DESIGN GUIDES FOR BIOLOGICAL WASTEWATER TREATMENT PROCESSES

Performance of Biological Treatment Processes

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SYMBOLS AND ABBREVIATIONS

a	fraction of substrate converted to bacterial mass
a'	oxygen consumed per unit of substrate removed
b	substrate removed per unit of time, by endogenous respiration
b'	endogenous respiration rate
BOD	biochemical oxygen demand
BODF	filtered (assumption: soluble) BOD_5
BODT	total BOD_5
^oC	degrees centigrade
c	oxygen concentration
COD	chemical oxygen demand
CODF	filtered (assumption: soluble) COD
CODT	total COD
^oF	degrees Fahrenheit
IC	inorganic carbon
k	reaction rate coefficient, related to MLSS
K	reaction rate coefficient, MLSS included
k_a	reaction rate coefficient, related to active MLSS
K_a	reaction rate coefficient, active MLSS included
K_h	heat transfer coefficient
k_m	substrate-concentration at which the reaction-rate equals one-half k_{max}
k_R	k_{max} divided by a
k_v	reaction rate coefficient, related to MLVSS
k_{max}	maximum removal rate, Monod - Michaelis-Menten equation

K_v	reaction rate coefficient, MLVSS included
K_v	equals K, K_a
K_T	reaction rate coefficient, at temperature T, ^oC
K_{20}	reaction rate coefficient, at 20^oC
MLSS	mixed liquor suspended solids
MLVSS	mixed liquor volatile suspended solids
NH_3	ammonia
NO_{2+3}	nitrite + nitrate
OUR	oxygen uptake rate
Q	flow
R	oxygen used, per unit weight of mixed liquor and per unit of time
S	soluble substrate concentration, tank
S_{o_s}	soluble substrate concentration, influent
ST_e	total substrate concentration, supernatant after settling
ST_o	total substrate concentration, influent
ΔS	soluble substrate removed, $S_o - S$
SS	suspended solids
t	hydraulic detention time, in completely mixed basins equal to the sludge age, $t = V/Q$
T	temperature
TKN	total Kjeldahl nitrogen
TC	total carbon
TOC	total organic carbon
TOCF	filtered (assumption: soluble) TOC
TOCT	total TOC

INTRODUCTION

The operation of treatment plants and evaluation of the performance of full-scale biological processes under a wide range of organic and hydraulic loadings are seldom undertaken. The principal objective of a plant operator after the system is constructed and equipment installed is to produce the best effluent possible. Therefore, in most cases the limitations of a particular process or of the total plant are not established until the organic or hydraulic capacity of the system is exceeded as indicated by deterioration of the plant effluent.

The study included an evaluation of the performance of the conventional activated sludge process, the contact stabilization process, the extended aeration process, the aerated lagoons, waste stabilization ponds, and trickling filters through a well-coordinated laboratory and field program.

The objectives of this study were:

a. The development of updated design guidelines for biological treatment processes.

b. The establishment of the effective operating ranges of hydraulic and organic loading by monitoring the effects of these variables on the performance of the various biological treatment processes.

c. The comparison of the performance of laboratory-scale treatment units with field installations to evaluate comprehensive scale-up information, mathematical correlations and operational guidelines.

d. Analysis and evaluation of laboratory and field procedures and results to improve and standardize laboratory methods and engineering design formulations.

Experimental programs involving the different processes were carried out at the various municipal wastewater treatment plants of the City of Austin, Texas. The design of these plants permitted unusual flexibility of operation of one or more of the biological systems treating municipal wastewater.

The development of this project at the City of Austin afforded unique advantages in that all of the aerobic biological processes with the exceptions of trickling filters were operational or were made operational with minor structural and hydraulic changes. Experimental trickling filter and pilot scale stabilization pond systems were installed at the Govalle Wastewater Treatment Plant. Therefore, comparative results were obtained on the same municipal wastewater which contained essentially no industrial wastewater.

The project encompassed four major phases:

a. construction and modification of existing facilities

b. collection of operational data from laboratory, pilot, and full-scale units

c. evaluation and correlation of data

d. preparation of design guidelines

The processes evaluated under various hydraulic and organic loading conditions included:

a. activated sludge operated as:
    1. the conventional activated sludge process with longitudinal flow-through tanks
    2. the contact stabilization process
    3. the extended aeration process
b. trickling filters using media consisting of:
    1. corrugated plastic sheets
    2. plastic rings
    3. rock
c. aerated lagoons
d. waste stabilization ponds

Analytical procedures for automated sample analyses were developed for the large number of analyses required for the simulataneous evaluation of the processes operated in parallel. An improved system of sample storage and handling was also developed.

The performance of the various processes was evaluated in terms of the removal of BOD, suspended solids, organic carbon and nutrients. These operating data were evaluated and compared with data reported for plants at other locations. This report includes all the operating data in Appendix A. Reports in which the data are evaluated are included in Appendix B. The design guidelines are submitted as a separate monograph.

DESCRIPTION OF THE TREATMENT PLANTS

Biological processes installed at the Govalle and Williamson Creek Wastewater Treatment Plants of the City of Austin, Texas were evaluated in this study. Pilot-scale trickling filter and waste stabilization ponds were constructed at the Govalle Plant.

A. Govalle Wastewater Treatment Plant

The original Govalle Wastewater Treatment Plant was constructed in 1936 in Austin, Texas and consisted of bar screens, aerated grit chamber, one primary clarifier, two aeration tanks, two final clarifiers and two two-stage digesters. The plant was expanded in the early 1950's to accommodate the increased flow and the biological treatment system was converted from the conventional activated sludge process to the contact stabilization system. At that time, additional bar screens, grit chamber, and two final clarifiers were added and the original primary clarifier was converted to a final clarifier. New carborundum diffuser tubes were installed in the aeration tanks. About one-half of the aeration tank volume was available for the contact portion of the process and after final clarification, the concentrated sludge was passed into the second half of the aeration tank for stabilization. Each aeration system was connected to separate clarifiers and had separate blowers, return sludge piping, and waste sludge pumps. The expanded plant had a capacity of 14 to 16 MGD.

In 1960, the plant was expanded once again and the third aeration tank and clarifiers were added to the existing plant. A grease skimming

tank was installed before the aeration tanks in order to minimize floating material in the final clarifiers. A second skimming tank was added in 1966. The floating material and grease removed in the skimming tanks were sent to the anaerobic digestion system. The solids that settled in the short detention time in the skimming tanks were pumped back into the wastewater entering the aeration basins.

A schematic flow diagram of the Govalle Wastewater Treatment Plant as it appeared during this particular study is shown in Figure 1. More descriptive flow diagrams are illustrated in Figures 2 and 3. During the final phase of this study, the Govalle Wastewater Treatment Plant was once again expanded. However, the new portion of the plant was not completed and the performance of the new addition was not evaluated in this study; therefore no description will be provided. Distribution of the incoming wastewater flow among the three contact stabilization units is as follows: one-third of the flow enters the most recently constructed plant (C Plant), and the remainder of the flow is distributed between the older portions of the plant, namely A Plant and B Plant.

The circular final clarifiers are equipped with sludge collection rakes and a skimming device. The floating solids are removed by skimming, thickened and discharged into the waste sludge wet well to be pumped to the sludge lagoons. The effluent of the final clarifiers is chlorinated and discharged into the Colorado River.

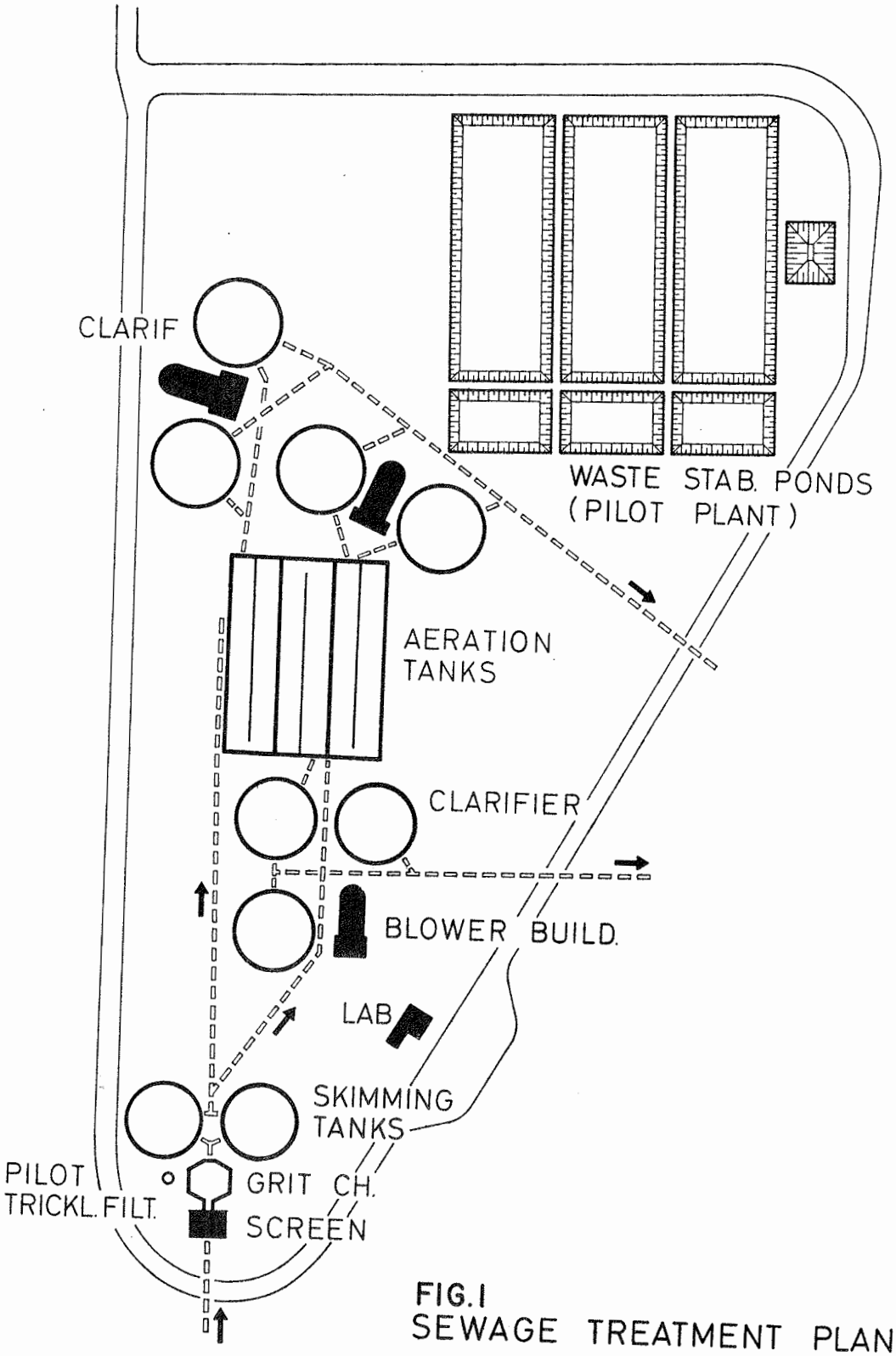

FIG.1
SEWAGE TREATMENT PLANT
"GOVALLE" AUSTIN, TEXAS

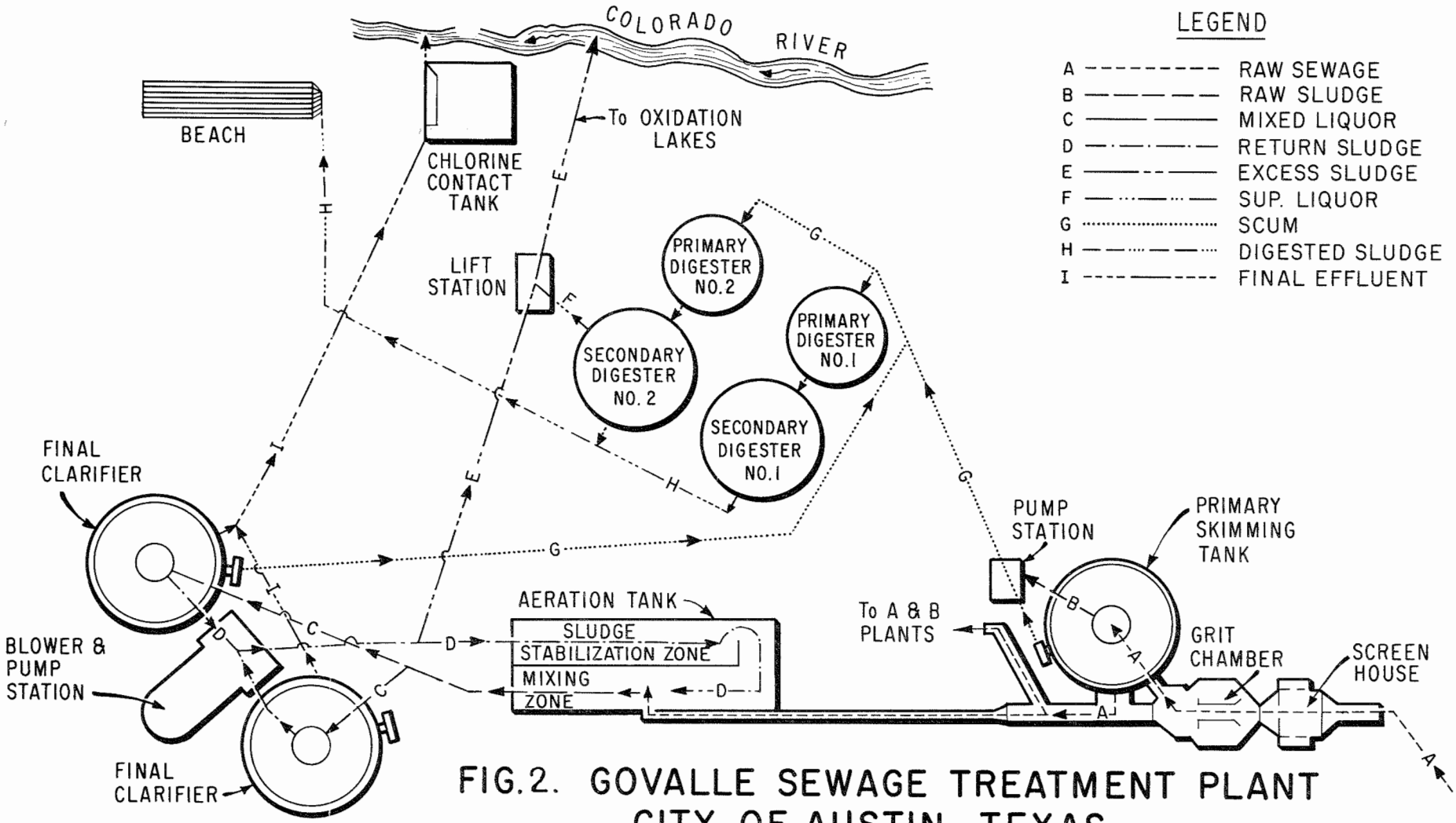

FIG. 2. GOVALLE SEWAGE TREATMENT PLANT
CITY OF AUSTIN, TEXAS

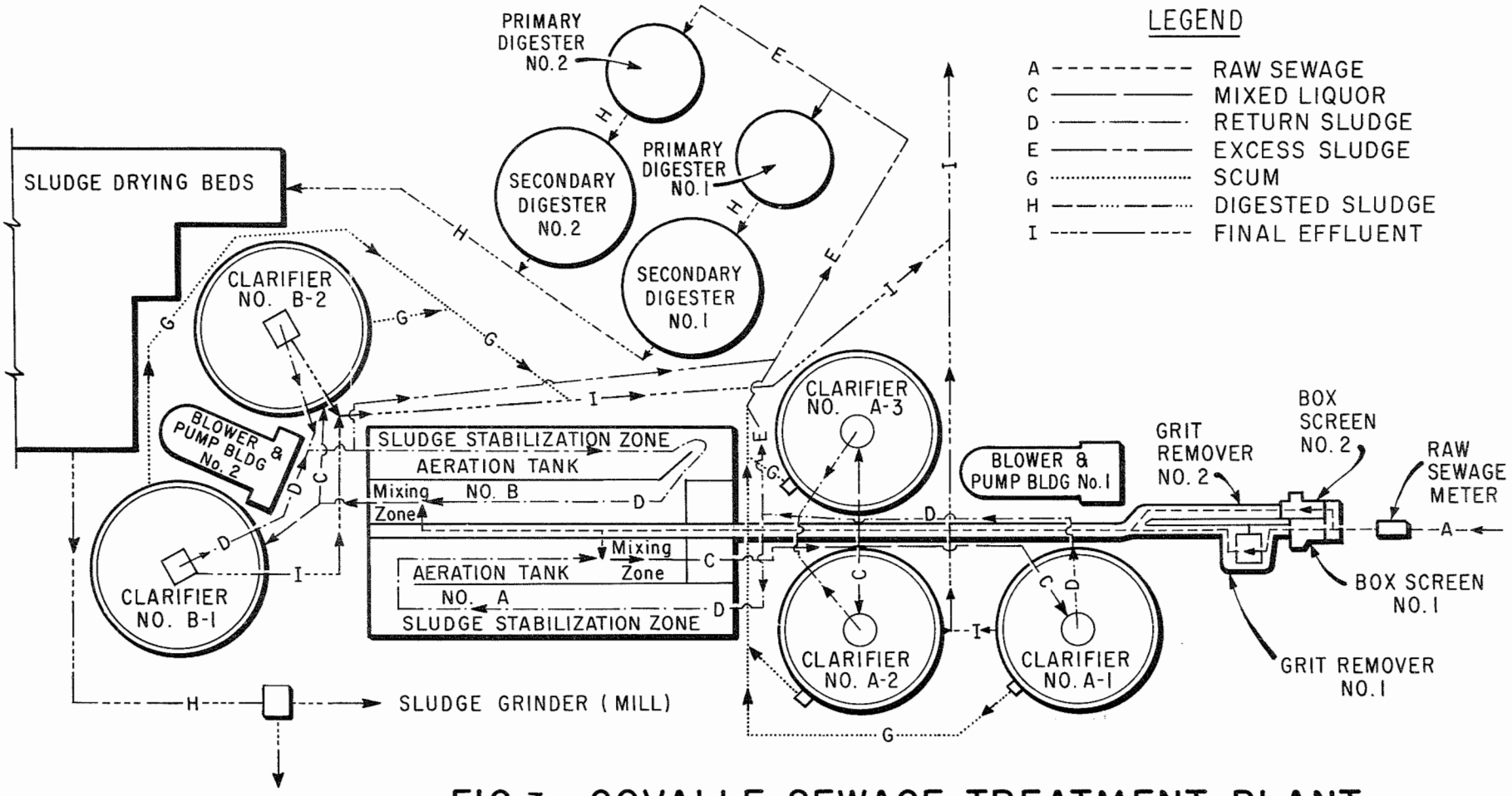

FIG.3 GOVALLE SEWAGE TREATMENT PLANT
CITY OF AUSTIN, TEXAS

Waste activated sludge from each aeration system is pumped separately from the main sludge return line. Normally one pump is operated and is controlled by an automatic timer. The waste sludge is pumped into a wet well for subsequent pumping to sludge disposal lagoons at Hornsby Bend, Texas which is located about five miles from the Govalle Plant site. The waste activated sludge is mixed with river water in the lagoons and is treated in this manner. These lagoons were constructed in 1956 at which time the anaerobic digestion system was also taken out of service.

1. Activated Sludge Process

The details of the aeration tank design are illustrated in Figure 4. The incoming wastewater can be introduced through any one of four inlet channels located along one side of each unit. The contact time can be varied by changing the point at which the wastewater enters the basin.

Some plant modifications were necessary in order to permit the operation of one of the aeration tanks as a conventional activated sludge process while the other two tanks were operated as contact stabilization systems. The A Plant was converted to the conventional activated sludge process since this system included the only aeration tank in which the wastewater and return sludge could be introduced at one end of the tank without extensive modification in the plant piping.

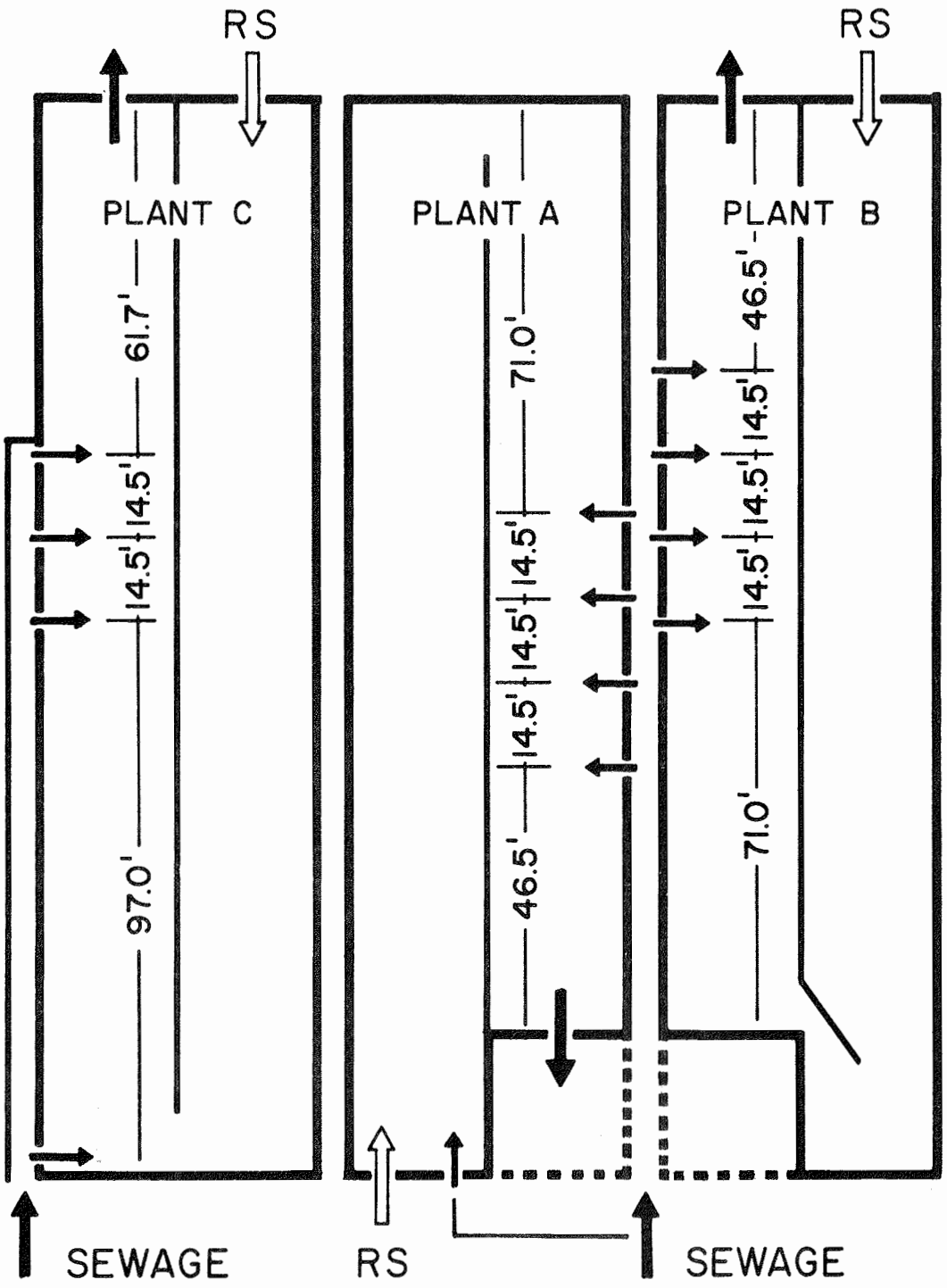

FIG.4
AERATION TANKS OF GOVALLE

A cross section of the aeration tanks is shown in Figure 5. The cross sectional area of the aeration tank is 360 square feet and the nominal dimensions of the tank are 25 feet wide and a side water depth of 15 feet. Air is distributed uniformly throughout the tank by carborundum diffuser tubes which are four feet long and spaced six inches on center. The diffusers are located about three feet off the bottom of the aeration tank and therefore are operated at a head of about 12 feet of water. The oxygen transfer for this aeration equipment is 28.5 pounds of oxygen per thousand gallons of aeration tank volume and the specific air flow rate is about 15 cubic feet of air per thousand gallons of aeration tank. This oxygen transfer is based on clean diffusers and as the diffuser tubes become clogged, the oxygen transfer is reduced. The volume of the aeration tanks and the capacity of pumps, blowers, and other basic information are summarized in Table 1.

a. Conventional Process

The wastewater flow which could be effectively treated by A Plant when operated as the conventional activated sludge process was limited by the size of the piping between the aeration tank and final clarifier as well as the hydraulic capacity of the clarifier. The maximum flow including incoming wastewater and return sludge which could be treated was approximately ten MGD. The rate of wastewater flow to the A Plant was recorded on flow meters which were so installed that the incoming wastewater to A Plant and B Plant were recorded on one meter and the flow to C Plant on the second meter. Flow meters were also installed to record

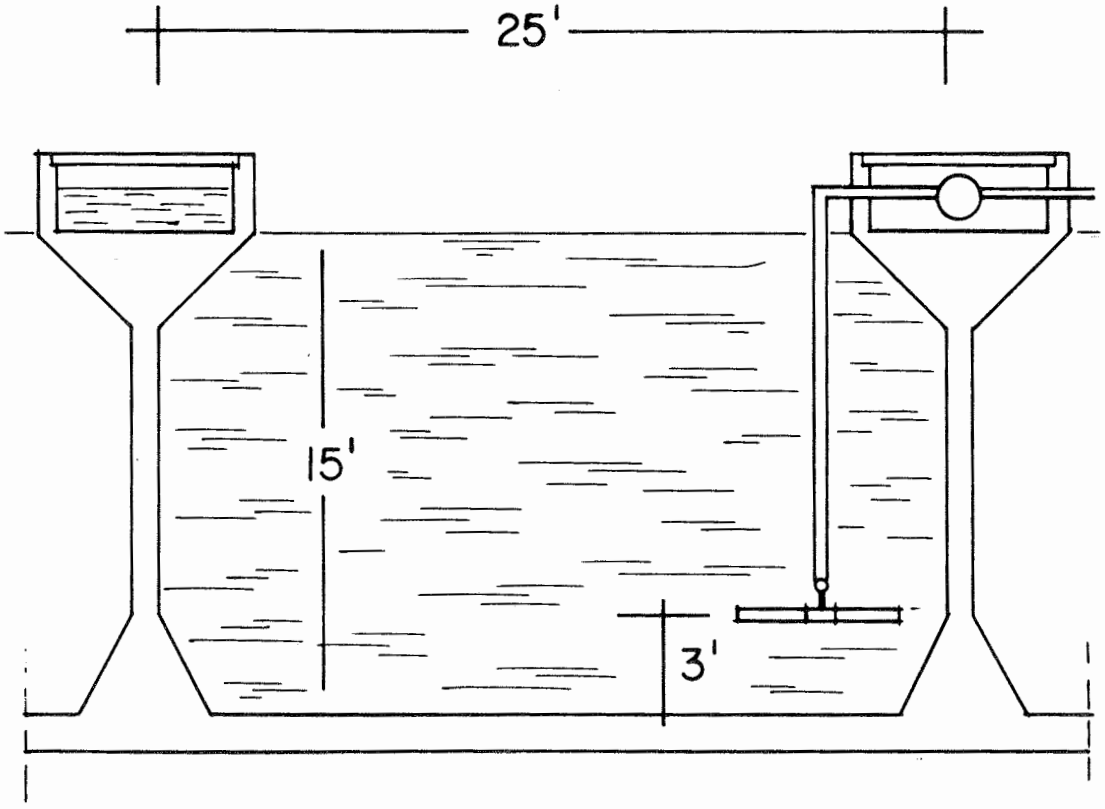

FIG. 5

AERATION TANK "GOVALLE"
CROSS SECTION

TABLE 1

GOVALLE SEWAGE TREATMENT PLANT, AUSTIN, TEXAS
SUMMARY OF PLANT CHARACTERISTICS

		Plant A	Plant B	Plant C
Aeration Tank Vol	mg	0.938	0.938	1.000
Return Sludge Pumps	mgd	1.584	3.456	3.744
	mgd	3.024	5.040	4.608
	mgd	5.040		5.760
Waste Sludge Pumps	mgd	0.288	0.288	0.432
	mgd	0.432	0.288	
Total Blower Capacity	scfm	6,000	14,100	15,880
Clarifier Surface Area	sq ft	13,251	11,350	11,350

the final effluent flow from the B and C Plant clarifiers. Therefore, by subtracting the effluent from the B Plant from the influent to the A and B Plants, the flow to the A Plant was determined. Control of the concentration of suspended solids in the return sludge and of the return sludge flow rate was almost impossible. The return sludge pumps operated at a constant rate and were controlled by an automatic timer. No flow meters were installed in the return sludge line; therefore the rate of return sludge was estimated based on the rated capacity of the pump. The results of the conventional activated sludge process evaluation should be reviewed in light of the limitations of the system both in capacity and control of operation.

b. Contact Stabilization Process

The B Plant and C Plant were operated as contact stabilization systems. The contact time and stabilization time were varied by introducing the incoming wastewater at different locations in the aeration tank. Influent gates are located along the side of the front portion of the aeration tanks. By changing the gate through which the wastewater entered the aeration tank, the contact time could be varied. There was an appreciable amount of back-mixing of the incoming wastewater with the stabilized sludge so that it was almost impossible to delineate the contact zone and to determine the exact contact time. The flow rates of the return sludge also varied throughout the study. No meters were installed in the return sludge line, therefore an estimate of the flow rate could only be obtained by using the rated capacity of the return sludge pumps in conjunction

with the record of the time during which the pumps operated. The capacity of the final clarifiers also limited the hydraulic loading that could be applied to the contact stabilization process. Overloading the final clarifiers causes a carryover of suspended solids in the effluent channel, giving an effluent that will not meet regulatory quality standards.

c. Sampling Program

The distribution of the wastewater to the various processes and basins at the Govalle Plant required a sampling program which would provide the sufficient data to permit the evaluation of the performance of the various processes. Samples of the influent wastewater were collected from the channel to each of the aeration tanks. Two samples were required since the flow in one channel went to the A and B Plants and the flow in the C Plant was in the second channel. The major difference in the composition of the two samples was that a slight fluctuation in the concentration of suspended solids was observed between the samples of the two channels. Effluent samples were collected from the effluent channels from the clarifier in each of the aeration systems. The samples were collected manually by the treatment plant personnel each hour , and the 24-hour composite samples were obtained by mixing a volume of each of the hourly samples which were proportioned to the rate of flow at the time the hourly sample was collected.

Samples of the mixed liquor and of the return sludge were collected every two hours. These two-hour samples were mixed to make up a 24-hour composite sample. The sampling procedure was somewhat different for the A Plant which was operated at a conventional activated sludge system and for the B and C Plants which were operated as contact stabilization systems.

The mixed liquor sample in Plant A was collected at the effluent of the aeration tank while the sample of return sludge was drawn from the main sludge return line. Samples of mixed liquor for the B and C Plants were collected at the effluent of the aeration tank after contact with wastewater as well as at a point 25 feet upstream from the influent. The concentration of suspended solids in the return sludge was determined from samples collected at the influent of the return sludge to the stabilization tank.

2. Pilot Trickling Filter

A pilot-scale trickling filter was constructed at the Govalle Plant Site since the original plant did not contain a trickling filter. The trickling filter consisted of a steel tower which was three feet in diameter and has a total height of about 25 feet. The steel tower was broken into three parts. The bottom section in which the effluent was collected on and in which space for eleven feet of packing material was available. The second section included a screen installed at the bottom to provide support for an additional eleven feet of packing material. The final section contained the motor driven distributor as well as the flow regulating devices. The model filter is illustrated in Figure 6. This design in the three sections provided the flexibility needed to operate the filter at different depths of media. Five sampling ports were located at four foot intervals along the depths of the tower as illustrated in Figure 6.

The influent wastewater to the trickling filter was pumped from the skimming tank at a point between the wall and the overflow weir. This location was selected in order to simulate the application of presettled wastewater to the trickling filter. This method of getting feed to

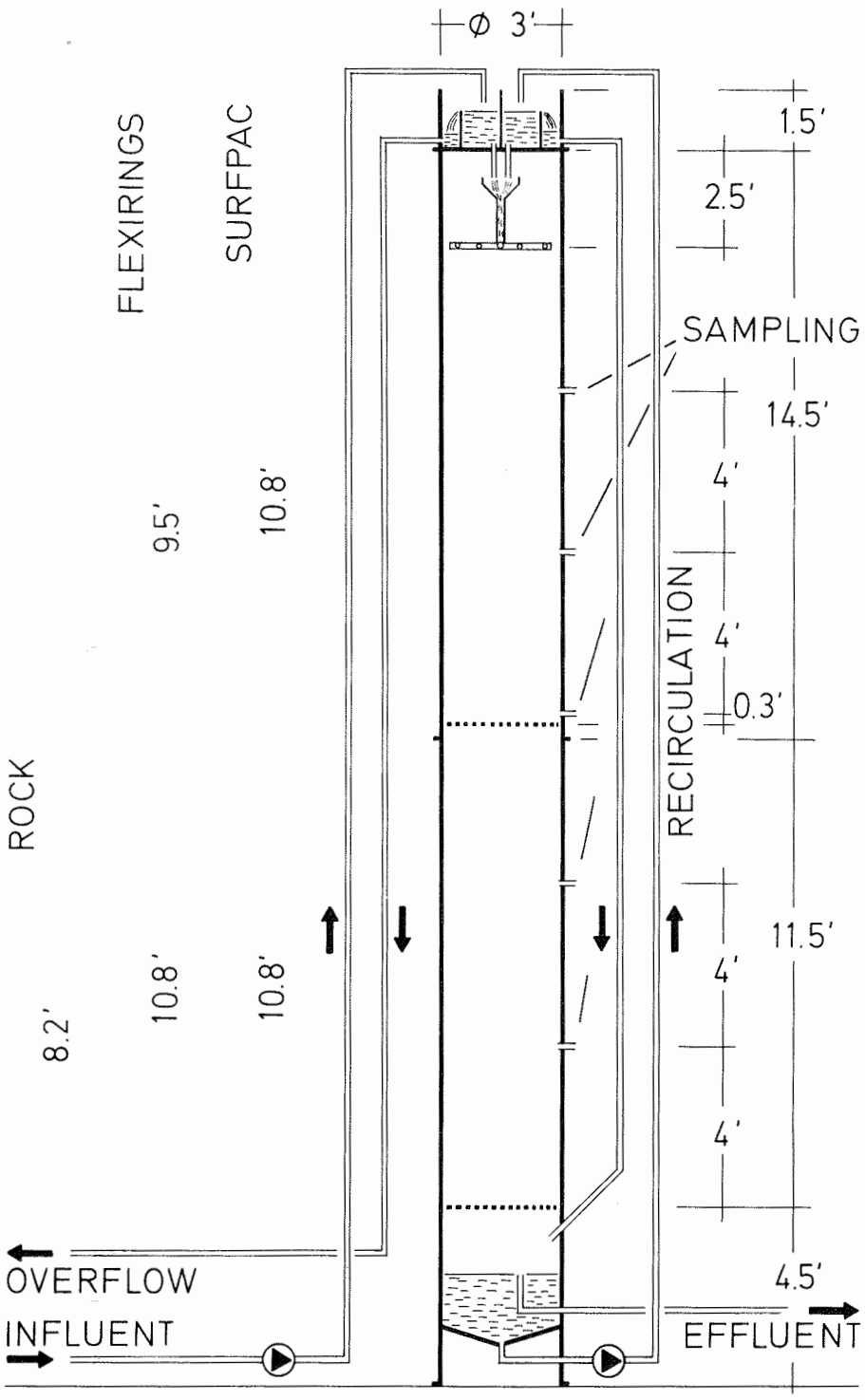

FIG.6
EXPERIMENTAL TRICKLING FILTER

the filter was fine as long as the medium in the filter was a vertical sheet of corrugated plastic. However, other media which were randomly placed resulted in clogging caused by removal and accumulation of suspended solids. For these particular runs, the suction line to the pump was modified as illustrated in Figure 7.

Three types of filter media were used in these experiments and include:

(a) corrugated sheets of saran, which are bonded together into modules which are 20 inches by 20 inches and have a specific surface area of about 27 square feet per cubic foot of media. The modules are light weight and structurally self-supporting and were cut to fit the three-foot diameter experimental filter. A total depth of 21.6 feet of corrugated saran modules were used.

(b) polypropylene rings which were $3\frac{1}{2}$ inches in diameter and $3\frac{1}{2}$ inches high. The rings are similar to those used for absorption towers and have a specific area of 30 square feet per cubic foot. These rings were dumped into the filter resulting in a random packing. The total depth of 20.3 feet of rings were used in these studies.

(c) graded limestone rock which was four to six inches in diameter was used in the third part of the study. These rocks have a specific surface of ten square feet per cubic and a total depth of 8.2 feet of rock was used.

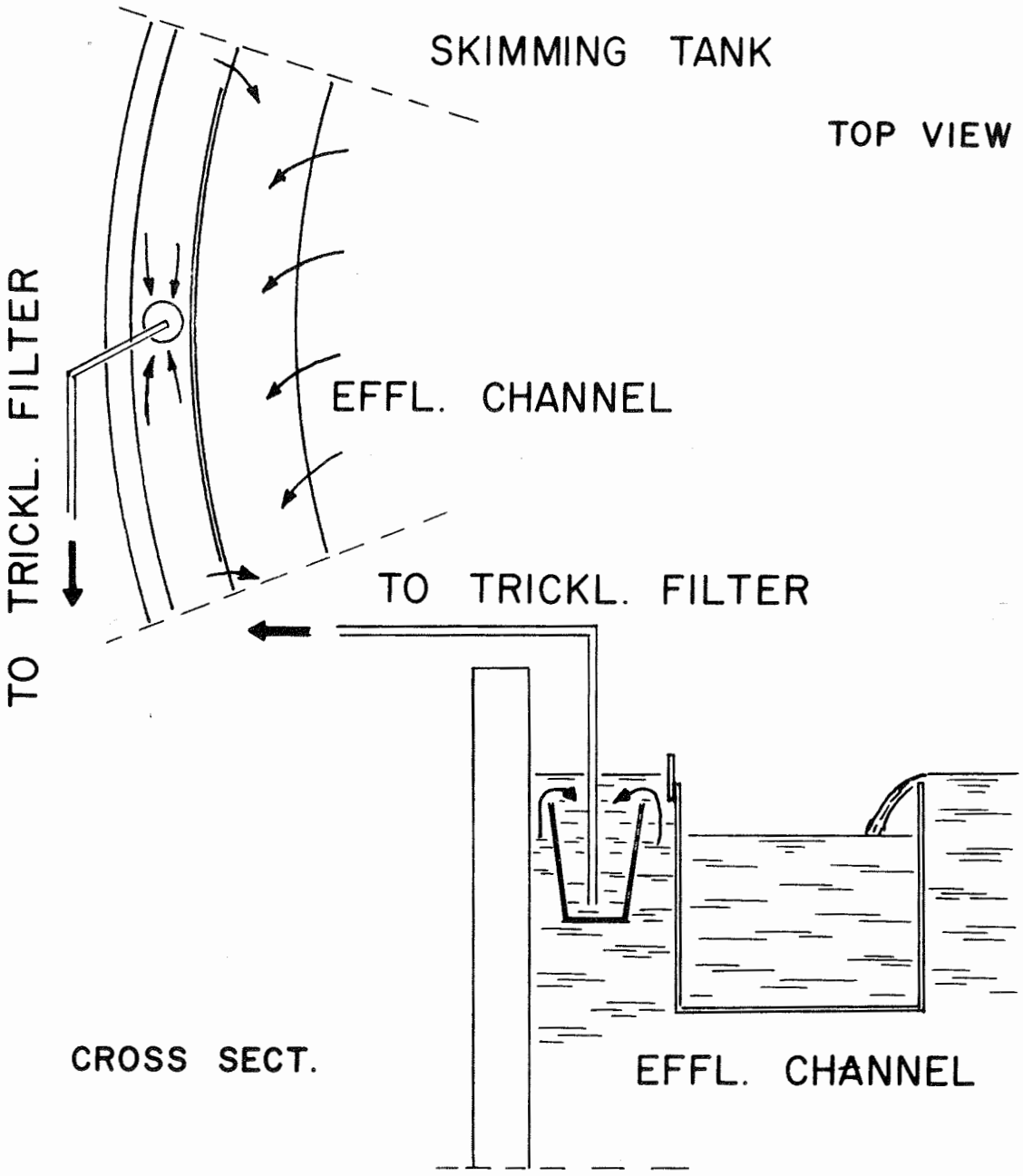

FIG. 7

SUCTION IMPROVEMENT FOR THE TRICKLING FILTER

3. Waste Stabilization Ponds

Pilot-scale waste stabilization pond systems were constructed at the Govalle Waste Treatment Plant. Each of the systems have the same total volume and the three systems are illustrated in Figure 8. System 1 consisted of three separate ponds, including an anaerobic pond, a facultative pond, and a maturation pond. A facultative pond in which a four foot deep trench was located in the bottom to provide the same volume of anaerobic zone as in System 1 and a maturation pond made up System 2. System 3 consisted of a facultative pond which is about six inches deeper than the other facultative ponds and a maturation pond. The volume and other information pertaining to the three pond systems is tabulated in Table 2.

Wastewater was pumped to the ponds from the grit chamber. A separate pump and feed line with flow metering devices were installed for each of the three pond systems. The average rate of flow into each of the ponds was the same. Operation of the pumps was controlled by automatic timers. Laboratory-scale aquaria were operated in parallel with the pilot-scale pond.

B. Williamson Creek Treatment

The wastewater treatment plant at Williamson Creek was constructed in 1963 to service the most Southwestern portion of the City of Austin, Texas. The plant consists of a lift station, screens, a comminutor, an extended aeration plant, two aerated lagoons operating in parallel and waste

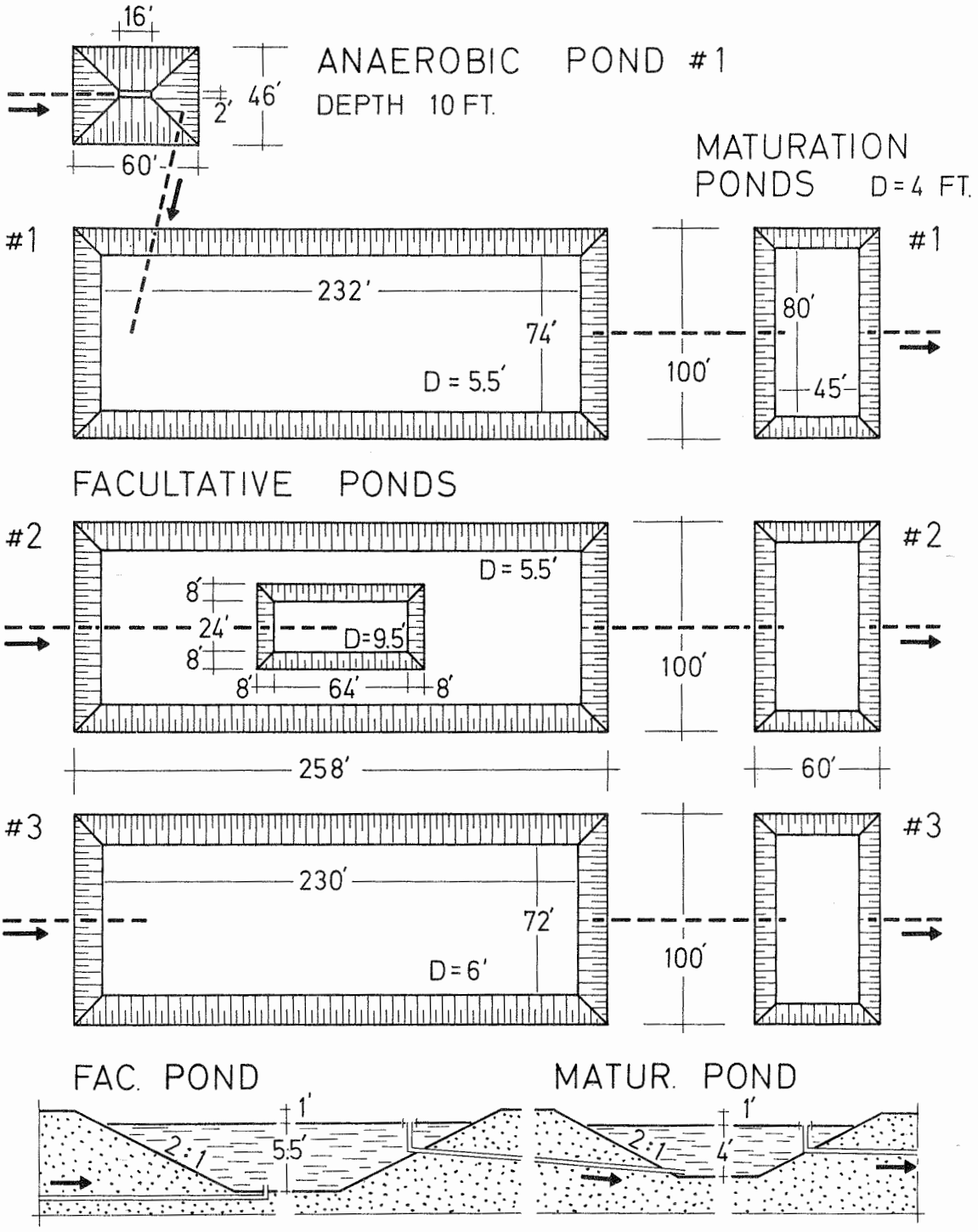

FIG. 8

PILOT WASTE STABILIZATION PONDS

TABLE 2

PHYSICAL CHARACTERISTICS OF THE
PILOT SCALE WASTE STABILIZATION PONDS

	Depth	Surface Area		Volume	
	ft.	sq ft	acre	cu ft	acre ft
I Anaerobic	10.0	2,780	0.064	8,970	0.207
I Facultative	5.5	25,800	0.592	117,530	2.690
I Maturation	4.0	6,000	0.138	18,130	0.415
II Facultative (with trench)	5.5	25,800	0.592	126,300	2.900
II Maturation	4.0	6,000	0.138	18,130	0.415
III Facultative	6.0	25,800	0.592	126,300	2.900
III Maturation	4.0	6,000	0.138	18,130	0.415

stabilization ponds. A schematic diagram of the plant site is presented in Figure 9. This plant was designed for flexibility of operation and to provide an experimental facility for upgrading biological waste treatment processes. The incoming wastewater enters the wet well of the lift station and is pumped 55 feet into a comminutor. The comminuted wastewater is then distributed among the aeration tanks. The aeration tanks are square and have a side wall slope of one vertical to three horizontal. The ponds are lined with concrete and a surface aerator is fixed to a platform which is located in the center of the basin. Operating conditions can be varied by changing the elevation of the effluent weir and by modifying the immersion of the mechanical aerators. The physical characteristics of the aerated lagoons and the extended aeration system are summarized in Table 3.

The pumps in the lift station operated intermitently when the wet well fills and surges on the aeration system occur. The influent wastewater flow to the extended aeration system (A Plant) was metered separately and an additional meter was available for recording the total flow to the two aerated lagoons (B Plant and C Plant). Therefore, regulation of flow into the aerated lagoons was accomplished by pumping wastewater to the aerated lagoon of B Plant from the discharge line into the aerated lagoons of C Plant. Operation of the smaller pump was synchronized with the operation of the larger pumps in the lift station.

The mechanical aerator in the aerated lagoon of B Plant was equipped with a variable speed transmission; therefore, the power level in

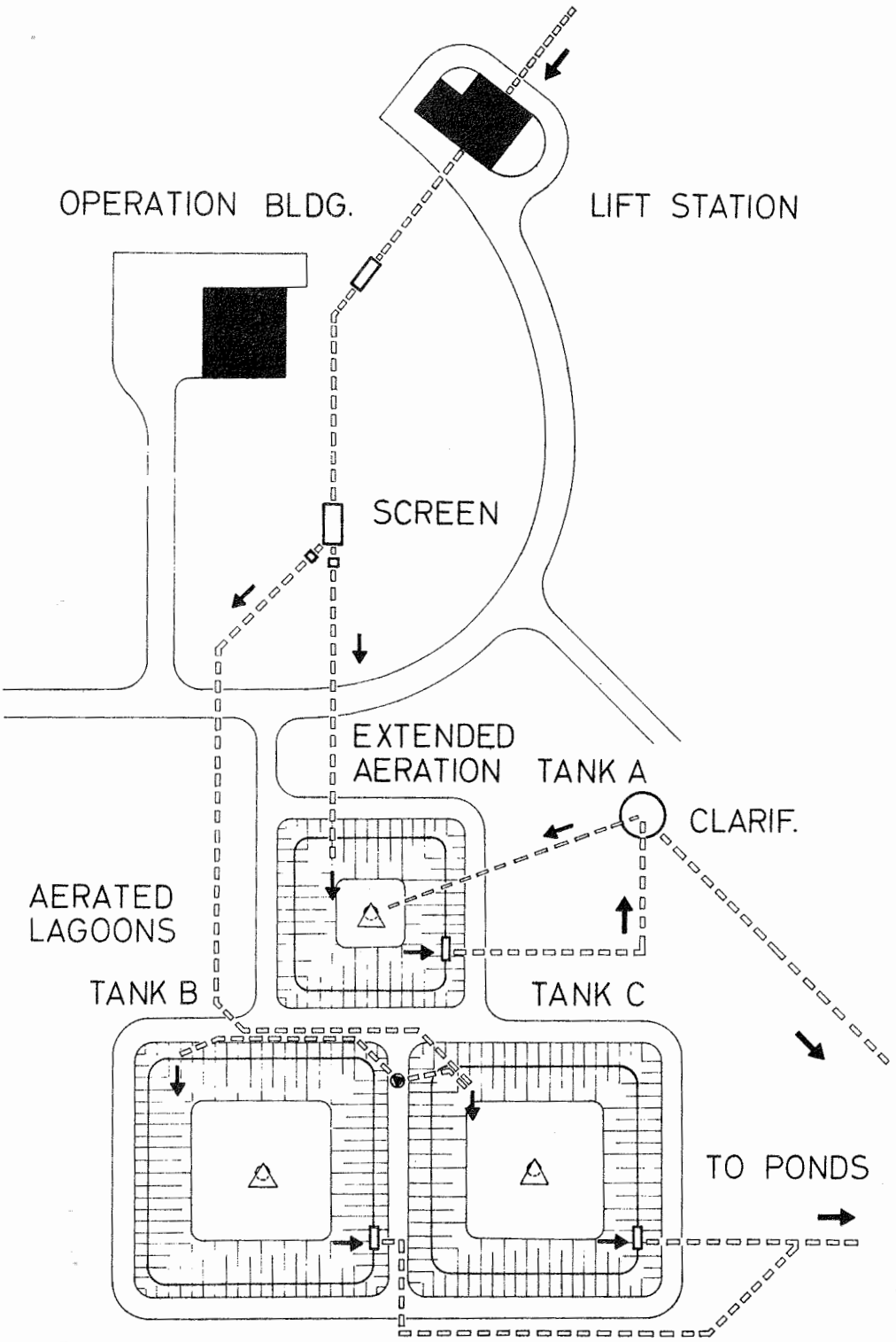

FIG.9

SEWAGE TREATMENT PLANT
"WILLIAMSON CREEK" AUSTIN, TEXAS

TABLE 3

WILLIAMSON CREEK SEWAGE TREATMENT, PLANT, AUSTIN, TEXAS
SUMMARY OF PLANT CHARACTERISTICS

EXTENDED AERATION (PLANT A)

Aeration Tank Volume	0.235	mg
Aeration Tank Depth	7.4	ft
Aerator Max Power	10.0	HP
Clarifier Surface	490	sq ft

AERATED LAGOON	PLANT B	PLANT C
Aeration Tank Volume	0.69 mg	0.56 mg
Aeration Tank Depth	8.0 ft	6.5 ft
Aerator Max Power	20 HP	20 HP

this particular lagoon could be varied from about ten to 50 horsepower per million gallons by changing the speed of operation. The power level could also be varied by changing the immersion depth of the aerator but this method was considered too complicated; therefore only one immersion depth was used.

LABORATORY PROCEDURES

A. Sample Handling and Chemical Analyses

Samples collected at the Govalle Waste Treatment Plant and at the Williamson Creek Plant were analyzed in one of three laboratory facilities. Some laboratory space was provided for the project at the Govalle Wastewater Treatment Plant. A mobile laboratory and equipment made available by the Environmental Health Engineering Laboratories and the Center for Research in Water Resources at The University of Texas at Austin was also made available during the duration of the project. The main analytical laboratory was located at the Balcones Research Center of The University of Texas at Austin.

Samples collected at the Govalle Wastewater Treatment Plant were analyzed in the field laboratory for suspended and volatile suspended solids and pH. A portion of the sample was also filtered or settled at the field laboratory, depending on the analyses to follow. The samples were then refrigerated prior to being transferred to the laboratory at the Balcones Research Center for more complete analytical work. The mobile laboratory also permitted some analyses for total organic carbon (TOC) and oxygen uptake rate studies. All samples for analyses were kept under refrigeration until the necessary data were collected. The data reported for each of the samples is summarized in Table 4. Specific information required for individual experimental programs are discussed in detail in the reports which are included in the Bibliography.

TABLE 4

ANALYSES OF 24 HOUR COMPOSITE SAMPLES

Analysis	Sample	Frequency	Procedure
	Influent and Effluent		
BOD	(filtered and unfiltered)	every 2nd day	Standard Method with modifications
TOC	(filtered and unfiltered)	daily	Beckman Model 915 Carbon Analyzer
COD	(filtered and unfiltered)	every 3rd day	Standard Methods (later filtered sample by autoanalyzer)
SS	(total and volatile)	daily	Standard Method
TKN	(filtered)	every 3rd day	Auto analyzer
NH_3	(filtered)	every 3rd day	Auto analyzer
$NO_2 + NO_3$	(filtered)	every 3rd day	Auto analyzer
PO_4	(filtered)	every 3rd day	Auto analyzer
	Mixed Liquor		
SS	(total and volatile)	daily	Standard Method
	Return Sludge		
SS	(total and volatile)	daily	Standard Method

A schedule of analyses was required in order to accommodate the laboratory personnel in a five-day work week. Samples for BOD analyses were set up on Wednesday, Thursday, and Friday. The Auto Analyzer was operated daily and a different analysis was programmed for each particular day. Total organic carbon was analyzed daily.

All of the analyses in a particular sample were generally completed within a ten-day period after the samples were collected. Only under extreme cases such as breakdown of the automatic equipment or illness of the laboratory technicians were samples stored for more than a ten-day period.

A method of storing the samples was developed in order to minimize the extent of biodegradation of materials in the samples which were in storage. The procedure involved transferring the sample into a household blender, introducing nitrogen and mixing for 30 seconds. This procedure was followed prior to storage of the samples. During this short time period, the dissolved oxygen concentration was reduced to zero. Preliminary results indicate that the combination of a zero oxygen content and refrigeration of a temperature of near $4^{o}C$ is effective for preservation of samples about one a week.

The results of the BOD, COD, TOC, ammonia, total Kjeldahl nitrogen, orthophosphate, nitrate and nitrite are affected by this method of blending and mixing nitrogen. Precautions should be taken to insure that the pH of the sample is maintained below pH 7.5 for those samples in which ammonia and Kjeldahl are to be determined. There is a tendency for the

ammonia to come out of solution at pH = 7.0 or higher. Sulfuric acid in a dilute solution can be used to reduce pH without noticeable effect on the analytical results. This method of sample preservation was started in May of 1970. The blending was done when the samples were received at the laboratory. Prior to this time, the samples were blended in the field.

B. Analytical Procedures

The analytical procedures used during this study were those outlined in Standard Methods for Examination for Water and Wastewater 12th edition, 1965, where these procedures were considered satisfactory. In many cases, the procedures had to be modified in order to compensate for discrepancies which were found in the analyses of the samples and standards. Those modifications of analytical procedures other than those found in Standard Methods are presented in this section.

1. BOD Analyses

During the early stages of this study, BOD dilution water was prepared by aerating the water which contained the necessary nutrients and seed. The aerated water resulted in saturation values of dissolved oxygen in the order of seven to eight milligrams per liter. During the later part of 1969 the procedure was modified to use compressed oxygen to oxygenate the dilution water. The oxygenated water provided a wider range of dissolved oxygen available provided for the biochemical processes in the BOD bottle and also eliminated the need in many cases for diluting the sample prior to analyses.

Twenty liters of distilled water which contained the necessary nutrients outlined in the Standard Methods and 20 milliliters of a seed obtained from the supernatant of settled activated sludge was oxygenated with oxygen from a compressed gas cylinder for one and one half to two minutes. The short period of oxygenation resulted in dissolved oxygen concentrations in the dilution water of 15 and 20 mg/l. The bubbling oxygen also mixed the nutrient and seed in the dilution water. Fine gas bubbles resulting from the oxygenation were cleared from the water by letting the dilution water stand from one to two minutes after the oxygenation stopped. The highly oxygenated dilution water was then added to the BOD bottles which already contained the samples to be analyzed.

The loss of oxygen from the samples bottle was evaluated by observing the oxygen concentration in dilution water. Samples of oxygenated water were analyzed periodically during the first hour after oxygenation and after five days of incubation. The loss of oxygen was negligible. Only one dilution of each sample was made because of the high initial dissolved oxygen concentration in the dilution water. A blank was run for each dilution. A modified Winkler procedure for determining the high oxygen concentrations was also used as outlined in Standard Methods.

2. COD Determination

The procedure developed by Standard Methods was used for some of the COD analyses. An automated procedure for the COD analyses had been developed. A flow diagram for the automated procedure is presented in Figure 10. This procedure is effective over a range of COD of zero to 100 mg/l of equivalent COD.

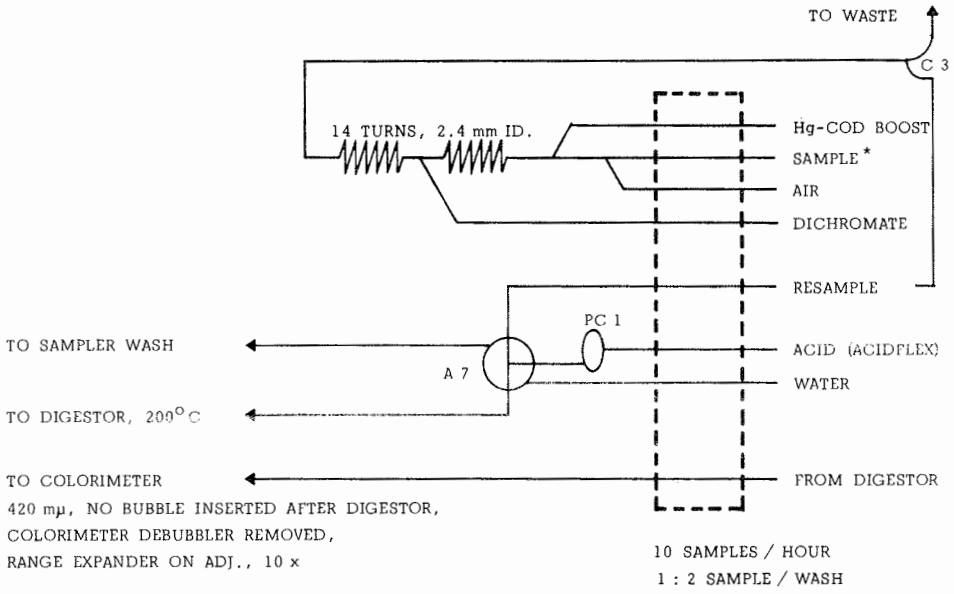

FIG.10
COD BY AUTO-ANALYZER (FLOW DIAG.)

sample that is to be analyzed by the automated COD method. Determine the COD of the sample in triplicate using the manual method. Prepare a standard curve using serial dilution of the original sample. Compare the manual COD value with the phthalate standard and determine the proper correction type. The corrected phthalate standards should be retained for normal use since they may be more stable than the original sample.

b. Chemical Basis

The COD test is based on the reduction of the yellow hexavalent potassium dichromate by oxidizable organic material in the sample. A green colored solution is typical of trivalent chromate. The hot sulfuric acid and the silver catalyst break down the organic matter. Silver is removed from the reaction by halogens; therefore, mercury in the form of mercuric sulfate is added to the sample to complete the halogens and prevent the removal of the silver. A further discussion of the chemical reactions dissolved in the COD procedure can be found in Standard Methods.

c. Procedure

The following procedure is established for the Technicon Auto Analyzer. Any other automated equipment would have to have the particular settings and procedure modified to fit the particular equipment. The digester should be set at 200°C on both heaters since it is not recommended to raise the temperature above the 200°C level. The 420 millimicron

filters should be installed in the colorimeter and the 50 ml flow cell used. A four ml flow cell would reduce the noise and increase the reproducibility of the analyses but a four ml cell was not available at the time. A range expander should be inserted between the colorimeter and the recorder. The settings of the range expander should be as follows: Range:10 x Pos: adj.

After the digester and colorimeter are adjusted, water is run through the system to set the 100 percent transmisibility and the zero percent. Select an apperture which gives the 100 percent transmittance on the recorder but about ten percent transmittance on the colorimeter dial. The reagents without any sample are then passed through the system. When the base line has been standardized the samples can be run. It is important that the base line be standardized and that the strongest standard be passed through the system first. Adjustments of the colorimeter setting and apperture can be made using the strongest sample so that standard reads about 95 percent transmittance. This procedure will insure that the optimum range and sensitivity are maintained throughout the analyses.

d. Discussion

The characteristics of a colorimeter used in the automated system are such that the difference between a concentration of zero and 600 mg/l equivalent COD is very small using the 420 millimicron filter. However, the sensitivity is greatly increased at concentrations between 600 and 800 mg/l. This change in sensitivity is illustrated in Figure 11.

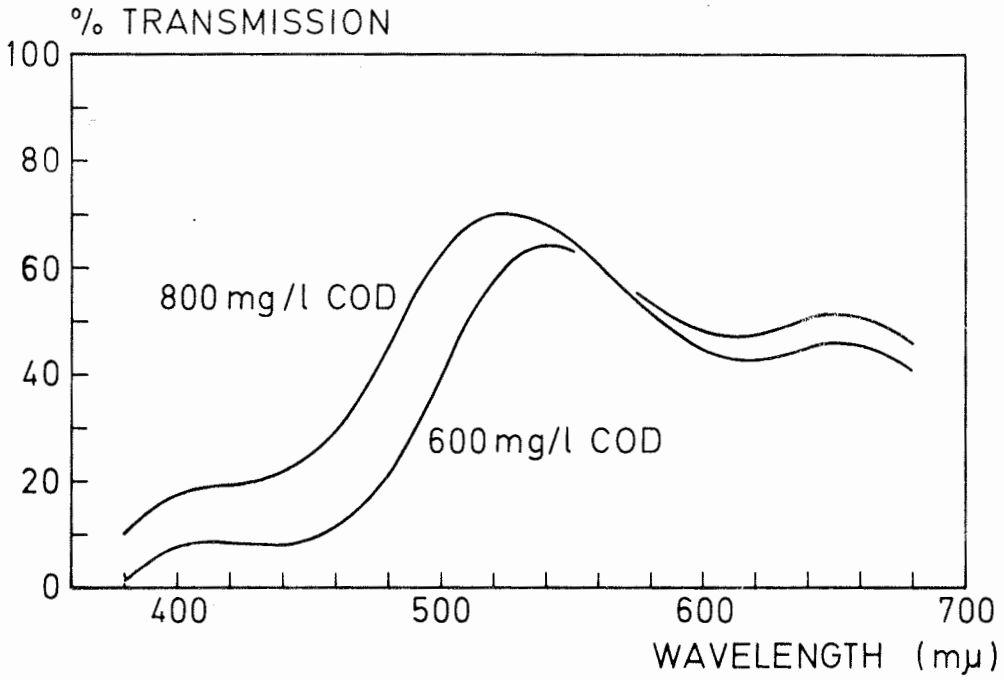

FIG.11

SPECTROGRAM FOR COD

The COD boost solution used in the standards sets the base line at a concentration of 600 to 800 mg/l. Once the base line is established for the particular boost solution, the concentration of COD in the samples are read in addition to this base line. The boost does not have to be subtracted from the read out since the standards also contain this solution.

The phthalate standards are quite easily digested compared to some of the complex samples. Therefore, it is recommended that the manual method be used to determine the COD of unknown solutions which may contain complex materials. The manual values can then be correlated to the phthalate standards as mentioned above.

3. Automated Analysis of Ammonia

The flow diagram for the Auto Analyzer module for analysis of ammonia is presented in Figure 12. This procedure is effective for ammonia concentrations up to 0.30 mg/l as nitrogen.

a. Reagents

The reagents should be prepared using distilled dionized ammonia free water.

(1) Sodium hypochlorite

The commerically available household bleaches, such as "clorox" and "purex" which provide approximately five percent solution of sodium hypochlorite can be used.

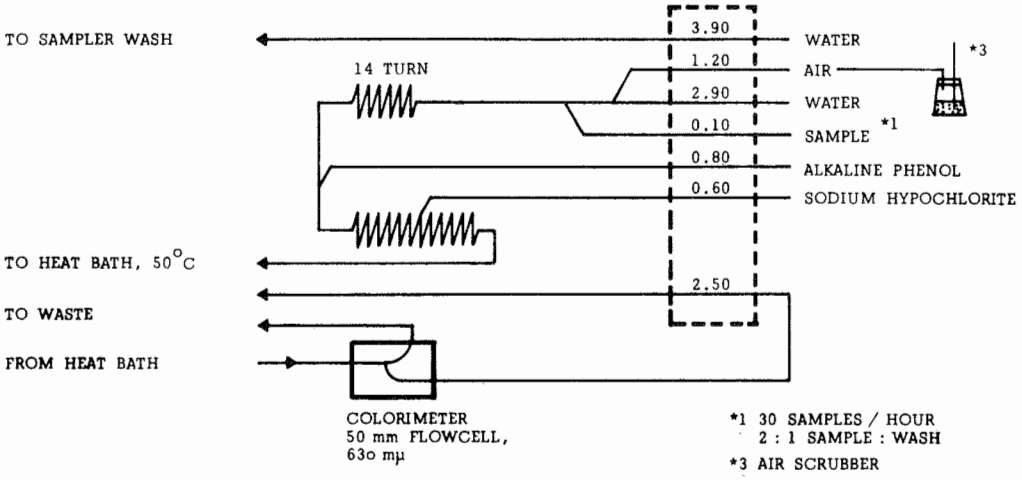

FIG.12
NH_3 BY AUTO-ANALYZER (FLOW DIAGR.)

(2) Stock solution of sodium hydroxide, 12 N

Dissolve 1000 grams of sodium hydroxide (NaOH) in one liter of distilled water in a Pyrex container. Cool and dilute the solution to two liters. A large quantity of this prepared reagent should be kept on hand.

(3) Sodium hydroxide, 10 N

Dilute 1670 ml of stock sodium hydroxide solution to two liters.

(4) Alkaline phenol

Slowly add 552 ml of 88 percent liquified phenol which is commercially prepared to a container holding one liter of the 10 N sodium hydroxide solution which is in a water bath. Stir the sodium hydroxide solution while adding the phenol. The solution should be cooled and diluted to two liters.

(5) Standards

Prepare stock solution which contains 4.7169 grams per liter of ammonia sulfate $(NH_4)_2SO_4$. The concentration of stock solution is 1000 mg/l of ammonia as nitrogen. Prepare serial dilutions in the proper means for standardization.

b. Chemistry

The quantitative determination of the ammonia on the colorimeter is based on a blue colored complex formed when a solution of ammonium

salt is reacted with sodium phenoxide followed by the addition of sodium hypochlorite (Berthelot Reaction).

c. Procedure

The reagents should be run through the automated system to establish the 100 percent transmittance. The blank side of the colorimeter should be set at zero percent transmittance. All samples should be at a pH of 7.5 or lower in order to insure that all the ammonia is present in the solution as the ammonia ion (NH_4^+). If the pH of the samples is above pH 7.5, some of the ammonia may be lost as a gas. All samples for ammonia analysis should have the pH adjusted by adding a dilute solution sulfuric acid.

4. Automated Analysis for the Total Kjeldahl Nitrogen (TKN)

The flow diagram for the automated system for analysis for the total kjeldahl nitrogen is presented in Figure 13. The procedure is applicable up to a concentration of 0.30 mg/l of TKN as nitrogen.

a. Reagents

All the reagents used in this procedure must be prepared using distilled dionized ammonia free water.

(1) Acid digestant

Slowly add 40 ml of 68 to 70 percent perchloric acid solution to 160 ml of distilled water. Dissolve six

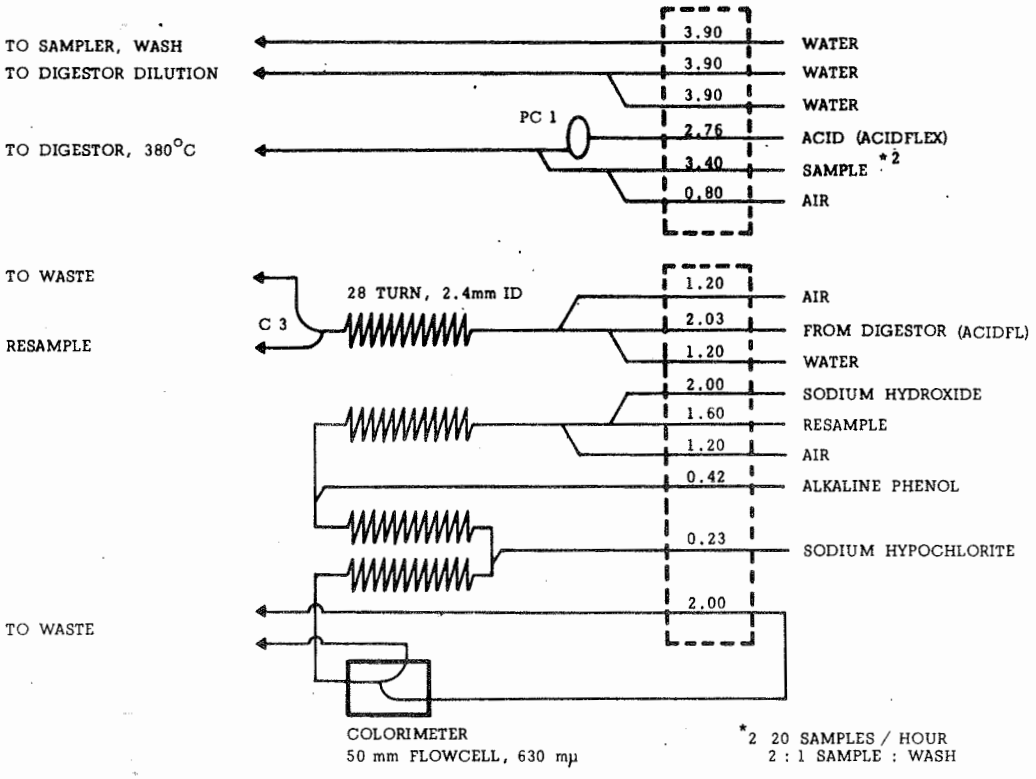

FIG.13
TKN BY AUTO-ANALYZER (FLOW DIAG.)

grams of selenium dioxide into this solution. Place this solution in a container in a water bath and while stirring slowly add 1800 ml of concentrated sulfuric acid. Cool this solution and dilute to two liters.

CAUTION: This reagent is extremely corrosive.

(2) Sodium hypochlorite

Commercially prepared household bleaches such as "Clorox" and "Purex" may be used in this reagent.

(3) Stock sodium hydroxide, 12 N

Dissolve 1000 grams of sodium hydroxide (NaOH) in one liter of distilled water. Cool and dilute to two liters. A very large quantity of this reagent should be kept on hand.

(4) Sodium hydroxide, 10 N

Dilute 1670 ml of the stock sodium hydroxide solution to two liters.

(5) Sodium hydroxide, 9 N

Dilute 1500 ml of stock sodium hydroxide solution to two liters.

(6) Alkaline phenol

Slowly add 552 ml of 88 percent liquid phenol while stirring one liter of ten normal sodium hydroxide solutions. The container with the sodium hydroxide should be placed in a water bath. Cool and dilute to two liters.

(7) Standard Stock Solution

Add 2.1439 grams of urea to one liter of water. The stock solution has a concentration of 1000 mg of nitrogen per liter. Prepare a serial dilution using this stock solution for the range of analysis.

b. Chemical Basis

The automated total Kheldahl nitrogen procedure is based on the manual method for total nitrogen found in Standard Methods. Organically bound nitrogen as well as other forms of nitrogen are converted to the ammonium ion by the digestion process. Organic carbon is oxidized at the high temperature by the sulfuric-perchloric acid oxidizing solution with the assistance of sodium hydroxide catalyst. Sulfuric acid is reduced to sulfur dioxide which is a strong reducing agent which in turn converts all nitrogen of a higher oxidiation state to a lowest trinegative state, ammonia (NH_3). However, at the low pH the ammonia is present as the ammonium ion. The quantitative determination of the ammonium ion on the colorimeter is based on a blue colored complex which is formed when the solution of ammonium salt reacts with the sodium phenoxide (alkaline phenol followed by the addition of sodium hypochlorite). All samples to be analyzed must have a pH equal to pH 7 or less in order to insure that all ammonia present is in the solution is in the ammonium form. Samples which have a pH greater than seven may lose some of the ammonia as gas; therefore, the pH of all samples should be adjusted by using a dilute sulfuric acid and stored at a pH of less than seven.

c. Procedure

The digester should have both heaters turned on and set at 380°C. The 630 millimicron filter and the 50 mm flow cell should be installed in the colorimeter. The automatic sampler should be set at 20 samples per hour and a ratio of sample to washwater should be 2:1. The 100 percent transmittance is set by passing only the reagents through the system. The blank side of the colorimeter should be set at zero transmittance. The optimum temperature for the digester can be established using a simple procedure. Place the sampling probe in one of the standards which has a high concentration. Continuously aspirate the sample; vary the temperature of the digester starting at 320°C and observe the recorder. The temperature to be used is that temperature at which the highest peak or most definition is observed.

5. Automated procedure for nitrite (NO_2) and nitrate (NO_3)

The automated procedure for the nitrite and nitrate determination is presented in Figure 14. This procedure is applicable for a range of zero to 2.00 mg/l as nitrogen.

a. Reagents

All reagents and dilutions must be prepared using distilled dionized ammonia free and nitrogen free water.

(1) Color developer

Add 200 ml of concentrated phosphoric acid to 1.5 liters of distilled water and mix. To this solution, add 80 grams

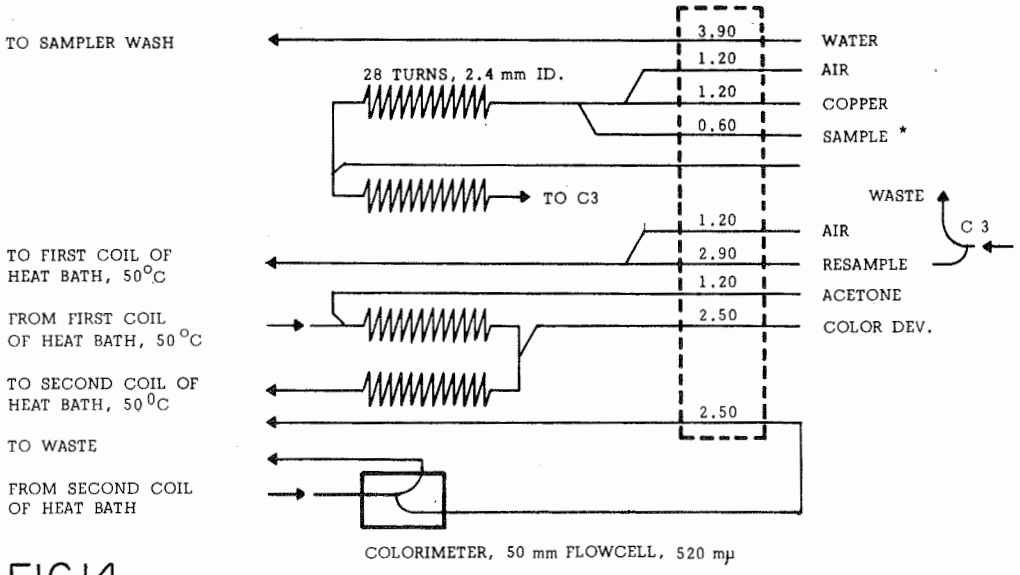

FIG.14
NO_2+NO_3 BY AUTO ANALYZER (FLOW DIAG.)

of sulfanilamide and heat if necessary to dissolve. Add 8.0 grams of normal (1-naphthyl)-ethylenediamine dihydrochloride to this solution and dissolve. Dilute the solution to two liters and store in the refrigerator. This reagent is stable for about one month.

(2) Copper stock solution

Dissolve 2.5 grams of copper sulfate ($CuSO_4 \cdot 5H_2O$) in one liter of distilled water. Dilute 20 ml of the stock solution to two liters for the working solution.

(3) Sodium hydroxide, 12 N

Dissolve 1000 grams of sodium hydroxide in 750 ml of water. Cool and dilute to two liters.

(4) Sodium hydroxide, 1.0 N

Dilute 165 ml of stock solution to two liters

(5) Sodium hydroxide, 3N

Dilute 55 ml of stock solution to two liters

(6) Acetone (10 percent)

Dissolve 200 ml of acetone in two liters of water. Keep this solution tightly stoppered to minimize loss of acetone by evaporation

(7) Hydrazine sulfate

Dissolve 27.46 grams of hydrazine sulfate in water and dilute to one liter. This reagent is stable for about six months.

(8) Hydrazine sulfate working solution

Dilute 50 ml of stock solution to two liters. This dilution of a stock solution is stable for one month

(9) Standard

Dissolve 0.7218 grams of anhydrous potassium nitrate and dilute to one liter. The concentration of this solution is equivalent to 0.1 mg of nitrogen per ml.

b. Procedure

The heating bath should be set at a temperature of 50°C. The 520 millimicron filter and the 50 mm flow cell should be installed in the colorimeter. The sampler should be set at 20 per hour at a ratio of two parts sample to one part wash. The reagents without any sample are passed through the module to set the 100 percent transmittance while the blank is used to set the zero transmittance.

6. Automated Phosphate Determination

The automated phosphate analytical procedure is presented in Figure 15. The range of applicability is 0.100 mg/l phosphate.

a. Reagents

All dilutions are made with distilled deionized water.

(1) Sulfuric Acid

Slowly add (200 ml per hour) one liter of concentrated

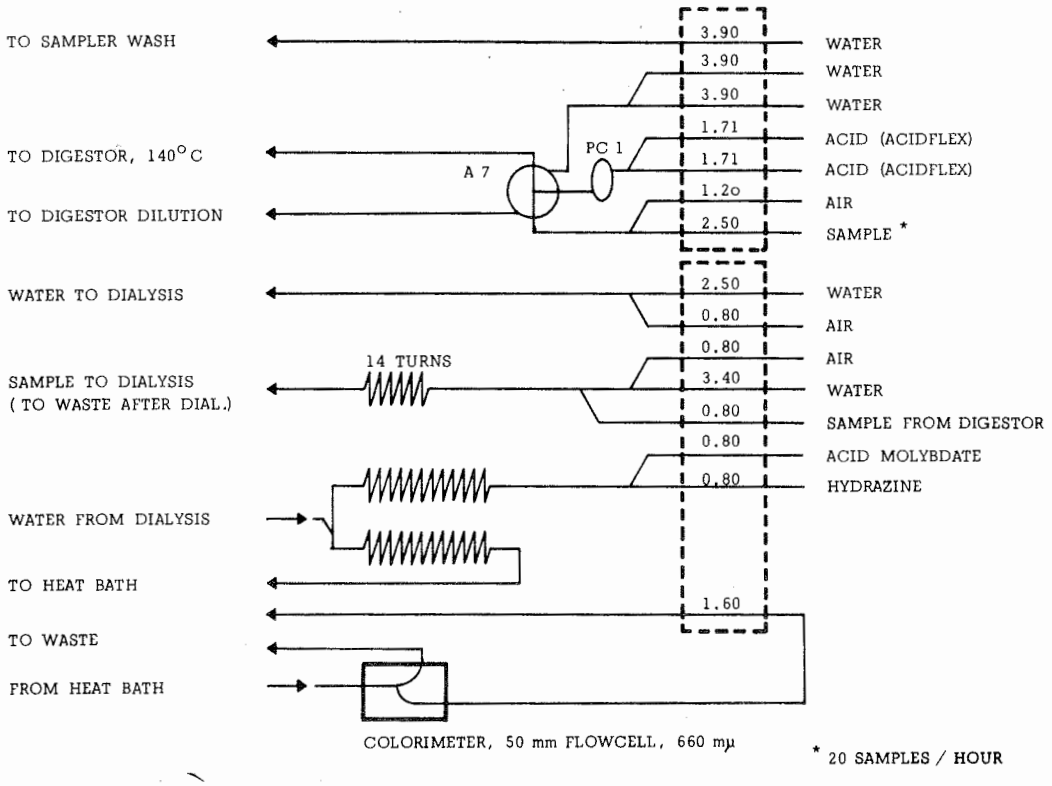

FIG. 15

PO_4 BY AUTO ANALYZER (FLOW DIAGRAM)

reagent grade sulfuric acid to one liter water while stirring. Cool between additions of acid.

(2) Acidic (3 N) ammonium molybdate (1 %)

Add 550 ml of concentrated sulfuric acid to one liter of water. After mixing and cooling, add 20.0 grams of ammonium molybdate. Dilute to two liters at a rate of 50 ml/hr.

(3) Hydrazine Sulfate Stock

Add 54.9 grams of hydrazine sulfate to 1.5 liters of water. Dilute to two liters. This reagent is stable for six months.

(4) Hydrazine Sulfate Working Solution

Add 30 ml of stock hydrazine sulfate solution to 1.5 liters of water. Dilute to two liters. This solution is stable for one month.

(5) Standard

Dissolve 1.443 grams of anhydrous potassium dibasic phosphate (KH_2PO_4) in water and dilute to one liter. The concentration of this stock solution is 1000 mg PO_4 per liter. Make serial dilution for proper range.

b. Chemical Basis

The automated procedure is based on the manual method. All forms of inorganic phosphate are converted to orthophosphate by digestion with the 10 N sulfuric acid solution at 140°C. The resulting orthophosphate reacts in the acid medium with the ammonium molybdate to form molybdophosphoric

acid. The molybdophosphoric acid is subsequently reduced by the hydrazine to form a blue molybdenum complex.

c. Procedure

The heating bath should be set at 80°C. The 660 millimicron filter and the 50 millimeter flow cell should be installed in the colorimeter. The sampler should be set at 20 per hour.

The reagents without any sample are passed through the module to set the 100 percent transmittance while a blank is used to set the zero transmittance.

WASTEWATER CHARACTERISTICS

The wastewater entering the wastewater treatment plants is primarily of domestic origin. A large portion of the working force in Austin, Texas is employed at various Federal and State offices and there are very few large water using industries. Some industrial wastewater from abattoirs, food processing plants, and chemical plants are also in the wastewater. These industrial wastes are insignificant in volume and concentration compared to the domestic sewage except for those times when chemical dumps are introduced into the system. The strength of the wastewater and the rate of flow are influenced to a great extent by infiltration of rain water into the collection system.

The variations in the flow rate and the concentrations of BOD and suspended solids during the duration of the project at the Govalle Wastewater Treatment Plant are presented in Figures 16 and 17. The data illustrated in Figure 11 is for the influent to the C Plant. There is some difference among the BOD and SS data reported for the influent to the A, B, and C Plants. These discrepancies reflect the error which may be introduced as a result of sampling procedures and analytical reproducibility. The sampling and analytical procedures will be discussed in more detail in Chapter V. The analytical data on which the evaluation of process performance is based is included in the references in the Bibliography.

Statistical analyses of the BOD and suspended solids data for the influent wastewater to the Govalle Plant are presented in Figures 18 and

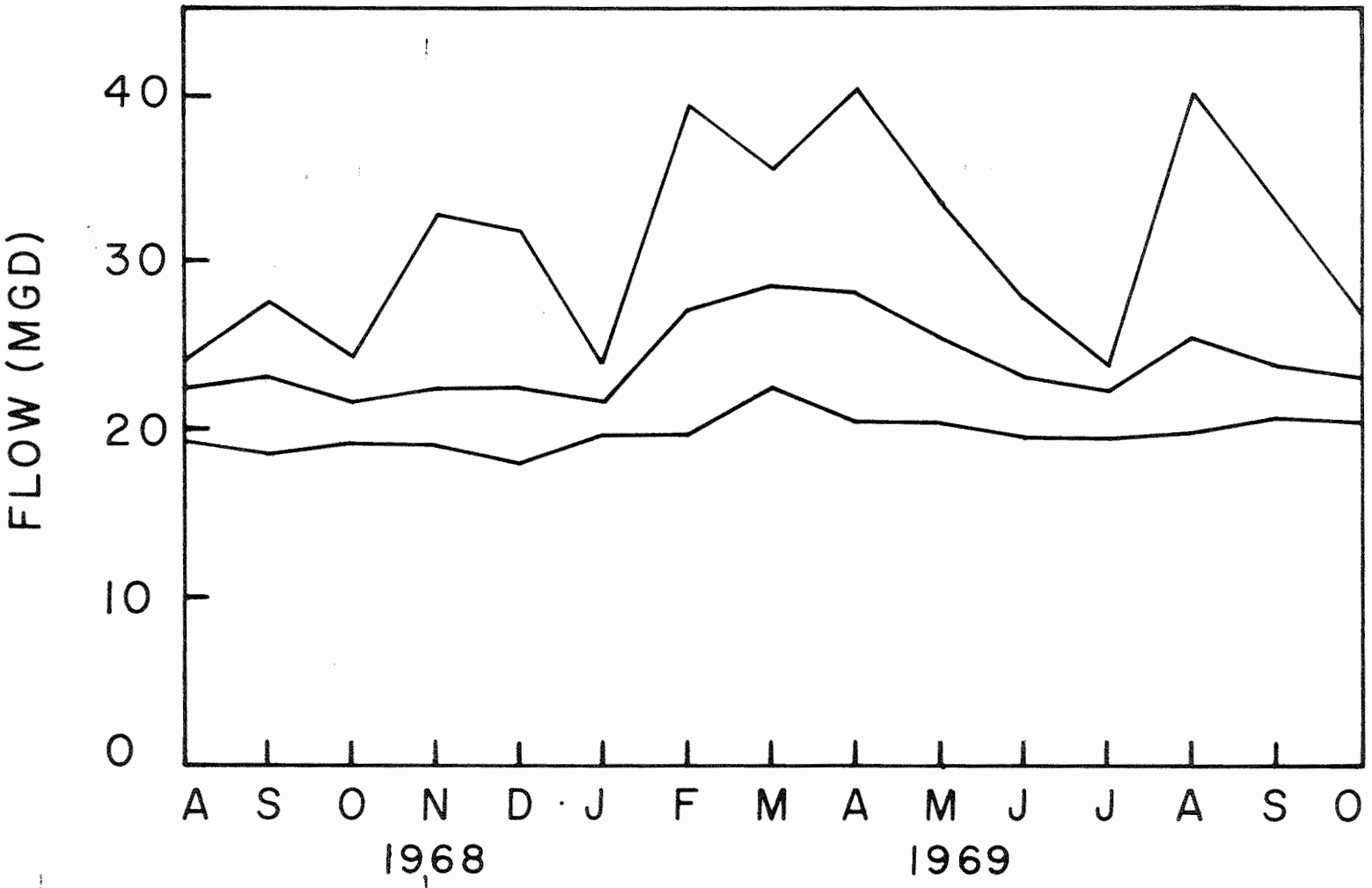

FIG.16
MONTHLY AVERAGE, MAXIUM, AND MINIMUM TOTAL FLOW
GOVALLE WASTEWATER TREATMENT PLANT

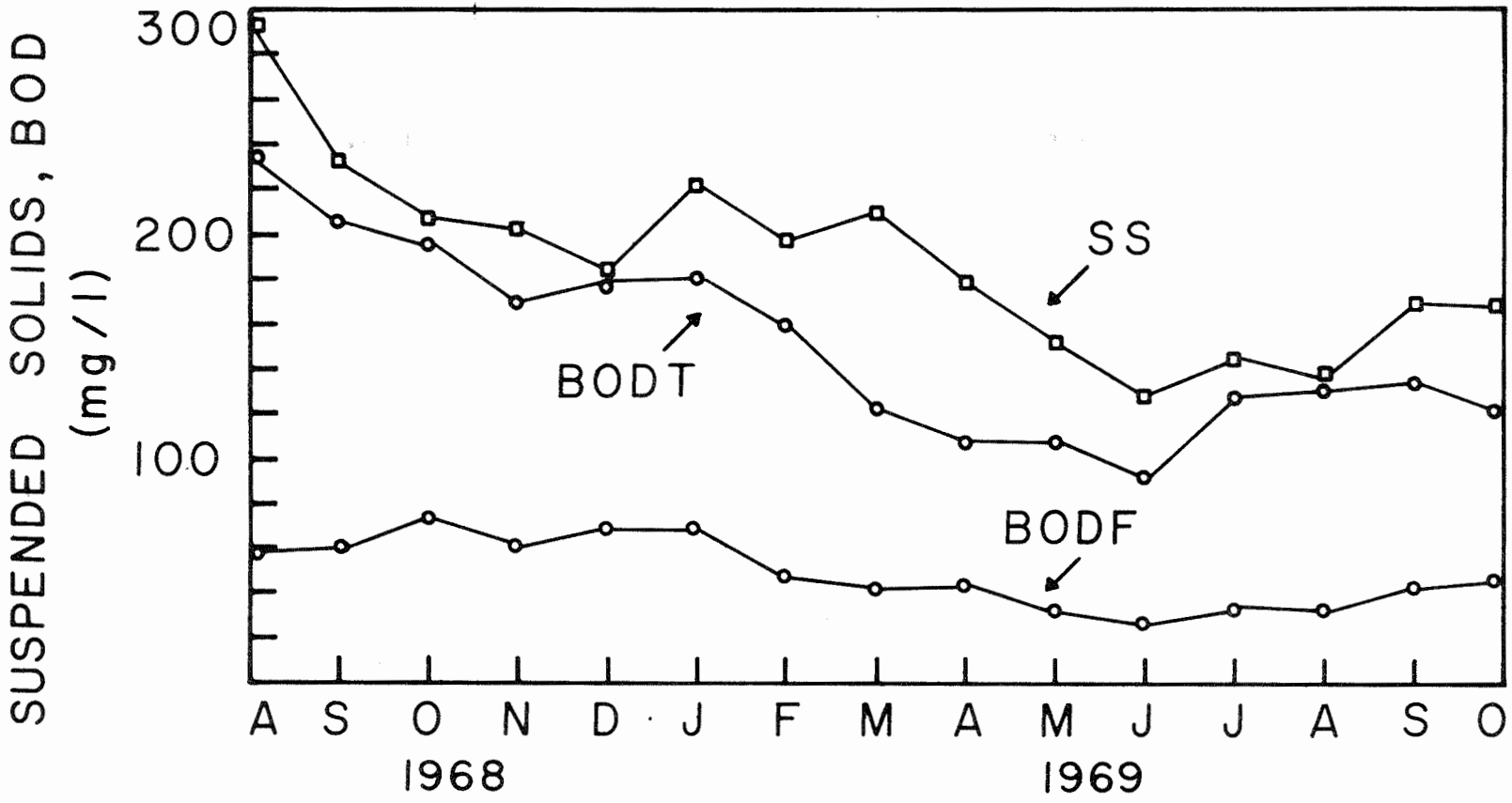

FIG.17
MONTHLY AVERAGE INFLUENT BOD AND SS TO C PLANT
GOVALLE WASTEWATER TREATMENT PLANT

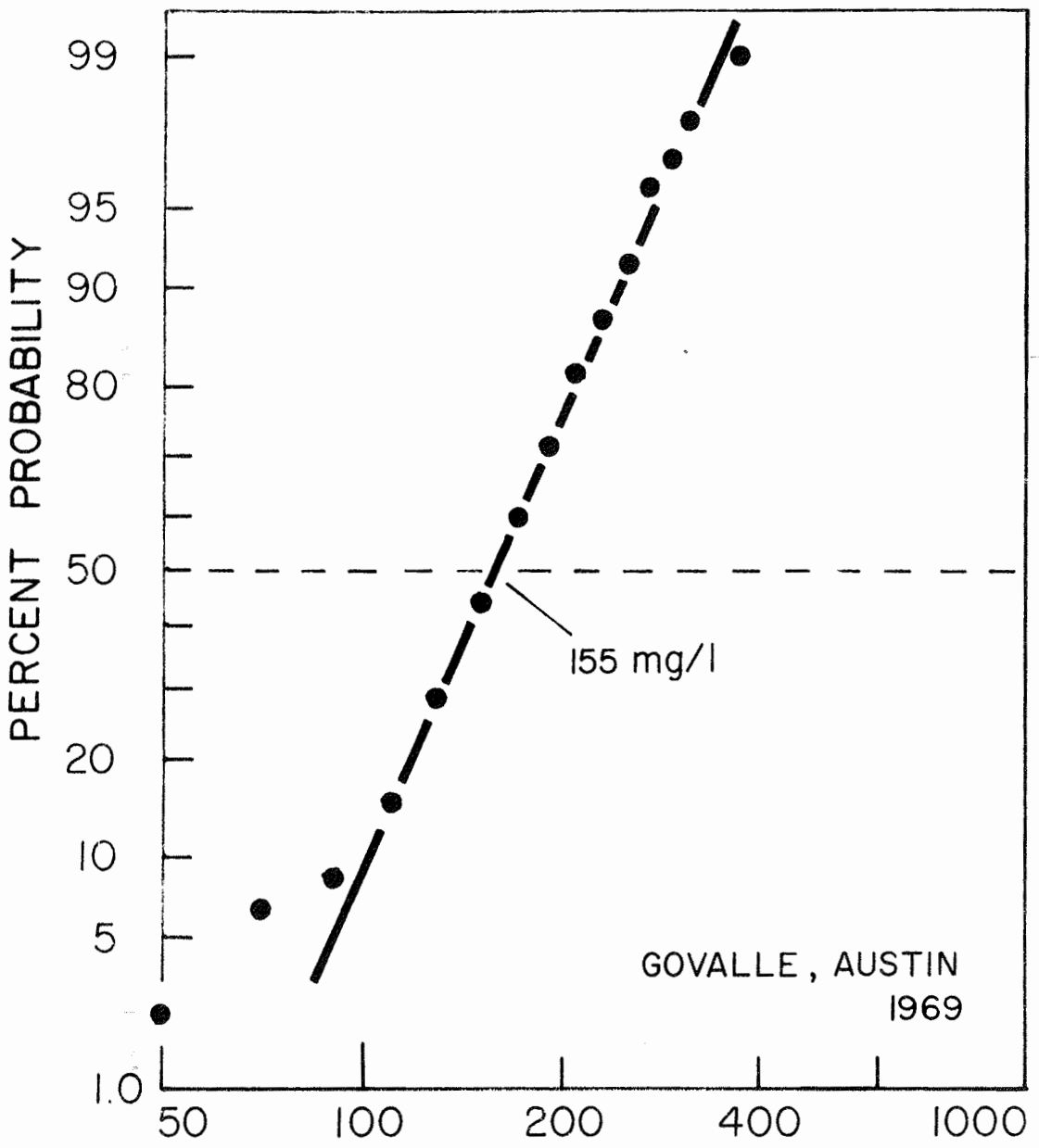

FIG.18

PROBABILITY OF BOD INFLUENT

19, respectively. The BOD data are described by a logarithmic or geometric distribution while the SS data are described by a normal or arithmetic distribution. There is some deviation from the linearity at a BOD concentration of less than 100 mg/l. The data indicate that during the two year period of record the average BOD and suspended solids concentrations of the influent wastewater at the Govalle Wastewater Treatment Plant was 155 mg/l for both parameters.

The interrelationship of flow, concentration of BOD in the wastewater and the total organic load to the treatment plant can be used to evaluate the extent of dilution of the wastewater which may occur at the higher flow rates which generally result from infiltration of wastewater into separate collection systems. The data presented in Figure 20 indicate the variations in wastewater flow, concentration of BOD and BOD load on the Govalle Wastewater Treatment Plant expressed in thousand pound per day. These data are the monthly average, the maximum, and the minimum values which occurred during the time period, July 1968 through June 1970. The variation in wastewater flow was much smaller compared to the variations in the concentration of BOD and the BOD loading. The wastewater flow ranged from slightly less than 20 MGD to over 45 MGD and represents about a two-fold variation. However, concentration of BOD varied from less than 50 mg/l to over 400 mg/l while the BOD load ranged from about 10,000 to 80,000 pounds of BOD per day. The ratio of maximum to minimum value for BOD concentration and load is about eight to one. Therefore, the range in the total BOD load to the treatment plant that is more dependent on the concentration of BOD than on the rate of wastewater flow.

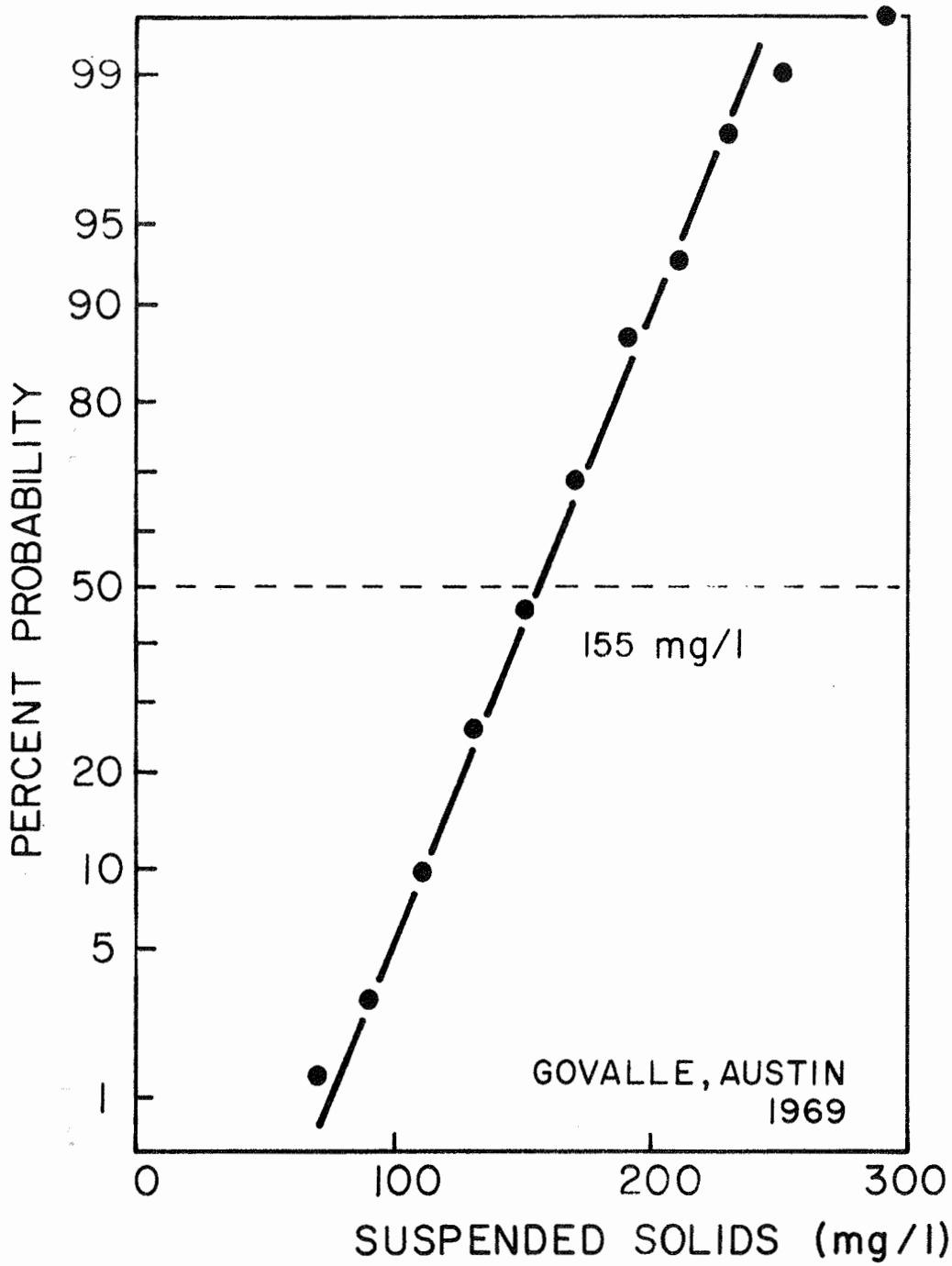

FIG.19
PROBABILITY OF SUSPENDED SOLIDS

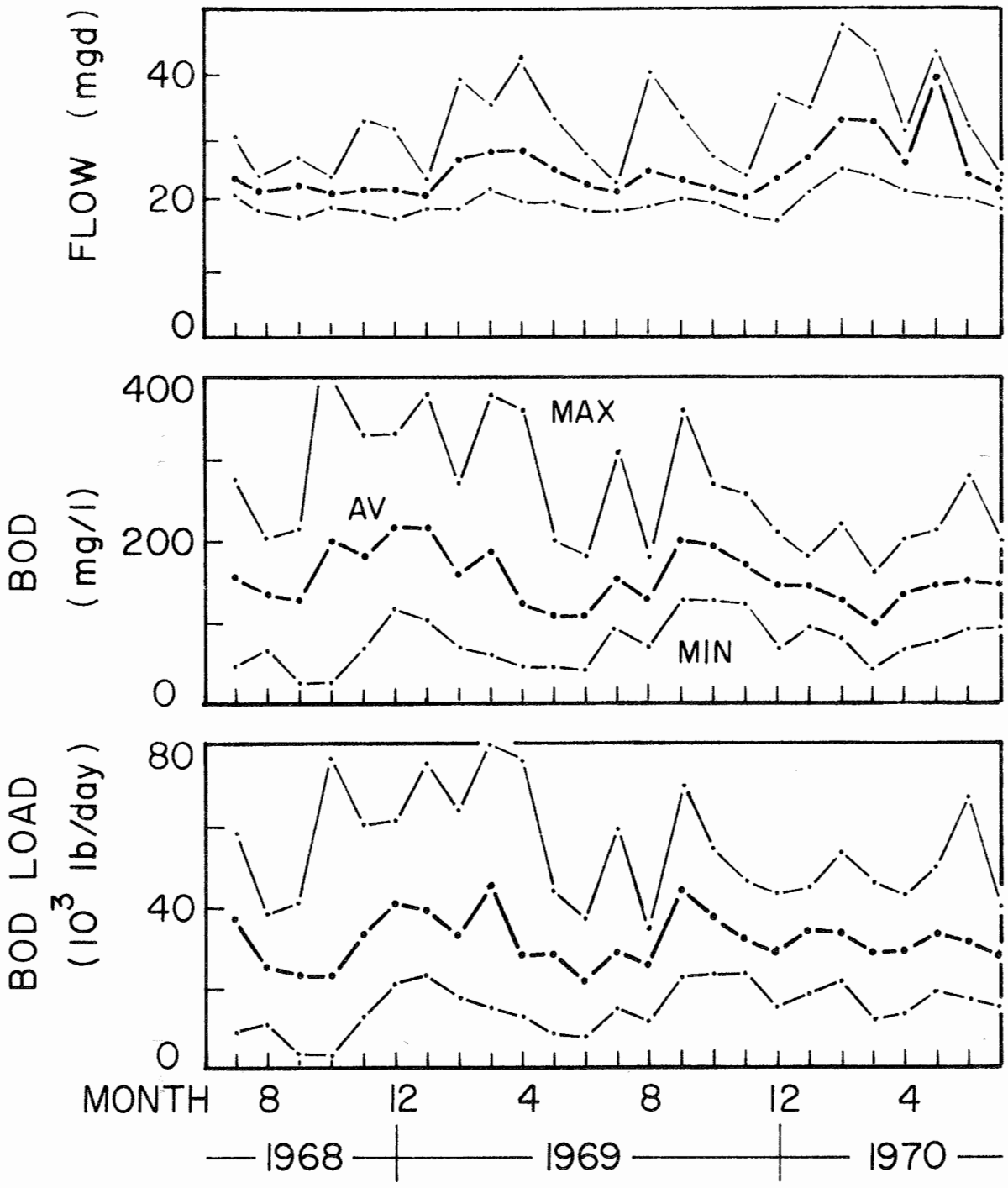

FIG.20

LOADING CHARACTERISTICS

MONTHLY AVG, MAX, MIN GOVALLE

Statistical analysis of the BOD load data indicates that these data are described by a logarithmetic distribution as illustrated in Figure 21. However, there is some deviation from linearity at a BOD load of less than 15,000 lb/day. The slope of the lines in Figures 18 and 21 are almost identical and this similarity indicates that the BOD load is affected much more by the variation in BOD concentration than by the wastewater flow variations.

The relationship of wastewater flow, BOD concentration, and BOD load during a 24 hour period is presented in Figure 22. The data in Figure 22 are based on the analyses of four-hour composite samples which were prepared by mixing hourly samples. These data were collected during July 16, 1970 and July 17, 1970 which represents a period of dry weather flow. The concentration of BOD is relatively constant between noon and midnight. However, the concentrationof BOD is highest for the sample collected during 12:00 noon to 4:30 p.m. period and decreases for each of the following four-hour composite samples. The sample collected between 8:30 p.m. and 12:30 a.m. is the lowest. The flow rate also is highest at noon and decreases steadily until 4:30 a.m. Combining the wastewater flow and the BOD concentration data results in a BOD load which is maximum during the four-hour period of 12:30 and 4:30 p.m., decreases steadily and reaches a minimum between 4:30 and 8:30 a. m. on July 17, 1970. The peak BOD load is 2500 pounds of BOD per hour while the average load was 1400 pounds per hour. The ratio of the peak to the average was 1.7. The average load during the day time period from 12:30 p.m. to 8:30 p.m. was approximately 2,200 pounds of BOD per hour. The ratio of this average load to the 24-hour average load is about 1.5. During this eight-hour period, about half of the total load arrives at the plant.

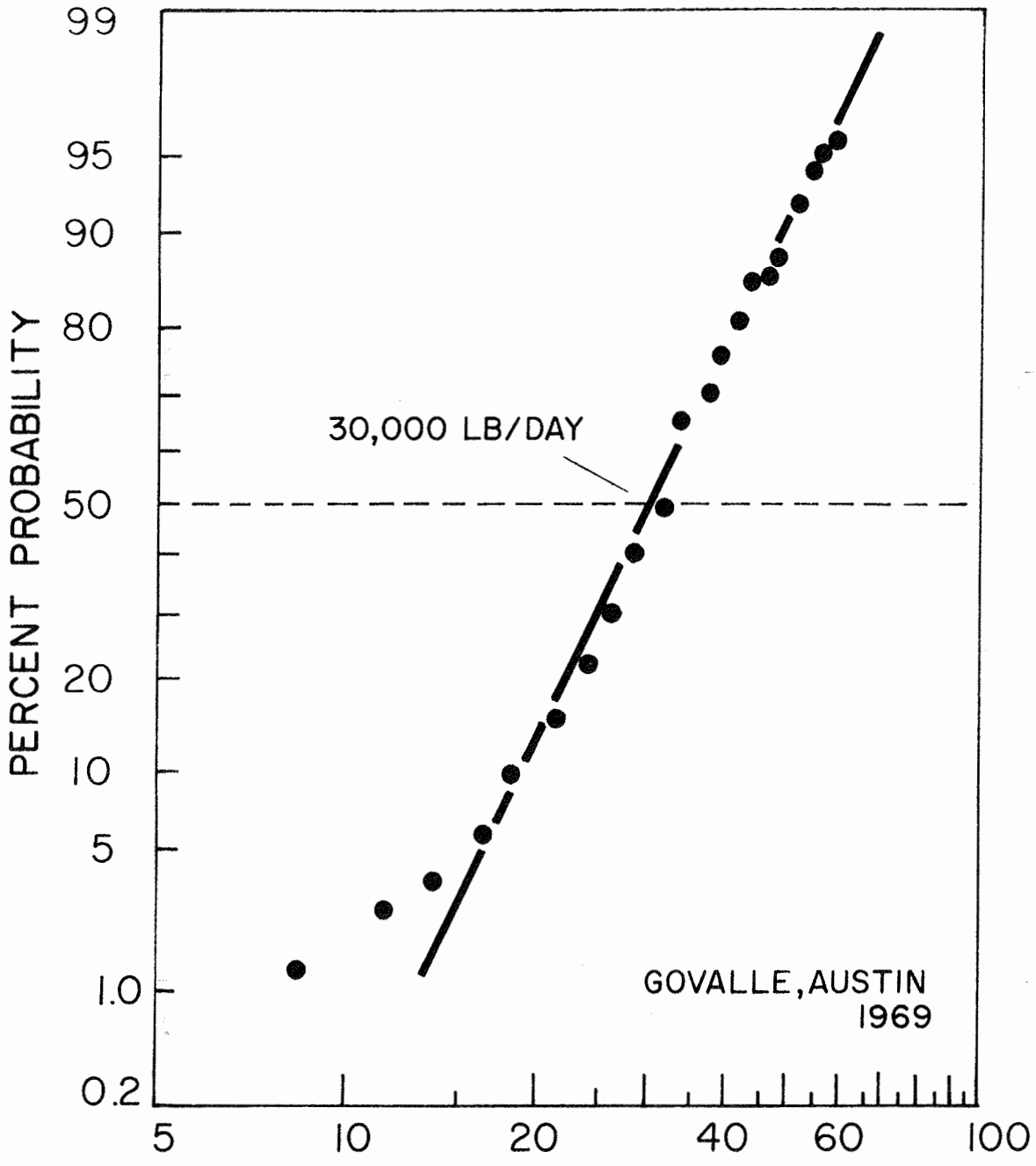

FIG. 21

PROBABILITY OF BOD LOAD

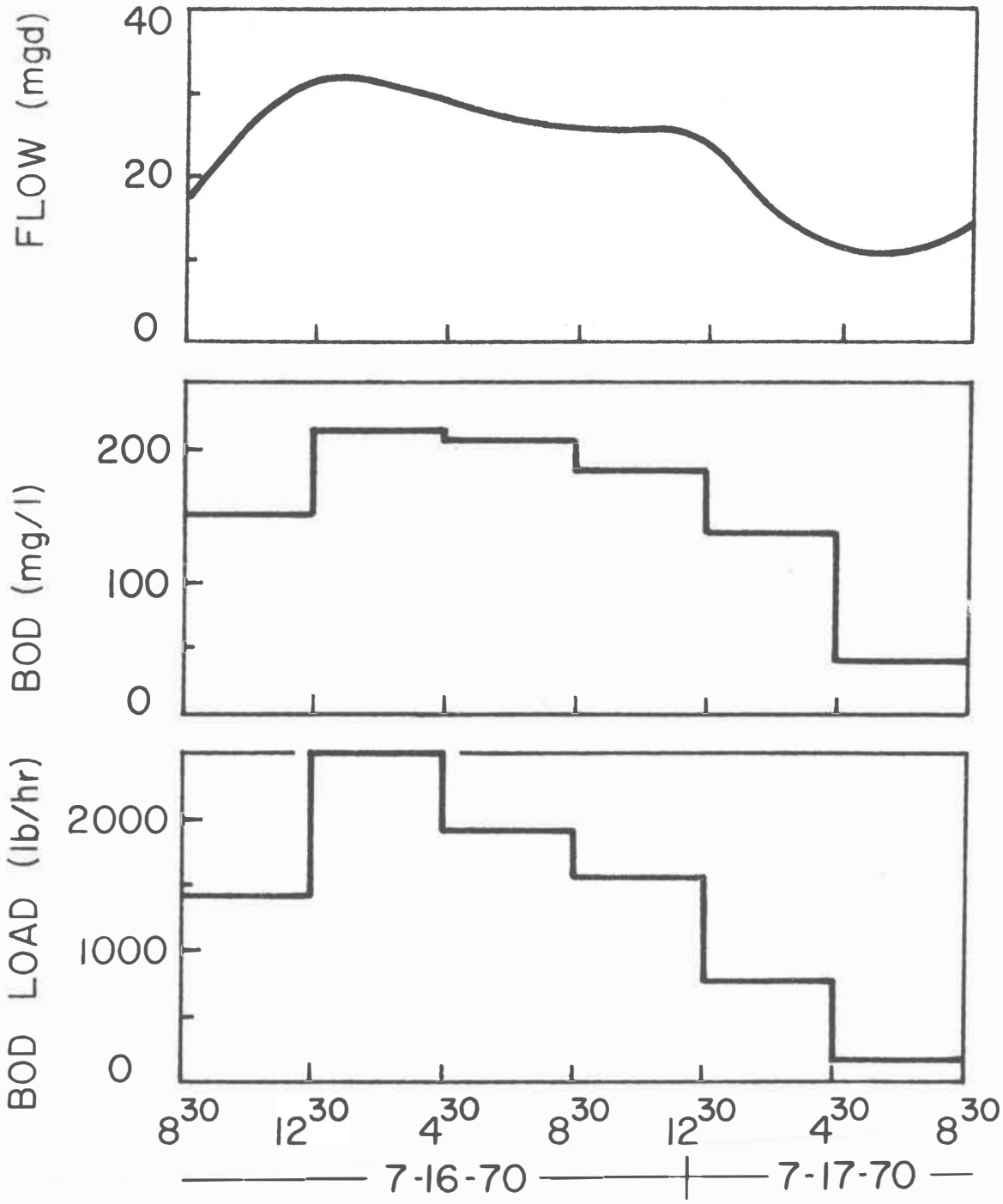

FIG. 22

BOD AND LOAD DISTRIBUTION

GOVALLE, AUSTIN, TEX

Seasonal variations in flow and in rainfall are presented in Figure 23. The average monthly and maximum and minimum daily flows at the Govalle Treatment Plant in Austin, Texas during 1969 are presented in this figure. There is some correlation between the monthly and maximum daily rainfalls and the average monthly and maximum daily flows. However, it is quite apparent that the intensity and duration of the rainfall have a more dramatic affect on the flow entering the wastewater treatment plant. A rainfall of low intensity and of long duration is much more likely to result in higher infiltration rates than a rainstorm of high intensity with only a short duration. A statistical evaluation of the average daily flow data for 1969 is presented in Figure 24. These data indicate that municipal wastewater flow can be divided into three separate statistical groups, namely the average flow for Sundays, the average flow for work days, and the wastewater flow including the infiltration of rainfall either on Sundays or on work days. The average flow at the Govalle Plant on Sunday is 20 MGD and on work days 23.5 MGD. The data in Figure 24 indicate that an average dry weather flow of 25 MGD will occur 90 percent of the time. However, a flow of 30 MGD will occur 90 percent of the time during periods of rainfall.

A daily variation in flow during dry weather flow (July 16-17, 1970) and during a rainy period (May 20-21, 1970) are presented in Figure 25. The difference in the average daily flow is 14 MGD and is attributed to infiltration. If this average rate of infiltration is subtracted from the flow recorded for the May 20-21, 1970 curve, the resulting curve will be very similar to the dry weather flow curve. The ratio of peak daily flow to average

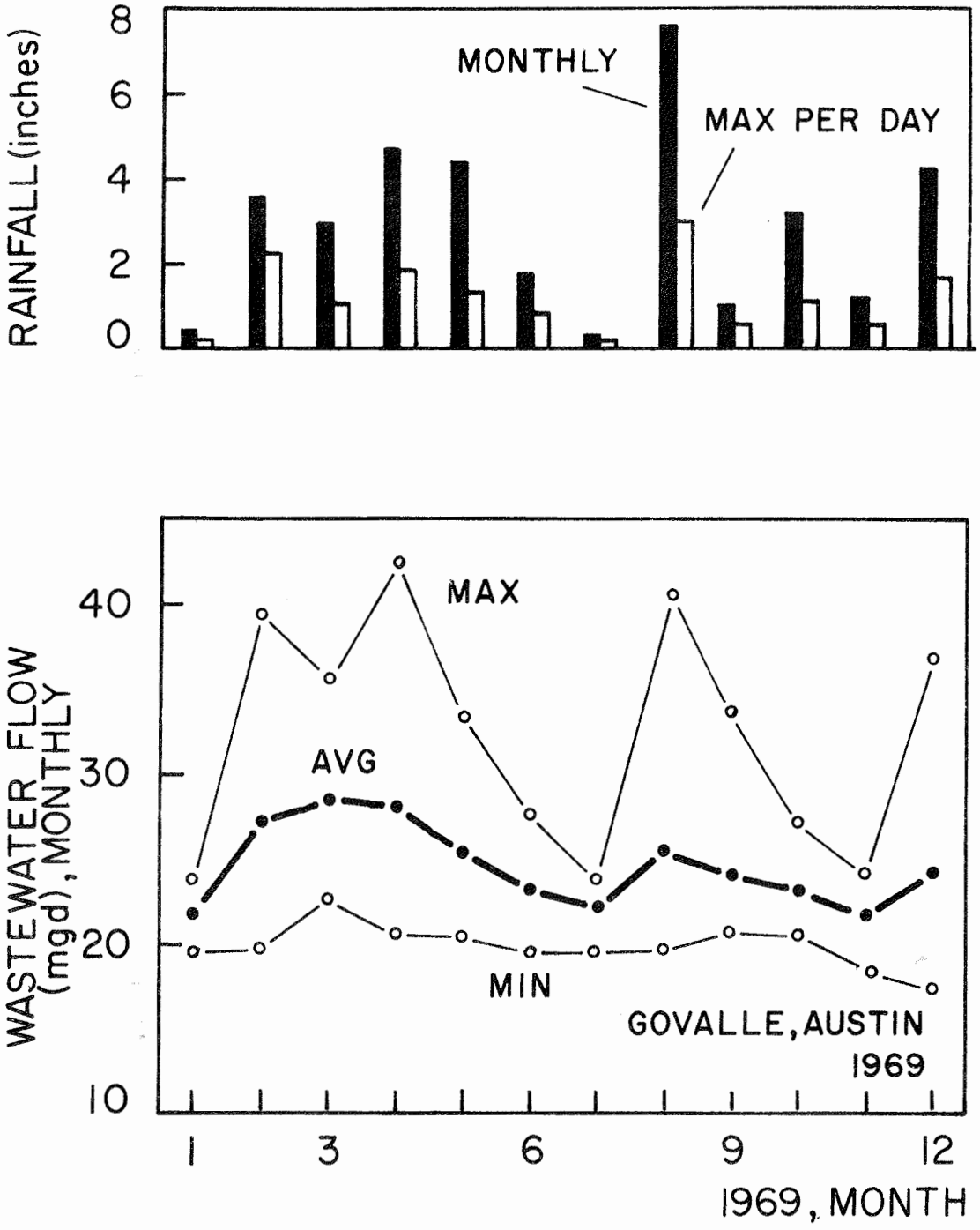

FIG. 23
RAINFALL AND WASTEWATER FLOW

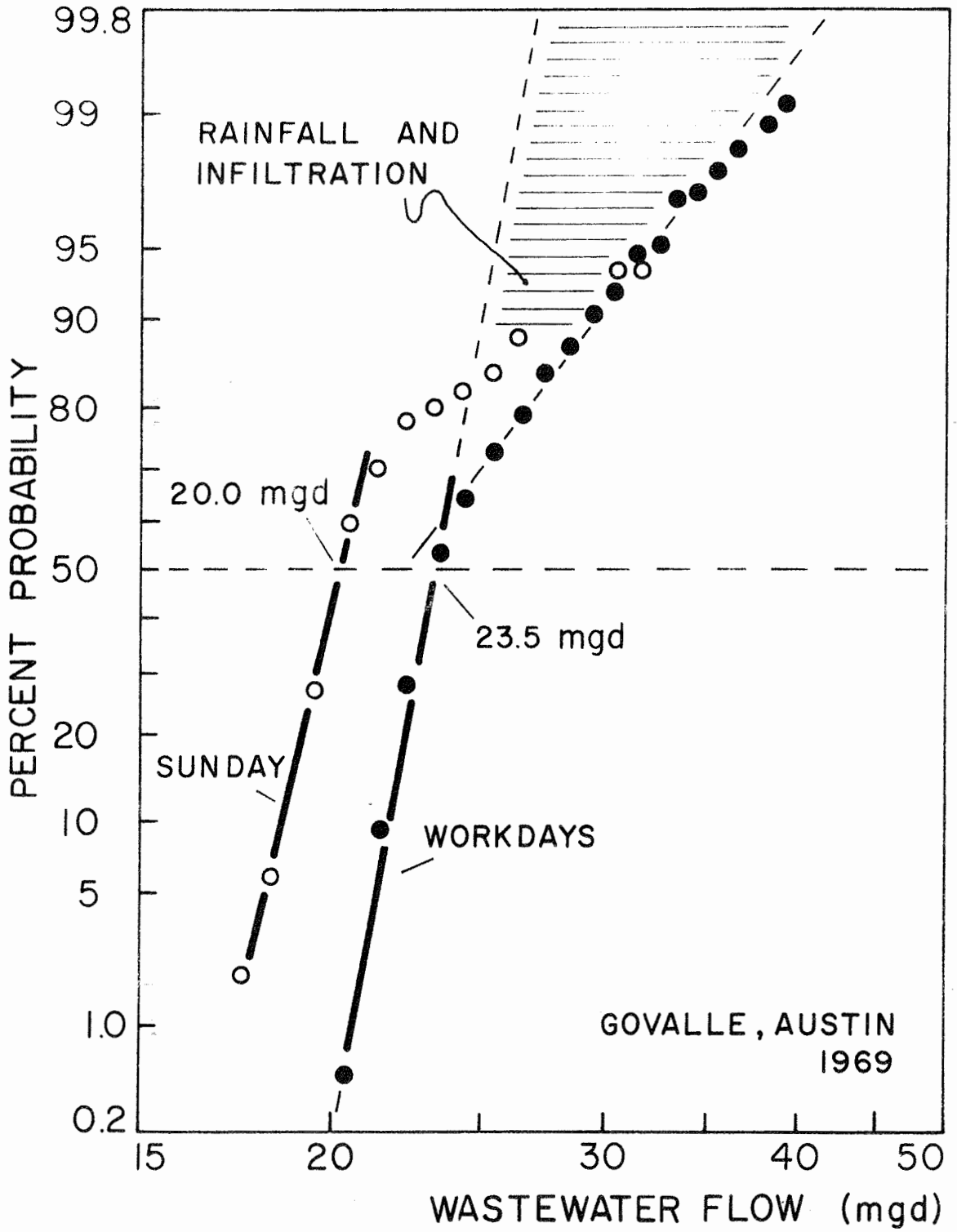

FIG. 24

PROBABILITY OF WASTEWATER FLOW

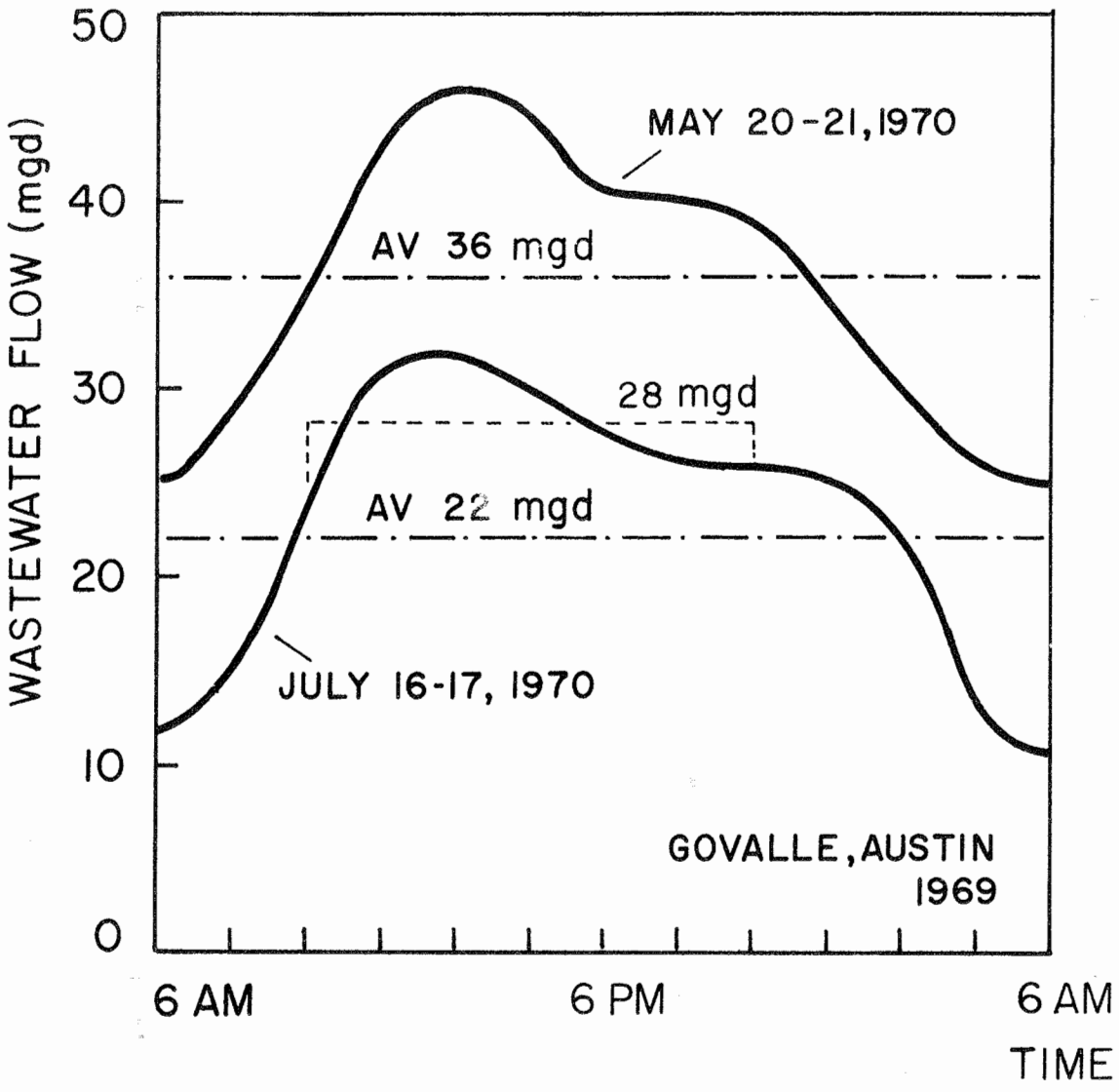

FIG. 25
WASTEWATER FLOW DISTRIBUTION
GOVALLE PLANT, AUSTIN

daily flow ranges from 1.27 to 1.45, respectively, for the wet weather flow and for the dry weather flow curves. The average flow during dry weather for the time period 10:00 a.m. to 10:00 p.m. is 28 MGD. The flow rate during this 12-hour period is about 1.26 million gallons per hour.

RESULTS

A summary of the experimental procedures and significant results are presented in this chapter. The schedule of the experimental programs is presented in Figure 26. Analytical data for each of the processes evaluated during this study are presented in Appendix A. Statistical parameters which describe the data are also included with the analytical results. More detailed discussions of the experimental and sampling as well as analysis of the results for specific processes are presented in Appendix B in the form of individual reports.

A. Conventional Activated Sludge Process

The purpose of this study was to determine the optimum operating conditions for the conventional activated sludge process and to establish the parameters used in conventional design models. A full-scale treatment plant was operated for several months at various detention times and organic loading rates.

The results of the field investigation on the operation of the conventional activated sludge system which included the evaluation of the routine data as well as the mixing in the aeration tank was developed by Halbert and Malina (1970). The effect of influent suspended solids concentration on the quantity of excess sludge produced and the oxygen uptake were evaluated in laboratory scale studies and reported by Schmidt and Eckenfelder (1970). These reports are included in Appendix B.

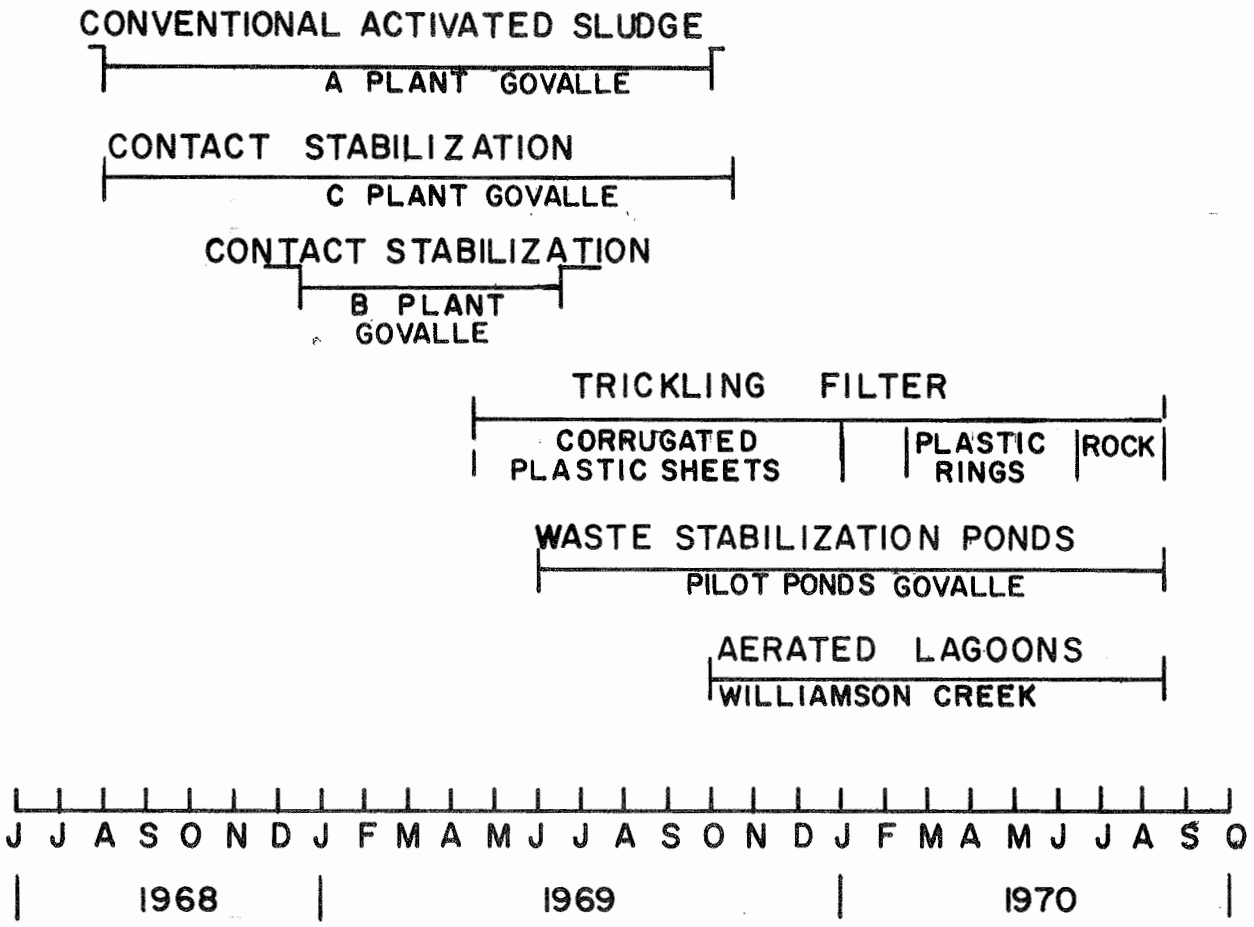

FIG.26
SCHEDULE OF EXPERIMENTAL PROGRAM

1. Experimental Program

The performance of the activated sludge process was evaluated from August 1968 through September 1969. A summary of the operating variables for each run is presented in Table 5.

TABLE 5

SUMMARY OF OPERATING CONDITIONS

Run No.	Date	Aeration* time-hour	Flow Wastewater MGD	Return Sludge MGD	lb BOD / lb MLSS-day
1 A	08/06/68 10/31/68	4.9 (7.50)	3.00	1.58	0.23
2 A	12/16/68 01/31/69	2.6 (5.83)	3.88	4.61	0.24
3 A	02/01/69 05/14/69	2.2 (3.94)	5.77	4.61	0.26
4 A	05/15/69 09/30/69	2.3 (4.23)	5.36	4.61	0.28

*Aeration time based on total flow including wastewater and return sludge. The number in parentheses ( ) is the aeration time based on the wastewater flow only.

A number of special studies were undertaken during the experimental period to supplement the data from the regular sampling schedule. These studies included oxygen uptake rates, organic utilization, physical mixing and batch kinetic evaluation. Oxygen uptake rates of the mixed liquor at various points along the length of the aeration tank were measured periodically to establish profiles of uptake rates.

The purpose of the mixing studies was to evaluate the extent of longitudinal mixing or deviation from plug flow. Three mixing studies were carried out for this particular around-the-end flow aeration system. The plant was operated under stable conditions established for the particular run. The data from the mixing studies were correlated with the soluble organic concentration and oxygen uptake data which were also recorded during the studies, which were performed between 2:00 and 6:00 p.m. when the flow into the plant remained fairly constant. The hydraulic loading conditions are summarized in Table 6.

TABLE 6

SUMMARY OF HYDRAULIC CONDITIONS

Run No.	Aeration time $\bar{t}$ (hrs)	Sewage flow (MGD)	Return Sludge flow (MGD)
1	1.90	8.25	3.60
2	1.95	7.94	3.60
3	2.70	5.60	2.74

The air flow distribution in the two parts of the basin was observed to be fairly uniform. The rated capacity of the air blowers was the only information available ragarding the air flow rates since no effective flow measuring devices were installed.

Radioactive sodium 24 was selected as the tracer because of the high activity of sodium requiring short counting times and because

sodium is a nonessential nutrient to the microorganisms. The short half-life of the sodium-24 prevented prolonged exposure and possible injury to the health of plant personnel. Profiles of the remaining soluble organics in the aeration tank and the oxygen uptake rates were established during the mixing studies.

The kinetics of soluble organic substrate removal was determined by batch unit experiments. Portions of return sludge and influent wastewater were mixed and the removal of soluble organics followed with time.

2. Results of Experimental Runs

The operating conditions and the characteristics of the samples collected at various locations in the activated sludge system are summarized in Tables 7 and 8. The average concentrations of suspended solids, BOD, TOC, COD, nitrogen and phosphate are shown for each run. The average aeration time, clarifier overflow rate and effluent temperature are also included. Meters were not available to measure the waste sludge flows and the rated capacity of the pumps were used. Since these data were very erratic they have not been included in the summary.

The aeration times based on sewage and return sludge flows varied from 2.2 to 4.9 hours. Shorter aeration times were not possible because of the hydraulic limitations of the plant.

TABLE 7

SUMMARY OF RESULTS

Run No.	Aeration Time (hr)	MLSS (mg/l)	MLVSS (mg/l)	RSS (mg/l)	RVSS (mg/l)	Overflow Rate (gpd/ft^2)	Average* Temp (^oC)
1 A	4.9	2490	2030	5610	4500	340**	29
2 A	2.6	3430	2950	5190	4300	300	24
3 A	2.2	2860	2380	5850	4690	450	22
4 A	2.3	2710	2350	6380	4960	420	28

Run No.	SS (mg/l)		BODT (mg/l)		%	BODF (mg/l)		%	TOCT (mg/l)		%
	Inf	Eff	Inf	Eff	Removed	Inf	Eff	Removed	Inf	Eff	Removed
1 A	137	12	171	23	87	71	3	96	111	19	83
2 A	172	19	187	29	84	73	6	92	128	28	78
3 A	109	22	117	21	82	44	7	84	89	28	69
4 A	150	22	124	20	84	37	7	81	91	23	75

* temperature of plant effluent

**only two of three clarifier operated in Run 1A

TABLE 8

SUMMARY OF RESULTS

Run No.	TOCF (mg/l) Inf.	TOCF (mg/l) Eff.	% Removed	CODT (mg/l) Inf.	CODT (mg/l) Eff.	% Removed	CODF (mg/l) Inf.	CODF (mg/l) Eff.	% Removed
1 A	48	14	71	376	69	82	159	42	74
2 A	48	20	58	389	76	80	138	49	64
3 A	35	19	46	254	65	74	110	40	64
4 A	28	17	39	331	65	80	99	43	57

Run No.	NH_3 (mg/l) Inf.	NH_3 (mg/l) Eff.	TKN (mg/l) Inf.	TKN (mg/l) Eff.	$NO_2 + NO_3$ (mg/l) Inf.	$NO_2 + NO_3$ (mg/l) Eff.	PO_4 (mg/l) Inf.	PO_4 (mg/l) Eff.
1 A	23.8	12.0	--	--	--	--	--	--
2 A	23.6	17.9	26.9	20.4	0.1	1.6	32.3	24.2
3 A	20.6	16.8	25.5	20.2	0.1	0.1	24.2	18.9
4 A	21.1	15.2	23.2	15.5	0.1	0.3	24.1	17.7

The reduction in the concentrations of the total BOD, TOC, and COD remained relatively constant for all runs. The results indicate a decrease in the percent reduction of soluble BOD, TOC, and COD as the aeration time decreased. However, the influent soluble BOD, TOC, and COD similarly decreased and probably explains the apparent drop in efficiency.

Although there were significant reductions in the concentrations of ammonia and total Kjeldahl nitrogen, very little nitrification occurred during any of the runs. There were also noticeable decreases in the phosphate contents.

Organic loading rates and removal rates, expressed as pounds BODT per day per pound of mixed liquor suspended solids in the aeration system were calculated based on the relationships presented in Equations 1 and 2.

$$\text{Loading Rate} = \frac{QS_o}{VX} \qquad (1)$$

$$\text{Removal Rate} = \frac{Q\,(S_o - S_e)}{VX} \qquad (2)$$

where:

Q = total daily flow

S_o = influent BODT

S_e = effluent BODT

VX = total suspended solids in the aeration system

A summary of these data are presented in Table 9.

TABLE 9

SUMMARY OF LOADING AND REMOVAL RATES

Run No.	BODT Loading Rate (lb/lb/day)	BODT Removal Rate (lb/lb/day)
1 A	0.234	0.198
2 A	0.244	0.206
3 A	0.258	0.216
4 A	0.278	0.234

The results of the loading rate studies indicated that approximately 85 percent of the total BOD was removed in all runs. The aeration time in the system ranged from 2.2 to 4.9 hours, however, the average total BOD loading rates varied only from 0.234 to 0.278 pounds BODT per pound MLSS per day. The narrow range of the loading rates was caused by a significantly lower influent total BOD concentration during Runs 3 A and 4 A. The influent total TOC and COD data showed a similar decrease in concentration which was attributed to dilution of the sewage by infiltrating rainwater during a period of heavy rainfall. The effluent filtered BOD, TOC, and COD results showed little variation for all runs.

The physical limitations of the test system limited the range of loading rates over which operational data could be collected. Thus the operating results for the Govalle Treatment Plant were insufficient for

complete evaluation of the parameters of conventional models. However, data from other plants were used in developing the design models.

The oxygen uptake rates of the full-scale test system indicated a linear relationship between oxygen uptake rate and distance along the length of the tank. Typical data are presented in Figure 27. A base level at which the uptake rate remained constant was also established. The oxygen uptake rate decreased in a manner similar to the decrease in the concentration of soluble organics. The maximum oxygen uptake rate decreased in a manner similar to the decrease in concentration of soluble organics. The maximum oxygen uptake rates were observed in Run 4 A when the temperature of the mixed liquor was 28°C. No direct relationship could be found between the BOD removal rate and oxygen uptake rate data for any of the runs.

The results of the mixing studies indicated that the dispersion model best characterized the mixing regime in the aeration system. This experiment was designed such that several points along the aeration tank were sampled thus reducing the number of tests required to establish the mixing characteristics.

The soluble organic fraction of the influent wastewater was removed within the first of the series of two aeration basins. As previously mentioned, the oxygen uptake rate decreased similarly to the concentration of soluble organic matter with only a base activity level noted in the second

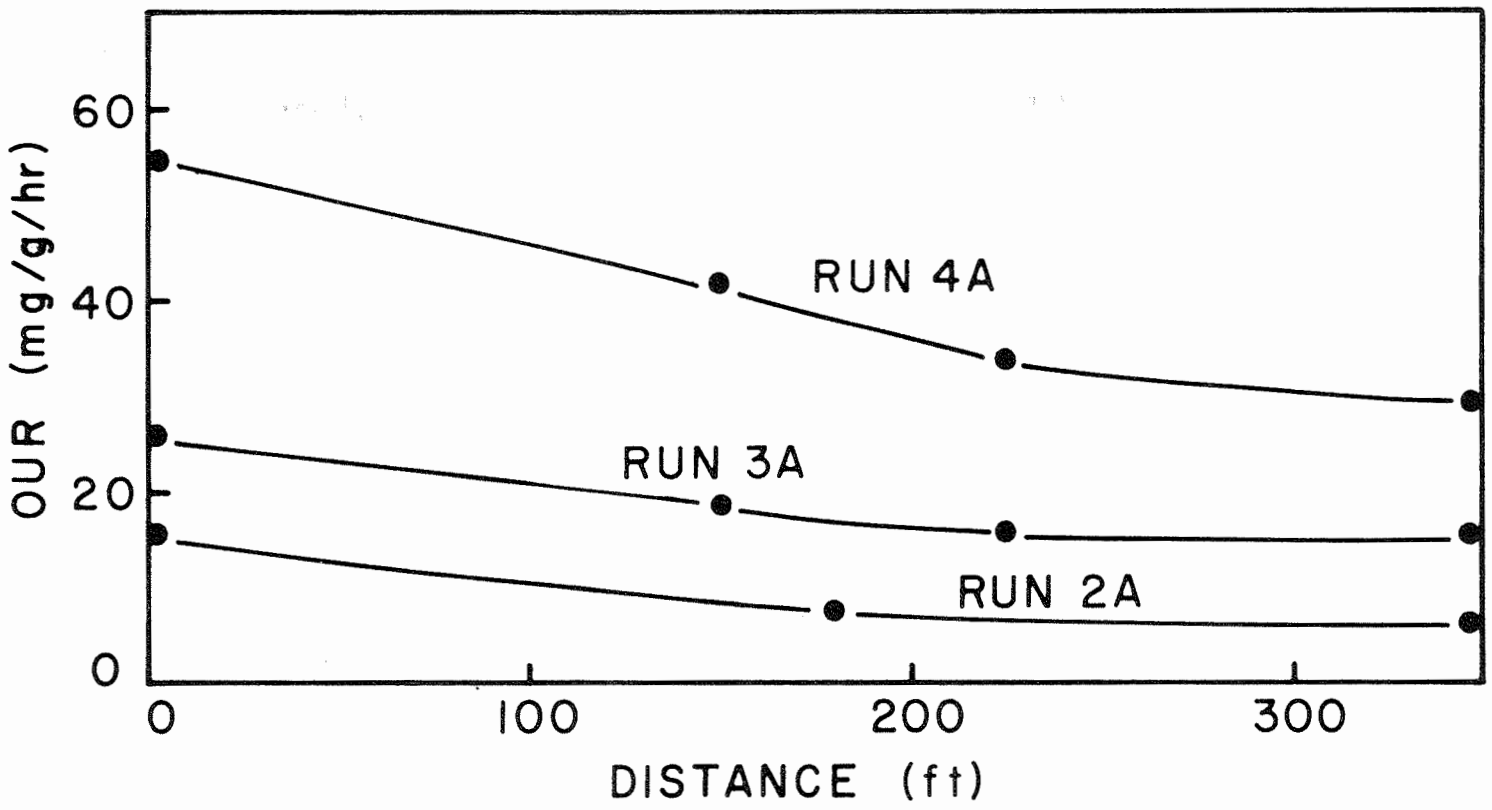

FIG.27 UNIT OXYGEN UPTAKE RATES — RUNS 2A, 3A, 4A

aeration basin. None of the predictive models described the performance of the aeration system available in this study. The relatively low influent soluble substrate concentration and rapid removal of the substrate upon contact with the return activated sludge may be the reason for these discrepancies.

B. Contact Stabilization Process

The purpose of this study was to define some of the design considerations for the contact stabilization process and to attempt to determine the optimum operating conditions of the process. Full-scale treatment plants were operated for a number of months using various contact times, stabilization times and organic loading rates. The project was limited to the evaluation of data from plants having a configuration as illustrated in Figure 2 in which there was no physical separation of the stabilization and contact zones. A laboratory-scale study was undertaken as a supplement to the full-scale experiments to determine the influences of contact time, stabilization time and loading rate on the performance of the contact stabilization process.

The results of the field investigation of the contact stabilization process are evaluated and compared with the results of batch tests in a report prepared by Myatt and Malina (1970). The results of the laboratory-scale study are discussed by Berryhill, Malina, and Kayser (1970). These reports are included in Appendix B.

1. Experimental Program

The performance of the full-scale contact stabilization process was evaluated at the Govalle Plant using the B Plant from December 15, 1968 to June 12, 1969 and the C Plant from August 6, 1968 to October 23, 1969. The contact times and stabilization times were changed for each run by using various flow rates and aeration tank volumes. A summary of the physical characteristics of each run is given inTable 10. The contact times given are based on the total wastewater and return sludge flows.

TABLE 10

SUMMARY OF OPERATING CONDITIONS

Run No.	Date	Contact Time (Hr.)	Stabilization Time (Hr.)	Flow Waste-water MGD	Flow Return Sludge MGD
1 C	08/06/68 - 10/29/68	0.52	1.07	11.96	11.20
2 C	12/15/68 - 03/05/69	0.38	3.14	9.92	5.76
3 C	02/13/69 - 05/05/69	0.28	2.00	11.72	8.93
4 C	05/10/69 - 06/18/69	0.99	3.21	8.48	3.74
5 C	06/21/69 - 07/21/69	0.30	3.30	10.32	5.76
6 C	07/23/69 - 08/31/69	0.84	3.21	11.52	3.74
7 C	09/01/69 - 10/23/69	0.71	1.80	11.94	5.76
2 B	12/15/68 - 03/05/69	0.45	4.13	9.46	4.02
3 B	03/06/69 - 05/05/69	0.42	4.25	10.22	3.90
4 B	05/10/69 - 06/12/69	0.28	3.57	9.65	5.17

The hydraulic capacity of B Plant limited the lower contact times which could be used. In addition, short stabilization times in combination with low contact times were not feasible because higher return sludge rates were not possible.

Oxygen uptake rates of the mixed liquor at various points along the length of the aeration tank were measured periodically during most of the experimental runs. Laboratory aeration tanks were used for batch tests to determine the difference in oxygen uptake rates during the stabilization and contact periods.

The operating conditions for the supplementary laboratory studies are summarized in Table 11 and a schematic drawing of the laboratory-scale unit is illustrated in Figure 28. The contact time was varied from less than ten minutes to 30 minutes and the stabilization time was maintained constant at one hour.

2. Results of Experimental Runs

The operating conditions for the contact stabilization process and the characteristics of samples collected at different points in the system are summarized in Tables 12, 13, 14, and 15. The average suspended solids, BOD, TOC, COD, and nitrogen and phosphate data are shown for each run. Since meters were not available to measure the waste sludge flows, the rated capacity of the pumps were used.

The MLSS in the contact zone ranged from 2000 to 3350 mg/l. The suspended solids in the stabilization zone (AESS) was based on the average of the return sludge and that at a point 25 feet from the contact zone and varied from 5350 to 7400 mg/l. During Run 5 C, some difficulties

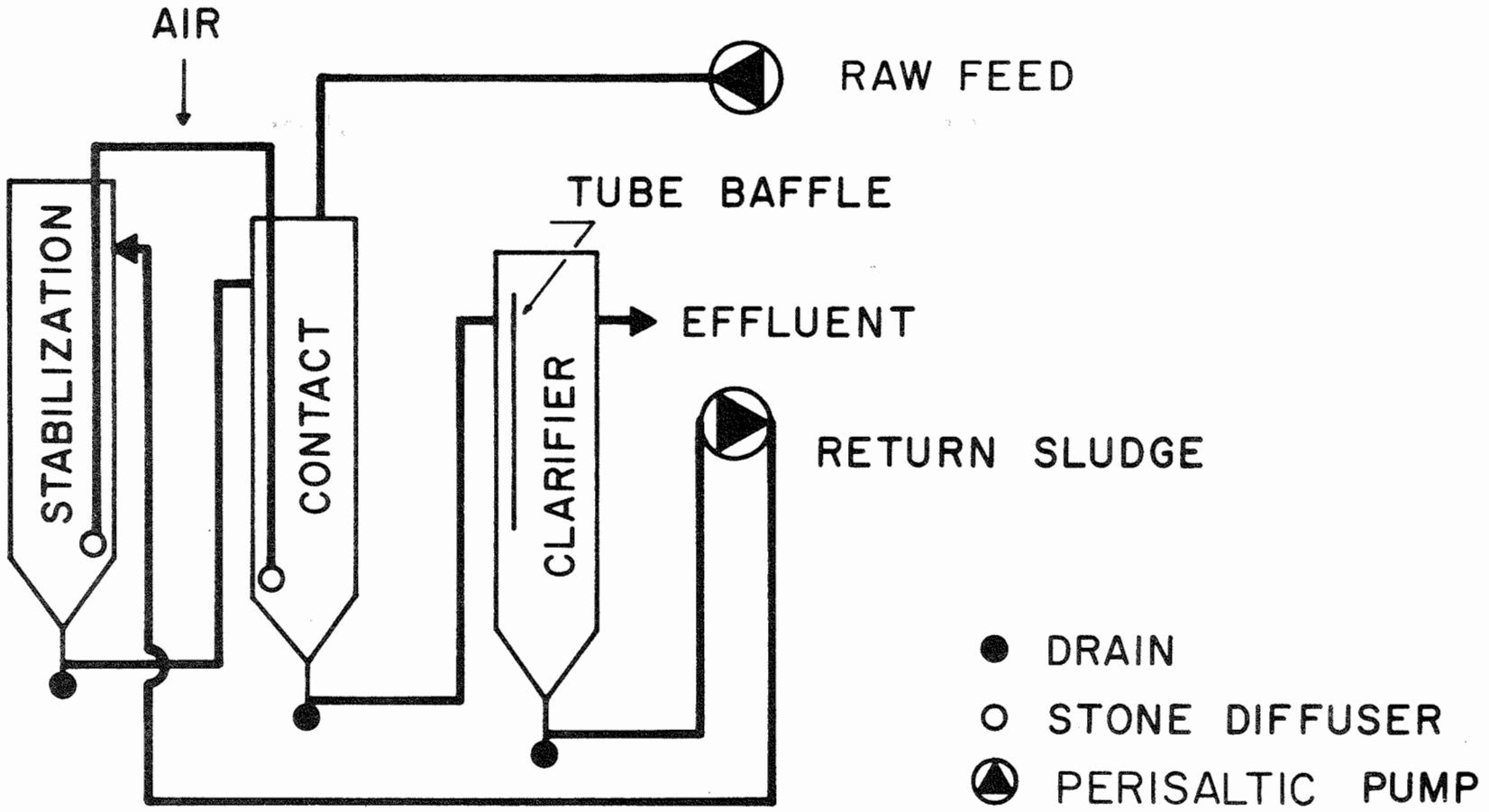

FIG.28
THE LABORATORY CONTACT STABILIZATION UNIT

TABLE 11

UNIT CONFIGURATIONS AND FLOW RATES

Run	Date: Start of Run	Date: End of Run	Unit 1: V_c (liters)	Unit 1: V_s (liters)	Unit 1: Clarifier Volume (liters)	Unit 1: Waste-water Feed Rate (l/hr)	Unit 1: Return Sludge Rate (%)	Unit 2: V_c (liters)	Unit 2: V_s (liters)	Unit 2: Clarifier Volume (liters)	Unit 2: Waste-water Feed Rate (l/hr)	Unit 2: Return Sludge Rate (%)
1	May 4	May 22	4	4	4	4.0	100	4	4	4	4.0	450* to 700
2	May 22	June 5	2	4	4	4.0	100	1	4	4	4.0	100
3	June 5	June 19	1/2	4	4	4.0	100	1	4	4	4.0	100
4	June 19	July 3	2	0	4	4.0	100	2	4	4	4.0	100

V_c = contact volume

V_s = stabilization volume

* = return sludge rate variation caused by the use of air lift pumping

TABLE 12

SUMMARY OF RESULTS

Run No.	t_c hr	t_s hr	MLSS mg/l	MLVSS mg/l	RSS mg/l	AESS mg/l	Overflow Rate gpd/sq ft	Average Temp. ^{o}C
1 C	0.52	1.07	3350	2750	7350	6800	1050	29
2 C	0.38	3.14	2900	2400	7550	7500	875	21
3 C	0.28	2.00	2400	2000	8100	6900	1050	22
4 C	0.99	3.21	2200	1800	7250	5950	750	25
5 C	0.30	3.30	2700	2200	6350	5700	925	29
6 C	0.84	3.21	2400	1900	7250	6000	950	30
7 C	0.71	1.80	2250	1850	6100	5350	1025	29
2 B	0.45	4.13	2400	2000	6950	6300	825	21
3 B	0.42	4.25	2000	1650	6700	5700	900	22
4 B	0.28	3.57	3100	2550	7450	6750	825	25

NOTE: Mixed liquor average volatile content was 82 percent

TABLE 13

SUMMARY OF RESULTS

Run No.	tc hr	ts hr	SS mg/l Inf.	SS mg/l Eff.	BODT mg/l Inf.	BODT mg/l Eff.	% Removed	BODF mg/l Inf.	BODF mg/l Eff.	% Removed
1 C	0.52	1.07	250	26	208	42	80	69	6	91
2 C	0.38	3.14	219	26	169	33	80	54	10	81
3 C	0.28	2.00	184	18	104	22	79	41	8	80
4 C	0.99	3.21	146	16	104	24	77	25	7	72
5 C	0.30	3.30	133	43	107	26	76	38	6	84
6 C	0.84	3.21	169	25	129	29	78	32	6	87
7 C	0.71	1.80	165	31	128	27	79	45	6	87
2 B	0.45	4.13	159	50	155	47	70	60	11	82
3 B	0.42	4.25	124	38	106	22	79	37	11	70
4 B	0.28	3.57	154	28	109	26	76	32	10	69

TABLE 14

SUMMARY OF RESULTS

Run No.	tc hr	ts hr	TOCT mg/l Inf.	TOCT mg/l Eff.	% Removed	TOCF mg/l Inf.	TOCF mg/l Eff.	% Removed	CODT mg/l Inf.	CODT mg/l Eff.	CODF mg/l Inf.	CODF mg/l Eff.
1 C	0.52	1.07	145	28	81	47	17	64	490	98	156	50
2 C	0.38	3.14	128	32	75	47	21	55	425	95	143	55
3 C	0.28	2.00	85	27	68	31	20	35	293	73	102	48
4 C	0.99	3.21	79	23	71	25	17	32	302	60	100	35
5 C	0.30	3.30	89	29	67	34	19	44	289	72	100	43
6 C	0.84	3.21	76	20	74	25	14	44	326	76	114	41
7 C	0.71	1.80	71	21	70	26	14	46	337	78	129	43
2 B	0.45	4.13	116	42	64	45	24	47	393	123	140	56
3 B	0.42	4.25	81	29	64	32	21	34	241	86	102	49
4 B	0.28	3.57	90	26	71	26	19	27	277	78	84	45

TABLE 15

SUMMARY OF RESULTS

Run No.	tc hr	ts hr	NH_3 mg/l		TKN mg/l		$NO_2 + NO_3$ mg/l		PO_4 mg/l	
			Inf.	Eff.	Inf.	Eff.	Inf.	Eff.	Inf.	Eff.
1 C	0.52	1.07	---	---	---	---	---	---	---	---
2 C	0.38	3.14	22.3	14.5	26.9	17.0	0.1	1.4	29.0	25.5
3 C	0.28	2.00	19.3	15.5	22.4	20.6	0.1	0.3	21.1	17.5
4 C	0.99	3.21	---	---	---	---	---	---	25.6	18.5
5 C	0.30	3.30	25.3	18.8	33.2	24.3	0.0	0.6	23.4	20.6
6 C	0.84	3.21	19.1	11.8	20.5	12.6	0.1	0.7	23.2	18.8
7 C	0.71	1.80	18.1	13.3	20.7	14.2	0.1	0.2	25.8	19.3
2 B	0.45	4.13	23.2	16.9	27.6	19.9	0.1	0.5	31.1	27.1
3 B	0.42	4.25	19.5	16.4	25.3	21.1	0.1	0.1	21.3	20.2
4 B	0.28	3.57	---	---	---	---	---	---	24.5	23.5

were encountered in obtaining a stable operation and the high effluent suspended solids may have been the result of irregular sludge wasting which would cause a sludge buildup in the clarifiers. The effluent suspended solids were also high during Runs 2 B, 3 B, and 4 B.

The removal of total BOD ranged from 70 to 80 percent and soluble BOD from 69 to 91 percent. There were small reductions in the concentration of ammonia and total Kjeldahl nitrogen. These ranged from 16 to 37 percent for NH_3 and from seven to 37 percent for TKN. Since the nitrate concentration of the effluent was very low, it was assumed that little nitrification took place. There were also slight reductions in the phosphate levels. The relationship between the average total BOD loading rate and total BOD removal rate for all runs is presented in Figure 29. The rates are expressed in pounds of BODT per day per pound of mixed liquor suspended solids in the total aeration system and were calculated using Equations 1 and 2 presented on page 70.

The data presented in Figure 29 indicate that the relationship between loading rate and removal rate is linear within the range of the loading rates used during this study. The slope of the line is 0.78 which indicates that the average reduction of BODT was 78 percent.

Typical unit oxygen uptake rates profiles for the contact stabilization process are illustrated in Figures 30 and 31. The arrows indicate the point at which the influent wastewater is introduced into the aeration tank. No direct relationship could be found between the BOD removal rate and oxygen uptake rate for any of the runs.

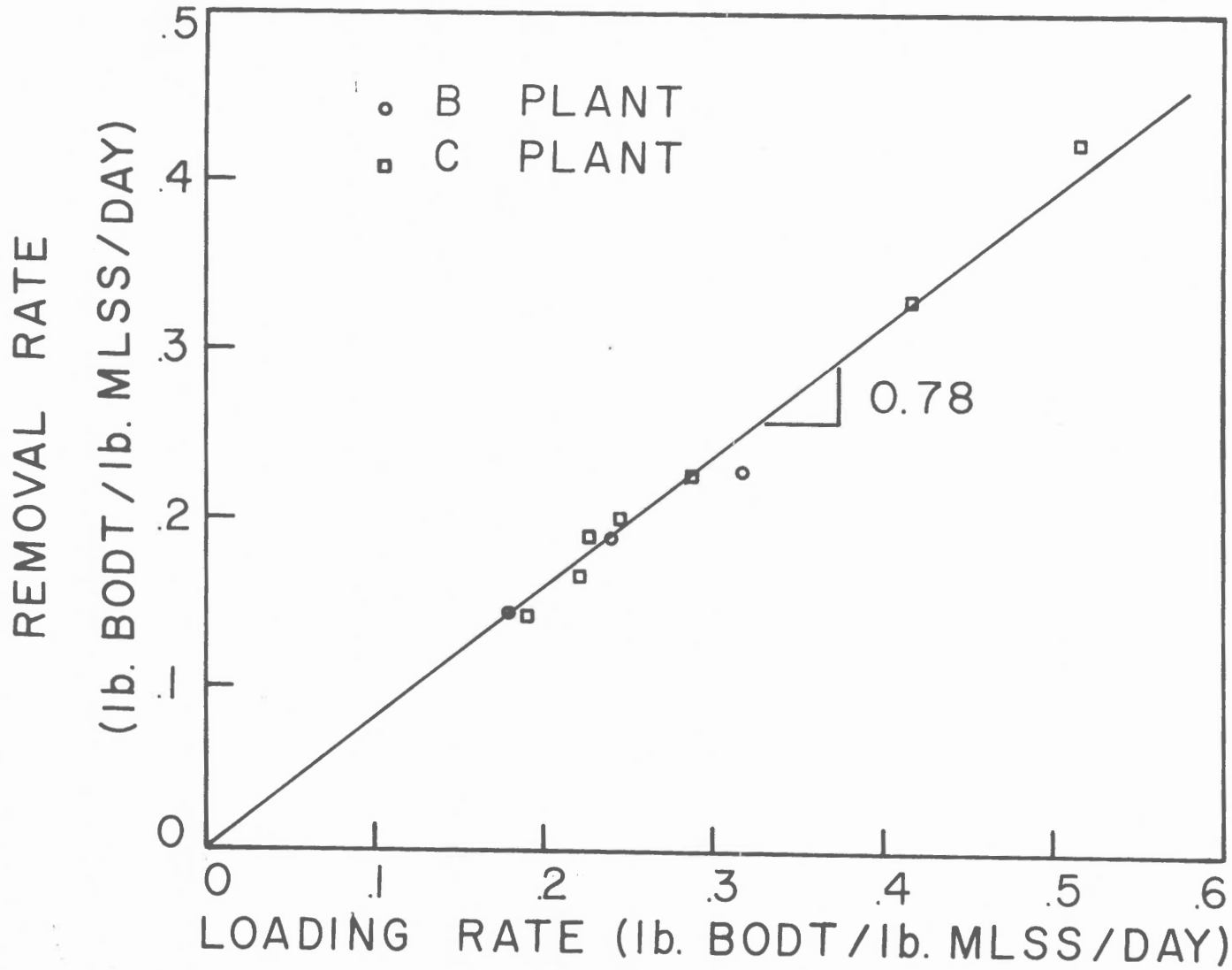

FIG.29 BODT LOADING RATE VS REMOVAL RATE

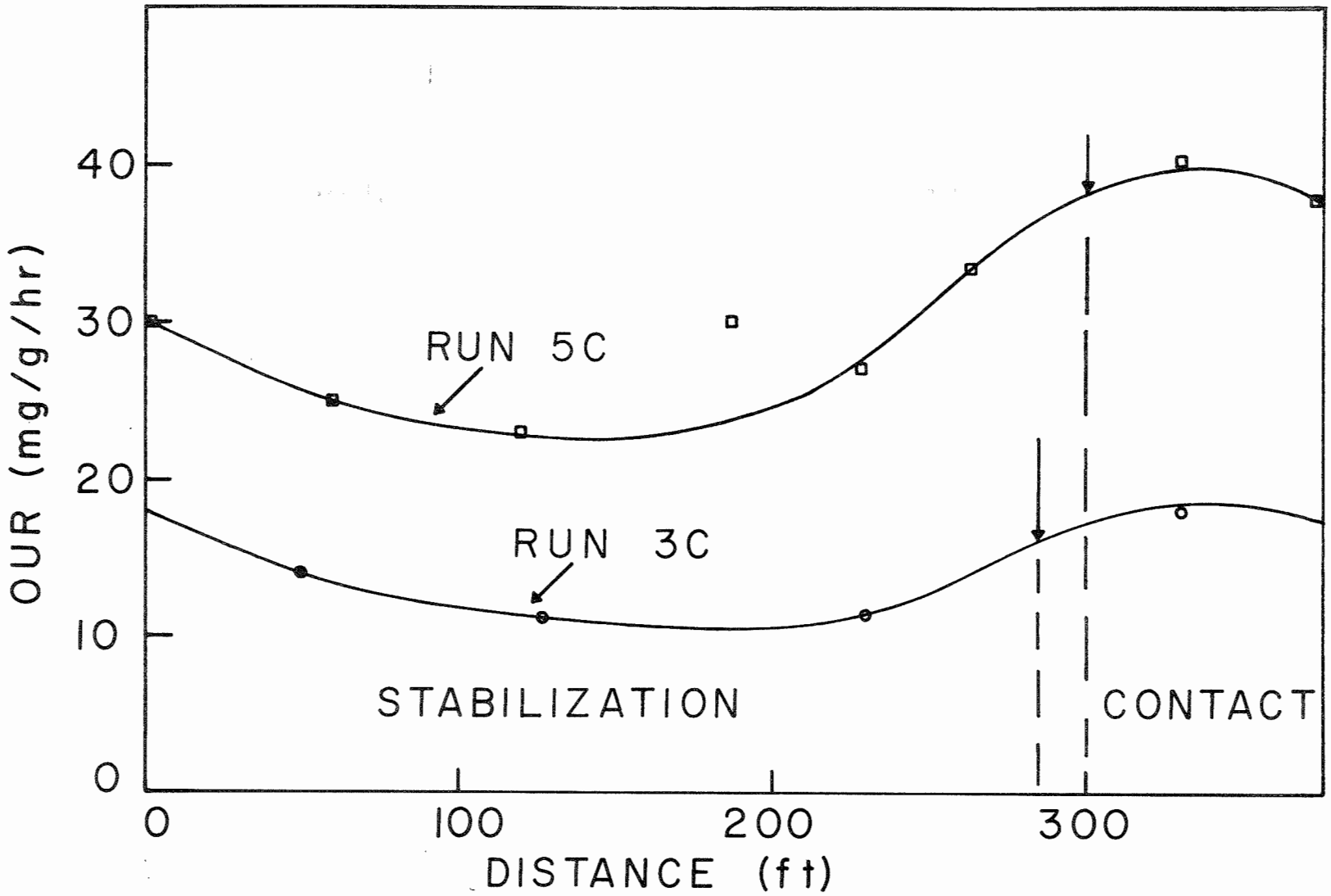

FIG.30 UNIT OXYGEN UPTAKE RATES — RUNS 3C, 5C

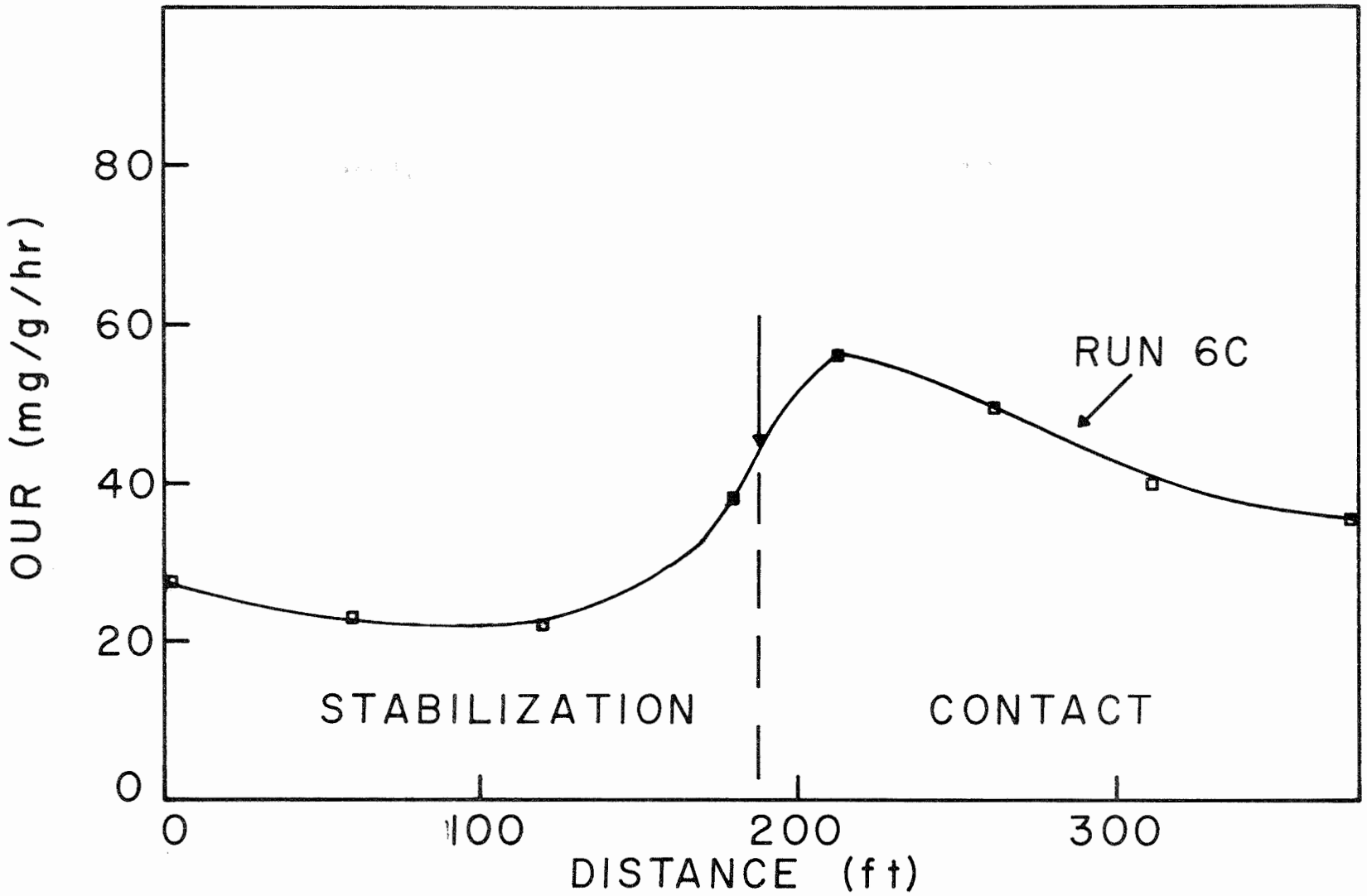

FIG.31 UNIT OXYGEN UPTAKE RATES-RUN 6C

These profiles are typical of a system in which there is no physical separation of the stabilization and contact zones. The data indicate that the unit oxygen uptake rate increases before the wastewater is added. This phenomenon is attributable to the fact that there is a considerable degree of backmixing because the zones are not physically separated. The profile illustrated in Figure 32 represents oxygen uptake rate data observed in batch studies and shows the shape of the profile for systems in which the contact and stabilization zones are physically separated. Summary data observed for the laboratory investigations of the contact stabilization process are presented in Tables 16 and 17. These results correlate well with those observed during the full-scale plant studies. The laboratory and field data are presented in Figure 33 to illustrate this correlation.

These data indicate that short contact periods of about ten to fifteen minutes were adequate to reduce the soluble BOD in the wastewater at the Govalle Wastewater Treatment Plant to acceptable low levels. This wastewater contained an average total BOD of about 100 mg/l.

Substrate removal limits were not exceeded in these studies, thus no minimum contact or stabilization periods could be established.

C. Trickling Filter Process

The primary purpose of the trickling filter experiments were to verify the existing mathematical models which describe the biodegradation of soluble

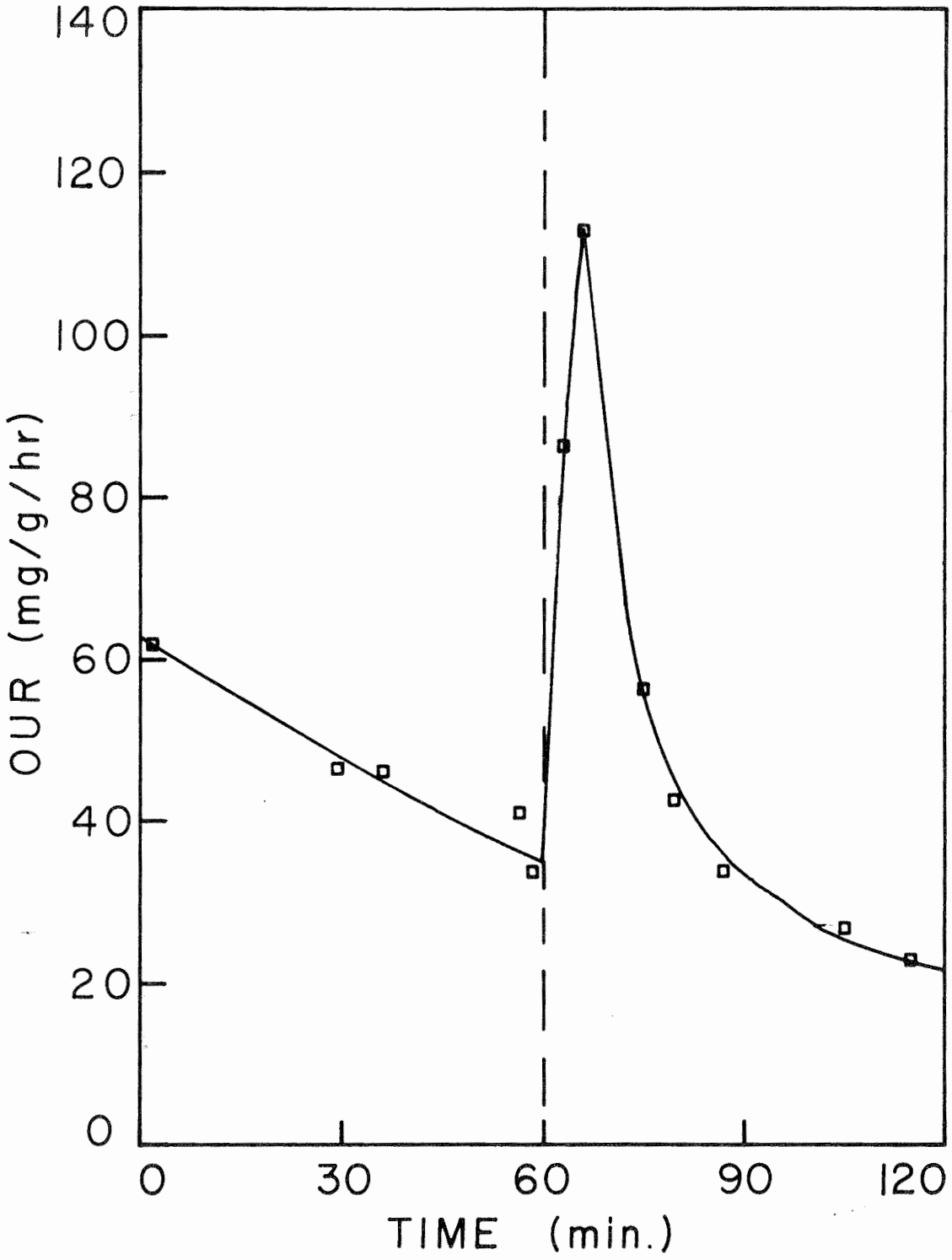

FIG. 32 OUR BATCH TEST NO. 2

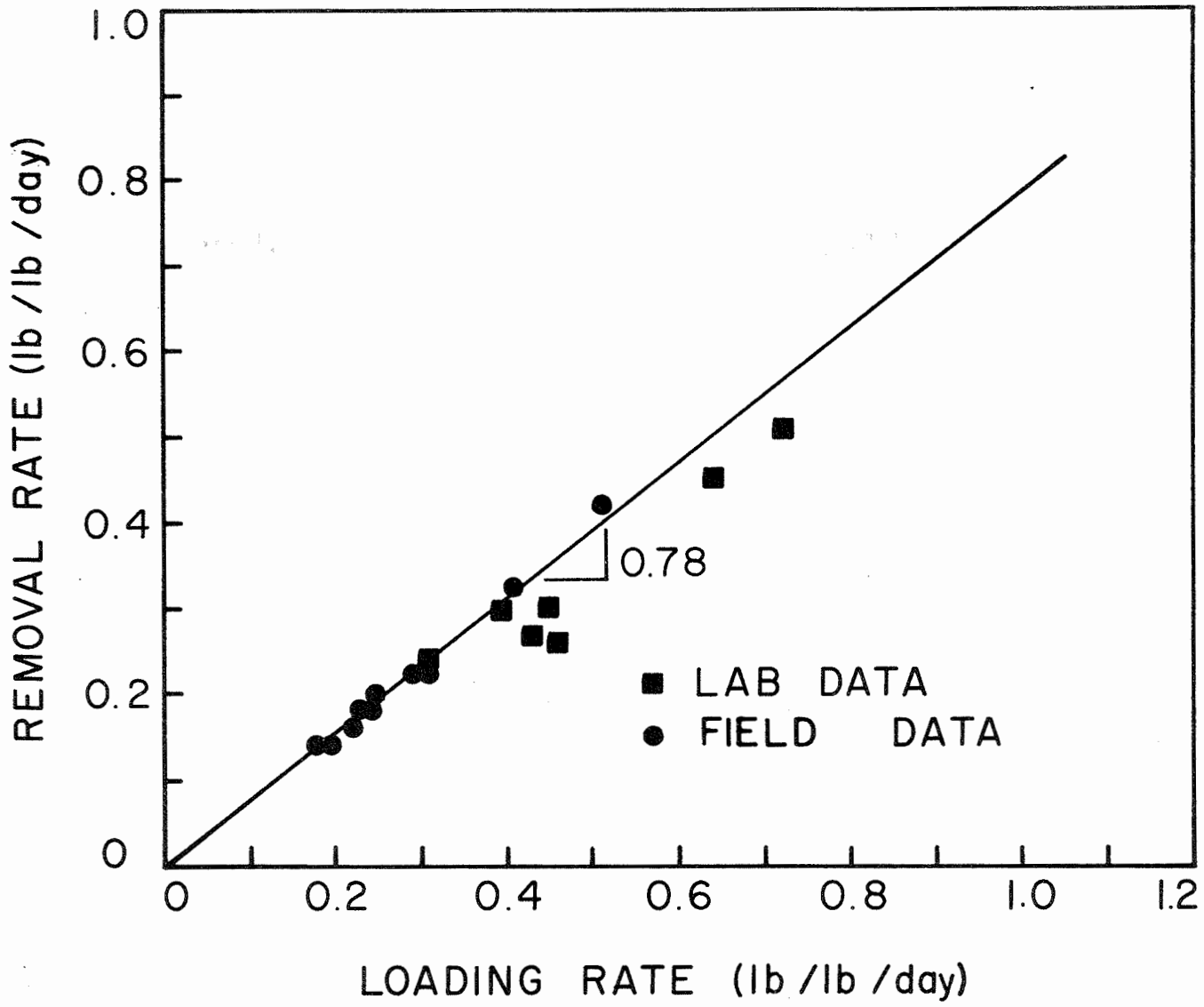

FIG.33 BODT LOADING VS REMOVAL RATE

TABLE 16

SUMMARY OF RESULTS

Run No.	Unit No.	°C		Tank Vol. (l)	MLSS RSS (mg/l)		TOCT (mg/l)	TOCF (mg/l)	BODT (mg/l)	BODF (mg/l)	CODT (mg/l)	CODF (mg/l)	SS (mg/l)
1	1	24	Cont	4	2750	INF	76	32	104	36	317	57	123
			Stab	4	5450	EFF	23	16	23	3	83	36	38
	2	24	Cont	4	2750	INF	76	32	104	36	317	57	123
			Stab	4	3350	EFF	28	15	29	3	118	38	63
2	1	24	Cont	2	2050	INF	55	20	73	28	235	47	108
			Stab	4	3100	EFF	16	13	27	4	68	33	24
	2		Cont	1	2550	INF	55	20	73	28	235	47	108
			Stab	4	3200	EFF	16	11	32	3	69	34	25
3	1	26	Cont	1/2	2950	INF	59	25	122	36	248	52	119
			Stab	4	3700	EFF	19	15	35	5	83	46	24
	2	26	Cont	1	3000	INF	59	25	122	36	248	52	119
			Stab	4	3850	EFF	20	15	36	5	75	46	27
4	1	28	Cont	2	4700	INF	80	39	102	41	301	66	170
			Stab	0	----	EFF	25	22	13	6	64	45	10
		28	Cont	2	2950	INF	80	39	102	41	301	66	170
			Stab	4	4250	EFF	25	19	30	5	73	43	23

TABLE 17

SUMMARY OF RESULTS

Run No.	Unit No.		Soluble Contact Load (S_o^*)	Soluble Contact Removal ($S_o^* - S_e^*$)	Total Load (S_o)	Removal ($S_o - S_e^*$)	Rate ($S_o - S_e$)
1	1	TOC	0.28	0.14	0.22	0.18	
		BOD	0.32	0.29	0.31	0.30	0.24
		COD	0.50	0.18	0.93	0.82	
	2	TOC	0.28	0.15	0.29	0.24	
		BOD	0.32	0.29	0.39	0.40	0.30
		COD	0.50	0.17	1.25	1.10	
2	1	TOC	0.47	0.17	0.32	0.25	
		BOD	0.66	0.56	0.43	0.40	0.27
		COD	1.10	0.33	1.37	1.17	
	2	TOC	0.75	0.34	0.35	0.28	
		BOD	1.05	0.94	0.46	0.44	0.26
		COD	1.77	0.49	1.47	1.26	
3	1	TOC	1.62	0.65	0.35	0.26	
		BOD	2.33	2.01	0.72	0.69	0.51
		COD	3.39	0.39	1.46	1.19	
	2	TOC	0.80	0.32	0.32	0.23	
		BOD	1.16	0.99	0.64	0.61	0.45
		COD	1.67	0.19	1.30	1.06	
4	1	TOC	0.40	0.17	0.82	0.59	
		BOD	0.42	0.36	1.04	0.98	0.91
		COD	0.67	0.21	3.07	2.62	
	2	TOC	0.64	0.33	0.34	0.26	
		BOD	0.67	0.59	0.43	0.41	0.30
		COD	1.07	0.37	1.26	1.08	

or organic compounds. The limitations of the model pilot plant filter made it impossible to evaluate all the factors which might have influence in the design and operation of a trickling filter. An evaluation of mathematical models used for evaluating trickling filters and the results of the studies using the corrugated media are presented by Gromiec and Malina (1970). This report is included in Appendix B.

1. Experimental Program

The operating schedule for the trickling filter of the different packing media is summarized in Table 18.

TABLE 18

TRICKLING FILTER OPERATING CONDITION

Package Material	Run No.	Period	Hydraulic Loading Rate gpm/sq ft	mgad
Corrugated Plastic Sheet	1	04/14/69 - 06/05/69	1.0	63.0
	2	06/11/69 - 08/28/69	1.5	94.5
	3	10/11/69 - 11/14/69	3.0	189.0
	4	12/02/69 - 01/07/70	+1.5 RC	+94.5 RC
Polypropylene Rings	5	02/15/70 - 04/16/70	1.0	63.0
	6	04/20/70 - 06/18/70	3.0	189.0
Limestone	7	06/25/70 - 07/15/70	0.7	45.0
	8	07/16/70 - 07/23/70	1.0	63.0
	9	07/24/70 - 08/06/70	2.0	126.0

A description of the media and some of the physical characteristics of the material are presented in Chapter II on page 18.

The pilot scale trickling filter was operated without a final clarifier. However, the effluent samples were allowed to settle for one hour in the field and a supernatant sample was analyzed as the settled effluent. However, this one hour settling does not really simulate actual settling on the field conditions; therefore, the effluent samples were filtered and only the soluble portion analyzed.

A 24-hour composite sample of the influent and effluent of the trickling filter was collected by means of a solenoid valve which was operated on a timer. A sample was collected every 30 minutes and stored in the refrigerator in 20-liter containers. Grab samples were also collected at various depths of the filter at regular intervals during the period of study. The sampling pipes extended into the filter medium. During the runs in which a rock medium was used no grab samples were possible because the sampling pipes could not be installed into the medium.

2. Experimental Results

The chemical characteristics of the composite influent and effluent sample collected during the experimental runs in which the performance of the corrugated plastic medium was evaluated are summarized in Tables 19 and 20. These data are presented to illustrate relative treatment efficiency of the trickling filter compared to the other biological processes operated at the Govalle Plant. More complete data observed for the various media are included in Appendix A.

TABLE 19

SUMMARY OF RESULTS

Run No.	BODT mg/l		BODF mg/l		TOCT mg/l		TOCF mg/l		CODT mg/l		CODF mg/l	
	Inf.	Eff.	Inf.	Eff.	Inf.	Eff.	Inf.	Eff.	Inf.	Eff.	Inf.	Eff.
1	68	20	40	9.5	33	21	28	15.5	143	69	104	56.6
2	82.5	32.5	44	14	58	26.5	33	20	214	94.5	105.5	63
3	96	45	54	27.5	61	34	32.5	20.5	309	135.5	146	96
4*	112	58	36	20	76	28.5	42.5	21.5	260.5	105.5	117.5	59.5

*In Run 4, undiluted influent is presented

TABLE 20

SUMMARY OF RESULTS

Run No.	Hydraulic Loading Rate MGAD	Organic Loading Rate lb BOD/1000 cu ft-day	SS mg/l		VSS mg/l	
			Inf.	Eff.	Inf.	Eff.
1	63	38	50.5	22	42	20.5
2	94.5	69	86.5	29	74.5	28
3	189	161	121.5	61	116	50.5
4	189 (N = 1:1)	142	131.5	35.5	119	30

Run No.	NH_3 mg/l		$NO_2 + NO_3$ mg/l		PO_4 mg/l		Temp. °C
	Inf.	Eff.	Inf.	Eff.	Inf.	Eff.	
1	21	18	--	1.71	26	24.5	24
2	25	21.5	--	.61	25	22.5	29.5
3	25	22	.13	.17	24	20	26
4*	16.5	14	.16	.35	23.5	19.5	23

*In Run 4, undiluted influent is present

The results in Tables 19 and 20 indicate an increase in the percent total BOD remaining as the hydraulic loading rate increased from 63 MGAD to 189 MGAD. Assessment of the performance based on the total substrate concentration is not very meaningful, since the pilot-plant trickling filter was not equipped with a final clarifier. Therefore, the performance was based on soluble substrate concentration. The percent soluble BOD, TOC, COD remaining increased with increasing hydraulic loading. The respective influent concentrations remained fairly constant for all runs. There was a slight reduction in the ammonia nitrogen and phosphate concentrations but very little nitrification occurred during the runs.

The relationship of soluble BOD removal with depth of medium is illustrated in Figure 34. The data plotted in Figure 34 represent average values of grab samples. The soluble BOD concentration of the composite samples are also included in Figure 34, for comparison purposes. The slopes of the curves decrease as the hydraulic loading increases indicating that the reduction in soluble BOD decreases with increasing hydraulic loading. The slopes of the curves varied from 0.027 to 0.012 for Runs 1 through 3. In general, the results of the composite samples agree well with the data observed for the grab samples. The introduction of recirculation into the operating procedure resulted in greater reduction of soluble BOD, TOC, and COD compared to results of Run 3 which had the same hydraulic loading rate. The percent reduction is based on the undiluted influent concentration and the results obtained at recirculation ratio 1:1 are those reported in Run 4.

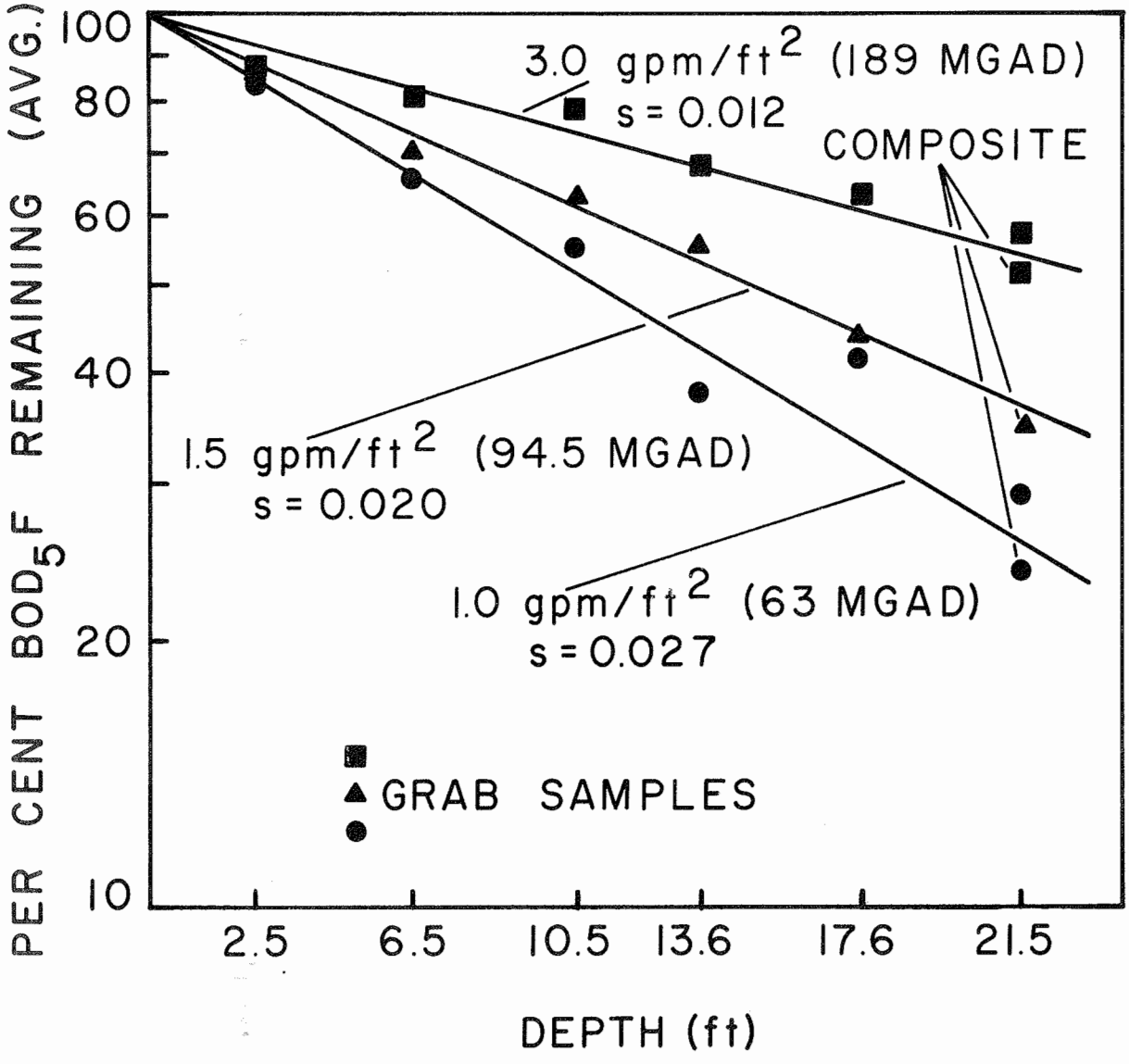

FIG.34 RELATIONSHIP BETWEEN BOD REMOVAL AND DEPTH (CORRUGATED PLASTIC MEDIUM)

The mathematical model which describes the overall performance of trickling filters in terms of soluble BOD remaining in the effluent may be written as Equation 3.

$$\frac{S_e}{S_o} = e^{-k_s' (20)^{\theta^{T-20}} A_v D/Q^n} \quad (3)$$

The performance in terms of BOD remaining ($\frac{S_e}{S_o}$) is a function of physical characteristics of the filter, such as specific surface area, A_v, and depth, D, of the filter medium, of the hydraulic loading rate, Q, of the wastewater temperature T, and of the constants, k_s', n, and θ.

The overall BOD removal rate constant k_s' is primarily a function of the characteristics of the influent wastewater; however, the hydraulic characteristics of the particular filter medium also affects the removal rate constant. The exponent n, which modifies the effect of the hydraulic loading on the mean detention time, is a function of the specific surface area and the configuration of the medium.

The data observed for the three media evaluated in this study are presented graphically in Figure 35 when the constant, n = 0.50. The performance of the two plastic media compare well and result in an overall BOD removal rate constant, $k_s' = 0.017$. However, the performance of the rock medium does not compare well with that of the plastic media.

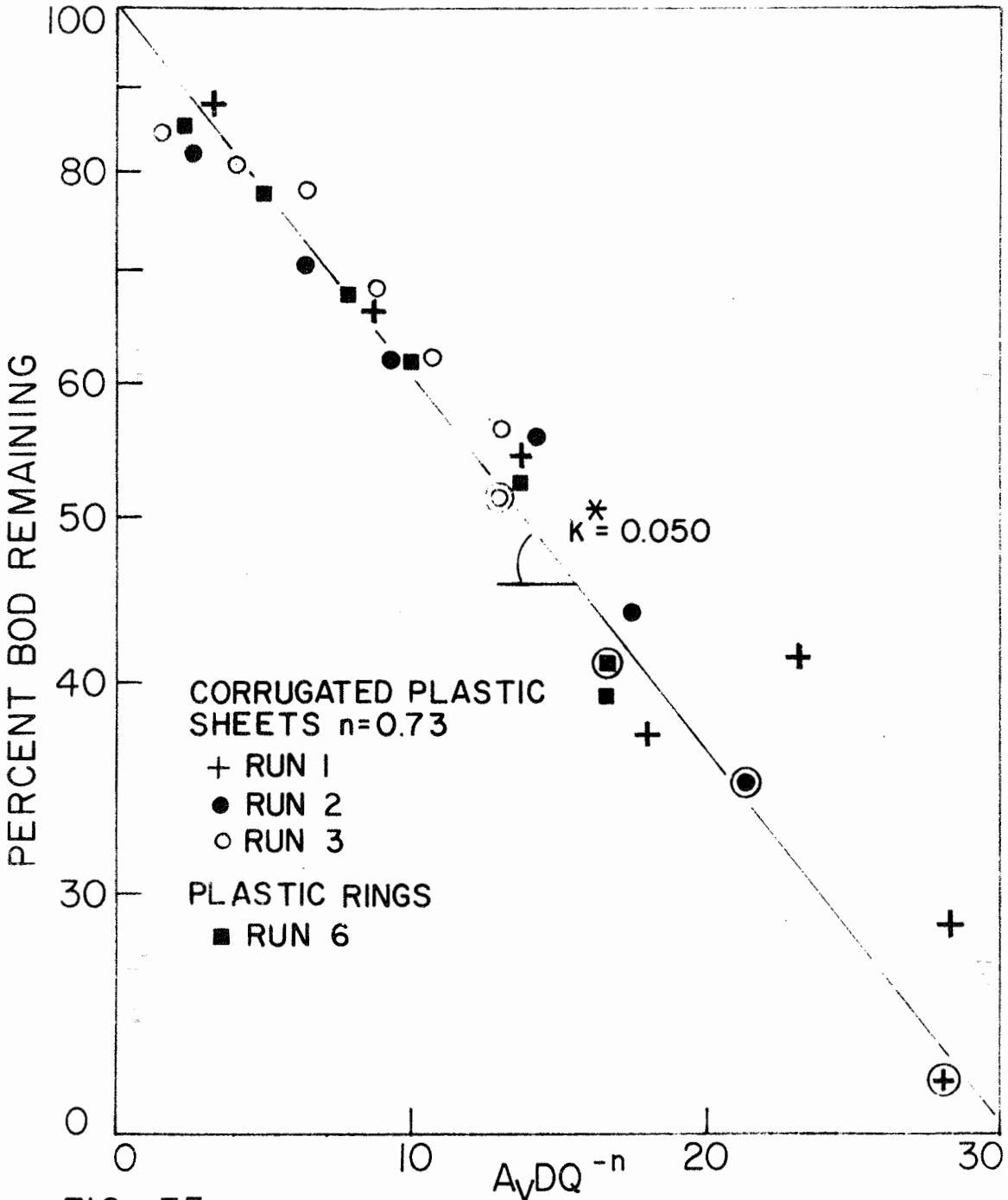

FIG. 35
DETERMINATION OF k^*
PILOT PLANT DATA: GOVALLE, AUSTIN

The data observed for the corrugated plastic, the polypropylene rings and the rock media indicate that the exponent n is 0.73, 0.70, and 0.36, respectively, for the specific medium. When these values of the exponents are used, the overall BOD removal rate constant, $k_s' = 0.050$ as indicated by the slope of the line which describes the data plotted in Figure 36. The exponent n, which modifies the effect that hydraulic loading is a function of the specific surface area and the configuration of the medium. The value of n for a particular medium should be determined from the relationship between mean detention time and hydraulic loading rate using tracer response techniques. Furthermore, the same value of n should be used in the model for any of the parameters which measure chemical characteristics.

D. Aerated Lagoons

The objective of the study was to evaluate the performance of full-scale aerated lagoons and to compare the performance with that observed in the laboratory-scale units operated under different conditions. In one of the lagoons, the power level was varied and the influence of this variable on the removal mechanism was studied. The theoretical advantages of soluble substrate-removal when aerated lagoons operate in series was also evaluated. Three tanks in series, two tanks in series and one tank were operated in parallel at a total detention time of six hours and twelve hours and the performance was compared. Tracer studies using radioactive sodium were also conducted in the lagoon of B Plant in order to evaluate the extent and degree of mixing in the basin.

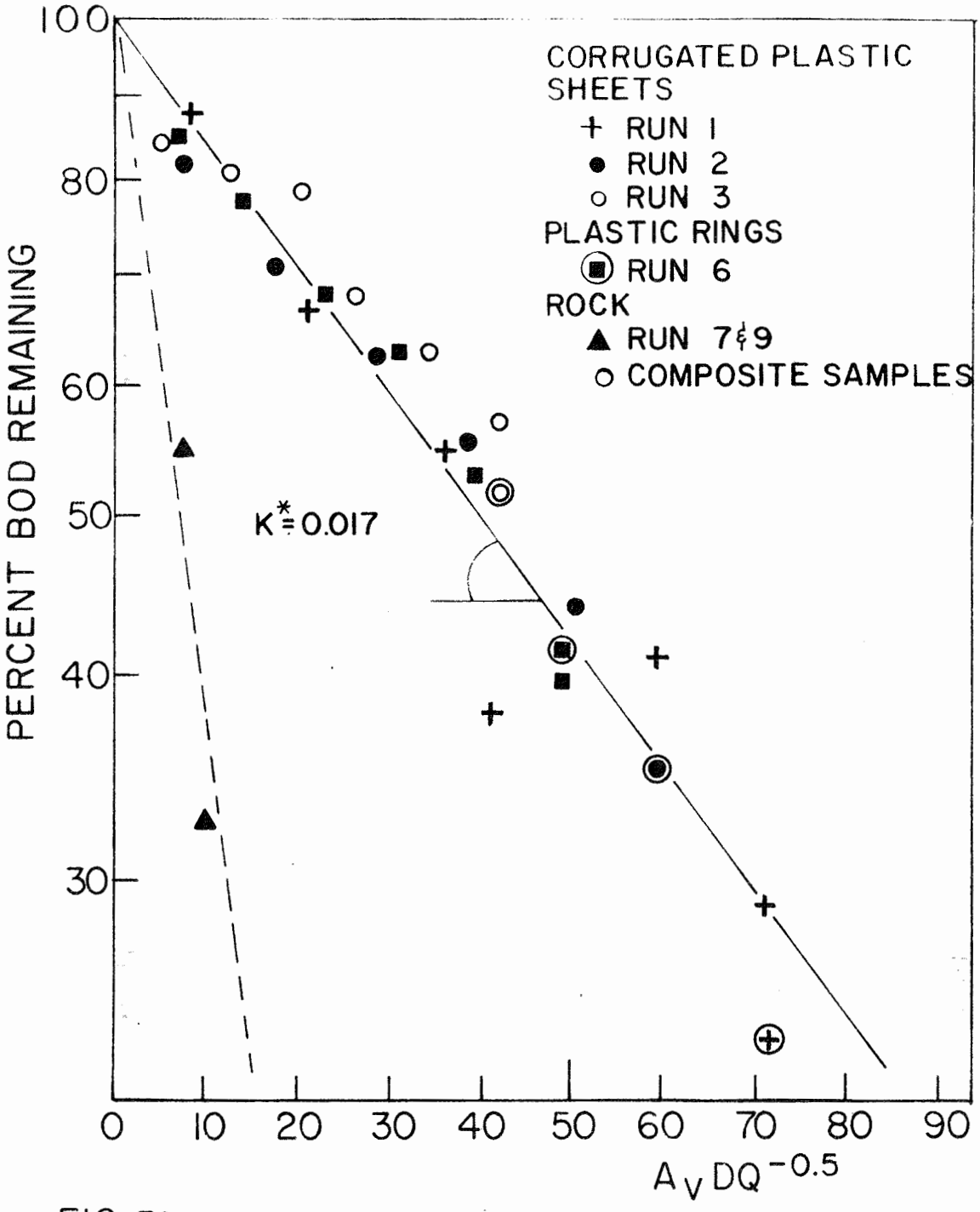

FIG. 36

DETERMINATION OF k^*

PILOT PLANT DATA, GOVALLE, AUSTIN

The performance of laboratory scale aerated lagoons and the comparison of single lagoon operation with the operation of lagoons in series was studied at the Govalle Sewage Treatment Plant and the results of this study were reported by Fleckseder and Malina (1970). This report is included in Appendix B. The results of the mixing study and a heat balance for the aerated lagoons was reported by Bishop and Malina (1971).

1. Experimental Program

The aerated lagoons at the Williamson Creek Plant were operated according to the schedule presented in Table 21. The aeration tank of the extended aeration plant, A Plant, was also operated as an aerated lagoon in one of the runs. The physical characteristics of the aerated lagoons were summarized in Table 3 on page 25.

The laboratory-scale experiments began in mid-April and ended in mid-July, 1970. Each laboratory tank was a plastic bucket equipped with diffuser stones. The tanks were located so that flow was by gravity from one tank to the next tank in the series. The general flow scheme is illustrated in Figure 37. The dimensions and other pertinent data are summarized in Table 22.

2. Experimental Results

The power levels in the aerated lagoon of A Plant and C Plant were 42.5 and 32 HP/million gallons respectively. These power levels indicate that these units were aerobic lagoons. The conversion of soluble

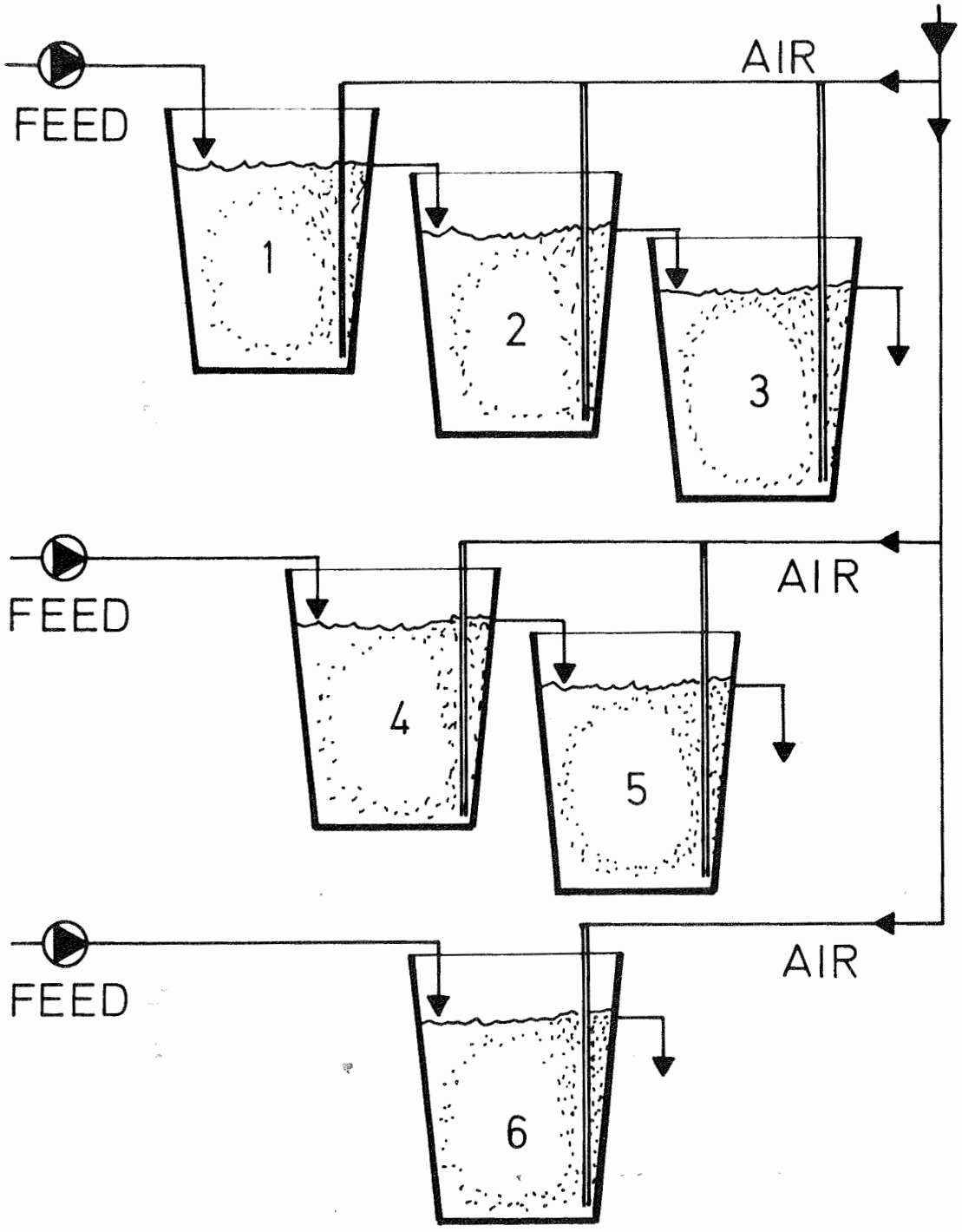

FIG.37
LABORATORY AERATED LAGOON SYSTEMS

TABLE 21

OPERATING CONDITIONS

Date	Detention Time (days)			Influent Characteristics			
	A Plant	B Plant	C Plant	TOCT mg/l	BODT mg/l	CODT mg/l	SS mg/l
10/07/69 - 10/30/69	---	8.0	0.93	81	125	308	139
11/03/69 - 11/13/69	---	8.0	0.93	78	124	301	164
11/16/69 - 11/30/69	---	7.8	0.97	74	113	280	159
12/01/69 - 12/22/69	---	6.9	0.83	64	91	235	161
01/04/70 - 01/29/70	---	6.9	0.70	83	129	320	131
02/01/70 - 02/26/70	---	6.3	0.56	66	96	245	142
03/02/70 - 03/11/70	---	6.1	0.42	49	62	172	100
04/13/70 - 04/30/70	2.6	6.6	----	57	78	206	124
06/24/70 - 06/30/70	---	3.35	----	69	103	261	142
07/01/70 - 07/16/70	---	3.45	----	68	100	254	138
07/19/70 - 08/16/70	---	3.63	----	70	104	263	143

TABLE 22

OPERATING PARAMETERS FOR LABORATORY STUDIES

RUN I	A			B		C
Tank	1	2	3	4	5	6
Volume (liters)	18	18	18	18	18	18
Detention time (hr)	4	4	4	5.75	5.75	12.85
Detention time, cumulative (hr)		12			11.5	12.85

RUN II	A			B		C
Tank	1	2	3	4	5	6
Volume (liters)	10	10	10	10	10	10
Detention time (hr)	2.2	2.2	2.2	3.2	3.2	8.3
Detention time, cumulative (hr)		6.6			6.4	8.3

RUN III	A		B		C		
Tank	1	2	3	4	5	6	7
Volume (liters)	10	10	10	10	18	18	18
Detention time (hr)	4	4	4	4	18	18	18
Detention time, cumulative (hr)	8		8		54		

material to biomass observed in the field lagoons was similar to that observed in the laboratory-scale experiments. The low values of effluent soluble substrate after one-day detention time gave rise to the laboratory-scale experiments. The data presented in Figure 38 represent the relationship between effluent substrate concentration and detention time. The difference between the laboratory data and the full-scale data can be attributed mainly to the differences in mixing intensity and temperature.

Dissolved oxygen measurements in the full-scale tanks of the experiments indicate concentrations of 1.0 to 3.0 mg/l. The total effluent substrate concentrations are very high in systems like C Plant; therefore, these units cannot be applied as a single treatment unit because there is no removal of suspended solids. When A Plant was operated as an aerobic lagoon, the suspended solids and total of filtered substrate concentrations measured after clarification were similar to that observed in the laboratory-scale plants with one-hour settling.

The lagoon in B Plant was operated as an aerobic-anaerobic lagoon at a power level of 11 and 18 HP/million gallons. The data for the B Plant are summarized in Figure 39. The increase in the effluent BODT with a decrease in temperature is evident. An algal population of varying size was present in the aerated lagoon in B Plant throughout the study even at hydraulic detention times of about 3.5 days. When the mixing studies were undertaken, it was observed that this algal population was distributed throughout the tank. The algal contribution to the effluent suspended solids concentration could not be estimated.

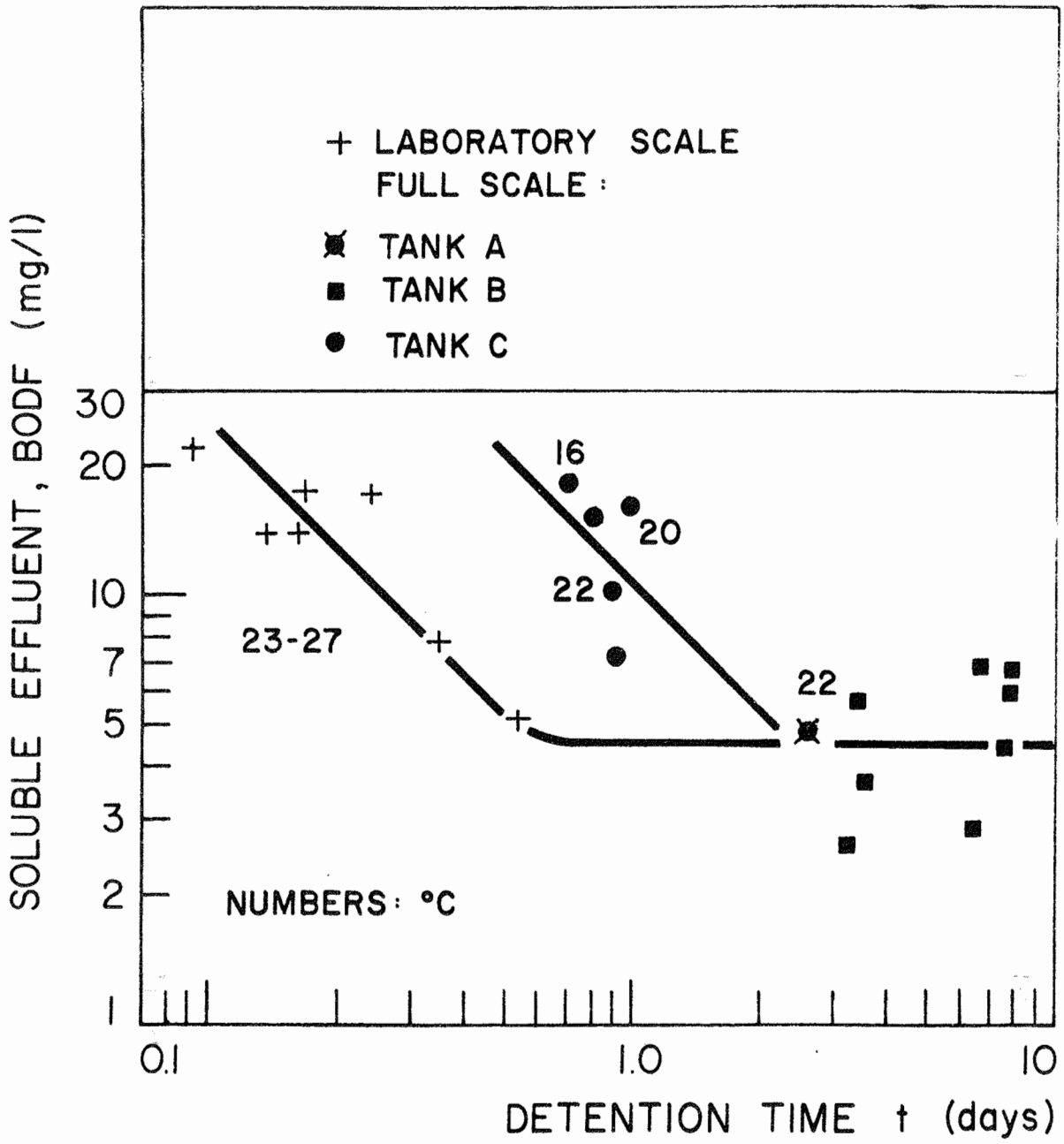

FIG. 38

SOLUBLE EFFLUENT - BOD VERSUS DETENTION TIME

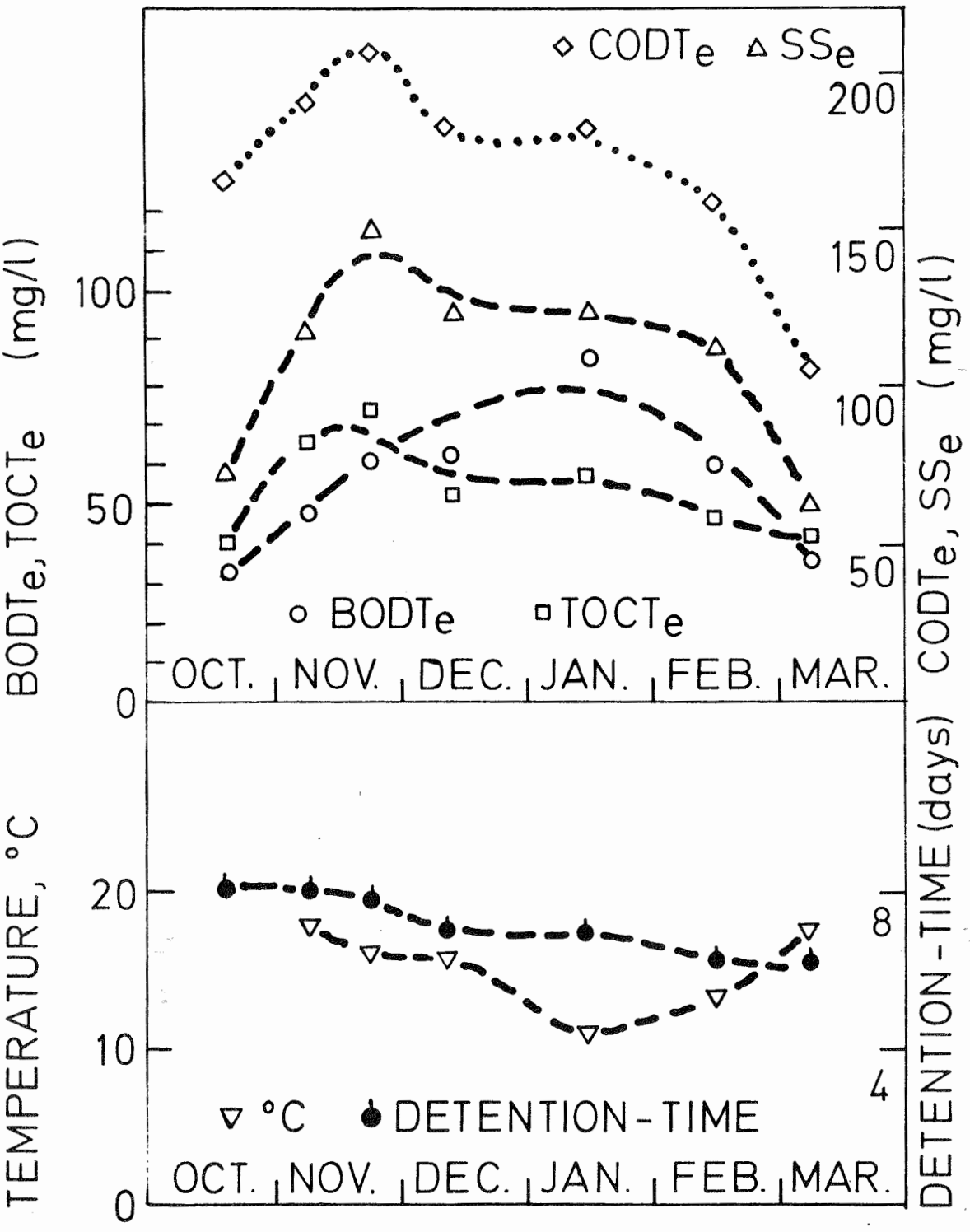

FIG.39

EFFLUENT DATA B PLANT

Dissolved oxygen measurements throughout the entire tank to within two inches of the bottom of the tank indicate concentrations of dissolved oxygen between 4.5 mg/l and 7.0 mg/l. All measurements were done during the daylight hours. No sludge-deposits could be located; however, the tank was in operation for a relatively short time (less than one year). Nitrification, determined by ammonia-reduction, was measured at all times; the detention time in C Plant was too short to have measurable nitrification.

The detention time in the laboratory-scale units ranged from 0.09 to 0.54 days (2.2 hours to 12.6 hours) and in the full-scale lagoons from 0.7 to 1.0 days and from three to eight days for the C Plant and B Plant, respectively. The data in Figure 37 indicate that the laboratory results seem to cluster at one part of the graph while the full-scale data tend to cluster in another area of the graph. The line through the two sets of data have a slope of -1 which indicates that the effluent BOD concentration is proportional to reciprocal time (1/t). The detention time does not seem to affect the concentration of BOD in the effluent beyond the detention time of 2.5 days. With these long detention times the effluent BOD concentration was relatively constant and the concentration of soluble BOD in the effluent ranged from three to seven mg/l. The data indicate that the rate constant for the removal of soluble BOD in the full-scale process was less than that observed for the laboratory-scale experiments.

It is doubtful that the effluent soluble BOD concentration could ever be reduced to below four to five mg/l even at longer detention times. The intersection of the line describing the laboratory data and a horizontal line at four to five mg/l effluent soluble BOD occurs at a detention time of about 0.6 days. This detention time is approximately equal to that at which the maximum active mixed suspended solids was reported in the laboratory-scale lagoons. Increasing the detention time beyond the point at which the maximum active mixed liquor suspended solids concentration occurs will not reduce the effluent BOD concentration, since at detention times beyond this point, the active mixed liquids suspended solids concentration decreases and therefore less soluble BOD will be removed. In full-scale systems, this breakpoint can be assumed to occur at a detention time of about 2.0 days. The difference in the rate constants can be attributed to different characteristics of the wastewater and the different degrees of mixing. The field-scale ponds were evaluated at the Williamson Creek Plant at which plant the influent concentration of BOD was lower than that at the Govalle Wastewater Treatment Plant where the laboratory-scale tests were carried out. Therefore, the minimum possible soluble BOD concentration in an effluent can be achieved in a completely mixed aerated lagoon which is operated at a detention time of about two days.

The effluent concentration of total BOD is markedly affected by the concentration of suspended solids in the effluent. The relationship between the effluent suspended solids and the effluent total BOD is presented in Figure 40. These data were recorded for the aerated lagoons located at

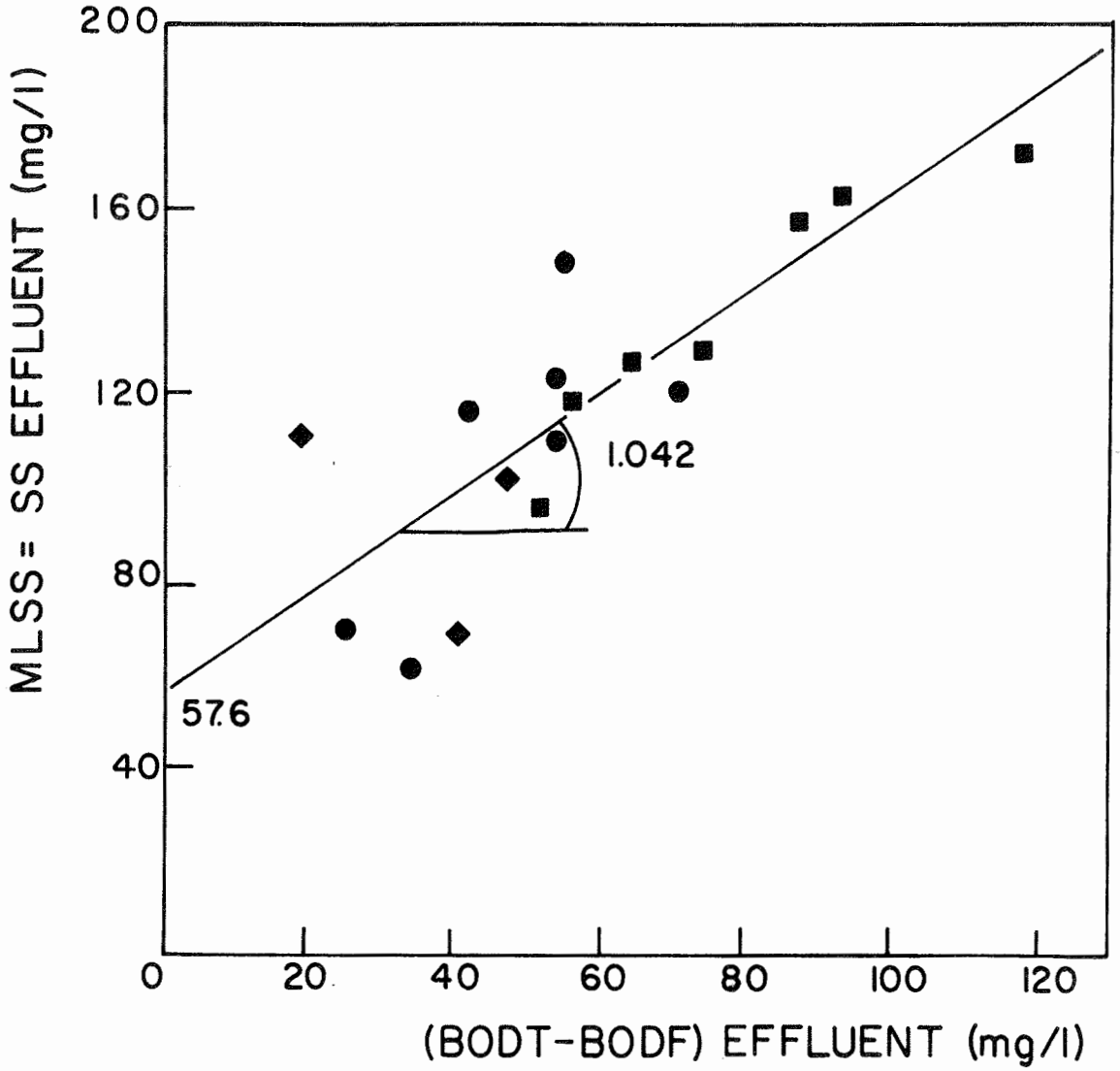

FIG.40

EFFLUENT SUSPENDED SOLIDS VERSUS EFFLUENT BODT MINUS EFFLUENT BODF

Williamson Creek Wastewater Treatment Plant. The average ratio of effluent total BOD to effluent suspended solids is about 0.55. The extended aeration plant at the Williamson Creek Treatment Plant was operated as an aerated lagoon with a detention time of 2.6 days for a short period of time. The effluent suspended solids concentration of the aerated lagoon and the final clarifier were under 25 mg/l and 32 mg/l, respectively. The BOD concentration in the effluent of the clarifier was about ten mg/l. The concentration of total BOD in the effluent from the aerated lagoon operated at detention times of three to eight days and ranged between 20 and 70 mg/l with an average value of 40 mg/l. However, at a one-day detention time, the average total effluent BOD varied from 50 to 120 mg/l. It should be pointed out at this time that the power level and therefore the mixing level in the aerated lagoon was very high and almost no settling of any of the solids was noticed.

The relationship between the suspended solids concentrations in the aerated lagoons and the operating power levels is illustrated in Figure 41. No stratification was obvious at any of the power levels applied. This experiment was run during one week. The single value at a power level of 10.3 HP/million gallons was measured after one week of operation at this power level. The increase in the suspended solids concentration can be attributed to the influent suspended solids. The results of tracer studies in the same tank are presented in Figure 42 and indicate that this tank is only well mixed at a power level greater than 30 HP/million gallons. These observations indicate that the non-existence of a suspended solids stratification is no proof of a well mixed tank.

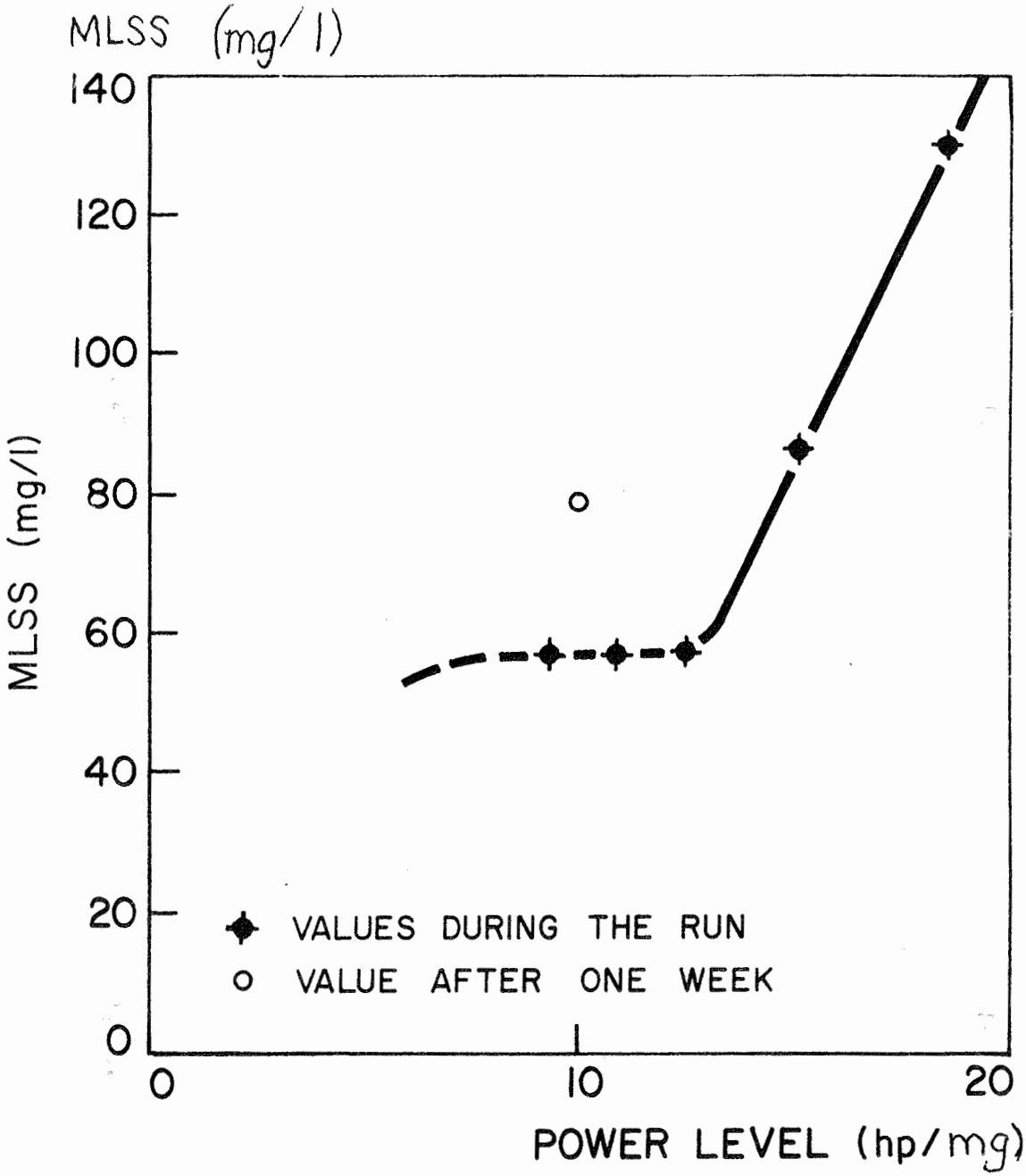

FIG. 41

POWER LEVEL VERSUS MLSS

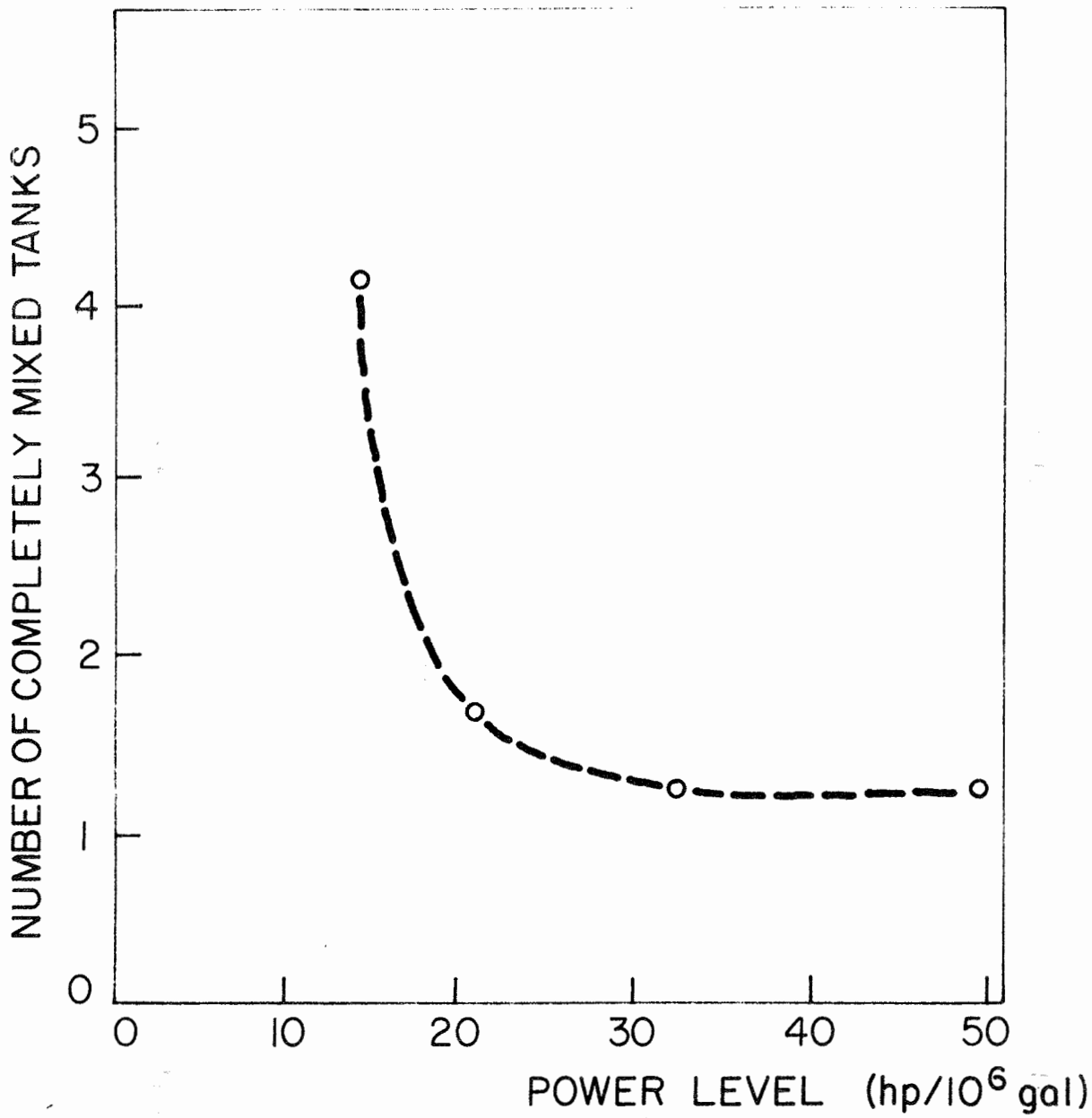

FIG. 42

POWER LEVEL AND DEGREE OF MIXING

The quality of the effluent of a lagoon that is not completely mixed and in which a portion of settleable of the suspended solids are permitted to settle depends to a large extent on the energy level, the detention time, the distance of the aerators from the effluent, as well as the type, size, and construction of the effluent weirs. In some of these incompletely mixed tanks, the concentration of algae which develop in the basin can also be a significant factor which must be considered in the design. As the power level decreases the concentration of mixed liquor suspended solids must also decrease and the concentration of algae will tend to increase.

A tube settler was installed in the lagoon in B Plant. The effluent quality remained unchanged. The configuration of the overflow weir of the lagoon was such that it was almost impossible to properly install the tube settler and maintain a uniform take-off of the effluent. Complete dewatering of the lagoons would have been necessary in order to properly install the tube settler. The effluent weir was only about four feet long, and the flow through the settler had to flow through a rather small cross-sectional area. Tube settlers provided an area of about 50 square feet and under normal operating conditions, the overflow rate was approximately 5400 gallons per square foot per day, which was considerably higher than would be desired for operation of the tube settler. Water waves caused by wind action and the action of the aerator extended caused a carryover of suspended solids in the effluent. The installation of additional baffles on the tube settler could have reduced some of the wave action and possibly improved the conditions under which the tube settler operates.

The data observed during the laboratory studies indicate that there is no advantage to operating a series of aerobic lagoons instead of a single lagoon treating municipal wastewater when the detention time and environmental conditions are the same.

E. Waste Stabilization Ponds

The objectives of this study included the verification of an existing design relationship improvement in the criteria for waste stabilization ponds and an evaluation of the influence of the location of anaerobic zones on the quality of pond effluents. An experimental facility consisting of three laboratory-scale and pilot-scale pond systems were installed and operated at the Govalle Wastewater Treatment Plant. The physical characteristics of the pond systems are included in Table 2 on page 22.

Specific data relating to the performance of the ponds has been reported spearately and these publications are included in Appendix B. The effects of the location of anaerobic zone on the pond performance was reported by Aguirre and Gloyna (1971). The effects of physical parameters including temperature, pH, dissolved oxygen, and suspended solids was reported by Chieu and Gloyna (1970). A determination of the BOD rate constant at the various locations in the pond was developed by Chiang and Gloyna (1970). Davis and Gloyna (1970) discussed the reduction in coliform organisms in the pond systems.

1. Experimental Program

The performance of the pilot-scale waste stabilization pond

systems was evaluated from June 1969 through August 1970. The schedule of operation of these systems is presented in Table 23.

TABLE 23

OPERATING SCHEDULE
WASTE STABILIZATION POND SYSTEMS

Run	Date	Flow Rate gpm
1	June, 1969 - January, 1970	10
2	February, 1970 - June, 1970	20
3	July, 1970 - August, 1970	35

Other operational characteristics for the pilot-scale pond systems are summarized in Table 24.

Laboratory-scale pond systems consisting of aquaria were operated from November 1968 through July 1970. The hydraulic loadings to the ponds were varied from 67 percent to 400 percent of the design capacity.

2. Results of Experimental Runs

The average quality of the effluent of the various pilot-scale pond systems is summarized in Tables 25, 26, and 27, respectively, for the three runs. The ponds were designed to provide 90 percent removal of BOD using a modification of the Hermann-Gloyna relationship which is presented as Equation 4.

$$\frac{QS_o\ (8.34)}{A} = \frac{544\ d}{1.085^{(35 - T)}\ (t_o)} \qquad (4)$$

TABLE 24.

OPERATIONAL CHARACTERISTICS OF THE
PILOT-SCALE POND SYSTEMS

Pond System	(gpm)	Flow Rate (ft^3/d)	Detention (days)	Surface Loading (lbs BOD/a - d) BOD_u	Surface Loading (lbs BOD/a - d) BOD_5	Volumetric Loading (lbs BOD/1000 ft^3 - d) BOD_u	Volumetric Loading (lbs BOD/1000 ft^3 - d) BOD_5
I							
Anaerobic	10	1925	4.7	460	302	3.1	2.0
	20	3850	2.3	920	604	6.1	4.0
	35	6738	1.3	1610	1057	10.8	7.1
Facultative	10	1925	61.0	19*	12*	0.9*	0.06*
	20	3850	30.5	37	24	0.18	0.12
	35	6738	17.4	65	43	0.32	0.21
II							
Facultative (w/Trench)	10	1925	65.6	47	30	0.22	0.14
	20	3850	32.8	94	60	0.44	0.28
	35	6738	18.7	165	105	0.76	0.50
I, II							
Maturation	10	1925	9.4	20**	13**	0.15**	0.10**
	20	3850	4.7	39	26	0.30	0.20
	35	6738	2.7	67	46	0.53	0.35
III							
Facultative	10	1925	65.6	47	30	0.22	0.14
	20	3850	32.8	94	60	0.44	0.28
	50	9625	13.1	235	150	1.10	0.72
Maturation	10	1925	9.4	20**	13**	0.15**	0.10**
	20	3850	4.7	39	26	0.30	0.20
	50	9625	1.9	99	65	0.75	0.50

* Load to Facultative Pond I based on 40% of load received by Anaerobic Pond I
**Load to Maturation Ponds I, II, and III based on 10% of influent raw sewage

TABLE 25

AVERAGE QUALITY OF PILOT-SCALE POND EFFLUENTS
(Load: 47 lb BOD_u/a - d; June, 1969 - January 1970)

Pond System	BODT (mg/l)	BODF (mg/l)	CODT (mg/l)	CODF (mg/l)	TOCT (mg/l)	TOCT (mg/l)	pH	Suspended Solids (mg/l)
Raw Sewage	155	---	403	---	---	---	7.5	164
Anaerobic I	104	56	197	103	66	42	7.2	88
Facultative I	22	13	130	79	41	29	8.9	70
Maturation I	35	10	131	70	48	28	8.7	55
Facultative II	39	13	167	81	60	30	9.0	104
Maturation II	52	10	168	70	57	30	9.4	83
Facultative III	34	10	160	72	59	29	8.8	99
Maturation III	36	10	158	75	60	29	8.9	92

TABLE 26

AVERAGE QUALITY OF PILOT-SCALE POND EFFLUENTS
(Load: 94 lb BOD_u/a - d; February - June, 1970)

Pond System	BODT (mg/l)	BODF (mg/l)	CODT (mg/l)	CODF (mg/l)	TOCT (mg/l)	TOCF (mg/l)	pH	Suspended Solids (mg/l)
Raw Sewage	130	---	362	---	---	---	7.4	152
Anaerobic I	49	19	160	61	39	27	7.4	48
Facultative I	30	2	147	48	47	27	8.7	67
Maturation I	22	2	135	55	48	26	9.0	85
Facultative II	22	4	164	63	51	29	8.2	82
Maturation II	23	2	156	60	43	28	8.5	86
Facultative III	29	2	162	61	51	27	8.6	79
Maturation III	32	2	188	56	59	25	9.0	101

TABLE 27

AVERAGE QUALITY OF PILOT-SCALE POND EFFLUENTS
(Load: 165 lb BOD_u/a - d; June - August, 1970)

Pond System	BODT (mg/l)	BODF (mg/l)	CODT (mg/l)	CODF (mg/l)	TOCT (mg/l)	TOCF (mg/l)	pH	Suspended Solids (mg/l)
Raw Sewage	143	---	362	---	---	---	7.5	209
Anaerobic I	61	40	192	70	34	25	7.3	76
Facultative I	41	9	205	73	56	27	8.9	119
Maturation I	43	3	240	74	52	26	9.2	118
Facultative II	55	7	280	66	69	25	8.2	162
Maturation II	49	7	246	64	64	24	8.8	161
Facultative III	68	7	282	65	76	36	8.6	201
Maturation III	54	5	243	68	53	25	8.8	145

in which

$\frac{QS_o(8.34)}{A}$	=	Areal BOD loading (lb/acre-day)
A	=	Surface area of the pond (acres)
d	=	depth of the pond (feet)
T	=	operating temperature, minimum monthly average (^oC)
t_o	=	optimum detention time for 90 percent BOD removal at 35^oC (days)
0.326	=	conversion from acre-feet to gallons
8.34	=	conversion of gallons to pounds

The results of the pilot-scale studies are represented as the relationship between effluent BOD concentration and areal loading in Figure 43. The effluent quality expressed as total BOD is highly dependent upon the areal loading rate. However, the soluble BOD of the effluent is independent of the loading rate within the range of loading conditions used in this study. Two theoretical effluent quality curves for facultative ponds are also included in Figure 43. These curves are based on calculated values using Equation 4 for operating temperatures of 10^o and 20^oC, respectively. The influent BOD (S_o) was assumed to be 200 mg/l and the relationship of optimum detention time, BOD removal and the rate constant. The two theoretical curves follow a similar pattern to the other data and indicate that the constants developed are in the useful range. However, for design purposes, these constants should not be over-emphasized, since the experimental data observed at pond temperatures of 20^oC and above resemble the performance

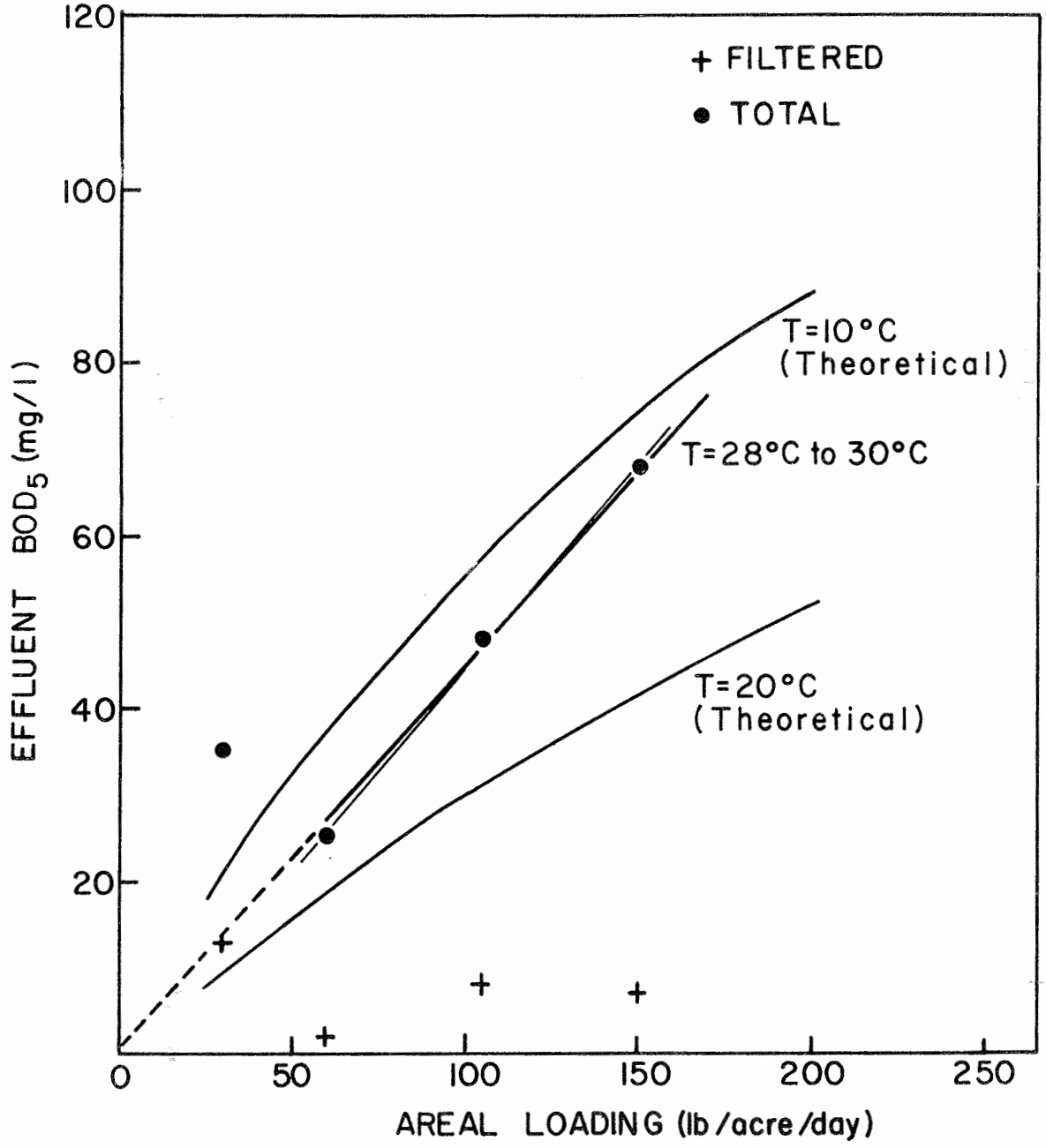

FIG. 43
EXPERIMENTAL AND THEORETICAL EFFLUENT QUALITY AS A FUNCTION OF AREAL LOADING

of a pond operating at 10°C. This information indicates that the rate constant in the pilot-scale stabilization pond was much lower than the assumed theoretical value. The calculated rate constants assuming completely mixed conditions are shown in Figure 44. The rate constant calculated from the straight line in Figure 44 indicates that k = 0.11. However, the dashed curve drawn through the points in Figure 44 indicates that the rate constant might be a function of the loading and is similar to those reported for other biological waste treatment processes.

The biodegradation rate constant (k_{10}) for the various ponds in the three systems are summarized in Table 28. These results indicate that the degradation rate constant of the filtered pond liquid decreased as the waste progressed through each pond system. The high rate (k_{10} = 0.27) observed for anaerobic pond of System I was reduced to one-half (k_{10} = 0.13) by the time the liquid had exited from the maturation pond. Although similar reductions were noted in the other two systems, the anaerobic pretreatment pond has a much more noticeable influence on the degradability of the soluble organic fraction of the raw wastewater. The soluble fraction of the raw wastewater had an approximate k_{10} of 0.15, which was considerably less than that found in the anaerobic pond and very similar to that of the maturation ponds. Therefore, the feed-back of anaerobic fermentation products contributed a significant fraction of readily degradable material to the pond liquid. This phenomenon was also observed to a lesser extent, in the facultative ponds where the feed-back of material from the anaerobic zone results in a degradation rate constant of about 0.19.

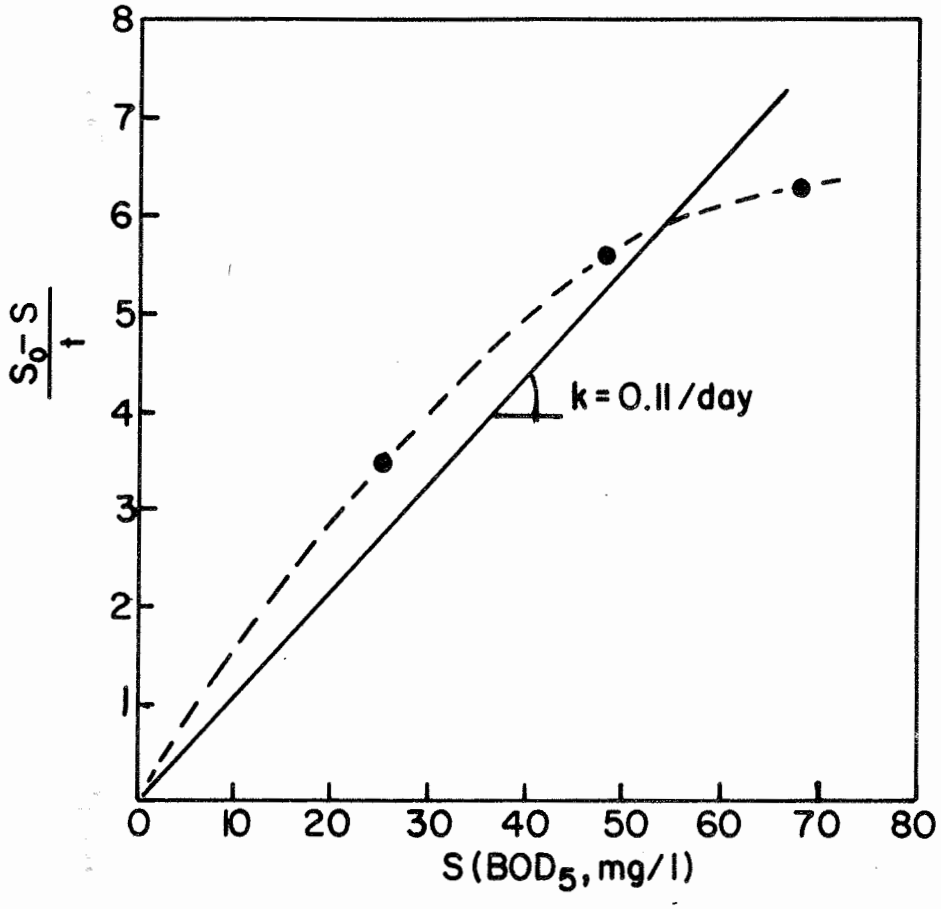

FIG. 44

CALCULATED BIODEGRADATION RATE CONSTANT, ASSUMING COMPLETELY MIXED CONDITIONS

TABLE 28

BIODEGRADATION RATES OF PILOT-SCALE
WASTE STABILIZATION PONDS

Sample	BOD_5 (mg/l)	k_{10} (d^{-1})
Raw Sewage		
Total	262	0.217
Filtered	114	0.148
Anaerobic I	57*	0.269
Facultative I	12	0.205
Maturation I	10	0.128
Facultative II	12	0.194
Maturation II	10	0.163
Facultative III	9	0.184
Maturation III	7	0.134

*BOD_5 of pond effluents on filtered samples only

The results of these investigations indicate that the three pond systems consisting of a separate anaerobic pond with a short detention time, followed by a facultative pond from which the effluent flowed into a smaller maturation pond produced the best effluent. It can be assumed that the three pond system provides a better effluent since approximately 50 percent of the BOD and organic carbon in the wastewater was removed in the anaerobic pond. Therefore, the BOD load to the facultative pond was reduced and the quantity of algal cells produced in the facultative pond was also reduced, resulting in an effluent of better quality.

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