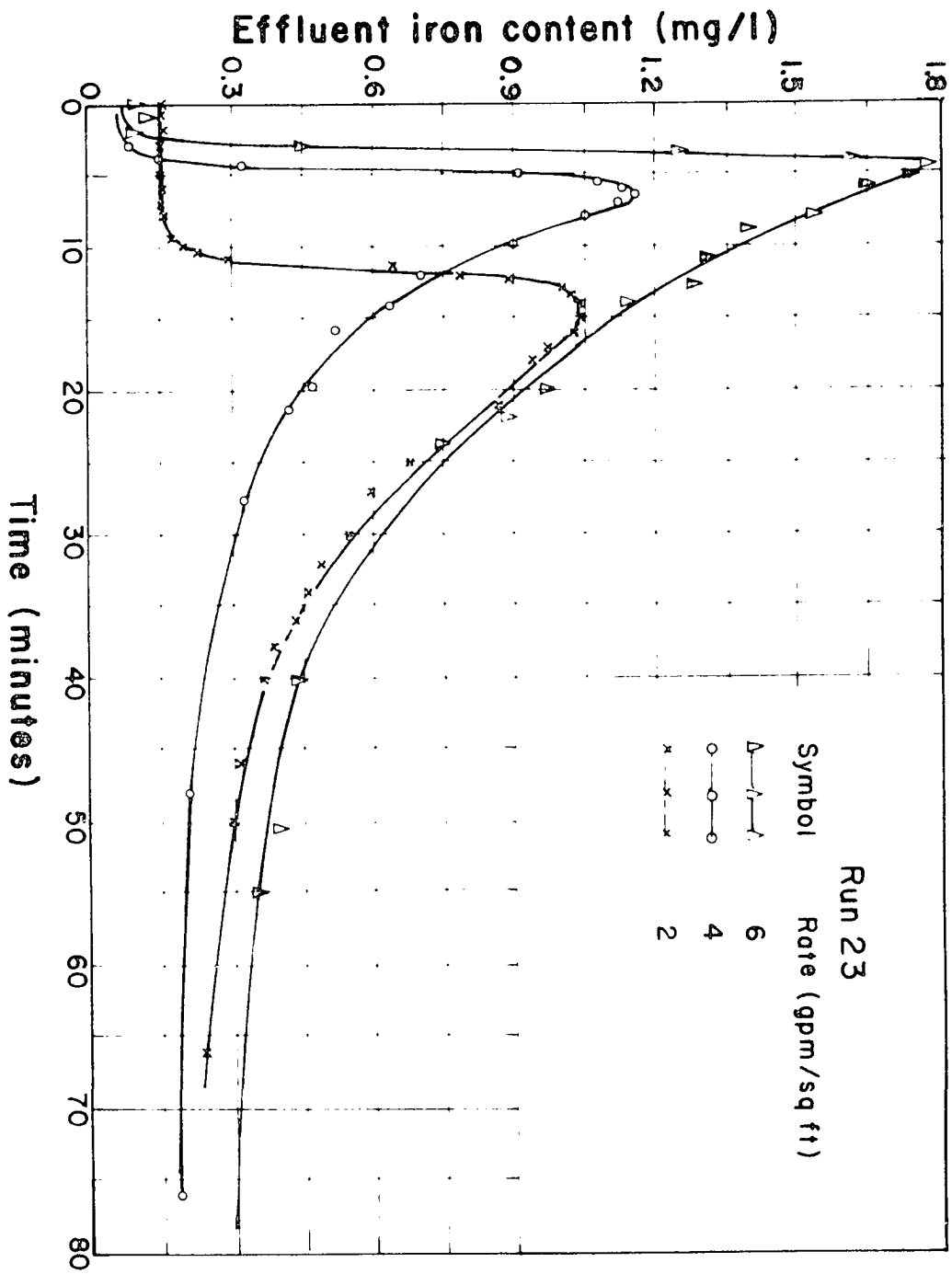

Effluent iron content (mg/l)
0
0.3
0.6
0.9
1.2
1.5
1.8
Time (minutes)
0
10
20
30
40
50
60
70
80
Run 23
Symbol
Rate (gpm/sq ft)
6
4
2

Figure 12. Run 23, effluent iron content vs. filtrate volume

Constant pressure run started at 6 gpm/sq ft and permitted to decline in rate as the head loss increased.

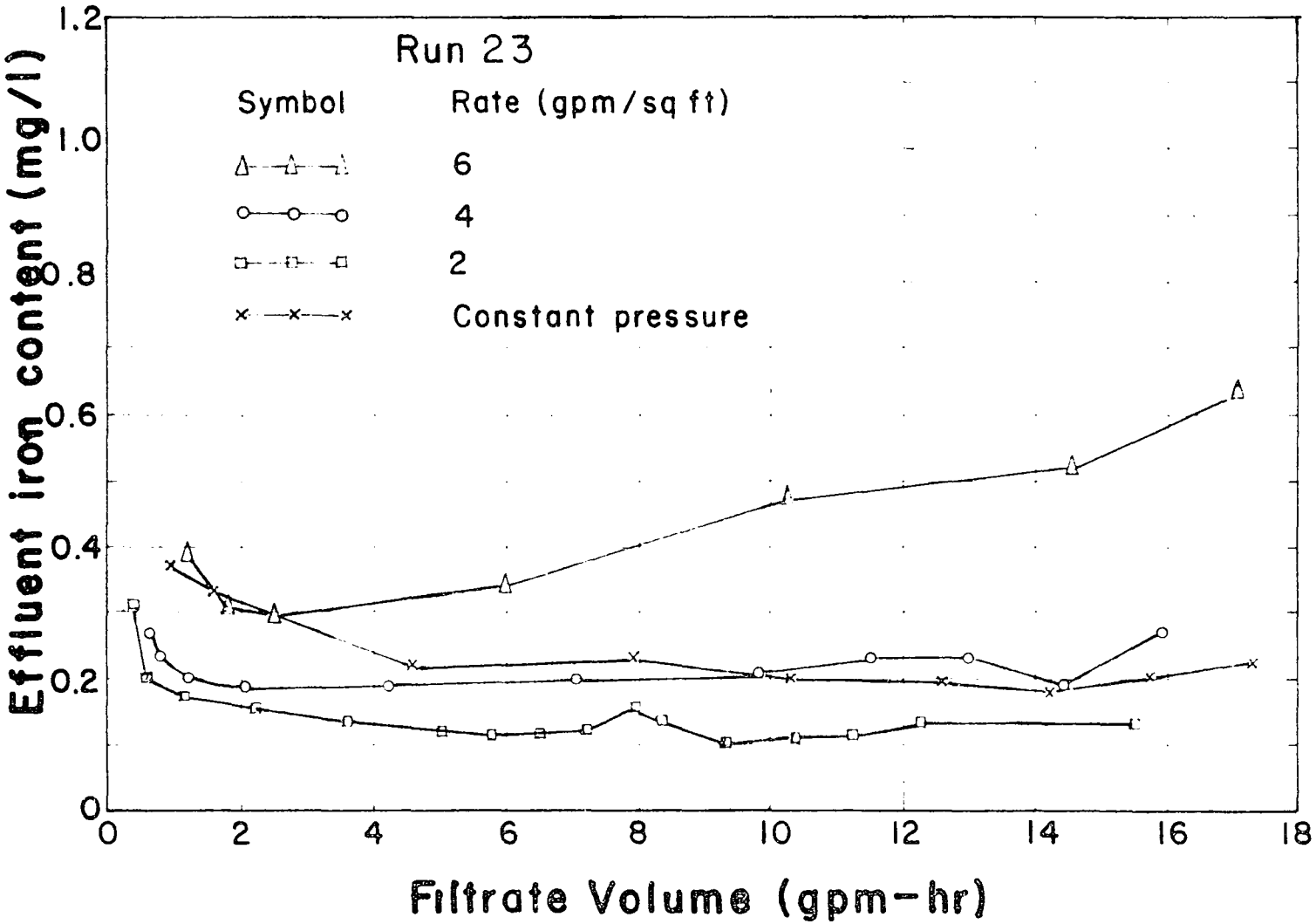

Run 23
Symbol
Rate (gpm/sq ft)
6
4
2
Constant pressure
Effluent iron content (mg/l)
Filtrate Volume (gpm-hr)
0
0.2
0.4
0.6
0.8
1.0
1.2
0
2
4
6
8
10
12
14
16
18

Table 8. Summary of effluent iron content, run 23

Filtration rate (gpm/sq ft)	Minimum effluent iron[a] (mg/l)	Initial iron passed[b] (mg/sq ft)	Average increase effluent iron[c] (mg/l)
1	0.08	25	0.001
2	0.09	97	0.005
4	0.18	123	0.007
6	0.28	581	0.031

[a]Lowest level of iron content at any time during the filter run.

[b]The total mg/sq ft of effluent iron passed above a level of 0.30 mg/l during the initial improvement period.

[c]Average increase in effluent iron due to the iron passage during the initial improvement.

G. Experimental Runs with a Pronounced Optimum Rate

When filtering the filter influent water of the Ames municipal lime-soda ash softening plant, a pronounced tendency for greater production per filter run was apparent as filtration rates were increased. In some filter runs, the production reached an optimum at some medium filtration rate, and decreased at higher filtration rates. Figures 13, 14, 15, and 16 for runs 18, 19, 24, and 25 respectively are typical total head loss vs. volume of filtrate curves for this type of filter influent water.

The average influent water quality during these filter runs is described in Table 9.

Certain general characteristics are apparent in Figures 13, 14, 15, and 16. At the lower rates of filtration, the head loss increases at an

Figure 13. Run 18, total head loss vs. filtrate volume

Filtering Ames filter influent water. Plant operating one well at 1100 gpm.

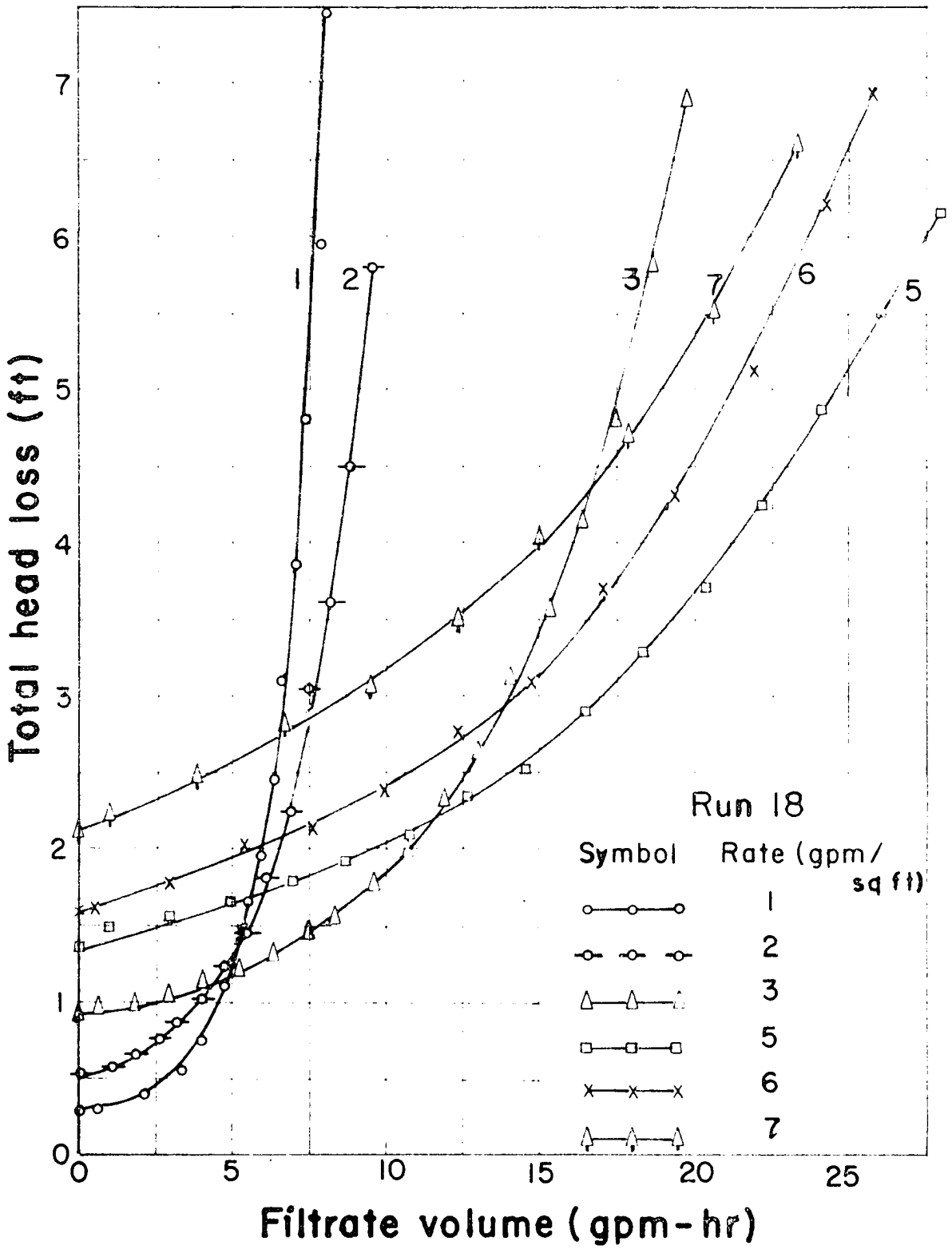

Total head loss (ft)
Filtrate volume (gpm-hr)
0
1
2
3
4
5
6
7
0
5
10
15
20
25
1
2
3
7
6
5
Run 18
Symbol
Rate (gpm/ sq ft)
1
2
3
5
6
7

Figure 14. Run 19, total head loss vs. filtrate volume

Filtering Ames filter influent water. Plant operating two wells at a total of 1400 gpm.

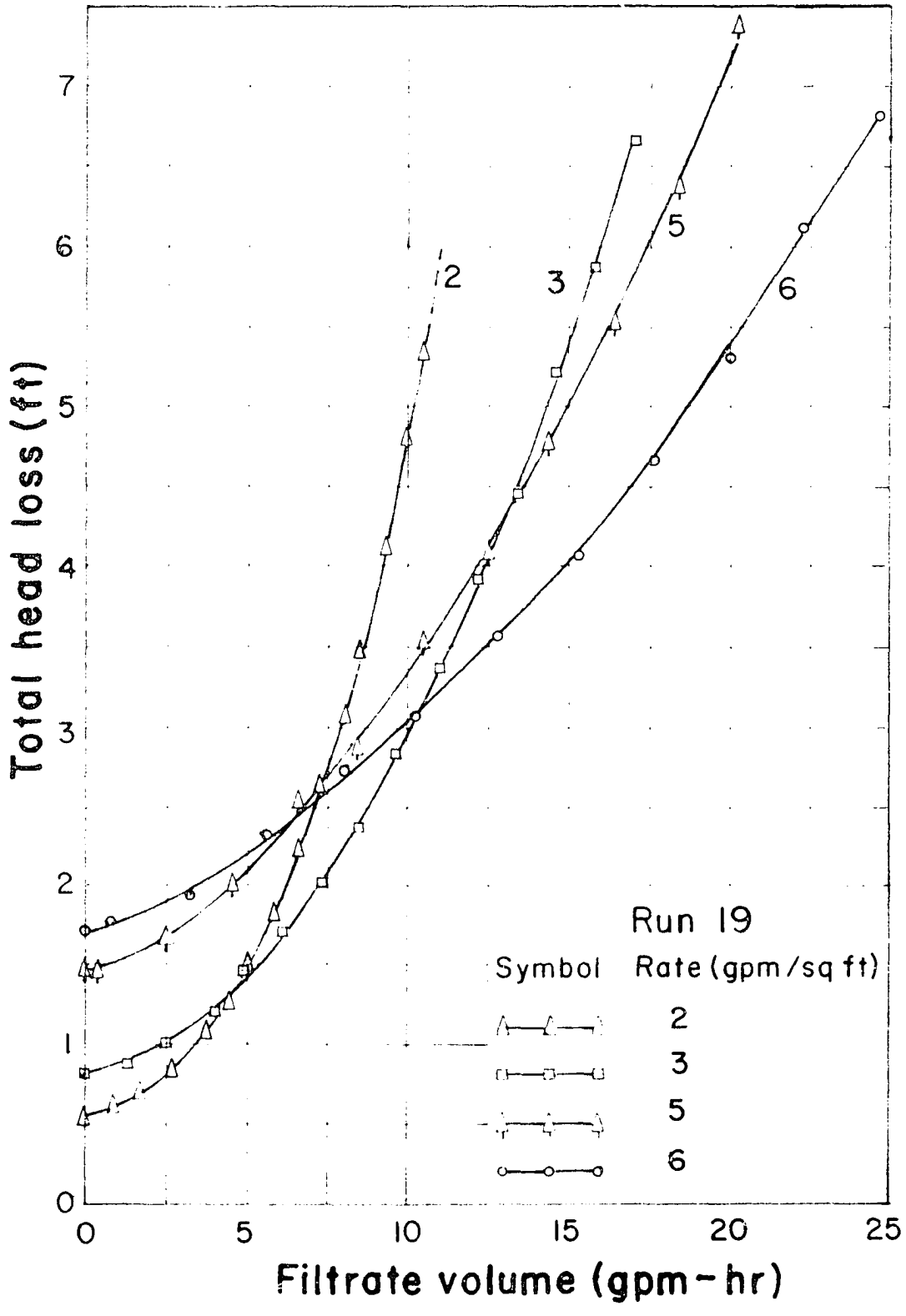

Total head loss (ft)
Filtrate volume (gpm-hr)
0
1
2
3
4
5
6
7
0
5
10
15
20
25
2
3
5
6
Run 19
Symbol
Rate (gpm/sq ft)
2
3
5
6

Figure 15. Run 24, total head loss vs. filtrate volume

Filtering Ames filter influent water diluted with an equal amount of tap water to regulate turbidity and stability. Plant operating with two wells at a total rate of 1650 gpm.

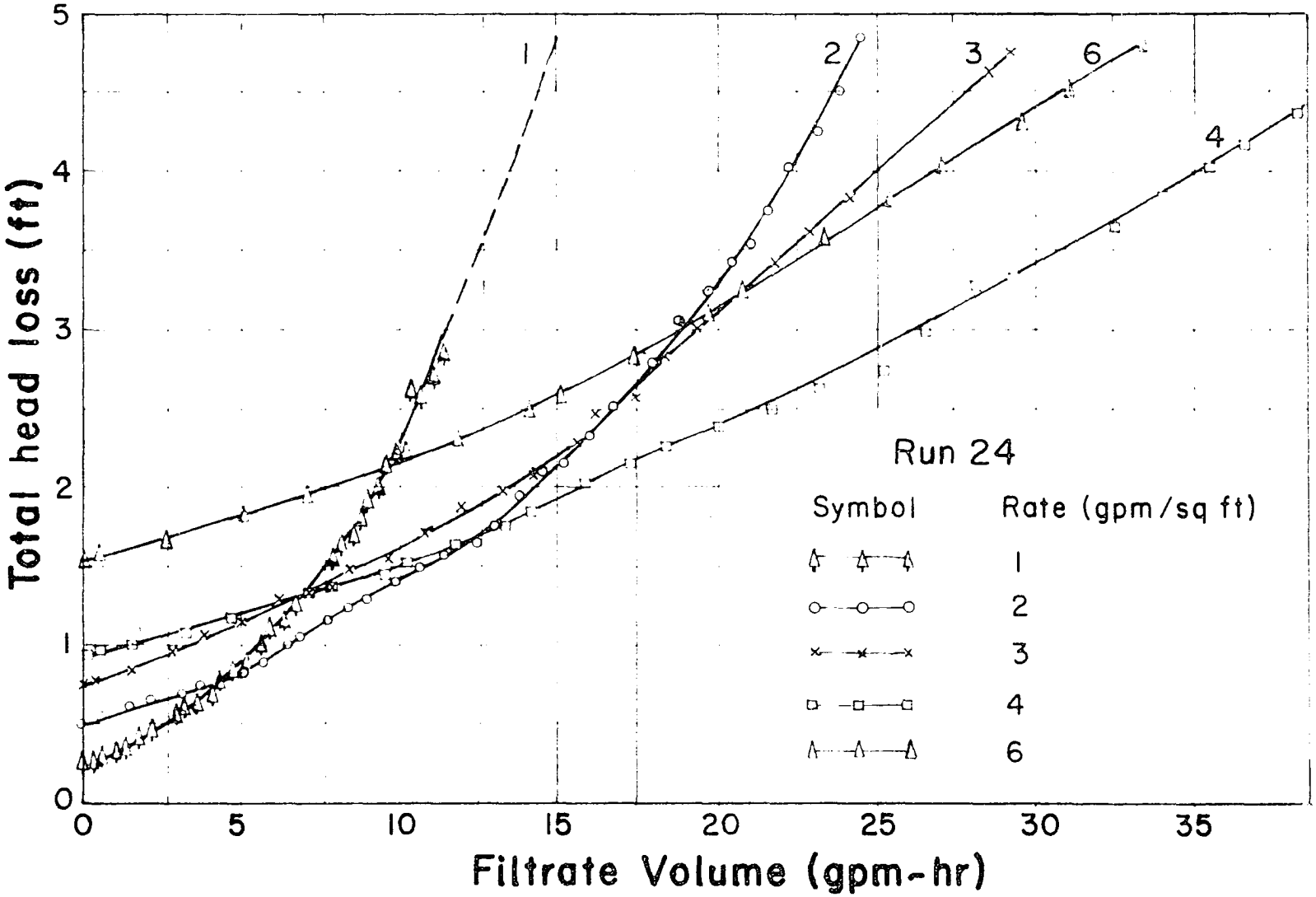

Total head loss (ft)
Filtrate Volume (gpm-hr)
0
1
2
3
4
5
0
5
10
15
20
25
30
35
1
2
3
6
4
Run 24
Symbol
Rate (gpm/sq ft)
1
2
3
4
6

Figure 16. Run 25, total head loss vs. filtrate volume

Filtering Ames filter influent water diluted with tap water to regulate turbidity and stability. Plant operating two wells at a total rate of 1650 gpm.

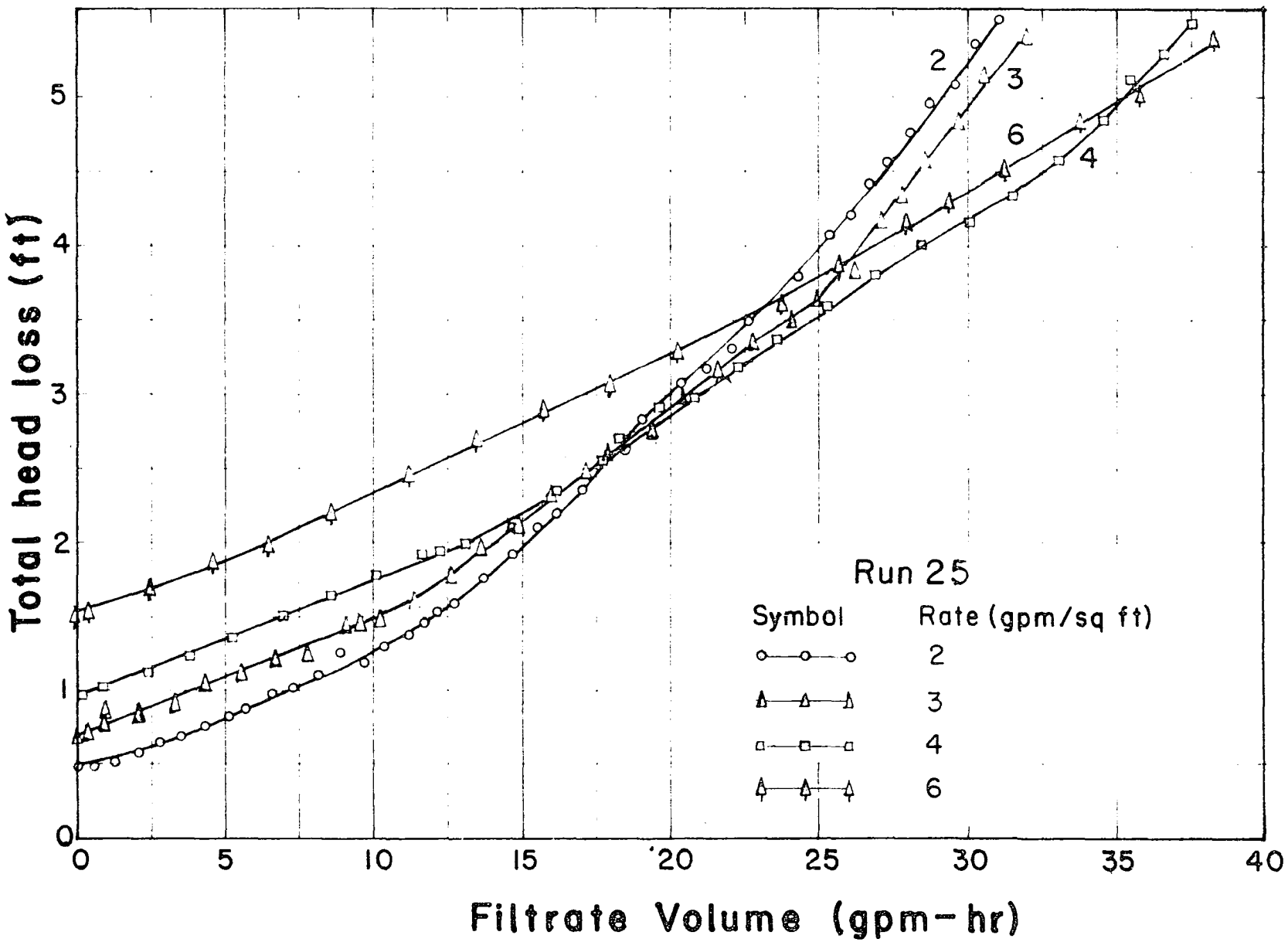

Total head loss (ft)
Filtrate Volume (gpm-hr)
0
1
2
3
4
5
0
5
10
15
20
25
30
35
40
2
3
6
4
Run 25
Symbol
Rate (gpm/sq ft)
2
3
4
6

Table 9. Average filter influent quality in runs 18, 19, 24 and 25

Run no.	Temp. °F	Total hardness[b] mg/l	Alkalinity[b] $(CO_3)^{=}$ mg/l	Alkalinity[b] $(HCO_3)^{-}$ mg/l	pH	Turbidity[c] units	Stability[a] mg/l drop in hardness	Comments
18	56-58	83	28	24	9.2	5	Unstable 9	Plant operating one well at 1100 gpm
19	56-57	82-89	26	25	9.3	9	Unstable 14	Plant operating two wells at 1400 gpm
24	59-63	79-85	22	15	9.2	7	Stable 0	Plant operating two wells at 1650 gpm. Influent diluted with tap water[d]
25	61-63	83-89	20	21	9.2	6	Stable 0	Same as run 24

[a]As indicated by drop in alkalinity and hardness through the dirty filter, mg/l as $CaCO_3$.

[b]Expressed as $CaCO_3$.

[c]As measured with Hellige turbidimeter.

[d]Diluted with tap water to control turbidity and stability.

increasing rate as the run progresses. At higher filtration rates, this tendency is reduced. The curves may approach linearity at high filtration rates as in Figure 15. In Figures 13 and 15, beyond the rate at which the curves most nearly become linear, higher rates resulted in head loss curves which were nearly parallel. The optimum rate, where maximum production to a given head loss will occur, is the lowest rate where the head loss development approaches linearity.

Effluent turbidity, Table 10, behaved in a manner similar to the effluent iron content described in the previous section. At the beginning of a filter run, there was a brief period of high turbidity, the peak occuring at approximately the theoretical displacement time of the filter and turbidimeter. The turbidity then dropped quickly to some minimum value. At the lower rates, this value remained nearly constant during the filter run. At the medium and higher rates of filtration, the effluent turbidity increased gradually as the run progressed.

Effluent turbidity was not always satisfactory at the higher filtration rates. Therefore, the optimum rate from the standpoint of production may not be a feasible rate due to unacceptable water quality. Table 11 is a comparison of relative production to a 5 ft terminal total head loss (including initial clean bed head loss) in runs 18, 19, 24, and 25.

Study of Tables 10 and 11 permits the following observations. When filtering a fairly high quality influent water as is produced with one well operating at the Ames municipal plant (run 18), the optimum filtration rate lies between 3 and 5 gpm/sq ft. Expected production per run will be over twice the production at the standard rate of 2 gpm/sq ft. Rates in excess of 5 gpm/sq ft result in reduced production and water

Table 10. Effluent water turbidity in runs 18, 19, 24 and 25

Run no.		Filtration rate (gpm/sq ft)						
		1	2	3	4	5	6	7
18[a]	Initial turbidity[b]	0.2	0.3	0.3		0.5	0.3	0.6
	Final turbidity[c]	0.2	0.3	0.3		0.7	0.3	0.6
19[a]	Initial turbidity		0.4	0.4		0.7	0.5	
	Final turbidity		0.5	0.7		0.9	1.7	
24	Initial turbidity	0.14	0.16	0.16	0.36		0.50	
	Final turbidity	0.14	0.21	0.34	1.16		1.20	
25	Initial turbidity		0.13	0.16	0.22		0.37	
	Final turbidity		0.32	0.46	0.70		1.04	

[a]Turbidimeters were not completely calibrated until after runs 18 and 19. Therefore, these turbidity values are less reliable than turbidity values in later runs.

[b]After initial improvement period.

[c]At a terminal head loss of 5 to 6 ft.

quality.

Run 24 and 25 are similar in some respects to run 18. In these runs a stable water of moderate turbidity was filtered. The optimum rate from a production standpoint was 4 gpm/sq ft, where 152 and 121 percent of standard rate production was obtained, respectively, in run 24 and 25. The terminal turbidity at the optimum rate was above the acceptable level of 0.5 unit, therefore, the desireable operating rate would be between 3 and 4 gpm/sq ft.

When filtering a more unstable, and more highly turbid influent water,

Table 11. Relative water production[a] in runs 18, 19, 24 and 25

Run no.	Filtration rate (gpm/sq ft)						
	1	2	3	4	5	6	7
18	0.80	1.00	1.93		2.65	2.32	2.07
19		1.00	1.42		1.47	1.84	
24	0.60[b]	1.00	1.23[b]	1.52		1.40	
25		1.00	1.03	1.21		1.21	

[a]To a terminal total head loss of 5 ft, with the production at 2 gpm/sq ft considered unity.

[b]Estimated by extrapolation.

as is produced when the Ames municipal plant operates more than one well, the optimum rate had not been reached even at 6 gpm/sq ft. Production increased steadily as the filtration rate increased. However, terminal water quality was not acceptable, even at the 3 gpm/sq ft rate. Therefore, with this type of water, the filtration rate should be governed by the acceptable water quality and established at between 2 and 3 gpm/sq ft.

It is evident that the optimum tendency is more pronounced in runs 18 and 19, in which an unstable water is being filtered, than in runs 24 and 25 in which the influent water is stable. The instability of the water results in precipitation of calcium carbonate in the filter bed. The unstable calcium carbonate is catalyzed by the calcium carbonate already removed in the filter. The amount of after-precipitation is practically zero on the clean sand at the beginning of a filter run, but soon reaches an equilibrium value which is then constant throughout the filter run.

After-precipitation in the bed is a significant load on the filter as evidenced by the lower production in runs 18 and 19 (Figures 13 and 14) than in runs 24 and 25 (Figures 15 and 16).

VI. REASONS FOR AN OPTIMUM RATE TENDENCY

A. General

The foregoing sections indicate that the two types of water being filtered contained particles which were distinctly different from the standpoint of head loss development. The water which contained hydrous ferric oxide particles caused nearly linear head loss development at all rates of filtration. The filter influent water of the Ames municipal treatment plant, which contained principally calcium carbonate particles, caused head loss to go up at an increasing rate as the run progressed. This tendency was most pronounced at lower rates.

With a typical linear head loss development, increased filtration rates result in reduced water production to a given terminal head loss. With a typical increasing rate of head loss development, higher rates result in greater water production to a given terminal head loss. In the latter case, an optimum rate may be reached as the head loss development curve approaches linearity and further rate increases will result in reduced production.

What are the basic reasons behind these two characteristic manners of head loss development? This chapter will try to answer this question.

B. Hydraulic Conditions Existing in a Dirty Filter

To evaluate the reasons which may cause an optimum rate tendency, we must first study the hydraulic conditions which exist in a filter bed as it becomes filled with sediment.

As previously mentioned, in Chapter II (p. 20), the observation of

laminar flow in a clean sand filter has been made by a number of investigators. This observation has also been duplicated in this research at all rates of filtration studied. The characteristic of flow through a dirty filter, however, has not been reported.

Hudson (36) has suggested that the excessive passage of material through the filter may be associated with the development of turbulent flow, and he developed a "filtrability index" for turbulent flow conditions (eq. 6, p. 26). He did not present valid experimental proof of the existance of turbulent flow.

According to Darcy's law for laminar flow through porous media (eq. 1, p. 21), head loss is directly proportional to the rate of flow. Thus, the observation of this proportionality in a dirty filter would support the presence of laminar flow conditions. On several occasions, the change in head loss with flow rate was observed at the end of a filter run when a filter was quite dirty. The head loss at various depths was observed as the rate of filtration was progressively reduced from the rate during the run to lower rates. Figures 17, 18 and 19 are examples of the typical curves for these observations. These figures show almost a linear relation of head loss to filtration rate for the full bed and for the upper layers where turbulence might be expected to develop due to the high degree of clogging. Figure 17 represents a typical run on water containing precipitated hydrous ferric oxide. Figures 18 and 19 represent a typical run on Ames filter influent water.

The curves of Figures 17 and 18 are slightly concave downward. Presence of turbulence would have had the opposite effect, since in turbulent flow, the head loss is proportional to approximately the second power of

Figure 17. Run 23, demonstration of laminar flow in a dirty filter

Following a run at 4 gpm/sq ft, filtering water containing hydrous ferric oxide.

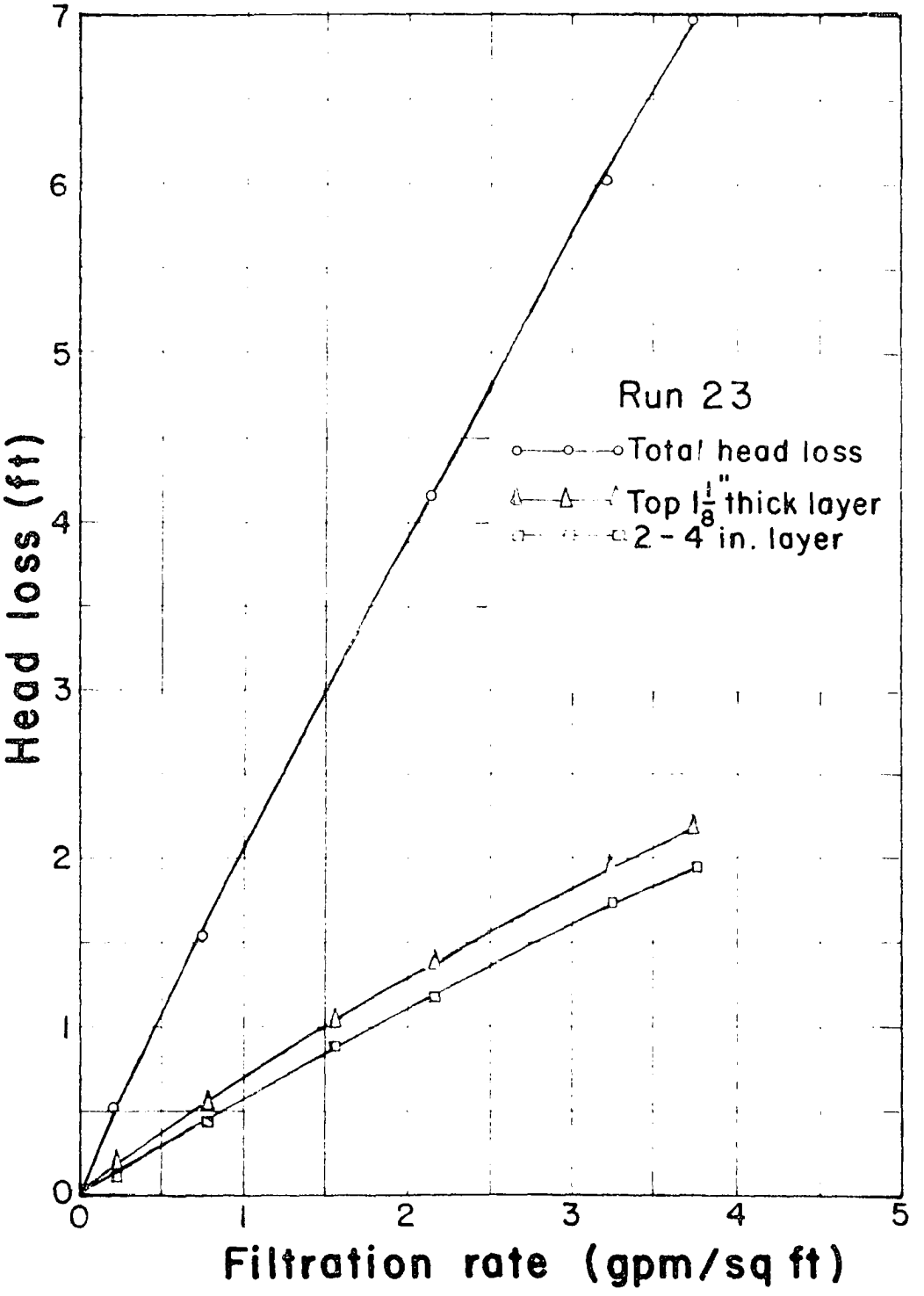

Run 23
Total head loss
Top 1 1/8" thick layer
2 - 4 in. layer
Head loss (ft)
Filtration rate (gpm/sq ft)
0
1
2
3
4
5
6
7

Figure 18. Run 24, demonstration of laminar flow in a dirty filter

Following a run at 4 gpm/sq ft, filtering Ames filter influent water.

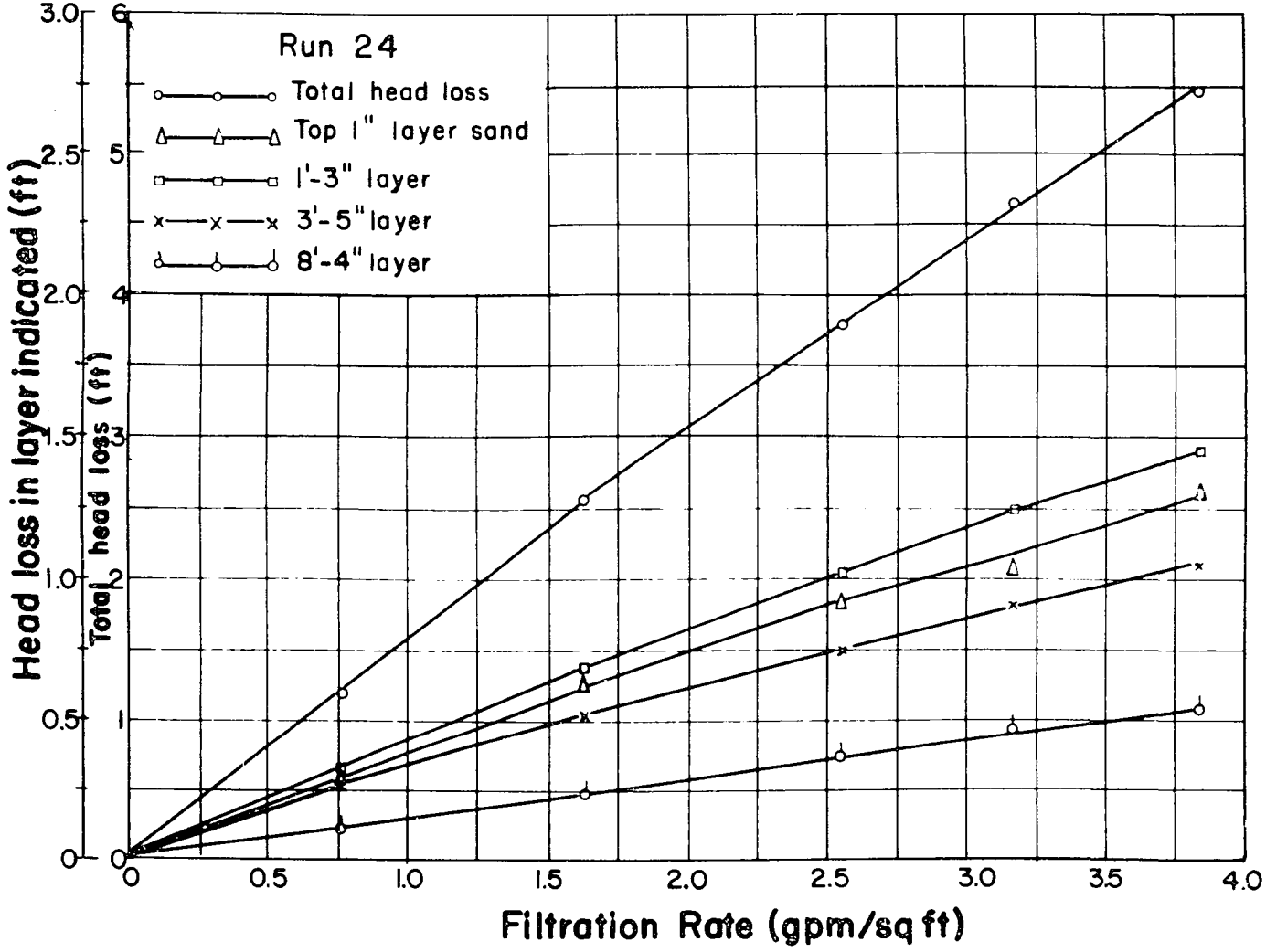
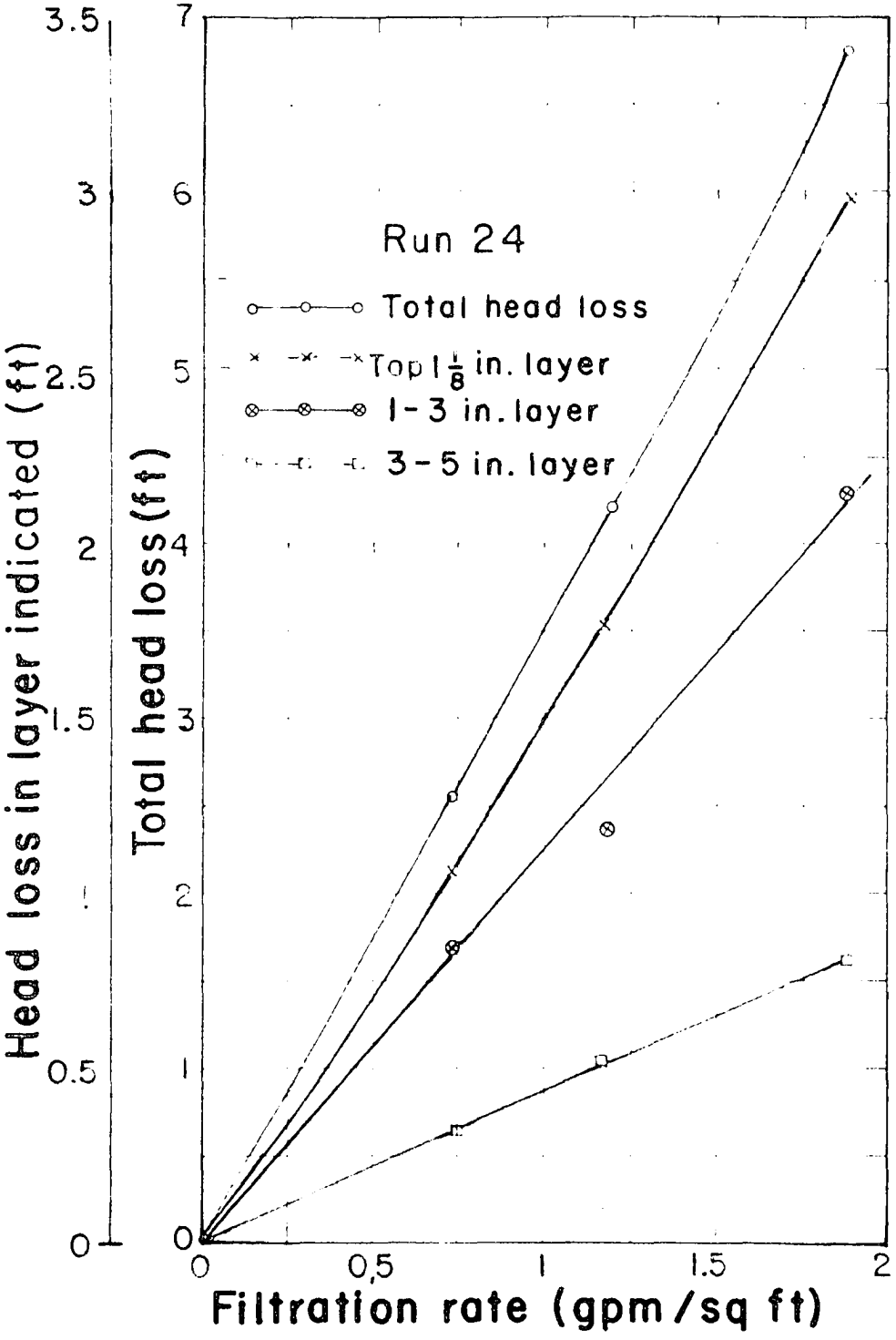

Run 24
Total head loss
Top 1" layer sand
1'-3" layer
3'-5" layer
8'-4" layer
Head loss in layer indicated (ft)
Total head loss (ft)
Filtration Rate (gpm/sq ft)
0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
1
2
3
4
5
6

Figure 19. Run 24, demonstration of laminar flow in a dirty filter

Following a run at 2 gpm/sq ft, filtering Ames filter influent water.


Run 24
Total head loss
Top 1 1/8 in. layer
1-3 in. layer
3-5 in. layer
Head loss in layer indicated (ft)
Total head loss (ft)
Filtration rate (gpm/sq ft)
0
0.5
1
1.5
2
2.5
3
3.5
0
1
2
3
4
5
6
7
0
0,5
1
1.5
2

the flow rate as shown in the following general equation:

$$H_f = r\, Q^n \qquad \text{Eq 7}$$

Where:

H_f = Head loss due to friction
r = Resistance coefficient
Q = Flow rate, volume per unit time
n = Exponent, approximately 2 according to most turbulent flow equations.

The unexpected concave downward tendency observed in Figures 17 and 18 must, therefore, be due to some increase in permeability as rates are decreased. The permeability increase is due to reduction of the compressive forces exerted by the hydraulic gradient which are reduced as the flow rate is decreased. The curvature is most pronounced in the upper layers and absent in the lower layers, supporting this hypothesis.

Figure 19 for run 24 shows nearly a linear relation between head loss and rate of filtration. These data were collected following a run on Ames filter influent water at 2 gpm/sq ft. The curves for head loss through the full bed and the top 1-1/8 in. layer are slightly concave upward. Turbulence would be most apt to develop in a run of this type at low rates with a strong tendency towards surface removal. If turbulence was present, it should be present in the top layers. The curve for the top layer is far from the exponential type curve experienced with turbulent flow.

It can be concluded that laminar flow conditions existed throughout all filter runs, in all layers of the filter, for all waters and for all filtration rates covered by this study.

C. Utilization of Filter Voids at Various Rates

Head loss observations were made on all piezometer connections at two hour intervals during each filter run. The following observations are the result of study of these head loss data.

At the beginning of a filter run, most of the sediment is removed in the upper layers of the filter. As the run progresses, the upper layers get filled with sediment and the burden of removal is carried to progressively deeper layers of the filter. This observations is particularly true at low to standard filtration rates and supports similar observations made by Eliassen (23).

A larger segment of the filter depth plays a significant role in sediment removal at high filtration rates than at low filtration rates. This results in better utilization of the sand bed and may be one reason for the presence of an optimum rate tendency. It view of this possibility, a study was made of the relative utilization of the void spaces in the sand at different rates and with different influent waters.

The existance of laminar flow conditions throughout the filter runs makes it possible to calculate the relative utilization of the voids within the sand. By Darcy's law for laminar flow:

$$v = KS \qquad \text{Eq 1 (p. 21)}$$

Where:

v = Velocity of flow
S = Hydraulic gradient ratio
K = Constant, coefficient of permeability.

And since:

$$Q = Av \qquad \text{Eq 8}$$

Where:

Q = Volume of flow per unit time
A = Area of flow
v = Velocity of flow.

Combining equations 1 and 8:

$$\frac{Q}{A} = KS \qquad \text{Eq 9}$$

The area available for flow per unit area of the filter at any instant, would be proportional to the porosity at that instant.

$$A = A'p \qquad \text{Eq 10}$$

Where:

p is porosity of the sand at any time.
A' is horizontal area of the filter.

Combining equations 9 and 10:

$$S = \frac{Q}{KA'p} \qquad \text{Eq 11}$$

Equation 11 indicates that the hydraulic gradient is inversely proportional to the porosity. If S_i and p_i designate initial values and S_n and p_n designate values at any time "n", then for constant rate filtration.

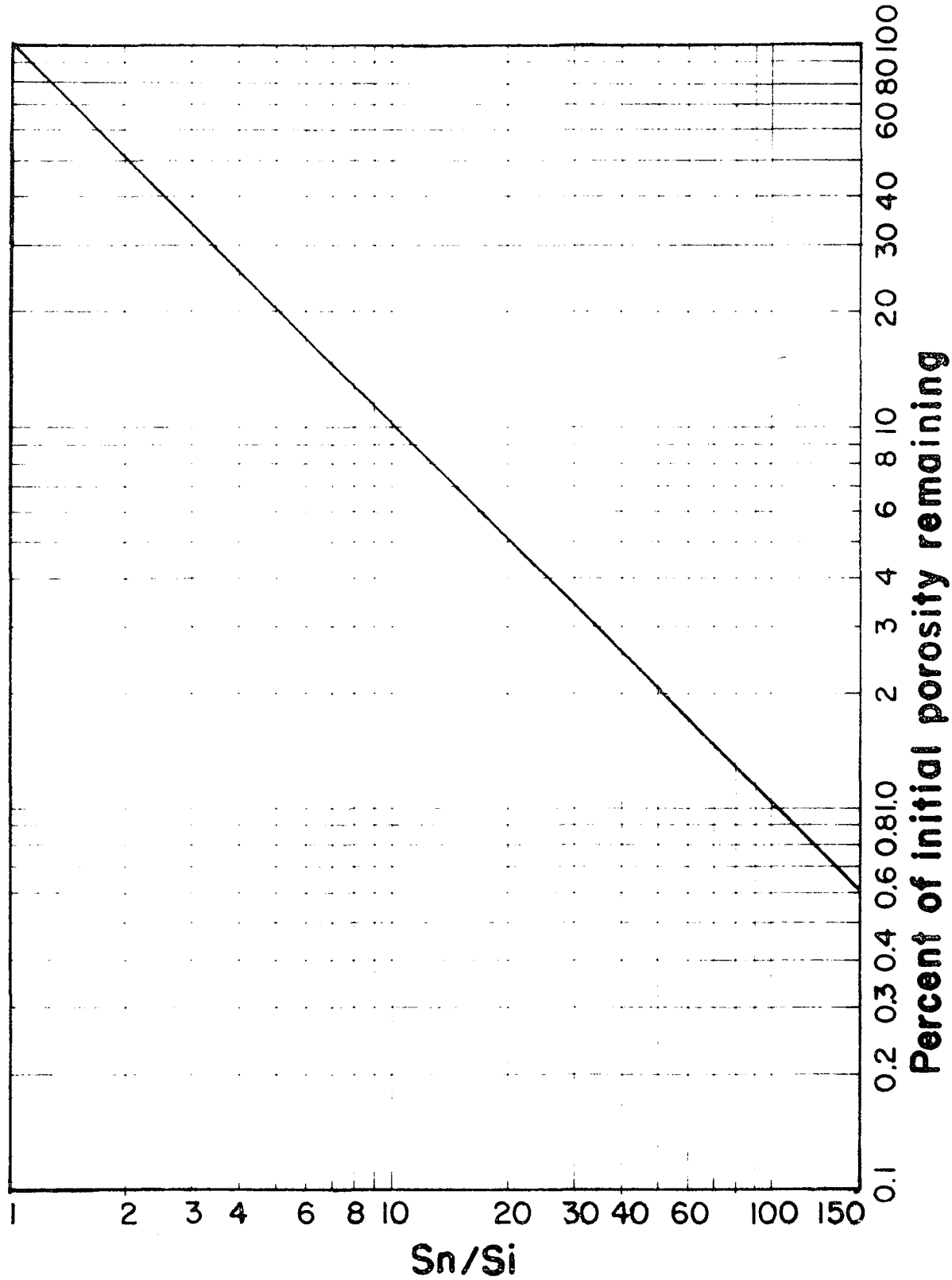
$$\frac{S_n}{S_i} = \frac{p_i}{p_n} \qquad \text{Eq 12}$$

Equation 12 is illustrated graphically in Figure 20.

It is evident from Figure 20 that slight increases in head loss represent utilization of relatively large amounts of storage within the filter. As the head loss or hydraulic gradient ratio increases, the corresponding returns in the form of material stored diminish rapidly. For example, a fifty fold increase in head loss occuring only in the top 2 in. layer

Figure 20. Relation of hydraulic gradient to porosity

S_n/S_i = ratio of head loss at any time "n" to the initial head loss.


Percent of initial porosity remaining
0.1
0.2
0.3
0.4
0.6
0.8
1.0
2
3
4
6
8
10
20
30
40
60
80
100
Sn/Si
1
2
3
4
6
8
10
20
30
40
60
100
150

would represent 98 percent utilization of the voids in that layer. On the other hand, the same total increase in head loss distributed over a 20 in. layer of uniform sand would represent 80 percent utilization of the voids in the 20 in. layer. The tremendous advantage of increased penetration is therefore evident.

Calculation of the void utilization in runs 16 and 18 illustrates the difference between a run with no optimum tendency and a run with a pronounced optimum tendency. Table 12a gives the percent utilization of voids to a 5 ft head loss. It is also possible to determine the depth of stored material in each layer in a similar manner. If an initial porosity of 40 percent is assumed, which is a fairly typical value for sand, a 1 in. layer of sand would have an equivalent void layer of 0.4 in. If these voids are 75 percent utilized, the depth of stored material would be 0.3 in. Table 12 b gives depths of stored material calculated in this manner for runs 16 and 18 for the void utilizations of table 12a.

Study of table 12b reveals a striking difference in the results from the two filter runs. Run 18, which had a strong optimum rate at 5 gpm/sq ft, shows a similar optimum in the amount of material stored within the bed. The predominance of surface removal is evident at the lower rates. Filter bed utilization increases as the rate increases, and it is apparent that the full bed contributed significantly to the storage of material at 5, 6, and 7 gpm/sq ft. The decreased storage obtained at 6 and 7 gpm/sq ft is the result of reduced time of operation due to higher initial head losses.

Run 16, in which no strong optimum rate was apparent, shows a similar lack of optimum in the amount of material stored. Some increased utiliza-

Table 12a. Percent void utilization in a dirty filter bed[a]

Rate gpm/sq ft	Filter lamina (in. from surface)						
	0-2	2-4	4-6	6-9	9-15	15-21	21-27
Run No. 16 (No optimum rate evident)							
0.7	99.5	98	88	75	38	0	0
2	97	96	92	83	38	0	0
3	95	90	86	75	50	0	0
4	90	86	77	64	41	17	0
5	89	84	71	67	40	17	0
6	88	80	52	82	47	22	8
Run No. 18 (Strong optimum rate evident)							
1	99.1	38	0	0	0	0	0
2	98	50	17	17	0	0	0
3	96	75	31	20	0	0	0
5	93	71	50	22	17	5	9
6	92	75	50	23	14	5	5
7	86	55	33	18	8	7	5

[a]At a terminal head loss of 5 ft.

Table 12b. Depth of stored material in a dirty filter bed (inches)[a]

Rate gpm/sq ft	Filter lamina (in. from surface)							Total	Water produced (gpm-hr)
	0-2	2-4	4-6	6-9	9-15	15-21	21-27		
Run No. 16 (No optimum rate evident)									
0.7	.79	.78	.70	.90	.91	0	0	4.08	13.2
2	.77	.76	.74	1.01	.91	0	0	4.19	14.0
3	.75	.72	.69	.90	1.20	0	0	4.26	15.0
4	.72	.69	.62	.78	.96	.41	0	4.18	11.0
5	.72	.67	.49	.80	.96	.41	0	4.05	10.8
6	.70	.64	.42	1.00	1.17	.53	.19	4.65	11.9
Run No. 18 (Strong optimum rate evident)									
1	.79	.30	0	0	0	0	0	1.09	7.5
2	.78	.40	.13	.20	0	0	0	1.51	9.1
3	.76	.60	.25	.24	0	0	0	1.85	17.7
5	.74	.57	.40	.26	.43	.12	.24	2.76	24.5
6	.73	.60	.40	.26	.33	.12	.12	2.56	21.5
7	.69	.43	.27	.23	.19	.17	.12	2.10	18.7

[a]At a terminal head loss of 5 ft, assuming 40 percent initial porosity.

tion of the lower portion of the bed is evident at higher rates. However, compared to run 18, this run without an optimum rate tendency is characterized by more uniform utilization of the bed at all rates. It is interesting to note the much greater storage at all rates than in run 18.

The optimum tendency in run 18 can be attributed, therefore, to a predominant surface removal with lack of penetration at the lower rates. This lack of penetration results in poor utilization of the filter bed and low production. In the runs without an optimum rate tendency, considerable penetration occurs at all rates, resulting in good utilization of the bed and good production even at low rates.

D. The Role of Surface Removal with an Optimum Rate

1. General

As suggested in the previous section, an optimum rate tendency is the result of strong surface removal and the lack of penetration of suspended matter at low filtration rates. However, increased penetration was observed at higher rates regardless of the presence or absence of an optimum rate tendency. Therefore, deeper penetration cannot be the only factor leading to the two characteristically different types of head loss development. Some other important factor must be responsible for the increasing rate of head loss development associated with the presence of an optimum rate tendency.

2. Comparison of surface and subsurface head loss development

A study of the head loss development in the top layer of sand and in the remaining depth of the filter bed permits some pertinent conclusions. In the case of water containing hydrous ferric oxide particles, the head

loss in the top 1 in. layer developed in nearly a linear manner as did the head loss in the entire bed. Figure 21 is a typical plot of the head loss in the top 1 in. layer for run 23. Figure 10 shows the typical linear head loss development for the entire filter bed for the same run.

On the other hand, when filtering Ames filter influent water, the loss in the top layer was almost entirely responsible for the typical increasing rate of head loss development. This is illustrated for run 24 by the head loss development curves for the top 1 in. of the bed shown in Figure 22 and for the remainder of the filter bed shown in Figure 23. It can be observed from Figure 23 that the head loss exclusive of the surface layer develops in much the same manner as with the water containing hydrous ferric oxide precipitate. The curves at different rates are nearly linear and parallel. The slight upward curvature evident in Figure 23 is probably due to increased load received by this portion of the filter bed as the surface layer gradually becomes so dirty that it removes a smaller and smaller portion of the applied load. Figure 22, for the top 1 in. layer, shows the typical increasing rate of head loss development at lower filtration rates, with a diminishing of this tendency as rates are increased.

It is apparent from these typical figures, that the surface layer is responsible for the different types of head loss development which result in the presence or absence of an optimum rate tendency.

3. Surface filtration observations by other investigators

Chemical engineers have observed typical exponential head loss development when filtering various types of compressible precipitates on cloth filters. Such filtration might be described as cake filtration since the deposited precipitate, or cake, acts as the filtering media ex-

Figure 21. Run 23, head loss in the top 1 in. layer vs. filtrate volume

Filtering water containing precipitated hydrous ferric oxide particles.

Curves for uncontrolled run and 1 gpm/sq ft run not shown.

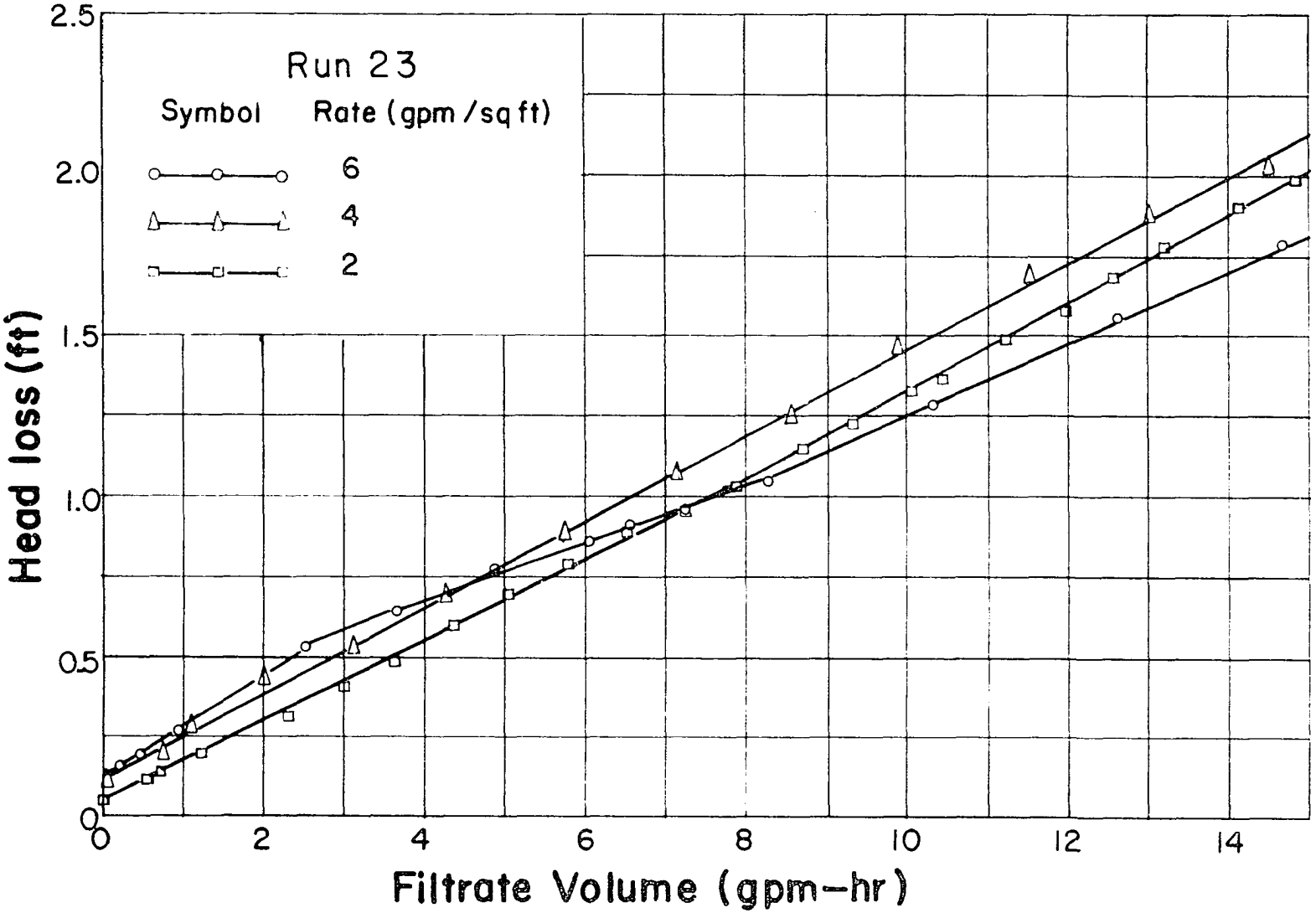

Run 23
Symbol
Rate (gpm / sq ft)
6
4
2
Head loss (ft)
Filtrate Volume (gpm–hr)
0
0.5
1.0
1.5
2.0
2.5
0
2
4
6
8
10
12
14

Figure 22. Run 24, head loss in the top 1 in. layer vs. filtrate volume

Filtering Ames filter influent water diluted with tap water to control stability and turbidity.

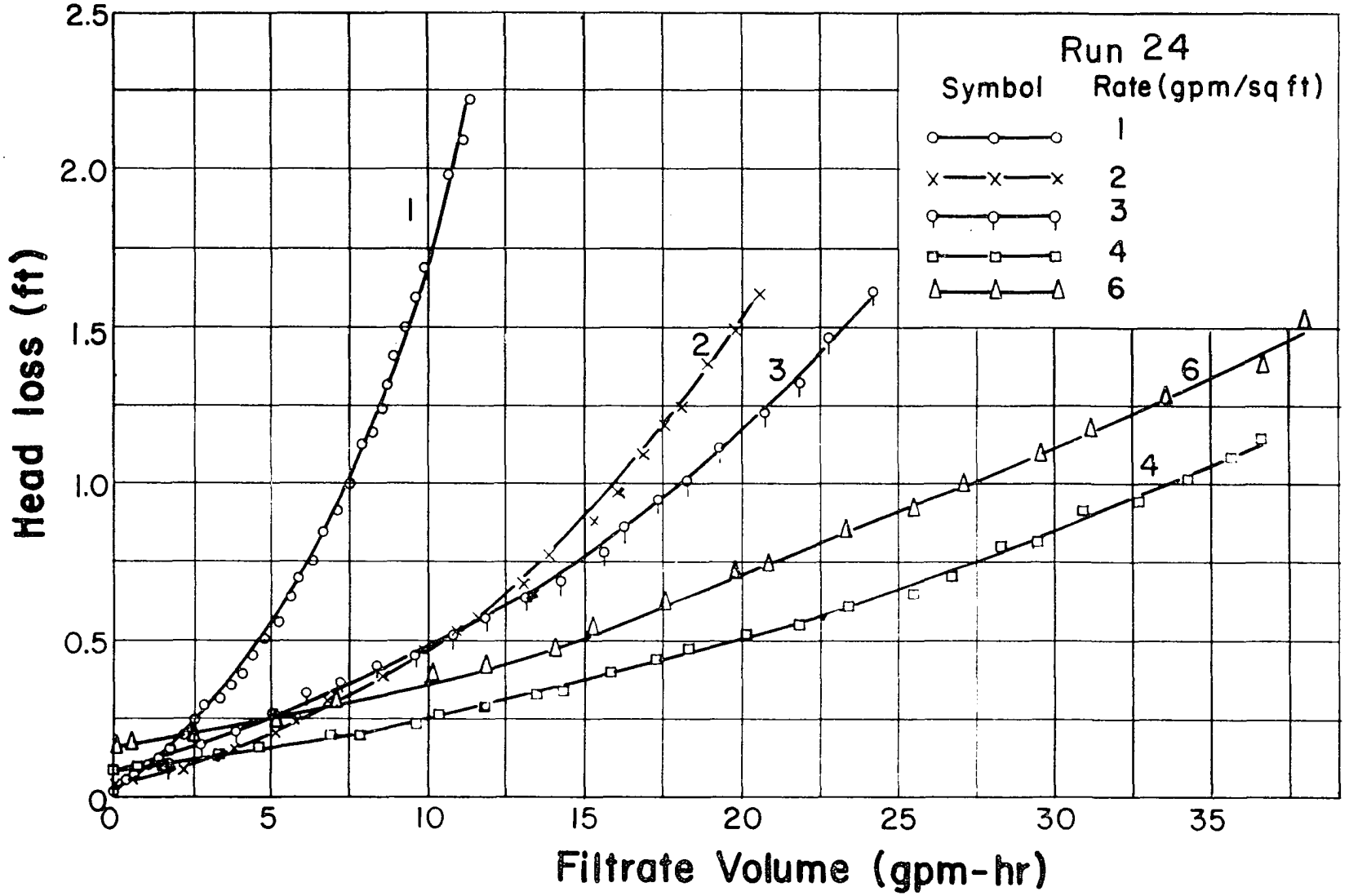
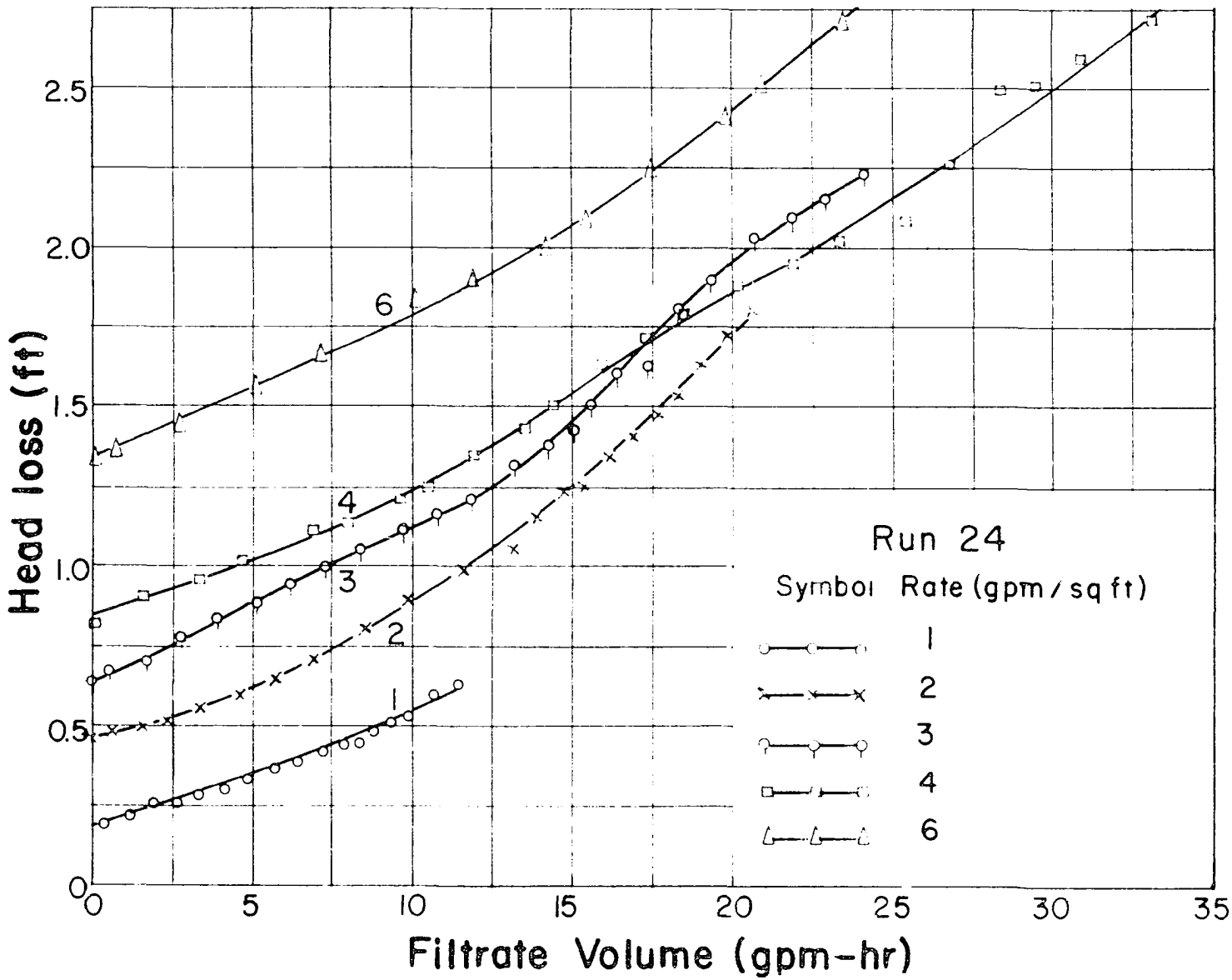

Run 24
Symbol
Rate (gpm/sq ft)
1
2
3
4
6
Head loss (ft)
Filtrate Volume (gpm-hr)
0
0.5
1.0
1.5
2.0
2.5
5
10
15
20
25
30
35

Figure 23. Run 24, head loss excluding top 1 in. layer vs. filtrate volume

Filtering Ames filter influent water diluted with tap water to control stability and turbidity.


Head loss (ft)
Filtrate Volume (gpm-hr)
0
0.5
1.0
1.5
2.0
2.5
0
5
10
15
20
25
30
35
Run 24
Symbol Rate (gpm / sq ft)
1
2
3
4
6
1
2
3
4
6

cept for a short period at the beginning of the filter run. Equations have been developed by Ruth (49) for both constant pressure and constant rate cake filtration. Neglecting the head loss in the filter cloth, his equation for head loss development during constant rate filtration is:

$$P = \frac{\rho w \nu \alpha}{A^2 g_c (1-mw)} \left(\frac{dV}{d\theta}\right)^2 \theta \qquad \text{Eq 13}$$

Where:

V = Volume of filtrate (cu ft)
θ = Time (hr)
P = Pressure difference across the cake (lb/sq ft)
A = Area of the filter cake normal to direction of fluid flow (sq ft)
ρ = Density of filtrate (# mass/cu ft)
w = Weight fraction of solids in slurry
ν = Viscosity of fluid (# mass/ft hr)
α = Specific cake resistance
m = weight ratio of wet cake to dry washed cake
g_c = dimensional constant, 4.17×10^8 (ft lb mass/lb force hr^2).

For a given slurry, all items in equation 13 are constant except P, θ, $\frac{dV}{d\theta}$, and α. Combining the various constants, equation 13 may be simplified as follows for constant rate filtration.

$$P = K \alpha \left(\frac{dV}{d\theta}\right)^2 \theta \qquad \text{Eq 14}$$

The specific cake resistance (α) has been found to be pressure dependent for cakes that exhibit some compressibility. In the usual range of pressures in industrial cake filtration, the relation between α and P has been expressed as follows (47, p. 965):

$$\alpha = \alpha' P^s \qquad \text{Eq 15}$$

Where:

α' = A constant determined largely by the size of the particles forming the cake.
s = Cake compressibility.

The cake compressibility (s) varies from 0 for granular incompressible cakes to 1.0 for highly compressible cakes. For most industrial slurries, s lies between 0.1 and 0.8. Combining equations 14 and 15 for an incompressible cake yields:

$$P = K \alpha' \left(\frac{dV}{d\theta}\right)^2 \theta \qquad \text{Eq 16}$$

Equation 16 indicates a linear development of pressure during the filter run. Combining equations 14 and 15 for a partially compressible cake yields:

$$P = K \alpha' P^s \left(\frac{dV}{d\theta}\right)^2 \theta \qquad \text{Eq 17}$$

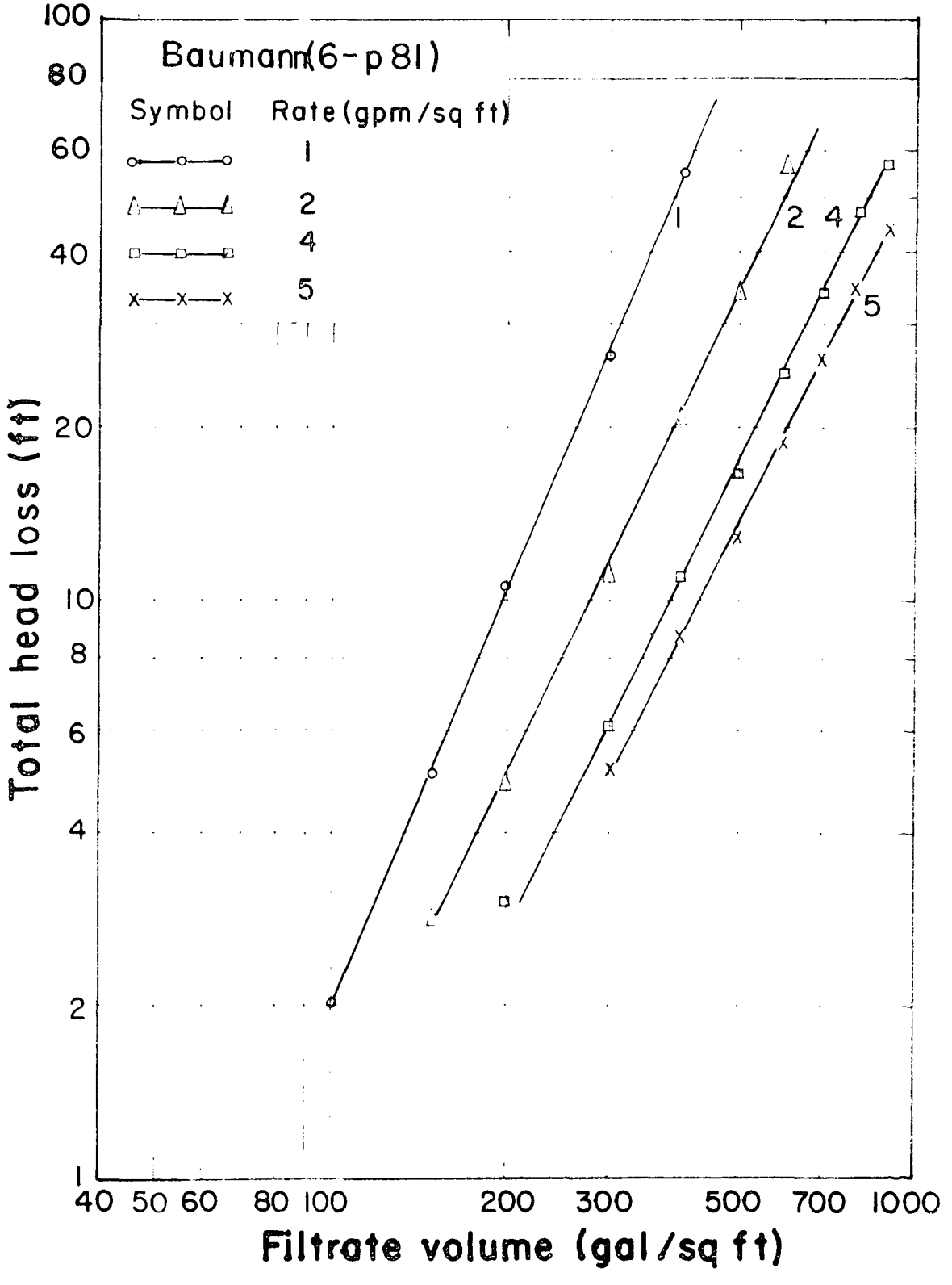
$$P = \left[K \alpha' \left(\frac{dV}{d\theta}\right)^2 \theta\right]^{\frac{1}{1-s}} \qquad \text{Eq 18}$$

For values of s between 0 and 1.0, equation 18 indicates an exponential development of pressure during the filter run.

Baumann (6) has observed the strongly increasing head loss curves when filtering iron coagulated water through diatomaceous earth filters without the aid of body feed. In these experiments with the fine precoat layer acting as the only filtering media, surface straining was the predominant mechanism of removal. When the surface layer became clogged, the head loss increased at a more and more rapid rate. Figure 24 is a typical plot of data from Baumann's studies of the effect of filtration rate on filtrate volume without the use of body feed. Figure 24 illustrates exponential head loss development at all filtration rates, and,

Figure 24. Total head loss vs. filtrate volume for diatomite filters without body feed

From Baumann (6, p. 81) filtering ferric chloride coagulated water with 0.15 lb/sq ft C535 precoat with no body feed. Influent turbidity = 1.0 units.


Baumann(6-p 81)
Symbol
Rate (gpm/sq ft)
1
2
4
5
Total head loss (ft)
100
80
60
40
20
10
8
6
4
2
1
40 50 60 80 100 200 300 500 700 1000
Filtrate volume (gal/sq ft)

therefore, conformity with equation 18 for cake filtration of a compressible precipitate. The slope of these curves is equal to $(1/1-s)$. Values of the cake compressibility (s) calculated for Figure 24 yield values of 0.51 at 2, 4, and 5 gpm/sq ft and 0.58 at 1 gpm/sq ft.

Baumann also observed that the use of adequate body feed to completely preclude the development of a surface sediment layer would cause the head loss curves to become linear.

4. Surface removal observations

On the basis of the foregoing equations for cake filtration, it can be concluded that the presence of a head loss vs. volume relationship which approaches an exponential curve is an indication of the formation of a partially compressible surface cake. To cause such exponential development of head loss, the surface cake must have adequate strength to bridge across the sand openings and resist the hydraulic forces tending to wash the deposited material deeper into the bed.

The presence of such a surface cake development was evident when filtering Ames filter influent water at low rates of filtration. For example, a layer of calcium carbonate precipitate was apparent on the sand surface at all filtration rates in run 18. At the lowest rate of 1 gpm/sq ft, this layer appeared 1/16 to 1/8 in. thick. When backwashed, the layer disintegrated into large pieces, many of which were up to one inch in major dimension. These particles could not be removed by backwashing and resisted further disintegration, indicating that the layer had been strongly compressed. The surface layer was not so thick at 3 gpm/sq ft and left particles unwashed of about 1/4 in. maximum major dimension. At 5 gpm/sq ft the layer was thin and broke into pieces less than 1/8 in. in major

dimension, most of which were removed in backwashing.

The development of a compacted surface layer was not evident in any of the filter runs on water containing precipitated hydrous ferric oxide. While the sand surface was generally covered with a layer of red iron precipitate, the layer was soft and backwashed from the filter without difficulty.

5. Demonstration of conformity with surface cake equations

It can be hypothesized from the foregoing observations that head loss development on a sand filter is the sum of the head loss caused by removal in the sand bed plus the head loss caused by the surface cake when such a cake develops during the run. The head loss caused by removal within the bed should develop in nearly a linear manner at constant sediment loading rates, since the sand will act as a rigid matrix to prevent compression of the sediment. The head loss caused by the surface cake should develop exponentially in accordance with equation 18.

To test the validity of this hypothesis, an attempt was made to separate these two components of head loss development, for the top sand layer, for all filter runs on Ames filter influent water. A tangent line representing head loss in the sand bed was fitted to the slope of the initial head loss development. The choice of slope was based on the assumption that the initial slope, before the surface cake had time to develop, would represent the linear development within the sand layer. The increase in head loss above this tangent line was assumed to be the development in the surface cake. This increase in head loss was plotted on log-log paper against filtrate volume to determine if it was exponential and in accordance with equation 18.

Figure 25 for run 24 is typical of these curves and indicates conformity with an exponential equation. The slope of the curves is equal to $(1/1-\varepsilon)$. Values of the cake compressibility (s) calculated for all runs on Ames filter influent water are recorded in Table 13. Some

Table 13. Particle compressibility "s" in runs 18, 19, 20, 24 and 25[a]

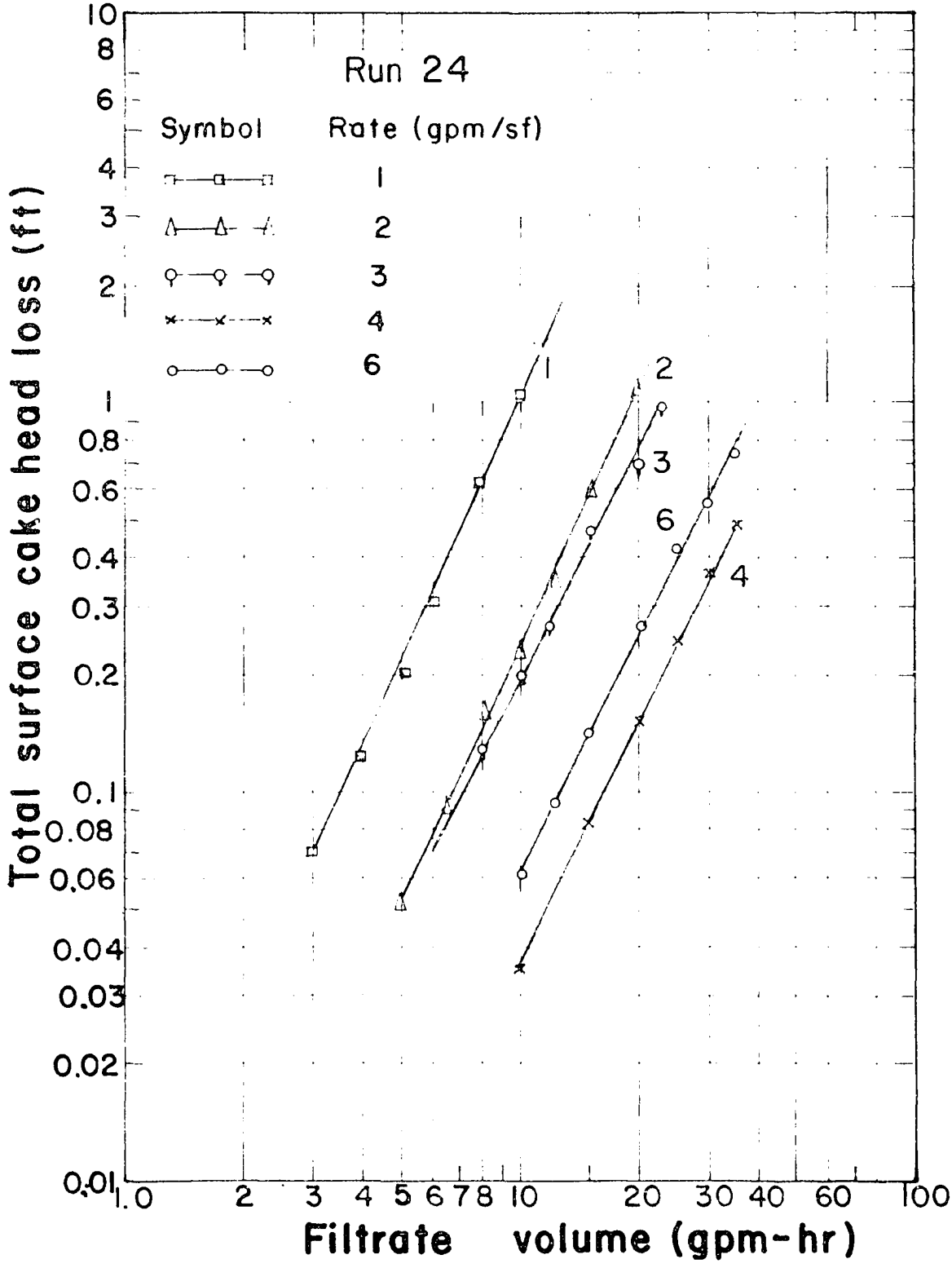
Run no.	Filtration rate						
	1	2	3	4	5	6	7
18	0.75	0.65	0.65		0.62		0.65
19		0.69	0.51		0.51	0.50	
20		0.60		0.60		0.60	
24	0.55	0.55	0.50	0.52		0.50	
25		0.63	0.55	0.52		0.45	

[a]Based on Eq 18.

decrease in cake compressibility (s) is apparent at higher rates. This is in accord with the observations of Ruth (50) who explains this as a time phenomena. Less time is available for compression of the cake at higher rates, and thus lower values of compressibility (s) are obtained. Theoretically, the cake compressibility (s) should be independent of the rate. Compressibility may vary for the different runs due to slight differences in the particle characteristics. The overall average value of cake compressibility is 0.59.

Figure 25. Run 24, surface cake head loss vs. filtrate volume

Filtering Ames filter influent water diluted with tap water to regulate turbidity and stability. Surface cake head loss component above estimated initial tangent to top 1 in. layer head loss curve shown in Figure 22.


Run 24
Symbol
Rate (gpm/sf)
1
2
3
4
6
Total surface cake head loss (ft)
Filtrate volume (gpm-hr)
10
8
6
4
3
2
1
0.8
0.6
0.4
0.3
0.2
0.1
0.08
0.06
0.04
0.03
0.02
0.01
1.0
2
3
4
5
6
7
8
10
20
30
40
60
100

E. Conclusions

From the foregoing observations, several conclusions can be made concerning the cause of an optimum rate tendency as follows:

a. Head loss which increases at a more rapid rate as the run progresses is the result of the formation of a compressible cake on the sand surface.

b. Total head loss development in such cases is the sum of the loss in the sand bed and in the surface cake. For constant loading rates, the loss in the sand bed probably develops in a linear manner, while the loss in the surface cake develops in an exponential manner in accord with established equations for cake filtration.

c. An optimum rate tendency can be expected when the particles being filtered are of such nature as to have adequate strength to form a surface cake which can resist hydraulic forces trying to wash it into the bed. The surface cake thus formed is subjected to increasing compressive forces as the run progresses. The gradual compression of the cake reduces its permeability and thereby increases the hydraulic gradient necessary to maintain constant flow rates. The reduced permeability improves the opportunity for further surface removal and further compression.

d. At higher filtration rates, deeper penetration takes place regardless of the presence or absence of an optimum rate tendency.

e. The improved production at higher rates, in runs with an optimum rate tendency, is the result of a smaller fraction of the suspended

matter being removed in the surface cake. This permits a larger volume of water to be produced before an appreciable surface cake develops.

f. The lowest rate at which the head loss development approaches linearity is the optimum rate. Higher rates, above the optimum, result in less production since the surface cake filtration influence has been reduced so that the head loss development is similar to that associated with suspensions which do not exhibit an optimum rate.

g. Suspensions for which head loss development is nearly linear do not exhibit an optimum rate tendency. With such suspensions, higher rates result in reduced production to a given total head loss due to the higher initial loss of head. The higher initial head loss leaves less head loss available for the increase which occurs during the run.

VII. EXPLANATION OF EFFLUENT QUALITY BEHAVIOR

The filter effluent quality behaved in much the same manner irregardless of the influent water type or the presence or absence of an optimum rate tendency. Certain general observations concerning effluent quality behavior are discussed in the following paragraphs.

A. Effect of Filtration Rate on Effluent Quality

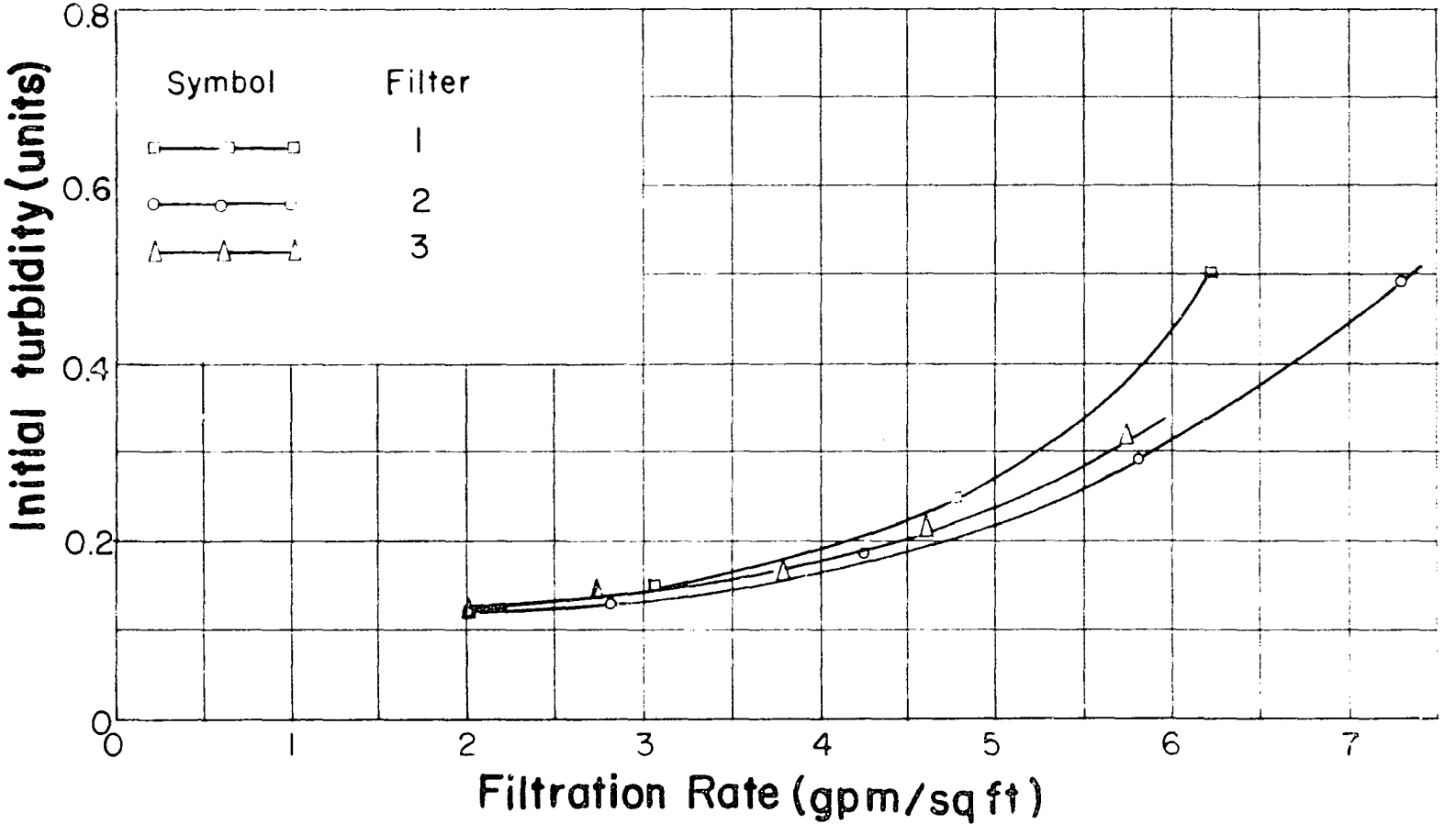
Effluent quality gets poorer at higher filtration rates. The effect of filtration rate on effluent quality was studied following run 25 in which Ames Filter influent water diluted with tap water was filtered. All three filters were operated at several different rates for a short period. The best water quality following the initial recovery period was observed. The results of this study are shown in Figure 26. At rates below 4 gpm/sq ft, the curve is quite flat with only slight changes in quality at different rates. Above 4 gpm/sq ft, the curves turn up sharply indicating more rapid degradation of initial quality as filtration rates are increased.

The effect of filtration rate on effluent quality does not seem to follow any simple mathematical relation. These data do not fit a straight line on either semi-log or log-log paper.

A water of zero turbidity or zero iron content was never achieved in any of the filter runs, even at rates as low as 0.7 gpm/sq ft. Figure 26 indicates this, since the curves are nearly horizontal at 2 gpm/sq ft. Apparently, some fraction of the particles, due to size or electrical charge characteristic, cannot be filtered even at extremely low rates.

Figure 26. Initial effluent quality vs. filtration rate

Quality following the period of effluent improvement at the beginning of the filter run. Special study of the effect of filtration rate on initial effluent quality conducted following run 25.


Initial turbidity (units)
0.8
0.6
0.4
0.2
0
Symbol
Filter
1
2
3
0
1
2
3
4
5
6
7
Filtration Rate (gpm/sq ft)

Such a behavior might be expected considering the fact that the suspensions being filtered were heterogeneous in size and, most likely, also in charge characteristic. The hydrous ferric oxide particles generally ranged from 1 to 20 μ in size with a majority about 5 μ. Calcium carbonate particles in the Ames filter influent water ranged from 1 to 10 μ with a few observations of particles up to 40 μ. These particle size distributions were observed with a Sedgwick-Rafter counting chamber, using a calibrated ocular micrometer in the eyepiece of the microscope. Observations were made at 100 and 430 power magnification and it is possible that some particles smaller than one μ were present but could not be readily observed.

With this range in particle size, one would expect the smaller fractions to be less easily filtered by interstitial straining than the large fractions. By the electrokinetic mechanism of filtration, those particles with the unfavorable charge characteristic would be most difficult to remove.

Therefore, whether the principal mechanism is straining or electrokinetic attraction, the increment of quality improvement at successively lower rates would become smaller and smaller due to the fact that the particles remaining are the most difficult fraction to remove. As rates are progressively increased, the quality will degrade more and more rapidly as the more easily removed particle fractions are forced through the bed. The best filtration rate would lie in the more horizontal portion of this curve below 4 or 5 gpm/sq ft. It is interesting that the optimum rate for maximum production per run was generally between 3 and 5 gpm/sq ft.

B. Filtrate Quality at Various Depths

The presence of some turbidity or iron in the effluent at all rates indicates that the entire filter is playing some active part in the clarification. To investigate the validity of this statement, the relationship between iron removal and depth was studied in runs 21 and 22. Water was permitted to flow continuously through capillary tubes attached to the piezometer sampling connections at successive depths in the filter. At periodic intervals, samples were collected at each depth and analyzed for total iron content. It was observed that some iron removal does take place between each piezometer connection at all rates of filtration. The full filter is thus contributing to the filtration action. The amount of iron removed in a lower lamina of sand, however, is much less than in a lamina near the surface. This suggests that perhaps any lamina in the filter can only remove some fraction of the iron which it receives. If this fraction were constant, the iron content of the water plotted against depth, for a clean filter, should plot as a straight line on semi-log paper.

Ives (44) has recently proposed a mathematical expression for the relation between time, depth, and concentration of remaining sediment in the filtrate. According to his equation for a clean isotropic filter bed, and for a suspension of unisize, homogenious particles:

$$I = I_0 \, e^{-kl} \qquad \text{Eq 19}$$

Where:

I = Concentration of suspended particles in the flow at any depth

I_0 = Concentration in the filter influent water

k = Constant called initial rate factor
l = Distance from the filter surfaces.

Such an exponential equation will plot as a straight line on semi-log paper. Iron concentration in the filtrate vs. depth is plotted in Figure 27 for runs 21 and 22. This figure indicates that Ive's equation is not completely valid when applied to a non-isotropic filter bed and a heterogeneous particle suspension.

The curves for all three rates are nearly linear in the top 6 in. of sand indicating conformity with Ives' equation. This portion of the sand bed appears to be nearly uniform in size. Below the 6 in. depth, the points are more eratic and follow a slight curve. The much flatter slope in this lower region indicates a much less efficient removal and a wide divergence from Ives' equation. This divergence is due both to a non-isotropic sand bed and to the heterogenious particle suspension described in the previous section. The filter sand is a graded sand and gets considerably coarser at greater depth. It is impossible to state, on the basis of the work done in this study, which of these two factors is more responsible for the divergence. Additional work should be done with a uniform sand filter to study the single effect of the heterogeneous particle suspension.

C. Effluent Quality During a Filter Run

At lower rates of filtration, the effluent quality remained fairly constant throughout a filter run. At higher rates, however, the effluent quality gradually degraded as is illustrated in Figure 12 and Table 10. In some cases, the effluent quality would remain fairly constant through

the early part of the run and then gradually degrade.

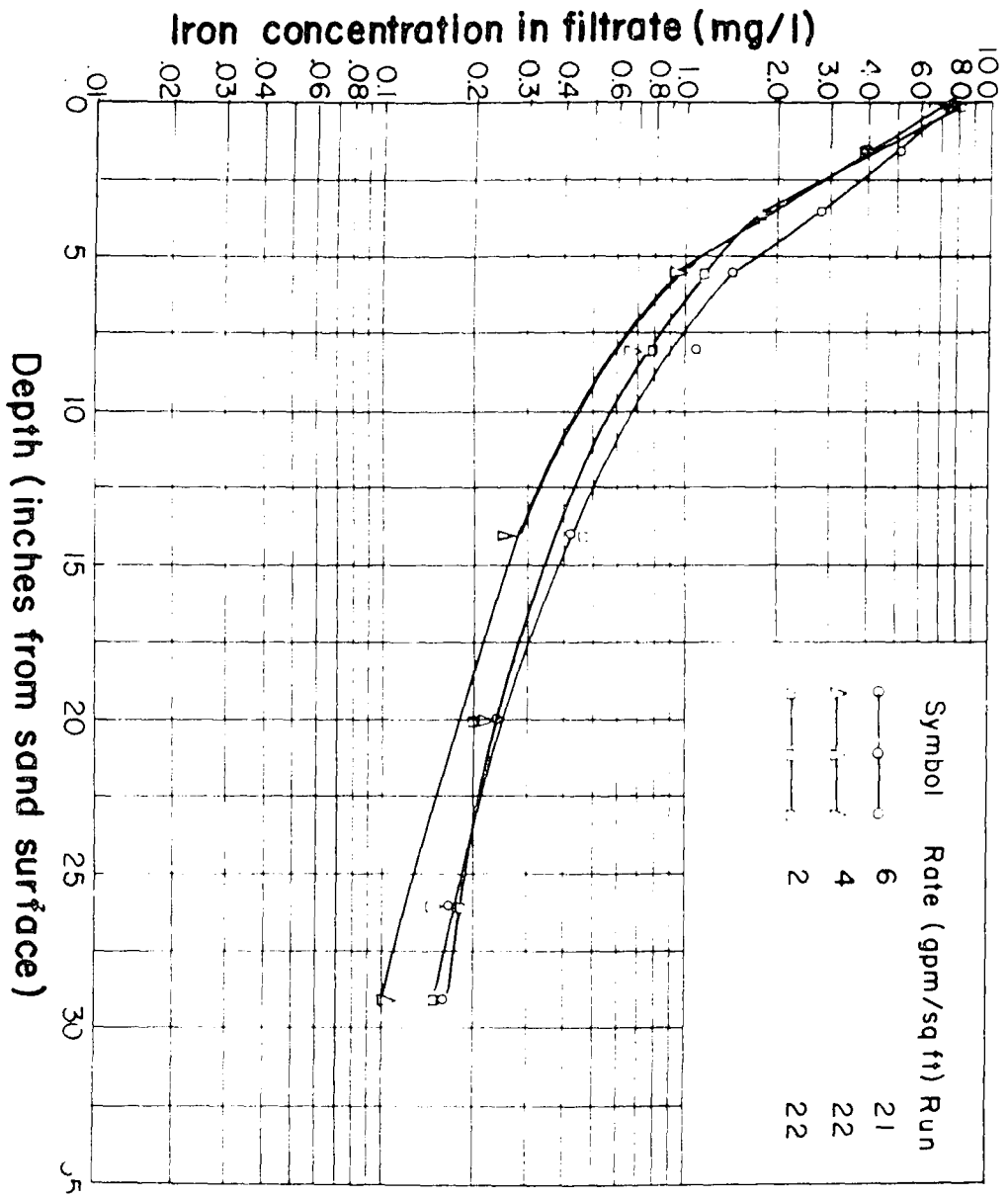
This behavior is rather unexpected since the entire filter bed is playing an active role in filtration. As the filter run progresses, the burden of removal moves deeper and deeper into the filter bed. As a greater burden is forced deeper into the filter bed, one would expect the effluent quality to degrade. This does not occur at the lower rates of filtration. A study of the relationship between iron concentration and depth at various intervals during the filter run may shed some light on this anomaly. Figure 28 shows this relationship at various times during run 22 conducted at a filtration rate of 4 gpm/sq ft. A family of straight lines is apparent on Figure 28. As the run progresses, the upper portions of the bed cease to follow Ives' equation 19 as they get dirty. They are replaced, however, by a deeper region in the filter which is still clean enough to follow Ives' equation. Fewer and fewer of the lower filter layers follow the flat slope observed on the clean filter. It is hypothesized that gradual effluent degradation will begin when the filter becomes sufficiently dirty that the straight line obeying Ives' equation continues to the bottom of the filter. It is further hypothesized that the heterogeneous particle dispersion is more responsible for the nonconformity with Ives' equation than the nonuniformity of the sand bed. These hypotheses are not verified since they are outside of the objectives of this study. They should receive further attention in a separate study.

D. Relationship of Curve Type to Effluent Quality

It would be convenient if an operator could detect the tendency toward passage of excessive material through his filters by observation of his

Figure 27. Iron concentration in the filtrate vs. depth for runs 21 and 22

At three different rates on a clean filter after 0.6 hours of operation.


Iron concentration in filtrate (mg/l)
100
80
60
40
30
20
1.0
0.8
0.6
0.4
0.3
0.2
0.1
.08
.06
.04
.03
.02
.01
0
5
10
15
20
25
30
35
Depth (inches from sand surface)
Symbol
Rate (gpm/sq ft)
Run
6
21
4
22
2
22

head loss vs. time charts. Hudson (38) has suggested that a linear curve would be an indication of this tendency. Figures 8, 9, and 10 would invalidate this statement as evidenced by good quality water produced at various rates in which the head loss vs. time curves were nearly linear.

On the other hand, a head loss vs. time curve in which the head loss goes up at an increasing rate as the run progresses does not necessarily preclude the passage of excessive material as evidenced by Figure 14 for run 19, in which excessive material was passed at high rates with this type of head loss development.

The only conclusion, if any, that can be drawn from these observations is that good effluent quality is not necessarily associated with any characteristic head loss vs. time curve.

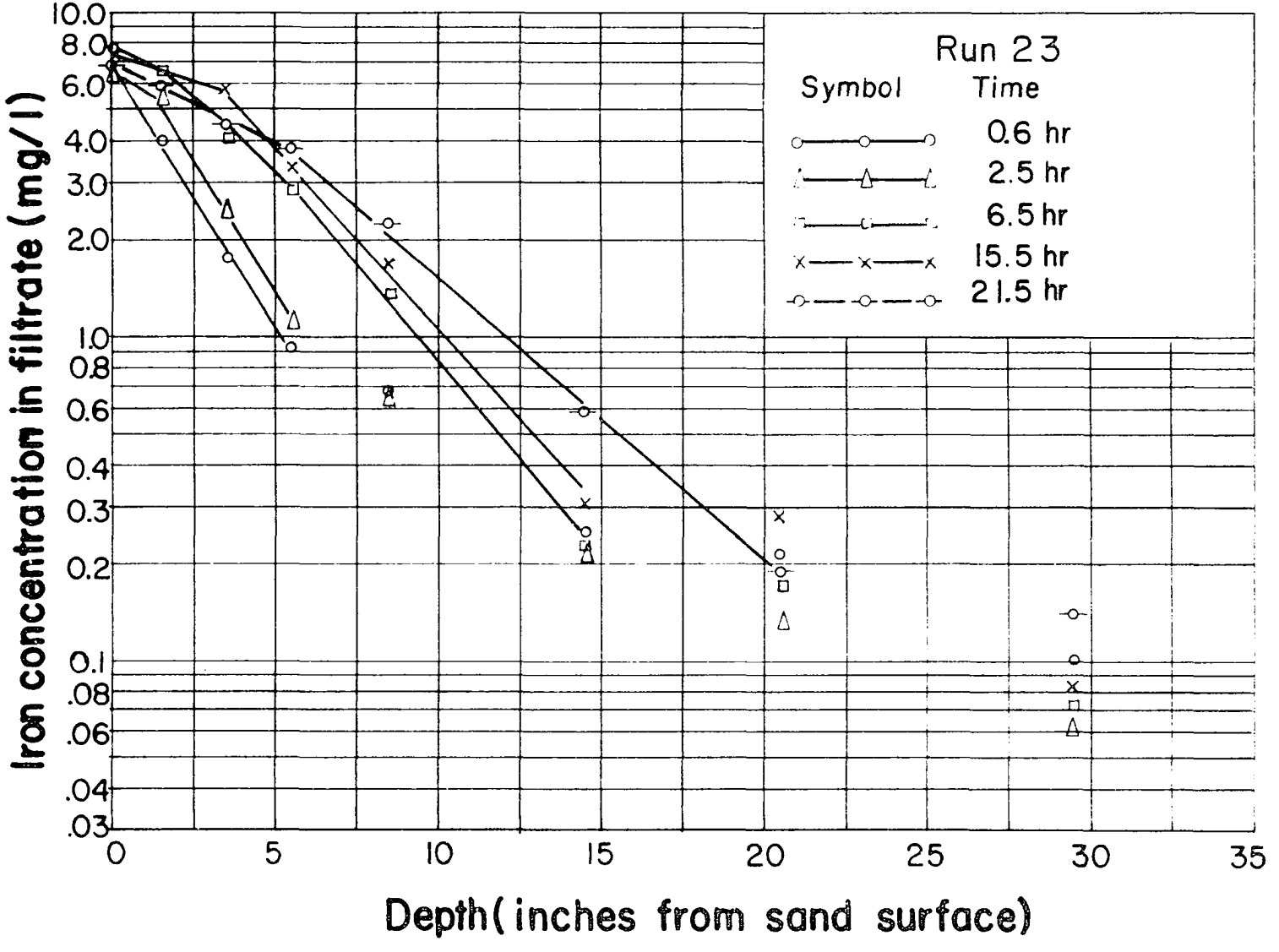
E. Evaluation of Predominant Removal Mechanism

Hydrous ferric oxide particles precipitated from the raw city water by aeration and slow mixing were observed to be extremely fine, as described in section VII A. With the coarse sand in use in the first eight filter runs (1.05 mm effective size) it was impossible to get adequate removal of these fine particles at rates above 2 gpm/sq ft. The particles averaged about 5 μ in size and ranged from 1 to 20 μ. Since the particles were small, experiments were conducted using ferric chloride solution in city tap water to try to precipitate a larger particle which might be filterable at somewhat higher rates on the coarse sand in use.

Laboratory jar tests and subsequent pilot plant operation indicated that a much larger particle could be produced in this manner. Particles between 20 and 50 μ predominated with a few as large as 100 μ and a few

Figure 28. Iron concentration in the filtrate vs. depth at various times during run 22

Rate of filtration was 4 gpm/sq ft.


Run 23
Symbol
Time
0.6 hr
2.5 hr
6.5 hr
15.5 hr
21.5 hr
Iron concentration in filtrate (mg/l)
10.0
8.0
6.0
4.0
3.0
2.0
1.0
0.8
0.6
0.4
0.3
0.2
0.1
.08
.06
.04
.03
0
5
10
15
20
25
30
35
Depth(inches from sand surface)

less than 20 μ . Run 4 was conducted using water prepared in this manner. Contrary to expected results, the large particles were even more difficult to remove than the 1-20 μ particles, with excessive iron passing through the filter at rates as low as 1 gpm/sq ft and at a head loss of only 1 ft. City tap water with 10 mg/l iron added in the form of ferric chloride solution had a pH of 7.6. This should be approximately the iso-electric point of the floc as observed by Stanley (51) and, therefore, it would be expected that the iron would be filtered with ease. In these runs, the head loss vs. filtrate volume curves were approximately linear. This type of particle would be described as a very "weak floc" since it has the filtering characteristics of a weak floc described by Hudson (35) (see p. 25).

In a further attempt to alter this particle in some manner to make it filterable, further jar tests were conducted using a commercial coagulant aid, Nalco 600. Large distinct particles averaging 100 μ in size were precipitated. This additive was used in run 5 with no improvement in the iron removal attained. Excessive iron passage occurred during the entire filter run at 2 and 3.5 gpm/sq ft. Iron began to pass at a head loss of 1.5 ft at 1 gpm/sq ft.

Run 6 was made in a further attempt to improve the filterability of the particles by adjusting the pH to a high value of approximately 10.6 which yielded a large heavy particle in laboratory jar tests. No improvement in iron removal was obtained.

It was apparent from runs 1 through 6 that the size of hydrous ferric oxide particles had little to do with the relative iron removal efficiency of the filter. The larger particles were less filterable than the smaller particles. This behavior may be interpreted, as follows, in terms of the

three mechanisms of filtration previously discussed, namely: interstitial straining, sedimentation, and electrokinetic attractive forces. If the removal mechanism is primarily mechanical in nature, such as interstitial straining or sedimentation, then larger particles should be more filterable than small particles. The opposite behavior may be explained by either of the following two possible hypotheses. The internal bonding of the particles, which are formed under very gentle mixing, may be of insufficient strength to resist the hydraulic shearing forces that exist within the filter; or, the external electrical charges carried by the particles may not be suitable for attachment either to the sand or to previously deposited particles. Particles which are formed and filtered at a pH other than the iso-electric point could be expected to exhibit both characteristics.

On the basis of these observations, electrokinetic forces appear primarily responsible for the removal of hydrous ferric oxide particles. This is in agreement with the conclusions of Stanley (51) and Stein (52). Particles removed by such a mechanism should have little preference for removal on any particular position on the sand grain. Such appears to be the case. When the dirty filters were observed with a hand lens, iron particles appeared to surround the sand grains completely. They exhibited no preference for horizontal surfaces or interstices. It was further observed on several occasions that the presence of some dissolved iron in the filter influent water improved the removal efficiency. Precipitation of the dissolved iron seemed to be catalyzed by the particles already deposited. Its precipitation within the bed seemed to aid in bonding the small particles in suspension to those already deposited in the filter.

Under these conditions, greater removal took place in the upper regions of the bed. Both of these observations support the importance of the electrokinetic mechanism in the removal of hydrous ferric oxide particles.

Particle size in the Ames filter influent water was observed to range from 1 to 10 μ ; thus, the aerated mixed raw water containing hydrous ferric oxide particles and the Ames filter influent water containing calcium carbonate particles had about the same particle size. Yet, the Ames filter influent water had a strong tendency toward surface removal; whereas, the water containing hydrous ferric oxide particles did not.

What are the reasons for the difference in the removal of these two types of particles? The calcium carbonate particles appeared granular and nearly spherical under the microscope, in sharp contrast with the gelatinous irregular hydrous ferric oxide particles. Since the particle size was about the same in both waters, one would suspect either the internal structure, or the external charge characteristic or both to be responsible for the different filtering characteristics. When the dirty filters were observed with a hand lens, the calcium carbonate particles appeared to predominate on the upper surfaces, at and above the interstices of the sand grains.

These observations indicate a predominance of interstitial straining and sedimentation as mechanisms of removal for this type of particle. A granular or less gelatinous particle of this type would lend itself to a mechanical removal. They should be capable of bridging at the interstices and resisting the hydraulic forces tending to cause their disintegration.

The term "strong floc", which is frequently used to describe an easily

filtered particle suspension, may have two connotations when viewed in light of the foregoing observations. It may mean a particle which possesses strong internal bonding, such as the calcium carbonate particles, and which can then be removed mechanically by interstitial straining or sedimentation. However, the removal of such particles may also be aided by electrokinetic forces if the external charge characteristic is suitable. Thus, the second connotation of the term "strong floc" refers to the existence of a suitable external charge characteristic on the particles to permit their removal by the electrokinetic mechanism. Some particles, such as the hydrous ferric oxide particles, may have weak internal bonding and are incapable of removal primarily by interstitial bridging. Such particles must depend on the electrokinetic mechanism for removal. If they do not bear suitable electrical charge characteristic, they will not be readily removed by the filter.

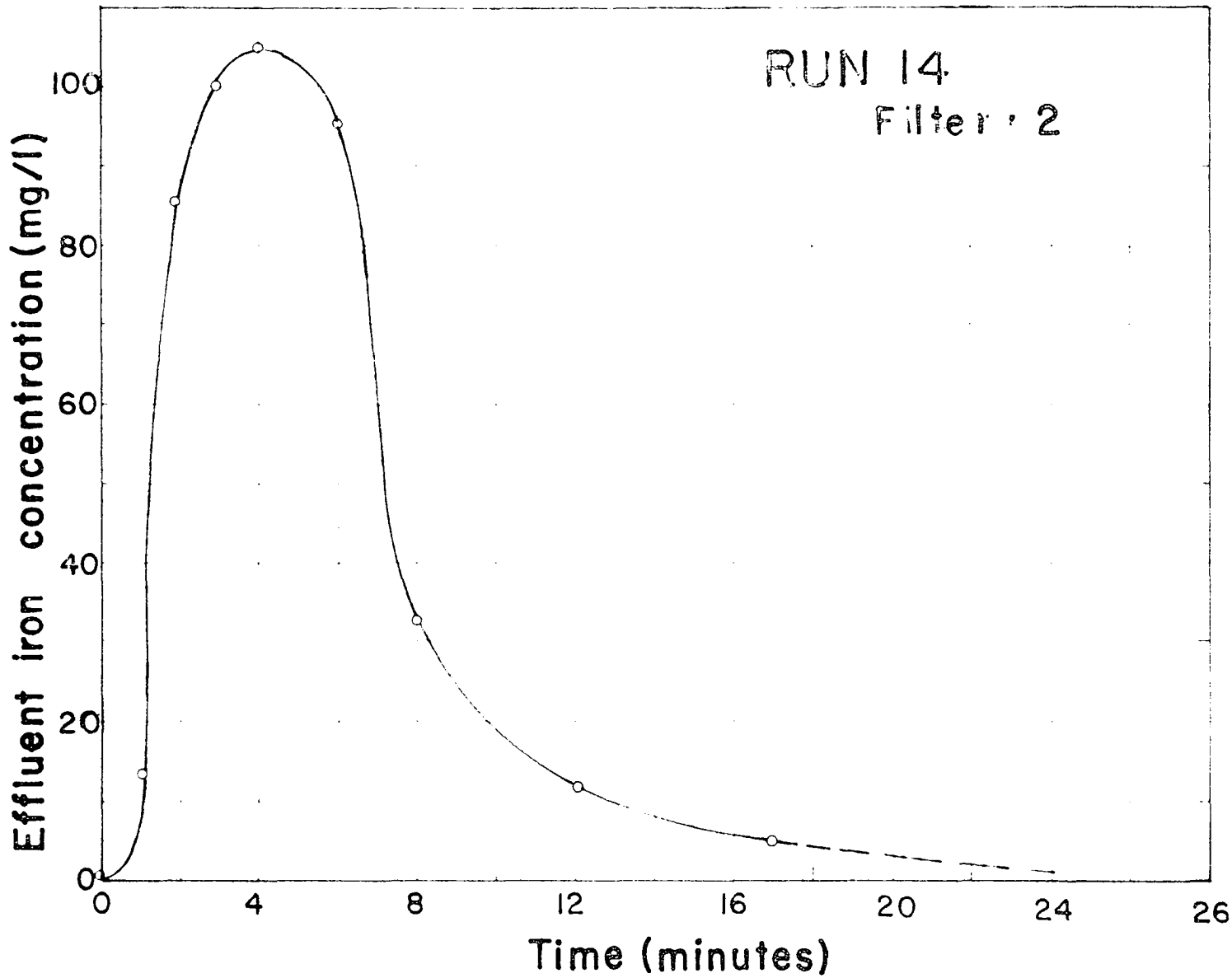
F. Effect of Rate Changes on Effluent Quality

On several occasions in the earlier runs, a disturbance in rate accidentally occurred for reasons previously outlined in section V E. Such a rate disturbance resulted in the passage through the filter of considerable material which had been previously deposited. The passage of such material was accompanied by a reduction in head loss. The filters recovered quite rapidly, and soon were producing water of quality equal to that produced prior to the disturbance.

This phenomenon was studied more objectively on several occasions by causing a moderate rate change on a dirty filter near the end of a filter run. Figure 29 presents the effect of such an experiment on the effluent

Figure 29. Run 14, effluent iron content vs. time following rate disturbance

Following run 14 on filter 2 at 3 gpm/sq ft on water containing hydrous ferric oxide particles. Rate increased from 3.0 to 4.20 gpm/sq ft for approximately twenty minutes, and then returned to initial rate.


RUN 14
Filter : 2
Effluent iron concentration (mg/l)
0
20
40
60
80
100
0
4
8
12
16
20
24
26
Time (minutes)

quality at the end of run 14 on filter 2. The rate was changed from 3.0 to 4.2 gpm/sq ft in this experiment. The area under the curve represents a total of 2.28 grams of iron flushed from the filter bed. While this is only about 3.3 percent of the total iron previously deposited in the bed, it was accompanied by a reduction in head loss of 28 percent.

Similar observations were made when filtering city filter influent water. Rate changes of as little as 10 percent were observed to have a detrimental effect; however, the greater the rate change, the greater the effect on effluent quality and head loss. The duration of the rate change had little effect. A change for less than a minute would have about the same effect as a longer change lasting 10 to 20 minutes. The amount of sediment passed increased with the run length at the time of the disturbance.

From these observations, it is evident that the material removed in the filter is attached rather delicately to the sand or to previously deposited material. The manner of distribution of the material within the filter is peculiar to the specific rate of operation. Any rate increases result in unbalancing the equilibrium between the attaching forces which hold the material and the shearing forces of the liquid which try to tear it loose. After sufficient material has been flushed through the bed, a new equilibrium will be reached between the attaching forces and the now reduced shearing forces. The effect of rate increases on head loss is most pronounced in the dirtiest portions of the filter. Table 14 summarizes the effect of a rate increase on the head loss for run 14, filter 2, before and after the rate change described in Figure 29. It is interesting to note that in the upper 9 inches, the reduction of head loss

Table 14. Effect of rate increase on head loss, run 14, filter 2

	Laminae of the sand filter (in. from surface)						
	0-2	2-4	4-6	6-9	9-15	15-21	21-27
Head Loss Before (ft)	3.50	1.68	1.25	1.15	1.49	0.60	0.13
Head Loss After (ft)	2.45	1.17	0.88	0.81	1.09	0.48	0.12
% Reduction	30	30	30	30	27	20	8
% Change in Porosity	1	2	3	3.5	6	9	5

was uniform. However, the change in porosity increased with depth. The upper layer which was the most filled with material had the greatest head loss change but the smallest change in actual porosity. This is quite unexpected. One might expect the dirtiest portion of the bed to lose the most material on a rate increase. The opposite observation would lend support to the hypothesis previously suggested that in a heterogeneous suspension the different particles are removed with varying degrees of ease. Some are so difficult to remove that they pass the filter even at very low rates. Others with intermediate ease of removal reach intermediate depths. The flushing action of the rate change has increasingly greater affect on those particles at lower depths, which being more difficult to remove, are less firmly held in the filter.

The same conclusion can be reached by analysis of the head loss change data of Table 14. Since laminar flow conditions exist, the uniform percentage drop in head loss in the 4 upper layers reflects a uniform percentage drop in hydraulic gradient. The absolute level of the gradient is much higher in the upper layers than in the lower layers both

before and after the rate change. Thus, those particles in the upper layers resist greater hydraulic shearing forces before and after a rate change than the particles at lower levels. The ability of the particles in the upper levels to resist greater hydraulic gradients indicates stronger attachment to the sand.

The most important conclusion to be reached from these observations is that a rate increase, during a filter run has a very detrimental effect on filter effluent quality. Such rate increases should be positively avoided. This fact is not recognized by many water treatment plant operators. It is the practice in some treatment plants to increase filtration rates on dirty filters when plant rates are increased. This and other sources of rate change such as air binding, hunting rate controllers, and filter bumping should be eliminated.

These detrimental affects have gone unobserved in the past due to the absence of continuous turbidity monitoring devices. The development and use of the device used in this research should do much to help the progressive operator see poor operational practices and correct them.

G. A Choice: Constant Rate, Constant Pressure or Uncontrolled Filtration

The time tested method of constant rate filtration advocated by Fuller (26) at the turn of the century has recently been challenged by two alternative methods of operation namely: constant pressure[1] and uncontrolled[2] filtration.

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[1]Sometimes called declining rate filtration.

[2]Sometimes called variable rate filtration.

1. Constant rate filtration

Rate of filtration is held constant by means of a rate controlling device which automatically opens or closes a filter effluent valve to maintain the pre-set rate of flow. A disadvantage of this system is the initial and operating cost of the rate controllers. An improperly functioning rate controller may continually hunt for the desired valve position, first going above and then below the desired rate. Such rate variations will reduce the effluent quality.

Many treatment plants operate at a constant rate for several days. The rate is dictated by the average consumption. Constant plant input simplifies chemical handling and feeding problems. One major advantage of constant rate filtration is the ease of balancing a constant plant input to the filter output. For a small plant with only a few filters, this is a distinct advantage. In addition, sixty years of constant rate experience have clearly shown that good effluent water quality can be obtained by this means of operation.

2. Constant pressure filtration

This method of operation involves setting the effluent valve to yield a desired maximum filtration rate at the beginning of the filter run and thereafter allowing the rate to decrease gradually as the filter becomes dirty. If the water level is held reasonably constant above the sand surface, the filter is actually operating with a total pressure drop through the sand, gravel, underdrains and piping which is constant throughout the run.

The principle disadvantage of this method is the difficulty of balancing a constant plant input to filter output. Since the rate on each

filter is constantly decreasing, it is difficult for a small plant to equate the total filtration rate to a constant plant input rate.

Advantages of this method include the elimination of the initial and operational costs of the rate controller and the elimination of associated problems such as the detrimental effect on water quality due to the hunting controller. Some rate measuring device is still necessary so that the full cost saving of the rate controller is not realized.

While relatively little experience has been obtained with constant pressure filtration, some writers claim an improvement of water quality, and water production per run by this operating procedure (31,39,40).

While a complete study of constant pressure filtration is beyond the scope of this study, one filter was operated in this manner during run 23. The filter was started at 6 gpm/sq ft and the rate was permitted to decline as the head loss through the filter increased. The head loss development is shown in Figure 10 and the effluent quality is shown in Figure 12. From Figure 10 it is evident that the volume of water produced to a given terminal head loss was nearly equal to that produced on the filter operated at a constant rate of 6 gpm/sq ft. The effluent quality was much better in the constant pressure run as shown in Figure 12. In fact, the constant pressure filter had better effluent quality than the constant rate filter at 4 gpm/sq ft. These observations support the reports (31,39,40) regarding quality, but do not support the reports of increased water production.

Hudson (40) has suggested that constant interstitial velocities are present during a constant pressure filter run. Since laminar flow conditions exist, constant velocity would be evidenced by constant head loss

during the run. It is readily apparent from the increasing head loss shown in Figure 10 that increasing velocities do take place and that Hudson's suggestion is erroneous.

The improvement in effluent quality with constant pressure filtration makes it an attractive operating procedure. In large plants where many filters are in service, the difficulties involved in balancing inflow and outflow rate, and maintaining a constant water level, may be fairly easy to overcome. In small plants however, no easy solution is evident.

3. Variable rate filtration

In some large plants, the filtration rate is controlled by the water level in the clear well following the filters. The filtration rate is permitted to increase as the clear well water level decreases; and the rate decreases as the clear well level rises reducing the available head for filtration. A notable example of this type of operation is the Chicago South District Filtration plant where some of the many filters are operated in this manner. Baylis (17) has reported no detriment to the water quality due to the variable rates. Since the effluent quality is detrimentally affected by increasing the rate of filtration on a dirty filter, one would expect reduced water quality to be the result of variable rate filtration. However, at Chicago the rate changes are probably quite gradual due to the size of the city. If the period of rate change is fairly long and the filter run length fairly short as they are in Chicago, no appreciable detriment may develop. In small towns, however, where significant rate changes may occur during a fraction of a run length on a dirty filter, appreciable material may be forced from the bed.

This method of operation should be avoided as should any other

operational method which permits rates to increase even a small amount for a short period on a relatively dirty filter.

VIII. SUMMARY AND CONCLUSIONS

A. Present Status of Water Filtration

The development of the rapid sand filter began in 1884 and was standardized by Fuller's (26) historic research reported in 1898. The long years of relatively successful operation of sand filters at 2 gpm/sq ft, as originally suggested by Fuller, led to the common acceptance of this rate as the maximum acceptable rate of filtration. However, Fuller suggested that considerably higher rates were possible.

This suggestion has been verified in the past twenty years by a large amount of research and plant scale experience at higher rates of filtration. Many plants are now operated at higher rates during peak demand seasons. However, no research has pointed the way to the selection of the best rate of filtration. The best rate of filtration will be the rate which produces the largest quantity of acceptable water per filter run.

It is to this problem that this research is directed, namely:

a. to determine whether there is an optimum rate of filtration for a given water supply and for a given filter from the standpoint of the volume of acceptable water produced per filter run.

b. to establish simple criteria by which the operator can determine whether an optimum rate can be expected for his particular water quality; and, if so, how the optimum rate can be identified.

c. to explain the factors which result in the presence or absence of an optimum rate of filtration on any water supply.

B. Investigations

A pilot plant was constructed which included three 6 in. ID sand filters, each with a 30 in. layer of graded sand supported by 9 inches of graded gravel. The pilot filters are capable of operation between 2 and 10 gpm/sq ft and are equipped with combined piezometer and sampling connections at short intervals through the filter depth. Twenty-five experimental runs were conducted on the pilot filters over a period of 18 months on two types of influent water described below:

a. Water containing precipitated hydrous ferric oxide in suspension. This water was obtained either by aerating and mixing the raw well water of the Ames municipal treatment plant, or by adding iron salts to the Ames tap water followed by mixing to cause precipitation. The iron precipitated from the raw water was found to be more filterable and was used more frequently.

b. Water containing primarily calcium carbonate particles in suspension. This water was obtained from the influent to the Ames municipal filters following a typical two stage, split treatment, lime-soda ash softening process. In some filter runs, this water was diluted with tap water to regulate turbidity and stability.

The three filters were operated at different constant rates during the various filter runs to observe the existence of an optimum rate. The relation between total head loss and filtrate volume was plotted and used to select the optimum rate and to detect the identifying characteristics and the possible cause of the optimum rate tendency.

Effluent quality was monitored on all filter runs with continuous

reading turbidimeters. During filter runs on water containing precipitated iron hydrate, effluent quality was also monitored by quantitative colorimetric iron determinations. This data permitted an evaluation of the effect of increased filtration rates on effluent quality.

In two filter runs, the filtrate quality at various depths in the filter was studied at intervals during the filter run. These data permitted the evaluation of the validity of recently proposed equations for the relation between filtrate quality and depth. It also formed the basis for several hypotheses explaining effluent quality behavior.

Head loss development was observed by readings of the piezometer tubes at two hour intervals. Analysis of the head loss development in the various layers of the sand bed provided an explanation of the principal cause of an optimum rate tendency. It also permitted observation of the relative utilization of the filter voids at various filtration rates, and the gradual transfer of the burden of removal deeper into the filter as the run progressed.

During one filter run, constant pressure filtration was compared with constant rate filtration from the standpoint of water production and effluent quality.

C. Conclusions

1. Presence and identification of an optimum filtration rate

In constant rate filtration, several conclusions can be made with regard to the presence and identification of an optimum rate of filtration as follows:

a. In the filtration of suspensions which cause head loss[1] to develop at an increasing rate as the filter run progresses, greater water production per run can be expected as rates are increased. An optimum rate may be reached beyond which further rate increases result in decreased production.

b. The optimum rate tendency described above is absent when filtering suspensions which cause head loss to develop in nearly a linear manner. Head loss development curves for such suspensions are nearly parallel at different filtration rates. Since the higher rates cause higher initial head losses, lower production can be expected to any given terminal head loss.

c. The optimum rate can be identified as the lowest rate at which the head loss development curve becomes most nearly linear.

d. At and above the optimum rate, head loss development curves are nearly parallel and resemble the curves of a suspension with no optimum rate tendency.

2. Cause of the optimum rate tendency

Several conclusions can be made with regard to the cause of the optimum rate tendency as follows:

a. The increasing rate of head loss development associated with an optimum rate tendency is caused by the development of a compressible surface cake on the sand surface.

b. The surface cake only develops when filtering a suspension which has a strong tendency for surface removal, and in which the par-

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[1]Total head loss vs. volume of filtrate relationships.

ticles have adequate internal strength to resist the hydraulic shear forces tending to wash the particles into the filter.

c. Total head loss in the filter bed is the sum of the surface cake head loss development and the head loss development within the sand bed.

d. The head loss within the sand bed develops in a linear manner due to the rigid matrix of the sand grains which prevents compression of the deposited material.

e. Head loss in the compressible surface cake increases exponentially as the filter run progresses. The exponent depends on the cake compressibility and averaged 2.4 for the Ames filter influent water. This development is in agreement with established equations for cake filtration.

f. Increased production at higher rates is the result of two factors; greater utilization of the filter bed due to deeper penetration, and a reduction of the fraction of the particles removed in the surface cake. The latter factor permits greater production before a significant surface cake is produced.

g. At the optimum rate of filtration, the surface cake influence has been minimized and the head loss development approaches linearity. Above the optimum rate, production to a given terminal total head loss decreases since the underlying cause of an optimum rate tendency has been minimized or eliminated.

3. Effluent quality behavior

Several conclusions can be made regarding the reasons for effluent quality behavior as follows:

a. During a brief period at the beginning of each filter run, relatively poor effluent quality is obtained. The poorest quality occurs at approximately the theoretical displacement time of the water in the sand, gravel, and underdrains.

b. Following the peak of poor quality at the beginning of the run, the suspended matter in the effluent rapidly decreases to some minimum level.

c. This quality may then remain fairly constant throughout the filter run, or may gradually degrade during the entire run or the latter portions of the run.

d. The effluent quality following the initial improvement period is decreased by increased filtration rates. The curve of quality vs. filtration rate for Ames filter influent water is nearly flat at the lower rate when quality is plotted as the ordinate. The curve becomes increasingly steep at higher rates, particularly above 5 gpm/sq ft. Thus, the effect of filtration rate on effluent quality degradation becomes more and more important at progressively higher filtration rates. The relation between quality and rate did not fit any simple mathematical formulation.

e. Effluent water quality may not be acceptable at the optimum filtration rate selected from production considerations. In such a case, the selected operating rate must be reduced to meet the desired quality.

f. Complete removal of suspended material was not attained at rates as low as 0.7 gpm/sq ft, indicating that some fraction of the particles are of such size or charge characteristic as to be

unfilterable.

g. Good effluent quality is not necessarily associated with any particular type of head loss development curve.

h. Based on studies with water containing precipitated hydrous ferric oxide particles, the relation of filtrate quality to depth for a clean filter followed a first order reaction in the top 6 to 9 in. of sand as predicted by Ives (44) for a homogeneous suspension and an isotropic filter. The nonconformity in the remainder of the filter depth is believed to be due primarily to the heterogeneous particle suspension. Further study of this relation is needed.

i. As the filter run progresses, a deeper and deeper segment of the filter depth follows the first order relation between filtrate quality and depth. It is hypothesized, but not verified, that effluent degradation takes place when this segment reaches the bottom of the filter.

j. Rate increases on a dirty filter result in the flushing of considerable material through the filter before a new equilibrium is reached and good effluent quality is again attained. Such rate increases, regardless of cause, should be avoided.

k. The electrokinetic mechanism seems primarily responsible for the removal of precipitated hydrous ferric oxide particles.

l. Interstitial straining and sedimentation appear to be the predominant removal mechanisms for the calcium carbonate particles in the Ames filter influent water.

m. Particles removed in deeper regions of the filter bed are more

difficult to remove and are less firmly held than particles removed near the surface. This is evidenced by the fact that particles in the lower portions of the filter are more easily washed out by a sudden rate increase than are the particles in the upper portions of the filter.

n. Constant pressure filtration results in better effluent quality than comparable constant rate filtration.

o. Laminar flow conditions existed throughout all filter runs, in all layers of the filter, for all waters, and for all filtration rates covered by this study.

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He later modified the index (36) and renamed it the "filtrability index".

$$\text{Filtrability index for laminar flow}^2 = \frac{qd^3h}{l} \qquad \text{Eq 5}$$

Where:

q is the rate of filtration and all other nomenclature as in the floc strength index.

Hudson suggested that turbulence may develop in the clogged portions of the bed towards the end of the filter run and that breakthrough of floc may be associated with the development of turbulence. In accord with this hypothesis, he developed an alternate "filtrability index" for turbulent flow.

$$\text{Filtrability index for turbulent flow} = \frac{qd^2h\,l/2}{l} \qquad \text{Eq 6}$$

Experimental evidence will be presented in a later section to question this hypothesis concerning the development of turbulence. Since most rapid sand filters continually pass a small amount of turbidity, the use and value of the "breakthrough index" hinge on some arbitrary turbidity level considered a breakthrough.

Hudson urges better control of filtration with a desired turbidity level of less than 0.2 units. He cautions against unsteady flow to the filters, on-off operation, and surges in filtration rates due to the detrimental effect on effluent water quality (37).

Iwasaki (45) studied in minute detail the penetration of colloidal material and bacteria into slow sand filters. Beginning with rational differential equations for the time rate of removal of suspended matter in

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[2]Now called the "breakthrough index" (39).