EFFECT OF PREAERATION ON PRIMARY TREATMENT OF SEWAGE

by

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I. INTRODUCTION

Preaeration is not a new idea in sewage treatment; its origins extend back more than 25 years. Nevertheless, it remains one of our least understood and more controversial approaches to better waste treatment. The place of preaeration can best be appreciated in terms of a capsule outline of current practice.

The first major step in the sewage treatment process is generally one of quiescent settling to drop out all possible settleable solids. The equipment used generally permits removal of much floating material as well. This first step is strictly physical; it constitutes primary treatment, and may suffice where the receiving stream provides adequate dilution.

Additional or secondary treatment is of a biological and biochemical nature. In it, primary effluent is brought in contact with microbiological masses which convert colloidal and dissolved materials to more stable forms with greatly reduced pollutional strength. This biological process is followed by final settling, which is the last step in conventional complete treatment.

Preaeration precedes the waste treatment process outlined above. As defined in the Glossary -- Water and Sewage Control Engineering (16, p. 166), preaeration is "a preparatory treatment of sewage comprising aeration to remove gases, add oxygen, or promote flotation of grease, and aid coagulation". For the purpose of this investigation, the term preaeration was further restricted to pretreatment for a period of 30 to 45 minutes without either the addition of chemicals required for true coagulation or the return of final settling sludge, digester supernatant, or any other material which might serve as a physical or biological aid to flocculation. As the term is used here, preaeration does not include the interesting variations appearing in practice and in the literature which approach modified activated sludge treatment or which are interspersed at points beyond the primary settling step.

The value of preaeration, even after years of practice, remains open to argument. The most fitting comment on present knowledge of preaeration is the following quotation from a survey of current research problems (23, p. 1159)

> Plant operation personnel are also urged to review the problems that are listed, for some of the answers may be attainable through analysis of plant-scale data, or from special studies in operating plants. For example, preaeration facilities are provided in many modern plants, yet there are almost no data on the true advantages and limitations of the process.

The purpose of this investigation has been, therefore, to evaluate the influence of preaeration on primary settling efficiency; to study the factors affecting this influence; and to evaluate the economic worth of the preaeration process.

II. THE PREAERATION PROCESS; DEVELOPMENT

A. Early Uses

The development of preaeration is neither very recent nor very clear. In 1931 a unique grease removal procedure was used at Los Angeles' Hyperion screening plant (38). The process consisted of brief aeration in an open channel by means of high velocity jets of screened sewage discharging downward about 2 in above the surface, followed by a flow section permitting 2 to 3 min detention at reduced velocities. Removals of over half the free grease and oil were achieved, relieving a severe screen blinding problem.

As early as 1933, nine plants in the Midwest and two in California used preaeration; benefits cited were odor control, some addition of oxygen, substantial grease removal and improved settling (45). Preaeration detention times at design flow were given for the Dodge City, Kansas, and Whittier, California, plants as 13 and 19 min, respectively.

In 1935, laboratory experiments were carried out at Ft. Worth, Texas on preaeration of raw sewage to remove hydrogen sulfide (31). Mild improvement was noted in the first 15 min of preaeration and none thereafter.

In 1937, work was done by the City of San Francisco in an experimental 0.15 mgd treatment plant to effect both grease and grit removal by preaeration at the rate of 0.05 cu ft/gal during a 15 min period (26). The results were sufficiently encouraging to merit incorporation of a preaeration step in the 15 mgd plant then in the design stage.

B. Rutgers Studies on Chemical Coagulation

The 1930's saw increased interest in chemical treatment as a basic sewage treatment process. Concepts and coagulants transplanted from water treatment technology were adapted to the design of waste treatment plants aimed at results in the range between primary and complete treatment, but with minimum first cost. Operating results and costs were flexible, in response to stream requirements, through variations in coagulant dosage.

The 1930's also saw the earliest and essentially the only basic research studies in areas which might be considered related to preaeration development. The first of these was an extended study entitled "Chemical Coagulation of Sewage". This work was published (1936-1941) as a series of fifteen Journal Series Papers of the New Jersey Agricultural Experiment Station, Department of Water and Sewage Research, Rutgers University. The result was a conclusive review of coagulants and factors affecting their economical use. The work was done on strong and somewhat stale domestic sewages from nearby communities.

1. Plain flocculation versus chemical coagulation

Three of these papers are of particular interest here. In the first (46), experimental laboratory results were reported on flocculation alone, without chemicals, both by mechanical stirring and by air diffusion. It was found that air and mechanical flocculation produced like removals in the absence of coagulants. With light coagulant dosage, paddle flocculation gave better results; with a heavy coagulant feed, air flocculation gave much better removals. It was theorized by the authors that the more turbulent air flocculation, even though only vigorous enough to prevent settling, acted to break up the weak floc structure of inadequate coagulation but had no such effect on the strong floc produced by adequate chemical dosage. In any case, the use of coagulants gave substantially better removals than plain flocculation. It was also found that mechanical stirring, even in the presence of coagulants, could be too vigorous, resulting in poorer ultimate removals with excessive paddle speeds.

The few experimental trials with plain flocculation gave surprising removals, although always less than with coagulants. The flocculation period was found to be of major importance. With 20 mg/l ferric chloride as the coagulant, for example, roughly 80 percent of the ultimate turbidity removal was accomplished with only 15 min flocculation, and more than 90 percent in 30 min. With plain flocculation, on the other hand, only 15 percent of the ultimate turbidity removal was accomplished in the first 15 min, 50 percent in 30 min and 90 percent in 60 min. Flocculation for more than 90 min did not appear to accomplish additional removals in either case.

2. Use of various gases

In the second of the Rutgers papers (47), results of laboratory-scale flocculation experiments conducted in atmospheres of air, ozone, oxygen, carbon dioxide and nitrogen were reported. The test runs were designed with gentle mixing for periods of 0.5, 2 and 5 hrs, followed by 2 hr settling in each case. Where 2 hr settling alone resulted in a 56 percent SS removal, 0.5 hr preflocculation without coagulants boosted this to an average removal of 74 percent, 2 hr preflocculation produced 77 percent removal and 5 hr preflocculation produced 82 percent removal. The gas employed made little difference.

In a subsequent run the above gases, along with hydrogen peroxide, were diffused directly into sewage samples in what was apparently the first considered observation of the effect of preaeration on primary settling. The authors indicate that this method gave even greater improvement over settling alone than did plain stirring, but the data presented are simply not adequate to support any sort of conclusion.

For the 0.5 hr diffusion period, the gas used seemed to have no effect; after 2 hrs and even more so after 5 hrs of such treatment, the oxygen-bearing gases did produce substantially greater BOD reductions, although this did not seem to be true with turbidity removal.

The conclusion was that flocculation without chemicals could improve primary BOD and SS removals considerably and that both mechanical stirring and air diffusion could accomplish this goal. The theory was that the effect was physical, involving primarily the suspended and semi-colloidal fractions of sewage.

3. Plain flocculation on plant scale

In the third Rutgers paper (14), plant-scale observations at New Brunswick, New Jersey are reported. Here it was possible to split the raw sewage flow so that half received mechanical flocculation, then settling, while the other half received equal settling without pre-treatment. Very early in the study it was found necessary to operate the paddle-type flocculators at a peripheral speed of at least 1.8 fps; lower rates permitted considerable settling in the flocculation tanks.

Mechanical flocculation was found to produce improved settling at this plant, even with a fairly weak sewage containing 125 to 175 mg/l SS. Settling time provided in normal plant operation was 4 hrs; flocculation periods ranged from 20 mins to $4\frac{1}{2}$ hrs. SS removal by settling alone averaged about 65 percent. It was indicated that the effect of 20 min flocculation was quite nominal, but that the same pre-treatment for a period of $4\frac{1}{2}$ hrs was capable of boosting primary SS removal to around 80 percent or higher. Supporting data were lacking, however.

In commenting on the earlier (46) laboratory finding that most of the benefit to be derived from flocculation was achieved in the first hour, and that nothing was to be gained beyond 90 min flocculation, Gehm theorized (14, p. 1074) that the difference here was "due to the character of the weak New Brunswick sewage, which contains no great amount of pseudo-colloidal matter which readily flocs out."

C. Early Gas Diffusion Studies

In their work with gas diffusion, Rudolfs and Gehm acknowledged earlier published research by Williams (57) at University College in London. Williams studied the flocculation effect of bubbling oxygen, air, nitrogen, and hydrogen through 1 l samples of sewage for 6 hr and 24 hr periods. Based on the reduction in organic carbon and total nitrogen, no material difference was found between gases in the 6 hr period, but oxygen and air gave best results over the longer period.

A test was also conducted to compare oxygen diffusion with mechanical stirring by glass paddles under air-free conditions. Analysis after a 2 hr period indicated a close comparison between the two, although oxygen diffusion produced twice the reduction in strength at 6 hrs. "These results might be taken as evidence for the view that the main effect of gas bubbling on sewage liquors is of a physical nature." (57, p. 357)

D. Dorr Company* Research on Mechanical Flocculation

1. Laboratory studies at nine plants

During the mid-1930's, the Dorr Company embarked on a study of mechanical flocculation which culminated in the only comparative plant-scale work on pre-flocculation reported (13). The study began with the

*Now Dorr-Oliver, Incorporated

development of a rectangular jar and paddle arrangement designed for flocculation of sewage samples of about 1 gal in volume. After it was learned that transfer to another container would break up much of the delicate floc formed, the sewage was allowed to settle in the flocculation test jar. Samples of supernatant were drawn off for analysis after specified settling periods.

With equipment and technique refined, the Dorr Company research staff visited nine sewage treatment plants from Connecticut to the Dakotas. At each plant, laboratory runs were made with combinations of flocculation times of 0, 10, 20, and 30 mins and settling times of 20, 40, and 60 mins. SS results for these runs are reproduced in table 1.

These data are unusually valuable because they represent a number of plants, a wide range of sewage characteristics, yet were derived by what surely must have been closely standardized technique. With this in mind, simple statistical analyses were carried out on data from seven of the nine plants. Coney Island, New York, was dropped because of incomplete data and Aurora, Illinois, was omitted because the raw waste was unusually low in strength, 89 mg/l SS, and appeared to yield erratic results.

Data for the remaining plants were analyzed to determine the effect of varying flocculation time, table 2a, and the effect of varying settling time following flocculation, table 2b. At all seven plants, flocculation served to improve primary settling efficiencies in these laboratory-scale tests; in fact, the longer the flocculation, the better the results. Flocculation was also found to accelerate settling; that

Table 1.* Laboratory results on mechanical flocculation of raw sewage, Dorr Company data

Plant	Coney Island, N. Y.		Norwalk, Conn.		Lima, Ohio		Aurora, Ill.		Cedar Rapids, Ia.	
SS	mg/l	% Rem.	mg/l	% Rem.	mg/l	% Rem.	mg/l	% Rem.	mg/l	% Rem.
Influent	186		197		171		89		343	
Effluent										
After No Floc.										
Settled 20 min			125	36.5	157	8.2	47	47.2	195	43.1
Settled 40 min			74	62.4	89	48.0	45	44.5	180	47.6
Settled 60 min	183	1.6	68	65.4	81	52.7	38	57.3	144	58.0
After 10 min Floc.										
Settled 20 min			125	36.5	76	55.6	37	48.4	193	43.7
Settled 40 min			72	63.5	68	60.3	19	78.6	190	44.6
Settled 60 min	102	43.9	60	69.5	62	63.7	52	41.5	103	70.0
After 20 min Floc.										
Settled 20 min			63	68.0	69	59.6	51	42.7	137	60.7
Settled 40 min			63	68.0	58	66.0	67	24.7	84	75.6
Settled 60 min	89	50.5	48	75.6	48	72.0	25	72.0	94	72.6
After 30 min Floc.										
Settled 20 min			55	72.6	58	66.0	38	57.3	104	69.7
Settled 40 min			47	76.2	52	69.5	22	75.3	74	78.5
Settled 60 min	69	63.0	45	77.0	52	69.5	27	69.7	73	78.8

*This table reproduced from Fischer and Hillman (13, p. 285).

Table 1. (Continued)

Plant	Ortonville, Minn.		Morehead, Minn.		Fargo, N. Dak.		Sioux Falls, S. Dak.	
SS	mg/1	% Rem.	mg/1	% Rem.	mg/1	% Rem.	mg/1	% Rem.
Influent	896		666		1302		688	
Effluent								
After No Floc.								
Settled 20 min	260	71.0	332	50.1	664	49.0	212	69.3
Settled 40 min	248	72.4	220	66.9	518	60.2	171	75.2
Settled 60 min	210	76.6	186	72.0	432	66.8	159	76.9
After 10 min Floc.								
Settled 20 min	138	84.5	112	83.1	502	61.5	119	82.7
Settled 40 min	176	80.4	90	86.5	324	75.1	108	84.4
Settled 60 min	165	81.5	97	85.4	316	75.8	95	86.3
After 20 min Floc.								
Settled 20 min	192	78.5	98	85.3	340	73.9	110	84.0
Settled 40 min	140	84.4	93	86.2	326	75.0	102	85.3
Settled 60 min	177	80.2	82	87.6	296	77.2	81	88.3
After 30 min Floc.								
Settled 20 min	167	81.3	86	87.0	317	75.6	83	88.0
Settled 40 min	142	84.0	81	87.8	293	77.4	75	89.2
Settled 60 min	135	85.0	77	88.4	280	78.4	60	91.2

Table 2a. Effect of varying flocculation time on SS removals, Dorr Company data

Flocculation time min	SS removal percent		Degree of improvement by flocculation	
	Settling* only	Flocculation and settling	Percentage points	Increased removal, percent
10	64	74	10	16
20	64	78	14	22
30	64	81	17	27

*Average of data for 40 min and 60 min settling.

Table 2b. Effect of varying settling time, following flocculation, on SS removal, Dorr Company data

Settling time min	SS removal percent		Degree of improvement by flocculation*	
	Settling only	Flocculation* and settling	Percentage points	Increased removal, percent
20	52	76	24	46
40	63	79	16	25
60	65	80	15	23

*Average of data for 20 min and 30 min flocculation.

is, considerably more of the ultimately settleable material was removed in early time increments than was the case with plain settling. Correspondingly, the margin of improvement shown by flocculation was greatest with short settling time and decreased with extended settling.

The opinion is widely held that any sort of preflocculation will pay dividends with stronger wastes, but that the effort is wasted on weak or normal sewage. These data do not indicate this. With three plants receiving fairly normal wastes, 171-343 mg/l SS, three receiving strong wastes, 666-896 mg/l SS, and one receiving unusually strong waste, 1302 mg/l SS, there was no significant difference between them in the degree of improvement effected by plain flocculation. The strong wastes showed excellent results with flocculation but, as a group, they also responded well to plain settling alone.

This interesting facet did appear; the better the job done by plain settling only on a particular waste, the less was accomplished by preflocculation, either in terms of added percentage points removal or increased degree of removal in percent. To repeat, these are laboratory-scale data representing wastes from seven different plants, but they are considered unusually helpful to this study.

2. Plant-scale work at Cedar Rapids, Iowa (19)

The Dorr Company next concentrated its efforts on a packinghouse waste pretreatment plant at Cedar Rapids, Iowa. This plant, located on the municipal treatment plant site, comprised a holding tank and rate-of-flow controller to balance the waste load, a flash mixer, a flocculator and two 50 ft diam clarifiers. After pretreatment, the packinghouse wastes were added to the raw domestic sewage and the combined waste was given complete treatment in the municipal trickling filter plant.

The pretreatment plant was arranged in the study to split the flow to subject one portion to flocculation and settling while the remainder received settling alone. This plant was designed for chemical treatment, and a series of experimental runs was conducted with varying chemical dosage and without any chemicals. These runs were made prior to the work on mechanical flocculation described herein. Their conclusions were that, while chemical treatment was highly effective, the results by plain flocculation were sufficiently attractive to render the cost of chemicals unjustified at that time.

For the runs made by the Dorr Company, the flow path was altered so that the flow through the pretreatment plant consisted of 10 to 20 percent packinghouse waste and the balance domestic sewage. Generally, these tests were intended to provide a comparison of plain settling with mechanical preflocculation followed by settling. Sampling and analyses were performed daily.

In the first set of runs extending about a month, the proportion of flow was varied in an attempt to achieve equal SS removal by each of the two bays of the test plant. The few data in the published report indicate that substantially more flow was handled through the flocculation-settling combination than through settling alone, with equal or better results achieved following preflocculation.

The second set of runs, also lasting about one month, provided a direct comparison at equal flows of 1.0 to 1.6 mgd through each side of the test plant. For the runs discussed here, settling detention time varied from 1.5 to 2.5 hrs, flocculation time from 30 to 54 mins, and the BOD and SS strengths of the waste treated were both in the general range of 250 to 400 mg/l.

SS removal averaged 54 percent by plain settling and 64 percent by flocculation and settling; a gain of 10 percentage points or roughly an 18 percent improvement. BOD removals were poor and erratic, but were improved from about 13 percent to about 20 percent by flocculation. The general relationships indicated in tables 2a and 2b from laboratory-scale work were also evident here. The improvement by flocculation was no greater for the stronger wastes, but was, if anything, less because these wastes responded so well to plain settling, leaving less room for improvement by any means. As expected, increased flocculation time added to the degree of improvement.

The importance of careful handling of the delicate floc formed was emphasized by laboratory settling tests on the flocculator effluent before and after passing through a 14 in siphon feed line to the clarifier. Above flow velocities of 1.5 fps through this line, settling results were diminished by 10 to 12 mg/l SS. This represented a loss of 3 to 4 percentage points in primary removals because of floc destruction.

3. Plant-scale work at Ypsilanti, Michigan

The next phase of the Dorr Company study was conducted at the Ypsilanti, Michigan municipal sewage treatment plant. A full-scale experimental unit was constructed embodying a circular, concentric mechanical flocculation chamber within a conventional clarifier structure. The unit was designed for continuous passage of the flocculated flow downward into the settling area, then outward and upward toward the overflow weirs. This unit was the pilot model of the Clariflocculator which is now manufactured and sold by the Dorr Company.

The test unit was operated in parallel with a conventional 40 ft diam clarifier which provided settling detention time equal to the combined flocculation and settling time in the Clariflocculator. The physical arrangement of the Clariflocculator provided one-sixth of total capacity for flocculation and the remaining five-sixths for settling.

One series of runs compared the operation of the two units for a period of fourteen weeks at detention times of from 2 to $2\frac{1}{2}$ hrs. The SS strength of the wastes treated during this period averaged about 150 mg/l. With equal flow to each unit, SS removals averaged 45 percent by plain settling and 57 percent in the Clariflocculator, an improvement of 21 percent. Runs with the flow unequally divided indicated that the Clariflocculator could handle 35 percent more flow than the conventional clarifier of equal overall dimensions with equal or slightly better results.

Additional plant runs with detention times of more than 5 hrs indicated average SS removals of 66 percent by plain settling and 75 percent in the Clariflocculator. It will be noted that, while the Clariflocculator still maintained an advantage of 9 percentage points, its relative margin of improvement had dropped to 14 percent as a consequence of the high SS removal by plain settling at very long detention periods.

The need for a means of obtaining an undisturbed sample of the flow leaving the flocculation chamber led to the development of a special sampling device. With it, a wide-mouth bottle was lowered to the sampling point upside down, then filled slowly by the controlled release of its entrapped air through a tube to the surface. It was then turned upright by a swivel arrangement and brought to the surface with the floc undisturbed. This ingenious device was the model for a somewhat similar sampler used in studies reported in later sections herein.

E. European Experiences with Flocculation

Mechanical flocculation also received its share of attention abroad. In 1942 preflocculation was said to have given excellent results in England (24). In 1943, laboratory experience in Germany with mechanical flocculation and operating data for a plant installation were reported (39). The floc formed were very fragile and tended to break up at flow velocities exceeding 0.4 mps (1.31 fps). In 1949, seven years of full-scale operation at Wolverhampton, England were reported in which a Clari-flocculator showed 20 percent better removals than a conventional clarifier of the same overall dimensions (25).

F. Aerochlorination and Grease Removal

Although not directly involved in the development of preaeration, some work supported by the Chlorine Institute, Inc., New York City, is of interest. In 1937 plant-scale use of $1\frac{1}{2}$ mg/l of chlorine was reported at Woonsocket, Rhode Island to improve grease removal by as little as 6 mins of very vigorous preaeration (12). The amount of air was not measured. The raw sewage had a high grease content from wool scouring and other textile process steps. Grease removal by aerochlorination, as this procedure was termed, was more than double the removal by aeration alone, in terms of wet weight of drained scum per mil gal of sewage. The scum was removed manually from baffled areas of the preaeration tank.

Further work was done in 1938 on domestic sewage at Baltimore (28) in 800 gal pilot test tanks. Runs were made with 5, 10 and 15 min preaeration periods, using 0 to 10 mg/l of chlorine. Generally, with 1 or 2 mg/l of chlorine, 2 to 3 times as much grease was removed on a dry weight basis as compared with preaeration only. With more chlorine, removals were as much as 5 to 7 times higher. Air use was essentially constant at about 0.07 cu ft/gal per 5 min increment of time.

In 1938, plant-scale aerochlorination results at Lancaster, Pennsylvania were reported (56). With only 3 to 4 mins of preaeration at air flows of 0.10-0.14 cu ft/gal, removals of drained wet scum ranged from 3 to 4 lb/mil gal with aeration only or chlorination only, to 12 or 14 lb/mil gal by aeration coupled with 2.0 mg/1 of chlorine. Without either air or chlorine, grease removal was about 1½ lb/mil gal. At this plant, grease was skimmed from the effluent end of primary clarifiers providing an average of 1.75 hrs settling time.

At Lancaster, grease removal was also determined by the analysis of raw sewage and primary effluent for the mg/1 grease content. The results indicated roughly 40 percent grease removal following either no pretreatment or chlorination only, 50 to 60 percent removal with preaeration only, and 80 percent removal following preaeration supplemented by chlorination. The degree of improvement over plain settling was still very significant but only about half as great in these terms as for wet scum weight; apparently a bulking factor of some sort was taking effect. SS and BOD data were lacking for the aerochlorination studies described.

In 1942 some laboratory-scale work was conducted at Rutgers on grease removal from a number of sewages (15). Comparative test series indicated an appreciable improvement over plain settling by preaeration and settling combined. The amount of air was not given. When mechanical flocculation was substituted for preaeration in identical runs, the removal of grease was considerably better. This comparison is open to question because the sewage samples used in the second series had much higher grease content.

Suspended solids results were also given for both test series. With raw sewage strength averaging 300-325 mg/l of SS, plain settling achieved 60 to 65 percent SS removal. The 1 hr pretreatment improved these removals by 6 to 9 percentage points, or by 10 to 14 percent.

Plant-scale data were also reported from South River, New Jersey, where a 30 min preaeration period was provided ahead of primary settling (15). The rate of air supply was 0.02 cu ft/gal. Scum was removed at the outlet end of the preaeration tank. Weekday scum removals over a 30 day period of plain preaeration averaged 39 lb/mil gal on a dry grease basis. With the aid of 8 mg/l of chlorine, this removal increased to 55 lb/mil gal over a 40 day period of similar dry-weather flows.

Following these successes, detailed studies which yielded consistent results -- consistently contradictory to previous experience -- were made at three military camp treatment plants. At plant A, experimental work on 20 gal samples showed no improvement in grease removal over plain settling by 10 min preaeration at a rate of 0.15 cu ft/gal, even when reinforced by 5 to 10 mg/l of chlorine. At plant B, mechanical preaeration for 15 minutes resulted in only the slightest improvement over plain settling, and that only with fairly high grease content. No improvement in SS removal resulted from pretreatment at either plant.

At plant C, a vacuum flotation unit was observed. Here, the combination of flotation and settling performed slightly better than either settling or flotation alone. The addition of chlorine up to 25 mg/l, however, was of no help whatever. All that was actually accomplished by pretreatment at these installations was the removal of some grease, especially slugs and chunks, and some reduction of the grease-removal burden on primary settling. The conclusion was (11, p. 313) that "plain settling is one of the most effective means of removing grease from raw sewage". Whether or not these results are peculiar to military camp wastes can only be conjectured.

G. Literature Review by Heukelekian

A superb literature review was published in 1941 covering forty-five references pertaining to the theory and practice of mechanical flocculation at that time (22). This review included this thoughtful definition (22, p. 507):

> Flocculation -- Coalescence of finely divided suspended matter in sewage in the absence of biologically active slime, primarily under the influence of physical forces. The term is applied to short period mechanical or air diffusion processes where biological action is at a minimum.

With this review, an era ended. The studies at Rutgers and the work done by the Dorr Company stand out in this period of development.

Two things are important at this juncture. First, the chemical treatment process, partly through high cost and scarcity of chemicals during World War II, partly through the success of plain flocculation without chemicals, fell into disuse and has not since been a factor in waste treatment, with a few exceptions. Second, the first sparse groundwork had been laid for the use of air as an alternate method of physical stirring or mixing to induce flocculation.

Such was the development of the preaeration process.

III. THE PREAERATION PROCESS; PRESENT STATUS

Preaeration, defined earlier as the air flocculation pretreatment of plain raw sewage for a period of 30 to 45 mins, remains controversial in the waste treatment field. Mechanical preflocculation has been accepted, as evidenced by the hundreds of Clariflocculators now in service, and by the similar treatment units developed by competing manufacturers. That preaeration provides several or all of such extra dividends as gas scrubbing for odor control, grease flotation, grit washing, the addition of DO and improved treatability is acknowledged. However, there is wide disagreement on whether or not preaeration also improves primary settling removals as seems quite well established for mechanical preflocculation.

The popularity of preaeration in view of the meager data on either development or operating experience with this process was noted in 1949 (58). At that time comparative plant-scale studies were urged to furnish information on the merits of preaeration. At least thirty plants were using this process according to a survey conducted in 1945 by the United States Public Health Service.

A. Recent Published Reports

1. Favorable operating experiences

In 1945 preaeration was claimed to have materially improved primary settling efficiency in several California plants (9). Actual operating results were not given. It was further stated that the DO content of the sewage was improved prior to secondary treatment and that odors formerly released by trickling filter spray were eliminated.

In 1948, probably the most enthusiastic discussion of preaeration was published (54). After an experience with a hopelessly septic, untreatable sewage which defied all efforts to establish an activated sludge culture, only to become completely amenable to treatment following 1.5 hrs of preaeration, the investigator became interested in a number of other preaeration installations. He cited 8 plants employing this pretreatment step; all with success and some with phenomenal primary removals. No direct comparative operating data with and without preaeration were available, however.

This interesting postulate was offered (54, p. 116):

> Preaeration is practiced only when sufficient oxygen has been added to the sewage to permit a sample, properly taken, to show at least 0.2 ppm DO upon standing in a DO bottle (submerged) for 60 minutes. This means that it may take 10 minutes or 10 hours of aeration depending upon how much oxygenation and agitation the waste requires, the intensity and efficiency of oxygenation, and other factors.

This concept is certainly an interesting one, although modern design practice would lean toward a modified or short-period activated sludge process if more than nominal air capacity and detention time were required.

Improved treatability resulting from preaeration was noted. This term defies specific definition except that it connotes a waste flow leaving primary treatment with biochemical characteristics which make it more amenable to aerobic secondary treatment, in contrast with a waste which might be septic upon arrival at the plant and would normally become still more so during its slow passage through the primary settling step.

As still further dividends to be received, odor and corrosion control by removal of hydrogen sulfide, and the addition of dissolved oxygen were cited (54, p. 118):

> It is time that considerable effort should be made to collect comprehensive data showing the effect of preaeration on primary settling and secondary treatment operation. There is little or none of this data available in the literature or in plant operation reports.

In recent operating experience with preaeration of a stale, septic raw sewage at Wichita, Kansas, the normal preaeration period is 1.8 hrs, with air provided at the rate of 0.17 cu ft/gal (29). A considerable accumulation of grease balls occurs, and is removed from the preaeration basin periodically.

A comparison of primary removal with and without pretreatment was made by operating without preaeration for the five week period April 16 through May 20, 1958. Raw sewage strength and volume for the period and for several weeks both before and after it were quite uniform; raw sewage strength averaged 225 mg/l of BOD and 260 mg/l of SS. BOD removal averaged only 34 percent during the 5 week test period, while averaging 40 percent for three weeks prior and 42 percent for the two weeks following. SS removal averaged 64 percent without preaeration and 67 and 69 percent, respectively, before and after the test period. Sampling and analyses were performed at approximately three day intervals throughout the ten weeks.

2. Unfavorable operating experiences

It is human to report successes and to mention failures only in passing, if at all. This is true regarding preaeration, although several writers tell of disappointing applications.

The Denver, Colorado plant was built with both a preaeration tank designed for a 15 min detention period with a 0.02 cu ft/gal air rate, and a preflocculation tank of identical capacity (21). It was reported (21, p. 1121) that in 1939 and 1940,

> several experimental efforts were made to employ the preaeration, flocculation and sedimentation units so as to secure improved results over those normally expected from primary treatment. These experiments were to no avail, however, . . .

until primary sludge was returned in a sort of activated pretreatment with a very high aeration rate, which was strikingly successful.

In 1946, the preaeration facilities at the Ley Creek plant, Syracuse, N. Y., which were designed to provide approximately 15 min detention with an aeration rate of 0.20 cu ft/gal were reported (32). Experimental work there in the mid-1940's indicated that preaeration was beneficial in leveling off shock loads which tended to upset the activated sludge secondary treatment, but that it did not produce any change in the DO of the raw sewage (34). Some solids which normally settled out by primary settling were broken up so that they did not settle following preaeration. Preaeration at the Ley Creek plant, while intended for grease removal, could not justify its cost in terms of the other benefits claimed for this process.

In 1957, a five year struggle with what must surely rank among the most difficult wastes in this country or anywhere else was reported (4). The sewage reached the Hyperion Plant in Los Angeles, after many miles and hours of flow, high in temperature, high in sulfides, high in immediate oxygen demand and strongly anaerobic. Here, the 0.5 hr preaeration provided ahead of primary settling has been of no value other than for corrosion control.

Valuable work has been done at Hyperion in correlating ORP with sewage characteristics and particularly with microbiological conditions in terms of aerobic, facultative, or anaerobic environment. The degradation of the sewage has progressed so far that, while preaeration raises it to a facultative state, the sewage is again anaerobic on leaving primary settling. Despite a large air supply rate to the aeration tanks, the sewage does not even remain aerobic during final settling. This would appear to be a valuable new insight into the concept of treatability. The authors' comments (4, p. 778) on preaeration follow:

> A careful and detailed study of the preaeration system was made as it affected the primary sedimentation effluent. One-half of the sewage flow was given preaeration, while the other half received none. The primary sedimentation effluent from the preaerated half was the same as the effluent received from the unpreaerated half insofar as BOD, suspended solids, settleable solids, and chlorine demand reduction were concerned. Preaeration did reduce the sulfide content somewhat, thus reducing the oxygen demand of the aerator influent a small degree. It is doubtful that the slight reduction in oxygen demand justifies the use of air for preaeration, and it is felt that the air would be used to better purposes in the aerators. In contradiction to this, preaeration has apparently been of value in other installations.
>
> It is believed that the concept of treatability can help to explain why preaeration works at some locations and fails at others. Preaeration of sewage which is only slightly septic and has a low to moderate oxygen demand might keep the sewage in the high facultative to low aerobic range,

preventing anaerobic breakdown, and thus decrease the load on the aerators. However, where the septicity is far advanced and the oxygen demand is high, a short preaeration period cannot meet the oxygen demand, and the little good accomplished by preaeration is quickly nullified by further anaerobic degradation in the primary sedimentation tanks.

This paper was supplemented by a thoughtful discussion of the ORP studies carried out prior to the design of preaeration facilities for the then-proposed San Jose, California, sewage treatment plant (40). This work indicated that preaeration of the waste in question for 30 mins would result in an ORP that was relatively stable, apparently in the facultative-aerobic range. Preaeration for 60 mins appeared to yield an even more stable ORP level which would not decline for several hours.

B. Operating Data Survey by Roe

The first solid survey and discussion of preaeration practice was reported in 1951 (44). The survey failed to turn up any set of comparative plant data in which plain settling was operated in parallel with the combination of preaeration and settling. Nevertheless, it was a valuable review of the art as then known.

Design and operating questionnaires were sent to 91 plants known to employ preaeration in early 1950. Of these, 38 replied. Data were also obtained from 13 other plants not originally listed, indicating that the total number of plants using preaeration then was more than 100. It was noted that the term preaeration meant something different to almost everyone using it, from detention times of a few minutes to several hours, and with negligible to noteworthy aeration rates. The term has even been used for long aeration of primary effluent prior to secondary treatment, as at Syracuse, N. Y. and Decatur, Ill., but this is more properly referred to as plain aeration and is not to be confused with preaeration of raw sewage, to which this study is limited.

It was stated that diffusion of air into the average raw sewage almost immediately sweeps out entrained CO_2 and H_2S, enhances the separation of grease and reverses the ORP of the sewage from negative to positive, thereby stopping reduction and markedly improving the treatability of the sewage. Further, continued aeration beyond the first few moments serves to provide additional available DO and to stimulate agglomeration and flocculation of solids.

The survey returns were unanimous in reporting odor control and at least some attack on septicity by preaeration. Of the 51 plants reporting, 18 indicated that grease removal was considered in the design of the preaeration unit, but only six gave data on actual grease removal. In addition, five plants reported grit removal by preaeration, although only three were so designed.

With regard to the important factor of improved primary treatment, SS removal by preaeration and 2 hr settling fell in the pattern shown in figure 1. Although some of the data were inconsistent, the relationships shown were generally valid. Settling time was a factor, in that data from plants with shorter or longer settling times fell below or above the curves, respectively. BOD removal reported by 26 of the plants showed no correlation whatever. It can only be theorized that, if preaeration does result in improved SS removal, the BOD must also be affected, simply by

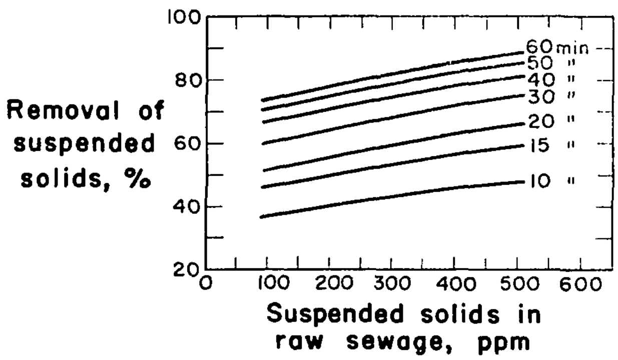


Fig. 1.* SS removal by preaeration and 2 hr settling

*This figure reproduced from (44, p. 130)

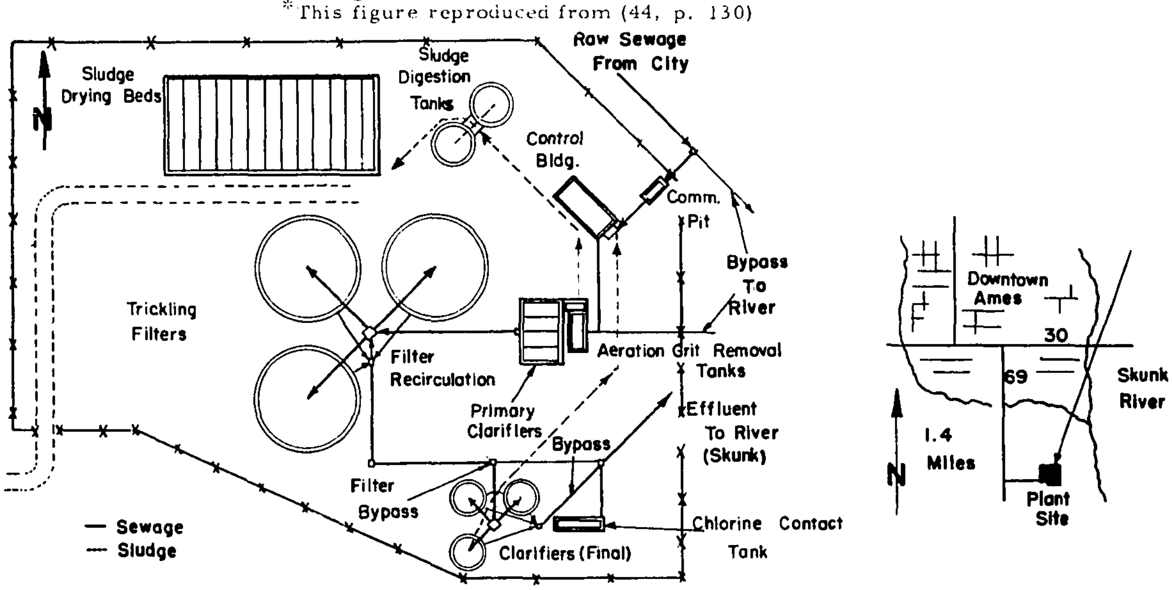


Fig. 2. Location and schematic plan of Ames, Iowa, sewage treatment plant

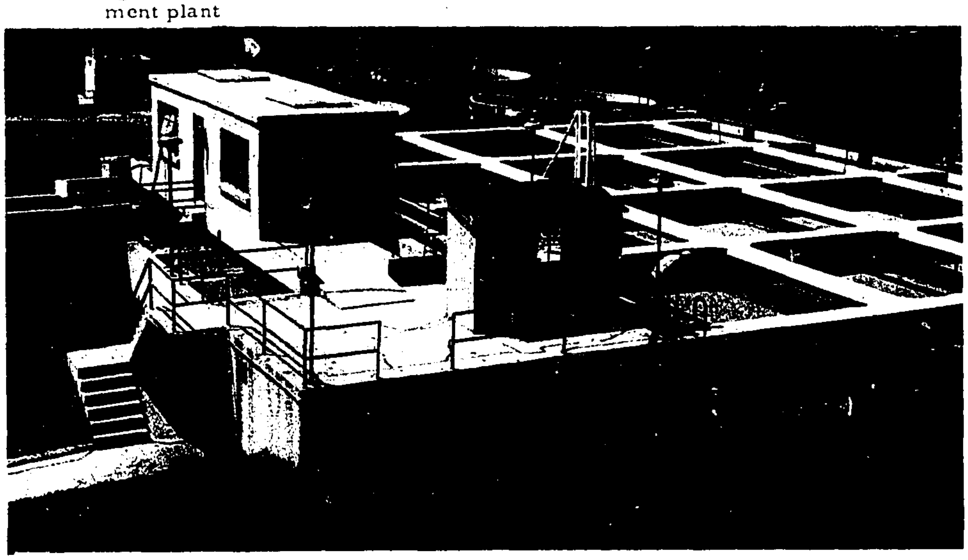


Fig. 3. View of preaeration and primary settling tanks, Ames plant

reduction of the amount of solids in the primary effluent. The survey data on air supply was also reviewed and the rate of aeration was found to vary in generally a straight-line relationship from about 0.07 cu ft/gal for 10 min preaeration to about 0.16 cu ft/gal for 60 min preaeration. These rates represent, of course, actual practice rather than design criteria.

In discussing this paper (33, p. 139),

> It is quite evident that most older preaeration units are using far more air than shown on Mr. Roe's curve of air requirements.

The discussion also suggested that air flocculation and preaeration are two different processes, each with its definite purpose.

The survey concluded that preaeration can be relied on to improve treatment results at almost any sewage treatment plant (44, p. 137):

> . . . preaeration accomplishes more in actual performance or analytical results per total dollar expended than any other step of primary or secondary treatment. Beyond this is the intangible benefit of conditioning sewage so that it is more treatable, thereby increasing the success and/or decreasing the cost of secondary oxidation processes.

C. Preaeration Equipment

1. Separate units

The physical arrangement of air diffusion devices in some of the proprietary preaeration systems then available were given (44). The primary function of the air is mechanical, to properly agitate and maintain all solids in suspension. The air required varies from 1 cfm/lin ft for a small, shallow channel to as high as 4 cfm/lin ft of conventional aeration tank comparable in design to that employed in the activated sludge process. Additional equipment systems available for preaeration and several industrial waste applications are discussed in a later paper (43).

2. Integral with settling

In an analysis of sedimentation, preflocculation was identified as an adjunct to primary settling which has been badly neglected (6). Designers take special care to transport the flow from the flocculation step to the settling tank without exceeding velocities of 2 to 3 fps, depending on pipe size, above which the rather delicate floc may be broken up. Better still, consideration should be given to the design of a flocculation area as an integral part of the settling basin.

Christy (7) also discussed this latter design feature, listing among its advantages the elimination of floc break-up by avoiding the travel which may cause it, and the excellent distribution of both flow and solids to the settling area. A further major advantage is the simple, positive removal of solids which tend to settle or float in the preaeration area by the same removal equipment serving the settling area proper. Several designs of such integral preaeration-settling tanks by the Link-Belt Company are described.

D. Other Applications

1. Grit removal

The use of a very short preaeration period solely for grit removal is one of the most interesting recent developments. This involves the application of a spiral-flow aeration tank modified to include a grit trough directly beneath air diffusers mounted along one wall, at the point where troublesome grit accumulations sometimes occur without encouragement.

The design basis and construction of the first such major installation by the Chicago Pump Company at Columbus, Ohio, consisting of four tanks with 13 ft operating depth has been given (18). One feature of the "Aer-Degritter" is that the velocity of the roll can be controlled by the amount of air used, and is essentially unaffected by flow variations in the incoming raw sewage.

At Columbus, surprisingly clean, fine grit was removed with a detention time of only 1.50 to 1.75 mins (27). Air use was a fraction of 0.01 cu ft/gal. Generally, an aeration rate of 3 cfm/lin ft of conventional aeration tank length is said to develop a spiral flow velocity of 2.0 fps which is more than sufficient to keep organic particles in suspension. It has also been determined that 0.2 mm sand particles will be rolled along the tank bottom by a velocity of 0.75 fps. Between these velocities, then, is the proper operating range of this process, which does not fall within the definition of preaeration but is one of its most interesting offshoots.

2. Long-period preaeration

Test runs at Philadelphia, in which 1.5 hrs of preaeration was used, gave improved results over plain settling (27). The plant-scale operation at Fort McClellan, Ala., in which a still longer preaeration period preceded primary settling was also discussed. The latter plant was apparently handling a difficult waste, since without preaeration primary removals were only about 30 percent and even the trickling filter effluent lacked DO. After the plant had adjusted to preaeration averaging 2.4 hrs on a 24 hr basis, primary removals of BOD and SS increased to 66 percent and 78 percent, respectively. Further, the trickling filter effluent contained 3.0 mg/l of DO although the preaeration tank itself was normally without DO except in the early morning hours of minimum flow.

Possibly the most interesting facet of the study at Fort McClellan was that, while the first month of preaeration was discouraging, the second month showed improvement, and treatment during succeeding months produced excellent results. Even without benefit of return sludge or effluent, a culture was apparently established in the preaeration tank which took some time to develop, then accomplished a good deal more than physical flocculation. This is unquestionably a worthwhile treatment variation, but it is beyond the scope of preaeration as defined for this study.

E. Design Criteria

1. "Ten-State" design standards

A manual of design criteria for sewer systems and sewage treatment has been issued by the ten states* of the Upper Mississippi River and Great Lakes area. This is a joint product of their combined sanitary engineering staffs (53). The standards are not rigid, but are rather intended as general guideposts to good practice under normal circumstances.

The complete statement on preaeration in the Ten-States Manual is reproduced as Appendix A. However, the following excerpt (53, p. 20) indicates the loose framework by which design engineers and regulatory agencies alike are guided in the use of preaeration. Note that nothing is said concerning the effect on primary treatment.

> C. Detention Period:
>
> (1) Coagulation: When air or mechanical agitation with chemicals is used to coagulate or flocculate the sewage, the detention period should be about 30 minutes but never less than 20 minutes at the design flow.
>
> (2) BOD Reduction: When air or mechanical agitation (either with or without the use of chemicals) is for the additional purpose of obtaining increased reduction in BOD, the detention period should be at least 45 minutes at design flow.

2. State Health Department policies

To determine current policies on preaeration in the Ten-State area

*Illinois, Indiana, Iowa, Michigan, Minnesota, Missouri, New York, Ohio, Pennsylvania and Wisconsin.

and for five additional midland states, all fifteen Departments were canvassed. Survey forms are included as Appendix B.

Of the fifteen states, fourteen indicated a willingness to give some credit for increased primary removal following preaeration under certain conditions. The restrictions and qualifications outlined varied from mild to rigorous; no two states approached this with the same viewpoint.

The allowance for increased BOD removal beyond a suggested 35 percent for plain settling varied from 0 to 15 additional percentage points, or a range of 35 to 50 percent BOD removal by preaeration followed by settling. Several replies suggested that 60 percent SS removal was a reasonable allowance for preaeration combined with primary settling. Detailed survey results are presented in table 3.

F. Summary of Present Status

Preaeration as discussed here comprises air flocculation of raw sewage for 30 to 45 minutes prior to primary settling; no recirculated or biologically active material is added to the process; the effect is primarily physical.

First, it is apparent that one can take any position whatever on preaeration and find company. Just as operating reports conflict, comments and opinions of those in the field vary from enthusiasm to disillusionment. Some believe that preaeration will achieve results with strong sewage and/or with certain industrial wastes, but not with sewage of normal strength. For others preaeration is wasted effort under any conditions.

Some consulting engineering firms include preaeration in all or most

Table 3. Summary of current design review policies on preaeration

State	1 Credit Given for Improved Primary Removals?	2 If so, up to what BOD Removal?	3 Comments
Colorado	Yes.	Considered individually; in some cases allowance as high as 45 percent is made.	Municipalities must comply with established effluent standards. Review of plans does not prejudice our enforcement of compliance with such standards.
Illinois	Prefer not to give credit unless pilot tests indicate improvement by preaeration.	In some instances 10 to 15 percent credit may be given (up to about 45 percent BOD and perhaps 50 or 60 percent SS), where strong sewage is treated, particularly if industrial wastes amenable to treatment are included.	Preaeration normally is not regarded as an efficient use of air when compared with activated sludge. We believe aerated grit chamber produces beneficial effects. Flexibility in preaeration is encouraged to include return of sludge and variable air supply.
Indiana	If preaeration period is at least one hour, some credit may be given.	Approximately 5 percent.	Preaeration is encouraged, particularly if the sewage is strong or stale.
Iowa	Yes for BOD; no increase for SS	40-45 percent removal of BOD.	Little operating data available.
Kansas	Yes, for BOD removal, if minimum aeration period of 30 min is provided.	40-45 percent BOD removal is allowed in primary sedimentation with preaeration; only 30 percent BOD removal is allowed for primary sedimentation alone.	
Michigan	Yes.	Up to 15 percent.	No operating experience as yet in Michigan plants for this type of equipment.
Minnesota	Yes.	Allowance of about 40 percent BOD removal and 60 percent SS removal would be considered reasonable for this treatment where fresh domestic sewage is treated followed by conventional sedimentation tanks without appreciable amounts of industrial wastes.	No units of this type are in operation in this State. Several places use air for flocculation or for mixing with shorter periods of detention. The BOD removal allowed for praeration is tied in with the design of the primary settling units and depends largely upon the individual case, including the physical and chemical characteristics of the sewage to be treated.
Missouri	Yes.	10 percent.	We have allowed 10 percent in several cases where strong industrial organic wastes were concerned. None of these plants have operated as yet; therefore, we have no factual data on actual operating experience.
Nebraska	Yes.	None beyond the 35-45 percent BOD removal. If preaeration is not given, doubt that 35-45 percent is attained.	Cannot give blanket approval; some sewage is benefited by preaeration, on the other hand, don't believe fresh sewage from a small community would be helped at all. Overall, believe that a 10 percent (additional) BOD removal could be reasonably expected in sewage that needs preaeration for either grease removal or for overcoming septic conditions caused by long sewer line.
New York	Yes.	For 45-min preaeration plus primary settling, we would consider 45 percent BOD removal and 60 percent SS removal.	The percentage removal of either BOD or SS depends greatly on the character of the raw sewage. We have very little actual data in New York to substantiate these figures.
Ohio	Yes.	For 45-min preaeration plus primary settling, we give credit for 45 percent BOD removal and 60 percent SS removal.	Actual operating results at Ohio plants substantiate these allowances. Several plants produce 50 percent BOD removal and 65 to 70 percent SS removal. The percentage removal of either BOD or SS depends greatly on the character of the raw sewage.
Oklahoma	No.		No preaeration units per your definition in state.
Pennsylvania	Yes.	10 to 15 percent.	We like to have at least 1-hr preaeration and provision for return of sludge or chemical additions in case needed. Less allowance, if sewage is weak.
Texas	No.		Usually preaeration is proposed for grease removal, incorporated with grit removal, or to help a septic sewage problem. Should a city be able to provide laboratory proof that preaeration of their sewage will give additional BOD reduction, then credit would be given in their design of plant additions.
Wisconsin	Yes.	For domestic sewage about 15 percent. For industrial wastes 5 to 10 percent, depending upon the character of the waste.	The reduction for domestic sewage and industrial waste is calculated separately. The total overall allowable reduction is determined from such separate calculations.

of their plant designs; others ignore it. Equipment firms report that interest in preaeration is high but that for no other treatment step does design practice vary so widely.

Second, the statement can still be repeated today with authority (23, p. 1159):

> preaeration facilities are provided in many modern plants, yet there are almost no data on the true advantages and limitations of the process.

Over 200 plants in this country are now equipped for this process; yet comparative, parallel operating data with and without preaeration are still lacking.

To this end, the present study is inclined; not with the thought that all questions regarding preaeration will be resolved, but rather that at least one intensive, continuing evaluation of the process be carried out and its findings made available.

IV. PURPOSE AND SCOPE OF THIS STUDY

The objectives of this study are as follows:

a. to evaluate the effect of preaeration on primary settling of sewage;

b. to evaluate the factors, such as the amount of air and detention time, which affect the results achieved; and

c. to evaluate the economics of preaeration as a sewage treatment process.

It has been noted that published reports of parallel plant operation with and without preaeration are lacking, although such data seem vital to any conclusions about the process. The Ames plant is particularly well suited for such a study since its primary stage could easily be arranged for operation as two parallel units with the flow split accurately between them.

It was recognized that the results of this study would be conclusive not for the process, but only for the Ames plant. However, Ames sewage is strictly domestic and of normal strength, providing a challenge to a process which some feel has little or no application with such a waste.

During preliminary work at the Ames plant and particularly after discussing preaeration informally with many others at technical meetings, the emphasis of this study shifted rather heavily to the first of the objectives listed above. The real challenge was the opportunity to establish whether the preaeration process actually benefited primary settling or whether this was only imagined by its enthusiastic supporters.

A very brief chronology of the course of this study is outlined as follows:

1. Spring and summer, 1956	Preliminary work; physical plant changes; detention tests.
2. September, 1956 through May, 1957	Plant-scale operating runs, continuously without inspection or changes, using original downdraft aerator.
3. July, 1957	Plant-scale runs with provision for overcoming grit problem; downdraft aerators used alternately.
4. August and September, 1957	Laboratory-scale experimental runs with various aeration and settling times.
5. Fall, 1957 through Winter, 1958	Work on developing and refining laboratory methods; installation and testing of air equipment.
6. March-August, 1958	Plant-scale operating runs at Ames plant, with varied air rates and varied equipment, for one-week intervals; intensive sampling and laboratory analyses.
7. August, 1958	Plant-scale operating runs at Grinnell, Des Moines, and Cedar Rapids, Iowa; with total flow receiving preaeration -- no parallel comparison possible.
8. November and December, 1958	Oxidation-reduction potential data gathered; cleanup of experimental work.
9. January through March, 1959	Plant-scale operating runs at Ames, with equal split flow but unequal settling times.

Discussion of this study and of the results achieved follow generally the chronological outline above.

V. PREPARATORY WORK AT THE AMES PLANT

A. Plant Arrangement and Operation

This plant is somewhat unusual in that it serves both the City of Ames and Iowa State University. Fixed and operating costs of the plant are shared by each in proportion to sewage volume contributed.

The flow diagram and normal operating procedures have been given in a Progress Report (5) covering the first period of this study. The flow diagram is included here in figure 2, and a general view of the preaeration and primary settling area is presented in figure 3.

The Ames plant is designed to treat an average flow of 3.0 mgd. The present population served is approximately 27,000. Actual volume load on the plant during the period of this project ranged generally from 2.1 to 2.8 mgd. Sewage strength in terms of both BOD and SS normally averages from 200 to 250 mg/l; overall plant removals range from 75 to 80 percent in winter to about 90 percent in summer. Float tests indicate that daytime sewage flow from the main business district reaches the plant in roughly 1¼ hrs, while the flow time from the University is approximately 2 hrs.

1. Treatment units

Major plant units, in order of flow through the plant, include:

a. A comminutor pit, with Parshall flume to measure total raw sewage flow.

b. A control building, housing raw sewage pumps, engine-generator unit, shop, office and laboratory.

c. Two aeration-grit removal tanks for preaeration and grit removal.

d. Four rectangular primary clarifiers for settleable solids removal.

e. Three standard-rate trickling filters to provide biological treatment.

f. Three circular final clarifiers to provide final settleable solids removal.

g. A chlorine-contact tank, which has never been in service, for chlorination of final plant effluent.

h. Two sludge digestion tanks, operated as a two-stage system; primary unit equipped with a floating cover, secondary digester with a gas holder.

i. Sludge drying beds for dewatering digested sludge in the summer.

j. A sludge lagoon to which sludge or supernatant can be diverted.

Unit sizes and design details for the units are summarized in table 4. Hydraulic loadings are shown for both a 2.0 mgd rate, which approximates dry-weather 24 hr average flow, and a 3.0 mgd rate which is in the range of normal daytime flow. It is apparent that loads on this plant are still moderate.

Table 4. Unit sizes and loadings; Ames sewage treatment plant

	Design factors		
	Per unit	At 2.0 mgd flow rate	At 3.0 mgd flow rate
Aeration-grit removal tanks (2 units) (preaeration)			
Surface area, sq ft (22.5 ft x 22.5 ft)	506		
Maximum depth (hopper), ft	20.0		
Volume, gal	40,000		
Detention, hr*		0.96	0.64
Surface overflow rate, gpd/sq ft		1,970	2,960
Primary clarifiers (4 units)			
Surface area, sq ft (59 ft x 20 ft)	1,180		
Average depth, ft	6.35		
Volume, gal	56,000		
Weir length, ft	154		
Detention, hr*		2.69	1.80
Surface overflow rate, gpd/sq ft		424	636
Weir overflow rate, gpd/lin ft		3,250	4,870
Trickling filters (3 units)			
Diameter, ft	135		
Average depth, ft	8.0		
Surface area, acre	0.33		
Volume, acre-ft	2.63		
Hydraulic loading, mgd/acre		2.03	3.05
BOD loading, lb/day/acre-ft (assuming 125 mg/l applied)		264	—
Final clarifiers (3 units)			
Surface area, sq ft (41 ft diam)	1,320		
Average depth, ft	8.25		
Volume, gal	81,500		
Weir length, ft	129		
Detention, hr*		2.94	1.96
Surface overflow rate, gpd/sq ft		505	757
Weir overflow rate, gpd/lin ft		5,170	7,750

*Theoretical displacement time.

2. Normal operation

Flow from both City and University reaches the plant through a common trunk sewer and passes through comminutors and a Parshall flume before reaching the raw sewage wet well. From here, it is pumped to the aeration-grit removal units at rates of approximately 2 or 3 mgd, the rate being governed by automatic programming of the float-controlled pumps to keep pace with the incoming flow.

At the aeration-grit removal units, the flow is split for parallel treatment in the two tanks, then recombined in a channel which provides distribution to the four rectangular primary clarifiers. Primary settling effluent is again recombined for flow through an underground conduit to the three trickling filters.

Filter effluent is transported similarly to the three final clarifiers, the effluent from which is discharged to the Skunk River approximately one-half mile from the plant. Gravity flow is provided throughout these treatment steps and to the river. All units are normally in operation except that one filter is taken out of service during the winter months to provide increased flow to the other two to minimize icing problems.

Primary settled sludge is drawn off at 6 hr intervals to a sludge well where it is concentrated for approximately 5 hrs. Shortly before the next scheduled draw, the heavy sludge is pumped from the bottom of this sludge well to the first-stage digester. The thin top liquor or supernatant remaining in the sludge well is drained back to the raw sewage wet well to be recycled through the plant. The sludge well is then momentarily empty before receiving the next raw sludge withdrawal.

Final settled sludge is recycled to the raw sewage wet well through a return line throttled by a float-actuated butterfly valve at the wet well. It is thus resettled in the primary clarifiers. The float is set to return little or no flow during the daytime, but opens late at night to bring back all final sludge along with a good deal of final effluent. In this way, a minimum night flow rate of 1.5 mgd is maintained, compared to the 0.8-0.9 mgd normal night flow arriving at the plant.

All raw sludge goes to the first-stage digester, whose contents are actively recirculated through an external heat exchanger. Regular sludge transfers are made to the second-stage digester which functions as a storage and concentration unit. Supernatant is normally returned to the raw sewage wet well for recycling through the plant. Digested sludge is drawn off to sand beds for drying when weather conditions permit. A large lagoon is also available as a safety valve for the sludge handling phase of the treatment process.

It is important to point out both now and later that the sludge and supernatant returns described were diverted from the raw sewage wet well when preaeration plant runs were in progress, since the object of the study was to evaluate preaeration in its simplest terms, i.e., without return material of any kind. Supernatant from the digesters and from the sludge well could be and were readily diverted to the sludge lagoon. Final settled sludge was held in the final clarifiers for varying periods, usually without serious detrimental treatment effects. During winter operation without return night flow from the finals, it was found advisable to operate only one trickling filter.

B. Preliminary Studies

The preliminary work pertaining to plant load patterns and actual detention times will be mentioned in passing because it was a necessary forerunner of an intelligent sampling program.

1. Sewage strength and volume patterns

In February and March of 1954, an intensive series of sampling runs was conducted jointly by sanitary engineering graduate students and plant personnel. Raw sewage samples were picked up hourly for one 24 hr period in each of five consecutive weeks and analyzed for BOD, SS and TS. The results, as shown in figures 4 and 5, were surprisingly consistent patterns of incoming raw sewage strength, representing essentially dry-weather flow.

Later, an analysis was made of flow records for the full year 1956, a severely dry year following several years of less than normal precipitation. Here again, the patterns were considered to be unusually free of abnormal influences. The 24 hr flow patterns were generally unaffected by the season (figure 6), or by a comparison of maximum and minimum weeks with the mean for the entire year (figure 7).

2. Primary clarifier detention tests

In a series of tests in February and March of 1956 actual detention times in the primary clarifiers were determined. This work has been reported in detail (1). Rock salt, fluorescein dye and $rubidium^{86}$ were

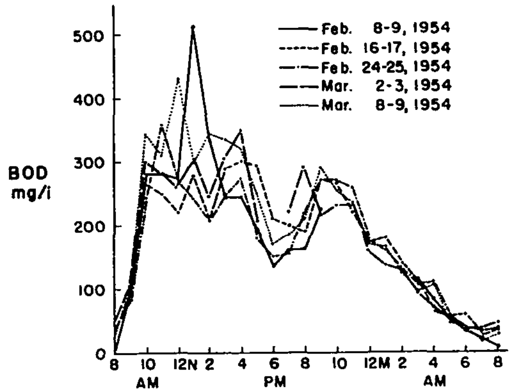


Fig. 4. Typical 24 hr patterns of Ames raw sewage BOD strength

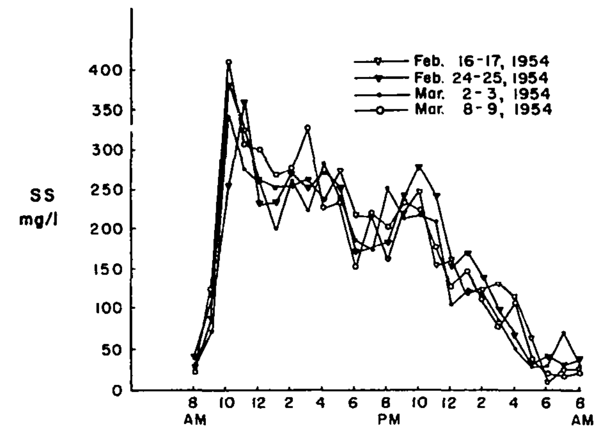


Fig. 5. Typical 24 hr patterns of Ames raw sewage SS strength

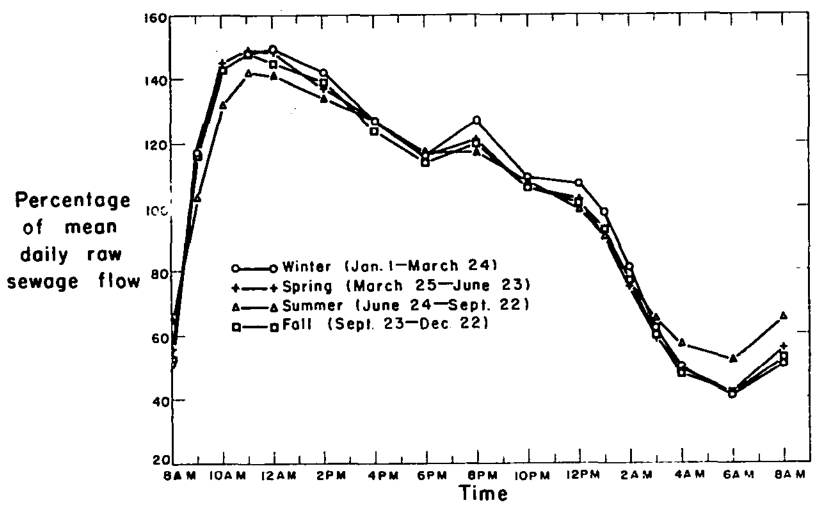


Fig. 6. Seasonal variations in typical 24 hr Ames flow pattern

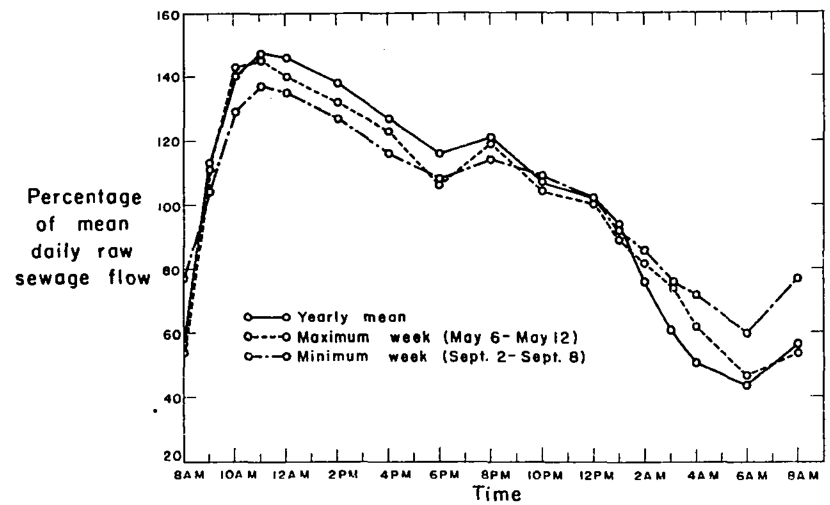


Fig. 7. Ranges in typical 24 hr Ames flow pattern

used as tracers in separate runs. The rock salt proved to be the least reliable, consistently giving mean detention times longer than calculated theoretical displacement times. It was theorized that the problem here was at least partly one of density currents. Additional difficulties were encountered because of the variable chloride content of the raw sewage.

The rubidium^{86} radiotracer gave mean detention times considerably less than theoretical, but not particularly consistent. Even with its advantages, the use of a radiotracer involves counting equipment and safety precautions which salt or dyes do not.

Fluorescein dye, the old reliable, not only proved the simplest to use and to read photometrically in the laboratory, but gave the most consistent and reasonable results. Actual mean detention times, as determined with fluorescein, ranged from 70 to 87 percent of theoretical in five of the six runs. In the sixth, this figure was 99 percent, which is not realistic. A value of 80 percent is probably most reasonable as a basis for sampling (figure 8). Calculated dye recovery averaged about 75 percent. Results of the detention tests on the primary clarifiers are summarized in table 5.

Table 5. Dye detention tests on Ames primary clarifiers

Test no.*	Clarifier no.	Flow rate mgd	Theoretical det. time mins	Actual mean det. time mins	Mean as percent of theoretical
2	1	.75	108	76	70
	2	.55	147	121	82
	3	.45	179	130	72
	4	.85	95	77	81
3	4	.85	95	94	99
4	4	.85	95	83	87

*Designation in (1)

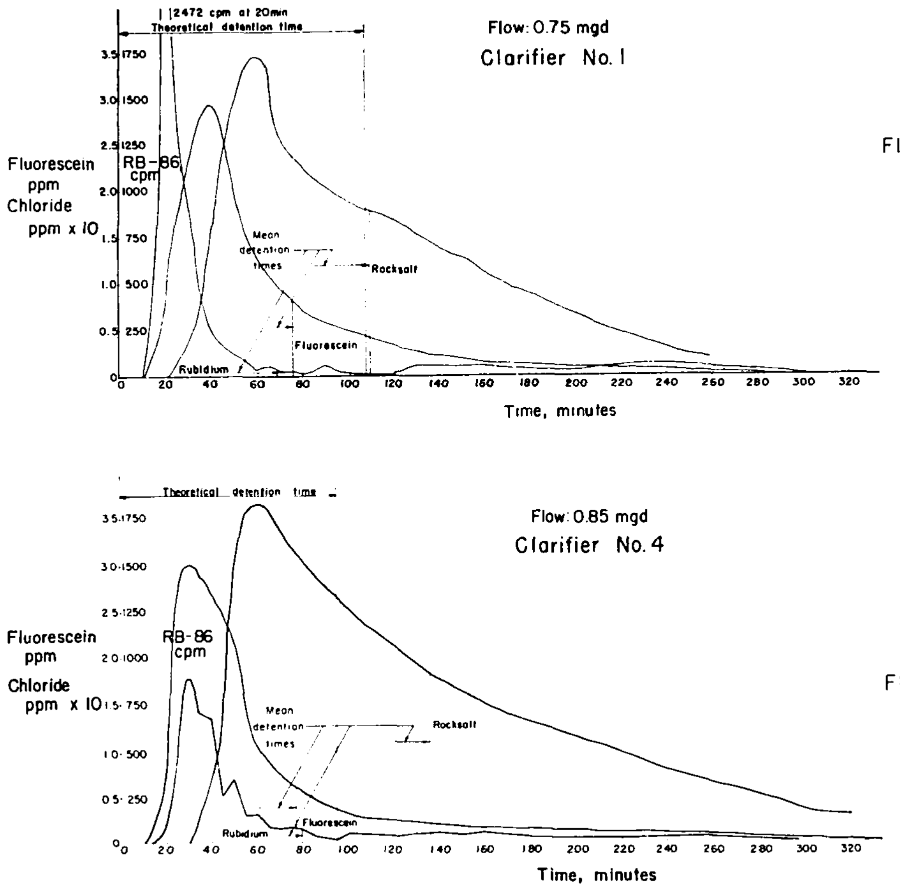


Fig. 8. Detention time tests on Ames primary clarifiers

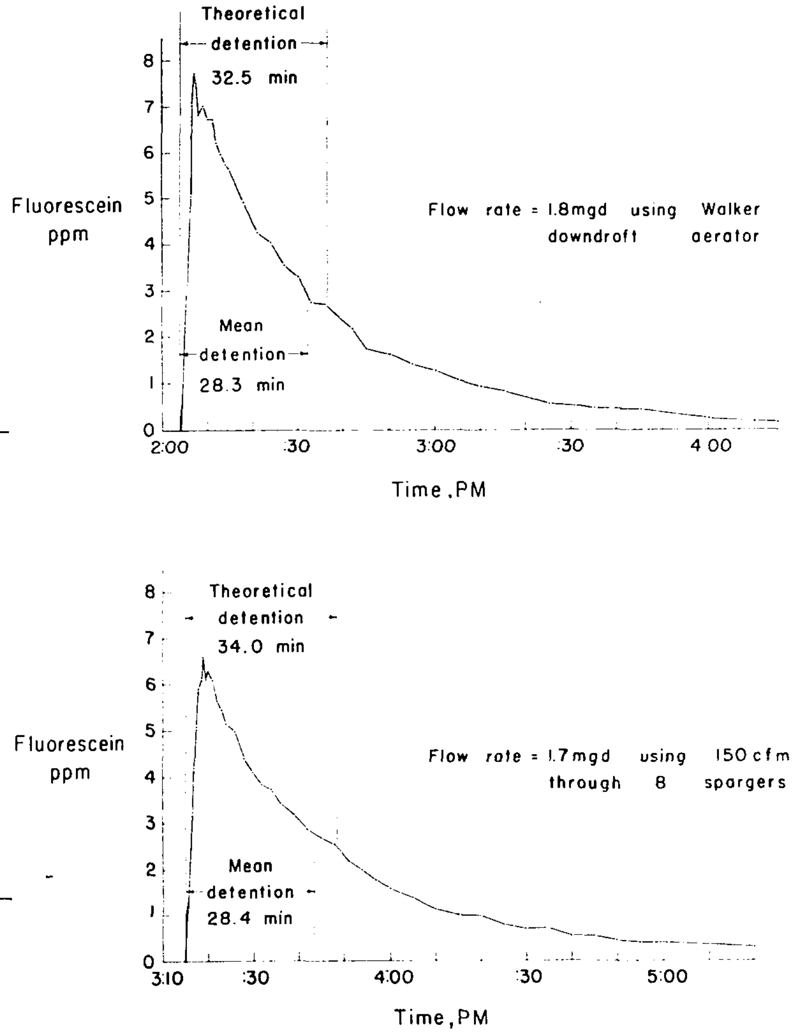


Fig. 9. Detention time tests on Ames south preaeration tank

3. Preaeration detention tests

Following these detailed experimental runs with several tracers, fluorescein was used to check actual detention times in the south preaeration tank. The procedure used, calibration curves, and typical data are in Appendix C.

Two runs were made at similar flow rates, one with the original Walker downdraft aerator in operation, the other with a blower delivering 150 cfm of free air to the tank through 8 spargers. In each of the two runs, 500 g of fluorescein dye was used; sewage flow through the unit was maintained at a steady rate for the 2½ hours required to complete the test runs.

The results were quite consistent, with actual mean detention times determined to be 90 and 88 percent of theoretical in the two tests (figure 9 and table 6). This work provided a valid basis for planned sampling of the preaeration step.

Table 6. Dye detention tests on Ames south preaeration tank

Test no.	Flow rate mgd	Theoretical det. time mins	Actual mean det. time mins	Mean as percent of theoretical
1	1.8	32	29	90
2	1.7	34	30	88

C. Plant Changes and Adjustments

1. Downdraft aerator rebuilt

The original equipment in each of the two preaeration units was a downdraft mechanical aerator driven by a 5 hp motor. This equipment was furnished by Walker Process Equipment Co. of Aurora, Illinois. This unit (figure 10) is basically a submerged impeller operating within the upper end of a submerged downdraft tube which extends to a depth of fourteen ft below the sewage surface. Air-inlet tubes extend from the atmosphere at the pump base to an area of reduced pressure just above the impeller. Rotation of the impeller forces a mixture of liquid and air downward through the draft tube, recirculating the tank contents. The air contributes only slightly to the energy involved in recirculation.

The amount of air added to the sewage by the downdraft aerator is small by usual aeration standards, but its distribution is excellent and it appears at the surface in very fine bubbles. According to the manufacturer, the rate of air intake is at a maximum when the impeller clears the air tube openings by only $\frac{1}{4}$ in and when the submergence is such that gentle vortexing occurs on the surface. Either more or less submergence will hinder the natural air suction. Later attempts to force additional air down the draft tube with this unit in operation succeeded only in badly hampering pumping action and in producing an area of boils as the air rose in masses next to the downdraft tube.

Grit removal, which at first was not considered to be a serious problem at the Ames plant, is accomplished in a somewhat unique manner. The

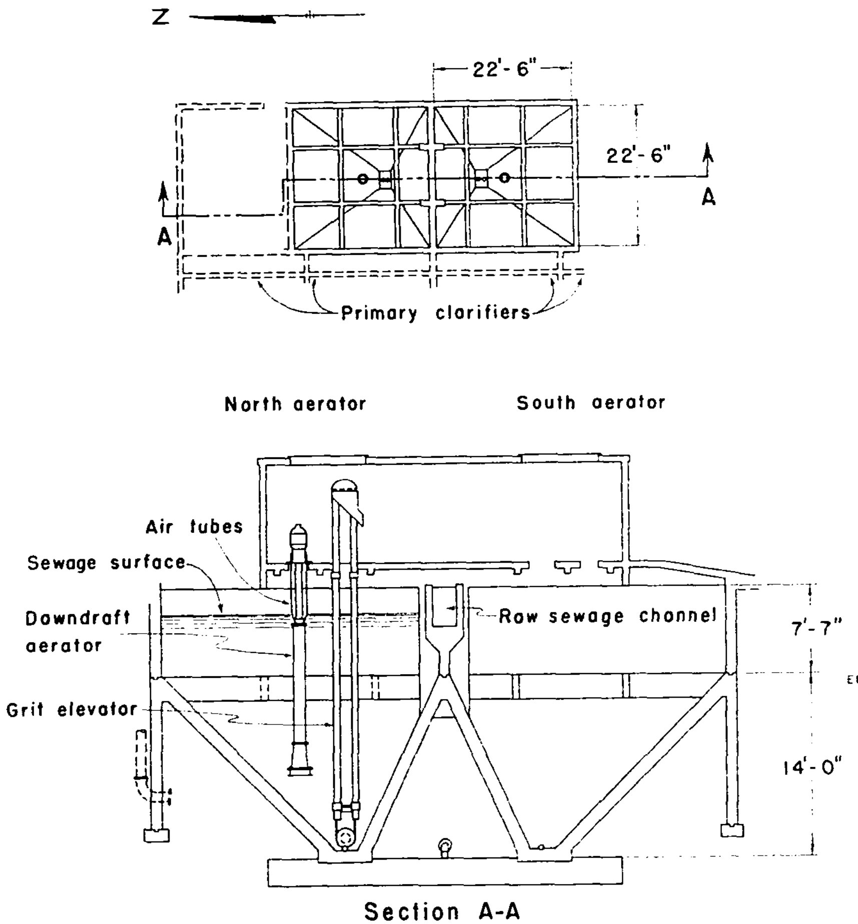


Fig. 10. Plan and section through Ames preaeration tanks

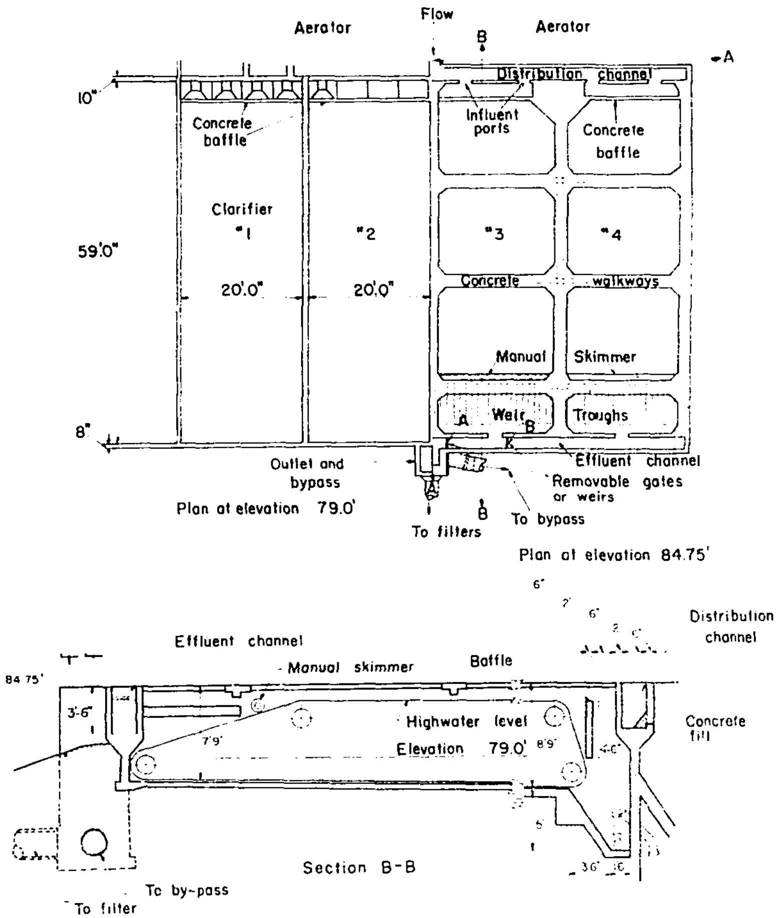


Fig. 11. Plan and section through Ames primary clarifiers

grit-washing action of preaeration appears to be successful in permitting the grit to settle below the draft tube while keeping the organic material in suspension. However, an accumulation of about one week's grit is necessary, lodged along the lower slopes of the hopper bottom, before any grit is removed by the bucket elevator. This inactive grit storage does not appear to increase appreciably, nor is there any evidence that it affects the characteristics of the sewage passing through these units. In normal operation, grit is elevated out of the unit once each operation shift.

In the summer of 1956, the mechanical aerator in the south preaeration tank was rebuilt and provided with a new impeller and larger air lines. The air lines were collected in a manifold at the pump base so that the total draft could be measured. Automatic reversing switch-gear was installed to reverse the impeller direction for two mins every hr. This served to clear the impeller blades and shaft of rags and other materials which would otherwise seriously reduce the effectiveness of the aerator within a few hours after manual reversing.

2. Primary effluent weirs

The rectangular primary clarifiers are equipped with endless chain collector flights which move surface scum to the effluent end of the tank and bring settled sludge back to hoppers beneath the influent ports (figure 11). All primaries are served by a common influent channel. Two large inlet ports are provided from the influent channel into each clarifier. A solid baffle wall 2 ft from the inlet openings is provided to dissipate the entrance energy and direct the flow downward.

It has always been apparent that the distribution of flow to the four primary clarifiers was unequal with all influent ports fully open. As a first step in better flow control, Cippoletti weirs were so installed in the primary effluent channel that the flow through each clarifier could be measured, either directly or by difference.

Considerable effort was expended in the calibration of these weirs as actually installed. Raw sewage was pumped directly to the primary clarifiers, bypassing the preaeration units, at a known rate until the flow over the weirs was stabilized and readings were taken. Then, the procedure was repeated at a different flow rate. Pumping rates were established by measurements at the Parshall flume, which had earlier been calibrated volumetrically.

Early runs following calibration of the Cippolletti weirs indicated just how poor the natural distribution was. The data in table 7 are typical of these runs.

Table 7. Unbalanced flow to primary clarifiers before adjustment

Clarifier	Flow, mgd
1	.75
2	.55
3	.45
4	.85
Total	2.60 mgd

3. Primary influent shear gates

The unequal flow distribution was overcome with the easily adjustable shear gate (figure 12). With a moderate amount of juggling, the flow to all four clarifiers was balanced and remained so for months without attention.

This arrangement did create a new problem of scum accumulation in the influent channel. The scum problem was in turn solved by cutting an opening in the two shear gates at the extreme ends of the channel and providing a sliding panel which was raised once a shift for a few moments to flush out the scum without tampering with the shear gate setting.

4. Installation of flow splitters

For comparison of primary treatment with and without preaeration, splitters were installed in the channel serving the preaeration tanks. The splitters (figure 13) consisted of redwood boards held in place by angle iron frames. A galvanized metal rudder on the inlet side split the nappe of the raw sewage flow entering this channel. This arrangement provided a high degree of accuracy in dividing the flow between the two primary treatment bays. Once set, this device was found to be stable for months without attention. The splitters and supplementary gates were used to direct half the flow through the south preaeration tank, then to primaries 3 and 4, while the other half passed directly to primaries 1 and 2. This was the basis of operation for many months of plant-scale comparative runs.

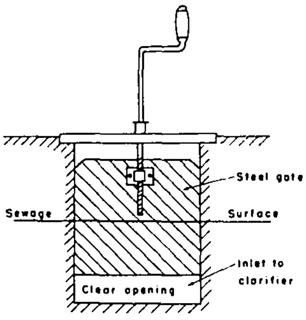


Fig. 12. Adjustable shear gate for flow control to primary clarifiers

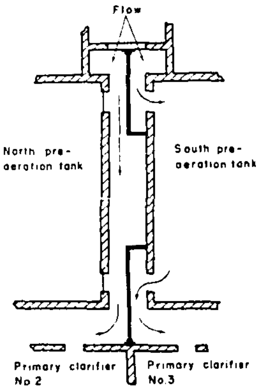


Fig. 13. Arrangement of splitters and shear gates during full-scale plant runs

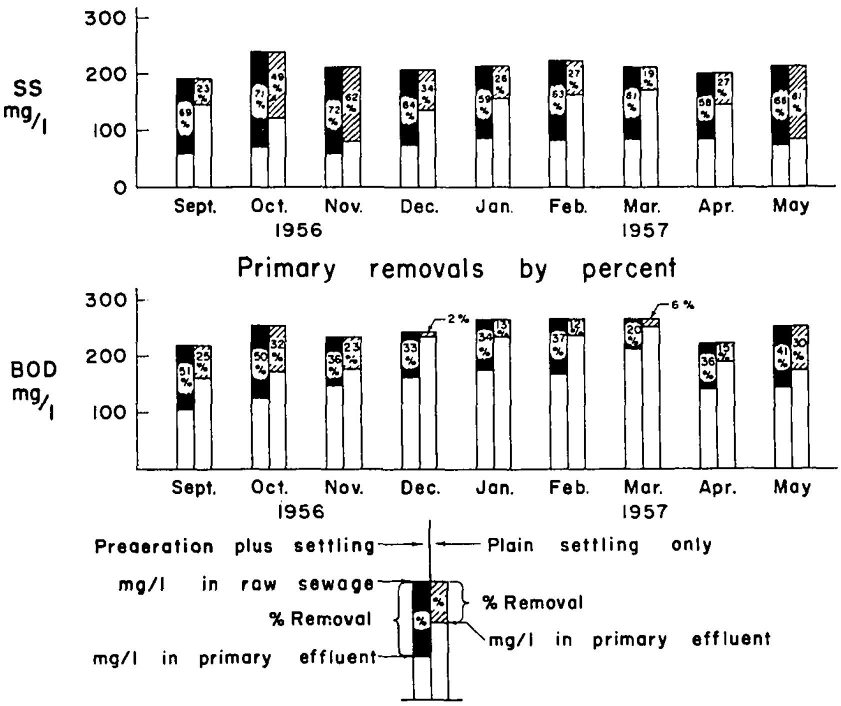


Fig. 14. Monthly average primary removals for full-scale Ames plant runs, 1956-1957

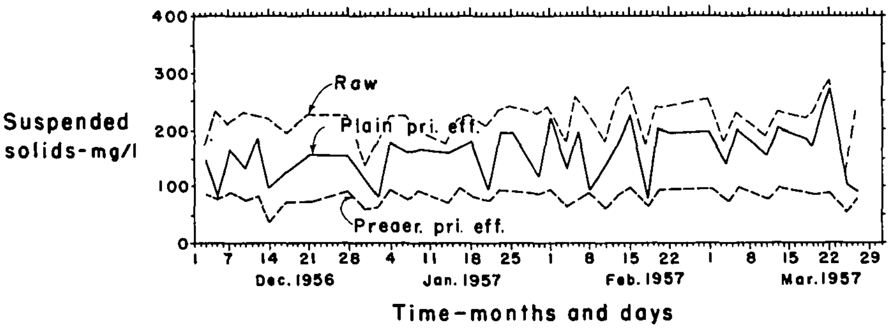


Fig. 15. Individual raw and primary effluent SS values, 1956-1957 Ames plant runs

D. Trial Plant Runs; July 26-September 14, 1956

1. Operating sequence

The object of these runs was to initiate a sampling and laboratory program even though all mechanical and plant changes were not yet complete.

Sampling for 24 hr composites was begun in late July with all raw sewage flow receiving rather brief preaeration in the north unit, then passing to the four primary clarifiers. The original Walker downdraft aerator was in place in the north unit. Two trickling filters and all three final clarifiers were in operation. Plant operation was normal, including nightly return of final effluent to the raw sewage wet well to supplement low night flows. Total plant flow during this trial run ranged from 1.42 to 1.87 mgd, averaging 1.70 mgd.

Sewage samples were collected at 2 hr intervals at the following five locations: plant influent, preaeration effluent, primary effluent, filter effluent and final effluent. The 2 hr sampling frequency was based on earlier studies which indicated that a shorter interval would accomplish little in terms of more representative sampling.

On August 15 the flow was split with half passing through the north preaeration tank as before then to the north pair of primary clarifiers, while the other half of the raw sewage flow bypassed preaeration and went directly to the south pair of primaries. This required sampling of two primary effluents, bringing total sampling points to six. At this time, procedures were initiated to hold sludge and supernatant out of the influent to the primary stage of treatment.

2. Plant results

Operating results for this period are summarized in table 8. Taken at face value, these results present preaeration in a very attractive light. Primary BOD and SS removals appear to be about 15 percentage points better than for plain settling, and overall plant efficiency was better when the

Table 8. Results of trial Ames plant runs, July-September 1956

Sampling point	Trial run A*		Trial run B**	
	BOD, mg/l	SS, mg/l	BOD, mg/l	SS, mg/l
Raw, or preaeration influent	269	169	185	168
Preaeration effluent	273	190	197	202
Preaerated primary effluent	193	67	106	57
Plain primary effluent	--	--	132	84
Filter effluent	(insufficient data)		58	38
Final effluent	40	18	38	27
Removals	BOD percent	SS percent	BOD percent	SS percent
Preaerated primary treatment	28	60	43	66
Plain primary treatment	--	--	29	50
Complete treatment	85	89	80	84

*All flow through north preaeration unit; 9 composites, July 26-August 15.

**Half of flow through north preaeration unit and half direct to primaries; 7 composites, August 23-September 14.

total sewage flow was preaerated. However, these results do not deserve this interpretation; the north aerator and several primaries were down for repairs during this period, and the division of flow was known to be approximate.

Most important, the local corn cannery was operating from August 2nd through 19th, with an organic load alone roughly equal to that from the rest of the community. This load at times came in heavy slugs, in addition to a fairly steady discharge from normal processing. The effect of this waste was apparent for a number of days after canning operations had ceased. Finally, these trial runs covered too short a period to be conclusive, as later runs were to emphasize all too sharply. Complete laboratory data for these trial runs are included in a project Progress Report (5).

On September 17, 1956, an accurate split of the raw sewage flow was achieved; the north preaeration unit was shut down and half the flow was bypassed directly to the north primaries. The south preaeration unit was placed in operation, with this half of the flow then channeled to the south primaries. This was the operating procedure until late spring of 1957, for the duration of the first intensive series of full-scale plant runs.

VI. FULL-SCALE PLANT RUNS, SEPTEMBER 1956-MAY 1957

A. Plant Arrangement and Operation

For the next eight months, the incoming raw sewage flow was split accurately, half passing through the south preaeration unit and the south primary clarifiers, the other half bypassing directly to the north primary clarifiers without pretreatment. The Walker downdraft aerator was in continuous operation in the south preaeration unit with hourly reversing.

All primary clarifiers were in continuous service. Settled sludge was drawn off at 6 hr intervals. An important change during this period was the diversion of supernatant from the raw sludge well to the sludge lagoon, preventing any return of material from this point to the incoming raw sewage. Another change in plant operation involved the trickling filters; on September 28 one filter was taken out of service, leaving only filter No. 1 in operation. On November 23 it became necessary to remove this filter from service and substitute filter No. 2, which then operated alone until late in May, 1957. Only one filter was used throughout the winter season to avoid icing problems aggravated by lack of the usual nighttime return flow from the final clarifiers.

The final clarifiers were all in service. However, final sludge return was restricted to the hours of 1 AM to 4 AM on nights following the conclusion of 24 hr sampling. During these hours, the preaeration control gates were shifted to bypass the south preaeration tank entirely, diverting all raw sewage together with its supplement of final settled sludge directly to the primary clarifiers. In this way final sludge was re-settled in the primaries as usual, while bypassing preaeration.

Digester supernatant was lagaooned during this period. Only raw sewage passed through the south preaeration tank; the north preaeration unit was never in service. Thus, the key objective of direct comparative operation with and without preaeration was satisfied.

B. Sampling Schedule

Six samples were collected at 2 hr intervals:

A Preaeration influent (raw)
B Preaeration effluent
C Plain primary effluent
D Preaerated primary effluent
E Filter effluent
F Final effluent

On sampling days, an initial sample was taken at each point at 10 AM; sampling then proceeded until 8 AM the following morning, comprising the 24 hr sampling period. The samples, picked up in 250 ml square, wide-mouth bottles, were stored in a 4° C water-bath cooler.

This sequence ignored the time for passage of the sewage through the plant, but it was assumed that over the months involved, sampling errors from this source would largely cancel out. The schedule used did have the advantage of permitting laboratory work to begin on all samples promptly following 8 AM. Sampling was carried out routinely on Sundays, Tuesdays and Thursdays, permitting laboratory work to be concentrated on Mondays, Wednesdays and Fridays.

Considerable attention was given to the technique of sampling at each point to insure that the small quantity brought into the laboratory was as representative as possible of the flow at that point. Operating personnel who performed the sampling were carefully instructed in technique to insure that their efforts would be as uniform as possible.

C. Laboratory Procedures

On removal from the cooler, the individual samples were first composited in proportion to flow. The first two composites, preaeration influent and effluent, were homogenized for 30 secs in a food blender to break down the larger particles. This procedure yielded considerably more reproducible analytical results than were otherwise possible. Primary, filter and final effluent composites were not blended. The composite samples were used for all laboratory determinations.

After much hard and often frustrating work, the Gooch crucible method for determining SS was abandoned in favor of the following adaptation of the standard evaporation procedure described in the section on Residue of Part II of Standard Methods (52). First, approximately 250 ml of the composite was filtered through Whatman No. 12 folded filter paper. Next, duplicate 100 ml portions of the original, unfiltered composite and duplicate 100 ml portions of its filtrate were evaporated to dryness. The residues constituted TS and DS, respectively; SS were calculated by subtraction of dissolved residue from total residue. This method was found to provide excellent reproducibility and was particularly adaptable to a laboratory situation in which many SS determinations were made by a number of people.

BOD determinations were made according to the azide modification of Standard Methods. An important short-cut here was the use of a Bausch and Lomb Spectronic 20 colorimeter to read the DO content of a sample after color development, thereby eliminating the titration step, the preparation of standard sodium thiosulfate and starch solutions, and saving a good deal of time. The development of this colorimetric procedure and its use with both clear and turbid samples has been discussed in an earlier publication (37).

During this run, some preliminary work was conducted with the Bausch and Lomb colorimeter toward the development of a photometric method for the determination of SS as evidenced by turbidity. This procedure will be described later.

D. Results

This lengthy plant-scale run was conducted to provide a direct comparison of primary settling removals with and without preaeration, all else being equal. It was further desired to learn the length of test run required for the primary units and for the plant as a whole to achieve a steady state following a major operational change and to observe the effect of marked seasonal change on treatment results.

The results of this plant-scale run were quite enlightening in a frustrating, obstructive, challenging way — certainly unlike anything anticipated. As cold statistics, the primary removals are summarized in table 9 and plotted in figure 14. Detailed daily laboratory results for

Table 9. Summary of results for full-scale Ames plant runs, September 1956-May 1957

Month	No. of composites	Average weekday flow, mgd	BOD				
			Raw	Plain primary effluent		Preaerated primary effluent	
			mg/l	mg/l	Removal percent	mg/l	Removal percent
Sept.	4	1.8	220	164	25	107	51
Oct.	2	2.0	256	174	32	128	50
Nov.	6	2.0	233	179	23	150	36
Dec.	10	2.0	244	238	2	164	33
Jan.	11	2.1	268	234	13	177	34
Feb.	9	2.15	268	237	12	170	37
Mar.	9	2.1	268	252	6	216	20
Apr.	13	2.3	222	190	15	143	36
May	7	2.5	251	175	30	147	41
			SS				
Sept.	4	1.8	192	148	23	60	69
Oct.	4	2.0	240	122	49	71	70
Nov.	6	2.0	211	81	62	60	72
Dec.	10	2.0	209	138	34	76	64
Jan.	11	2.1	212	157	26	87	59
Feb.	9	2.15	224	163	27	82	63
Mar.	9	2.1	214	173	19	84	61
Apr.	13	2.3	202	148	27	84	58
May	7	2.5	215	83	61	73	66

the entire period are tabulated in the first Progress Report (5).

Generally, primary BOD and SS removals in winter were poorer than for either fall or spring. It also appeared that a steady state was reached in primary treatment, at least, in much less than one month. Beyond these general observations, it can only be reported that, while preaeration and settling produced BOD and SS removals in the 35 percent and 65 percent range, respectively, plain settlingremovals went from poor to bad, then became still worse and remained so as long as the operating procedure remained unchanged.

The run was permitted to continue, possibly far beyond practical limits, to see what the final outcome would be, even after the cause of the trouble had been theorized. The cause was GRIT, as later established positively by inspection, sampling and further runs in May, 1957.

Without the grit removal usually afforded by the preaeration unit, a certain fraction of rather coarse grit succeeded in building really impressive and unbelievably solid banks of grit in the sludge hopper area of the north primary clarifiers. These grit banks contained a high content of putrescible material. This digesting mass apparently was sufficiently active to literally poison the entire clarifier to an extent which knocked SS removals down to a level of 15 to 25 percent for many weeks. At times, primary effluent BOD strength exceeded that of the incoming raw sewage.

This phenomenon became rather predictable after some work with Imhoff cones. If effluent samples from the plain settling primaries contained $3\frac{1}{2}$-4 ml/l of settleable solids, compared to $5\frac{1}{2}$-7 ml/l in the raw sewage, BOD removal would be little or nothing. If, on the other hand, Imhoff cone tests indicated 0.1-0.2 ml/l of settleable solids in the effluent from these primaries, as was uniformly the case with the preaerated primary effluent, it could be expected that BOD and SS removals by plain settling would, for that day, be fairly respectable. Not once, however, did plain settling results equal parallel removals by preaeration and settling combined. Figure 15 presents the day-by-day pattern of SS results during these months of plant-scale operation.

In summary of this first major, albeit abortive, phase of the project, preaeration appeared to benefit primary settling at this plant consistently. This run was certainly not a quantitative indication of benefit; it is hard to characterize it as a quantitative indication of anything at all. More positive was the lesson taught on "Grit, care and handling of . . . "

VII. FULL-SCALE PLANT RUNS, JULY 1957

A. Plant Arrangement and Operation

With the cause of the extremely poor plain primary settling results known and under control, further plant runs were made to evaluate the benefits of preaeration, if any, under normal conditions. These runs were also planned to provide a check on the flow and treatment balance between the four primary clarifiers.

In most respects, the arrangement was unchanged from the long test run of the preceding winter and spring. The incoming raw sewage flow was accurately split; half received preaeration, then primary settling, while the other half was bypassed directly to identical plain primary settling. The two Walker downdraft aerators were still in place and were operated alternately as described below.

The July test run involved 24 hr sampling each Tuesday and Wednesday for four weeks. In preparation for the sampling period, the following routine was observed. On Monday morning, the sludge hoppers in all four primaries were jetted free of grit and sludge while the primaries remained in service. On Monday afternoon, digester and sludge well supernatants were diverted to the sludge lagoon, and final sludge recirculation was shut down. One preaeration unit was bypassed, and the plant received no return flow of any kind until conclusion of the two-day sampling period.

During the first and third weeks' runs, the south preaeration unit was operated with the south primary clarifiers while the other half of the flow was bypassed directly to the north primaries. During the second and fourth week, this arrangement was reversed to provide preaeration ahead of the north primaries and plain settling only in the south primaries.

In other respects, primary clarifier operation and sludge withdrawal was unchanged from the previous run. Two trickling filters were in service throughout the July run, as were all three final clarifiers. Final settled sludge was held in the final clarifiers during sampling.

B. Sampling Schedule

Sampling began at 10 AM Tuesday and extended through 8 AM Wednesday, constituting a 24 hr set of samples for compositing. Sampling then continued from 10 AM Wednesday through 8 AM Thursday to provide a second 24 hr composite.

Six samples were taken:

A	Preaeration influent (raw)
B	Preaeration effluent
D-1	No. 1 primary clarifier effluent
D-2	No. 2 primary clarifier effluent
D-3	No. 3 primary clarifier effluent
D-4	No. 4 primary clarifier effluent

Samples were collected in 250 ml square, wide-mouth bottles and stored in a 4° C water-bath cooler. In addition, grab samples were brought in as time permitted for immediate DO analysis and for determination of settleable solids.

C. Laboratory Procedures

Laboratory procedures used in compositing the samples and in carrying out BOD and SS determinations were unchanged from those previously described. For determining settleable solids, Imhoff cones were used in accordance with Standard Methods (52).

Since the incoming raw sewage was often deplete of DO during the daytime, the following procedure was used to arrive at a value for negative DO. A 305-ml BOD bottle was filled quietly with dilution water. A second BOD bottle was just as quietly filled to half capacity with dilution water. Upon arrival of the sewage sample in the laboratory, the remaining half of the second bottle was immediately filled by siphon from the sewage sample, and DO reagents were promptly introduced into both bottles. A negative DO value for the sewage sample was obtained as in this sample calculation:

Dilution water DO: 7.0 mg/l

50:50 mixture DO: 3.0 mg/l

Multiply DO mixture x 2, yielding 6.0 mg/l

then, 2 x mixture DO - dilution water DO

= 6.0 - 7.0 = -1.0 mg/l.

It is recognized that this procedure lacked refinement, yet it did provide helpful information not available by any other means.

In connection with these runs, further effort was expended on the photometric method for determining SS. The results of the July test runs are reported, however, on the evaporation basis only for consistency.

D. Results

This short run went smoothly, marred only by the unexpected startup of the local corn cannery during the fourth week. This resulted in fitful discharges to the sewage treatment plant of heavy loads of corn waste, clean water, and corn waste in unpredictable sequence.

The plant-scale comparative data indicated consistent and significant improvement in primary removals following preaeration. Detailed BOD and SS results are given in table 10; these results are summarized in table 11 and in figures 16 and 17. For the first three weeks, average primary BOD removals were 35 percent without preaeration and 43 percent with preaeration. Comparable SS removals were 61 and 69 percent, respectively. Removals were much lower the fourth week, but the improvement due to preaeration was still significant in spite of the abnormal waste flow.

Settleable solids results were of little or no help in evaluating the difference in primary removals. When there was any difference, it was in the range of a reading of 0.1 ml/l following plain settling, contrasted with a trace reading in the preaerated primary effluent.

As indicated by individual composites for the four primary clarifiers, the balance between them was excellent. No one unit showed even mildly better removal than the others. It would also appear that both preaeration units were achieving reasonably like results.

The DO work proved valuable, less so in interpretation of the July results than as a background for later work with oxygen values. The DO results are summarized rather simply in table 12. These values, typical

Table 10. Detailed results of July 1957 full-scale Ames plant runs

Date		Preaeration influent	Preaeration effluent	Primary clarifier effluent*			
				No. 1	No. 2	No. 3	No. 4
9	BOD						
	strength, mg/l	146	154	101	106	83	92
	removal, percent			30.8	27.4	43.2	37.0
	SS						
	strength, mg/l	216	155	60	77	45	52
	removal, percent			72.2	64.3	79.2	75.9
10	BOD						
	strength, mg/l	144	140	109	101	86	95
	removal, percent			24.3	29.8	40.3	34.0
	SS						
	strength, mg/l	156	147	64	61	48	44
	removal, percent			59.0	60.9	69.2	71.7
16	BOD						
	strength, mg/l	167	132	95	91	102	113
	removal, percent			43.1	45.5	38.9	32.3
	SS						
	strength, mg/l	146	153	46	54	63	41
	removal, percent			68.5	63.0	56.8	71.9
17	BOD						
	strength, mg/l	146	131	80	77	84	89
	removal, percent			45.2	47.2	42.5	39.1
	SS						
	strength, mg/l	122	154	48	50	54	60
	removal, percent			60.6	59.0	55.7	50.8
23	BOD						
	strength, mg/l	160	166	110	107	106	96
	removal, percent			31.2	33.1	33.7	40.0
	SS						
	strength, mg/l	158	162	70	64	53	49
	removal, percent			55.6	59.5	66.5	69.0
24	BOD						
	strength, mg/l	168	150	93	90	79	76
	removal, percent			44.6	46.4	53.0	54.7
	SS						
	strength, mg/l	160	147	55	56	48	43
	removal, percent			65.6	65.0	70.0	73.1

*Underlined data indicate preaerated primary effluent; data not underlined indicate plain primary effluent.

Table 10. (Continued)

Date		Preaeration influent	Preaeration effluent	Primary clarifier effluent			
				No. 1	No. 2	No. 3	No. 4
30**	BOD						
	strength, mg/l	264	224	182	184	176	160
	removal, percent			31.1	30.3	33.3	39.3
	SS						
	strength, mg/l	129	127	82	78	83	89
	removal, percent			36.4	39.6	35.6	31.0
31**	BOD						
	strength, mg/l	127	142	60	75	99	78
	removal, percent			52.7	41.0	22.1	38.5
	SS						
	strength, mg/l	98	101	57	55	67	63
	removal, percent			41.8	43.9	31.6	35.7

**Local corn cannery in operation.

Table 11. Summary of July 1957 Ames plant results

Date and day		Flow mgd	Mean sewage temperature °F	Average primary BOD removal, percent		Average primary SS removal, percent	
				Plain	Preaerated	Plain	Preaerated
9	Tu	2.92	70	29.1	40.1	68.2	77.5
10	W	2.88	70	27.1	37.2	60.0	70.5
16	Tu	2.76	71	35.6	44.3	64.4	65.8
17	W	2.70	70	40.8	46.2	53.3	59.8
23	Tu	2.65	70	32.2	36.9	57.6	67.8
24	W	2.44	71	45.5	53.9	65.3	71.6
30	Tu*	2.96	72	36.3	30.7	33.3	38.0
31	W*	2.65	72	30.3	46.9	33.7	42.9

*Local corn cannery in operation.

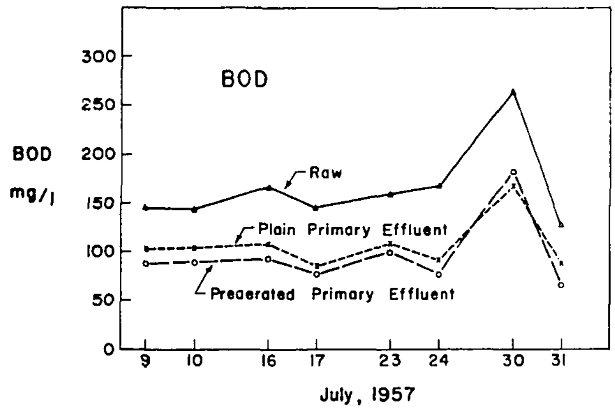


Fig. 16. Primary BOD removals for July 1957 Ames plant runs

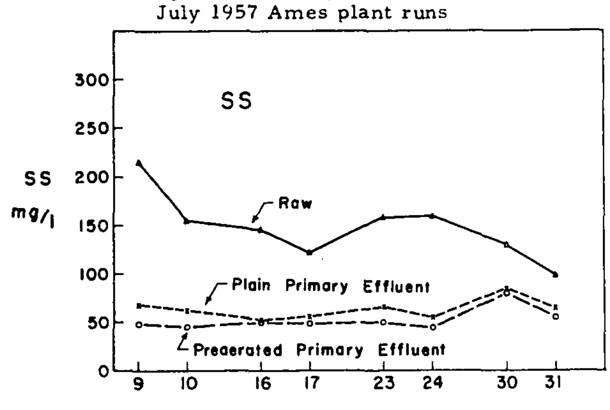


Fig. 17. Primary SS removals for July 1957 Ames plant runs

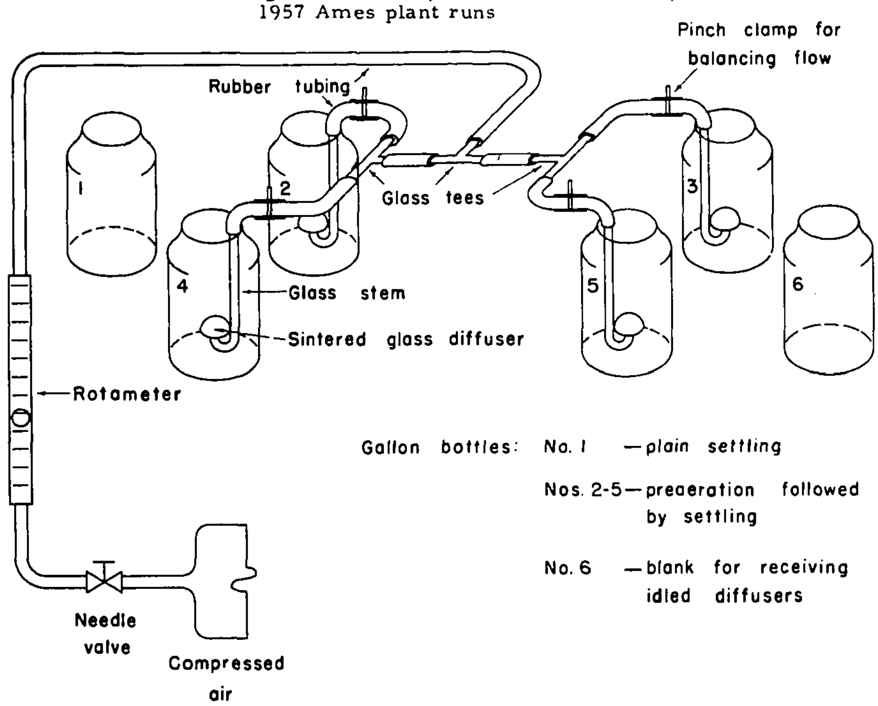


Fig. 18. Sketch of arrangement for laboratory-scale preaeration tests

Table 12. Typical afternoon DO values during July 1957 Ames plant runs

Operating arrangement	Range of average DO values, mg/l*			
	Preaeration influent	Preaeration effluent	Primary clarifier effluent No. 1 and 2	Primary clarifier effluent No. 3 and 4
North preaeration unit operated in conjunction with primaries 1 and 2; South preaeration unit bypassed.	$-\frac{1}{2}$ to -1	-1 to -2	-1 to -2	-1 to -3
	(slight loss)		(held even)	(drop)
Reverse of above; South preaeration unit in operation preceding primaries 3 and 4.	$-\frac{1}{2}$ to -1	0 to -1	-1 to $-2\frac{1}{2}$	0 to -2
	(held even or slight gain)		(drop)	(held even or slight drop)

*These results are representative of fairly stable afternoon conditions, determined by hourly sampling.

of fairly stable afternoon conditions, indicate that preaeration with downdraft aerators was maintaining the DO level reasonably constant as raw sewage passed through the unit. However, the DO level in the primary clarifiers seemed to drop more slowly following preaeration than it did in the plain settling primaries. The rebuilt south aerator somewhat outperformed the north aerator by this criterion, which is not surprising.

Again in general terms, the DO content of the incoming raw sewage dropped swiftly from as high as 4 to 6 mg/l in the early morning hours to zero by 10:15 or 10:30 AM, the height of the morning flush. The DO level continued down to as low as -3 mg/l in the late forenoon; recovered to the fairly stable afternoon plateau of -0.5 to -1.0 mg/l and usually did not reoccur as a plus figure until a short interval around the supper hour. In the late evening the DO level of the raw sewage became definitely positive and remained so until the following forenoon.

For this waste at this plant during this month, preaeration produced significantly better primary removals than did plain settling. The way was cleared for work on the factors affecting successful use of the preaeration process.

VIII. LABORATORY-SCALE TESTS, AUGUST-SEPTEMBER 1957

A. Laboratory Facilities

1. Objectives

Among the factors affecting preaeration, these would appear to be most critical:

a. aeration rate,

b. length of preaeration period, and

c. length of settling time following preaeration.

It is important that any one variable be analyzed independently of all others insofar as possible. It would be most helpful to study each of these factors at length in plant scale, but this is simply not practical in any existing installation.

At the Ames plant, equipment was installed to vary the aeration rate over wide limits, making it possible to study this factor in full-plant scale. It would have been awkward to vary either the length of preaeration or the settling time followingpreaeration while keeping all other conditions constant. These two factors, therefore, were studied by an extensive series of test runs in the laboratory.

2. Experimental arrangement

Wide-mouth gallon bottles served as the test containers for these runs. The experimental arrangement shown in figure 18 consisted of the test containers, a compressed air source, a needle valve and rotameter to insure constant air flow, and a set of four sintered diffusers for introducing air into the sewage samples. Following preaeration, the samples were permitted to settle undisturbed in the bottles to avoid the risk of floc break-up by transfer to other containers. Air flow to individual diffusers was balanced visually by pinch clamps to give approximately the same intensity of turnover in each bottle. A fifth bottle was provided for plain settling, and a sixth for receiving diffusers when individual preaeration periods were concluded. In this way, a balanced, constant air supply was maintained to all four diffusers throughout the run.

The aeration rate was established at 140 cu cm/min to each gallon bottle, or a total of 560 cu cm/min. This rate was derived as the arithmetic equivalent of a 200 cfm aeration rate in one of the Ames preaeration tanks. The 200 cfm rate was the capacity of the blower secured for project use. The 140 cu cm/min rate to each gallon bottle was equivalent to the following aeration rates for the times shown:

Table 13. Preaeration rates during laboratory-scale tests

Preaeration period, mins	Aeration rate, cu ft/gal
15	0.074
30	0.148
45	0.223
60	0.297

3. Procedure

After several trial runs, the test procedure was standardized as follows. Five of the gallon containers were filled quickly with raw sewage and brought to the laboratory. This constituted a grab rather than a composite sample, collected most commonly at 10:30 or 10:45 AM or at 1:30 or 1:45 PM. An aliquot of the sample was drawn for BOD and SS analysis. Preaeration was initiated immediately in four of the containers; the fifth was permitted to settle quietly. The sixth bottle was filled with tap water. According to a predetermined schedule, preaeration time would vary, followed by uniform settling time, or settling time would vary, preceded by uniform preaeration.

As the preaeration period for each bottle was concluded, its diffuser was moved to the bottle of tap water and permitted to continue aerating; this involved no adjustment of air supply to the other bottles. At the conclusion of preaeration and settling for each bottle, a sufficient aliquot was immediately drawn by siphon from just below the surface, and determinations of BOD and SS were made.

Two sets of laboratory-scale runs were projected in line with the objectives described above. In the first group, comprising test series A and B, four 1 gal samples were preaerated for 15, 30, 45 and 60 min, respectively. Preaeration was followed by a settling period which was identical for these four samples and for the fifth sample receiving plain settling only. In series A, separate runs were made with settling times of 1, 2, 3 and 4 hr. In series B, the sequence of settling times was changed to 0, ½, 1 and 2 hr. With replicates, a total of eighteen separate runs was made in this group, designed to determine in the laboratory the effect of length of the preaeration period.

The second group of laboratory-scale runs was intended to show the effect of varied settling time following preaeration, all else being equal. In each run, four samples were preaerated for an equal length of time, then given settling periods of 1, 2, 3 or 4 hr in series C, or 0, $\frac{1}{2}$, 1 or 2 hr in series D. A total of twenty-four separate runs was made in this second group of tests. The preaeration periods used were 15, 30, 45 and 60 min.

The local corn cannery operated sporadically during most of the six weeks of these laboratory test runs. This had the nuisance effect of doubling normal sewage strength with little notice, but it did provide a variety of waste flows for this phase of the project. Although an attempt was made to carry out each specific test combination both with and without cannery waste, it did not appear that there was a significant difference in the results obtained.

B. Test Results

1. Varied preaeration time

Individual analytical results for the eighteen test runs in Series A and B are plotted for comparison in figures 19 and 20. The most perfunctory glance at these data indicates that some preaeration seemed to be better than none, and that where some was good, more appeared to be still better.

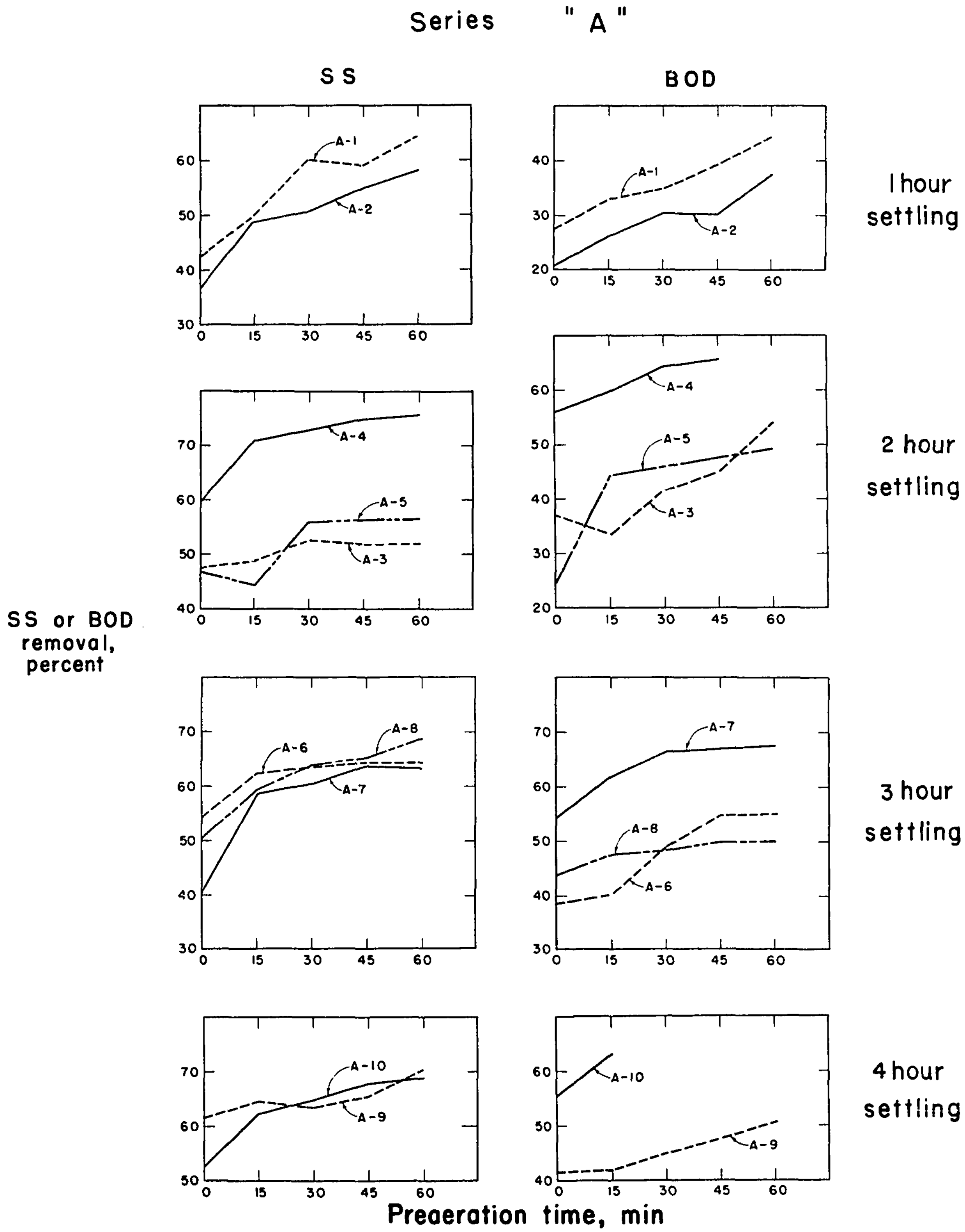


Fig. 19. Effect of varied preaeration time on removals; laboratory series A, 1 to 4 hr settling

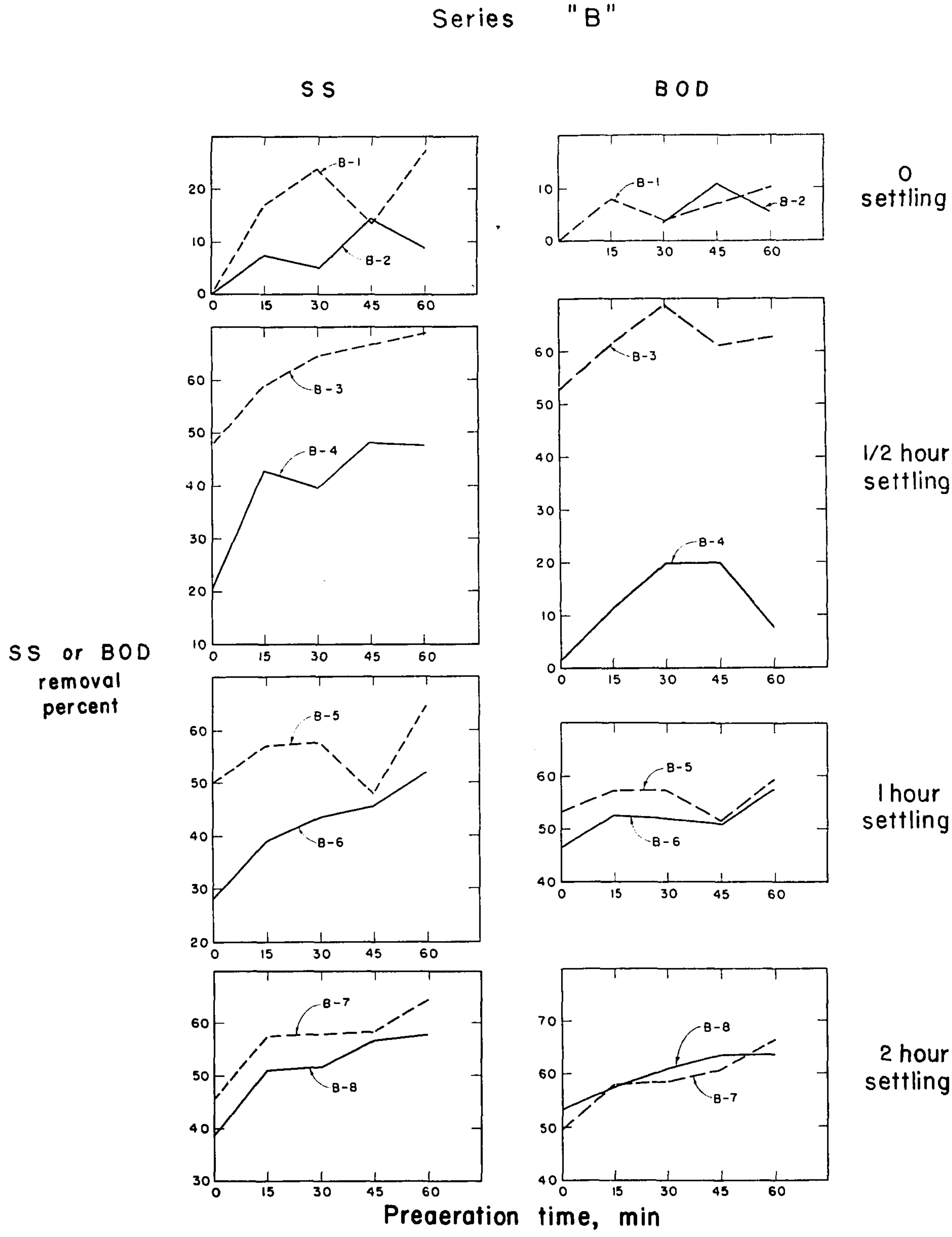


Fig. 20. Effect of varied preaeration time on removals; laboratory series B, 0 to 2 hr settling

In analyzing these results, the improvement in primary settling due to preaeration is defined in terms of percentage points. This is more meaningful than an expression of percent improvement. Suppose, for example, that primary BOD removal on Monday was boosted from 10 percent by plain settling to 20 percent by preaeration and settling combined. Assume further a corresponding improvement on Tuesday from 40 to 50 percent removal. The degree of improvement might be cited as 100 percent on Monday and only 25 percent on Tuesday; yet the actual level of improvement was 10 percentage points each day, a much more realistic appraisal of the benefit accruing from preaeration.

From table 14 it is apparent that preaeration always improved SS removal, and that longer aeration increased the benefit. The improvement was most marked with short settling time, decreasing in relative importance as the settling time lengthened. Table 14 also indicates that BOD removal was benefited by preaeration, to a degree corresponding generally with length of the aeration period. In the BOD there is no clear indication that the benefit of preaeration was proportionately greater with short settling time. However, it would seem that BOD and SS removals must parallel, at least generally, and in this case the SS analytical results were more reliable.

It was observed that removals of both BOD and SS were indicated by preaeration without settling in these laboratory test runs. This was never indicated in plant-scale results. No explanation of this phenomenon is offered.

The summary of proportional improvement (table 15 and figure 23) was

Table 14. Effect of varied preaeration time on removals; summary of laboratory results, Series A and B

No. of runs*	Settling time hrs	Preaeration time, mins 0	15	30	45	60	Overall improvement by preaeration, in percentage-points
		SS removal, percent					
2	0	0	12	14	14	18	18
2	½	34	51	52	57	58	24
4	1	39½	48½	53	55	60	21½
4	2	47½	56	59½	61½	63	15½
3	3	48½	60	62½	64½	65½	17
2	4	57	63½	64	66½	69½	12½
		BOD removal, percent					
2	0	0	4	4	6	8	8
2	½	27	36½	-	40½	(41)**	(14)**
4	1	37	42½	43½	44½	49½	12½
5	2	44	51	54½	56½	60	16
3	3	45½	50	54½	57	57½	12
2	4	48½	52½	(55)	(57½)	(60)	(11½)

*Each run comprises five 1 gal samples, all receiving <u>same</u> settling as shown, following <u>varied</u> preaeration as shown.

**Data in part extrapolated.

Table 15. Proportional improvement by increments of laboratory preaeration

	Preaeration time, mins 0	15	30	45	60
	percent of ultimate* improvement over plain settling				
SS	-	55	73	87	100
BOD	-	47	68	78	100

*Arbitrarily assuming for calculation purposes that full ultimate improvement over plain settling is achieved by 60 min preaeration.

based on an arbitrary calculation assuming that complete or ultimate benefit over plain settling was attained in 60 mins of preaeration. On this basis, about half the expected improvement was achieved in the first 15 mins of aeration; some 70 percent in 30 mins and over 80 percent in 45 mins. This, of course, was in the laboratory under ideal conditions, with treatment of the total sample for the full period without short circuiting or similar difficulties encountered in plant operation.

Further analysis of the test data from Series A and B indicated that neither the benefit of preaeration, nor the time element just discussed, seemed to be affected by the strength or character of the raw sewage. Strong sewage was benefited no more than the weak. In contrast, the correlation shown in figure 24 between the improvement in percentage points credited to preaeration and the removal by plain settling was particularly interesting. The poorer the job done by settling alone, the greater was the benefit resulting from preaeration. If this were valid in actual plant operation, the preaeration process would be unusually useful.

2. Varied settling time

Series C and D runs consisted of 24 individual tests with settling times of 1, 2, 3 or 4 hrs and of 0, ½, 1 or 2 hrs for the four 1 gallon samples. Detailed analytical results are plotted for comparison in figures 21 and 22.

SS and BOD data for this group of runs are summarized in table 16. The results of these runs were somewhat erratic and are open to a variety of interpretations. As expected, removals improved with additional

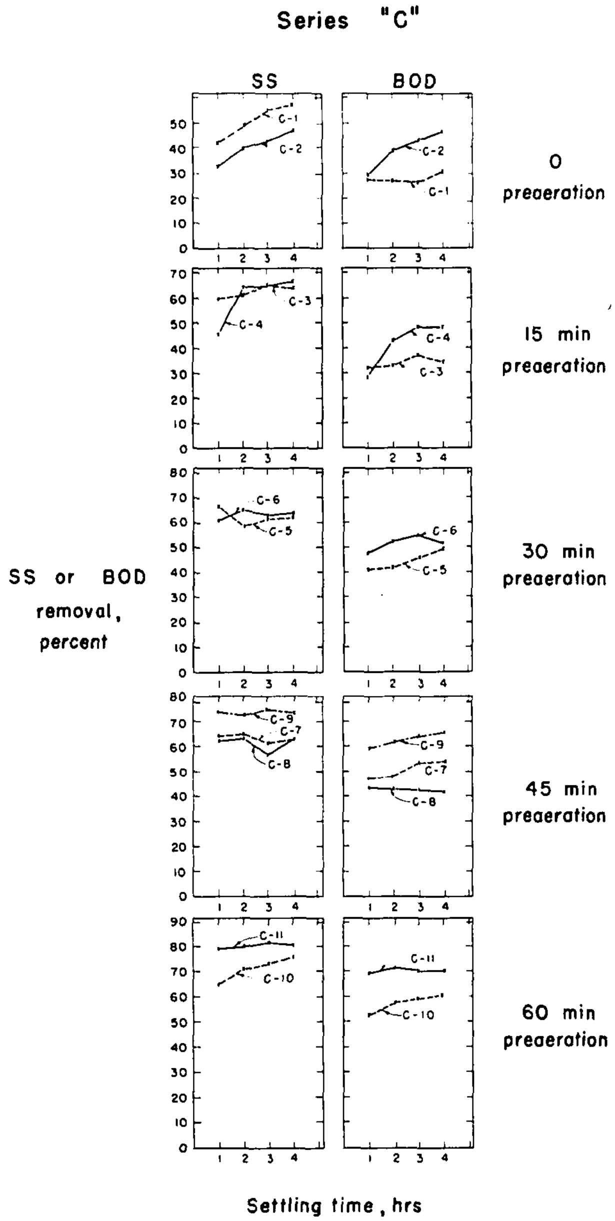


Fig. 21. Effect of varied settling time on removals; laboratory series C, 0 to 60 min preaeration

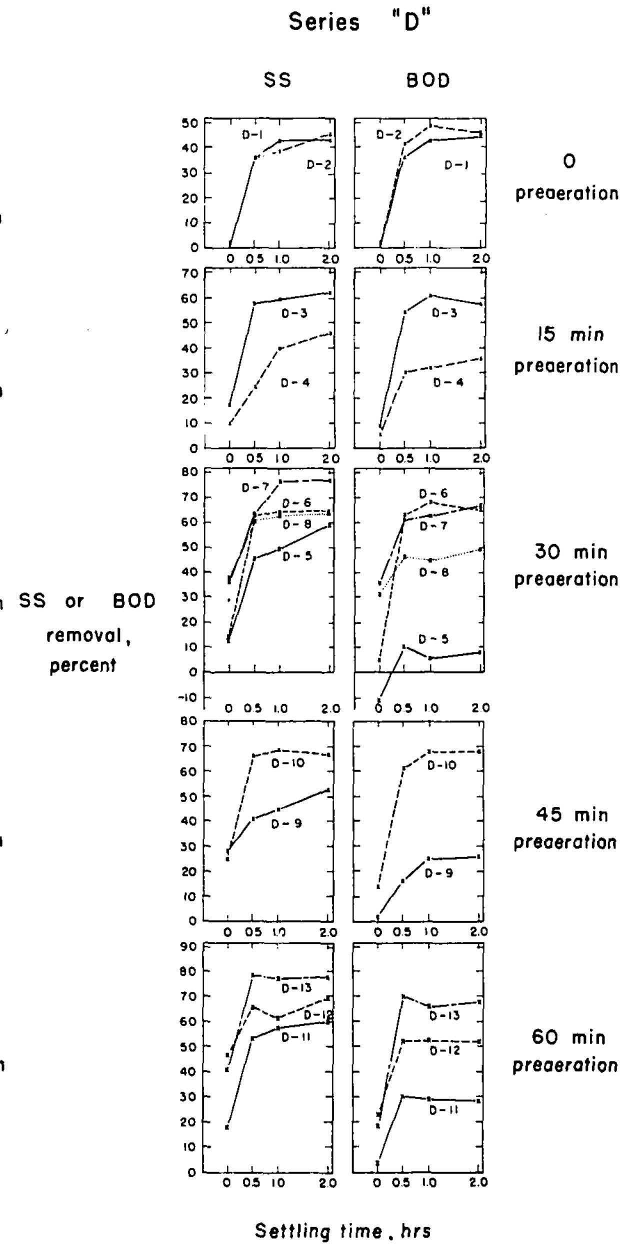


Fig. 22. Effect of varied settling time on removals; laboratory series D, 0 to 60 min preaeration

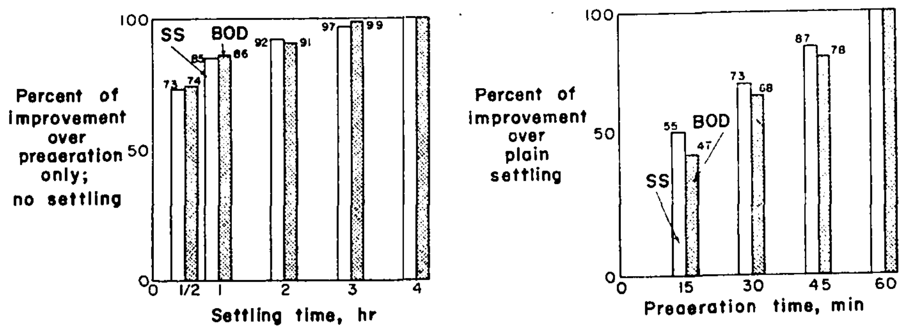


Fig. 23. Proportional improvement in primary removals by increments of laboratory preaeration and settling time

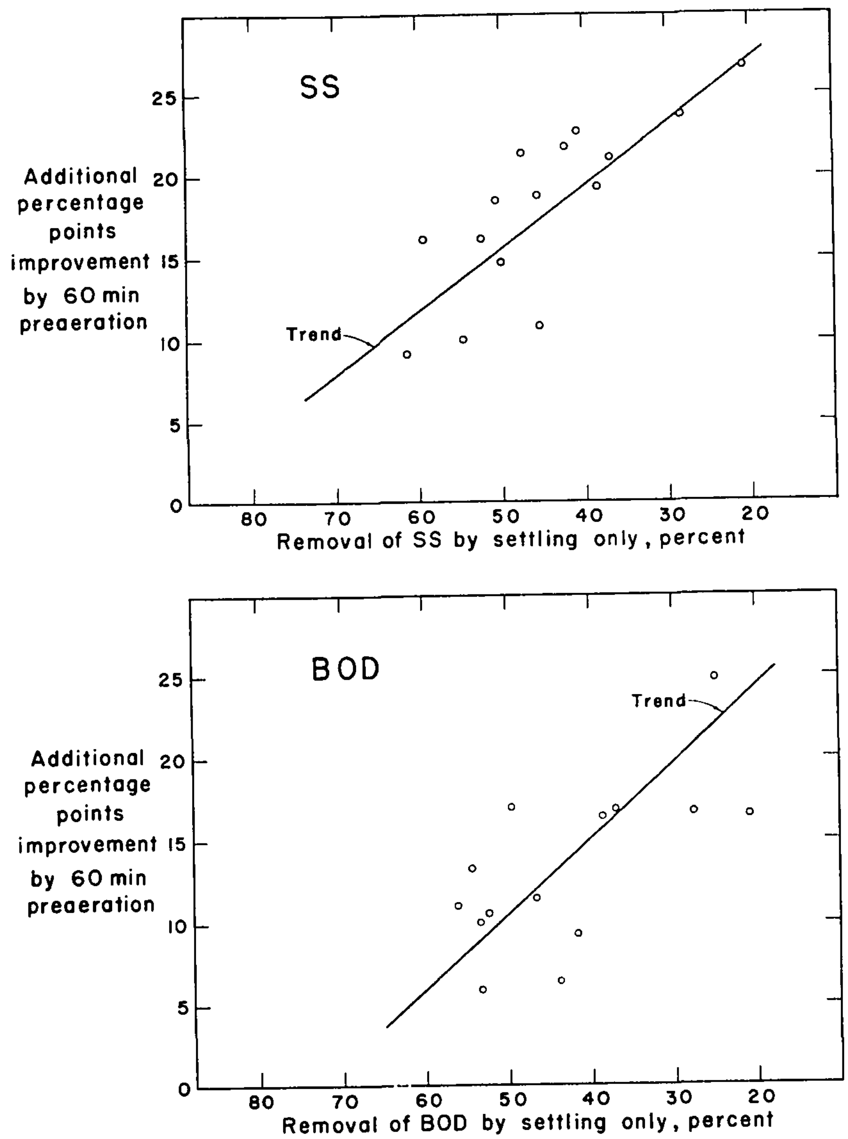


Fig. 24. Relationship of improvement by preaeration to plain settling removals

Table 16. Effect of varied settling time on removals; summary of laboratory results, Series C and D

No. of runs*	Preaeration time mins	Settling time, hrs 0	½	1	2	3	4
		SS removal, percent					
2-4	0	0	36½	39	44½	49	52½
2-4	15	13½	41½	51½	58½	65½	66
2-6	30	25½	58	63½	64½	62	63
2-5	45	27	54	62½	65	65	66½
2-5	60	35	66	68	71½	77½	78
		BOD removal, percent					
2-4	0	0	39	37	39	34½	39
2-4	15	7½	42	43	47	53	51½
2-5	30	18	45½	49	55	50	50½
2-5	45	8½	34½	49	50	53½	54
2-5	60	15½	51	54	55½	64½	65

*Each run comprises five 1 gal samples, all receiving the <u>same</u> preaeration followed by varied settling as shown.

settling. These data also appeared to confirm the results of the first group of runs in that the removal, scanning the columns from top to bottom, improved with increased preaeration time. Although reasonable, this is not a valid inference, since every preaeration value tabulated represents a different sample. These runs were designed to evaluate only the effect of varied settling time.

As with the first group, it appeared that preaeration alone was responsible for removal of BOD and SS. Again, this was contradicted in later plant-scale runs.

Table 17, the data for which are also shown in figure 23, summarizes the effect of varied settling time, arbitrarily assuming ultimate benefit

Table 17. Proportional improvement by increments of laboratory settling

	Settling time, hrs					
	0	½	1	2	3	4
	percent of ultimate* improvement over preaeration only					
SS	-	73	85	92	97	100
BOD	-	74	86	91	99	100

*Arbitrarily assuming for calculation purposes that full ultimate improvement over preaeration only (no settling) is achieved by 4 hr settling.

from 4 hr settling. In these terms, over 90 percent of ultimate benefit was achieved in 2 hrs, and 80 percent of that was attained in the first half hour of settling. It appears that preaeration strongly enhances the efficiency of the early moments of settling.

3. Composite results

Tabulated in composite form, analytical results of all 42 laboratory-scale runs are summarized in table 18. These composite results, in themselves, are more interesting than weighty. They support the beneficial effect of preaeration; they also provide in contour form a general indication of different combinations of preaeration and settling which, in the laboratory, produced equivalent results.

For example, 30 min preaeration followed by ½ hr settling outperformed plain settling of 2 or even 3 hr duration. Similarly, 60 min preaeration with ½ hr settling was roughly equivalent to 15 min preaeration and 3 hr settling, and quite superior to plain settling of 4 hr duration or longer.

Table 18. Composite results of all laboratory tests

No. of runs	Settling time hrs	Preaeration time, mins				
		0	15	30	45	60
		SS removal, percent				
4-6	0	0	13	21½	20½	28
4-6	½	35	46	56	55½	63
8-10	1	39	50	59	59	64
8-10	2	46	57	62½	63½	67½
5-6	3	48½	62½	62½	65	70
4-5	4	55	65	63½	66½	74
		BOD removal, percent				
4-5	0	0	6	12	7	12
4	½	33	39	(42)*	37½	(47)*
8-9	1	37	43	56½	49	52
9-10	2	41½	49	54½	53	57½
5-6	3	41	51½	52½	55½	60½
4-5	4	44	52	(53)*	(56)*	(62½)*

*Data in part extrapolated.

In these terms, preaeration appears very attractive. It also appears that the key to its optimum use may be in the judicious substitution of preaeration detention time for settling detention time. Finally, it must be cautioned that there are wide and numerous gaps between what can be done in the laboratory and what can be done in full-scale plant operation.

IX. FINAL LABORATORY AND TEST PROCEDURES

Prior to the discussion of the plant-scale runs carried out during the spring and summer of 1958, this section will be devoted to a detailed description of the laboratory procedures standardized during the preceding winter, and of the aeration equipment and control facilities at the Ames plant.

A. Oxygen Values

1. Dissolved oxygen (DO)

The DO determination was carried out in accordance with the Winkler method, azide modification, as described in Standard Methods (52). The only variation from this procedure was the direct photometric determination of DO with a Bausch and Lomb Spectronic 20 Colorimeter after color development (figure 25) rather than by titration with sodium thiosulfate. This photometric procedure was developed in the early months of this project and its use for both clear and turbid samples has been described earlier (37).

For a clear sample, the transmittance was observed and the corresponding DO read directly from a plot (figure 26). This calibration in tabular form, together with detailed procedure, is included in Appendix D. The calibration table was prepared by titration of numerous DO samples for which the transmittance was also determined.

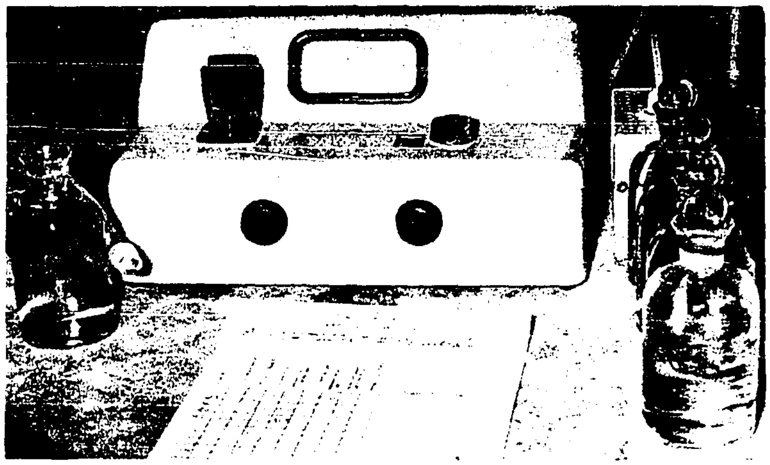


Fig. 25. Bausch and Lomb Spectronic 20 Colorimeter used in DO determination

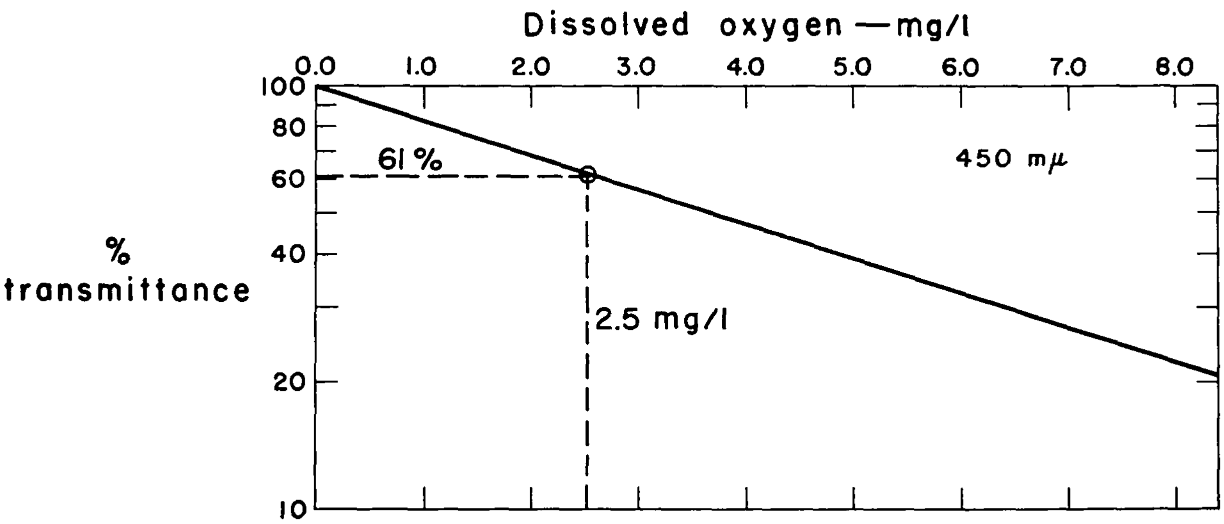


Fig. 26. DO calibration for use with Spectronic 20 Colorimeter

Fig. 27. Varied color development in series of sample bottles for DO depletion rate test

The procedure for a turbid sample is of special interest because it provided the key to an approximate photometric method for SS. A photometric reading merely indicates the amount of light reaching the photocell. The intereference caused by color and by turbidity are cumulative, and can only be separated by an indirect method. In the procedure followed for a turbid sample, color was developed and the transmittance read; then the iodine color was destroyed by oxidation with sodium thiosulfate and the transmittance read again. From the combined Lambert-Beer law (37) the interference credited to the color alone is expressed by the ratio of the two readings. This ratio is equivalent to the transmittance for the iodine color alone, and the DO of the sample can be determined with this calculated transmittance ratio directly from figure 26 as before.

2. Negative dissolved oxygen

Raw sewage arriving at the treatment plant may have free DO present, or may be totally deplete, or may have deteriorated to the point where some chemically combined oxygen has been consumed and aeration would be necessary to restore the presence of DO in solution.

For lack of a better definition, this may be termed a state of *negative DO*. To obtain some measure of this condition, equal volumes of dilution water and of the sewage sample were combined. With the DO of the dilution water and of the mixture determined, the DO of the original sewage sample was readily calculated. The procedure and sample calculations are detailed in Appendix D.

3. Biochemical oxygen demand (BOD)

a. Procedure. In determining the BOD of raw sewage and preaeration effluent, an aliquot portion of the composite sample was homogenized for 30 secs in a food blender. Several trial series indicated that blending improved the repeatability without significantly changing the numerical result of the 5 day BOD test. All other laboratory determinations were made on the unblended composite sample. Blending was omitted entirely for primary effluent samples, since it appeared to contribute nothing to laboratory technique in their case.

Standard Methods procedure was followed, including the use of dilution water supplemented with phosphate buffer, magnesium sulfate, calcium chloride and ferric chloride solutions. Plunging the sample while pipetting off portions for seeding the dilution bottles was religiously observed. The dilution bottles were incubated at 20° C for 5 days. Following iodine color development, DO determinations were made photometrically as described above.

The photometric shortcut eliminated the preparation and frequent standardization of sodium thiosulfate and saved many hours of laboratory work. This modification has since been adopted for routine plant control at several Iowa sewage treatment plants. A sample BOD data and calculation sheet is included in Appendix D.

b. Limitations. Intensive work with the BOD test serves to make one aware of its limitations and vagaries. Even with great care, the numerical results of a single set of dilutions may not fall within a 10 percent spread. This is certainly discouraging in a research effort for which a

10 percent variation may indicate whether or not a trial operating change is worthy of further study.

On several occasions, inhibitors were apparently present either in the sewage reaching the plant or in the dilution water. The results were either sharply depressed oxygen demand or highly erratic numerical results, or both; and the first hint of this did not appear until five days later. When the dilution water was at fault, chances were good that some or all of the work of the intervening five days was equally faulty. In spite of its limitations, however, the 5 day BOD determination remains the standard laboratory approach to oxygen demand evaluation.

4. DO depletion rate

In an effort to learn something about the rate of short-term oxygen demand of raw sewage upon its arrival at the plant, several simple experimental methods were tested. The result was a procedure which consisted of violently agitating a sewage sample to a DO level of 6 or 8 mg/l, then observing the rate of DO depletion over a period of 45 mins to an hour. After agitation, a series of small bottles was filled immediately by siphon from the sewage sample. The DO level was then determined in one bottle at a time at 5 or 10 min intervals. The result (figure 27) was a series exhibiting decreasing color intensity, and thus DO content, with time. Detailed procedures and sample calculations are described in Appendix D.

For normal daytime raw sewage, the rate of DO depletion was in the range of 5 to 10 mg/l per hr. With strong cannery waste, this rate was recorded as high as 17 mg/l per hr. A sample of raw sewage taken at 7:10

AM, still clear night flow in appearance, was found to have a depletion rate of about 0.1 mg/l per hr. The normal rate for primary effluent was found to be in the range of 2 to 4 mg/l per hr.

This determination was helpful in evaluating the rate of oxygen acceptance during the preaeration step. If the raw sewage neither gained nor lost DO during preaeration, its oxygen demand rate was just being met, and the depletion test then provided a numerical measure of the rate of both demand and supply. If the DO level rose or dropped enroute through preaeration, the rate of acceptance could be approximated by taking into account probable detention time. Although not particularly scientific, this test contributed an understanding of the aeration phase of preaeration which would not have been possible otherwise.

5. Oxygen acceptance rate during aeration

Knowing the DO depletion rate and the DO levels entering and leaving the preaeration step, it was possible to approximate the oxygen acceptance rate in terms of mg/l per hr. This rate was found to vary with the equipment used and with the rate of aeration.

The procedure involved sampling both the preaeration influent and effluent at 10 min intervals for at least an hour. The sampling periods were staggered to reflect the approximately half-hour actual mean detention time in the preaeration unit at normal daytime flow. Reagents for the DO determination were added to these samples within less than a minute. A typical DO sampling run is reproduced in Appendix D.

During this period, DO depletion tests on preaeration influent were being run in the laboratory. With the data on DO levels and depletion rates, the oxygen acceptance rate was calculated as shown in Appendix D.

6. Oxygen-reduction potential (ORP)

ORP determinations were made for a limited time only, near the conclusion of plant-scale work late in 1958. In the waste treatment field, ORP remains a research tool rather than a routine laboratory procedure. Grune and Chueh (20) have discussed the meaning of ORP in sewage treatment work. Its potential value has been summarized as follows (48, p. 93):

> I would just like to add one comment. Fundamentally, the ORP is a measuring stick that can be used to measure the intensity of anaerobic conditions. For example, consider the dissolved oxygen test -- you might run a DO and find no dissolved oxygen. That is worth knowing, but it doesn't tell how bad off you really are. The ORP measurements allow you to go way down on the scale and measure degrees of oxygen deficiency indicating: when all nitrates are exhausted; when all sulfates are exhausted and so forth. It has real value there.
>
> I don't mean to say the ORP is limited to these very things but I think it will find its greatest application in sensing anaerobic conditions and how bad they are. It possibly can be used also for sensing the degree of aerobic conditions but my experience with it under aerobic conditions is that after the DO gets above 1 ppm it doesn't tell you very much.

The ORP data collected in this project were of interest, but a much broader foundation would be necessary before these data would be pertinent to either experimental or plant control work. The instrument used in determining ORP is shown in figure 28. The procedure and a sample calculation are included in Appendix D.

B. Solids Determinations

1. Suspended solids by evaporation

As discussed earlier, project personnel finished a poor second in a bout with the Gooch crucible procedure for SS. An alternate procedure was adopted in which TS were determined by evaporation as described in Standard Methods (52). A sufficient quantity of the same sample was also filtered through Whatman No. 12 folded filter paper to permit a parallel determination of DS by evaporation of this filtrate. SS were then calculated as the difference between TS and DS. The procedure and sample calculations are presented in Appendix E.

This method is simple but time-consuming. With filter paper of satisfactory quality, its results are highly reproducible. The only edge held by the Gooch crucible method over this procedure would appear to be the weight of tradition.

2. Photometric determination of SS

a. Development. Several descriptions of photometric approaches to the measurement of SS in sewage have appeared in the literature. Those described have been confined to single measurements of the interference to light transmission. This is essentially the turbidity determination. As long as the waste characteristics and weather remained the same, the results were fairly consistent. An important change in either, however, usually resulted in broad changes in the relationship of transmittance to SS content.

The major problem with sewage in this regard is that sewage is quite strong and quite variable in color. After some work with the photometric technique for determining DO of turbid samples, it appeared that the same general approach might apply here.

In the DO procedure for turbid samples, the transmittance is read both before and after decolorizing to cancel out the effect of turbidity. These readings provide a ratio which is equivalent to the transmittance of the color alone. In the photometric procedure for SS, transmittance of the sewage sample is read both before and after filtration, thus cancelling out color; the resulting ratio is the transmittance for SS alone.

In two years' use for both project and plant control work, this procedure has demonstrated its merit. Project personnel have learned to respect its limitations but remain enthusiastic over its advantages. Part of the laboratory arrangement is shown in figure 29. Procedures, sample calculations and calibration tables are presented in Appendix E.

b. Limitations. The chief limitation of this photometric procedure for SS is that it simply lacks the potential precision which could make it a primary standard. However, in spite of its hallowed standing, the Gooch crucible method is notorious for scattergun reproducibility; anything would be an improvement.

The characteristics of sewage are constantly changing; the color varies; the DS content varies with time of day, season, amount of groundwater infiltration and type of waste. For equal SS content as determined by evaporation, photometric response is significantly different at each major step of the sewage treatment process.

For this procedure, the Bausch and Lomb Spectronic 20 Colorimeter was also found to be a source of some variation. Both the light source and the photocell gradually weaken or expire and must be replaced. With a change in light components, the sewage sample may give mildly different readings, indicating a corresponding drift in calibration.

The SS calibration curves prepared and used during the 1958 plant-scale runs are shown in solid lines in figure 30. Further calibration work during January 1959, following several changes of light components, provided the dotted lines shown. Note that although the difference is quite strong, the curves are essentially parallel. It is not known at this writing whether this shift was primarily seasonal, or due to lack of photometric refinement.

c. Advantages. Whatever the cause of the variance described, the photometric method has the advantage of consistency. For example, if SS results by the evaporation method were found to be appreciably higher than by this method, this was observed for all samples of the group, much as if the calibration curve had been displaced to a new, parallel position. Since all results appeared to have been affected proportionally, calculated removals and comparisons of efficiency were virtually unaffected by such a shift in calibration.

The rapidity of this method is particularly valuable. SS determinations can be completed for a dozen samples in 30 mins. For the Ames plant studies in which four points were sampled every half hour for 12 to 24 hrs a day, SS determinations for each individual sample would have been physically impossible by any other method.

The photometric method described is one which the small plant operator can use. The likelihood of the small sewage treatment plant being provided with an analytical balance is as remote as the operator's familiarity with it. As a result, no SS determination of any kind is performed in nine out of ten of the smaller plants, nor in a surprising number of the larger ones. With any type of photometric equipment, preferably simple, the small plant operator could learn to perform BOD and SS analyses. The results might not be highly refined, but they would be valuable nevertheless.

C. Settling Rate, Oulman Settlimeter

Early in this project it was theorized that if preaeration were of benefit to primary treatment, it would accentuate physical flocculation of the raw sewage solids and thereby accelerate settling. Several experimental photometric arrangements were tested for observing the settling rate; the result was a device which was subsequently termed a settlimeter. Its functional design is shown in figure 31; a detailed description and test procedures describing its use appear in Appendix F.

In practice, a sample of sewage was collected in the 2 in diam settling cell and placed in the cell holder. As settling progressed, light transmission through the cell increased. This was measured by the photocell and indicated by the galvanometer. More rapid settling was readily apparent by more rapid change in galvanometer readings. Several trial runs with the light source placed at the 50 percent depth indicated no important change in settling patterns from tests made at the 20 percent depth. The settlimeter in its final form is shown in figure 33. Four

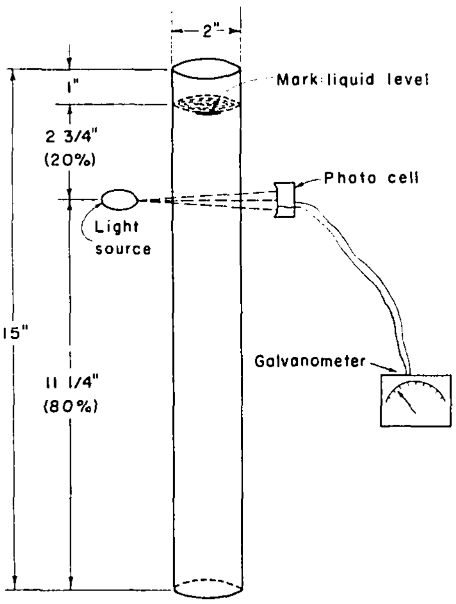


Fig. 31. Functional sketch of Oulman settlimeter

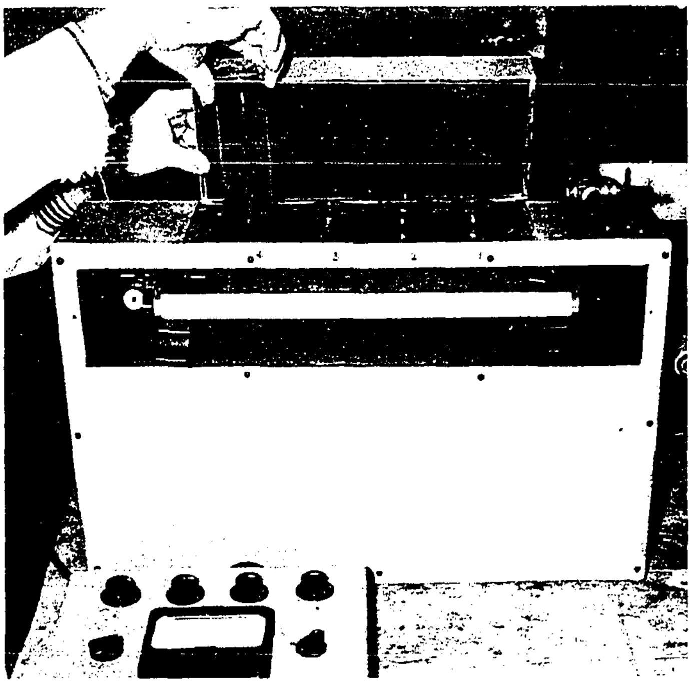


Fig. 33. Laboratory observation of settling characteristics with the Oulman settlimeter

Fig. 32. Sampson sampler used in collecting undisturbed samples for settleability determination

cell positions were provided. A selection knob permitted galvanometer readings to be made for each in turn as rapidly as necessary.

It was apparent that for valid settling results, samples should be brought into the laboratory in as nearly an undisturbed state as possible to avoid damaging the floc particles. For this purpose, a special sampler was designed by Mike Sampson, chief mechanic at the Ames plant. Its use is demonstrated in figure 32. With this device an inverted sampling cell was slowly filled with sewage by controlling the release of its entrapped air, then righted and removed from the sample holder.

Comparative settling rates were observed by sampling preaeration influent and preaeration effluent, spaced by a time interval representing probable mean detention time in the preaeration tank. A method was developed for conversion of galvanometer readings to mg/l of SS, by means of supplemental SS determinations with the Spectronic 20 Colorimeter. The settlimeter proved to be quite reliable in predicting and checking actual plant performance in full-scale runs at the Ames plant. It was particularly valuable in pointing up the effect of preaeration on the early moments of settling.

The settlimeter was of key importance in plant-scale preaeration studies at three other Iowa plants. At none of the three was it possible to split the flow for comparison of results with and without preaeration. Consequently, the primary criterion on which the benefit of preaeration to primary removal at these plants was based was the laboratory determination of settling characteristics of the wastes before and after preaeration.

D. Aeration Devices and Control

1. Sutorbilt blower

To undertake plant-scale studies with varied aeration rates, two things were needed: a source of air, and a means for measuring air flow. The first was provided by the loan of a Sutorbilt blower through the courtesy of Zimmer and Francescon, manufacturer's representatives of Moline, Illinois. Name-plate data for this blower were as follows:

Size 7H
.35 displacement
Serial 1274
Date 8-57

This blower was rated nominally at 200 cfm against 7 psi pressure, varying somewhat with speed. The blower and a 10 hp 440 v motor were mounted on a skid. The drive was with 3 V-belts, geared to a blower speed of 800 rpm, which produced somewhat more than 200 cfm at normal operating pressures. Clean air was assured by intake through oil-bath air cleaners requisitioned from an abandoned mobile air compressor.

2. Orifice calibration

Air flow was measured by careful calibration of two orifice plates in a length of 4 in diam cast iron pipe. Following calibration, this pipe section, with manometers, was moved to a permanent location in the building over the preaeration tanks.

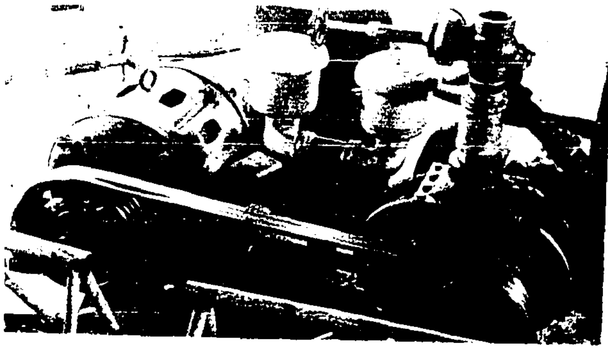
The orifice holding arrangement was rather simple, a flanged connection near the outlet end of the length of 4 in pipe. The holder was designed for insertion of an orifice plate with flange holes drilled to match. Steel plate 5/64 in in thickness was used in fabricating the orifice plates; 3/32 in vellumoid gaskets were placed on either side. After some preliminary work, the two plates calibrated were 1 1/8 in and 1 7/8 in in orifice diameter.

Air flow was metered through two large factory-rebuilt and calibrated gas meters. At high rates both were used, while at lower rates one was bypassed (figures 34 and 35). Air flow was controlled by combined use of the bypass valve at the blower and the throttling valve at the outlet of the orifice section. Replicate runs were made at various pressures at the orifice. Air flow data were calculated to standard conditions at atmospheric pressure (figure 36). Normal operating pressure proved to be about 7 psi.

3. Aeration devices

a. Walker Process downdraft aerators. This is the original equipment installed in the Ames preaeration tanks. Each aerator has a 5 hp motor driving a turbine impeller operating at a shallow liquid depth; as the impeller revolves, atmospheric air is drawn through suction tubes to the impeller area and this air-sewage mixture is pumped down through a draft tube to the lower tank area. Both recirculation and some aeration of the tank contents are achieved. The major source of energy for tank recirculation is, however, pumping. This equipment was, of course, operated without the blower.

By manifolding the air suction tubes and connecting this manifold through a large hose to the orifice line, the intake draft of the original



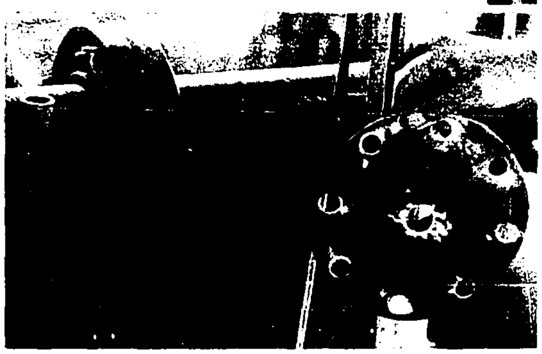
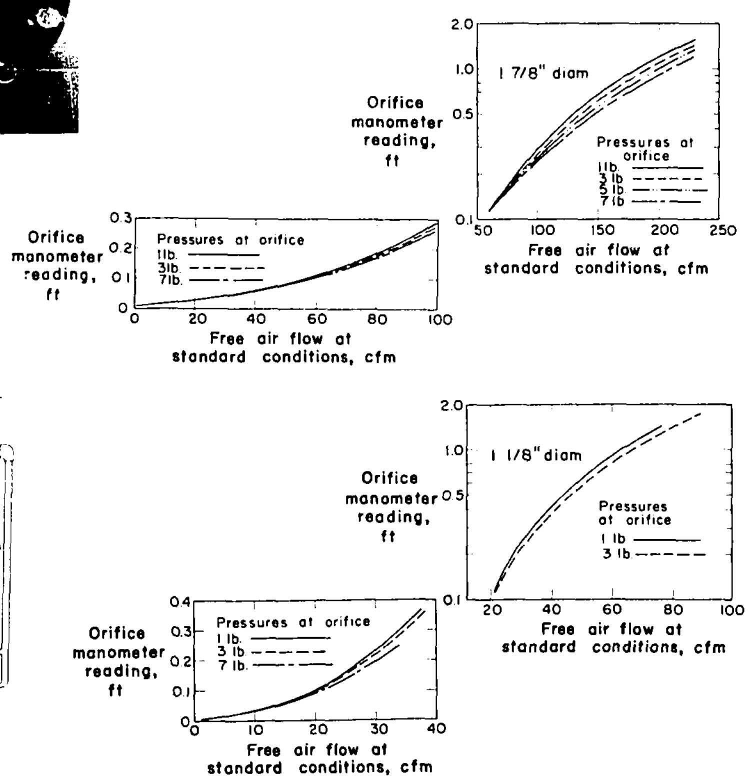


Fig. 34. Views of air orifice calibration arrangement



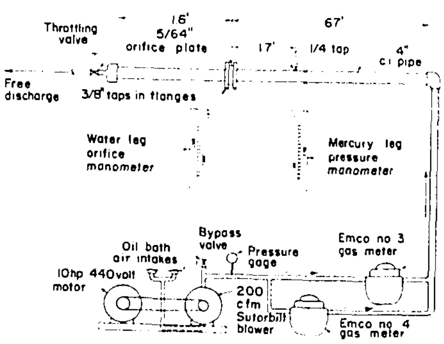


Fig. 35. Schematic diagram of orifice calibration arrangement

Fig. 36. Calibration curves for air orifices used in Ames plant preaeration studies

Walker Process aeration equipment could be measured with reasonable accuracy. Under ideal conditions, just after reversing the normal direction of rotation of the pump impeller to clear the impeller and air tubes of fouling materials, air intake was at a maximum of 15 cfm. A more normal average of 12 cfm was observed at random many times. The general tank circulation pattern at full natural draft was upward near the center, then out toward the corners and down again. This was in addition to the downdraft at the center of the tank evidenced by near-vortexing around the impeller shaft.

Test runs were made at reduced natural draft, with air intake throttled to about 8 cfm. The tank surface was then somewhat quieter, but fine bubble emergence was still widespread. The really surprising thing was that the pattern of general tank circulation was reversed; flow was upward in the corners, then moved inward toward the center of the tank. This is an interesting reflection on tank geometry.

Several plant runs were also made with the Walker Process downdraft unit completely starved of air, with the suction tubes capped. This was an approach to mechanical flocculation without the usual equipment for it. Virtually no air was carried down to reappear as surface bubbled. The pattern of tank circulation was as described above for reduced draft but even less active. It was apparent that the impeller design is based on optimum efficiency with the air-sewage mixture obtained at full natural draft (figure 37).

On one occasion, an effort was made to force-feed air to this unit by connecting the blower to the air suction manifold. Natural draft was

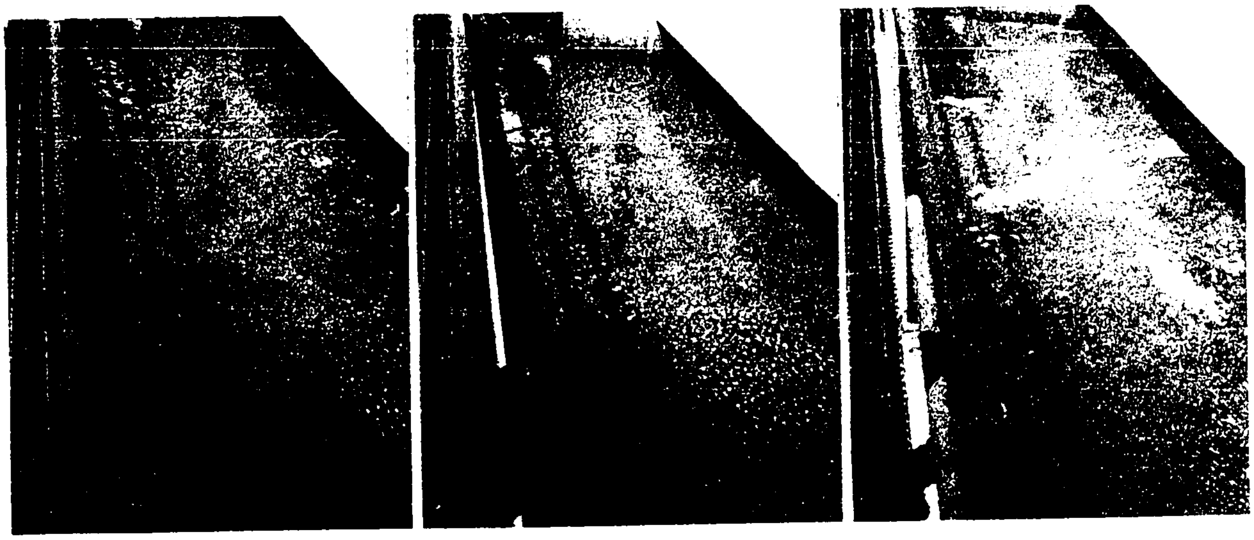


Fig. 37. Ames preaeration tank appearance at downdraft aeration rates of 0, 8 and 12 cfm

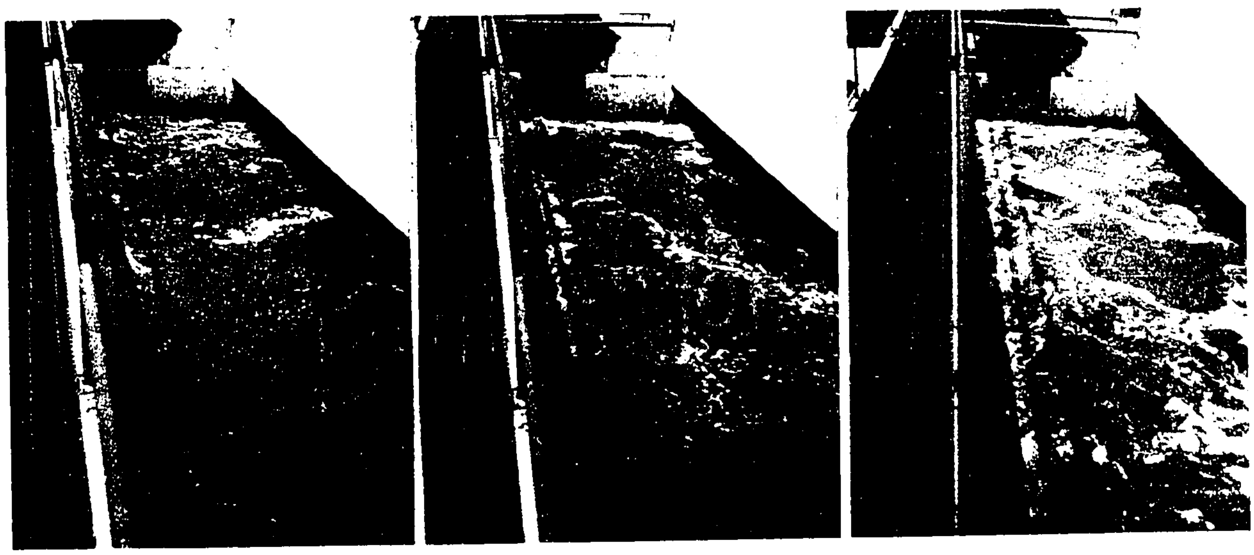


Fig. 38. Ames preaeration tank appearance at blower aeration rates of 65, 125 and 200 cfm

recorded at 15 cfm. As additional air was gently supplied, there was some evidence of interference at 18 cfm; larger bubbles were rising near the outside of the downdraft tube and the sewage surface appearance was not normal. At just past 30 cfm, all semblance of normal circulation and aeration disappeared, to be replaced by violent eruption of air pockets directly above the downdraft tube. Force-feeding air to the Walker Process aerators was abandoned.

b. Walker Process SPARJERS. As an aeration device, the SPARJER is a small cast piece mounted over a tapped pipe opening (figure 39). The casting is designed with an orifice in each of its 4 lateral arms. Spargers furnished for this project were 7/32 in orifice size. Sixteen were mounted in a square grid pattern in the south preaeration tank at a depth equal to the outlet of the downdraft tube (figure 41). The downdraft aerator was not disturbed.

A number of runs were made with all sixteen spargers in service. Runs were also made with alternate spargers sealed, leaving only eight in service. The difference in operating pressure was negligible for the aeration rates used (figure 38).

It is important to point out that all the energy for tank recirculation was provided by the air alone. It appeared that an air rate of 40 to 50 cfm was the minimum required for maintaining recirculation of the tank contents.

c. General Filter Company diffusion tubes. The diffusion tubes provided for this study were stock units manufactured by General Filter Company of Ames, Iowa, for use in diatomite filtration of water. Each

Fig. 39. Walker Process SPARGER used in Ames plant preaeration runs

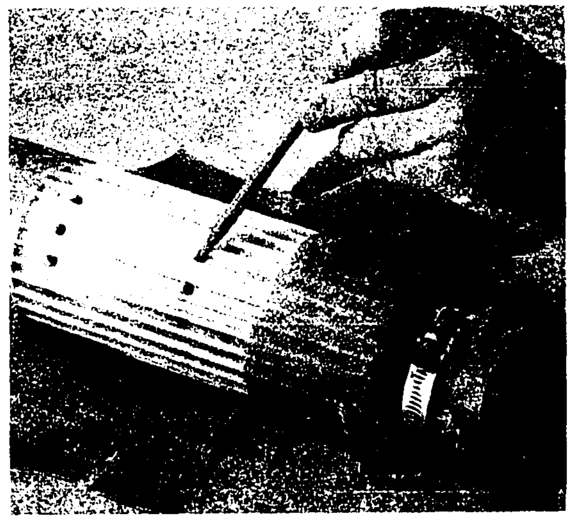


Fig. 40. General Filter Company diffusion tube used in Ames plant preaeration runs

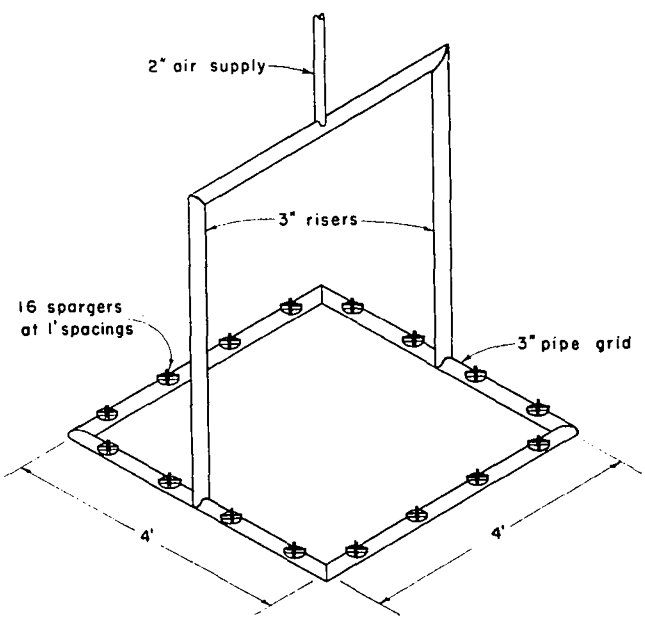


Fig. 41. Sketch of Walker Process SPARGER arrangement in Ames preaeration tank

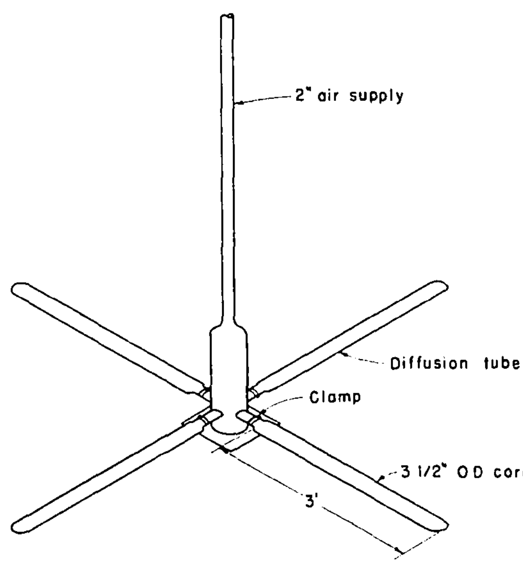


Fig. 42. Sketch of General Filter Company diffusion tube arrangement in Ames preaeration tank

diffusion tube (figure 40) is of a high-impact styrene core $3\frac{1}{2}$ in OD and 3 ft in length, with a sock-type covering of Dialon, a monofilament polyethylene material.

Four of these tubes were assembled (figure 42) around a welded cross served by a vertical air header. This assembly was placed at a depth equal to the outlet of the downdraft tube, without disturbing the aerator. The diffusion tubes were used only a short time but gave highly satisfactory service.

d. Comparison. The primary objective in using various aeration devices was to determine if the preaeration process was responsive to such changes. Evaluation of these devices in terms of aeration efficiency was a secondary consideration.

X. PLANT-SCALE STUDIES, MARCH-AUGUST 1958

A. Plant Arrangement and Operation

With trial plant runs completed and with sampling and laboratory procedures standardized, the stage was set for a lengthy series of full-scale plant runs. The objectives of these runs were to establish whether or not preaeration was of benefit to primary settling in actual plant operation, and to determine whether the aeration rate was an important factor.

1. Plant operation

Arrangement of the plant units was substantially the same as for the July 1957 test series. Incoming raw sewage flow was split accurately, half bypassing directly to the north primary clarifiers, while the other half was directed to the south preaeration unit and then to the south primaries. All four primaries were operated in a normal manner, with sludge withdrawal at 6 hr intervals.

The three trickling filters and three final clarifiers were in operation throughout the 1958 test series. Final settling sludge was held in the final clarifiers during the week and run back to the raw sewage wet well only on weekends. Digester and sludge-well supernatants were drained to the sludge lagoon throughout these runs. These arrangements were designed to provide for the operation of the primary treatment units without recirculation or return of material of any kind to the raw sewage.

To avoid a recurrence of the grit problem which so thoroughly disrupted the first plant runs, the primary sludge hoppers were jetted free of sludge and grit each Monday morning. At the same time, the south preaeration unit was completely emptied and flushed down. These precautions were successful in eliminating any accumulation of grit or septic sludge during these runs.

2. Preaeration arrangements

Plant studies were begun with the original downdraft aerator in place in the south preaeration tank. A number of replicate runs were made with this aerator operated at natural draft of about 12 cfm, with the air intake throttled to about 8 cfm, and with the draft tubes capped to exclude any aeration in the normal manner. With the draft tubes capped, the only aeration accomplished was through surface agitation. This condition was intended to simulate mechanical flocculation insofar as possible at this plant.

Following installation of the spargers and diffusion tubes in the preaeration units as previously described, plant studies were conducted with these devices at aeration rates from 65 to 200 cfm. Air for these runs was provided by the Sutorbilt blower and measured through a calibrated orifice. For the spargers and diffusion tubes, all energy required for recirculation of the tank contents must be provided through aeration, while most of this energy is furnished by a turbine pump in the downdraft aerator.

Raw sewage flow to the plant during the 1958 studies varied from 2.05

to 2.81 mgd, averaging roughly 2.4 mgd. This does not include a short period in July following extreme rains and unusual groundwater conditions when the flow to the plant exceeded 3 mgd.

The south preaeration tank, with a capacity of 40,000 gals, provided a theoretical displacement time of 48 mins at the 2.4 mgd average flow rate. This corresponds closely to the 45 mins usually recommended in design. Air rates, based on the 2.4 mgd average flow, ranged from 0.08 cu ft/gal at 65 cfm to 0.24 cu ft/gal at 200 cfm. Details on aeration rates and sewage flows appear in the tabulation of operating results.

B. Sampling and Laboratory Procedures

1. Sampling schedule

During much of the 1958 plant studies, sampling began at four points at 10:30 AM andcontinued through 9:30 PM. The samples were composited generally in step with flow-through times, as shown. The result was a reasonably faithful record of the same 10 hr flow as it received primary treatment.

Sampling point	Composite period
Preaeration influent (raw)	10:30 AM - 8:00 PM
Preaeration effluent	11:00 AM - 8:30 PM
Plain primary effluent	11:30 AM - 9:00 PM
Preaerated primary effluent	12 Noon - 9:30 PM

For some of the runs, sampling continued on an hourly basis through 10 AM the following morning. All samples were collected in 250 ml square, wide-mouth bottles and stored in a 4° C water bath cooler. After compositing in proportion to flow, photometric SS determinations were made on each individual sample.

Sampling for DO levels and oxygen depletion rates was confined to mid-afternoon when sewage flow and strength were reasonably constant. After the procedure became standardized, DO samples were taken at ten minute intervals for an hour, and samples for depletion tests were taken near the start, middle and end of that hour.

Determinations of ORP were not made during the March-August period, but were confined to a short period of special DO and ORP studies near the end of 1958. For ORP, samples were taken at intervals of forty minutes to an hour, as time permitted.

For settling rate studies with the Oulman settlimeter, samples of preaeration influent and effluent were collected with a time lag reflecting the probable mean detention time in the preaeration tank. This procedure was intended to determine the settling characteristics of essentially the same increment of flow as it arrived at the plant and again as it left the preaeration tank. Most of the settlimeter work was also done in the afternoon.

2. Laboratory work

Reference is made to Section IX, Final Laboratory and Test Procedures, for details of the laboratory procedures followed during the 1958 plant studies.

C. Results

1. Primary removals; data summary only

The results of all the 1958 Ames plant runs are plotted in figure 43 in chronological order, showing the SS strength of each individual sample of preaeration influent and effluent and plain and preaerated primary effluent. Also indicated on the daily plot are the preaeration system in use and the aeration rate.

Data on composite samples for these runs are summarized in table 19a, also arranged chronologically. This tabulation includes the flow rate, the preaeration arrangement, BOD and SS analyses on composites from the four sampling points, and calculated BOD and SS removals based on the average of preaeration influent and effluent strengths.

Following several weeks' work with the original downdraft aeration equipment, spargers were installed. Plant tests were conducted first with sixteen, then with only eight spargers in operation. The tests were disrupted for two weeks by heavy rains and sewer flooding which necessitated bypassing the plant completely for a few days. The plant runs made with General Filter Company diffusion tubes following this interruption were influenced somewhat by the high flows and reduced sewage strength reflecting heavy groundwater infiltration.

The spargers were then used again for a short series of tests, followed by a final two weeks' study of the downdraft aeration system. The reason for repeating the same types of runs over and over, sometimes under similar conditions, was that the results would indicate a trend of sorts

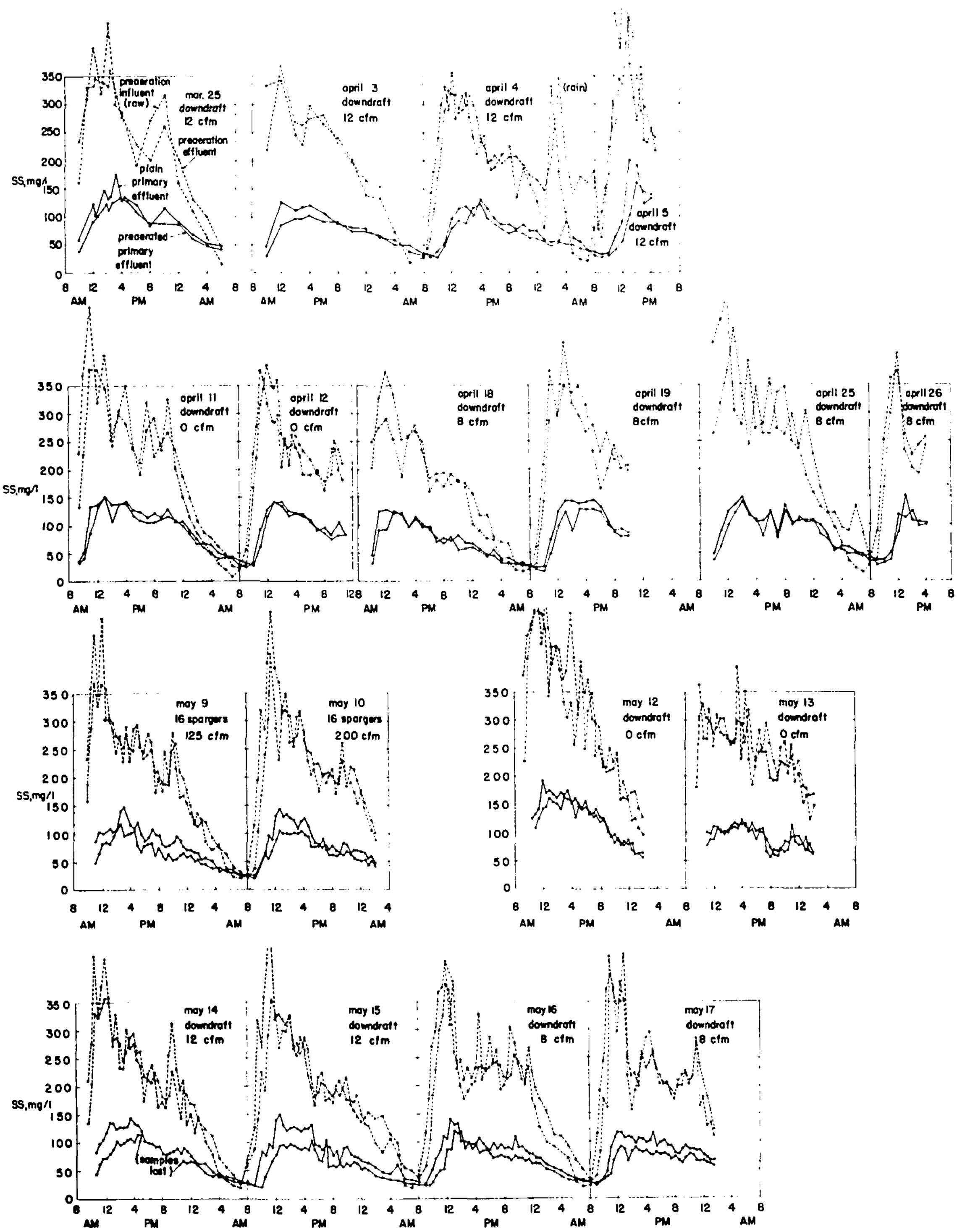


Fig. 43. Daily patterns of raw and primary effluent SS strength; 1958 Ames plant parallel runs

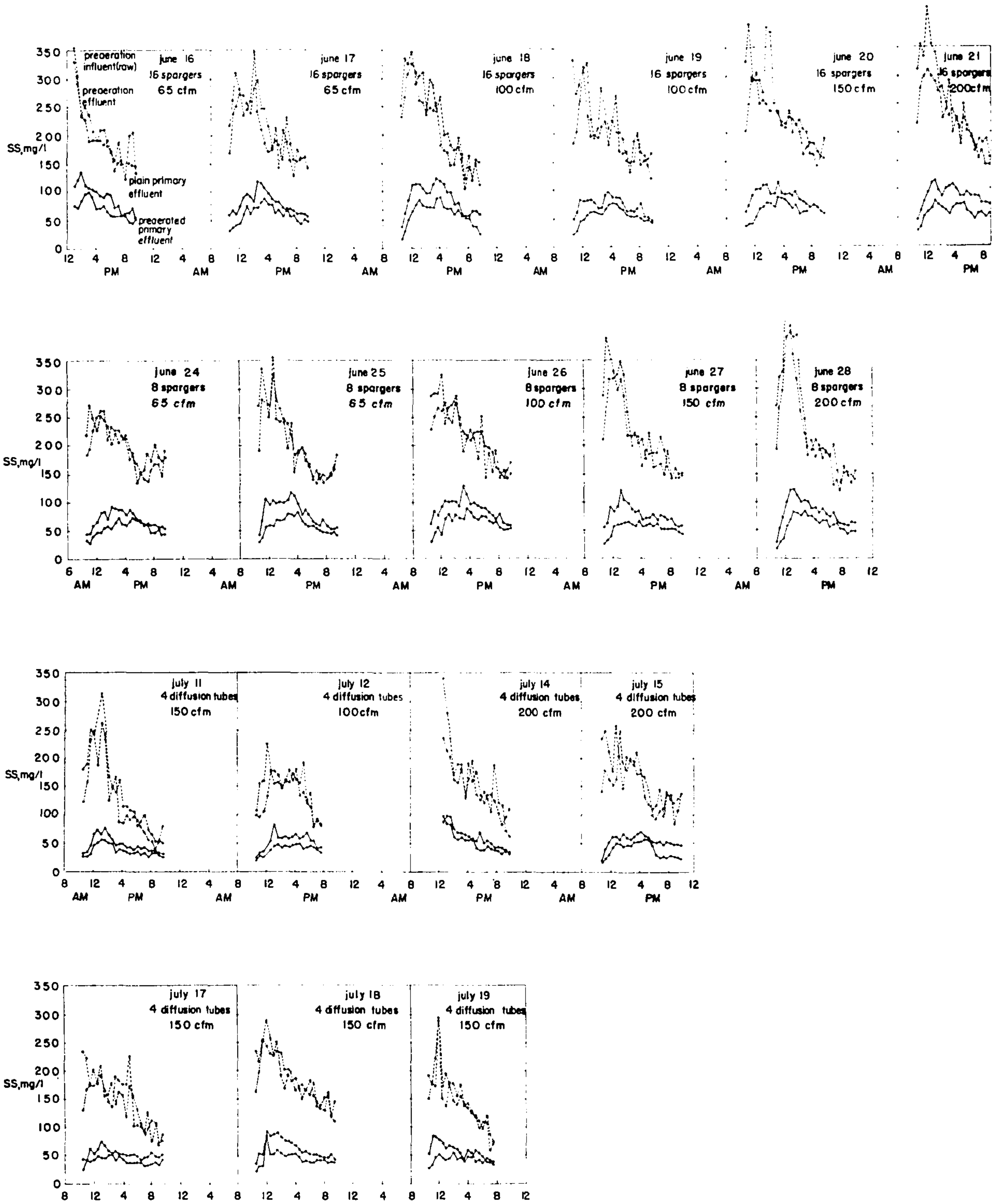


Fig. 43. Continued. Daily patterns of raw and primary effluent SS strength; 1958 Ames plant parallel runs

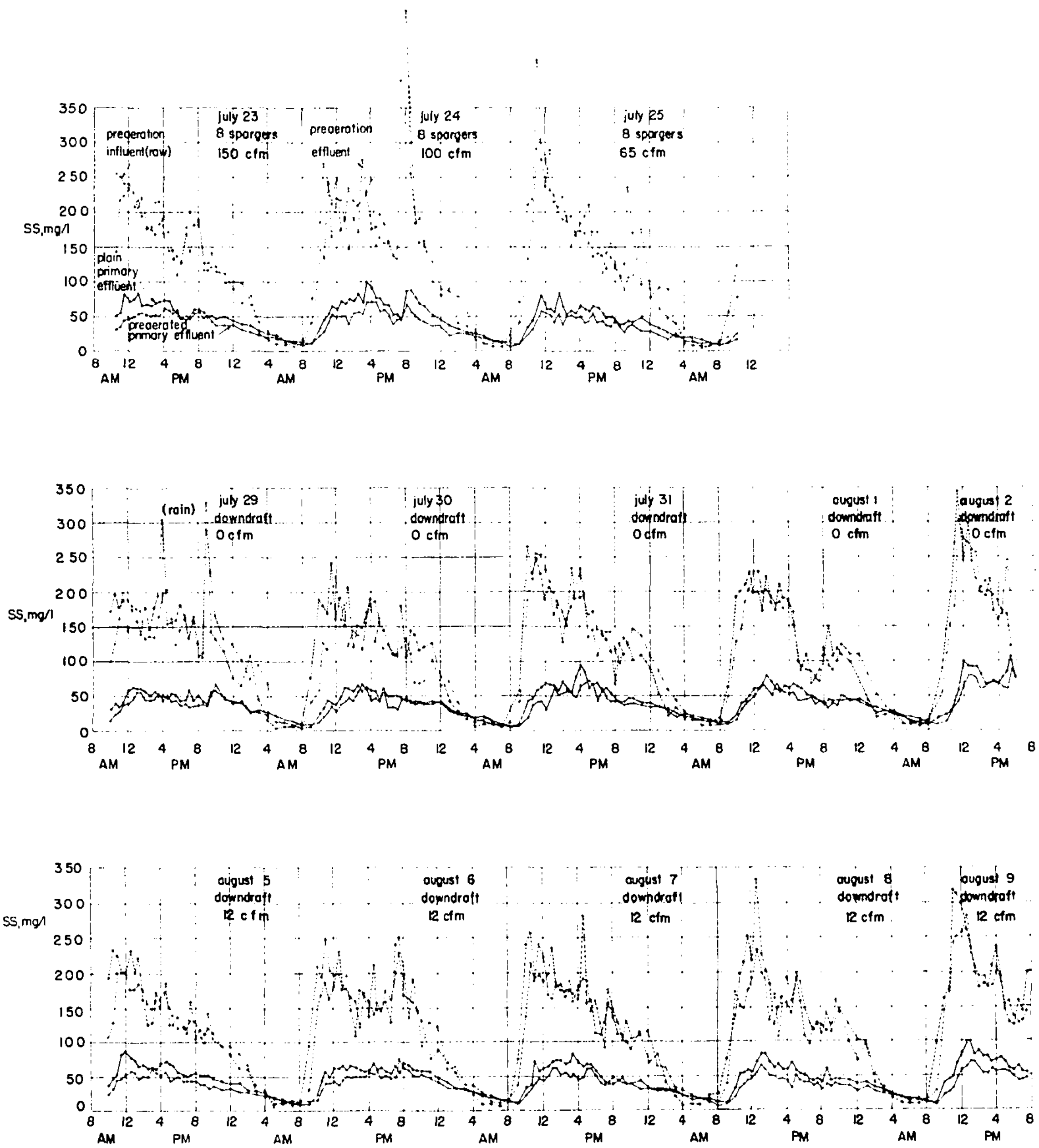


Fig. 43. Continued. Daily patterns of raw and primary effluent SS strength; 1958 Ames plant parallel runs

Table 19a. Summary of 1958 full-scale Ames plant runs

Date	Day	Raw Sewage flow, mgd	Preaeration arrangement	Air flow, cfm	Composite sampling period	SS method*	Analyses--all in mg/l: Preaerator Influent BOD	Preaerator Influent SS	Preaerator Effluent BOD	Preaerator Effluent SS	Primary 1 + 2 effluent (plain settling) BOD	Primary 1 + 2 effluent (plain settling) SS	Primary 3 + 4 effluent (preaeration) BOD	Primary 3 + 4 effluent (preaeration) SS	Primary Removals**, all in %: BOD 1 + 2 effluent (plain settling)	BOD 3 + 4 effluent (preaeration)	SS 1 + 2 effluent (plain settling)	SS 3 + 4 effluent (preaeration)
1958																		
3/25	Tu	2.27	downdraft	12	24 hr	P	200	251	170	227	135	113	115	91	27.1	37.8	52.7	61.9
					(10 hr)	(P)		312		289		123		108			59.1	64.1
4/3	Th	2.33	downdraft	12	24 hr	P	295	239	300		210	95	190	95	29.4	36.2	60.2	60.2
					(10 hr)	(P)		282		284		111		94			60.8	66.8
4/4	F	2.07	downdraft	12	24 hr	P	160	222	190	225	140	78	140	73	20.0	20.0	65.1	67.3
					(10 hr)	(P)		260		264		103		92			60.7	64.9
4/11	F	2.58	downdraft	0	24 hr	P	187	205	200	220	135	105	140	98	30.2	27.7	50.6	53.9
					(10 hr)	(P)		317		290		132		122			56.5	59.8
4/12	Sa	2.32	downdraft	0	(10 hr)	(P)		250		265		112		110			56.5	57.2
4/18	F	2.53	downdraft	8	24 hr	P	150	201	150	193	145	93	132	78	3.3	12.0	52.8	60.4
					(10 hr)	(P)		232		260		101		98			58.9	60.1
4/19	Sa	2.34	downdraft	8	(10 hr)	(P)		290		282		124		108			56.6	62.2
4/25	F	2.49	downdraft	8	24 hr	P	225	248	255	248	150	109	160	98	37.5	33.3	56.1	60.5
					(10 hr)	(P)		348		324		117		112			65.2	66.7
5/9	F	2.59	16 spargers	125	10 hr	P	235	287	220	273	148	105	133	91	35.0	41.6	62.5	67.5
5/10	Sa	2.47	16 spargers	200	10 hr	P	290	266	265	266	235	97	215	80	15.3	22.5	63.5	69.9
5/12	M	2.83	downdraft	0	10 hr	P			300	450	216	160	210	141	'8.0	30.0	64.4	68.6
5/13	Tu	2.73	downdraft	0	10 hr	P	202	285	212	300	155	100	151	109	25.1	27.0	65.8	62.8
					10 hr	E		281		294		146		106			49.2	63.1
5/14	W	2.69	downdraft	12	10 hr	P	270	289	230	258	177	116	163	97	29.2	34.8	58.5	64.5
5/15	Th	2.55	downdraft	0	10 hr	P	248	268	235	257	151	101	145	93	37.5	40.0	61.5	64.5
					10 hr	E		244		286		129		92			51.3	65.3
5/16	F	2.61	downdraft	8	10 hr	P	215	278	190	258	160	110	150	100	21.0	25.9	59.0	62.7
					10 hr	E		232				106		68			54.3	70.7
5/17	Sa	2.43	downdraft	0	10 hr	P	238	295	230	273	150		136		35.9	41.9		
5/27	Tu	2.44	16 spargers	125	10 hr	P	340	250	350	271	275	105	275	88	20.3	20.3	59.7	66.2
					10 hr	E		225		245		104		78			55.7	66.8
5/28	W	2.40	16 spargers	125	10 hr	P	330	243	320	266	219	106	174	79	32.6	46.5	58.5	69.0
					10 hr	E		228		247		114		76			52.0	67.1
5/29	Th	2.43	16 spargers	125	10 hr	P	256	315	232	305	195	122	177	106	20.1	27.5	60.6	65.8
					10 hr	E		295		315								
6/16	M	2.41	16 spargers	65	8 hr	P	196	223	202	217	162	82	124	69	18.6	37.7	62.7	68.6
					8 hr	E		196		238		88		63			59.5	71.0
6/17	Tu	2.26	16 spargers	65	10 hr	P	214		206		130		128		38.1	39.0		
					10 hr	E		210		216		78		68			63.4	68.1
6/18	W	2.30	16 spargers	100	10 hr	P	252	247	244	221	180	82	169	72	27.4	32.4	64.9	69.2
					10 hr	E		221		235		102		86			55.3	62.3
6/19	Th	2.37	16 spargers	100	10 hr	P	210	205	205	181	189	72	180	57	8.9	13.3	62.7	70.5
					10 hr	E		180				99		63			45.0	65.0
6/20	F	2.21	16 spargers	150	10 hr	P	220	234	213	222	140	81	120	54	35.3	44.5	64.5	76.3
					10 hr	E		242		248		82		63			66.5	74.4
6/21	Sa	2.05	16 spargers	200	10 hr	P	235	233	235	239	140	91	135	67	40.4	42.5	61.4	71.6
					10 hr	E		249		230		93		56			61.2	76.7
6/24	Tu	2.17	8 spargers	65	10 hr	P	210	191	210	180	136	74	120	56	33.7	41.5	60.1	69.8
					10 hr	E		214		208		83		60			60.7	71.5
6/25	W	2.16	8 spargers	65	10 hr	P	206	208	228	200	156	78	122	63	28.1	43.8	61.7	69.1
					10 hr	E		219		221		75		71			66.0	67.7
6/26	Th	2.15	8 spargers	100	10 hr	P	210	225	195	210	159	90	150	69	21.5	25.9	58.5	68.2
					10 hr	E		213		227		101		50			54.1	77.3
6/27	F	2.20	8 spargers	150	10 hr	P		222	170	215	118	80	108	61	30.6	36.5	63.4	72.1
					10 hr	E		215		220		86		62			60.5	71.5
6/28	Sa	2.13	8 spargers	200	10 hr	P	180	252		254		97		70			61.7	72.3
					10 hr	E		243		269		85		69			66.9	73.0
7/11	F	3.21	4 diffusion tubes	150	10 hr	P	125	140	127	138	100	45	70	35	20.6	44.4	67.6	74.8
7/12	Sa	2.89	4 diffusion tubes	100	8 hr	P	136	160	128	146	96	56	70	48	27.3	47.0	63.4	68.6
7/14	M	3.00	4 diffusion tubes	200	8 hr	P	153	155	145	147	110	58	90	44	26.2	39.6	61.6	70.9
7/15	Tu	2.76	4 diffusion tubes	200	10 hr	P	160	143	150	153	90	60	80	43	36.7	45.1	59.5	70.9
7/17	Th	2.55	4 diffusion tubes	150	10 hr	P	150	160	155	156	85	52	75	43	44.3	50.8	67.1	72.8
7/18	F	2.69	4 diffusion tubes	150	10 hr	P	170	195	150	184	95	61	88	48	40.6	45.0	67.8	74.6
					10 hr	E		190		204		74		58			62.4	70.5
7/19	Sa	3.09	4 diffusion tubes	150	8 hr	P	150	167	180	152	135	49	115	37	18.2	30.3	69.3	76.8
7/23	W	2.81	8 spargers	150	10 hr	P	174	186	170	188	113	67	89	49	34.3	48.3	64.2	73.8
					24 hr	P	130	147	120	133	82	48	70	37	34.4	44.0	65.7	73.6
7/24	Th	2.79	8 spargers	100	10 hr	P	152	292	150	246	120	67	100	50	20.5	33.8	75.1	81.4
					24 hr	P	121	208	120	186	86	62	67	43	28.6	44.5	68.5	78.2
7/25	F	2.67	8 spargers	65	10 hr	P	156	195	145	197	94	67	70	53	37.5	53.5	65.8	73.0
					24 hr	P	117	137	115	137	85	52	72	40	26.7	37.9	62.1	70.8
7/29	Tu	2.66	downdraft	0	10 hr	P	160	179	160	177	88	53	88	43	45.0	45.0	70.2	75.8
7/30	W	2.75	downdraft	0	10 hr	P	125	147	120	134	70	45	70	43	42.9	42.9	68.0	69.4
7/31	Th	2.80	downdraft	0	10 hr	P	140	184	150	185	99	72	87	59	31.7	40.0	61.0	68.0
8/1	F	2.74	downdraft	0	10 hr	P	150	168	140	174	90	63	90	54	38.0	38.0	63.1	68.4
8/5	Tu	2.62	downdraft	12	10 hr	P	143	176	145	172	99	62	82	52	31.3	43.0	64.3	70.1
8/6	W	2.57	downdraft	12	10 hr	P	140	196	142	182	96	66	84	54	31.9	40.4	65.1	71.4
8/7	Th	2.67	downdraft	12	10 hr	P	188	196	188	204	140	76	125	58	25.6	33.5	62.0	71.0
8/8	F	2.59	downdraft	12	10 hr	P	135	190	140	171	90	61	85	48	34.6	38.2	66.2	73.4
8/9	Sa	2.42	downdraft	12	8 hr	P	145	208	150	192	109	79	96	59	26.1	34.9	60.5	70.5

*P = photometric; (P) = photometric; calculated composite; E = evaporation.
**Calculated removals based on average of Preaerator influent and effluent strengths.

Table 19b. Notes to summary of 1958 plant runs

Date	Remarks
3/25	Reduced flow; between quarters at Iowa State University.
4/3, 4	Flow again reduced; Easter weekend. Wind and some showers.
5/9, 10	First plant-scale operation with Sutorbilt blower and Walker Process SPARGERS.
5/16, 17	Veishea weekend at Iowa State University, with some effect on load patterns.
5/27-29	For this 3 day run, SS determinations were not made on the individual samples; therefore, the daily plots were omitted.
5/29	Last day of public school system year.
6/13	Last day of spring quarter at Iowa State University.
7/1, 3	Very heavy rains, flooding sewer system, sending creek out of its banks and causing shutdown of sewage treatment plant until July 6.
7/10	Preliminary test with General Filter Company diffusion tubes in place.
7/11, 12	Sewage flow and strength still influenced strongly by groundwater infiltration.
7/24	Heavy rain at 6:15 PM.
8/9	Conclusion of 1958 plant-scale runs at Ames plant.

one week, then a contrary trend the following week. The primary variable throughout these plant runs was the rate of air supplied, and every effort was made to arrive at a valid conclusion as to its effect on the process.

A comment may be in order on the effect of the preaeration step itself on the raw sewage strength. There appeared to be none. Over the weeks of tests, preaeration effluent would sometimes be higher in BOD and

SS than the influent, sometimes lower. There was no pattern to these variations, however, and their overall result was in balance. Grit removal was effectively accomplished by preaeration, and this was earlier determined to be of vital importance to primary settling, but it did not amount to as much as 1 percent removal of SS.

The preaeration step did provide a mild buffering effect on the raw sewage flow, in that peak SS strength of the preaeration effluent was often noticeably less than that of the influent. However, the peaks and valleys in preaeration effluent SS patterns coincided fairly well with influent patterns, separated by the expected time lag reflecting detention time. In the early morning hours, both preaeration influent and effluent normally dropped to a SS level of 5 to 20 mg/l. There was no evidence of a solids accumulation or inventory in the preaeration tank. The most casual scanning of the plotted results will indicate that preaeration almost always proved an aid to primary settling.

2. Oxygen values

a. Tabulation. After procedures were standardized, sampling for oxygen values was confined to the midafternoon, when DO levels and depletion rates were found to be most consistent (table 20).

The depletion rate procedure, described in detail earlier, was a rather arbitrary test intended to give a measure of the short-term oxygen demand of the raw sewage. By violent shaking, a DO level of 6 to 8 mg/l was induced in the sample. Analysis for DO at 5 or 10 min intervals thereafter indicated the rate of demand or depletion. Under normal

Table 20. Summary of oxygen values for Ames plant

Date, 1958	Day	Preaeration arrangement	Air flow, cfm	Inflow sampling time, PM	Inflow rate, mgd	Dissolved oxygen levels in preaerator mg/l Influent	Effluent	Observed depletion rate, mg/l/hr	Calculated acceptance rate, mg/l/hr	Oxygen transfer efficiency, %*
3/24	M	downdraft	12	2:10-3:30	3.0	-0.2	1.2	6	9	20
3/25	Tu	downdraft	12	1:00-3:00	3.1	1	2.5	6.5	9.3	21
4/4	F	downdraft	12	2:30-3:30	2.8	1	2			
4/5	Sa	downdraft	12	1:30-3:00	2.7	1	3			
4/12	Sa	downdraft	0	2:00-3:20	3.2	1	-2			
4/19	Sa	downdraft	8	3:00-4:30	3.1	0.0	0.8	5.5	7.1	24
4/26	Sa	downdraft	8	1:40-2:40	3.3	0.4	1.8	5.0	7.8	26
5/9	F	16 spargers	125	2:20-2:50	3.4	0.1	1.9	6.8	10.6	2.3
5/10	Sa	16 spargers	200	2:20-3:40	3.2	0.2	2.8	6.2	12.2	1.6
6/16	M	16 spargers	65	2:20-3:20	3.1	0.6	-0.2	6.5	4.6	1.9
6/17	Tu	16 spargers	65	2:40-3:40	3.0	0.6	0.2	6.8	6.0	2.5
6/18	W	16 spargers	100	3:00-4:00	3.1	0.0	0.0	6.1	6.1	1.6
6/19	Th	16 spargers	100	2:40-3:40	3.2	0.4	0.4	6.3	6.3	1.7
6/20	F	16 spargers	150	2:40-3:40	2.9	0.2	2.3	6.3	10.1	1.8
6/21	Sa	16 spargers	200	2:40-3:40	2.7	0.2	1.8	6.7	10.0	1.3
6/24	Tu	8 spargers	65	2:30-3:30	2.9	0.2	-0.1	5.2	4.7	1.9
6/25	W	8 spargers	65	2:20-3:20	3.0	0.3	0.2	6.6	6.2	2.5
6/26	Th	8 spargers	100	2:20-3:20	2.9	0.2	0.8	6.3	7.3	1.9
6/27	F	8 spargers	150	2:20-3:20	2.9	0.6	2.1	6.7	9.4	1.7
6/28	Sa	8 spargers	200	2:20-3:20	2.8	0.1	2.4	6.7	10.7	1.4
6/30	M	downdraft	12	2:00-3:00	3.2	0.0	-0.2	7.5	7.1	16
7/1	Tu	downdraft	8	2:20-3:20	3.1	0.4	0.2	6.9	6.6	22
7/11	F	4 diffusion tubes	150	2:20-3:20	4.0	0.6	2.9	3.8	9.4	1.7
7/12	Sa	4 diffusion tubes	100	2:00-3:00	3.6	0.5	0.2	4.3	3.7	1.0
7/14	M	4 diffusion tubes	200	2:20-3:20	3.8	0.0	2.4	4.5	10.0	1.3
7/19	Sa	4 diffusion tubes	150	2:00-3:00	3.8	0.5	2.4	3.6	8.0	1.4
7/29	Tu	downdraft	0	2:10-3:10	3.3	0.6	-0.3	4.9	3.0	
12/22	M	downdraft	12	2:20-3:20	2.6	0.5	0.3	6.7	6.4	14
12/23	Tu	8 spargers	200	1:45-2:55	2.4	2.5	4.5	8.0	11.0	1.5
12/24	W	downdraft	0	2:00-3:20	2.2	0.5	0.0	6.5	5.8	

*Assuming no surface aeration

afternoon conditions, this rate was consistently in the range of 6 to 7 mg/l/hr.

Raw sewage reaching the plant in midafternoon was always low in DO, usually $\frac{1}{2}$ mg/l or less, but rarely past the point of depletion to a state of negative DO. After preaeration, the DO level was less, sometimes negative, at low aeration rates; roughly the same at moderate aeration rates, and 2 to $2\frac{1}{2}$ mg/l higher at maximum aeration capacity of 200 cfm.

From observations on preaeration influent and effluent DO levels and on DO depletion rates, it was possible to calculate the rate at which DO was being accepted by the sewage during preaeration. Even under the maximum air rate of 200 cfm, this rate of acceptance did not exceed 10 to 11 mg/l hr, or roughly twice the usual depletion rate. Finally, the aeration or oxygen transfer efficiency was calculated in terms of the percent of oxygen supplied mechanically which was accepted by the sewage during preaeration.

b. Analysis. Aeration or oxygen transfer efficiencies of 4 to 6 percent are considered excellent performance in activated sludge practice with conventional air diffusion equipment. In the activated sludge process, however, both the biological population and the food supply are maintained at high levels by recirculation of active material. Despite aeration rates of 1 to $1\frac{1}{2}$ cu ft/gal or more, DO levels usually hover at 1 mg/l or less, and the rate of air supply, or more accurately the rate of oxygen transfer, emerges as the limiting factor in the process.

Analysis of the calculated oxygen transfer efficiencies for the 1958 Ames plant studies proved interesting. Assuming first that all the oxygen accepted by the sewage was introduced by the aeration equipment, efficiencies for the downdraft aerator ranged from 20 to 25 percent, but for the spargers only about 1½ to 2 percent. The calculated efficiency in each fell as the aeration rate was increased.

Close inspection of the data revealed that oxygen acceptance did not increase proportionally with aeration rates. This implied that oxygen was also being absorbed by the sewage simply through surface agitation. Actually, this has long been known to occur, and was further confirmed by data from downdraft aeration runs with the air draft tubes completely capped.

Even at full natural draft, surface aeration apparently accounted for about half the oxygen transfer achieved by the downdraft equipment. The same relationship was indicated for the spargers and diffusion tubes at the 100 cfm aeration rate. At lower aeration rates, surface transfer was of more benefit than sub-surface diffusion. Table 21 presents a summary of aeration rates and calculated oxygen transfer efficiencies, with and

Table 21. Ames plant oxygen transfer efficiencies

Preaeration system	Aeration rate cfm	Aeration rate cu ft/gal*	Average oxygen transfer efficiency percent: Assuming no surface aeration	Average oxygen transfer efficiency percent: Deducting effect of** surface aeration
Downdraft	8	.007	25	10
	12	.011	20	10
Spargers	65	.054	2.2	1.2
	200	.18	1.4	1.1

*Calculated for 3.2 mgd daytime flow.

**Surface aeration estimated to contribute 4-5 mg/l/hr during downdraft operation; 2-3 mg/l/hr during sparger operation.

without allowance for surface aeration. It is particularly interesting to note that, when surface aeration is evaluated separately, calculated transfer efficiencies are essentially constant regardless of aeration rate. In a physical rather than biological process step such as this, such a finding appears reasonable.

The important lesson from this work on oxygen values may be that it is not possible to transfer DO in more than token amounts to the sewage at this point because the oxygen demand, in the form of active biological life and a corresponding food supply, simply does not exist. To maintain the incoming DO level during preaeration, an aeration rate of about 0.1 cu ft/gal is indicated for the Ames plant. Regardless of the preaeration method, surface agitation appears to be an important source of oxygen transfer.

3. Settling characteristics; Oulman settlimeter

a. Purpose. The settlimeter was not developed to establish the benefit of preaeration to primary settling at the Ames plant. This was determined conclusively by parallel operation with and without preaeration. Rather, the objective was a device to reflect in the laboratory actual settling performance in the plant. Without such a procedure there was little to be gained by studying preaeration at other Iowa sewage treatment plants where the flow could not be divided as at Ames. The Oulman settlimeter met this objective.

b. Typical data plots. A dozen typical plots of paired settlimeter data are shown in figure 44. Each consists of the settling curve for a

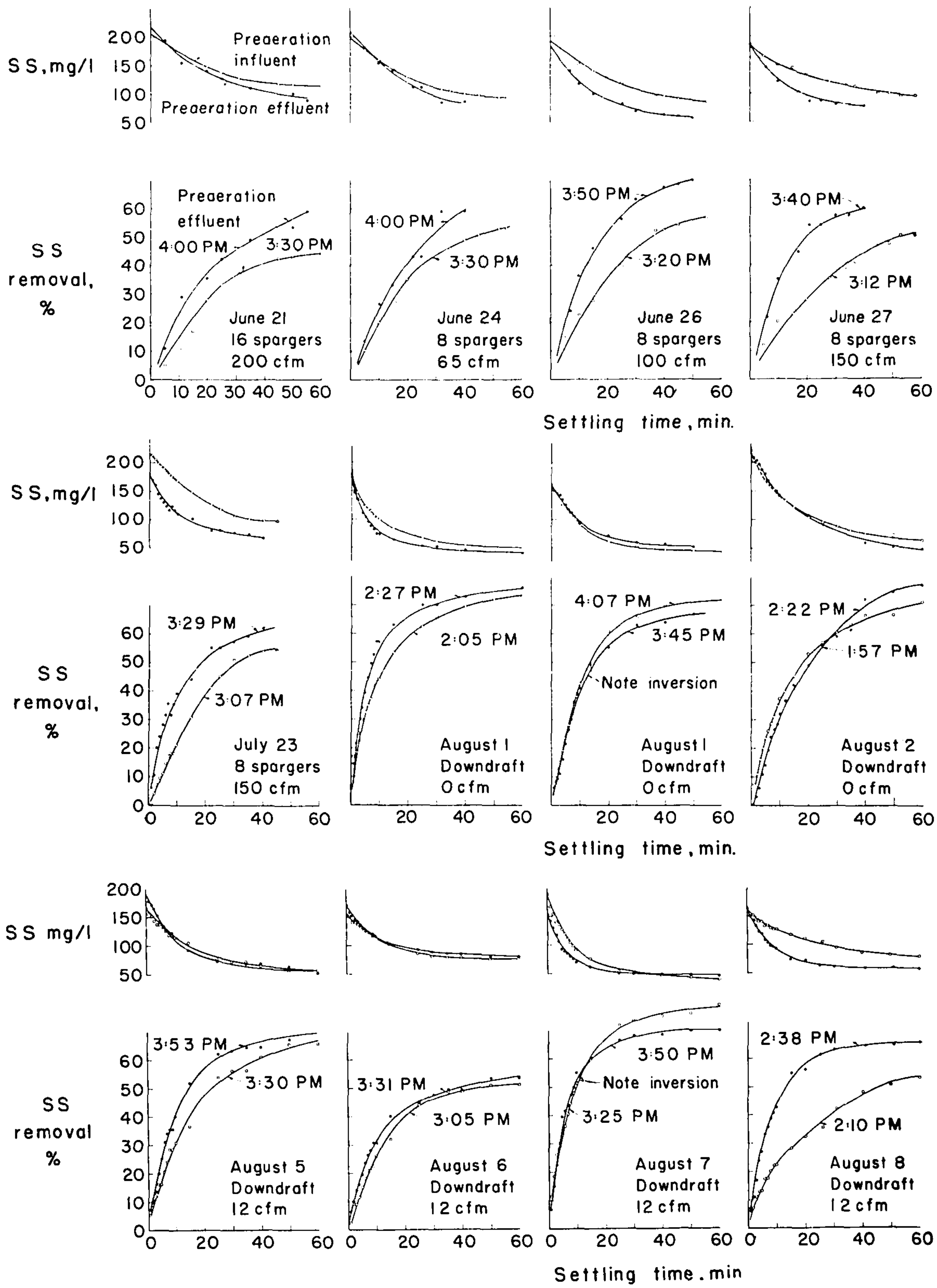


Fig. 44. Representative settling characteristics of preaeration influent and effluent samples at Ames plant, as determined by Oulman settlimeter

preaeration influent sample and a similar curve for a preaeration effluent sample collected following a time interval considered to be the probable aerator mean detention time. These plots span the most active period of plant tests and include results showing preaeration to be a good, bad and indifferent influence on primary settling. All represent midafternoon conditions of reasonably consistent sewage strength and flow.

A detailed discussion of the use of the settlimeter and the calculation of results appears in Appendix F. One interesting but very awkward problem arises here. The photometric SS determination by the Spectronic 20 Colorimeter is used in converting settlimeter readings in ua to SS content in mg/l. However, the photometric calibrations for raw and for settled sewage differ considerably. This means that the conversions must be based on a flexible calibration which will represent both the raw sewage sample at the outset of the settlimeter test and the well-settled sample at its conclusion, as well as the sample's changing character in the intervening period. Such a hybrid calibration will serve some tests faithfully, others not as well. Fortunately, this is of minor importance because the relative settling patterns of a data pair are only slightly affected by a change in calibration; the indication of benefit or lack of it is not easily masked.

c. Analysis. A general review of the paired data plots (figure 44) was made to determine how well these tests confirmed actual plant results (table 22).

For the first group of five tests, during which plant SS removals were consistently being boosted ten percentage points by preaeration, the

Table 22. Summary of settling characteristics as indicated by Oulman settlimeter*

Group of 1958 tests	Indication from settlimeter data plots	Results from plant-scale runs
June 21-July 23 5 tests	Strong benefit; average about 10 percentage points SS improvement	Average about 10 percentage points improvement; from 60 to 70 percent actual SS removal
August 1,2 3 tests	Little or no benefit; average a few percentage points SS improvement	Erratic; removals improved from nothing to about 5 percentage points SS
August 5,8 4 tests	Moderate benefit; from 8 percentage points to negative effect.	Average about 5 percentage points improvement; from 65 to 70 percent actual SS removal.

*For which settlimeter data are plotted in figure 44.

settlimeter plots also indicated from eight to twelve percentage points higher SS removal. On August 1 and 2, plant removals were being very little improved by preaeration, and this was sharply reflected by settlimeter test patterns which were much alike before and after preaeration. During August 5 to 8, removals were only moderately improved by preaeration, possibly because plain settling was doing such an excellent job. This too was reflected in overall settlimeter results, although several of the tests for these days exhibited abnormal settling patterns. It appears that the Oulman settlimeter provides a generally valid indication of settling characteristics.

In the paired data plots where preaeration markedly improved settling, most of the advantage, sometimes all of it, was gained in the first fifteen to twenty minutes of settling. If this is also true in plant scale, its implications for both design and operation are of vital importance.

4. Oxidation-reduction potential

a. Representative data. Determination of ORP was not done during the main series of plant tests but was limited to a few days' work late in 1958. The purpose of this very limited study was to gain an indication of ORP levels and their response to preaeration.

During several days of operation with all raw sewage flow receiving preaeration, grab samples were collected and the rate of change of ORP was observed. Then, with the flow split as for the plant test series, samples from the usual four points were collected hourly or oftener and analyzed for both DO and ORP. Data for December 1, with total flow preaeration, and for December 22-24 with split flow, are plotted in figure 45. During the 3 day run with split flow, preaeration rates were 12 cfm, 200 cfm and 0 cfm respectively in that order.

b. Analysis. ORP is a measure of the tendency of a physical or chemical system to yield electrons (oxidation) or to gain them (reduction). Since most bacterial cultures are reducing systems due to the metabolism of their organisms, raw sewage is either negative in potential or becomes so rapidly upon depletion of its original DO content. The more highly reducing a system, the greater will be its negative potential. Correspondingly, a rapid change in potential indicates a high rate of bacterial metabolism.

At the Ames plant, the daytime raw sewage ORP ranged 20 to 30 mv on either side of neutral. Its rate of change in the negative direction was

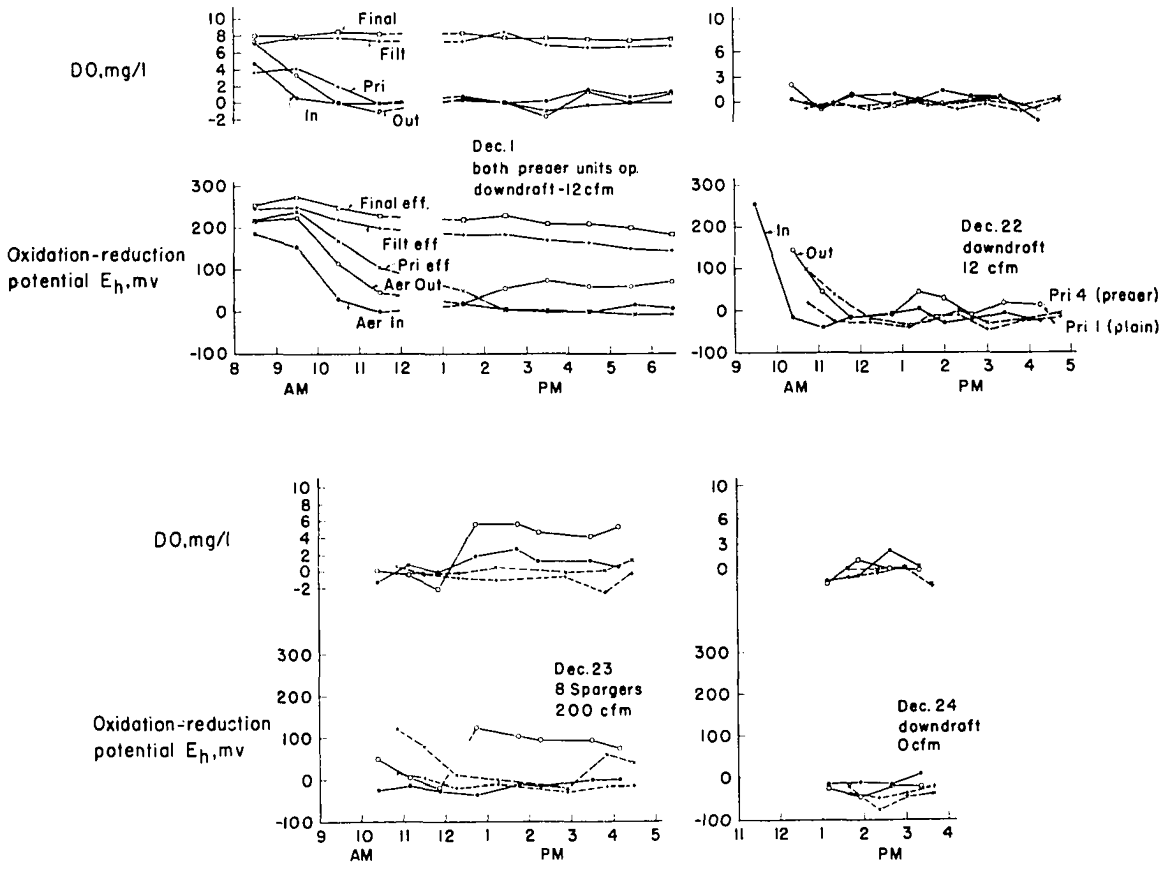


Fig. 45. Daytime patterns of ORP and DO levels at Ames plant, December 1958

30 to 50 mv/hr and this rate of change was essentially unaffected by preaeration or by primary settling. Following secondary treatment, however, the ORP level was found to be in the range of plus 200 mv, and its rate of change only 15 to 20 mv/hr, although still in the negative direction. These results, along with a DO level of 6 to 8 mg/l, confirm the character of the final plant effluent as a fairly stable, highly oxidized system.

Preaeration was found to boost ORP levels slightly in the positive direction even when failing to maintain DO constant. When a high aeration rate increased the DO, it raised the ORP to a sufficiently positive level to survive primary settling without reverting to a negative potential. Whether or not this is economically justified in terms of improved secondary stage treatability is beyond the scope of this report.

D. Analysis of Primary Removals

Whether properly or not, Will Rogers is credited with the remark that: "A batch of statistics is a lot like garbage; after you collect it you have to do something with it". This section is devoted to simple statistical treatment of the primary removal results for the 1958 Ames plant runs.

1. Plain primary settling removals

a. Effect of detention time. Before analysis of the benefit of preaeration, the results of plain settling were reviewed. Figure 46 is a plot of plain primary BOD and SS removals against mgd raw sewage flow, used as a parameter of detention time in the primary clarifiers. The

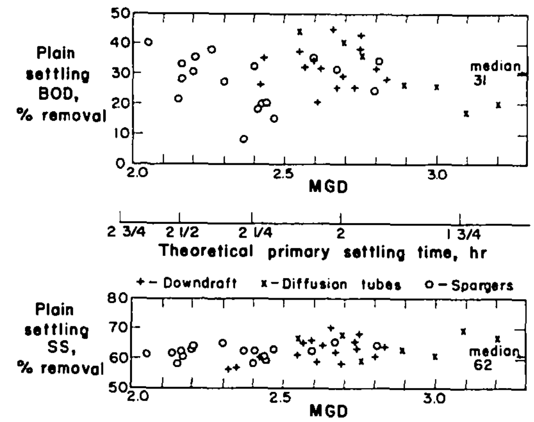


Fig. 46. Effect of detention time on plain primary removals; 1958 Ames plant runs

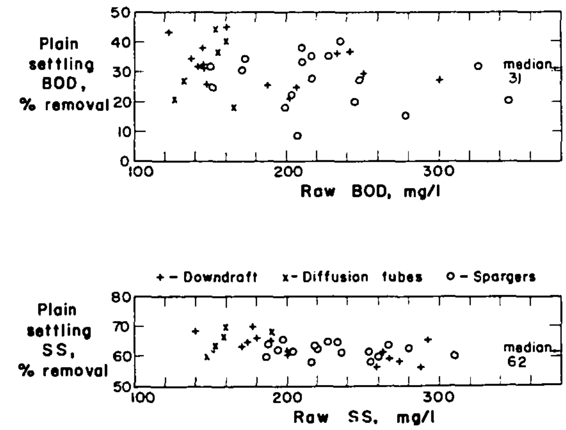


Fig. 47. Effect of raw sewage strength on plain primary removals; 1958 Ames plant runs

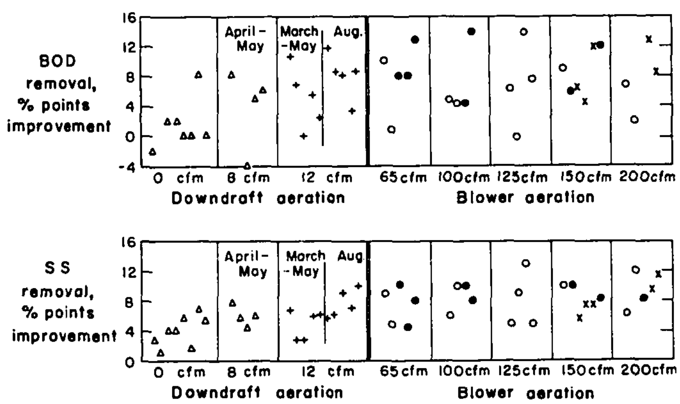


Fig. 48. Effect of aeration device or aeration rate on the improvement due to preaeration; 1958 Ames plant runs

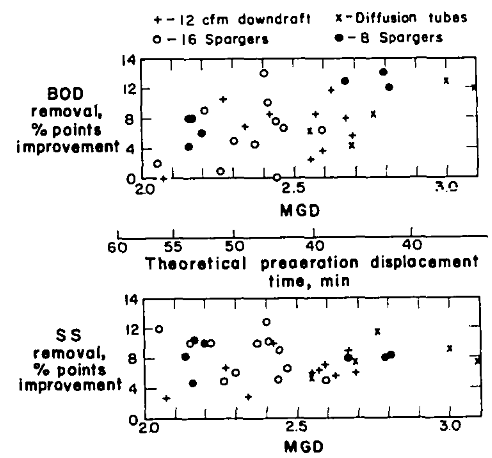


Fig. 49. Effect of preaeration detention time on the improvement due to preaeration; 1958 Ames plant runs

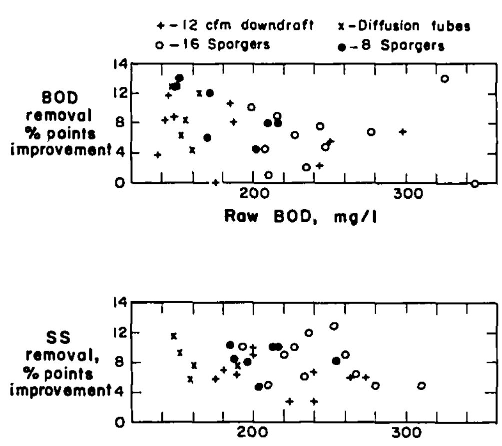


Fig. 50. Effect of raw sewage strength on the improvement due to preaeration; 1958 Ames plant runs

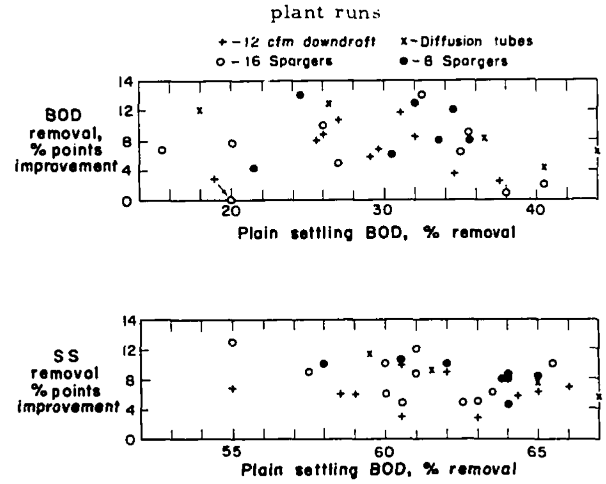


Fig. 51. Correlation of improvement due to preaeration with plain primary removals; 1958 Ames plant runs

scattering shown did not indicate any sort of trend. However, virtually all the data fall within limits of $1\frac{1}{2}$ to 2 hr settling detention time, a nominal variation in its effect on settling efficiency.

Median SS removal by plain settling was 62 percent. The overall range of SS results was from 56 to 70 percent removal, a surprisingly consistent grouping. BOD results, on the other hand, showed an extreme scatter of from less than 10 to more than 40 percent removal by plain settling. Some of the BOD data are more than slightly suspect because they conflict grossly with SS results for the same day. Comment was made earlier on the difficulties experienced with the BOD determination. Median BOD removal by plain primary settling was 31 percent.

b. Effect of sewage strength. Plain primary removals were further analyzed with relation to the strength of the raw sewage, using here for consistency the average of preaeration influent and preaeration effluent values. Plotted in these terms in figure 47, the data indicate a mild tendency toward improved plain settling removals with weaker sewage. Since the weaker strengths generally reflect higher flows and shorter detention times, this tendency is improbable. Close review of the data shows that inordinately high removals in the middle and last week of July, even in the face of reduced sewage strength, are responsible for this apparent trend. Disregarding the data of these two weeks, plain settling removals were essentially unaffected by raw sewage strength.

All data shown in figures 46 and 47 represent 10 hr composite sampling. From corresponding 24 hr results available, it appears that 24 hr SS removals averaged some three percentage points less than the 10 hr results. Insufficient comparative BOD data were available from which to draw conclusions.

2. Improvement by preaeration

a. Effect of method or rate of aeration. The degree of benefit attributed to preaeration is discussed here in terms of percentage points improvement in BOD or SS removals over that by plain primary settling. First, a comparison was made of the benefit achieved by downdraft aeration, by spargers and by diffusion tubes, at various aeration rates, disregarding other factors for the moment (figure 48).

In general, benefit from downdraft operation with air intake lines capped, at 0 cfm, was quite modest with regard to SS removal and appeared to be nil for BOD removal. With downdraft aeration at 8 cfm, improvement was good, and at 12 cfm it was better.

For blower aeration, neither the air diffusion arrangement nor the amount of air seemed to influence the results. For SS, the improvement ranged generally from 4 to 12 percentage points; the median value was 8 points. For BOD, the benefit varied more widely, from 0 to 14 percentage points, with a median value of 7½ points. It is interesting to note that, while plain settling shows a 2:1 ratio of SS to BOD removal, the benefit by preaeration is essentially the same for both.

Indications are that downdraft aeration may not be on a par with the more conventional diffusion methods (figure 48), but careful study of the data refutes this impression. The final week of operation at 12 cfm, corresponding closely in time and operating conditions to the blower runs, produced equally good results. Since operation in the spring at 8 cfm and at 12 cfm appeared to be comparable, it can be inferred that downdraft operation either at full or at reduced air intake is as beneficial, at least to primary removals, as the other methods tested.

Downdraft aeration with air intake lines capped, simulating mechanical flocculation, was of little benefit, but tank recirculation under these conditions was sharply reduced. This particular operating situation cannot be considered indicative of either preaeration or flocculation.

b. Effect of preaeration detention time. Having established that that aeration method or rate appeared to have no effect on the results, the next approach was that of detention time, via mgd flow to the plant. Figure 49 is a plot of improvement by preaeration against mgd flow, with preaeration displacement time also shown. For this analysis, and for the two following it, data from all 12 cfm downdraft runs were included; data for 8 cfm and 0 cfm runs were eliminated. Data from all blower runs were included except for the July 11 and 12 diffusion tube tests, which were unduly distorted by high flows and weak sewage strength from receding flood waters.

Within probable mean detention time limits of roughly 35 to 50 mins, no valid trend of benefit with varying preaeration detention time was apparent.

c. Effect of sewage strength. An analysis was next made of the relationship of preaeration benefit to strength of the incoming raw sewage. Raw sewage strength was again taken as the average of preaeration influent and effluent strength (figure 50). While there appears to be a slight tendency toward greater benefit with weaker sewage, this impression is probably due to the location of a few extreme values on the plot. This can be tested rather easily by covering single data points or small groups and observing the completely different appearance resulting. These data do not indicate a correlation of benefit with sewage strength.

d. Correlation with plain primary removal. The final analysis concerns the correlation of benefit with the removal accomplished by plain primary settling alone. A trend is apparent, and one which is confirmed by prior laboratory and plant-scale results (figure 51). The omission of a few well-chosen data can change the visual impression of the plot, but not sufficiently to destroy the ruling trend. Improvement by preaeration of roughly 10 percentage points is indicated over the poorer plain settling removals, to only 5 or 6 points improvement when plain settling was doing unusually well.

When plain settling is at its best, preaeration has less margin in which to produce results. Far more important is the implication that preaeration is of greatest benefit when for some reason plain settling is at a disadvantage -- and when primary treatment, and total plant performance, is most in need of help.

XI. AMES PLANT RUNS WITH VARIED SETTLING TIME; JANUARY-MARCH, 1959

A. The Tests

As careful evaluation of plant results at Ames and the three other plants progressed, it became increasingly apparent that preaeration strongly influenced the early stages of settling. Time and resources were lacking for extended further study of this phenomenon. However, two weeks were spent in January and several days in March 1959 in a preliminary study of what might be achieved in plant operation with reduced settling time following preaeration.

The plant arrangement was as given earlier for full-scale runs. The flow was accurately split, half passing directly to the north bay of primary clarifiers; the other half was directed first to the south preaeration tank, then to only one of the south primaries. All return flows were held or diverted to again provide an operating comparison with raw sewage only. The vital precaution of de-gritting all primary units was observed. Preaeration was at the rate of 100 cfm throughout the two weeks in January, then 12 cfm by downdraft aeration in March.

Sampling was conducted and composites prepared for 24 hr periods. Samples were collected every half hour during the daytime when practical and hourly otherwise. Laboratory procedures were as previously described. BOD and SS determinations were made on the composite samples, and the SS content of each individual sample was determined as well. Time did not

Table 23. Outline of comparative plant runs with unequal settling time

Date and days (1959)	Preaeration and settling (south bay)		Plain settling (north bay)
Jan. 13-15 3 days	Preaeration plus 1 hr settling; No. 4 primary	vs	Plain 2 hr settling; Nos. 1 & 2 primaries
Jan. 16, 17 2 days	Plain 1 hr settling alone; No. 4 primary	vs	Plain 2 hr settling; Nos. 1 & 2 primaries
Jan. 20-23 4 days	Preaeration plus 1 hr settling; No. 4 primary	vs	Plain 1 hr settling; No. 1 primary
March 2-4 3 days	Preaeration plus 1 hr settling; No. 4 primary	vs	Plain 1 hr or 2 hr settling; No. 1 or Nos. 1 & 2 primaries

permit laboratory work on oxygen values or settleability (table 23).

The primary objective of these runs was a direct operating comparison of plain primary settling with a parallel plant comprising preaeration followed by only half the settling capacity. Plant sewage flows during these runs were such as to provide nominal 2 hr displacement time through the two (north) plain primary clarifiers, and 1 hr displacement through the one (south) primary clarifier preceded by preaeration.

B. Results

Detailed composite BOD and SS results are presented in table 24, and individual SS strength patterns for raw sewage and for the two primary effluents are shown in figure 52. The SS results are quite interesting.

During the first three days, it was a pleasant surprise to find preaeration and 1 hr settling outperforming 2 hr plain settling. In BOD

Table 24. Summary of 1959 full-scale Ames plant runs

Date	Day	Raw Sewage flow, mgd	Preaeration arrangement	Air flow, cfm	Composite sampling period	SS method*	Analyses--all in mg/l: Preaerator Influent BOD	Preaerator Influent SS	Preaerator Effluent BOD	Preaerator Effluent SS	Primary 1 + 2 effluent (plain settling) BOD	Primary 1 + 2 effluent (plain settling) SS	Primary 4 effluent (preaeration) BOD	Primary 4 effluent (preaeration) SS	Primary Removals**, all in %: BOD 1 + 2 effluent (plain settling)	BOD 4 effluent (preaeration)	SS 1 + 2 effluent (plain settling)	SS 4 effluent (preaeration)
1959																		
1/13	Tu	2.50	8 spargers	100	24 hr	P	224	221	225	219	167	88	161	84	25.7	26.4	60.0	61.6
					24 hr	F		217		211		84		77			60.7	64.0
1/14	W	2.45	8 spargers	100	24 hr	P	170	217	175	210	133	[illegible]	134	56	22.[illegible]	22.3	58.3	59.6
					24 hr	E		209		219		86		[illegible]			58.[illegible]	67.7
1/15	Th	2.57	8 spargers	100	24 hr	P	190	211	190	211	157	[illegible]0	155	61	17.4	16.5	57.4	61.6
					24 hr	E		215		223		82		66			62.6	[illegible]
													4 plain			4 plain		4 plain
1/16	F	2.59	preaer. bypassed		24 hr	P	200	204			144	[illegible]0	158	106	25.[illegible]	21.0	55.[illegible]	51.[illegible]
					24 hr	E		197				[illegible]		111			54.3	43.6
1/17	Sa	2.50	preaer. bypassed		12 hr	P	186	252			155	[illegible]7	18[illegible]	111	16.7	[illegible]	61.[illegible]	56.0
					12 hr	E		24[illegible]				100		[illegible]			5[illegible].8	51.9
											1 plain		1 preaerated		1 plain	4 preaerated	1 plain	4 preaerated
1/20	Tu	2.50	8 spargers	100	24 hr	P	230	193			188	54	172	72	18.3	25.3	56.5	62.[illegible]
					24 hr	E		192				78		70			5[illegible].4	63.6
1/21	W	2.46	8 spargers	100	24 hr	P	235	186			175	7[illegible]	175	61	17.9	24.[illegible]	57.[illegible]	67.2
					24 hr	E		178				78		[illegible]2			56.2	59.8
1/22	Th	2.42	8 spargers	100	24 hr	P	230	221			180	88	1[illegible]3	[illegible]	21.6	25.[illegible]	60.2	6[illegible].2
					24 hr	E		208				[illegible]		5[illegible]			53.[illegible]	60.1
1/23	F	2.40	8 spargers	100	24 hr	P	195	213			155	[illegible]	15[illegible]	[illegible]	2[illegible]	22.[illegible]	[illegible].2	[illegible]
					24 hr	E		212				100		55			52.6	60.8
											1 + 2 plain				1 + 2 plain		1 + 2 plain	
3/2	M	2.42	downdraft	12	24 hr	P	184	183	175	183	133	74	121	7[illegible]	2[illegible]	32.[illegible]	[illegible]	60.1
					24 hr	E		180		167		[illegible]2		[illegible]			47.3	51.3
											1 plain				1 plain		1 plain	
3/3	Tu	2.30	downdraft	12	24 hr	P	177	170	177	180	145	86	12[illegible]	74	18.1	27.1	50.7	57.7
					24 hr	E		167		218		108		[illegible]2			46.7	54.6
											1 + 2 plain				1 + 2 plain		1 + 2 plain	
3/4	W	2.36	downdraft	12	24 hr	P	195	1[illegible]			160	72	140	[illegible]	15.6	28.2	5[illegible].8	[illegible]3.1
					24 hr	E		200				192		77			[illegible]	61.8

*P - photometric, (P) - photometric, calculated composite, E - evaporation.
**Calculated removals based on average of Preaerator influent and effluent strengths.

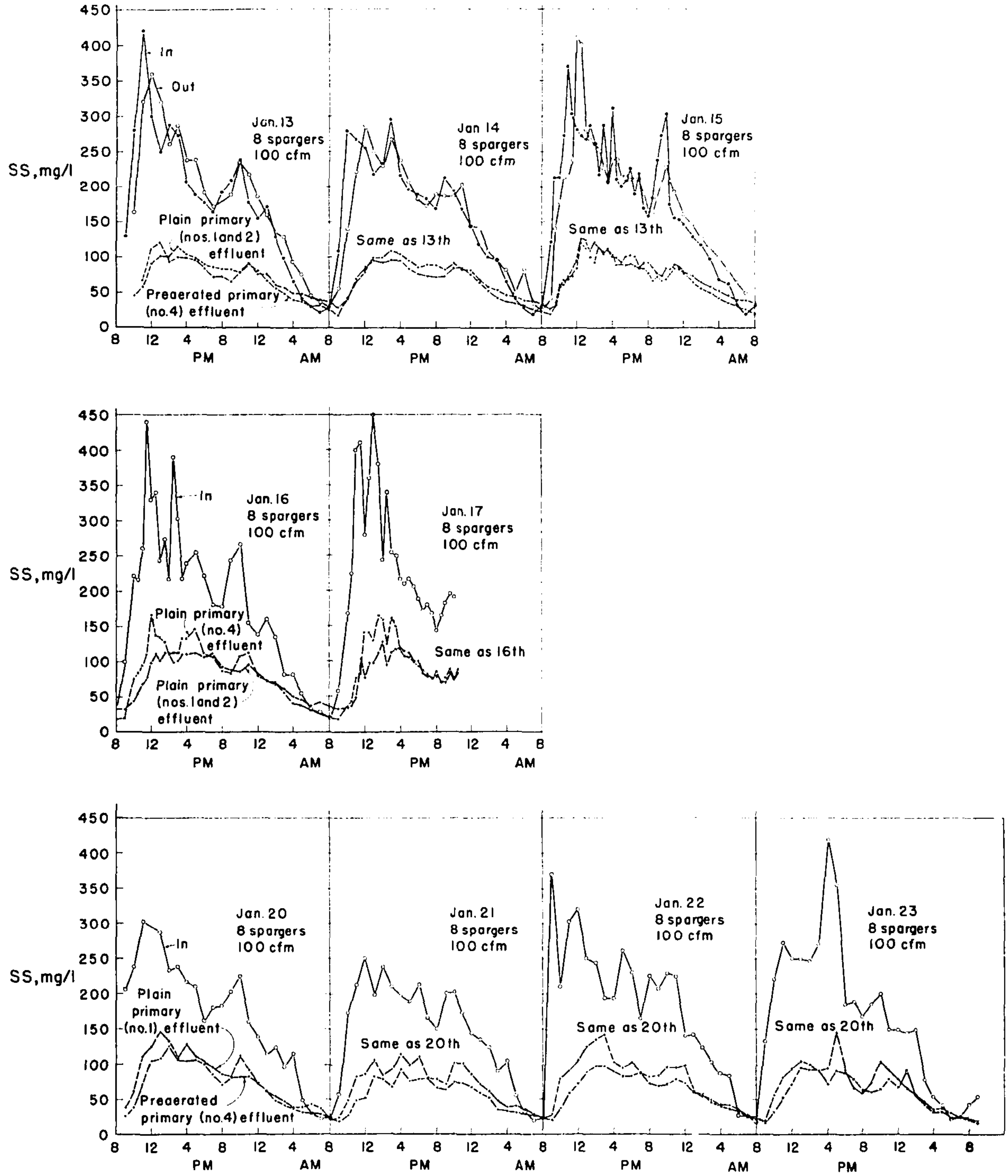


Fig. 52. Daily patterns of raw and primary effluent SS strength; 1959 Ames plant runs with varied settling time

removal, the difference was negligible, averaging a mere one percentage point. In SS removal, however, the margin ranged from about two to five points and appeared fairly consistent throughout the day, except for the forenoon period of peak raw sewage strength and flow volume.

Primarily to determine whether the success of this combination was in fact due to pretreatment, the south preaeration unit was bypassed the following two days. No. 4 primary clarifier promptly confirmed its expected limitations by dropping five to ten percentage points below No. 1 and 2 clarifiers in both BOD and SS removal. No. 4 clarifier, when operating without the help of preaeration, was at most serious disadvantage in handling forenoon peak loads and daytime loads generally, although holding its own at times of reduced volume and strength when settling time was longer on both sides. These results also indicate that the buffering effect of the preaeration step may be more substantial than assumed originally.

The following week, the plant arrangement was adjusted for parallel operation with 1 hr settling on both sides, but with preaeration again preceding No. 4 primary clarifier. The improvement by preaeration here was consistent and strong, ranging from five to ten percentage points in SS removal and holding steady at about seven points in BOD removal for three of the four days.

Later, an operating sequence was begun in which it was intended to pit preaeration followed by 1 hr settling against 1 hr or 2 hr plain settling on alternate days. After only three days this test series was brought to a rude halt by the worst snowstorm in eight years. However, the results again indicated a strong advantage for preaeration and 1 hr settling over 1 hr settling alone, and confirmed the merit of the former in competition with 2 hr plain settling. This would appear to be a major consideration in the intelligent application of the preaeration process in sewage treatment practice.

XII. STUDIES AT OTHER IOWA PLANTS

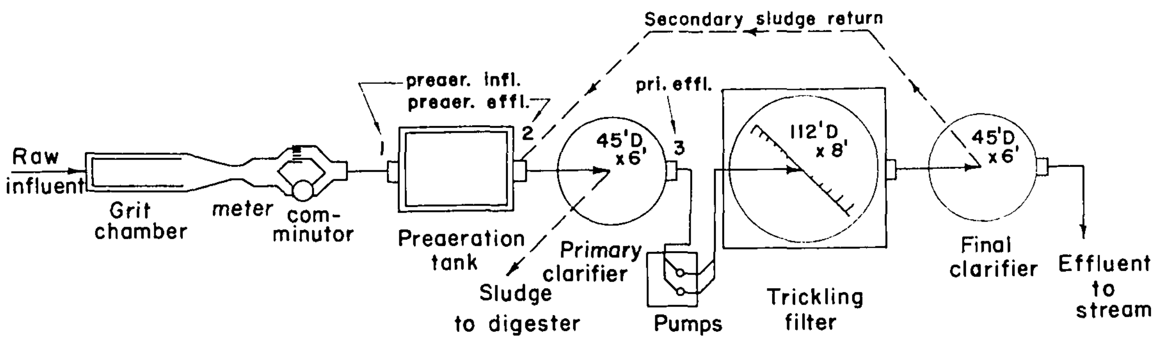
In August, 1958, brief plant-scale studies were made at the Grinnell, Des Moines, and Cedar Rapids sewage treatment plants. All use preaeration as a first step in the treatment process. A major difference from the Ames plant is that in none of the three was it possible to split the flow for comparative operation with and without preaeration. Consequently, the only indication of benefit from preaeration at these plants was obtained through intensive laboratory use of the Oulman settlimeter. In addition, as full a background of supporting data on oxygen values, SS and BOD was gathered as time permitted. In December, an additional day was spent at both the Grinnell and Cedar Rapids plants to gain a general idea of ORP values.

A. Grinnell

1. Plant arrangement

This plant, built in 1951, serves a residential community of 8,000 population, including Grinnell College (51). The plant is located in a rural area about two miles from the city proper. The treated effluent enters a small creek affording limited dilution.

Complete treatment is provided (figure 53). Following grit removal and comminution, the incoming sewage is preaerated and settled before being pumped to a trickling filter. Final settling completes the flow



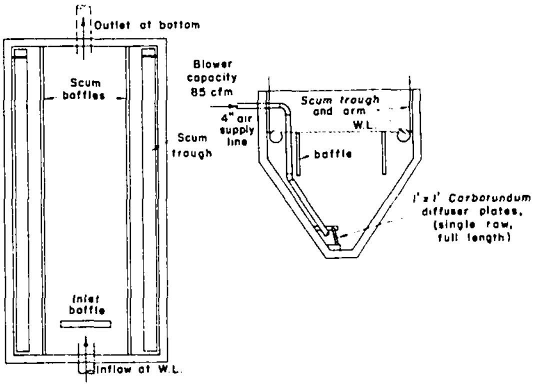


Fig. 53. Schematic diagram of Grinnell, Iowa, sewage treatment plant and preaeration facilities

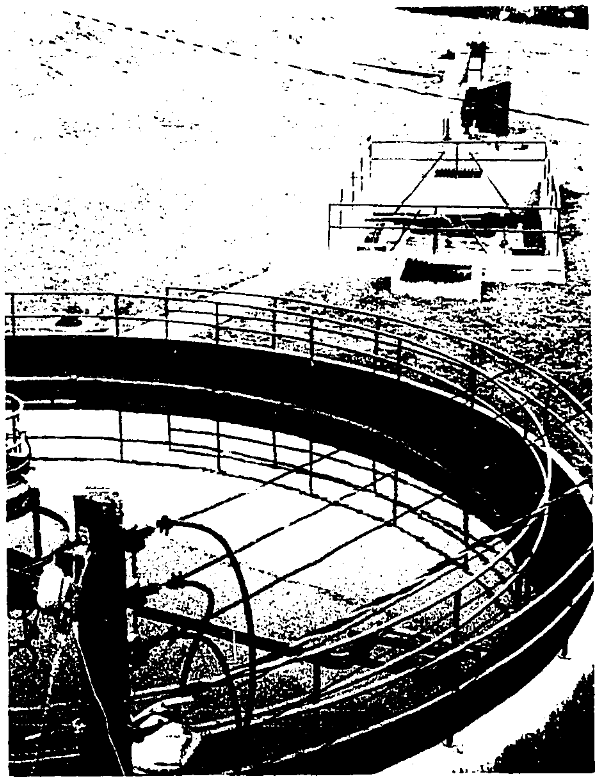


Fig. 54. View of Grinnell primary treatment facilities

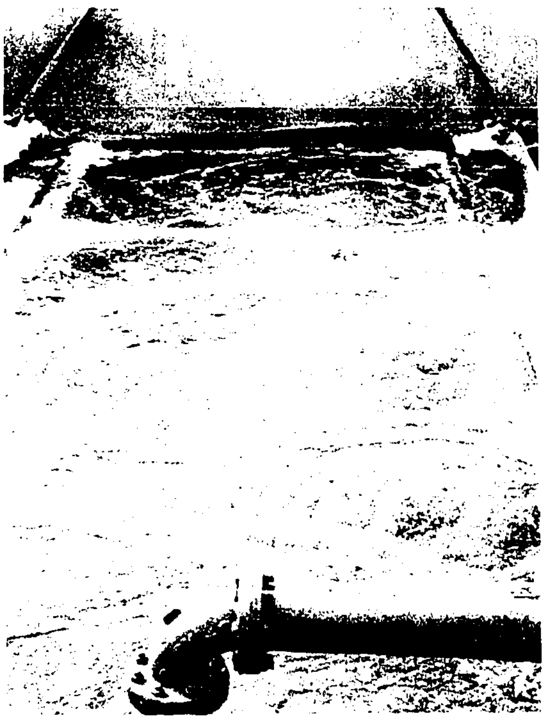


Fig. 55. Grinnell preaeration tank appearance at 85 cfm (0.14 cu ft/gal) aeration rate

diagram, except for continuous recirculation of final sludge and subnatant back to a point ahead of primary settling. Table 25 summarizes the size of the primary treatment units and their loading at the 0.9 mgd rate representing typical daytime flow during these operating runs.

Table 25. Sizes and loadings of primary units; Grinnell, Iowa sewage treatment plant

	Capacity per unit	Loading at 0.9 mgd daytime flow rate
Preaeration tank (1)		
Surface area, sq ft (30 ft x 14 ft)	420	
Maximum depth (hopper), ft	11.5	
Volume, gals	25,000	
Detention, hrs*		0.67
Air supply (1)		
Blower rating, cfm	85	
Aeration rate, cu ft/gal		0.14
Primary clarifier (1)		
Surface area, sq ft (45 ft diam)	1,590	
Side-water depth, ft	6.0	
Volume, gals	71,500	
Weir length, ft	141	
Detention, hrs*		1.91
Surface overflow rate, gpd/sq ft		565
Weir overflow rate, gpd/lin ft		6,380

*Theoretical displacement time.

Preaeration is provided in a single rectangular, hopper-bottom tank fitted with a single full-length row of carborundum diffuser plates. Two full-length baffles provide stilling areas for collection of grease which is removed periodically by tilting scum troughs (figures 54 and 55). Air is supplied to the diffuser plates by a blower of 85 cfm rated capacity driven by a 5 hp motor. No attempt was made to gage the actual air delivery. Grit does not settle out in this tank, nor has there been a problem with clogging of the diffuser plates.

2. Operating studies

Plant-scale studies made on August 12, 13 and 14 proceeded generally without incident. The weather was warm and humid, but the lack of appreciable rainfall for several weeks insured operation under normal dry-weather conditions. Total 24 hr flow reaching the plant averaged roughly 0.70 mgd.

Recirculation from the final clarifier was shut down Monday afternoon, August 11, to provide for primary treatment of incoming raw sewage only. No supernatant or other return material entered the primary units throughout these plant studies. On Tuesday, August 12, the actual detention time of the preaeration step, and indirectly for the primary clarifier also, was determined with fluorescein dye. The results of this work (figure 56) indicate that actual mean detention times for both units are close to theoretical displacement periods.

Preaeration influent and effluent and primary effluent were samples at half-hour intervals during the daytime and composited proportional to flow. The composite periods were staggered to compensate for probable detention times. DO and SS were determined on the half-hour samples; oxygen depletion rates were run hourly and as many settlimeter runs were made as practical. It was found necessary to establish new calibrations for the photometric SS method specifically for Grinnell Sewage.

On Tuesday, December 30, a return trip was made to the Grinnell plant. The weather was mild, around 20° F, and calm. The noon sewage

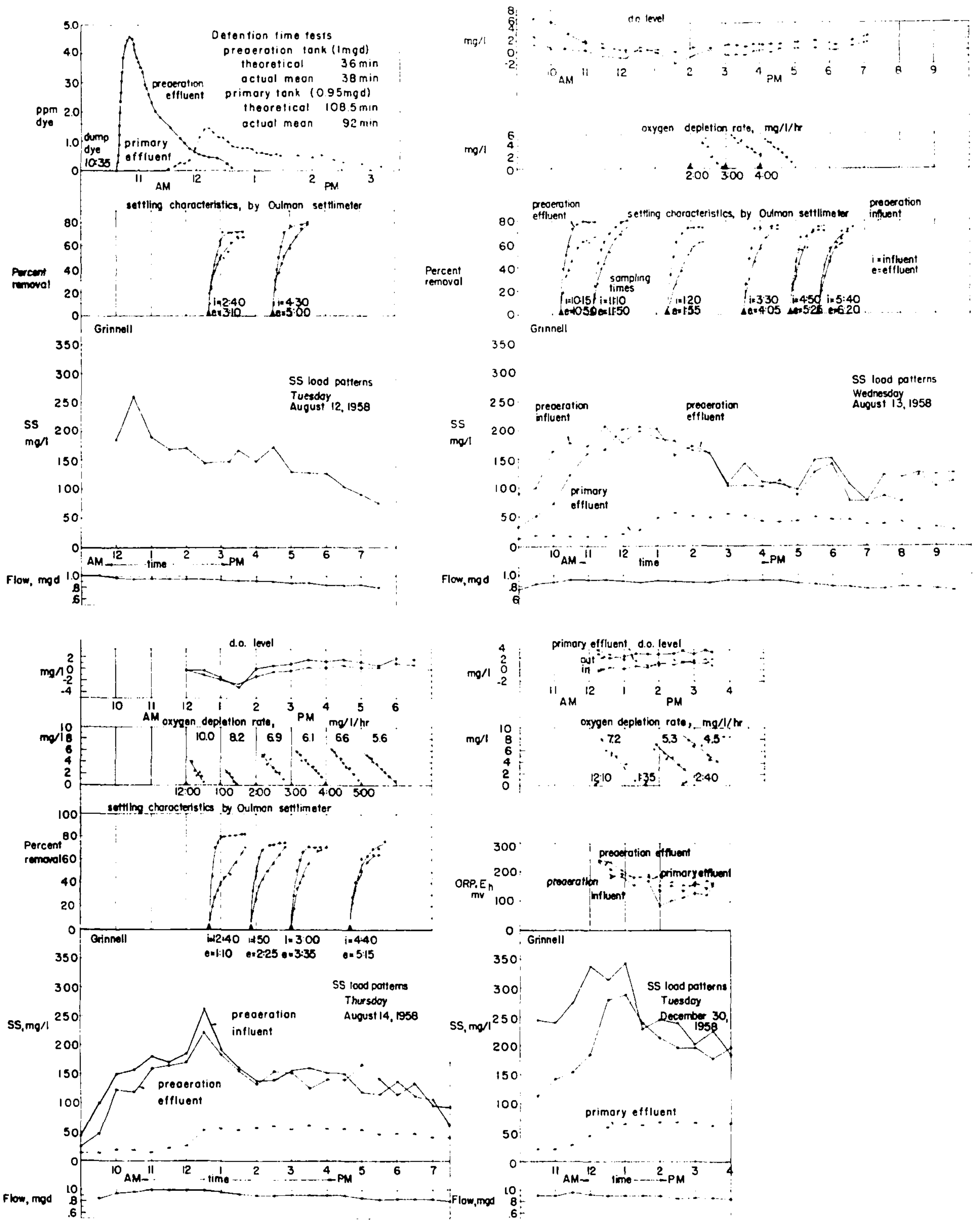


Fig. 56. Graphic data summary of preaeration studies at Grinnell, Iowa, sewage treatment plant

temperature was 57° F. The flow level was quite like that in August but the sewage strength was about half again as great. The objective of this visit was to gain a general indication of ORP levels in the raw sewage and in the preaeration and primary effluents. Other than the substitution of ORP determinations for settlimeter runs, sampling and laboratory work were essentially the same as in August. The composite period was limited to four hours.

3. Results

a. Primary removals. The analytical data for these four days (figure 56 and table 26) show that primary removals were excellent, averaging over 50 percent of influent BOD and over 70 percent of influent SS strength.

b. Oxygen values. Preaeration boosted the DO level from little or nothing in the raw sewage to a level of 1 to 2 mg/l during most of the day. In August, this was insufficient to maintain DO through primary settling, but in the December test the primary effluent was found to carry a small amount of DO consistently.

Oxygen transfer efficiencies (table 27) were just under 2 percent. The oxygen transfer efficiency and the oxygen depletion and acceptance rates compare closely with results at the Ames plant. Both plants are treating a domestic waste of very ordinary composition. ORP values proved to be consistently positive despite the long flow time to the plant. Preaeration appeared to be able to maintain the ORP level even at peak load and to raise it by some 50 mv in the afternoon. The data on ORP are too limited for other than the most general comments.

Table 26. Sewage characteristics and primary removals; Grinnell, Iowa

	Aug. 12 Tuesday	Aug. 13 Wednesday	Aug. 14 Thursday	Dec. 30 Tuesday
Preaeration influent				
Composite period	12 Noon - 7:30 PM; 8 hrs	9:30 AM - 7:00 PM; 10 hrs	9:30 AM - 7:00 PM; 10 hrs	10:30 AM - 2:00 PM; 4 hrs
TS, mg/l	1197	1185	1252	
DS, mg/l	1051	1021	1066	
SS, mg/l	146	164	186	265
BOD, mg/l	107	101	131	101
Preaeration effluent				
Composite period	not sampled	10:00 AM - 7:30 PM; 10 hrs	10:00 AM - 7:30 PM; 10 hrs	11:00 AM - 2:30 PM; 4 hrs
TS, mg/l		1186	1236	
DS, mg/l		1012	1079	
SS, mg/l		174	157	270
BOD, mg/l		115	135	181
Primary effluent				
Composite period	not sampled	12 Noon - 9:30 PM; 10 hrs	11:30 AM - 7:30 PM; 8½ hrs	12:30 PM - 4:00 PM; 4 hrs
TS, mg/l		1057	1073	
DS, mg/l		1012	1032	
SS, mg/l		45	41	69
BOD, mg/l		51	60	95
Primary removals				
BOD, percent	--	53	55	49
SS, percent	--	73	76	74

Table 27. Summary of oxygen values; Grinnell, Iowa

	Aug. 13 Wednesday	Aug. 14 Thursday	Dec. 30 Tuesday
Time	2-4 PM	2-4 PM	2-3 PM
Aeration rate, cfm	85	85	85
Aeration rate, cu ft/gal	0.14	0.14	0.14
Flow rate, mgd	0.89	0.90	0.89
Average DO levels, mg/l			
Preaeration influent	-0.3	-0.3	+1.2
Preaeration effluent	+1.0	+1.2	+3.0
Observed depletion rate, mg/l/hr	5.9	6.5	4.9
Calculated acceptance rate, mg/l/hr	8.1	9.0	7.9
Oxygen transfer efficiency, percent (assuming no surface aeration)	1.7	1.9	1.6

c. Settling characteristics. Results with the Oulman settlimeter are more easily judged in graphical form than described. The sample pairs collected 30 to 35 mins apart appeared to show much more rapid settling following preaeration. Sometimes the preaeration influent sample fell short of the settling ultimately achieved by the preaeration effluent sample, often it eventually did as well.

The important difference was the very striking advantage shown by the preaerated samples in the early moments of settling. For nine of the twelve pairs run, preaeration effluent samples averaged 17 and 22 percentage points advantage, respectively, over the matching influent samples after 10 min and 20 min settling. For the other three pairs, all collected around 5 to 6 PM, the early advantage was only five to ten percentage points, and it was not sustained throughout the test. The explanation for this is not readily apparent.

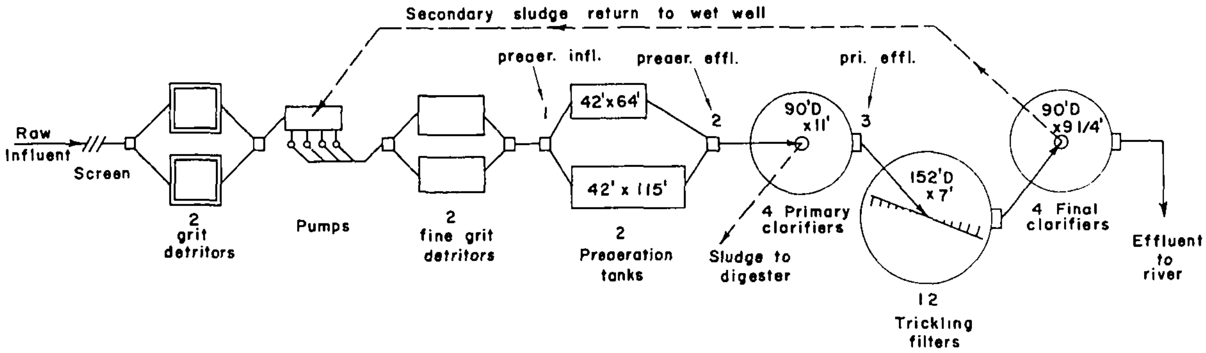
These results are highly favorable to preaeration; they are confirmed to some extent by the unusually high primary removals actually being achieved at the Grinnell plant. Nevertheless, it must be cautioned again that there are important gaps between laboratory results and plant practice.

B. Des Moines

1. Plant arrangement

This plant, completed in 1939 and expanded in 1955, now serves a metropolitan Des Moines area of approximately 225,000 population. Added to this domestic load is a heavy charge of packinghouse waste. The result is a raw sewage loaded with fine grit, grease, paunch manure and occasionally blood, but deceptively low in BOD and SS strength in spite of its appearance. The hydraulic capacity of the plant is limited, and there are times when the incoming flow must be backed up or partly bypassed without full treatment. The plant is located in an industrial area some four to five miles from the main business district. Plant effluent discharges to the Des Moines River.

The schematic flow diagram of the plan appears in figure 57. Following mechanical screening and the first stage grit removal, the raw sewage is pumped to second stage grit removal from which it flows by gravity through the rest of the plant. Preaeration is followed by conventional complete treatment on trickling filters. Final sludge is



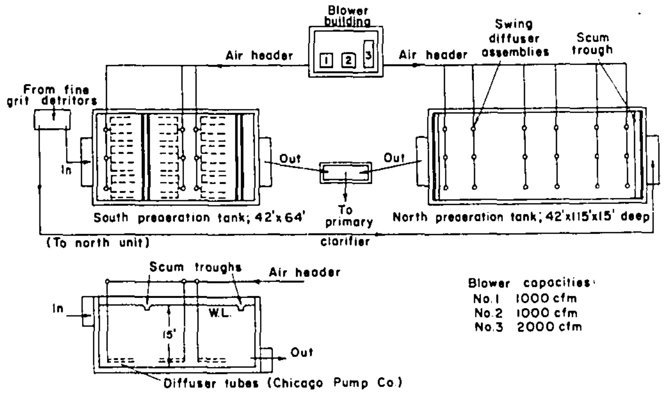


Fig. 57. Schematic diagram of Des Moines, Iowa, sewage treatment plant and preaeration facilities

Fig. 58. Des Moines preaeration tank appearance at 1,000 cfm (0.05 cu ft/gal) aeration rate

Fig. 59. Des Moines preaeration tank appearance at 2,000 cfm (0.10 cu ft/gal) aeration rate

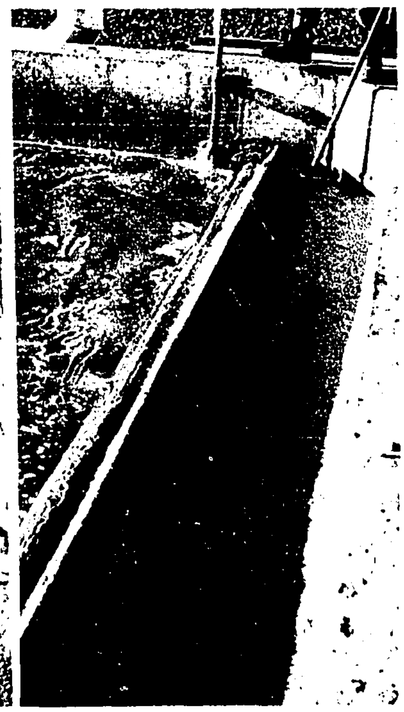


Fig. 60. Skimming grease from Des Moines preaeration tank surface

recirculated back to the raw sewage wet well during the early morning hours only for re-settling in the primary clarifiers. Digester supernatant can either be returned to the raw sewage wet well or discharged to lagoons.

As completed in 1939, the plant included a preflocculation step ahead of primary settling. As part of the 1955 expansion, the mechanical flocculators were removed from this unit and air diffusion equipment was installed instead, along with new scum troughs. This unit is now the south, or smaller, preaeration tank. The north tank and the blower building were also constructed in 1955. Air is provided by two blowers of 1,000 cfm capacity and by a third blower of 2,000 cfm capacity. The blowers are powered by 50 hp and 100 hp motors, respectively. Actual air capacity tests were not run. The air is distributed by swing diffusers placed in bays across the direction of flow through the tanks. In normal operation either of the two smaller blowers is operated; occasionally the 2,000 cfm blower is run for a short time to force open partially clogged diffuser pores and thus improve air distribution. Gradual clogging of the diffusers has proved to be an operating problem. Figures 58 and 59 picture the surface of the south preaeration unit at aeration rates of 1,000 and 2,000 cfm. Table 28 summarizes sizes and load factors for the primary treatment units.

Grease and grit are serious operating problems. If it accomplished nothing else, preaeration at Des Moines would probably be justified on the basis of grease removal alone. During 1958, grease was collected for rendering by a firm which paid the city $10 per ton in addition to all work of loading and hauling. For one recent peak month, the grease

Table 28. Sizes and loadings of primary units; Des Moines, Iowa sewage treatment plant

	Capacity per unit	Loading at 32 mgd daytime flow rate
Preaeration tanks (2)		
North unit		
Surface area, sq ft (42 ft x 115 ft)	4,830	
Depth, ft	15.0	
Volume, gals	543,000	
South unit		
Surface area, sq ft (42 ft x 64 ft)	2,688	
Depth, ft	15.0	
Volume, gals	302,000	
Detention, hrs (both units combined)*		0.64
Air supply (3)		
Blower No. 1 rating, cfm	1,000	
Blower No. 2 rating, cfm	1,000	
Blower No. 3 rating, cfm	2,000	
Aeration rate, No. 1 only; cu ft/gal		0.045
Aeration rate, Nos. 1 and 2; cu ft/gal		0.09
Aeration rate, Nos. 1, 2, 3; cu ft/gal		0.18
Primary clarifiers (4)		
Surface area, sq ft (90 ft diam)	6,360	
Side-water depth, ft	11.17	
Volume, gals	530,000	
Weir length, ft	283	
Detention, hrs (four units combined)*		1.60
Surface overflow rate, gpd/sq ft		1,260
Weir overflow rate, gpd/lin ft		28,300

*Theoretical displacement time.

salvage totalled 96 tons. According to the operators, much of this material formerly passed through primary settling and accumulated on the trickling filters in such quantity that severe ponding and clogging resulted. Figure 60 shows the operator just beginning to skim and indicates the extent of grease removal by preaeration.

Even following two grit removal stages which remove many yards of material daily, fine grit settles in the preaeration tanks and accumulates to such an extent that it tends to bury the air diffusion units. This is relieved partially by blowdown of the tanks, but it eventually becomes necessary to empty them for cleaning.

2. Operating studies

The first series of plant-scale studies extended over the period August 19 to 21. It was not practical to halt the return of final sludge for this long a time; however, the sludge return was stopped at 6 AM or shortly thereafter each morning to allow several hours for purging of the preaeration tanks before sampling began. No supernatant was returned to the sewage flow during these studies. Thus, the preaeration step was limited to treatment of raw sewage only, as far as practical.

Sampling for individual SS determinations and for compositing was done at 20 min intervals. The DO levels of preaeration influent and effluent were checked at approximately half-hour intervals and oxygen depletion runs were made hourly when practical. Settlimeter tests were run as often as possible. As at Grinnell, it was found necessary to prepare new calibrations for the photometric SS determination on this waste.

On Tuesday, August 19, the weather was warm and humid. Wednesday was the same until noon when the temperature dropped sharply, and a thunderstorm began to form. A heavy storm passed through the heart of town, reaching the plant area at 2:25 PM. Within the hour, the incoming raw sewage also changed sharply, bringing with it a heavy load of grit and similar solids. The rain was recorded as 1.6 in. As this flow reached the primary plant units, sampling and compositing were necessarily concluded for the day.

On Thursday the weather remained cool and the flow was still influenced, though very mildly, by the previous day's rain. Sampling was suspended from noon to 1:40 PM due to an unexpected plant shutdown. The composites were extended around this interruption as realistically as possible. At 3 PM a second blower was started, providing 2,000 cfm for preaeration as compared with the 1,000 cfm rate maintained previously.

On Monday, December 29 an additional day was spent at the Des Moines plant. The operation was comparable with that in August, and the sampling was also similar except that ORP determinations were substituted for settlimeter runs. The weather was calm and mild, about 15° F. The sewage was noticeably stronger in packinghouse waste. A single 1,000 cfm blower was operated until 2:45 PM, when the second small blower was started, providing a total aeration rate of 2,000 cfm.

3. Results

a. Primary removals. Primary removals for the four days averaged roughly 40 percent BOD and 70 percent SS (table 29 and figure 61).

b. Oxygen values. During the August plant studies, the DO of the

Table 29. Sewage characteristics and primary removals; Des Moines, Iowa

	Aug. 19 Tuesday	Aug. 20 Wednesday	Aug. 21 Thursday	Dec. 29 Monday
Preaeration influent				
Composite period	11:00 AM - 5 PM; 6 1/3 hrs	9:20 AM - 3:00 PM; 6 hrs	10:00 AM - 4:40 PM; 7 hrs*	11:30 AM - 4:00 PM; 5 hrs
TS, mg/l	1105	1034	1128	
DS, mg/l	690	634	677	
SS, mg/l	415	400	451	630
BOD, mg/l	300	320	350	580
Preaeration effluent				
Composite period	11:40 AM - 5:40 PM; 6 1/3 hrs	9:40 AM - 3:20 PM; 6 hrs	10:20 AM - 5:00 PM; 7 hrs*	12 Noon - 4:30 PM; 5 hrs
TS, mg/l	1046	1055	1125(e)	
DS, mg/l	650	637	684	
SS, mg/l	396	418	441(e)	525
BOD, mg/l	295	300	360	680
Primary effluent				
Composite period	1:00 PM - 7:00 PM; 6 1/3 hrs	11:30 AM - 5:00 PM; 6 hrs	10:40 AM - 5:20 PM; 7 hrs*	1:00 PM - 5:30 PM; 5 hrs
TS, mg/l	784	736	747	
DS, mg/l	655	600	634	
SS, mg/l	129	136	113	160
BOD, mg/l	185	175	145	400
Primary removals				
BOD, percent	38	43	59	37
SS, percent	68	67	75	72

*Interrupted by plant shutdown 12:00 Noon to 1:30 PM.

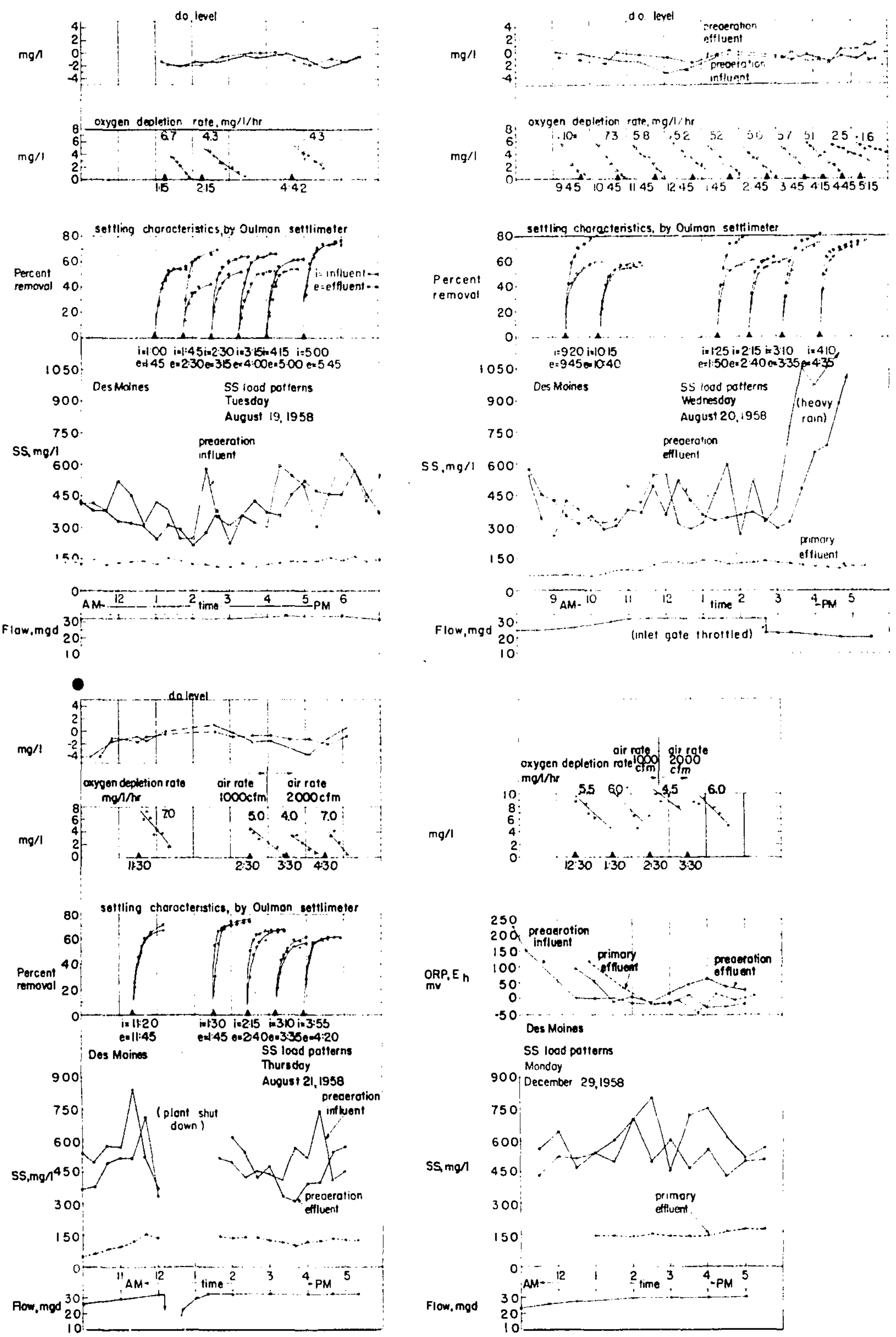


Fig. 61. Graphic data summary of preaeration studies at Des Moines, Iowa, sewage treatment plant

preaeration influent was rarely positive, and ranged more commonly 1 to 2, and as low as 4 mg/l on the negative side. With the stronger sewage flows, the preaeration effluent was usually still lower in DO; at times of more moderate oxygen demand, such as midafternoon, the DO level was raised slightly by preaeration.

Table 30 summarizes the DO levels, depletion rates and transfer efficiencies observed in the Des Moines studies. The oxygen transfer efficiency with only one blower operating was calculated to be in the

Table 30. Summary of oxygen values; Des Moines, Iowa

	Aug. 19 Tuesday	Aug. 20 Wednesday	Aug. 21 Thursday
Time	1-2 PM	1-3 PM	2-3 PM
Aeration rate, cfm	1,000	1,000	1,000
Aeration rate, cu ft/gal	0.045	0.045	0.045
Flow rate, mgd	32½	32	32½
Average DO levels, mg/l			
Preaeration influent	-1.7	-1.0	-2.0
Preaeration effluent	-1.5	-0.5	-0.9
Observed depletion rate, mg/l/hr	5.5	5.2	4.5
Calculated acceptance rate, mg/l/hr	5.8	6.1	6.4
Oxygen transfer efficiency, percent (assuming no surface aeration)	3.5	3.6	3.8

range of 3 to 4 percent. This was considerably higher than for Ames or Grinnell, but represents a low aeration rate of 0.045 cu ft/gal. The few hours of operation on Thursday at 2,000 cfm did not appear to affect the preaeration effluent significantly, but time was too short for a fair evaluation.

The oxygen data of December 29 were highly erratic and contradictory. The DO determinations yielded nothing of value except that both preaeration influent and effluent were deplete throughout the day. The daytime ORP levels were in the range of zero. Although here again the time was limited, it appeared that the 1,000 cfm aeration rate was able to maintain the ORP level generally constant, while the 2,000 cfm rate boosted it some 50 mv.

c. Settling characteristics. The influence of preaeration on settle-ability, as indicated by the settlimeter, varied over rather wide limits. On Tuesday, August 19, four of the six runs showed substantial improvement following preaeration, most of it occurring in the first ten to twenty minutes. On Wednesday, the improvement shown in several of the runs was very good, while in the others it was modest. On Thursday, the settli-meter tests indicated little or no benefit from preaeration. This was at least partly because the settleability of the preaeration influent was unusually good, as confirmed by plant primary removals substantially better than during the first two days. In general terms, preaeration at the Des Moines plant appeared, on the basis of laboratory tests, to benefit primary removal.

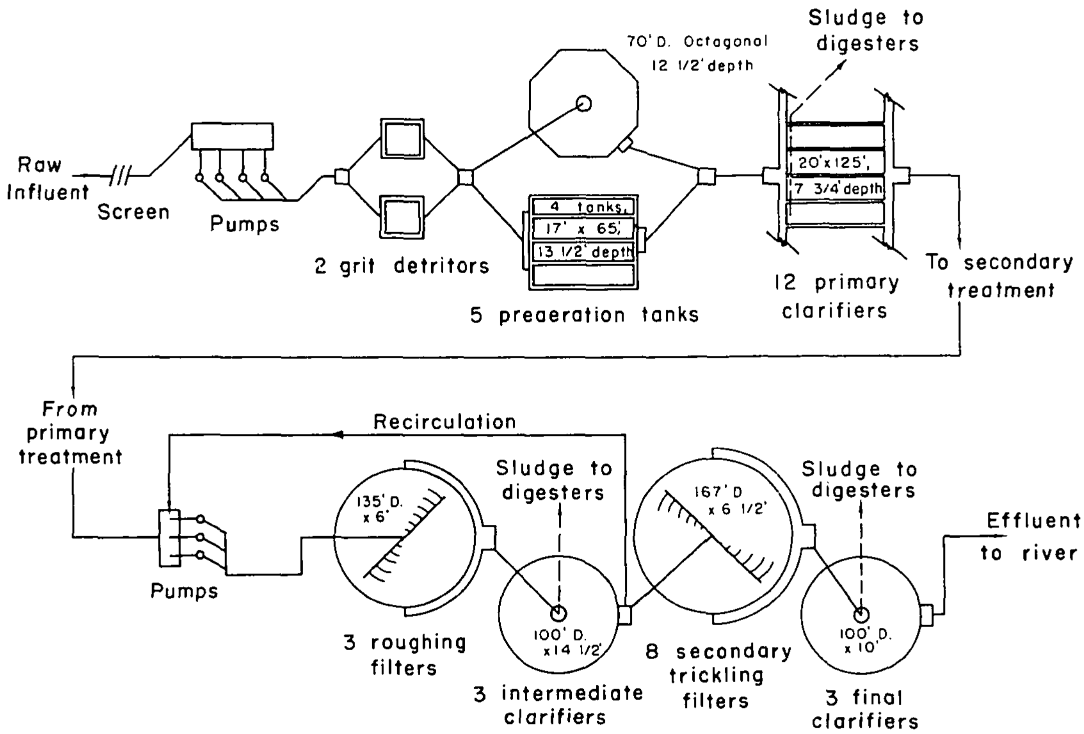
C. Cedar Rapids

1. Plant arrangement

This plant, when placed in operation in 1935, won national attention for its joint treatment of both domestic and packinghouse wastes and for the cooperative financing arrangement between city and industry. It was the first plant to employ several treatment procedures (30). An expansion program completed in 1958 more than doubled the plant capacity. The present waste load from a population of some 90,000, and from a vigorously expanding industrial sector, constitutes a population equivalent approaching 400,000. Wastes from cereal and grain processing and from meat packing are among the major contributors. The plant is located about two miles from the downtown business district and plant effluent discharges to the Cedar River.

The incoming raw sewage passes through mechanical screens, then is pumped to grit removal units (figure 62). Preaeration and settling in long rectangular clarifiers complete the primary phase of treatment. The primary effluent flows by gravity to a wet well from which it is pumped to high-rate roughing filters, then settled in intermediate clarifiers. Part of the intermediate effluent is diverted back for recirculation to the roughing filters while the remainder goes to standard rate trickling filters and final settling. Digester supernatant is returned to the raw sewage wet well in the evenings. No other return flow is mixed with the raw sewage ahead of primary treatment.

As part of the recent expansion program, four tanks which had



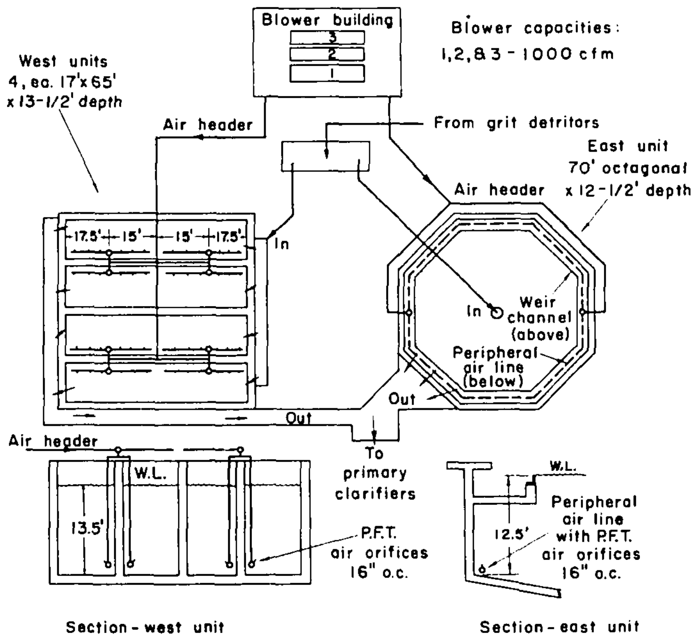


Fig. 62. Schematic diagram of Cedar Rapids, Iowa, sewage treatment plant and preaeration facilities

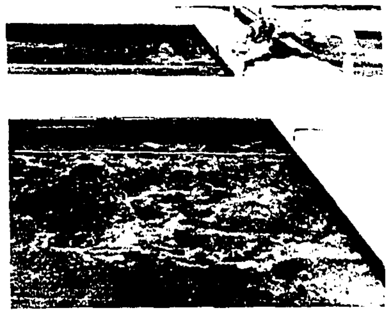


Fig. 63. Cedar Rapids preaeration tank appearance at 1,000 cfm (0.10 cu ft/gal) aeration rate

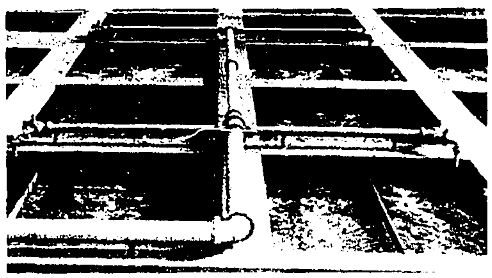


Fig. 64. View of Cedar Rapids preaeration facilities

previously served as primary clarifiers were adapted for preaeration. This consisted almost entirely of simple structural changes to accommodate the increased flows and the installation of air diffusion equipment. The sludge removal equipment in the tanks was not disturbed. Air is provided by a battery of three 1,000 cfm blowers, each driven by a 40 hp motor. Any one or two, or all three, may be operated. No check was made on actual air delivery. Air distribution is by special orifice castings mounted at 16 in spacing on cast iron pipe headers around the lower periphery of the octagonal tank and lengthwise along the four rectangular preaeration bays (figure 62).

At the time of these plant studies, work had not been completed on the octagonal tank, and the full flow was routed through the rectangular units (figures 63 and 64). The operators reported no grit accumulation in these tanks. A sparse accumulation of grease balls was evident, but the total grease problem here and on the primary clarifiers appeared to be moderate. Operating experience was not yet sufficient to determine if clogging of the air orifices would be a problem. Sizes and unit loadings for the primary treatment units are summarized in table 31.

2. Operating studies

Plant studies were conducted at Cedar Rapids on Monday and Tuesday, August 25 and 26. Both the weather and the flow volume were normal for late summer; there had been no recent heavy rains. The rate of flow was approximated by occasional checks on the operating cycles of the raw sewage pumps, whose capacities were known, since the total flow meter was

Table 31. Sizes and loadings of primary units; Cedar Rapids, Iowa sewage treatment plant

	Capacity per unit	Loading at 17 mgd daytime flow rate
Preaeration tanks (2)		
West unit (4 rectangular bays)		
Surface area, sq ft (65 ft x 68 ft)	4,420	
Depth, ft	13.5	
Volume, gals	450,000	
East unit (octagonal)		
Surface area, sq ft (70 ft diam)	4,060	
Depth, ft	12.5	
Volume, gals	380,000	
Detention, hrs*		
Flow through west unit only		0.63
Flow through both units		1.17
Air supply (3)		
Blower rating, cfm (each of 3)	1,000	
Aeration rate, one blower only, cu ft/gal		0.085
Aeration rate, all three blowers, cu ft/gal		0.25
Primary clarifiers (12)		
Surface area, sq ft (20.25 ft x 125 ft)	2,531	
Depth, ft	7.75	
Volume, gals	147,000	
Weir length, ft	198	
Detention, hrs (twelve units combined)*		2.49
Surface overflow rate, gpd/sq ft		560
Weir overflow rate, gpd/lin ft		7,150

*Theoretical displacement time.

inoperative. Supernatant return on these days was limited to the evening hours following sampling, thus providing for preaeration of raw sewage only.

Sampling for composites and for individual SS determinations was at 20 min intervals for preaeration influent and effluent, and at 30 min intervals for primary effluent. DO determinations were made each 20 mins, and oxygen depletion tests were made approximately once an hour. Settlimeter tests were made as often as time and facilities permitted. As at Grinnell and Des Moines, it was found necessary to establish new calibrations for the photometric SS method to permit its use on this waste.

On Monday, a single 1,000 cfm blower was operated. On Tuesday, a second blower was started, providing 2,000 cfm for preaeration until 1:45 PM when the rate was cut back to 1,000 cfm for the remainder of the day. Several incidents demonstrated the variable nature of this waste. On Monday a heavy charge of blood was apparent in the raw sewage reaching the plant from 6:15 to 6:30 PM. On Tuesday after 4 PM, the raw sewage began to develop a peculiar color and odor which the operators characterized as a starch waste from cereal processing. By 4:30 PM this was so strong that the acidification of DO samples for the oxygen depletion test produced a vivid purple coloration instead of the orange characteristic of iodine.

3. <u>Results</u>

a. <u>Primary removals</u>. For the two days, primary removals averaged 42 percent BOD and 75 percent SS. The SS removal seems unusually high but may have been influenced favorably by rather lengthy primary settling.

Sewage characteristics and removals are presented in table 32 and figure 65 presents graphically the detailed analytical data from the Cedar Rapids plant studies.

b. Oxygen values. Generally, the DO level of the preaeration influent was 1 mg/l or better, while the preaeration effluent was deplete with 1,000 cfm aeration and just positive with an aeration rate of 2,000 cfm. Sampling of the detritor effluent just ahead of preaeration indicated a lack of DO at this point. The only possible, and certainly an interesting, implication here is that the DO present in the sewage flow a few feet beyond the detritors is gained in the course of a free fall of several feet between the two points. The DO found in the preaeration influent is definitely dissolved rather than merely entrained.

Oxygen depletion tests were particularly erratic, yielding some improbably low test results and others which defied interpretation. As a result, the summary of oxygen values in table 33 must be discounted accordingly. Oxygen transfer efficiency is shown to be only about one-half percent; this may properly reflect characteristics of this particular waste which hinder its acceptance of oxygen by aeration.

c. Settling characteristics. Judging from laboratory tests with the settlimeter, preaeration was of substantial and fairly consistent benefit to primary settling. The settlimeter runs indicated an advantage of ten to fifteen percentage points in removal after only 10 min settling, and from fifteen to twenty points after 20 mins (figure 65). This advantage was usually sustained throughout the settling test. For only one of the sample pairs was no advantage shown for preaeration. These results were

Table 32. Sewage characteristics and primary removals; Cedar Rapids, Iowa

	Aug. 25 Monday	Aug. 26 Tuesday
Preaeration influent		
Composite period	1:40 PM - 7:00 PM; 5 2/3 hrs	9:20 AM - 5:00 PM; 8 hrs
TS, mg/l	1660	1750
DS, mg/l	1250	1280
SS, mg/l	410	470
BOD, mg/l	600	460
Preaeration effluent		
Composite period	2:00 PM - 7:20 PM; 5 2/3 hrs	9:40 AM - 5:20 PM; 8 hrs
TS, mg/l	sample lost	1725
DS, mg/l		1275
SS, mg/l		450
BOD, mg/l		475
Primary effluent		
Composite period	3:30 PM - 8:30 PM; 5½ hrs	11:30 AM - 7:00 PM; 8 hrs
TS, mg/l	1340	1345
DS, mg/l	1240	1235
SS, mg/l	100	110
BOD, mg/l	360	260
Primary removals		
BOD, percent	40	44
SS, percent	75	76

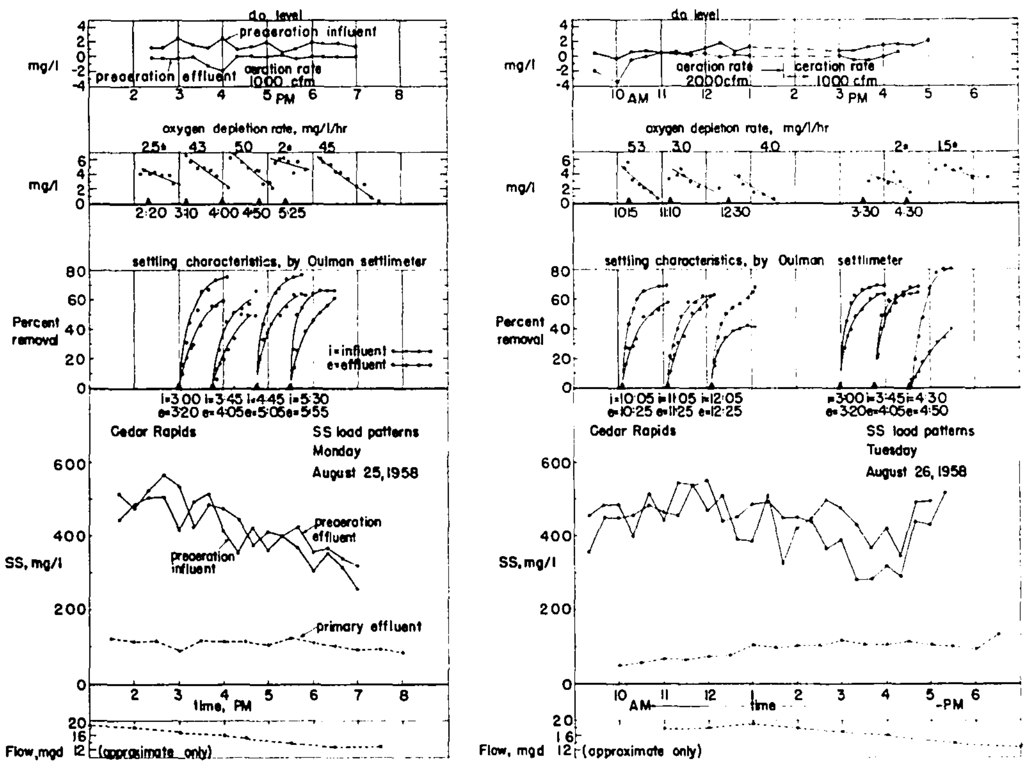


Fig. 65. Graphic data summary of preaeration studies at Cedar Rapids, Iowa, sewage treatment plant

Table 33. Summary of oxygen values; Cedar Rapids, Iowa

	Aug. 25 Monday	Aug. 26 Tuesday
Time	3-5 PM	10 AM-1 PM
Aeration rate, cfm	1,000	2,000
Aeration rate, cu ft/gal	0.09	0.16
Flow rate, mgd	16	18
Average DO levels, mg/l		
Preaeration influent	1.5	0.8
Preaeration effluent	0.0	0.3
Observed depletion rate, mg/l/hr	4.5	4.0
Calculated acceptance rate, mg/l/hr	2.0	3.0
Oxygen transfer efficiency, percent (assuming no surface aeration)	0.6	0.5

confirmed to some extent by the excellent SS removals achieved in plant scale during these studies. However, it cannot be re-emphasized too strongly that the settlimeter test is at best a laboratory approach to a question which can only be answered in full-plant scale.

XIII. ECONOMIC EVALUATION

A. Approach; Annual Cost Method

Engineering economy is defined succinctly by Grant (17) when he asks simply "Will it pay?" Somewhere in the development of every project or proposal this question must be injected if costs are of any concern.

Such a question involves first the outlining of possible alternatives. The next step is the analysis of prospective differences between these alternatives. Only the items that differ are relevant in such a comparison because the others cancel out. Finally, the physical differences are evaluated on a dollar basis. This answers the question, "Will it pay", on a strictly monetary basis. In practice, the choice between alternatives must also give weight to those factors, minor or major, which cannot be reduced to money terms.

The Annual Cost method is well suited to an evaluation of alternatives such as several tentative plant designs producing the same result. A simple example of the method follows, involving one plan with higher first cost and a second plan with higher operating cost. Amortization or capital recovery is based on a 20 yr life, with interest at $3\frac{1}{2}$ percent, for which the annual payment factor is 0.0704.

Alternative A - first cost, $10,000

Annual fixed charge = $10,000 x 0.0704	=	$ 704
Annual operating and maintenance charge	=	1,000
Total annual cost	=	$1,704

Alternative B - first cost, $7,000

Annual fixed charge = $7,000 x 0.0704	=	493
Annual operating and maintenance charge	=	2,000
Total annual cost	=	$2,493

Barring some overpowering argument alternative A would be the logical choice, at an annual cost advantage of $789.

In sewage preaeration, cost comparisons can be made between plants designed for equivalent treatment results with and without preaeration.

B. Treatment Plant Designs

Tentative plant designs were prepared following the Ten States standards (53) and conventional design practice. The design basis used was a population of 25,000, an average daily sewage flow of 3.0 mgd, and BOD and SS strength of 200 mg/l. Complete treatment was provided, with trickling filters comprising the secondary stage of treatment.

1. Standard-rate trickling filters

In the conventional plant design without preaeration, figure 66, 2 hr primary settling was provided. Assuming 35 percent BOD removal in this step, 3,250 lb of BOD passed to the filters, where 5.0 acre-ft satisfy

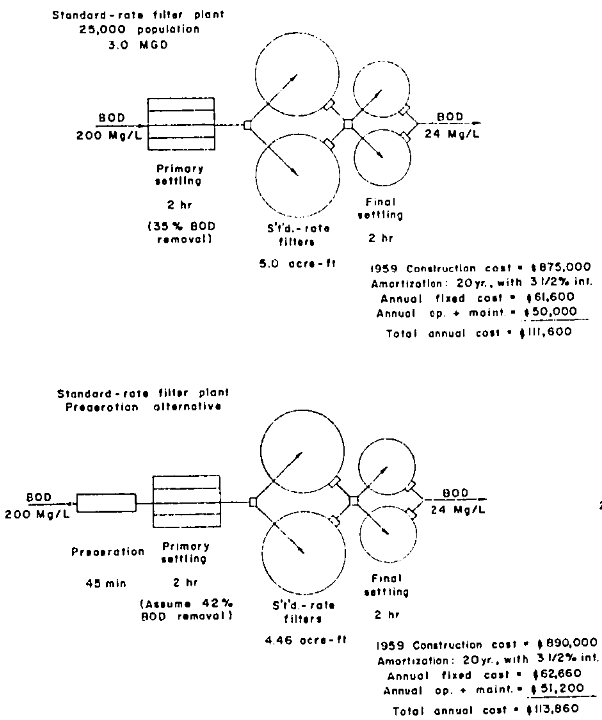


Fig. 66. Alternative standard-rate trickling filter plant designs for economic evaluation

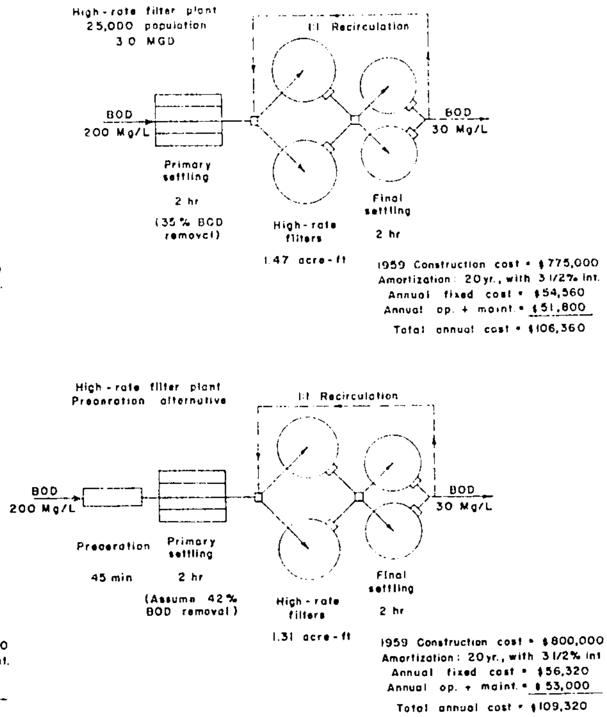


Fig. 67. Alternative high-rate trickling filter plant designs for economic evaluation

loading limits for standard-rate filters. Treatment was concluded by 2 hr final settling. Sludge handling facilities, not shown, were arbitrarily established as 4.0 cu ft/cap for digestion capacity and 1.3 sq ft/cap for sludge drying beds. The sludge handling facilities were identical for all plant designs. Overall BOD removal for this design would be 88 percent, yielding an effluent of 24 mg/l BOD.

2. Standard-rate plant; preaeration alternative

An alternative design was prepared, figure 66, including a 45 min preaeration step. Primary and final settling were both held at 2 hr detentions as in the basic design above. The only other change was in trickling filter capacity.

Assuming that preaeration and primary settling combined will accomplish BOD removal of 42 percent, seven percentage points improvement over plain settling, the BOD load to the filters was reduced to 2,900 lb. This permitted a reduction of trickling filter capacity to 4.46 acre-ft while maintaining loadings exactly like those in the basic design without preaeration. The final effluent BOD would also be the same, 24 mg/l.

3. High-rate trickling filters

The basic high-rate filter design, figure 67, comprised 2 hr primary settling, 1:1 recirculation of plant effluent back to the filter influent, and doubled final settling capacity to provide 2 hr detention for the total flow receiving secondary treatment. As before, 35 percent BOD removal was assumed for plain primary settling, leaving a BOD load of 3,250 lb in the primary effluent. Following the NRC design formula (36), this load can be handled by 1.47 acre-ft of high-rate filter, resulting in a final settled effluent strength of 30 mg/l BOD. The overall plant BOD removal efficiency would be 85 percent.

4. High-rate plant; preaeration alternative

The alternative design, including 45 min preaeration, is shown in figure 67. Primary and final settling were identical to that in the basic high-rate design. The only other change was in the filter capacity provided.

Assuming again a 42 percent BOD removal by preaeration and primary settling combined, the load to the filters is reduced to 2,900 lb of BOD. With 1:1 recirculation, 1.31 acre-ft of filter capacity are sufficient to handle this load with a final settled effluent BOD of 30 mg/l.

5. Design criteria

It is recognized that some features of these designs, such as rigid adherence to 2 hr detention time for both primary and final settling in all cases, are open to argument. However, since these elements, as well as the sludge handling facilities, are identical for both the basic and alternative designs, they do not affect the cost comparisons between the two. The preaeration time of 45 min is generally accepted, and the filter designs follow well-defined requirements. Only these two plant elements affect the cost comparisons.

C. Estimated Costs

1. Construction

Overall plant construction costs were approximated with the aid of recent studies (35, 10, 41). Adjusted to an ENR construction cost index level of 785 (April 1959), a cost of $31/cap is indicated (10) for a 3.0 mgd complete treatment plant in the midwestern area. A similarly adjusted plot (41) indicates a cost of $35/cap for complete treatment for a design population of 25,000. Total cost estimates for this study fall generally within these limits.

For a breakdown of costs by functional plant units, "Economics of Sewage Treatment" (49) was followed closely. The costs in this treatise were collected when the ENR construction cost index was around the 200 level. However, the original work was so fundamentally sound that appropriate adjustment of those unit costs to current index levels yielded overall plant cost figures which confirm today's $31 to $35/cap range. Detailed construction cost estimates appear in tables 34 and 35.

In estimating the cost of the preaeration units, it was assumed that these structures with piping and equipment would be reasonably similar in cost to settling tanks complete with piping and equipment; the same unit price of $2.40/cu ft was therefore applied. A lump sum of $2,000 was allowed for purchase, installation and wiring of a 200 cfm blower; an additional lump sum of $2,000 was allowed for the additional building space required and for air piping.

The additional first cost of preaeration would be offset somewhat by

Table 34. Construction cost estimates for standard-rate filter plant designs (ENR index - 785)

A. Standard-rate filter plant

Primary settling	- 33,400 cu ft	@ $2.40	$ 80,000
Trickling filters	- 5.0 acre-ft	@ $60,000	300,000
Final settling	- 33,400 cu ft	@ $2.40	80,000
Digestion system	- 100,000 cu ft	@ $1.55	155,000
Sludge drying beds	- 32,500 sq ft	@ $2.60	85,000
Auxiliaries, land, building, etc.			90,000
Engineering and contingencies			85,000
Plant A Total			$875,000

B. Standard-rate filter plant with preaeration alternative

Add:

Preaeration tanks with air diffusion equipment	12,500 cu ft	@ $2.40	$ 30,000
200 cfm blower with 10 hp motor, installed			2,000
Building space and piping			2,000
Total add		=	$ 34,000

Deduct:

Decrease in trickling filter capacity from 5.0 to 4.46 acre-ft, or 0.54 acre-ft.

Pro-rata difference would be 0.54 acre-ft @ $60,000, or $32,400. However, because this represents a rather nominal reduction in size of these filters, the actual saving would hardly be more than 60 percent of this amount, or about $19,000.

Total deduct	=	$ 19,000
Net add = $34,000 - 19,000	=	$ 15,000
Plant B Total		$890,000

Table 35. Construction cost estimates for high-rate filter plant designs (ENR index - 785)

C. High-rate filter plant

Primary settling	- 33,400 cu ft	@ $2.40	$ 80,000
Trickling filters	- 1.47 acre-ft	@ $80,000	118,000
Final settling	- 66,800 cu ft	@ $2.25	150,000
Digestion system	- 100,000 cu ft	@ $1.55	155,000
Sludge drying beds	- 32,500 sq ft	@ $2.60	85,000
Auxiliaries, land, building, etc.			107,000
Engineering and contingencies			80,000
Plant C Total			$775,000

D. High-rate filter plant with preaeration alternative

Add:

Preaeration tanks with air diffusion equipment	12,500 cu ft	@ $2.40	$ 30,000
200 cfm blower with 10 hp motor, installed			2,000
Building space and piping			2,000
Total add		=	$ 34,000

Deduct:

Decrease in trickling filter capacity from 1.47 to 1.31 acre-ft, or 0.16 acre-ft.

Pro-rata difference would be 0.16 acre-ft @ $80,000, or $12,800. However, because this represents a rather nominal reduction in size of these filters, the actual saving would hardly be more than 70 percent of this amount, or about $9,000.

Total deduct	=	$ 9,000
Net add = $34,000 - 9,000		$ 25,000
Plant D Total		$800,000

a saving in filter construction costs. This was approached somewhat arbitrarily by calculating the apparent cost difference on a pro-rata unit cost basis for the difference in acre-ft required, then assuming that only a portion of this saving would be realized. This assumption is an attempt to recognize that a difference of some 11 percent in diameter definitely would not result in an 11 percent cost difference. The assumptions made here are open to argument; however, only extreme adjustments would materially affect the final cost comparisons.

2. Operation and maintenance

Operating cost data are not generally available except from individual plant reports. Based on one compilation (8) of such data, a basic annual operation and maintenance cost of $2/cap was considered reasonable for conventional standard-rate trickling filter treatment. The other plants described differ in operation and maintenance requirements from this conventional treatment, and their additional costs are estimated as reasonably as possible in table 36.

It is almost certain that all of the four plants would be staffed alike. There would thus be no major difference in labor costs. The difference in trickling filter sizes between the basic and alternative designs is not sufficient in either case to involve any significant difference in operating duties or maintenance expense.

The cost of power for preaeration and for recirculation is assumed as 1.2 c/kwhr. For annual maintenance of the blower and air diffusion system, a lump sum of $410 was allowed. For annual maintenance of the

Table 36. Plant operation and maintenance cost estimates

A. Standard-rate filter plant		
Basic assumption: use $2/cap for annual cost for this, the simplest of complete treatment flow diagrams.		
Annual operating and maintenance cost: 25,000 pop. x $2/cap		= $50,000
B. Standard-rate; preaeration alternative*		
Added costs would be for operating and maintenance of preaeration units. Power: 10 hp = 7.5 kwhr x 24 hr = 180 kwhr/day		
Power cost = 180 kwhr x 1.2¢ x 365 days	= $ 790	
Maintenance on entire air system, est. at	410	
Annual added cost for preaeration	$ 1,200	
Annual base cost (above)	50,000	
Annual operating and maintenance cost		$51,200
C. High-rate filter plant		
Added costs here would be for recirculation.		
Pumping: 5 kwhr/mgd/ft; 3.0 mgd pumped 18 ft		
Power = 5 kwhr x 3 mgd x 18 ft = 270 kwhr/day		
Power cost = 270 kwhr x 1.2¢ x 365 days	= $ 1,180	
Maintenance on recirculation system, est. at	620	
Annual added cost for recirculation	$ 1,800	
Annual base cost (above)	50,000	
Annual operating and maintenance cost		$51,800
D. High-rate; preaeration alternative*		
Added costs here would be for both preaeration and recirculation.		
For preaeration as above, added cost	= $ 1,200	
For recirculation as above, added cost	= 1,800	
Annual base cost (above)	50,000	
Annual operating and maintenance cost		$53,000

*Cost savings through reduced filter size would be negligible.

recirculation pumps and control system, $620 was estimated. Total annual operation and maintenance costs were computed as the sum of the basic $2/cap figure plus the appropriate additional costs.

D. Cost Comparisons

Comparisons may now be made between equivalent treatment plans designed with and without preaeration. Amortization or capital recovery is based on a 20 yr life, with interest at $3\frac{1}{2}$ percent, for which the annual payment factor is 0.0704.

1. Standard-rate trickling filter treatment

A. Basic design - first cost, $875,000

Annual dixed charge = $875,000 x 0.0704	=	$61,600	
Annual operating and maintenance charge	=	50,000	
Total annual cost			$111,600

B. Preaeration alternative - first cost, $890,000

Annual fixed charge = $890,000 x 0.0704	=	$62,660	
Annual operating and maintenance charge	=	51,200	
Total annual cost			$113,860

Cost difference $2,260/yr, or 2.0 percent.

2. High-rate trickling filter treatment

C. Basic design - first cost, $775,000

Annual fixed charge = $775,000 x 0.0704	=	$54,560	
Annual operating and maintenance charge	=	51,800	
Total annual cost			$106,360

D. Preaeration alternative - first cost, $800,000

Annual fixed charge = $800,000 x 0.0704	=	$56,320
Annual operating and maintenance charge	=	53,000
Total annual cost		$109,320

Cost difference $2,960/yr or 2.8 percent.

3. Economic choice

The preaeration process, in its role of challenger, appears to be at a disadvantage both in first cost and in annual operating cost. This disadvantage is around 2 to 3 percent in total annual cost. Therefore, based on conventional design criteria and giving weight to dollar differences only, preaeration apparently fails to measure up to the question "Will it pay?"

Two other considerations are important here, however. The first is the demonstrated effect of preaeration on the early moments of settling. If 45 min preaeration followed by only 1 hr settling would be accepted as the equal of plain 2 hr settling, the preaeration alternative would be substantially on a par with conventional design in first cost.

The second consideration is the proven effectiveness of preaeration in separating grit from the raw sewage flow. Where preaeration could eliminate the need for a separate grit removal unit, an additional saving in both first and operating costs would be realized.

Where the above considerations are given credit, it appears that the preaeration alternative will enjoy an advantage in terms of dollar differences.

XIV. SUMMARY AND CONCLUSIONS

A. Origin and Present Status

The development of the preaeration process in sewage treatment extends over a period of around 25 years. At least 150 and probably more than 200 waste treatment plants in this country now employ preaeration ahead of primary settling, with the number increasing monthly. Yet, a thorough evaluation of the process has not previously been made.

As studied and discussed here, preaeration is limited to an air-agitation pretreatment of raw sewage ahead of primary settling; 30 to 45 min detention; no chemicals added, nor return of supernatant, sludge or any other material which might serve as a physical or biological aid to flocculation.

The origin of preaeration can be credited partly to early efforts at de-greasing with air flotation, partly to early work with mechanical flocculation which was in itself a modification of chemical treatment as practiced in the 1930's. Laboratory and plant-scale development work done on mechanical flocculation prior to World War II led the way to acceptance of the concept of flocculation as a pretreatment aid to primary settling.

Its proponents assert that preaeration accomplishes flocculation which significantly benefits primary settling and thus increases primary BOD and SS removal. Other advantages claimed for the process are:

grease flotation,

grit separation,
freshening or improved treatability in biological secondary treatment,
addition of DO,
scrubbing of noxious gases and odor,
nominal BOD satisfaction,
mixing and dispersal of waste "slugs".

Increased BOD and SS removal is a tangible benefit capable of being measured in the laboratory and evaluated in dollars. Grease and grit separation are also tangible, although their value lies more in reducing or eliminating operating headaches. The other factors tend toward the intangible and their evaluation is beyond the scope of this study.

Design practice varies widely. Preaeration is credited with substantial improvement in primary settling efficiency in some states. In other states no such credit is allowed. Regulatory policies range the full spectrum between these limits. Opinions and convictions of consulting engineers and plant operating personnel range over equally wide bounds.

The need for actual operating data to help resolve this confusion is widely acknowledged. To meet that need, this study was initiated to evaluate:

(a) the effect of preaeration on primary settling,

(b) the factors influencing its use, and

(c) the economic worth of preaeration as a sewage treatment process.

B. Ames Plant Investigations

The Ames plant is of conventional standard rate trickling filter design, treating almost exclusively domestic wastes from a combined city and university population of approximately 27,000. For purposes of this study, the primary treatment stage was separated into two physically identical bays, each having one hopper-bottomed preaeration tank and two rectangular primary clarifiers. Incoming raw sewage flow was split accurately between these two bays. During the test runs, half the flow was bypassed directly to the north primary flarifiers. The other half was preaerated, then routed to the south primaries. All recirculation, supernatant and other return flows were either diverted or held during these runs to permit an evaluation of preaeration with raw sewage only, in direct parallel comparative operation.

1. September, 1956-May, 1957 plant runs

This long, uninterrupted test series was carried out with the plant arrangement just described. The original downdraft aeration equipment was in service in the south preaeration unit. During the early weeks of this run the combination of preaeration and primary settling was found to have an advantage over plain settling. Soon this advantage increased to surprising proportions, but only because the efficiency of plain primary settling deteriorated almost to the vanishing point. This was purposely permitted to continue for some months to observe its ultimate outcome. In the Spring of 1957, it was established by tests and by observation that the cause of the trouble was grit. Grit was passing directly to the north primary clarifiers and accumulating in putrescible banks of such mass that these primaries were literally poisoned. The south primaries, preceded by preaeration, were free of this condition and performed consistently and well.

2. <u>July, 1957 plant runs</u>

A short series of runs was made in July, 1957, with the flow split as described, but alternating the north and south preaeration units in operation. The original downdraft equipment was in service in both units. For these runs, the primary clarifiers were flushed free of grit each Monday. At the same time, the preaeration tank to be used was also emptied and flushed.

The results for the first three weeks were quite consistent, indicating an average of 8 percentage points improvement in both BOD and SS removal due to preaeration. During the fourth week the local corn cannery began operations, upsetting the pattern of normal removal for both plain and preaerated primary settling.

3. <u>August-September, 1957 laboratory studies</u>

Laboratory preaeration and settling studies were made with gallon samples of raw sewage. A number of replicate runs were made with varied preaeration times and with varied settling time following uniform preaeration periods. Throughout these tests, the aeration rate was maintained at 0.22 cu ft/gal based on 45 min preaeration time. Preaeration consistently benefited settling in these laboratory-scale tests.

a. Varied preaeration time. It was apparent that longer preaeration provided increased benefit. However, the increment of benefit decreased sharply as preaeration continued. For example, half the ultimate benefit achieved by 60 min preaeration was accomplished in the first 15 mins, and 70 percent of this amount in the first 30 mins. The strength of the sewage sample did not appear to be a factor in the results obtained. On the other hand, preaeration seemed to be of greater benefit to those sewage samples which responded least well to plain settling.

b. Varied settling time. The effect of preaeration on the early moments of settling was very evident. For example, 30 min preaeration and $\frac{1}{2}$ hr settling was found to outperform 2 or even 3 hr plain settling. It must be cautioned that these results were obtained in the laboratory under ideal conditions, without short-circuiting or other such problems found in actual plant operation.

4. March-August, 1958 plant studies

The operating arrangement during this series of plant studies was as previously stated, with split flow for parallel operation with and without preaeration on raw sewage flow alone. The first operating chore of the week was always a thorough flushing and de-gritting of primary clarifier sludge hoppers and of the south preaeration tank, used exclusively during the 1958 runs. In addition to the original downdraft equipment, operated with 12, 8 or 0 cfm air intake, the south preaeration unit was also operated with spargers and diffusion tubes at aeration rates varying from 65 to 200 cfm, representing air flows of 0.08 to 0.24 cu ft/gal.

a. Benefit to primary removals. Preaeration was found to benefit primary settling almost without exception. The average improvement for all the plant runs in this series was 7½ percentage points of BOD removal and 8 points SS removal. Neither the method nor rate of aeration, within the limits cited, appeared to have any effect on these results. Also within the limits of plant-scale tests, the preaeration detention time did not seem to affect the results. Neither was any variation with sewage strength observed. The only definite correlation was with the efficiency of plain settling; preaeration seemed more beneficial when plain settling removal was poor, and of less benefit when plain settling was at peak efficiency. This could be quite helpful in actual plant operation.

b. Settleability. As determined by the Oulman settlimeter, the effect of preaeration was of prime importance in the first moments of settling. Often in these laboratory settling determinations the full margin of improvement in the preaerated sample was in the first ten to fifteen min of settling. The settlimeter is not presumed to be a precise instrument, but it gave a reasonable indication of primary settling efficiency and of the improvement resulting from preaeration.

c. Dissolved oxygen transfer. From frequent sampling of preaeration influent and effluent DO levels and from laboratory measurement of short-term DO depletion or demand, the oxygen transfer efficiency was calculated for various aeration conditions. For the downdraft equipment, the calculated transfer efficiency was more than 20 percent, with half or more due to surface agitation rather than sub-surface diffusion. For spargers or diffusion tubes, calculated transfer efficiencies ranged from 1 to 2 percent, the major portion of which was through diffusion.

At the Ames plant, an aeration rate of approximately 0.1 cu ft/gal was necessary to maintain a constant DO level in the preaeration tank during relatively stable afternoon conditions. During the forenoon period of peak flow and strength, not even the maximum aeration rate of 0.24 cu ft/gal could meet the short-term DO demand rate.

d. Oxidation-reduction potential. Based on very limited data collected late in 1958, a low aeration rate insufficient to maintain the influent DO level raised the ORP slightly. On the other hand, aeration rates high enough to raise the DO level also boosted the ORP level appreciably. Since ORP no doubt has a bearing on treatability in secondary stage biological processes, this aspect of preaeration would bear further study.

e. Miscellaneous. Preaeration did not appear to have any effect on the mg/l of SS in the sewage passing through it, despite its influence on settling characteristics. No doubt a few mg/l of oxygen demand, as BOD, are satisfied during preaeration, but this was not apparent within the limits of accuracy of the BOD test.

Preaeration provides a mild buffering action, reducing slightly the intensity of peak strength loads and 'slugs' on their way to primary settling. There was no evidence of a solids inventory in the preaeration tank sufficient to carry through the night. Preaeration proved its merit in grit separation; provision for grit removal should be included in the design of preaeration facilities. An aeration rate of roughly 0.1 cu ft/gal appeared to provide adequate turnover of preaeration tank contents.

5. January-March, 1959 unequal settling time runs

As a postscript of sorts to this project, several weeks were spent in split-flow, parallel operation of preaeration followed by only 1 hr settling against plain 2 hr settling. The results were impressive but probably not surprising in the light of earlier work on settling characteristics. The combination of 45 min preaeration followed by only 1 hr settling always produced better removals than did plain settling for the conventional 2 hr period. This points up the possibility of substantial aid to overloaded primary clarifiers by providing preaeration ahead of them.

C. Other Plants

Preaeration studies were carried out for several days each at the Grinnell, Cedar Rapids, and Des Moines, Iowa, waste treatment plants. In none of the three was it possible to split the flow for parallel operation with and without preaeration. The only indication of benefit was by laboratory work with the settlimeter. In each of the plants, this instrument indicated definite improvement in settleability following preaeration. Whether or not this improvement was fully reflected in actual plant results is not known, although primary removals were very good in each case.

At Grinnell, preaeration increased the DO content of the raw sewage and maintained a positive DO level in the preaeration effluent throughout the day. At Cedar Rapids the reverse was true, with the preaeration effluent usually deplete or negative with respect to DO. At Des Moines the raw sewage was strongly negative, and despite some improvement, left the preaeration tank still with a negative DO level. The grease separation and removal achieved at Des Moines was particularly impressive and was a key factor in successful operation of secondary treatment facilities. The following observation is made from a background of experience at the Minneapolis-St. Paul Sanitary District (42):

> It is my opinion that many sewage particles which should ordinarily settle out in the settling tanks during the normal detention periods are prevented from so doing because of the buoyant effect of grease, oils, and fats adhering to the particles. Preaeration of such raw sewages it, in my opinion, an action whereby the air bubbles diffuse through the sewage flow causing the sewage particles to bounce against one another effecting: (1) release of some of the grease, oils, and fats adhering to the particles; (2) agglomeration of the sewage particles causing their improved settling in the sedimentation tanks. The grease, oils, and fats released float to the surface to be removed in the skimming operations.

D. Economic Evaluation

Preliminary designs were prepared for standard rate and high-rate trickling filter plants with and without preaeration, adhering strictly to conventional design standards . For plain 2 hr settling, 35 percent BOD removal was assumed. Where preaeration preceded primary settling, credit of 7 percentage points was given, bringing primary BOD removal up to 42 percent for purposes of this evaluation. A corresponding reduction in filter size was permitted for the preaeration alternative designs.

Preliminary cost estimates indicated that the reduction in filter cost would not quite balance the added cost of preaeration facilities.

Further, the power necessary for aeration would mean an added operating cost. On a total annual cost basis, the preaeration process appeared to be at a disadvantage of 2 to 3 percent. However, a design alternative of only 1 hr settling following preaeration would probably provide a small advantage in first cost and might be roughly equal to conventional 2 hr primary settling on the basis of total annual cost.

It must be emphasized that this economic comparison was concerned only with a specified improvement in primary settling efficiency, and does not attempt to give credit in any way for other possible operating benefits of preaeration.

E. Conclusions

Preaeration proved to be of consistent benefit to primary settling. As a result of this study, the following conclusions are cited:

1. Intensive plant studies at Ames indicated an average improvement of 7 to 8 percentage points in both primary BOD and S^S removals following preaeration.

2. The degree of improvement or benefit seemed not to be affected by

    a. the aeration method,

    b. the aeration rate, or

    c. the strength of the raw sewage.

3. The degree of improvement or benefit did show a correlation with the efficiency of plain settling. Preaeration was of most help when plain settling was least effective, and of least benefit when removals by plain settling were excellent.

4. The effect of preaeration on the early moments of settling appeared to be of prime importance. In laboratory settling tests, the full margin of improvement in the preaerated sample was often achieved in the first 10 or 15 mins of settling.

5. In laboratory tests with 1 gal samples under ideal conditions, the combination of 30 min preaeration and $\frac{1}{2}$ hr settling was found to outperform 2 or even 3 hr plain settling.

6. In brief plant-scale runs, the combination of 45 min preaeration and 1 hr settling consistently outperformed 2 hr plain settling, confirming the importance of preaeration on the early moments of settling.

7. At the Ames plant, an aeration rate of about 0.1 cu ft/gal was found necessary to match the short-term oxygen demand and thus maintain a constant DO level in the preaeration tank during relatively stable afternoon conditions.

8. An aeration rate of 0.1 cu ft/gal appeared to provide adequate turnover of the preaeration tank contents.

9. The oxygen transfer efficiencies achieved with conventional air diffusion equipment were fairly low, probably due largely to the absence of an active biological culture in the preaeration tank.

10. The preaeration process appeared to have no effect on the mg/l SS content of the sewage passing through it, despite its influence on settling characteristics. If the BOD strength was affected, the amount was too slight to be observed in the BOD test.

11. Preaeration was found to provide a mild buffering action on peak loads reaching the plant. However, there was no evidence of a solids

inventory established in the preaeration tank.

12. The effectiveness of preaeration in grit and grease removal were underscored by operating experiences at Ames and at the other Iowa plants visited. Provision for at least grit removal should be included in the design of preaeration facilities.

13. An alternative plant design including preaeration will permit an appropriate reduction in the secondary treatment facilities provided. However, if conventional design standards are followed strictly, this will result in a slight increase in both construction and operating costs, placing the preaeration alternative at a 2 to 3 percent disadvantage in total annual treatment cost. If primary settling capacity can be sharply reduced following preaeration, this cost disadvantage will be overcome. This cost comparison does not take into account any of the other benefits of preaeration, some tangible, some not.

14. Because of its striking effect on the early moments of settling, preaeration could provide economical relief to seriously overloaded plants by restoring primary settling to normal efficiency. The settlimeter would be of value in determining the effectiveness of preaeration on the waste being treated.

15. The applicability of the preaeration process can best be judged on its merits for each specific waste treatment. Construction and operating costs are nearly the same whether or not preaeration is provided. Therefore, the engineer's judgment is particularly important with respect to such secondary considerations as grit, grease and treatability, and with respect to preaeration's effectiveness in coping with them.

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The author gratefully acknowledges the help provided by graduate and undergraduate students who assisted in the project, particularly Jerry Edwards, Dennis Gere, Charles S. Oulman and Donald K. Smith.

XVII. APPENDIX A. STATEMENT ON PREAERATION FROM TEN-STATES MANUAL OF DESIGN STANDARDS*

4. PRE-AERATION AND FLOCCULATION

A. General: Flocculation of sewage by air or mechanical agitation, with or without chemicals, is worthy of consideration when the raw sewage is strong or when it is desired to reduce the strength of sewage to such a degree that subsequent treatment units can produce a satisfactory plant effluent.

B. Arrangement: A unit should be designed so that it may be removed from service without affecting any settling unit.

C. Detention Period:

(1) Coagulation: When air or mechanical agitation with chemicals is used to coagulate or flocculate the sewage, the detention period should be about 30 minutes but never less than 20 minutes at the design flow.

(2) BOD Reduction: When air or mechanical agitation (either with or without the use of chemicals) is for the additional purpose of obtaining increased reduction in BOD, the detention period should be at least 45 minutes at design flow.

D. Stirring Devices:

(1) Paddles: Paddles should have a peripheral speed of 1½ to 2½ fps to prevent deposition of solids.

(2) Mechanical Aerators: Mechanical aerators should provide self-cleansing velocities across floor of tank.

(3) Air: Diffused air mix may utilize any of the types of equipment used for activated sludge aeration tanks. The quantity of air should be sufficient to provide self-cleansing velocities. The rate of application of air should be adjustable.

E. Details: Inlet and outlet devices should be designed to insure proper distribution and to prevent shortcircuiting. Convenient means should be provided for removing grit.

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*Reproduced from (53, p. 20, 21).

F. Quick Mix: At plants where there are two or more flocculation basins utilizing chemicals, provision shall be made for a quick mix of the sewage with the chemical so that the sewage passing to the several flocculation basins will be of the same composition. The detention period provided in the quick-mix chamber should be very short -- ½ to 3 minutes.

XVIII. APPENDIX B. SURVEY FORMS FOR PREAERATION DESIGN POLICIES IN 15 STATES

To: State Sanitary Engineer: Colorado, Illinois, Indiana, Iowa, Kansas, Michigan, Minnesota, Missouri, Nebraska, New York, Ohio, Oklahoma, Pennsylvania, Texas, Wisconsin

Attention: Sewage Treatment Plan Review Section.

Subject: Sewage Preaeration

Gentlemen:

Iowa State College and the City of Ames have been cooperating on intensive plant-scale research on sewage preaeration. This work is being supported by a USPHS grant.

The original incentives for this study were first, the contradictory claims for (and against) the process in the literature and in conversation, and second, the lack of valid plant-scale operating data on preaeration.

We are now at a point where your help is urgently requested. We wish to determine present practice of the regulatory agencies in the ten-states and midwest areas with regard to preaeration.

Thus far, we are limiting ourselves to the simplest definition of the process, as follows: an air agitation pretreatment of raw sewage, ahead of primary settling; 45-minute detention; no chemicals added, nor return of supernatant, sludge or any other material which might serve as a coagulant or as a biological 'activator'.

Only a few minutes are needed to fill out the attached questionnaire. As soon as the replies are in, you will receive a tabulation of results.

A fairly complete summary of our plant-scale results to date will also be available soon.

Thank you most sincerely for your help.

Harris F. Seidel, Sup't.
Water & Sewage Treatment
City of Ames, Iowa

cc Paul Houser
Iowa S. D. H., Des Moines

State ______________________________

By ________________________________

Definition: Preaeration: An air agitation pretreatment of raw sewage, ahead of primary settling; 45 minute detention; no chemicals added, nor return of supernatant, sludge or any other material which might serve as a coagulant or 'activator'.

1. In design review, is credit given for increased BOD and SS removal by primary settling if preaeration is provided?

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2. If credit is given, how much is allowed beyond, for example, 35 percent BOD removal by primary settling alone?

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3. Your comments will be appreciated.

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Thank you. As soon as the survey replies are in, you will receive a tabulation of results.

Harris F. Seidel, Sup't.
Water & Sewage Treatment
City of Ames, Iowa

XIX. APPENDIX C. USE OF FLUORESCEIN DYE IN DETENTION TESTS

Discussion:

Fluorescein is a dye material with the characteristic of imparting a brilliant green color to liquids. The color is readily visible in clear water if present in concentrations of less than 0.1 mg/l. In sewage, a green tinge is quite noticeable with as little as 0.5 mg/l present. Because of this characteristic, it is an excellent, economical tracer material.

A photometric laboratory method for determining the concentration of fluorescein in sewage was developed using a Bausch and Lomb Spectronic 20 Colorimeter. The method is based on the interference to light transmission by the fluorescein color over and above the interference by natural color and colloidal matter in the sewage. In accordance with the Lambert-Beer Law, the amount of interference due to the added color is determined following calculation of the ratio of transmittance with and without the added color. A calibration curve in these terms is presented as figure 68.

Materials needed:

Colorimeter, with necessary test cells or tubes.
Calibration curve for fluorescein dye in sewage.
Fluorescein dye; 500 g is sufficient for a flow of 1200 gpm.
Whatman No. 12 folded filter paper, 18.5 cm size.
Funnels, racks, flasks, etc. for filtering step.
Sample bottles of 200 ml capacity or more.

Procedure:

(a) Set Bausch and Lomb Spectronic 20 Colorimeter for operation at a wave length of 485 mu. Switch on about 30 min before using to provide a warm-up period.

(b) Before adding dye to the sewage, take several grab samples of the sewage for photometric base-line readings. Pass these samples through Whatman No. 12 folded filter paper, and determine the transmittance of their filtrates. If a parallel treatment unit is in operation during the dye-test run, it is helpful to sample it at 15 min intervals for continuing base-line data. Barring this, the influent to the treatment unit should be sampled at intervals for this purpose.

(c) Dissolve 500 g of fluorescein in a gallon of warm water; introduce the dye solution at the inlet to the treatment unit at a predetermined time.

(d) Sampling of the effluent of the treatment unit should begin 5 to 10 min before the dye is introduced. As soon as dye begins to appear in the effluent, sampling should be stepped up to once every minute, if not oftener. As time goes on, this can be drawn out every 2 min, then 3, then 5, etc., in the judgment of the sampler and depending on the accuracy desired.

(e) In the laboratory, pour about 75 and not over 100 ml of the sample into a Whatman No. 12 folded filter paper. Collect roughly the first 30 ml in a 1 in test tube and read the transmittance of this filtrate in the colorimeter. Consistency in procedure is important; the samples should be run as soon as possible.

Calculation:

Divide the transmittance of the colored filtrate by the transmittance accepted as the base-line value for plain sewage filtrate. With the decimal ratio of transmittancies resulting, determine the corresponding dye concentration from the calibration curve.

Example:

Time, PM	Colored sewage filtrate, Tr_c	Plain sewage filtrate, Tr_f	Ratio of transmittancies	Dye concentration, mg/l
3:09	-	70.5		
3:12	-	71.5		
3:15 (dump)	-	69.5		
3:16	43.5	71.5*	.61	1.25
3:16:30	20	71.5	.28	3.25
3:17	13.5	71.5	.19	4.2
3:17:30	9.5	71.5	.133	5.2

Comment:

It is recognized that this method does not give highly quantitative results. An important source of error is a possible poor assumption of, or a change in, base-line transmittancy for plain sewage filtrate. However, such an error is likely to have a consistent effect on calculated and plotted results but no important effect on their general trend.

*Average plain sewage filtrate transmittance, accepted as base-line value.

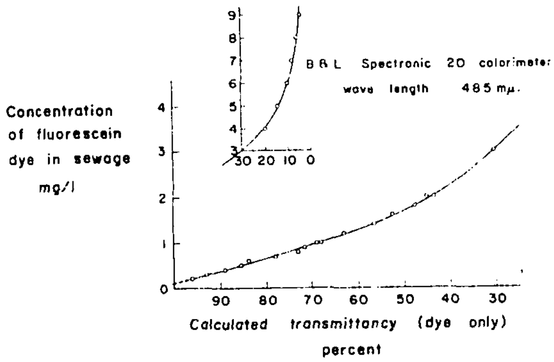

Fig. 68. Photometric calibration curve for fluorescein dye in sewage

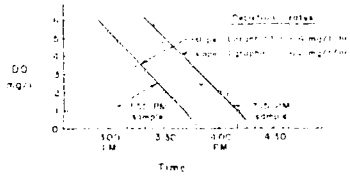

Fig. 69. Graphical determination of DO depletion rate

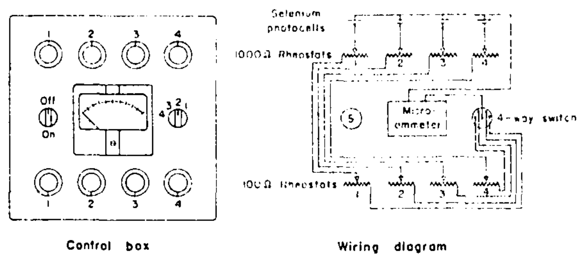

Fig. 71. Wiring diagram for Oulman settlimeter control box

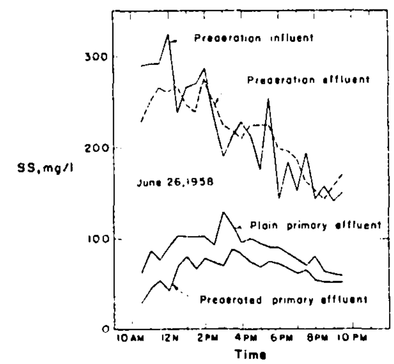

Fig. 70. Typical pattern of raw and primary effluent SS results

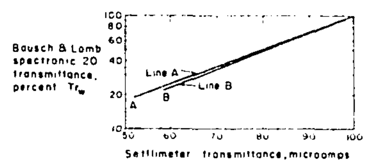

Fig. 72. Conversion of settlimeter readings to Tr_w transmittance

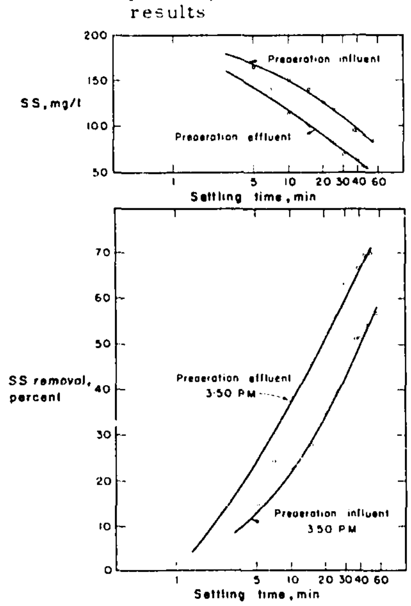

Fig. 73. Typical settling patterns for preaeration influent and effluent shown on semilog scale

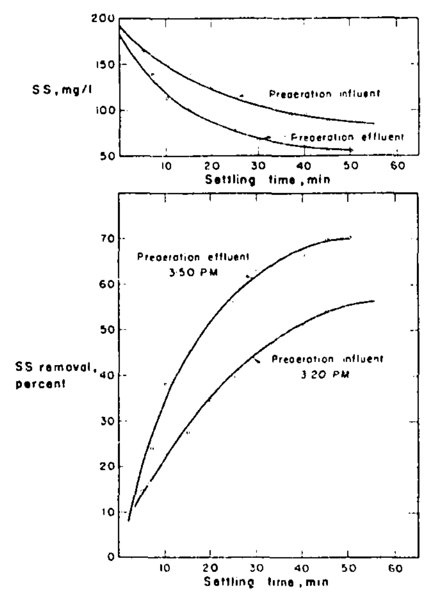

Fig. 74. Typical settling patterns for preaeration influent and effluent shown on rectangular scale

XX. APPENDIX D. LABORATORY PROCEDURES USED IN THE DETERMINATION OF OXYGEN VALUES

A. Photometric Determination of DO in Clear Samples

Materials needed:

Chemicals and glassware for Standard Methods DO procedure, through color development.

Bausch and Lomb Spectronic 20 Colorimeter.

Matched ½ in test tubes for Spectronic 20.

Procedure:

Follow Standard Methods through development of iodine color in BOD bottle.

Plug in Spectronic 20; set at 450 mu and allow warm-up period; then adjust for 0 percent reading without test tube in holder, and for 100 percent reading with distilled water blank.

Invert BOD bottle for thorough mixing of sample; rinse test tube with sample several times; then place in holder; read and record percent transmittance; from graph or tabulation, determine corresponding mg/l DO. See table 37.

Example:

61 percent transmittance = 2.50 mg/l of DO. (Table 37)

Table 37. DO calibration for use with Bausch and Lomb Spectronic 20 Colorimeter; 450 mu

Scale readings (transmittance)	DO, mg/1									
	0	1	2	3	4	5	6	7	8	9
0										
10										8.40
20	8.16	7.92	7.69	7.47	7.34	7.06	6.87	6.68	6.50	6.32
30	6.15	5.98	5.82	5.66	5.51	5.36	5.22	5.08	4.94	4.81
40	4.68	4.55	4.42	4.30	4.18	4.06	3.95	3.84	3.73	3.62
50	3.52	3.42	3.32	3.22	3.12	3.02	2.93	2.84	2.75	2.66
60	2.58	2.50	2.42	2.34	2.26	2.18	2.11	2.04	1.96	1.89
70	1.82	1.75	1.68	1.61	1.54	1.47	1.40	1.34	1.27	1.20
80	1.13	1.07	1.00	.94	.88	.82	.76	.70	.65	.59
90	.53	.48	.42	.37	.31	.25				

B. Photometric Determination of DO in Turbid Samples

Materials needed:

Chemicals and glassware for Standard Methods DO procedure, through color development.

Spectronic 20 Colorimeter and matched ½ in test tubes.

0.1N sodium thiosulfate.

Procedure:

Develop iodine color.

Adjust colorimeter.

Invert BOD bottle for thorough mixing of sample; rinse test tube with sample several times; then place in holder; read and record percent transmittance.

Remove test tube from holder; add two drops 0.1N sodium thiosulfate to sample in the test tube; invert several times to mix thoroughly, destroying the iodine color; then insert tube in holder and again read and record the percent transmittance.

Calculate the ratio of the two readings; with this as the calculated transmittance for the iodine color alone, determine the corresponding mg/1 of DO.

Example:

$$\frac{\text{Turbid and colored}}{\text{Turbid only (decolored)}} \quad \frac{:\ 18}{:\ 60} = 30 \text{ percent calculated transmittance.}$$

30 percent transmittance = 6.15 mg/1 of DO. (table 37)

C. Procedure for Determining Negative DO

Materials needed:

Chemicals and glassware for Standard Methods DO procedure, through color development.

Spectronic 20 Colorimeter and matched ½ in test tubes.

BOD dilution water.

Extra BOD bottles.

Rubber tubing for siphons.

Procedure:

Fill one BOD bottle quietly with dilution water. Fill a second BOD bottle equally quietly to the half-full mark with dilution water.

Collect the sewage sample without aeration; immediately siphon, without aeration, sample to the second BOD bottle until full.

Immediately add reagents, and proceed to develop the iodine color in both bottles; determine DO colorimetrically.

Sample calculation:

Dilution water DO = 7.0 mg/l
Sewage-dilution mixture DO = 3.0 mg/l.
Multiply mixture DO x 2; product = 6.0 mg/l.

Then, 2 x mixture DO - dilution water DO

= 6.0 - 7.0 = -1.0 mg/l of DO.

Check:

50:50 mixture of dilution water (7.0 mg/l) and sewage sample (-1.0 mg/l) would result in DO of 3.0 mg/l as determined above.

D. Sample of Data Sheet Used in BOD Determination

City of Ames
Sewage Treatment Plant

BIOCHEMICAL OXYGEN DEMAND OF SEWAGE

24-hr composite sample   Sampling 10 AM Tuesday 12th to 8 AM Wed. 13th
Set up 10 AM 8:13 1958.   Taken off 10 AM 8:18 1958

Bottle No.	ml of Sample seeded	% Dilution, (ml seed / 305)	When read (I or 5)	Spectronic 20 % transmittance	DO reading, mg/l	Observed depletion mg/l DO	Calculated demand mg/l 5 day BOD	Average BOD result	Sample point: Remarks
36	0		5	28	6.50				Raw
37	3		"	44	4.18	2.32	236		(Preaeration
38	4		"	49½	3.57	2.93	224		in)
39	4		"	50	3.52	2.98	227		
40	5		"	60	2.58	3.92	239	231 mg/l	
41	5		"	58½	2.70	3.70	226		
42	6		"	69½	1.85	4.65	236		
43	8		"	82	1.00	5.50			
44	0		5	28	6.50				Primary
45	6		"	46	3.95	2.55			Effluent
46	7		"	53½	3.17	3.33	145		
47	8		"	57½	2.80	3.70	141	144 mg/l	
48	9		"	65	2.18	4.32	146		
49	10		"	76½	1.37	5.13			

Sample Calculation: $\text{mg/l BOD} = \frac{D_1 - D_5}{\%\ \text{Dilution}} = \frac{7.50 - 4.30}{6/305} = 3.2\left(\frac{305}{6}\right)$

$= 163$

E. Procedure for Determining DO Depletion Rate

Materials needed:

Chemicals and glassware for Standard Methods DO procedure, through color development.

Spectronic 20 Colorimeter with matched ½ in test tubes.

0.1N sodium thiosulfate.

10 quart sampling pail.

4 liter flask.

Rubber tubing for siphon.

Several dozen 4 oz round, narrow-mouth, screw-cap bottles.

Procedure:

Collect sewage sample in 10 quart pail.

After brief plunging, pour about 2 liters into 4 liter flask.

With violent shaking, raise the DO level in this sample to 6 or 8 mg/l; this will require not more than 1 or 2 minutes, depending on the sample.

After allowing a moment for entrained air to escape, siphon from the flask into a series of 8 to 12 small bottles; fill the bottles full, including some overflow for mild purging; cap carefully to exclude air bubbles.

After all bottles are filled and capped, introduce DO reagents into the first of the series, noting the time; after a specified interval (5 to 10 minutes) add reagents to the second; repeat this procedure with the others at intervals.

Determine DO colorimetrically.

Calculate DO and plot DO vs time to determine depletion rate, as in figure 69.

Example:

Time reagents added, PM	Transmittance: color/decolored	Transmittance ratio, percent	DO mg/l
	Sample picked up 2:55 PM		
3:00	20/57½	35	5.35
:05	22/56½	39	4.8
:15	28½/56	51	3.4
:20	31/59	52½	3.25
:30	36½/54½	67	2.05
:35	40/56	71½	1.7
:45	48/53	90½	0.5
:50	51½/52½	98	--

Table 38. Intensive sampling and analysis of DO levels before and after preaeration

Time PM	Preaeration influent Transmittance Color/decolored	Ratio percent	DO mg/l	Preaeration effluent Transmittance Color/decolored	Ratio percent	DO mg/l
2:20	66/72	92	.4			
2:30	60/72½	83	.95			
2:40	63/75	84	.9			
2:48				47/72	65½	2.15
2:50	59½/66½	89½	.55			
2:58				58/79	73½	1.55
3:00	63/67	94	.3			
3:08				54/78	69	1.9
3:10	60½/68	89	.6			
3:18				54½/79	69	1.9
3:20	61/67½	90½	.5			
3:28				47½/74½	64	2.25
3:38				45/72	62½	2.4
3:48				45/72½	62	2.4
		Sum	4.20			14.55
		Mean	0.6 mg/l			2.1 mg/l

F. Procedure for Calculating Rate of Oxygen Acceptance During Preaeration

Date: June 27, 1958

Sample calculation:

DO levels:

Average preaeration influent	:	0.6 mg/l
Average preaeration effluent	:	2.1 mg/l
Gain	:	1.5 mg/l DO

DO depletion rates:

2:30 PM :	6.5 mg/l/hr
2:50 PM :	7.0 mg/l/hr
3:10 PM :	6.6 mg/l/hr
Average DO depletion rate	6.7 mg/l/hr

Sewage flow rate: 3.0 mgd

Probable mean detention time = 0.54 hr

Rate of DO gain = 1.5 mg/l in 0.54 hr, equivalent to a rate of 2.8 mg/l/hr

Oxygen acceptance rate

= measured depletion rate or	6.7 mg/l/hr
plus calculated rate of gain or	2.8 mg/l/hr
= oxygen acceptance rate of	9.5 mg/l/hr

G. Laboratory Determination of ORP

Materials needed:

Beckman Model G pH meter (battery-operated).

Platinum and calomel electrodes.

Sample beakers.

Procedure:

Follow manufacturer's instructions in adjusting instrument for use. Change connections as directed for determining ORP.

Place beaker with sample in position; lower electrodes.

The original reading in millivolts is not particularly significant since it takes some time for the system to become poised. The procedure followed was to take a reading approximately 1 min after the electrodes were immersed in the sample; then, depending on the rate of drift, to continue checking the reading until the drift was negligible. This required from 2 to 5 min depending on the character of the sample and on the final ORP reading.

Sample calculation:

$EMF_{calomel}$ = -210 mv.

Adjusting for calomel-hydrogen correction of +245 mv.

$EMF_{hydrogen}$ = -210 + 245 = +35 mv. = ORP, millivolts.

XXI. APPENDIX E. LABORATORY PROCEDURES USED IN THE DETERMINATION OF SEWAGE SOLIDS

A. Evaporation Method for SS Determination

Materials needed: (for single sample, duplicate determinations)

Analytical balance.

103° C drying oven.

Steam bath.

4 100 ml porcelain evaporating dishes.

Dessicator.

100 ml pipet.

Funnels, flasks, beakers for preparing composite and for filtering.

Whatman No. 12 folder filter paper.

Procedure:

Dry 4 clean evaporating dishes in 103° C oven for one hour.

Place them in dessicator to cool; allow at least 30 mins.

Weigh the dishes and place on steam bath.

While plunging continuously, pipet from unblended composite and then discharge into each of two dishes exactly 100 ml of the whole (unfiltered) sample.

Pass 250-300 ml of whole sample through Whatman No. 12 folded filter paper; collect this filtrate.

Pipet enough from this filtrate to rinse the pipet; waste this portion.

Pipet from the filtrate and discharge into each of two dishes exactly 100 ml of this filtrate.

After evaporation is complete, place the dishes in the oven; dry for a full hour after the 103° C temperature is reached.

Place in dessicator to cool; then weigh.

For sample calculation:

See next page. Note that this method also provides TS and DS results.

B. Sample of Data Sheet Used in Evaporation Method

City of Ames
Sewage Treatment Plant

SOLIDS IN SEWAGE

Date 6/26/58

Sample	Preaeration influent			Sample	Preaeration effluent	
			WHOLE			
Dish No.	66.80	64.70		66.17	66.85	Dish No.
	66.8508	64.7918	2 Dried solids in dish 2	66.2241	66.9301	
	66.7507	64.6929	1 Initial wt. of dish 1	66.1259	66.8323	
	1001	989	Total Solids	982	978	
			(2) Dried solids in dish(2)			
	______	______	3 Ash left on ignition 3	______	______	
			Total Vol. Solids			
			FILTERED			
Dish No.	65.63	58.88		65.79	65.87	Dish No.
	65.7005	58.9500	2 Dried solids in dish 2	65.8115	65.9160	
	65.6223	58.8718	1 Initial wt. of dish 1	65.7365	65.8404	
	782	782	Dissolved Solids	750	756	
			(2) Dried solids in dish(2)			
	______	______	3 Ash left on ignition 3	______	______	
			Dissolved Vol. Sol.			

Influent			Effluent		
Av'g.	TS:	995	Av'g.	TS:	980
Av'g.	DS:	782	Av'g.	DS:	753
	SS:	213 mg/l		SS:	227 mg/l

B. (Continued)

Date ____________

Sample	Primary 1 + 2 Effluent					Sample	Primary 3 + 4 Effluent	
				WHOLE				
Dish No.	62.64	62.45				62.38	62.70	Dish No.
	62.7061	62.1752	2	Dried solids in dish	2	62.4000	62.7799	
	62.6232	62.0933	1	Initial wt. of dish	1	62.3205	62.7010	
	829	819		Total Solids		795	789	
			(2)	Dried solids in dish	(2)			
	______	______	3	Ash left on ignition	3	______	______	
				Total Vol. Solids				
				FILTERED				
Dish No.	61.50	61.96				61.43	61.10	Dish No.
	61.7128	62.0235	2	Dried solids in dish	2	61.6047	61.1996	
	61.6408	61.9509	1	Initial wt. of dish	1	61.5324	61.1277	
	720	726		Dissolved Solids		723	719	
			(2)	Dried solids in dish	(2)			
	______	______	3	Ash left on ignition	3	______	______	
				Dissolved Vol. Sol.				

Av'g. TS: 824
Av'g. DS: 723
SS: 101 mg/l

Av'g. TS: 792
Av'g. DS: 721
SS: 71 mg/l

C. Photometric Determination of SS

Materials needed:

Bausch and Lomb Spectronic 20 Colorimeter.

Matched 1 in test tubes for Spectronic 20.

Whatman No. 12 folded filter paper.

Funnels, flasks, etc. for filtering.

Procedure:

Plug in Spectronic 20, set at 450 mu and allow warmup period; then adjust for 0 percent reading without test tube in holder and for 100 percent reading with distilled water sample.

Agitate the sample thoroughly; pour quickly, almost filling 1 in test tube; wipe dry, shake and place in holder; read transmittance quickly before settling can occur in the test tube.

Pour this sample from the test tube into a filter paper; collect the filtrate to predetermined volume, such as 1/3 or ½ the capacity of the test tube. Consistency here will improve accuracy of results.

Place filtrate in same test tube in the holder and read the transmittance again.

Calculate the ratio of the two readings; with this ratio, determine the SS content from the calibration curve or table.

Example for raw sewage:

$$\frac{\text{Whole sample transmittance, } Tr_w}{\text{Filtrate transmittance, } Tr_f} = \frac{21}{60} = 35 \text{ percent calculated ratio.}$$

35 percent = 195 mg/l SS content from calibration table 39.

Table 39. SS calibrations for use with Bausch and Lomb Spectronic 20 Colorimeter; 450 mu

Calculated ratio: Tr_w/Tr_f	SS, mg/l									
	0	1	2	3	4	5	6	7	8	9
					Raw sewage					
0								600	560	530
10	500	480	460	440	420	400	385	370	355	340
20	325	310	298	287	276	266	256	247	239	232
30	225	219	213	207	201	195	190	184	179	174
40	170	165	160	155	150	145	141	136	132	127
50	123	119	115	111	108	104	100	96	93	90
60	87	83	80	76	73	70	68	65	62	59
70	56	53	51	48	46	43	41	38	36	34
80	32		28		24		21		18	
90	15									
					Primary effluent					
0										
10										
20						175	170	165	160	155
30	150	145	140	135	131	127	123	119	115	111
40	107	103	100	97	93	90	87	84	81	78
50	75	72	69	67	65	62	60	58	56	54
60	52	50	48	46	44	43	41	39	38	36
70	35		32			28		26		
80	22		20			16		14		
90	10									

D. Tabulation and Plotting of SS Data

For daytime sampling runs, four points were sampled, usually beginning at 10:30 AM:

Preaeration influent
Preaeration effluent
Plain primary effluent
Preaerated primary effluent

Sampling continued at all four points through 9:30 PM. Using ½ hr and 1 hr as generally representative of preaeration and settling detention times, respectively, samples were composited in proportion to flow for these 10 hr periods:

Preaeration influent:	10:30 AM - 8:00 PM	inclusive
Preaeration effluent:	11:00 AM - 8:30 PM	"
Plain primary effluent:	11:30 AM - 9:00 PM	"
Preaerated primary effluent:	12:00 Noon - 9:30 PM	"

Duplicate photometric SS determinations, or more if indicated, were made on each composite sample.

The SS strength of each individual sample was determined photometrically, and the results tabulated as shown in table 40. This data was used for plotting the patterns of raw and primary effluent SS strengths as shown in figure 70.

Table 40. Typical SS results for individual samples, June 26, 1958

Time	Preaeration influent			Preaeration effluent		
	Tr_w/Tr_f*	Calculated ratio	SS mg/l	Tr_w/Tr_f	Calculated ratio	SS mg/l
10 AM						
:30	16½/72½	23	290	20/68	29½	228
11	16/71	22½	292	18/68	26½	251
:30	16/71½	22½	292	16/64	25	266
12 N	13/65	20	325	15½/61	25½	261
:30	19½/69½	28	239	16/64	25	266
1	19½/77½	25	266	18½/68	27	247
:30	17/69½	24½	271	18/64½	28	239
2	14/61	23	287	15/62½	24	276
:30	19½/68	28½	235	16/60½	26½	251
3	25½/71	36	190	19/63	30	225
:30	22/69	32	213	20/64½	31	219
4	21½/72½	29½	229	21/64½	32½	210
:30	22/69	32	213	21½/71	30	224
5	28½/74	38½	176	22½/75	30	225
:30	18½/70½	26	254	22½/75	30	225
6	35½/78	45½	143	26/76	34	200
:30	28½/77	35	184	28/80½	35	196
7	35/80	43½	152	28½/78	36½	187
:30	28/78½	35½	193	32½/78	41½	162
8	35/77	45½	143	33/76	43½	152
:30	32½/76	42½	157	35/77	45½	143
9	33½/73	46	141	32½/76	43	156
:30	32½/74	44	150	31½/79	40	170
10 PM						

Preaeration influent: 10 hr proportional composite, 10:30 AM - 8 PM, inclusive. (duplicate det'n.)

22/74	29.8
22/73½	30.0

Composite: <u>226 mg/l</u>

Preaeration effluent: 10 hr proportional composite, 11 AM - 8:30 PM, inclusive.

23/71	32.4
23½/71½	32.9

Composite: <u>209 mg/l</u>

*Transmittance (whole)/Transmittance (filtrate).

Table 40. (Continued)

Time	Plain primary effluent			Preaerated primary effluent		
	Tr_w/Tr_f	Calculated ratio	SS mg/1	Tr_w/Tr_f	Calculated ratio	SS mg/1
10 AM						
:30	43½/79	55	62	65½/86½	75	29
11	35/75	46½	86	51½/82	63	46
:30	37½/75½	49½	76	46/78½	58½	54
12 N	32/72½	44	92	52/80	65	43
:30	28/68½	41	103	38/74	51½	70
1	29½/71	41½	102	35/72	48½	80
:30	30½/73½	41½	102	38½/72	53½	66
2	30/72½	41½	102	35/71½	49	78
:30	31½/71½	44	93	37/73	50½	73
3	23½/68	34½	129	36/70	51½	70
:30	26/68	38	115	30/66	45½	88
4	30/69½	43	96	33/69½	47½	83
:30	29½/70	42	100	32½/64	51	73
5	30/69	43½	95	33/63½	52	69
:30	31½/71	44½	91	33/66	50	75
6	32/71	45	90	35/68½	51	72
:30	35/74	47½	83	37/70	53	67
7	36/73	49½	76	38½/70	55	62
:30	38½/75	51½	70	39/72	54	65
8	37½/77	48½	80	42/72	58½	54
:30	42½/77½	55	63	44½/74	60	52
9	42½/77	55	61	45½/75	60	52
:30	43½/76½	57	58	46/76½	60	52
10 PM	43½/76½	57	58	46/76½	60	52
10 hr proportional composite, 11:30 AM - 9 PM, inclusive. (duplicate det'n.)				10 hr proportional composite, 12 N - 9:30 PM, inclusive.		
	33/73	45.2		39/74½	52.4	
	32½/73	44.6		39/75	52.0	
	Composite:		90 mg/l	Composite:		69 mg/l

XXII. APPENDIX F. LABORATORY AND CALCULATION PROCEDURES FOR THE OULMAN SETTLIMETER

Materials needed:

Bausch and Lomb Spectronic 20 Colorimeter, with matched 1 in test tubes.

Filter paper, funnels, flasks, etc. as described for photometric SS determination.

Oulman Settlimeter, comprising light source, photocells and galvanometer (figure 71).

Matched pairs of 2 in cylindrical glass settling cells.

Sampson sampler for undisturbed filling and withdrawal of sample cell.

Sampling bottle, 250 ml, as used for individual SS samples.

Stop watch.

Procedure:

Plug in Spectronic 20, set at 450 mu and allow warmup period; then adjust for 0 percent reading without tube in holder and for 100 percent reading with distilled water sample.

Plug in Oulman Settlimeter and allow warmup period; adjust for 100 percent reading with distilled water sample in cell positions 1 and 2; leave these cells in place for the time being.

Take the matched 2 in cell for position 1, a 250 ml bottle and the Sampson Sampler to the point of inflow to the preaeration tank.

Place the cell in inverted position in the sampler; submerge it completely; slowly release entrapped air from within the cell by controlling its exhaust through the air tube.

While this cell is being filled, take a grab sample at the same place with the 250 ml bottle.

When the 2 in cell is filled, rotate the hinged sampling assembly to turn the cell upright; then bring it to the surface and disengage the settling cell.

Bring both samples into the laboratory promptly. First shake the 250 ml sample vigorously, then add an aliquot portion to fill the 2 in cell to overflowing.

Then gently invert the 2 in cell several times to get optimum uniformity of its contents; conclude this by flipping out approximately the top 3/4 in of its contents, to bring the level down to a uniform depth mark. Invert gently once more; then remove distilled water sample from cell position 1, insert the sample cell, place the fitted lid over the cell holder and read the galvanometer immediately. Every second is important at this point since settling occurs very rapidly at first, as reflected by transmittance (galvanometer) readings.

Read the galvanometer at 1 min intervals for 10 mins, then at longer intervals for 45 mins to an hr or longer if desired.

As soon as possible, determine the SS content of the remainder of the 250 ml grab sample photometrically.

At a later time which reflects probable detention time in the preaeration tank, collect a sample of preaeration effluent in an identical manner and determine its settling rate in cell position 2. Since individual readings take but a few secs, it is not impractical to have all four positions in use. However, it is wise to have an extra pair of hands available in this case, for sampling, for cleaning glassware between runs and for companion determinations with the Spectronic 20 Colorimeter.

Calculation:

Determining the SS content of a sewage sample theoretically identical with that used in the settling rate test makes possible an evaluation of settling results in terms of SS. The steps are illustrated with data from a pair of actual tests.

As shown in table 41, the settlimeter reading at time zero for the preaeration influent sample was 52 ua. The B and L transmittance readings for the companion sample were: Tr_w, 19 percent and Tr_r, 56 percent. Calculated ratio, 19/56 = 34, which indicates (table 41) a SS content of 193 mg/l.

In the semilog plot, figure 72, locate point A at 52 ua and 19 percent transmittance. Connect point A with the intersection representing 100 percent transmittance for both instruments; this is identified as line A.

At time 5 mins, the settlimeter reading was 55 ua. In figure 72, 55 ua is intersected by line A at 21 percent transmittance. This provides a new calculation Tr_w/Tr_r of 21/56 = 37½, equivalent to

165 mg/1 of SS. Thus, as settling continues, settlimeter (galvanometer) readings increase; they can be expressed in terms of decreasing SS content, and the percent removal calculated, as in the last column of table 41.

For the preaeration effluent sample, point B is established from the settlimeter reading of 58½ ua at time zero, and the Spectronic 20 reading of 22 percent transmittance.

This method may appear arbitrary because lines A and B are drawn to points of 100 percent transmittance established for distilled water. However, samples drawn from the settling cells at various times during a run and analyzed photometrically for SS content will fall on these lines, within the limits of experimental and human error.

Plotting results:

For the pair of samples used in the illustration above, the settling rate following preaeration showed significant improvement over that for raw sewage. This can be shown by plotting either mg/1 of SS or percent removal against time, either on semilog scale as in figure 73 or on rectangular coordinates as shown in figure 74.

The semilog relationship serves to mask the difference in settling characteristics, although it does demonstrate the straight-line function which obtains after approximately 5 mins of settling. Extended tests for as long as 2 or 2½ hrs indicated this straight-line function to hold, making the results of such long laboratory runs reasonably predictable.

By contrast, the rectangular plot indicates how sharply the settling rates diverge in the first 10 to 15 mins. Often in 20 mins almost all the potential margin of improvement by preaeration has occurred and the settling rates beyond this point are generally comparable. This is of prime significance.

Table 41. Typical data tabulation for determining settling characteristics by Oulman Settlimeter

Sample: Preaeration influent picked up 3:20 PM, June 26
Spectronic 20 readings on this sample: Tr_w/Tr_f: 19/56; calc. ratio = 34

Time PM	Increment min	Settlimeter reading, ua	Corresponding Tr_w by B & L	Calc. ratio Tr_w/Tr_f**	Calc. mg/l SS (B & L calib)	Calc. percent removal of SS by settling
3:25	0	52	19*	34	193	--
30	5	55	21	37½	165	14½
35	10	56½	22	39½	150	22½
40	15	57½	23	41	140	27½
45	20	58½	24	43	126	34½
50	25	60	25	44½	117	39½
4:02	37	62	27	48	95	51
05	40	62	27	48	95	51
10	45	63	27½	49	89	54
20	55	63½	28	50	84	56½
Sample: Preaeration effluent picked up 3:50 PM, June 26; Spectronic 20 readings on this sample: Tr_w/Tr_f: 22/62½; calc. ratio = 35						
3:55	0	58½	22*	35	185	--
4:02	7	62½	25½	41	140	24
05	10	65	28	45	114	38
10	15	66½	29½	47	101	45½
20	25	68½	31½	50½	81	56
25	30	70	33½	53½	68	63
35	40	71	34½	55	62	66½
40	45	71½	35½	57	56	69½
45	50	72	36	57½	55	70

*Read directly in Bausch and Lomb Spectronic 20: all succeeding values in this column are read from semilog conversion plot figure 72.

**In calculating this ratio, Tr_f is constant at 56 for the preaeration influent sample and constant at 62½ for the preaeration effluent sample.

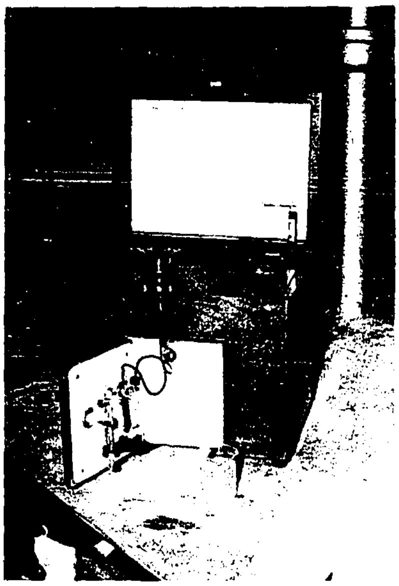

Fig. 28. Beckman Model G pH meter used in ORP determination

Fig. 29. Filtration of sewage samples for photometric SS determination

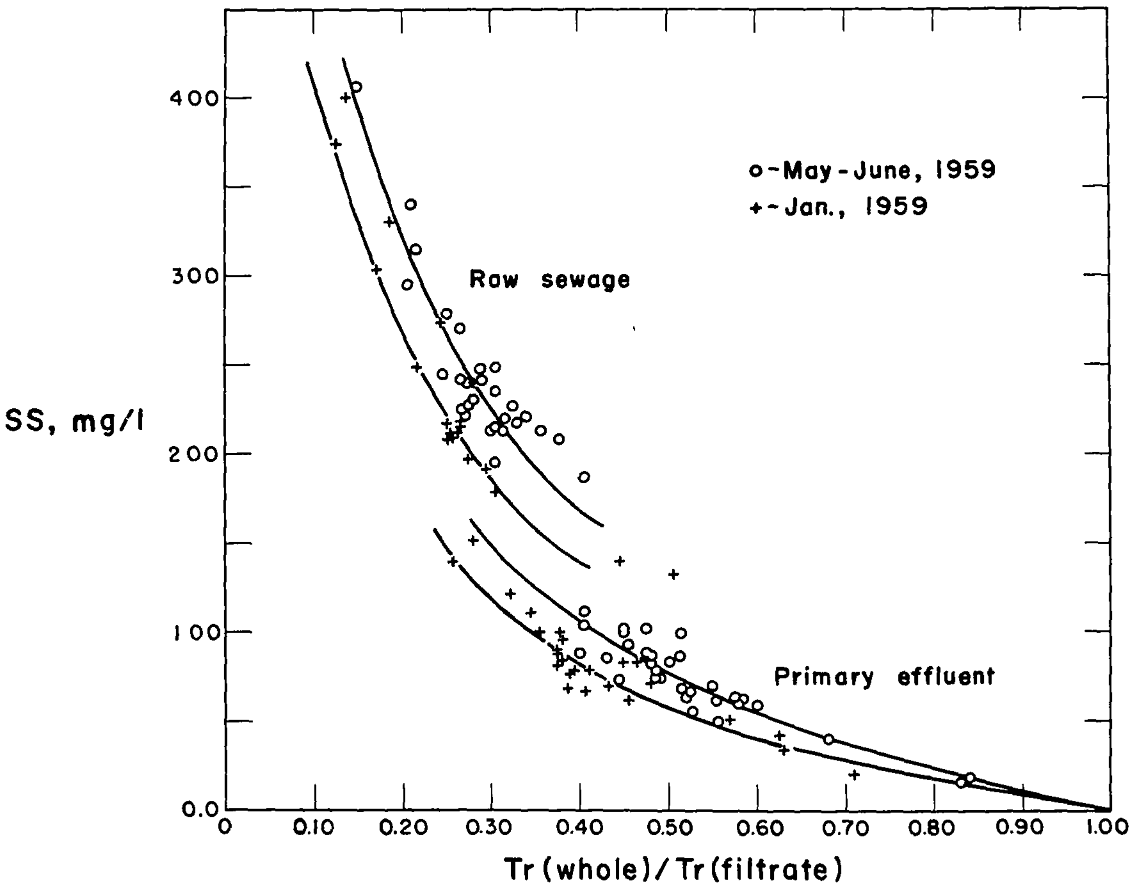

Fig. 30. SS calibrations for use with Spectronic 20 Colorimeter