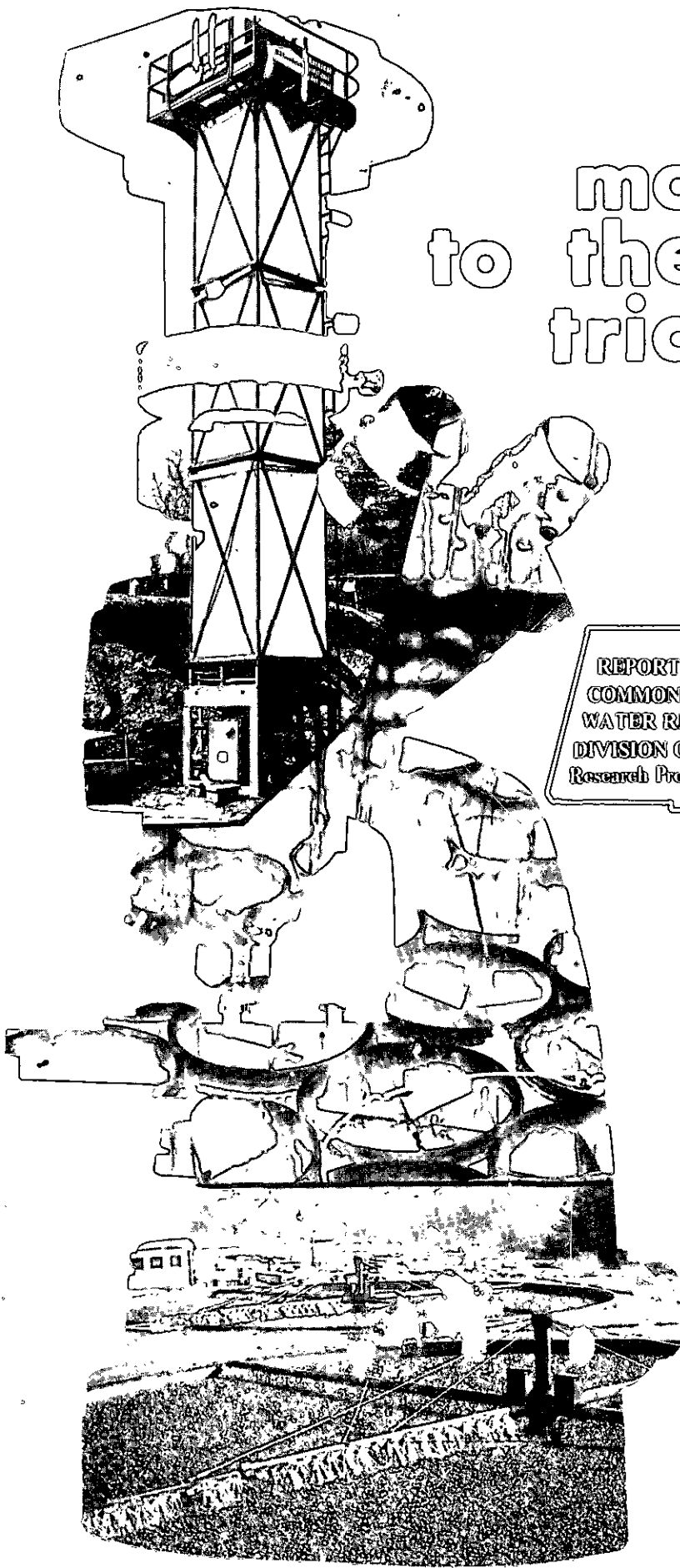# modifications to the high-rate trickling filter process

REPORT TO THE
COMMONWEALTH OF MASSACHUSETTS
WATER RESOURCES COMMISSION
DIVISION OF WATER POLLUTION CONTROL
Research Project 68-1

OCTOBER 1970

M&E

Report to

The Commonwealth of Massachusetts

Water Resources Commission

Division of Water Pollution Control

Modifications to High-Rate Trickling Filter Process

October 1970

Research Project 68-1

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November 20, 1970

Commonwealth of Massachusetts
Division of Water Pollution Control
100 Cambridge Street
Boston, Massachusetts 02102

Attention: Mr. Thomas C. McMahon
             Director, Division of Water Pollution Control

Subject: Research Project 68-1

Gentlemen:

In accordance with the contract agreement between the Commonwealth of Massachusetts, Division of Water Pollution Control and Metcalf & Eddy, Inc. dated June 27, 1968 and August 14, 1969, we have conducted studies at the Marlborough, Massachusetts wastewater treatment plant upon "Modification to the High-Rate Trickling Filter Process." This report contains the results of our studies.

Very truly yours,

METCALF & EDDY, INC.

David A. Duncan
Vice President

SUMMARY OF CONCLUSION

SUMMARY OF CONCLUSIONS

A summary of the findings and conclusions of the accompanying technical report are summarized herewith for convenience.

1. Rock trickling filters will produce an effluent of about 45 mg/L* BOD** and 55 mg/L suspended solids at loadings of 45 lb BOD/1,000 cu ft (cubic feet).

2. Plastic trickling filters will produce an effluent of about 35 mg/L BOD and 20 mg/L suspended solids at loadings of 45 lb BOD/1,000 cu ft and 45 mg/L BOD at loadings of 70 lb BOD/1,000 cu ft.

3. Lime treatment of rock trickling filter effluent will produce an effluent quality of about 40 mg/L BOD and 20 mg/L suspended solids at a pH of 10.8.

4. Lime treatment will not upgrade trickling filter effluent to produce an effluent of quality comparable to a well-operated activated-sludge unit.

5. Trickling filters may be considered as an alternate to the activated-sludge process if the filters are to be used as roughing filters or if the filters are to be followed by certain nutrient removal processes.

6. When trickling filter effluents were treated with lime, the lowest phosphorus concentrations were obtained at pH levels between 9.9 and 11.0.

7. The total phosphorus in the lime-treated effluent at pH 10.8 was less than 1.0 mg/L and the inorganic phosphorus concentration was reduced to less than 0.5 mg/L as P.

8. "Luxury uptake" of phosphorus in the activated-sludge process was nonexistent, and phosphorus removals were about 20 percent when no chemical was used.

9. Alum was added to the last pass of the aeration tank for optimum phosphorus removal.

*Milligrams per liter.
**Biochemical oxygen demand.

10. The addition of alum improved the settleability of the mixed liquor and improved the SVI (sludge volume index).

11. Alum to inorganic phosphorus ratios of 18 to 1 attained an average effluent quality of about 10 mg/L BOD, 20 mg/L suspended solids, 1.5 mg/L total phosphorus and 1 mg/L inorganic phosphorus.

12. Lime sludge may be dewatered for ultimate disposal or reclamation by the use of centrifuges.

13. Mixtures of waste-activated sludge containing alum and primary sludge may be dewatered by vacuum filtration.

14. The ammonia stripping process is not practical in the colder climates because of temperature limitations. Severe scaling and air pollution problems make this process unattractive at any location.

15. Nitrogen removal through the pathway of nitrification-denitrification seems to offer the most hope at this time.

16. Settled trickling filter effluent, with BOD in the range of 20 to 70 mg/L, proved to be a satisfactory feed for the nitrification unit.

17. The nitrification unit produced an effluent with an average of 10 mg/L BOD, and 30 mg/L suspended solids. Ammonia nitrogen was normally less than 0.3 mg/L.

18. Factors which affect the time to attain complete nitrification are feed ammonia concentration, pH, temperature, and MLVSS (mixed liquor volatile suspended solids) concentration.

19. The ammonia concentration did not inhibit nitrification in concentrations of less than 60 mg/L.

20. pH did affect the rate of nitrification. Optimum pH was found to be 8.4.

21. Temperature did affect the rate of nitrification. The rate increased through the range of 5 deg C to 30 deg C, in reasonable agreement with the van't Hoff-Arrhenius law.

22. The time required for nitrification is directly proportional to the amount of nitrifiers present in the system.

23. Instantaneous increases in BOD concentration from 5 to 110 mg/L did not affect the rate of nitrification. However, it would be expected that a change in the average BOD concentration of the feed would affect that percentage of MLVSS which is composed of nitrifiers, and as a result would affect the time to achieve complete nitrification.

24. Complete nitrification can be obtained in 1.8 hours contact with a MLVSS of 1,450 mg/L, average BOD feed of 50 mg/L, pH of 8.4, 20 deg C temperature, and an ammonia feed concentration of 20 mg/L. If the temperature were reduced to 10 deg C and the pH reduced to 7.0, the required time would increase to 9.45 hours.

25. Upflow contact units, both rock and plastic, were found to be unreliable denitrification systems.

26. In general, more work needs to be performed on the factors of denitrification and the control of the methanol feed.

REPORT

CHAPTER 1

INTRODUCTION

Trickling Filters

One of the common methods of wastewater treatment employed over the course of the years, particularly in the smaller cities and towns, has been to employ trickling filter systems using native rock as the contact material. As a result, a considerable capital investment has been made on the part of the communities involved. In Massachusetts alone, there are over 30 trickling filter installations, and prospects are that more will be built if sufficient evidence can be accumulated to demonstrate that they are compatible with projected requirements for additional higher degrees of treatment for a reasonable number of years in the future.

Although trickling filters have been known as the "work horses" of the treatment field, they have been shown to suffer from low temperatures in northern climates. As a result, although performance may be acceptable during the summer and fall, they may fail to meet the desired standards during the winter and spring when wastewater temperatures are lowest. In addition, trickling filters may not offer an opportunity for simple cationic removal of phosphorus such as the activated-sludge process does.

Because of the great investment in trickling filter systems in Massachusetts and in the nation, the growing need for higher quality effluents, and a need for algae nutrient (nitrogen and phosphorus) removal in many locations, it was conceived that the quality of trickling filter effluents might be upgraded to a satisfactory level by treatment of filter effluent with lime prior to final settling. Hopefully, such treatment would yield effluents with acceptable levels of BOD, suspended solids, and phosphorus with nominal additional capital costs. Furthermore, it was argued that lime treatment would raise the pH of the wastewater to a level where ammonia nitrogen could be removed by air stripping, if desired. The primary purpose of the investigations was to study these possibilities. Marlborough, Massachusetts was selected as the site because of location, facilities, and need.

Activated Sludge Process

Because of the many contingencies involved in the trickling filter studies relative to their outcome and a considered need for information

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about nutrient removal, both nitrogen and phosphorus, it was deemed advisable to conduct investigations, in parallel, on an alternative method of wastewater treatment. The activated-sludge process offered the greatest potential, so it was chosen as the competetive process. During the first year, phosphorus removal was incorporated as an integral part of the process so that direct comparison could be made with the trickling filter—lime precipitation system.

Second Phase of Study

At the close of the first year of investigations, it became quite apparent that a rather drastic realignment of the studies should be made to obtain specific answers to problems at Marlborough which would have statewide application, as well as to further define the future role of trickling filters.

The results of the first year clearly demonstrated that trickling filter performance at Marlborough was abnormal. Since it was not clear whether this condition was due to a deficiency of the filters or to some other abnormality, a decision was made to construct a new pilot rock filter to provide trickling filter effluent for all nutrient removal studies. Since the question of whether to use rock or plastic media in the filter presented itself, a decision was made to construct both types and operate them in parallel.

Because of problems encountered in stripping ammonia from lime-treated wastes, a decision was made to explore nitrogen removal via the nitrification—denitrification route. Since such a system was being studied in at least two other locations in the United States following activated-sludge treatment, we chose to conduct similar studies employing the effluent from the trickling filter system as feed to the nitrification unit.

Laboratory Support

Essentially all analytical testing required for the investigations was conducted in a mobile laboratory (Figure 1) stationed at Marlborough.

The analytical procedures used were in accordance with Standard Methods,* except in a few instances where changes were made to improve the sensitivity of the tests or to reduce time requirements. All BOD tests were run employing a 7-day incubation period to equalize the daily work load and to eliminate the need for Saturday and Sunday attention.

*Standard Methods for the Examination of Water and Wastewater, 12th Edition, 1965.

1-2

Personnel

The studies were conducted by two full-time employees of Metcalf & Eddy, Inc., working on a five-day per week basis. They were given considerable support during the regular work week by City of Marlborough, Department of Public Works personnel. On Saturdays and Sundays, the Public Works personnel assumed complete responsibility, with Metcalf & Eddy personnel on call.

Equipment

Much of the equipment used was purchased as prefabricated units. Site preparation and installation were accomplished mainly by City of Marlborough personnel.

Duration of Studies

The field studies were initiated during November 1968 and terminated in May 1970.
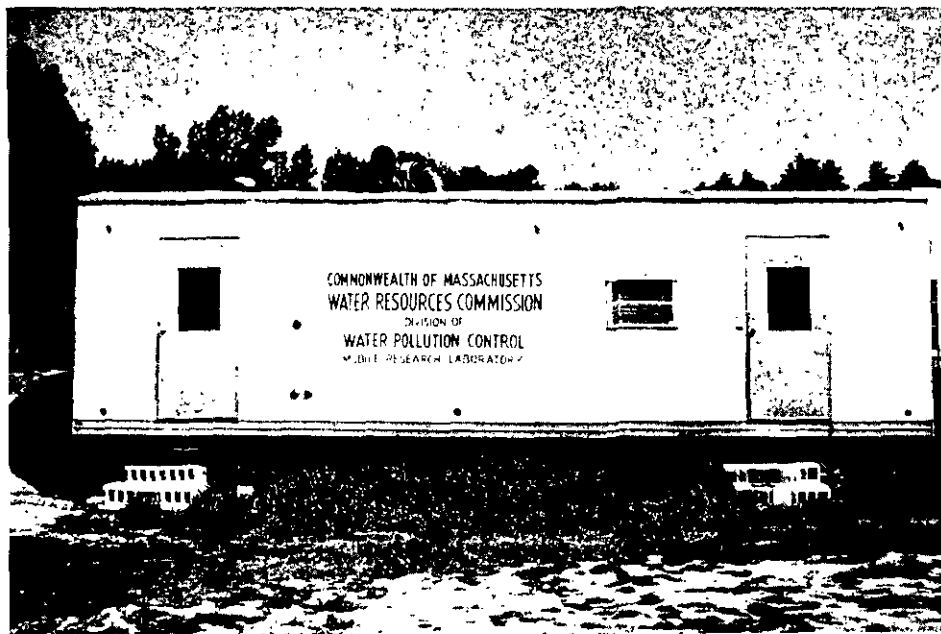
Fig. 1   View of Laboratory Trailer

CHAPTER 2

TRICKLING FILTER STUDIES

Preliminary Considerations

The major purpose of the trickling filter studies was to determine whether the quality of the effluent could be upgraded to a degree comparable to that expected from activated-sludge treatment plants, under both summer and winter conditions.

Preliminary studies conducted by Metcalf & Eddy, Inc., for the City of Marlborough during the summer of 1967 had demonstrated that final effluent quality could be markedly improved by coagulation of the TFE (trickling filter effluent) with alum or lime prior to final settling. Such treatment also removed a large part of the phosphorus, an important consideration at Marlborough because of the ponds downstream through which the effluent flows on its way to the Sudbury River. Since there was reason to believe that nitrogen removal might also be needed at Marlborough, a decision was made to investigate the use of lime for coagulation of the TFE because it could raise the pH sufficiently to allow removal of ammonia nitrogen by air stripping, in addition to phosphorus removal by precipitation.

Marlborough Trickling Filters

The trickling filters at Marlborough were designed as high-rate units employing the flow pattern shown on Figure 2. Recycling of final effluent to the filters was on an intermittent basis and occurred only when wastewater flows were low enough to allow recirculation without hydraulically overloading the filters.

Fig. 2   Flow Diagram of High-Rate Trickling Filter System at Marlborough

2-1

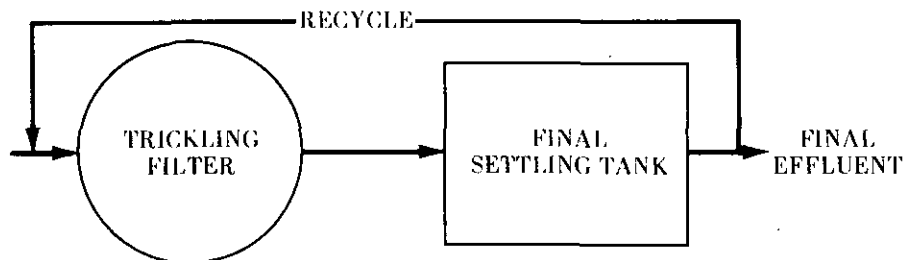Figure 3 shows the minor changes that were considered necessary at Marlborough to accomplish the studies on upgrading effluent quality and on phosphorus removal employing lime as the coagulant and precipitant. Because of problems related to conducting the tests on a plant scale according to the plan shown on Figure 3, a decision was made to alter the recycle system as shown, but to conduct the lime-treatment studies in a pilot plant. TFE was pumped from the sump to supply feed to the pilot unit.
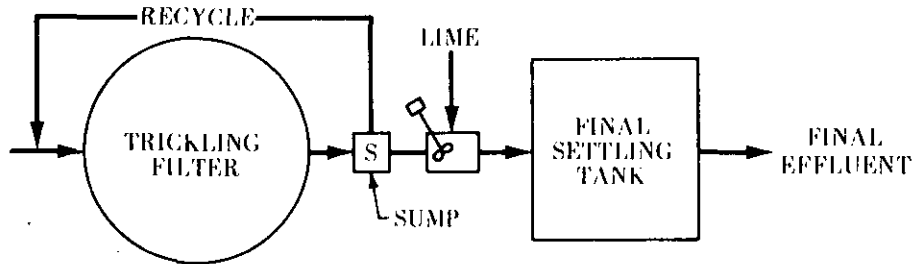
Fig. 3 Modified Flow Diagram of Trickling Filter for
Upgrading Effluent Quality Employing Lime

Lime Treatment of Marlborough Trickling Filter Effluent

The pilot plant studies on lime treatment of TFE were all conducted in a combination coagulation—clarifier unit employing an upflow solids contact chamber. The feed rate averaged about 5 gpm (gallons per minute) and the pH was controlled automatically between desired setpoints.

Effluent Quality — Historically, the quality of wastewater treatment plant effluents has been judged largely upon suspended solids and BOD. Other parameters of concern, particularly where nutrient removal is concerned, are ammonia and organic nitrogen. The latter was included in this evaluation because it would not be removed along with ammonia in a subsequent air stripping operation.

Figure 4 presents information on the quality of final effluent following lime treatment of TFE, in terms of the parameters expressed above.

The BOD of the final effluent varied radically from about 10 to 100 mg/L with an average of about 40 mg/L. The data were highly variable at all pH levels. The reason for this condition is discussed in some detail in the Appendix.

In general, suspended solids in the final effluent were below 50 mg/L, averaging about 20 mg/L at all pH levels below 11. At pH levels above 11,

Fig. 4 Effluent Quality Versus pH
Following Lime Treatment of Trickling Filter Effluent

2-3

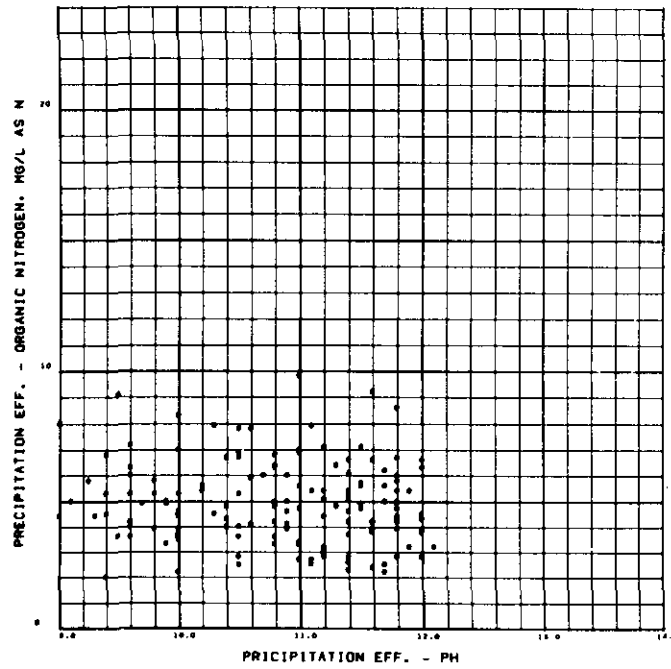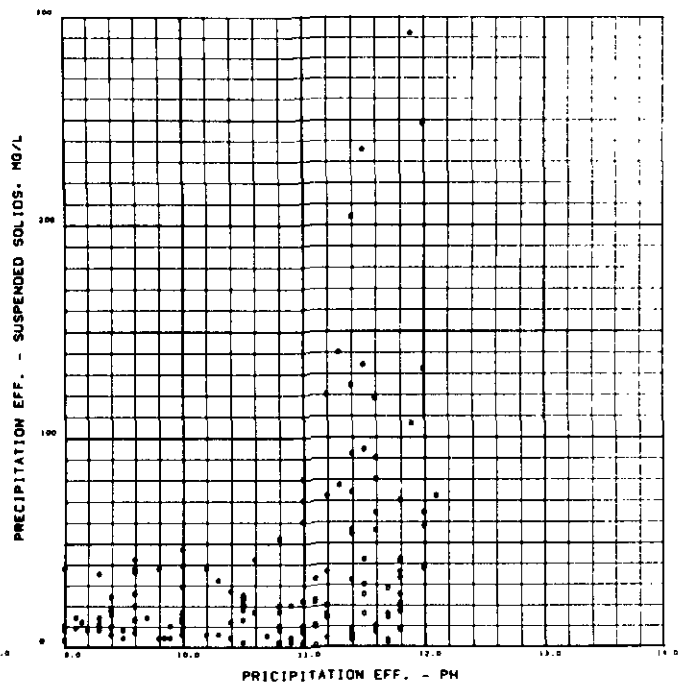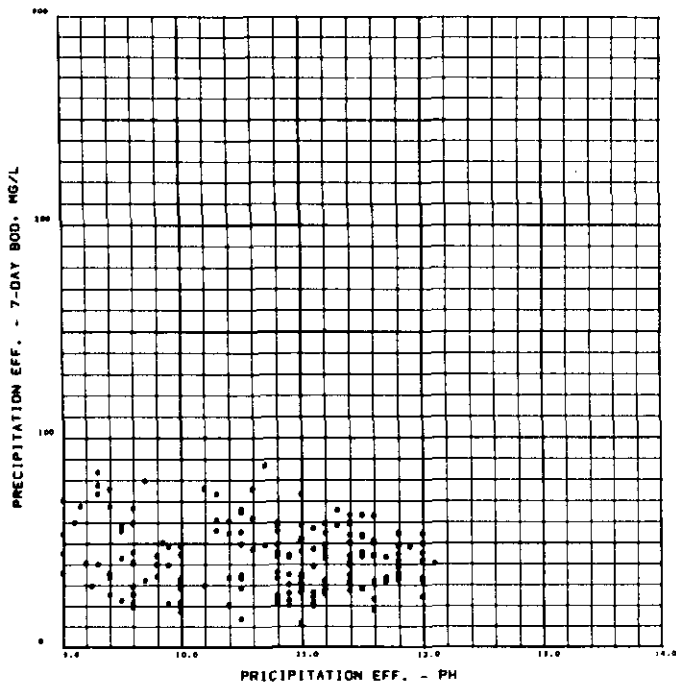results were highly erratic with values ranging up to 200+ mg/L. The high suspended solids at pH 11 and above were primarily due to loss of flocculant magnesium hydroxide and some magnesium ammonium phosphate.

The organic nitrogen content of the final effluent following lime treatment was found to vary radically at all pH levels, with some indication of improvement as pH increased. This parameter was, undoubtedly, closely associated with BOD and responds to the same conditions.

Summary — In general, the suspended solids content of the lime-treated effluent was at acceptable levels at all pH values below 11. However, both BOD and organic nitrogen were about twice as great, on an average, as those expected in a good quality activated sludge effluent.

Because of unusual circumstances at the Marlborough wastewater treatment plant with respect to wastewater flows, low temperatures, and condition of the trickling filters, it seemed unwise to condemn all trickling filter treatment plants as being unable to meet modern standards of treatment. On the assumption that a major part of the problem was related to the condition of the existing filters following 22 years of service, a decision was made to conduct further studies employing pilot filters.

Pilot Plant Trickling Filters

When it became apparent that further investigations on trickling filters at Marlborough should be conducted on a pilot plant scale, a decision was made to construct rock and plastic media filters which would be operated in parallel.

Rock Trickling Filter — The rock filter was a model of the existing filters at Marlborough. It was 13 feet in diameter, contained a depth of 6 feet of 2.5- to 3.5-inch rock, and was equipped with a rotary distributor and standard high-rate drainage system. (See Figure 5.) Imhoff tank effluent was fed to the unit at a rate varying from 400 to 700 gpd/sf (gallons per day per square foot). Trickling filter effluent was recirculated to the filter at the rate of 30 gpm, to provide a recirculation ratio of approximately 1-1/2 to 1. The filter was operated from November 18, 1969 until April 19, 1970.

Plastic Trickling Filter — The plastic media filter, shown on Figure 6, was four feet square and the media was 24 feet deep. Imhoff tank effluent was pumped to the unit to provide a feed rate of 1.0 gpm/sf (gallons per

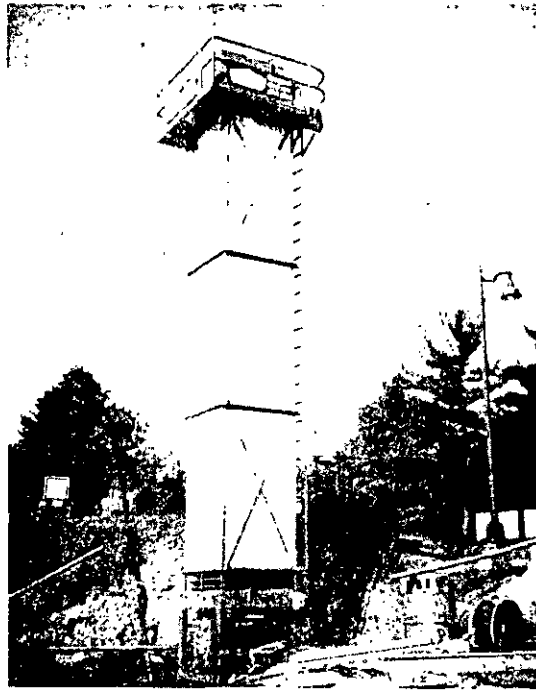Fig. 5 View of Pilot Rock Trickling Filter

Fig. 6 View of Pilot Plastic Trickling Filter

2-5

minute per square foot). No recirculation was used. The filter was operated from October 13, 1969 until April 16, 1970.

Effluent Quality — A comparison of the performance of the pilot rock and plastic filters and the main plant filter in terms of BOD remaining after settling is shown on Figure 7. These data show that the quality of the effluent from the pilot rock filter was somewhat poorer than that from the plastic filter which operated at considerably greater BOD loadings. In the range of 20- to 40-pound loadings the plastic filter gave far superior results. The main plant filter, although operating under unsteady state hydraulic loadings, gave nearly as good results as the pilot filters operating under steady state conditions, indicating that the main plant filters were not in as bad a condition as originally assumed.

Figure 8 presents data on the suspended solids content of settled *effluents from the pilot and plant filters.* The quality of all effluents, in terms of this parameter, was extremely erratic, especially that from the pilot rock filter. In the case of the plant filter, there appeared to be improved quality as the BOD loading increased; however, the deterioration of quality was due to the high flows which hydraulically overloaded the settling tanks at low BOD loadings. With the plastic pilot filter there was a definite deterioration as BOD loadings exceeded 80 lb BOD/1,000 cu ft.

Summary — All data shown on Figures 7 and 8 were obtained during the period of November 1969 through April 1970 and, therefore, are typical of winter operation. From the data, it seems reasonable to conclude that operation of high-rate rock filters, under winter conditions, at BOD loadings up to 40 lb/1,000 cu ft will result in effluents with BOD and suspended solids averaging about 40 mg/L and that operation of plastic filters under similar conditions, at BOD loadings up to 60 lb/1,000 cu ft will result in effluents averaging 30 and 20 mg/L of BOD and suspended solids, respectively. Thus, it appears that plastic filters will produce higher quality effluents than rock filters at comparable loadings and comparable results at somewhat higher loadings.

Lime Treatment of Pilot Trickling Filter Effluent

From November 16, 1969 through January 26, 1970, the pilot rock TFE was used as feed stock to the lime precipitation unit. The feed rate averaged about 8 gpm and the pH was controlled automatically between desired setpoints.
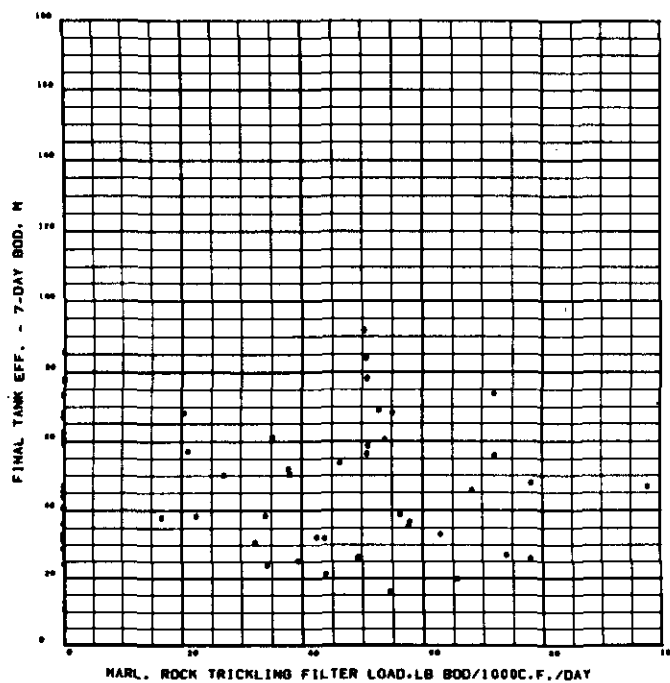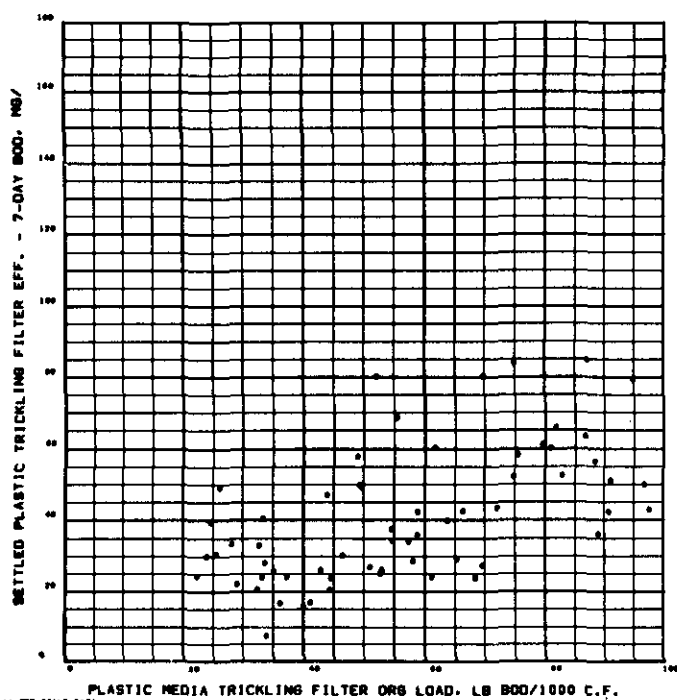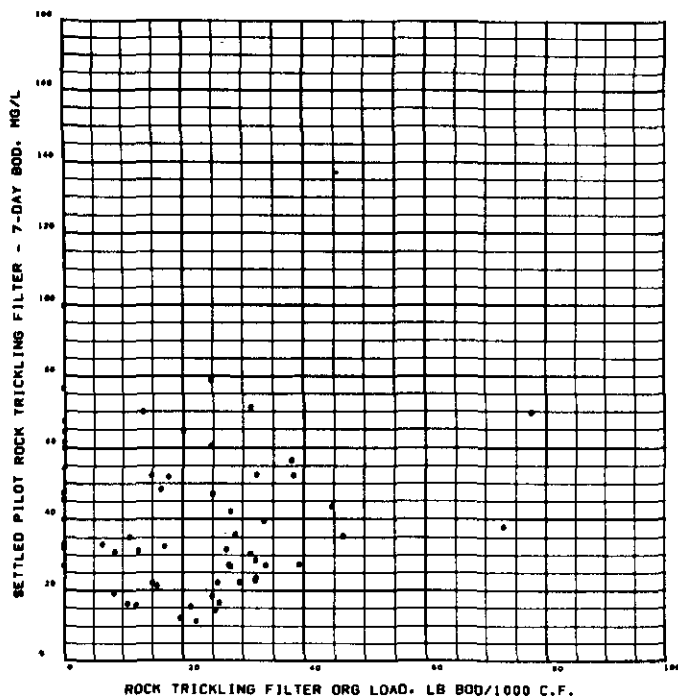
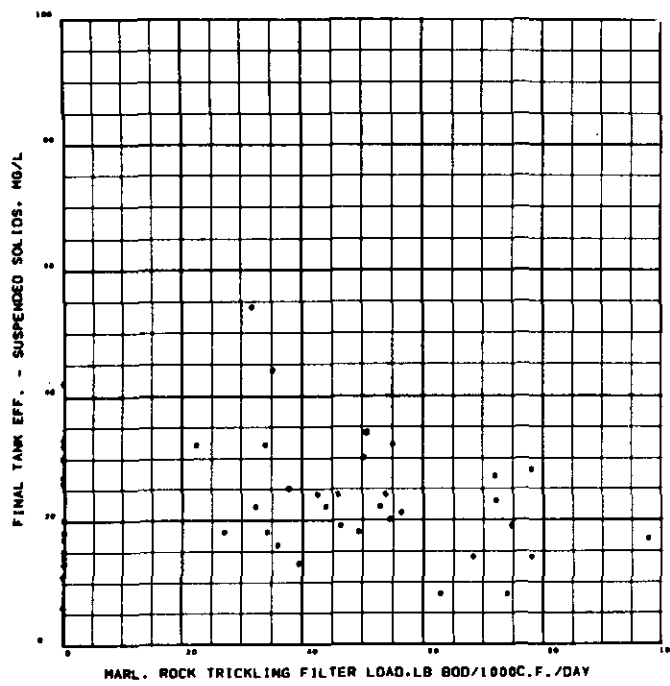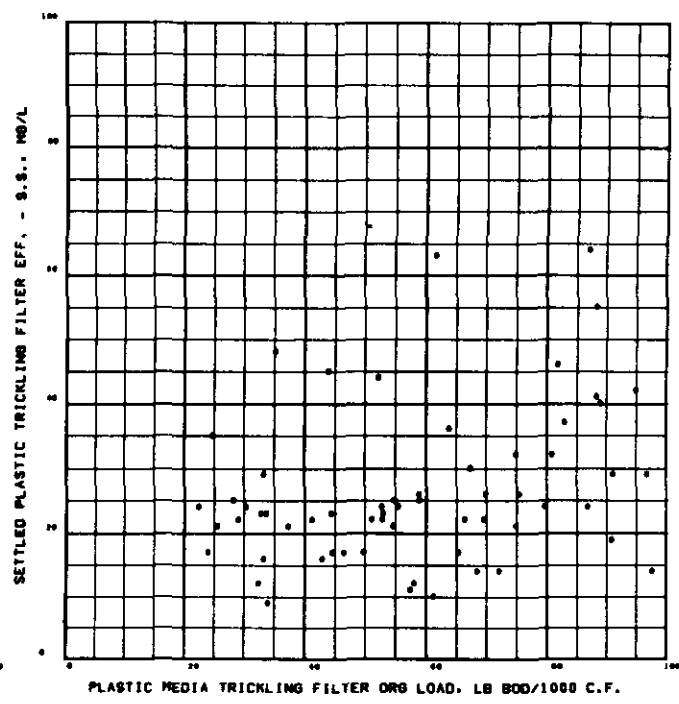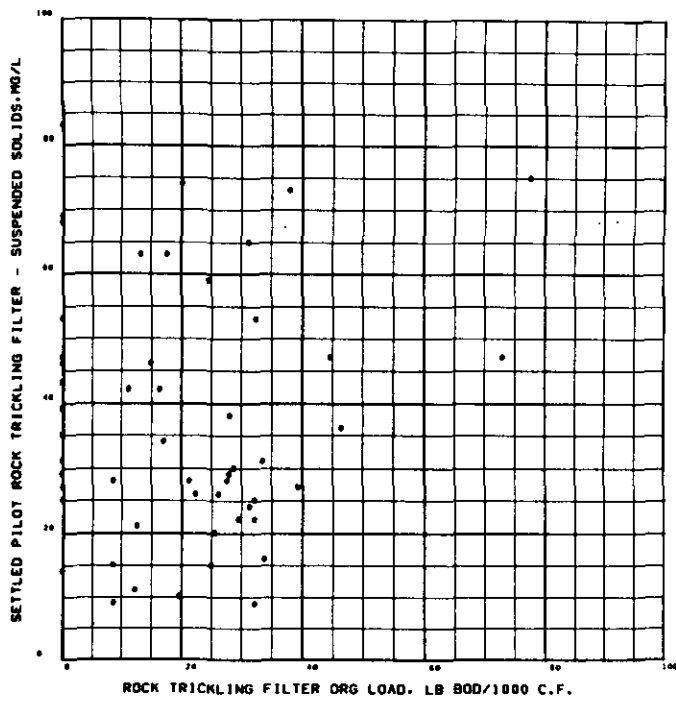Fig. 7 Performance of Pilot and Plant Trickling Filters in Terms of BOD

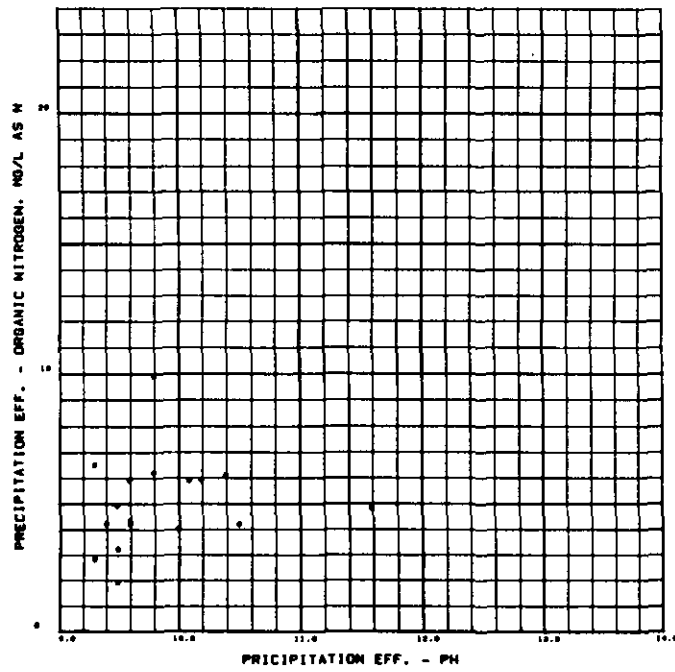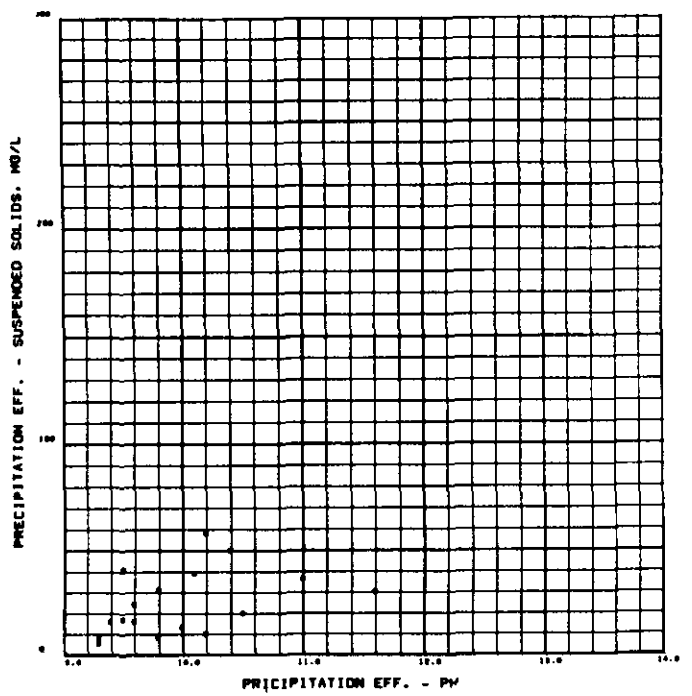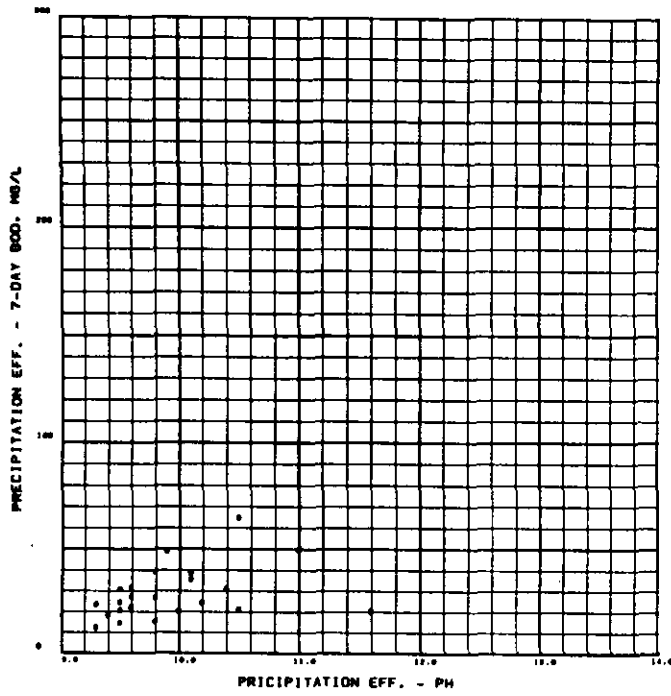Fig. 8   Performance of Pilot and Plant Trickling Filters in Terms of Suspended Solids

2-8

Fig. 9   Effluent Quality Versus pH Following Lime Treatment
of Pilot Rock Trickling Filter Effluent

2-9

Effluent Quality – The effluent quality in terms of BOD, suspended solids, and organic nitrogen, following lime treatment, is presented as Figure 9. Unfortunately, a majority of the data were obtained at pH values less than 10. The indications are that the rock filter effluent can be improved to the extent that its BOD and suspended solids would be reduced to the range of 25-30 mg/L. Organic nitrogen residuals in the lime-treated effluent were highly variable, averaging about 5 mg/L.

Summary – In general, the suspended solids content of the lime-treated effluent was at acceptable levels at all pH values below 11. However, both BOD and organic nitrogen were about twice as great, on the average, as would be expected in a good quality activated-sludge effluent.

Conclusions

A comparison of the data on Figure 9 with those on Figure 4 shows no outstanding differences in quality between the lime-treated effluents of the pilot rock and plant filters. There was somewhat less variation in BOD when treating the pilot TFE because the pilot unit was operated under steady-state hydraulic conditions while the plant filter was subject to normal diurnal and wet-weather flow variations.

It may be concluded on the basis of these investigations that lime treatment of TFE will improve the quality of the effluent, but it can be expected that residual BOD and suspended solids will average about 40 and 20 mg/L, respectively.

Because of the increasing stringency of standards imposed upon the discharge from waste treatment facilities, the question of the adequacy of trickling filters to meet these standards must be answered. It must be concluded from these studies, that the effluent from trickling filters is of lesser quality than that from the activated-sludge process and that the effluent quality deteriorates as the temperature decreases. While these two factors do not necessarily eliminate trickling filters from further consideration as treatment processes, they do place serious limitations upon their use. The trickling filter should continue to be considered as an alternate to the activated-sludge process in situations where the filter will be used as a roughing filter, or where the filter will be followed by certain nutrient removal processes capable of improving effluent quality to levels expected when treating activated-sludge effluents.

CHAPTER 3

PHOSPHORUS REMOVAL

Removal of phosphorus (inorganic and organic phosphates) from domestic and some industrial wastewaters is considered mandatory in all situations where drainage is to lakes and reservoirs, because of its alleged effect on stimulating eutrophication of such bodies of water. The importance of this subject is attested to by the pressure currently being exerted upon the manufacturers of synthetic detergents to eliminate phosphates from their products and edicts advanced to accomplish at least 80 percent removal of phosphorus from all wastewaters discharged to the Great Lakes.

Because of the particular location of the Marlborough Easterly wastewater treatment plant, phosphorus removal was considered a necessity and, because of the considerable existing investment in trickling filters, special attention was given to treatment of their effluent for the reasons expressed in Chapter 1.

The activated-sludge process is a competitive biological treatment process that is easily adapted to phosphorus removal through cationic precipitation of phosphates. Since it seemed a likely substitute or supplementary process for the trickling filter system, studies employing it were also conducted in a pilot plant.

Lime Treatment of Trickling Filter Effluents

The investigations on phosphorus removal from TFE were initiated during January 1969 and continued through January 1970. All studies were conducted upon unsettled TFE for reasons previously expressed. The TFE from the existing plant was used from January 1969 until October 1969. From the latter date to February 1970, TFE from the pilot rock filter was used as feed to the phosphorus removal unit.

Operation of the plant trickling filter was as described in the Appendix and the pilot units were run as described in Chapter 2.

Equipment — All studies on phosphorus removal by lime treatment of TFE were conducted in the combination precipitation upflow solids contact clarifier shown on Figure 10. A schematic diagram of this unit with ancillary

pH and lime feeding equipment necessary for continuous operation at desired pH levels is presented as Figure 11.

TFE was fed to the phosphorus removal unit at a normal rate of 6 gpm. Lime additions were paced by an automatic pH controller to maintain the desired pH in the mixing and coagulation zone. Since there was normally a significant change in pH through the system, a second pH recorder was installed on the effluent line to maintain a record of effluent quality for subsequent studies on nitrogen removal.

Phosphorus Removal — Although a considerable fund of information existed in the literature concerning lime requirements in terms of wastewater alkalinity versus pH and phosphorus residuals versus pH, the studies at Marlborough were designed to explore the efficacy of lime treatment over a relatively wide range of pH values. This was done by attempting to hold the pH in the coagulation-mixing zone within a relatively narrow pH range for several consecutive days before making changes. A major reason for studying pH effects over a rather wide range was related to the fact that the effluent from the phosphorus removal unit served as feed for the ammonia stripping unit, on which it was desirable to vary the pH over a relatively wide range.

The amount of lime required to reach any desired pH level was found to be related to the alkalinity of the wastewater as has been reported by other investigators. Figure 12 shows the amount of lime required to obtain various pH levels in Marlborough wastewater under dry-weather conditions.

Data showing residual phosphorus levels obtained during the course of the Marlborough studies are presented on Figure 13. Parts "A" and "B" of this figure show results obtained during the first 300 days, while TFE from the main plant was used to feed the lime precipitation unit. The data indicate that the best phosphorus removals, both inorganic and total, were obtained at pH levels between 9.9 and 11.0. The somewhat poorer quality effluent produced at pH levels between 11 and 12 was believed to be due to the higher suspended solids in the effluent caused by the formation of flocculant magnesium hydroxide and possibly magnesium ammonium phosphate, as noted in Chapter 2.

From the data obtained on the main plant TFE it seemed reasonable to conclude that lime treatment at pH levels between 10 and 11 would normally ensure the production of effluents with less than 0.5 mg/L of inorganic phosphorus and less than 1.0 mg/L of total phosphorus without filtration.
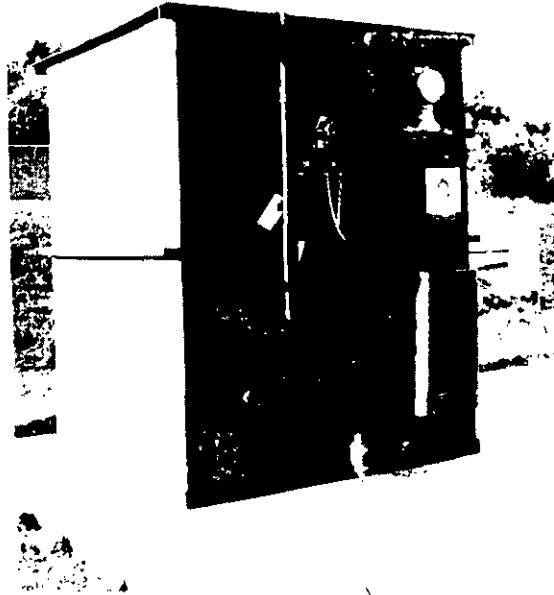
Fig. 10   Combination Precipitation-Upflow Solids
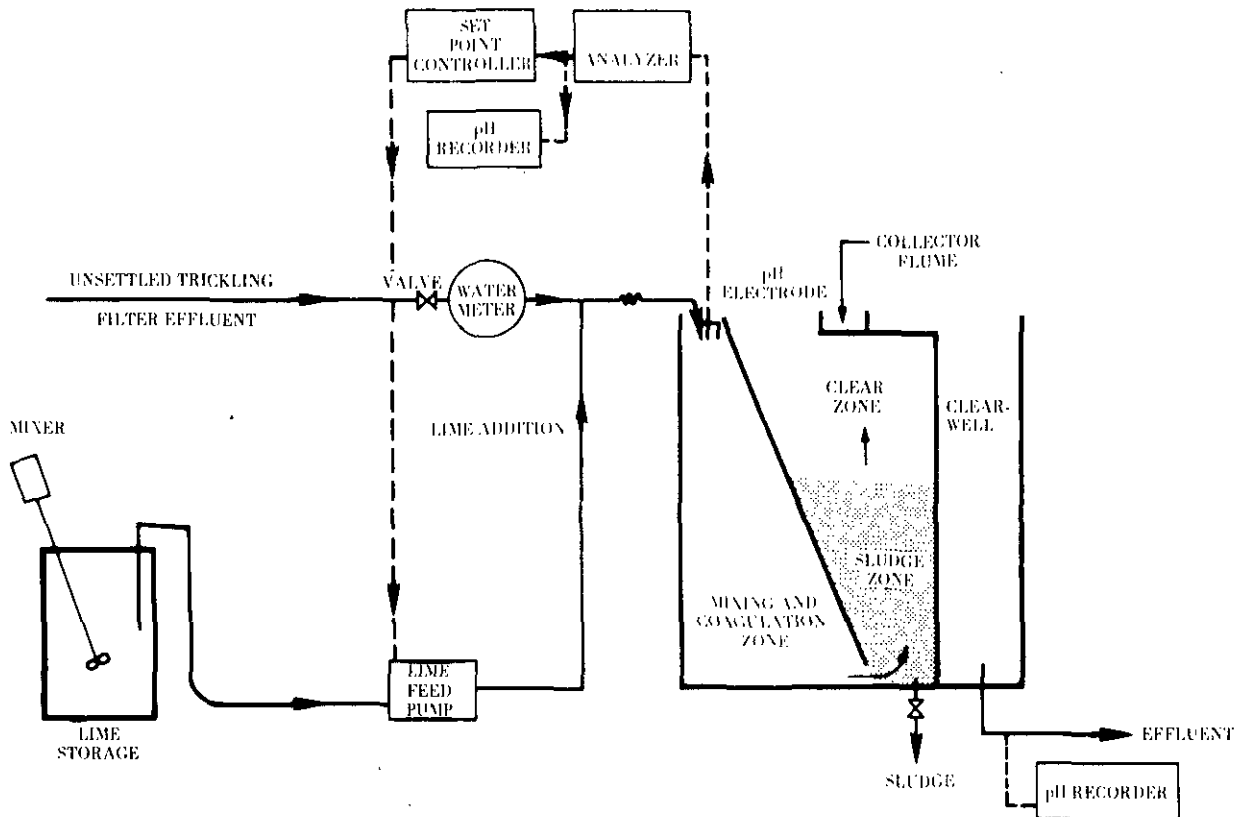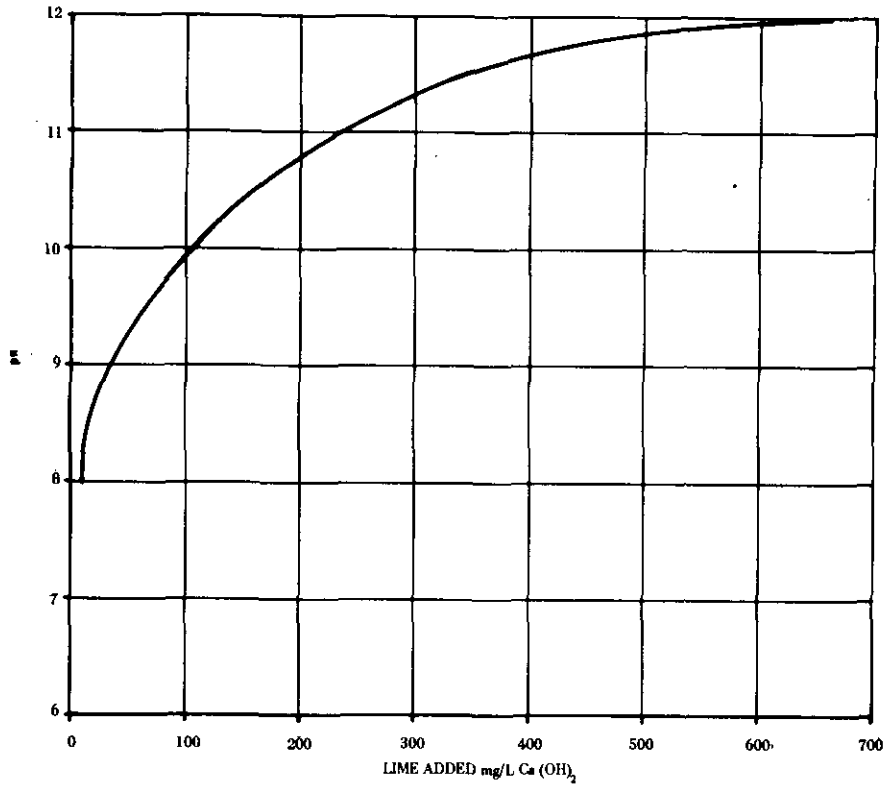Content Clarifier Used for Phosphorus Removal

Fig. 11   Pilot Plant for Phosphorus Removal with Lime

Fig. 12 Lime Requirements Versus pH at Marlborough

On November 16, 1969, biological growths on the pilot rock filter had matured to a degree that it was considered feasible to use its effluent as feed to the lime precipitation unit. The results obtained from that day through January 27, when the unit was closed down, are given in parts "C" and "D" of Figure 13. It will be noted that the results obtained on the pilot TFE were not nearly as favorable as those obtained on the plant TFE. Nearly one-half of the time the pH was below the favorable pH of 10. On the basis of the days when the pH was actually above 10, the data indicate that effluents with inorganic phosphorus of 1 to 2 mg/L and total phosphorus 1.5 to 3.0 mg/L would be normal. The great increase in these values as compared to the results obtained on the plant TFE was a cause of concern and efforts were made to determine why.

Because studies on the pilot TFE were conducted during a period when the Marlborough wastewaters had very low temperatures, the first inclination was to suspect it as a cause. Figure 14 presents a tabulation of all inorganic phosphorus data obtained during the survey in chronological order, with day 0 being December 31, 1968. Also given on Figure 14 are pH data for each of
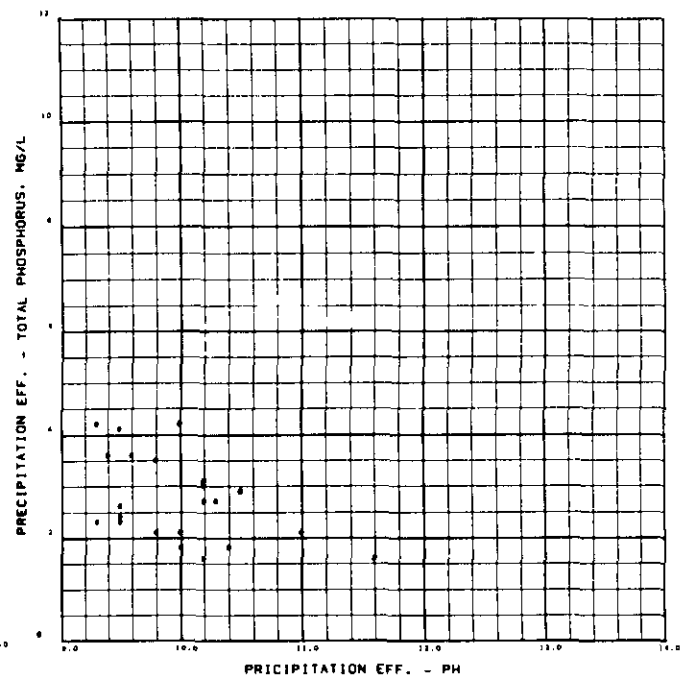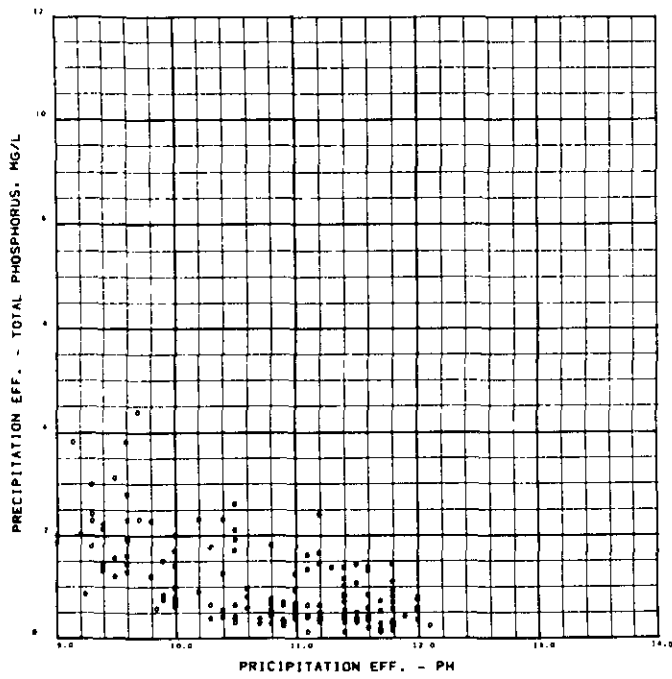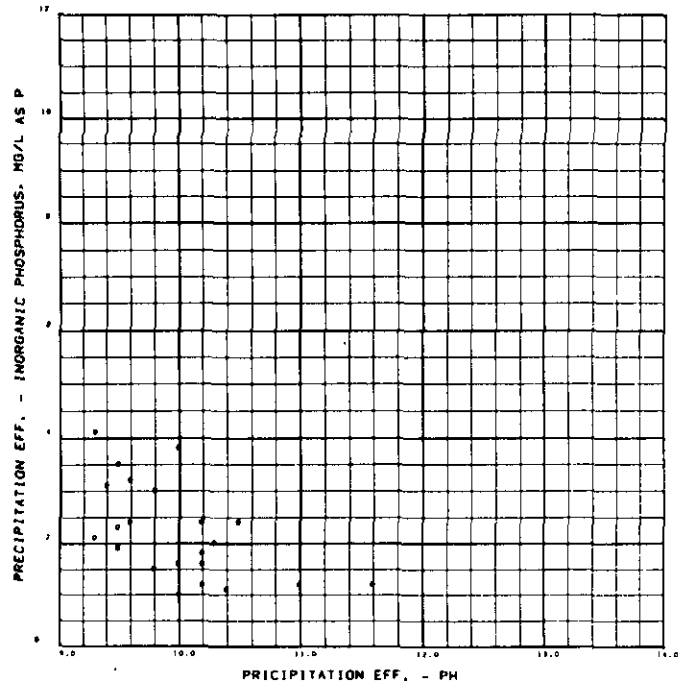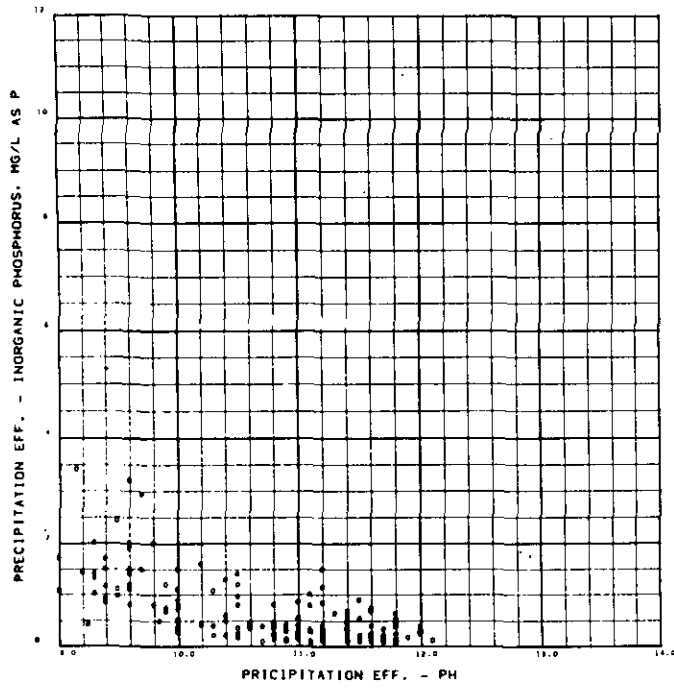
3-4

Fig. 13   Residual Phosphorus Versus pH with Lime Precipitation

3-5

the days on which residual phosphorus was measured. Inspection of the data will show that several days of operation at pH levels of 10 or above occurred during both the winter of 1969 and of 1970. In 1969, many values of inorganic phosphorus below 0.5 mg/L were observed; in 1970, none. Obviously, the reason for the discrepancy was not low winter temperatures.
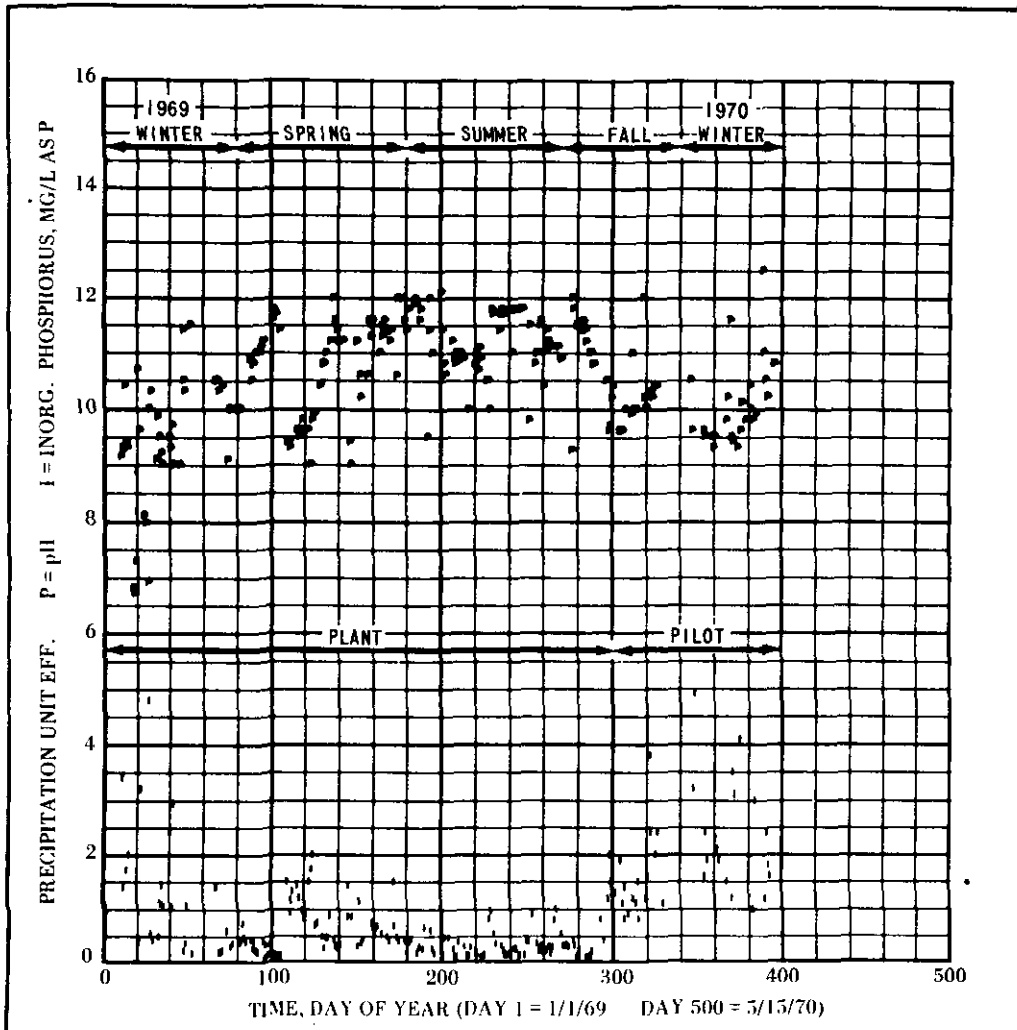
Fig. 14   Residual Phosphorus versus Time and pH

Further investigation has disclosed that when the change was made from using plant TFE to pilot TFE, a change in hydraulic regime occurred. Under the new conditions, a greater flow of TFE was fed to the precipitation unit. The poorer results obtained during the winter of 1970, therefore, are believed to have been caused by this hydraulic overload. It must be concluded, therefore, that production of effluents with low phosphorus levels of less than 1 mg/L will require that close attention be given to hydraulic loadings on clarifiers.

3-6

Summary — In general, the best quality effluent in terms of phosphorus occurs at pH levels between 9.9 and 11.0. This would ensure the production of effluents with less than 0.5 mg/L of inorganic phosphorus and less than 1.0 mg/L total phosphorus. In order to further insure this quality effluent, close attention must be given to the hydraulics of the clarifiers.

Activated-Sludge Process

Although several reports have been made indicating that the activated-sludge process is capable of high removals of phosphorus through "luxury uptake," if proper dissolved oxygen conditions are maintained, subsequent studies have shown that "luxury uptake" does not occur at all locations, and that where it does occur it is due primarily to cationic precipitation of phosphates. During the course of the studies at Marlborough several opportunities developed for determination of "luxury uptake" when freezing conditions prevented the addition of chemicals. Under such conditions and, in the presence of high dissolved oxygen levels, phosphorus removals averaged about 20 percent.

Because of the many reports in the literature indicating that high removals of phosphorus could be reliably obtained in the activated-sludge system of wastewater treatment by the judicious addition of aluminum or ferric salts, a decision was made to investigate this method of treatment as a comparison to the trickling filter system.

Equipment — The package-type activated-sludge plant shown on Figure 15 was installed at Marlborough in a location selected to facilitate use of Imhoff tank effluent as feed. The unit was selected from a considerable number of manufacturers' products because it included a circular clarifier with mechanical sludge scrapers to deliver sludge to an air-lift sludge return line. As received, the unit had a single aeration tank designed to operate on the completely mixed principle. Since most large municipal plants have been designed to operate as multiple-pass systems, the aeration tank was modified in a manner, as shown on Figure 16, to allow operation as a multiple-pass aeration tank and to allow operation of any of the modifications of the activated-sludge process.

Operation — The activated sludge plant was placed in operation as a stepped aeration system during the fall of 1968. Based upon studies conducted at other locations by Metcalf & Eddy, Inc., it was determined that the optimum application point for alum (the coagulant chosen for precipitation of phosphorus) was the last pass of a multiple-pass aeration tank.
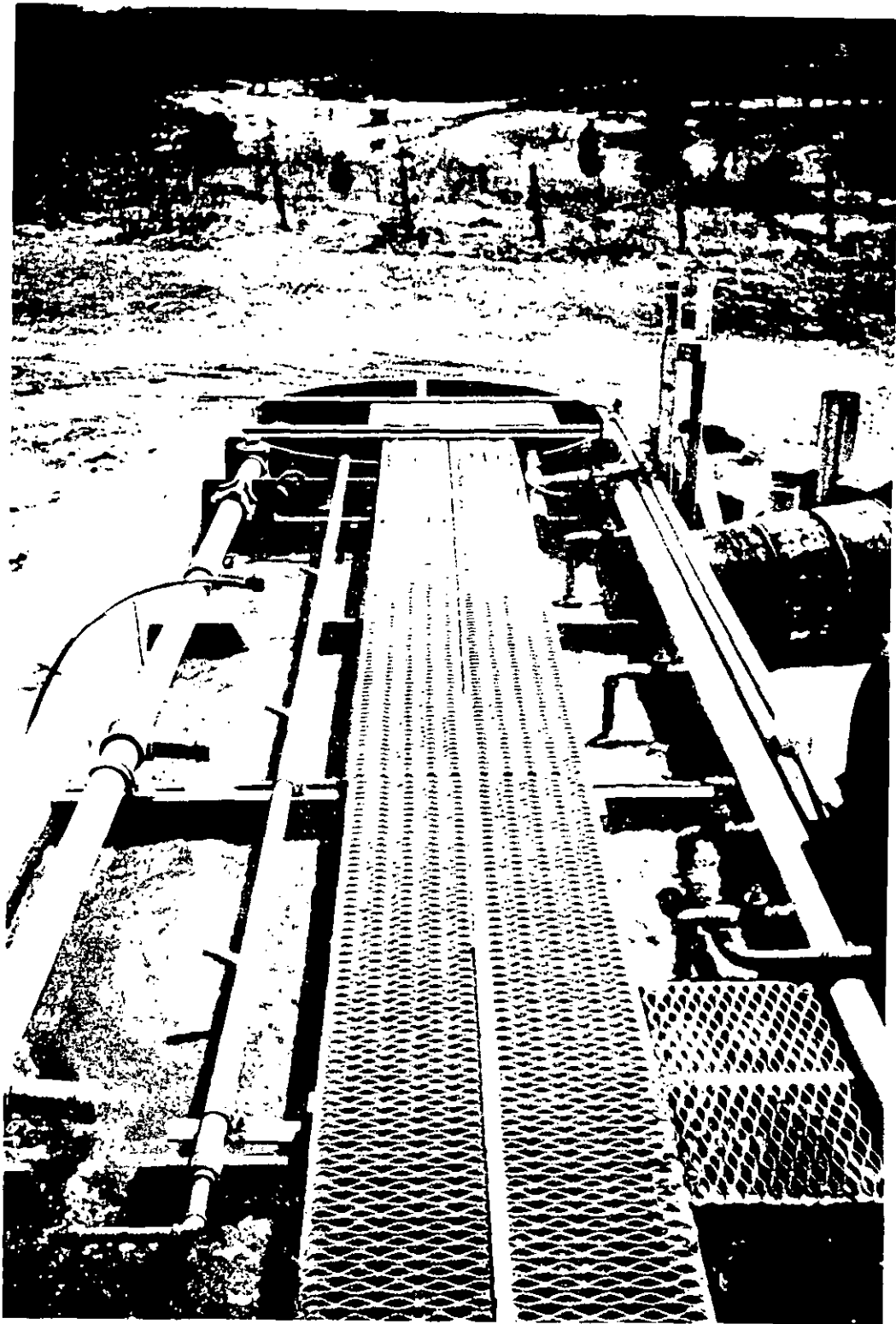
3-7

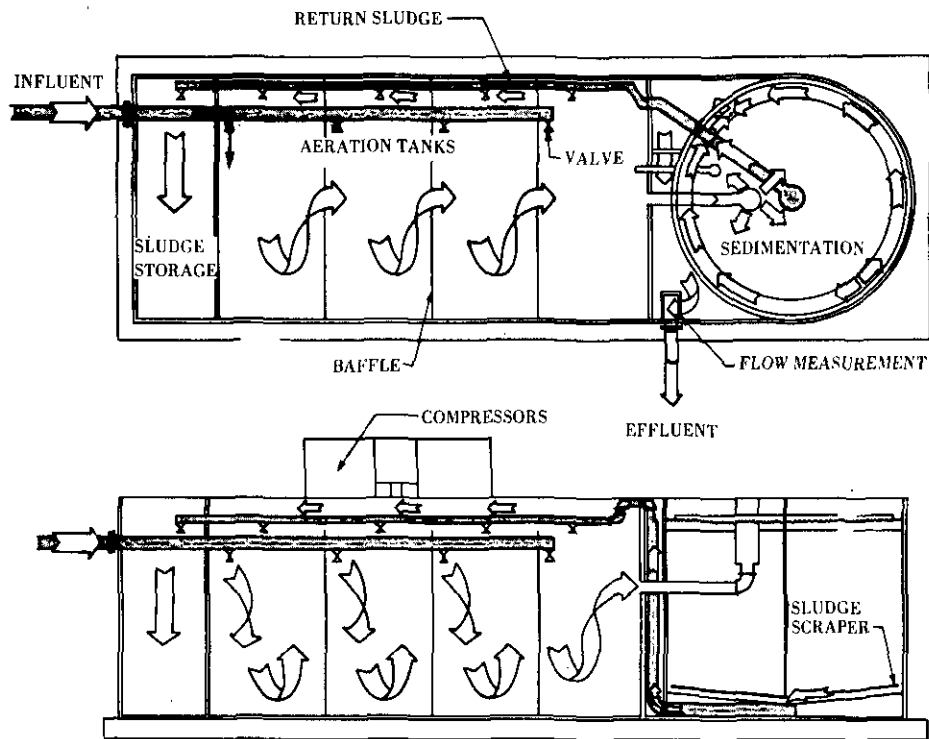Fig. 15   View of Activated-Sludge System

Fig. 16 Modification of Aeration Tank to Multiple Pass System

Routine operations were begun on January 2, 1969 and were continued until August 8. During extremely cold weather, alum additions were impossible with the facilities available, because of problems caused by freezing. The wastewater flow to the activated-sludge plant was maintained at a steady state of 19 gpm.

Phosphorus Removal — Before initiating the plant scale studies, considerable laboratory effort was expended in attempting to evaluate *hour-to-hour and daily variations in the inorganic phosphorus content of the* Imhoff tank effluent at Marlborough, as an aid to estimating desired alum dosages, in lieu of expensive continuous laboratory or automatic control. The results of these studies showed relatively minor diurnal variation as compared to the daily variation.

In planning the phosphorus removal studies, it was intended to hold the alum to inorganic ratios at a definite value for several days in succession and then change to a different ratio.

The data presented on Figure 17 show how the quality of the influent *(Imhoff tank effluent) to the activated sludge unit varied from day to day in*

3-9

terms of inorganic phosphorus and BOD. Because of the great daily variations in phosphorus, the ratio of alum to inorganic phosphorus changed considerably from day to day as shown, in spite of the intention of holding fairly uniform ratios for extended periods of time.

Figure 18 is a summary which shows how the common operational parameters of MLSS, F/M (food to microorganism) ratio and DO (dissolved oxygen) varied during the course of the studies. It also shows the settling character of the MLSS in terms of SVI. It will be noted that a considerable day-to-day variation occurred in spite of the steady state hydraulic load imposed. Reference to the BOD information on Figure 17 will show that the great variations in F/M ratios and DO were related to the BOD. The ability to maintain SVI values of 100 or less during such wide variations in operational parameters is definite evidence of a special benefit from the addition of alum for phosphorus removal.

Because of the considerable daily variation in alum to phosphorus ratios, the data concerned with effluent quality have been analyzed in terms of the ratios as shown on Figure 19. From these data it may be seen that alum dosages of at least 18 to 1* of phosphorus were needed to reduce inorganic phosphorus to average levels of less than 1 mg/L, 25 to 1 to ensure less than 1 mg/L, and at least 28 to 1 to produce effluents with 0.5 mg/L or less of inorganic phosphorus.

Alum to phosphorus dosages of 20 to 1 were required to produce an average effluent of 1.5 mg/L of total phosphorus and at least 30 to 1 for any appreciable possibility of effluents with less than 1 mg/L. These data were all obtained on unfiltered samples. The fact that there were 12 occasions when total phosphorus values of 0.5 mg/L or less were measured when alum to phosphorus ratios were 30 or above indicates that phosphorus levels can be reduced to very low levels employing alum in the activated-sludge process.

Further study of Figure 19 shows that the BOD of the effluent was normally less than 25 mg/L, averaging about 15 mg/L. There appeared to be little relationship of residual BOD to the alum to inorganic phosphorus ratio, except that the poorest quality effluent seemed to occur when the ratio was between 20 and 24.

With respect to suspended solids in the activated sludge effluent, there appeared to be a definite improvement as the alum to phosphorus ratio
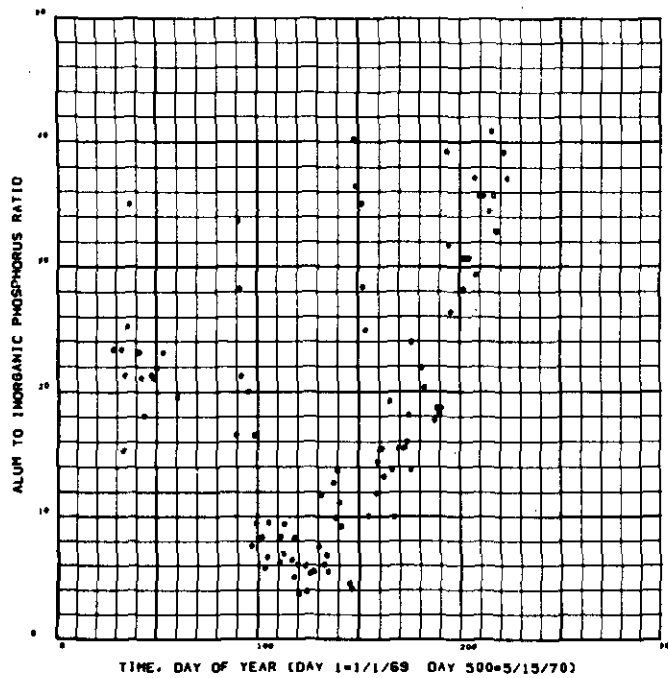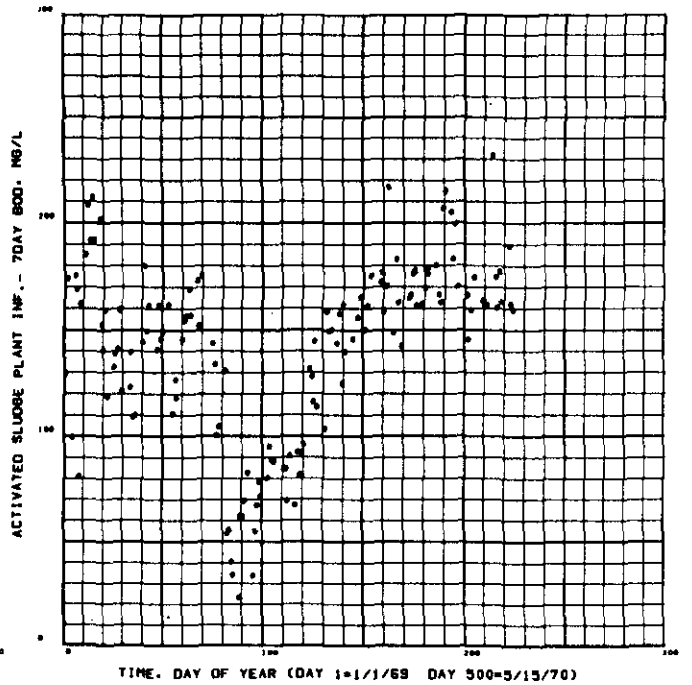
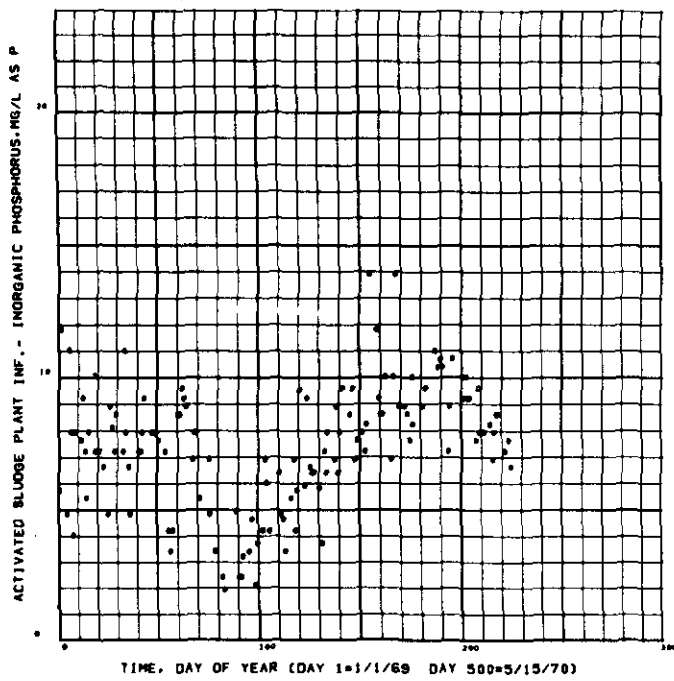*Alum: P ratio x 0.09 = Al:P.

Fig. 17 Phosphorus and BOD of Influent to Aeration Tank and
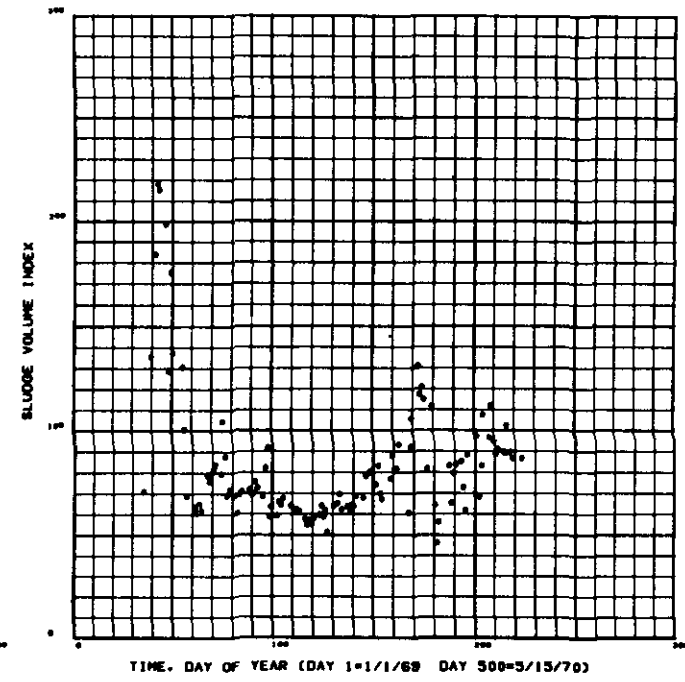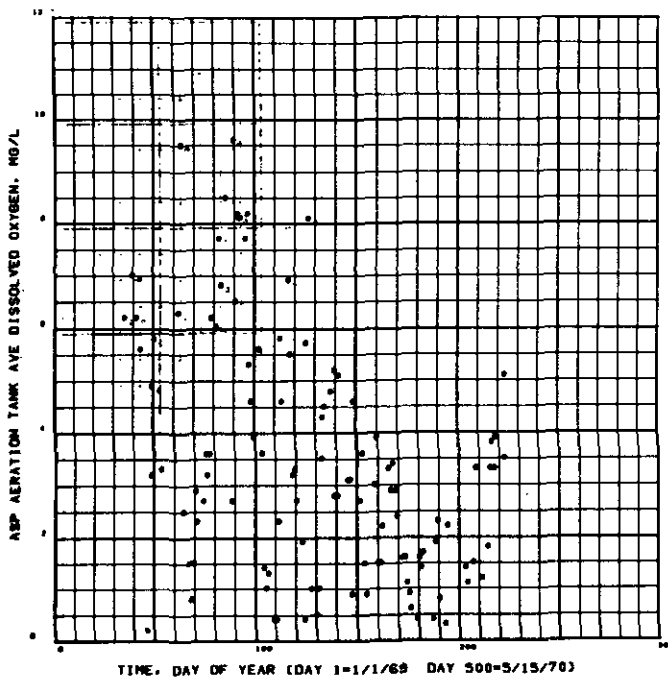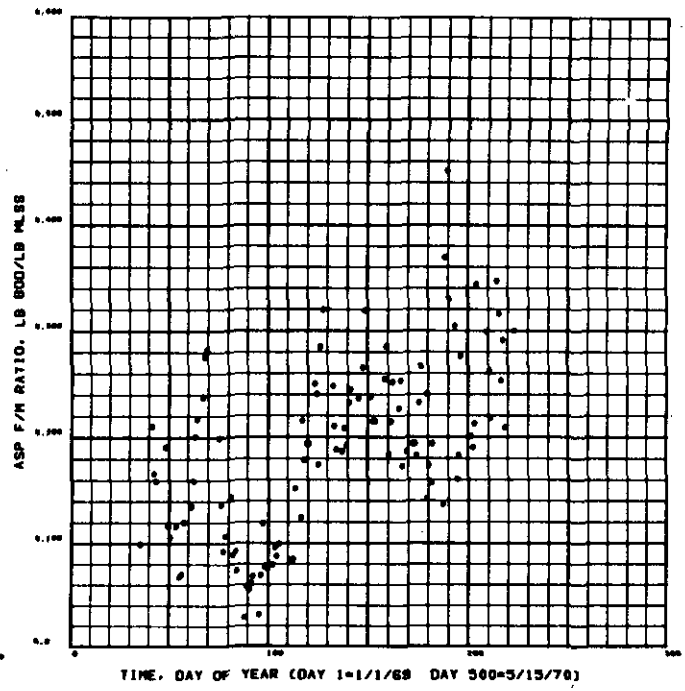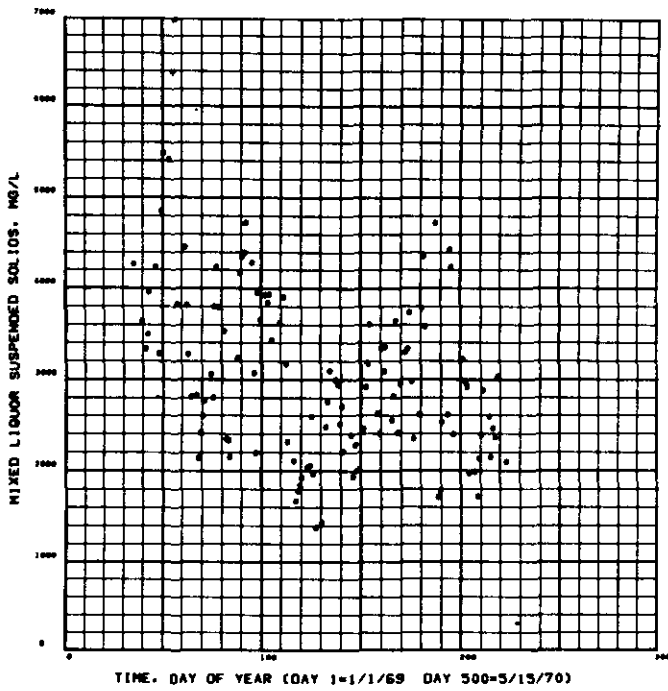Alum to Inorganic Phosphorus Ratio Employed

3-11

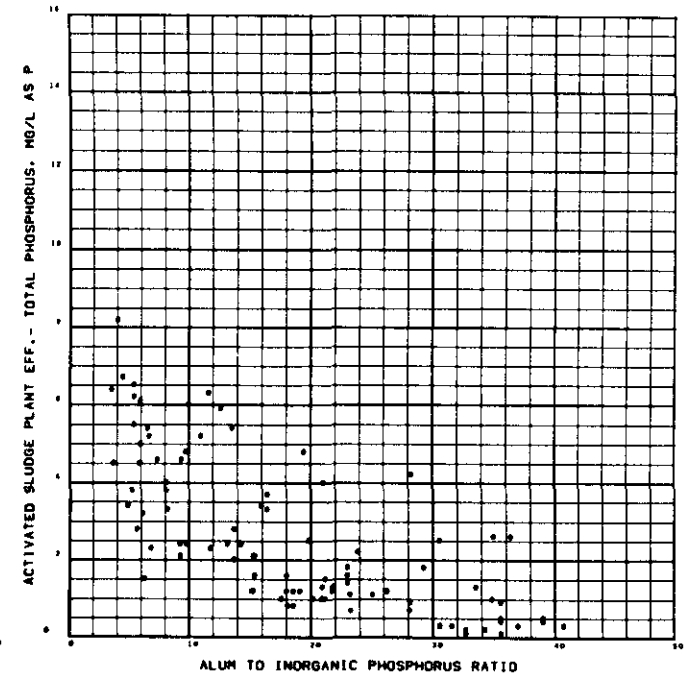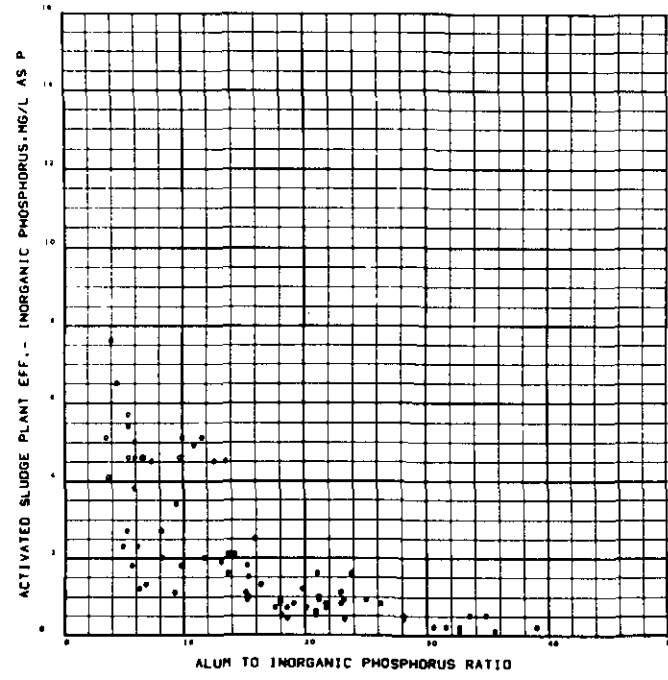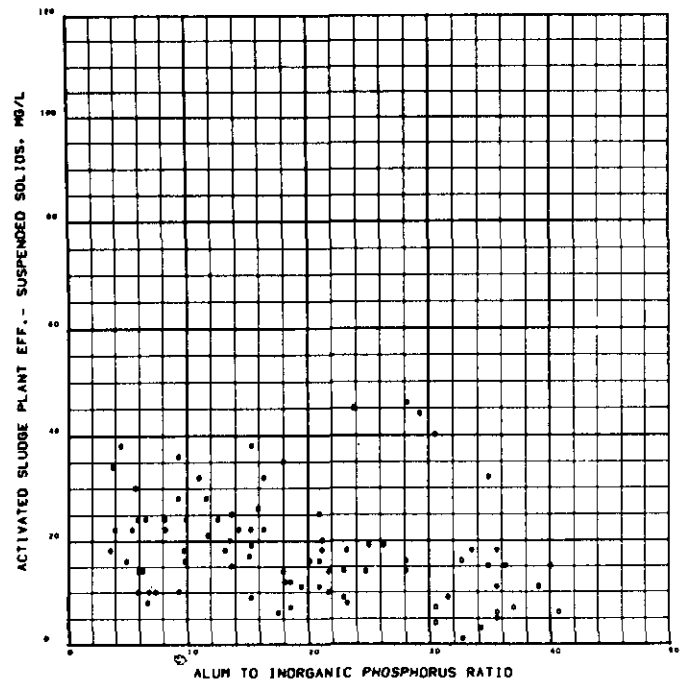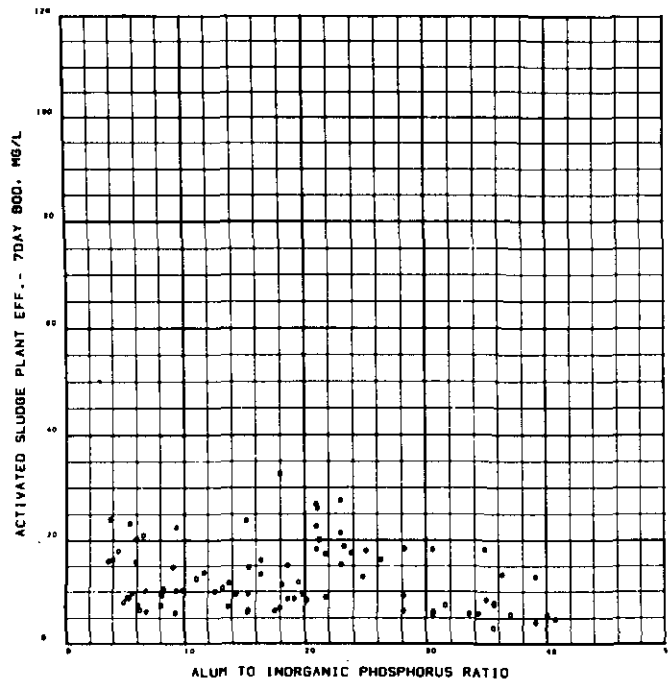Fig. 18   Activated Sludge Plant Operational Parameters

3-12

Fig. 19 Effluent Quality Versus Alum to Inorganic Phosphorus Ratios

3-13

increased. At ratios above 18 to 1, which were necessary to accomplish good phosphorus removal, suspended solids were normally less than 20 mg/L.

Summary — From the data on Figures 17, 18, and 19 we conclude that good quality effluents can be produced at Marlborough, in terms of residual phosphorus, BOD, and suspended solids when alum to phosphorus ratios are maintained at 18 to 1 or above.
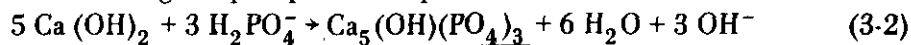
Sludge Disposal

Sludge disposal is a major problem and cost factor in any method of wastewater treatment that generates floating or settleable solids. Both of the methods of phosphorus removal investigated at Marlborough generate settleable solids that increase the total amount of solids requiring disposal.

Lime Treatment of Trickling Filter Effluent — Lime treatment of domestic wastewaters results in the precipitation of a large part of the bicarbonate alkalinity as calcium carbonate

$$Ca(HCO_3)_2 + Ca(OH)_2 \rightarrow \underline{2\ CaCO_3} + 2\ H_2O \qquad (3\text{-}1)$$
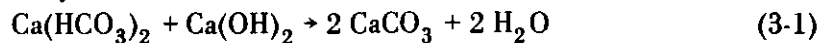
and much of the inorganic phosphates as apatite

$$5\ Ca(OH)_2 + 3\ H_2PO_4^- \rightarrow \underline{Ca_5(OH)(PO_4)_3} + 6\ H_2O + 3\ OH^- \qquad (3\text{-}2)$$

If the pH is carried to levels of 11 or above, magnesium ion will be precipitated as magnesium hydroxide

$$Mg^{++} + Ca(OH)_2 \rightarrow \underline{Mg(OH)_2} + Ca^{++} \qquad (3\text{-}3)$$
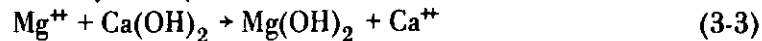
Elevation of the pH to values above 10 requires the use of excessive lime. The excess is normally removed by the addition of carbon dioxide (recarbonation) which results in the formation of additional calcium carbonate.

$$Ca(OH)_2 + CO_2 \rightarrow CaCO_3 + H_2O \qquad (3\text{-}4)$$
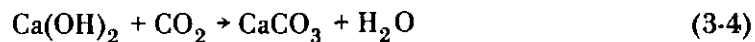
In the case of lime treatment of TFE, the organic solids sloughing from the filter would be separated in the final clarifier in combination with calcium carbonate and apatite from reactions 3-1 and 3-2. If sufficient lime were added to raise the pH above 11, then the final clarifier sludge would also contain magnesium hydroxide. Recarbonation is normally accomplished in separate facilities, so handling of sludge from it is a separate problem and not unlike that experienced in water works practice.

Lime precipitation of TFE results in the production of a sludge with a high percentage of inorganic matter. The amount of calcium carbonate will be in direct relationship to the alkalinity of the wastewater when the pH is raised to 10 or above. At Marlborough the volume of sludge wasted varied

considerably from day to day in order to maintain a desirable depth of sludge in the upflow unit. The average volume wasted was 400 gpd with an average dry-solids content of 1.3 percent, containing about 20 percent volatile matter.

Because of the low volatile content of the sludges and potential recovery of lime through recalcination, we chose to investigate separate handling of the sludges removed from the lime precipitation unit. Laboratory studies on vacuum filtration showed the sludge to be very difficult to dewater; consequently, attention was directed to dewatering by centrifuging.

Arrangements were made with the Bird Machine Company, Walpole, Massachusetts, to conduct dewatering tests using their equipment upon sludges transported to their plant by tank truck. Four batches of sludge were subjected to testing with and without the use of coagulants. The first batch of sludge was not typical so results obtained are not shown. Data obtained upon the other three batches are presented on Figures 20, 21, and 22.
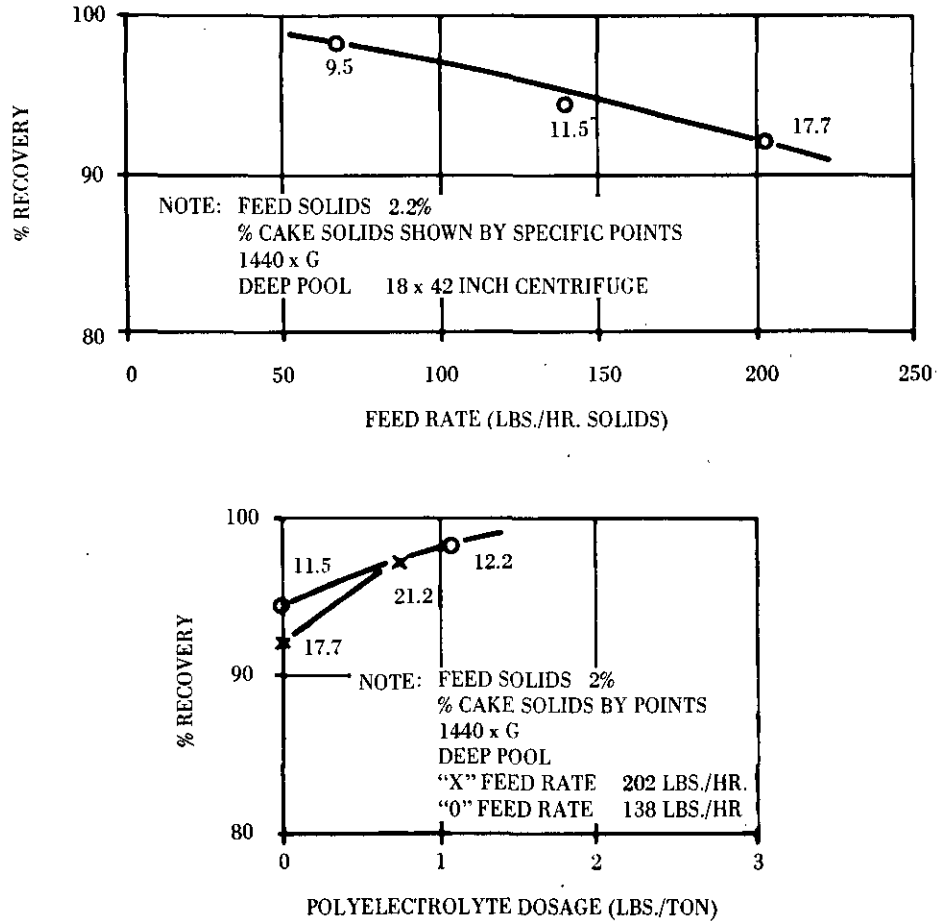

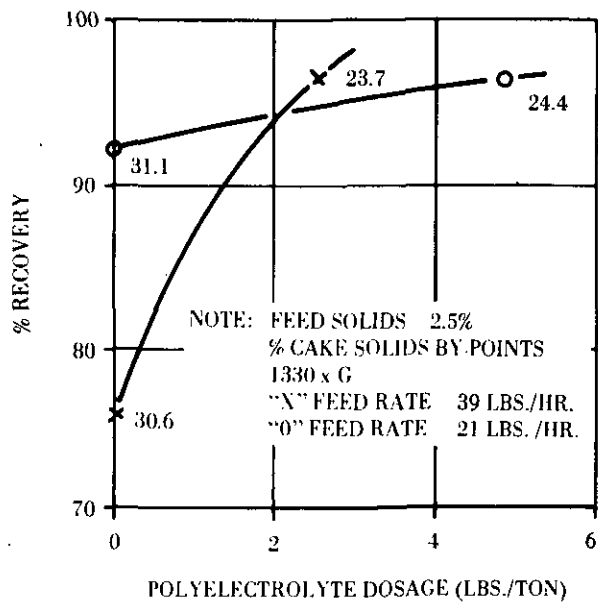Fig. 20 Centrifuge Studies on Sludge from Lime Precipitation of TFE
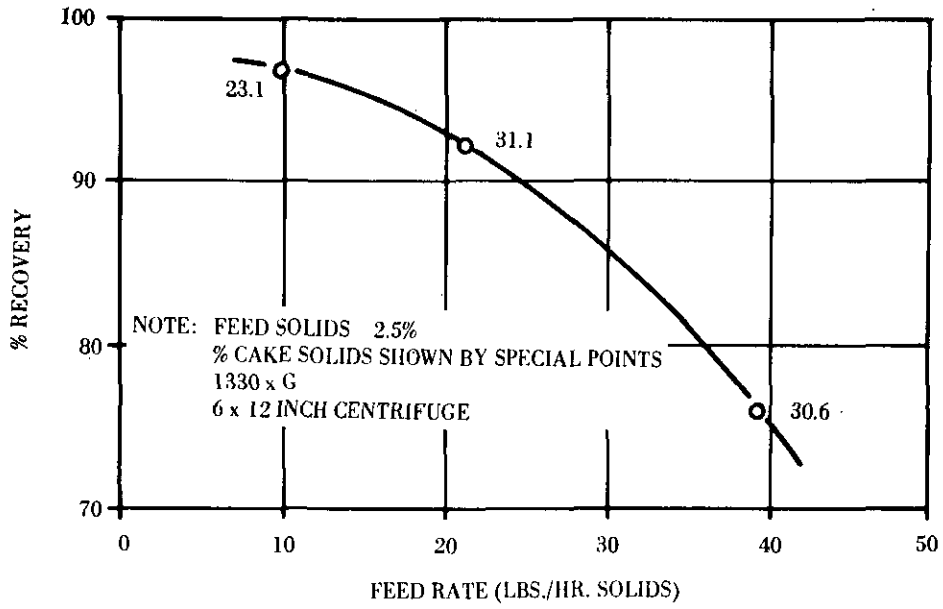
Fig. 21   Centrifuge Studies on Sludge from Lime Precipitation of TFE
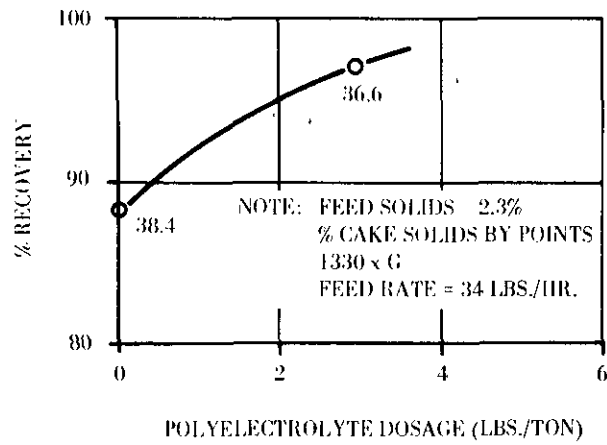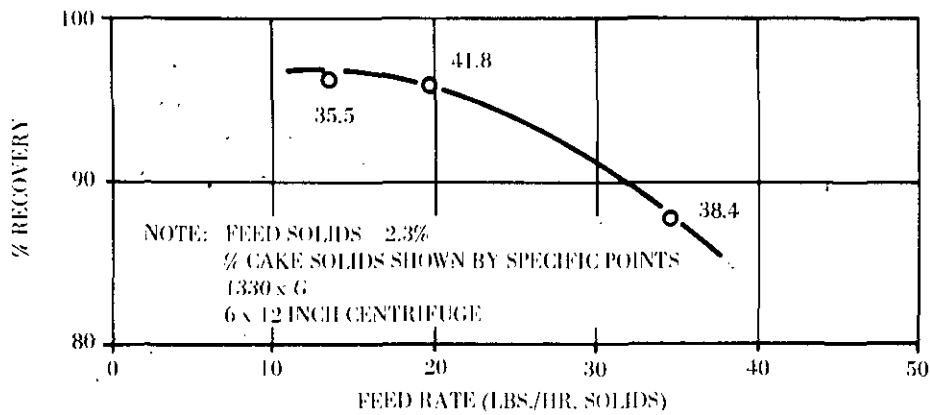
Fig. 22 Centrifuge Studies on Sludge from Lime Precipitation of TFE

The sludges for which test results are shown contained from 2.2 to 2.5 percent solids. The first batch was tested in a large 18- by 42-inch centrifuge. Cake solids increased from 9.5 to 17.7 percent solids on unconditioned sludge as feed rates increased from 65 to 200 lb/hour. Capture of solids was in excess of 92 percent in all cases. With about 0.8 lb of polyelectrolyte coagulant per ton of solids and a feed rate of 202 lb solids/hour, the cake solids were increased to 21.2 percent and the capture to about 97 percent. percent.

Figures 21 and 22 show the results of centrifuging two different batches of sludge in a 6- by 12-inch centrifuge. It was found possible to produce cakes with more than 30 percent solids from unconditioned sludge in both instances. The capture of solids was less than 80 percent in one case. Conditioning of the sludges with about 3 percent of polyelectrolyte increased the capture of solids at the highest feed rates tested to about 95 percent. The percent in the cake decreased from 30.6 to 23.7 in

3-17

the case where the solids capture on unconditioned sludge, Figure 21, had been so poor. This condition was not unexpected, since retention of the lighter solids in the cake would tend to decrease the degree of compaction. With the third sludge, Figure 22, addition of polyelectrolyte did not decrease the solids content of the cake significantly. This was to be expected in view of the excellent capture of solids with the unconditioned sludge.

Summary — Sludges from lime precipitation of TFE can be dewatered satisfactorily for recovery of lime by centrifuging. Solids capture can be controlled by addition of a relatively small amount of polyelectrolyte, approximately 3 pounds per ton of solids. The solids not captured which escape in the centrate have been shown by other investigations to contain a large part of the apatite. Thus, centrifuging constitutes a means of purifying the sludge by accomplishing a high degree of phosphate separation before the solids are recalcined to recover lime. The centrate containing a majority of the phosphate is normally mixed with primary and secondary sludges for dewatering and ultimate disposal.

Activated-Sludge System Employing Alum Removal of Phosphates — Addition of either aluminum or ferric salts to activated-sludge systems results in the removal of phosphates through precipitation reactions that are not well understood. The reactions are generally indicated to occur as follows:

$$*M^{+3} + HPO_4^{-2} \rightarrow \underline{MPO_4} + H^+ \tag{3-5}$$

and excess M^{+3} ion reacts with alkalinity of the wastewater to produce flocculant aluminum or ferric hydroxide.

$$2M^{+3} + 6\ HCO_3^- \rightarrow \underline{2\ M(OH)_3} + 3\ H_2O + 6\ CO_2 \tag{3-6}$$
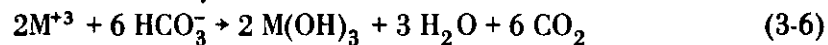
Because the hydroxides formed are highly flocculant, the use of excess aluminum or ferric salts is to be avoided because of detrimental effects on the settling characteristics of the activated sludge.

As was noted in an earlier section, the sludge produced was more in weight but less in volume than normally expected. In fact, because of the favorable effect of coagulant addition on the sludge volume index, resultant waste sludge volumes were found to be slightly less than in the conventional activated sludge system.

Thickening of the alum waste activated sludge was attempted in both the laboratory and the field. When a flotation unit was run at 0.35 gpm/sf and a 5 to 1 recycle to sludge ratio, the sludge thickened to 6.2 percent solids and resulted in a 99 percent recovery of the solids.

Where M^{+3} is either Al^{+3} or Fe^{+3}.

During the course of the pilot plant investigations, separate batches of mixtures of waste activated sludge, including chemical precipitates and primary sludge, were prepared for laboratory study of their filtration characteristics. The primary sludge was collected from a bucket suspended in the digestion section of the Imhoff tank in such a manner to collect the raw primary solids.

Combinations of $FeCl_3$ and $Ca(OH)_2$ (hydrated lime) were investigated as conditioning agents in a number of Buchner funnel tests. After required chemical dosages had been established by this method, a few tests were performed with a 0.1 square foot filter leaf. The conditions and data obtained during the most pertinent leaf tests are summarized in Table 1. It was found that the use of lime and $FeCl_3$ resulted in acceptable filtration. Filter cakes with total solids contents of 22 to 28 percent were obtained. Cakes ranged from 1/8-inch to 3/8-inch thick, were firm, and readily discharged from the media. Filter yields of 3.0 to 3.8 pounds per square foot per hour were calculated.

It was concluded from these studies that these sludge mixtures would be no more difficult to filter than would combinations of raw and conventional activated sludge.

Table 1. Summary of Laboratory Filter Leaf Studies

Sludge characteristics

	1	2	3	4	5	6	7	8	9	10
Test number	1	2	3	4	5	6	7	8	9	10
Total solids, percent	3.8	3.8	3.8	3.3	3.3	3.3	3.1	3.1	3.1	0.5
Mixture, percent primary – percent waste-activated sludge	73-27	73-27	73-27	60-40	60-40	60-40	50-50	50-50	50-50	0-100

Chemical conditioning

$FeCl_3$ dose, percent of total solids	3.2	3.2	3.2	3.8	3.8	3.8	9.0	9.0	9.0	(2)
CaO dose, percent of total solids	7.6	7.6	7.6	9.0	9.0	9.0	17.0	17.0	17.0	(2)
pH	9.8	9.8	9.8	–	–	–	–	–	–	

Test conditions

Media	(1)	(1)	(1)	(1)	(1)	(1)	(1)	(1)	(1)	(1)
Drum submergence, percent	25	25	25	25	25	25	25	25	25	25
Drum speed, minutes per revolution	4	5	6	4	5	8	5	6	8	6
Vacuum, inches of mercury	12	12	12	12	12	12	12	12	12	12

Filter cake

Total solids, percent	28.0	27.6	26.0	22.0	21.7	21.3	20.7	19.6	20.2	10.1
Thickness, inches	1/8	1/8	3/16	1/8	1/8	1/8	1/8	1/8	1/8	5/16
Yield, lb/sq ft/hour	7.56	6.28	5.3	4.55	4.2	3.8	3.9	3.9	3.5	1.78

Filtrate solids

Suspended solids, mg/L	2,866			1,840	1,230			1,560		

1. Wire screens, 60 x 70 mesh twill (approximates filter characteristics of coil media).

2. Polymer dosages of 12 pounds per ton dry solids (0.6 percent by weight) of Dow 1188 plus 12 pounds per ton Hercofloc 821. Used in lieu of $FeCl_3$ and CaO.

Separate samples of the primary sludge and the alum sludge were collected and taken to Bird Machine Company, to be run on a centrifuge. The samples were delivered by a tank truck on the day prior to testing the specific sludge. The primary and waste activated sludges were run on an 18-inch by 42-inch ccf (countercurrent feed) unit at 1,440 x G (2,400 rpm).

The two sludges were run independently. By using basic solids balancing principles developed by the Bird Machine Company, a prediction can be made as to the ability of the centrifuge to dewater various combinations of the sludges when mixed. As an example, if we were to mix the following: two parts of primary sludge by weight, dewatered alone to 30 percent cake solids, and one part of waste activated, dewatered alone to 6 percent cake solids, we could expect the mixture to dewater to about 22 percent cake solids.

The waste-activated sludge contained feed solids of approximately 0.6 percent by weight. Cake solids were generally in the range of 5.0 percent to 7.5 percent by weight. Feed rates without the addition of chemical flocculation varied from 20 to 105 pounds per hour of dry feed solids with recoveries varying from 87 percent to 98 percent as shown on Figure 23. At a feed rate of approximately 82 pounds per hour of dry feed solids, the recovery was increased from 92 percent to above 98 percent with the addition of 2 to 4 pounds of polymer per ton of dry feed solids.

The batch of primary sludge submitted for testing by centrifuge was not typical, since it contained only 1.2 percent solids instead of the usual 4 to 6 percent. Cakes containing from 23 to 33 percent solids were obtained but solids capture was less than 70 percent on unconditioned sludge and less than 80 percent on polyelectrolyte-conditioned sludge.

Summary — Dewatering of combinations of primary and activated sludge was best accomplished by vacuum filtration. Centrifuging was not acceptable because of poor solids recovery.

Conclusions

A comparison of the data on Figure 13 with those on Figure 19 shows that the phosphorus concentration in the effluent can be reduced to 0.5 mg/L inorganic phosphorus and 1.0 mg/L total phosphorus by either lime treatment of TFE or alum additions to the activated sludge process.

The pH required to attain these phosphorus concentrations with the addition of lime was 10.8. The alum dose required to assure the same
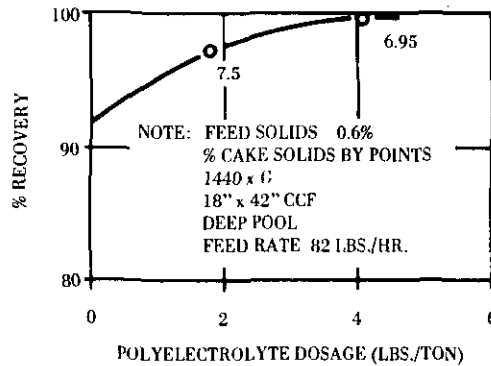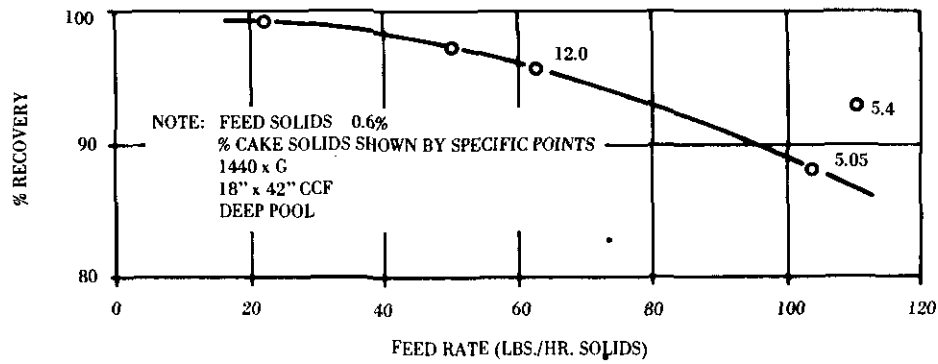
Fig. 23 Centrifuge Studies on Waste-Activated Sludge

concentration of phosphorus in the activated sludge process effluent is a 28 to 1 ratio of alum to inorganic phosphorus concentration, or 2.5 parts of aluminum for each part of phosphorus.

The lime sludges can be classified for lime recovery or be dewatered for ultimate disposal by the use of a centrifuge. The sludges produced through the use of alum for removal of phosphorus were dewatered better by the use of a vacuum filter than a centrifuge.

3-21

CHAPTER 4

NITROGEN REMOVAL

Because nitrogen is the major inorganic nutrient (fertilizer) required in the growth of algae and the relatively high concentration of it that occurs in domestic wastewaters, it is reasonable to assume that removal will be required in many locations.

Removal of fixed forms of nitrogen from wastewaters is somewhat more complicated than that of phosphorus because of the several forms in which it occurs, i.e., as ammonium, nitrite or nitrate ions, or as organically bound ammonia in the form of protein matter. Fortunately, nitrogen occurs in domestic wastewaters largely as ammonium ion or its derivatives — urea and protein. Effective nitrogen removal is dependent upon converting as much of the nitrogen as possible either to ammonium ion or to nitrate ion.

Removal as Ammonia by Air Stripping

During biological treatment, ammonium ions are released from urea through hydrolysis and from proteins by biological oxidation of the organic matter according to the following equations.

$$CO(NH_2)_2 + 2 H_2O \xrightarrow{\text{Bact}} 2 NH_4^+ + CO_3^= \tag{4-1}$$

$$Org\text{-}N + O_2 \xrightarrow{\text{Bact}} NH_4^+ + HCO_3^- \tag{4-2}$$
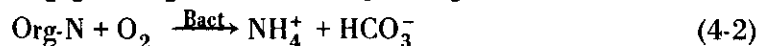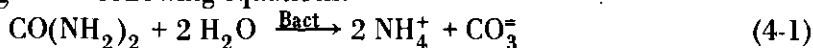
With the high-rate trickling filter or high-rate activated-sludge methods of treatment, all of the fixed forms of nitrogen except for small residual amounts of organic nitrogen, are converted to ammonium ion, consequently the effluents from both processes are especially amenable to reduction of their nitrogen content by removal of ammonia.

At the time the studies were initiated at Marlborough, air stripping of ammonia appeared to be a practical method in light of reports on studies at Lake Tahoe. Lime treatment to remove phosphates at elevated pH levels and subsequent stripping of ammonia as shown in equation 4-3 appeared to be an economical solution to the nutrient removal problem.

$$NH_4^+ + OH^- \rightarrow NH_3 \uparrow + H_2O \tag{4-3}$$
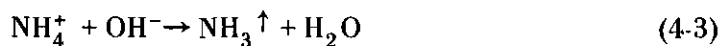
Accordingly, the ammonia stripping device shown as Figure 24 was purchased and installed at Marlborough to accept and treat the high pH effluent from the lime precipitation unit shown as Figure 10. The ammonia stripper was operated in a horizontal position rather than vertical because of its small size and ease of sampling. A prototype would be from 25 to 30 feet high. The suppliers of the equipment made assurances that results obtained

Fig. 24   View of Ammonia Stripping Unit

with the unit in a horizontal position would be comparable to those obtained in a vertical position. The air flow was countercurrent to that of the water.

Although the stripping unit was available for operation by December 1968, it was impossible to operate it at air temperatures below 40 deg F because of icing problems within the unit.

Five operational variables were considered as having possible significant effects upon the efficacy of the ammonia stripping unit. These were as follows and data were obtained to evaluate the effect of each.

1.   Air to wastewater ratio

2.   Air temperature

3.   pH of wastewater

4.   Relative humidity

5.   Hydraulic loading

A total of 38 runs was made starting on March 10, 1969 and continuing through August 25. Relative humidity affects the temperature drop of the

wastewater through the unit, and this drop affects stripping efficiency; however, other variables seemed to have more effect upon performance. Hydraulic loadings below 2 gpm/sf have no effect upon performance. The influence of the first three items shown above is presented on Figure 25. Airflows of at least 550 cu ft/gal were necessary to insure removals of 85 percent of the ammonia, and about 800 cu ft/gal was needed to accomplish removals in excess of 93 percent, under summer conditions.

Air temperatures have a most important effect on ammonia removals. Since air and wastewater temperatures rise and fall somewhat simulatneously, the measured effect must be considered to be a combined one. The data on Figure 25 indicate that removals above 85 percent would be dependent upon having air temperatures above 70 deg F. At temperatures below 70 deg F, removals decrease to about 60 percent at 40 deg F. Below 40 deg the process ceases to be dependable because of icing problems, if wastewater temperatures are low.

The effect of pH upon ammonia removal also is shown on Figure 25. The most astonishing revelation of the data is that pH levels of 11.5-12 did not insure high removals of ammonia. The reason for this paradox is not known because air-water ratios in excess of 500 and temperatures in excess of 70 deg F were used in all cases when pH was above 11.5. In general, however, the effect of pH is believed to be represented by the line shown on the figure.

During the course of the ammonia removal studies, severe scaling of the media in the stripping tower occurred. This problem has also been encountered at Lake Tahoe and Washington, D.C. It represents a most serious limitation of the process.

Summary — Removal of ammonia nitrogen by air stripping is not practical except when air and wastewater temperatures are above 60 deg F. Severe scaling of media can be expected at all pH levels above 10.5, and below pH 10.5 efficiency of removal is less than 75 percent. Under the most ideal conditions and with pH levels of 11.5 to 12, approximately 2 mg/L of ammonia nitrogen will remain in the effluent. Total nitrogen in effluents from ammonia strippers can be expected to range from 4 to 6 mg/L, depending upon the amount of organic nitrogen in the influent.

A serious limitation of air stripping of ammonia is related to the potential air pollution problem it creates. Serious question can be raised as to how long such a practice would be tolerated.
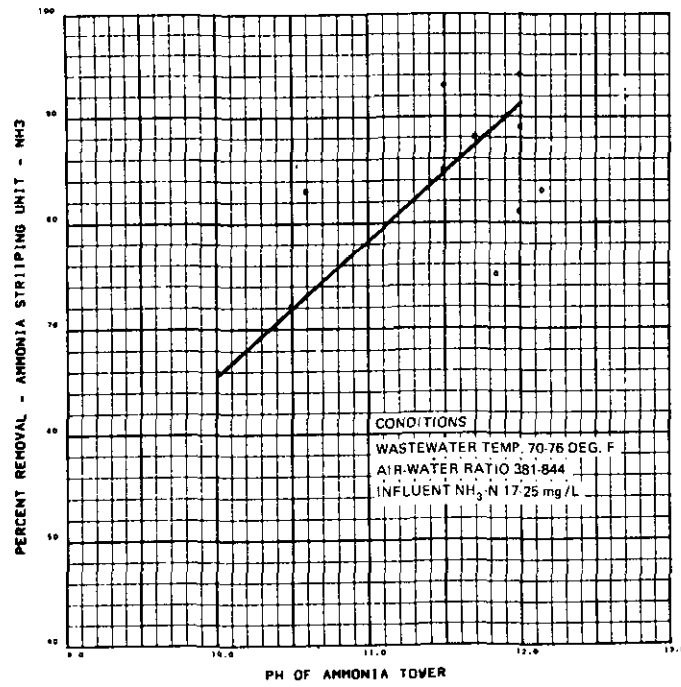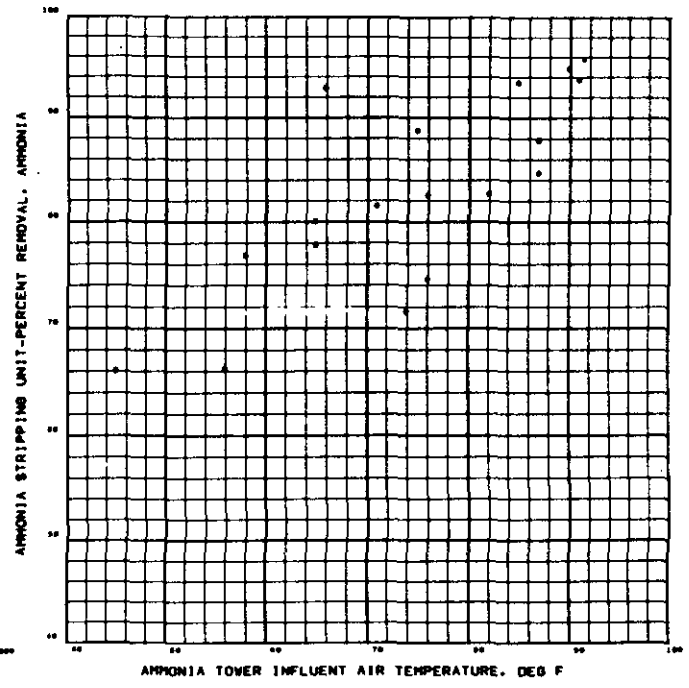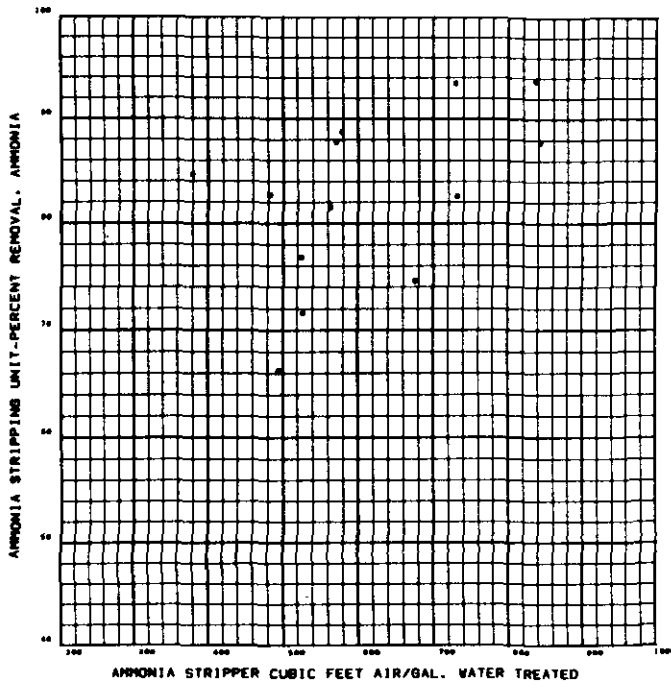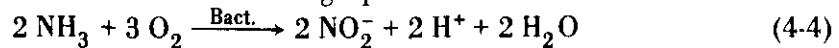
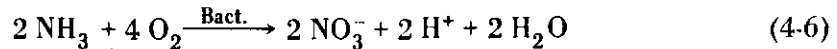Fig. 25   Effect of Certain Operational Variables upon Ammonia Stripping
Tower Performance

Removal by Nitrification and Denitrification

Nitrification followed by denitrification is a major pathway through which fixed forms of nitrogen are restored to elemental or gaseous nitrogen. It is nature's way and because the end product is nitrogen, no threat of air pollution is created.

Nitrification — During biological treatment of wastewaters, organic forms of nitrogen and urea are converted to ammonia, actually ammonium ion, by the action of saprophytic bacteria as shown in Equations 4-1 and 4-2. In many biological treatment systems, particularly during the warm summer months, part or even all of the ammonia may be oxidized to nitrites and nitrates, in accordance with the following equations.

$$2\,NH_3 + 3\,O_2 \xrightarrow{\text{Bact.}} 2\,NO_2^- + 2\,H^+ + 2\,H_2O \qquad (4\text{-}4)$$

$$2\,NO_2^- + O_2 \xrightarrow{\text{Bact.}} 2\,NO_3^- \qquad (4\text{-}5)$$

or

$$2\,NH_3 + 4\,O_2 \xrightarrow{\text{Bact.}} 2\,NO_3^- + 2\,H^+ + 2\,H_2O \qquad (4\text{-}6)$$

Experience has demonstrated that complete oxidation of all of the ammonia in wastewaters cannot be guaranteed in normal activated sludge or trickling filter plants under winter conditions such as occur in Massachusetts. In order to ensure complete oxidation at all times, it has been proposed that nitrification should be accomplished in a completely separate biological system following conventional biological treatment. In this manner the nitrifying bacteria responsible for nitrification are isolated and not subject to competition with the saprophytic bacteria.

The biological treatment sequence proposed for systems designed to accomplish nitrogen removal by the nitrification-denitrification route is shown as Figure 26. It will be noted that three separate stages of treatment are considered essential. This is because specialized bacterial cultures are required in each stage. In the pilot study the first stage of treatment (carbonaceous oxidation or BOD removal) is accomplished by trickling filters.

The proposed second stage of treatment involves use of a system exactly like that used in the conventional activated-sludge process, with the additional provision that plug flow characteristics should be maintained in the aeration tank. For our purposes, the activated sludge system used in the phosphate removal studies, shown on Figures 15 and 16, was ideally suited to accomplish nitrification, since the aeration tank had been modified to provide four aeration compartments to be operated in series.

4-5

```
┌──────────────┐      ┌──────────────┐      ┌──────────────────┐
→│ BOD REMOVAL  │──→───│ NITRIFICATION│──→───│ DENITRIFICATION  │──→
└──────────────┘      └──────────────┘      └──────────────────┘
```

Fig. 26 Biological Treatment Sequence for Nitrogen Removal by
Nitrification-Denitrification

The activated-sludge system was converted from its original use to a nitrification unit, employing settled TFE as feed, on October 7, 1969 and continued on that basis until April 30, 1970. The settled TFE used to feed the unit had characteristics shown on Figure 27. Ammonia, organic nitrogen, suspended solids and BOD all varied considerably during the 200-day period of the study. These great variations were related to variations in dry and wet weather wastewater flows and Imhoff tank performance, as discussed in the Appendix.

During the operation of any biological system, a considerable number of variables must be reckoned with, besides the quality of the feed. Most of them can be controlled except for temperature. For systems involving fluidized suspensions of microorganisms, the following variables are of concern:

1. Temperature

2. pH

3. Dissolved oxygen

4. Detention time or hydraulic loading

5. Mixed liquor suspended solids or bacterial numbers

6. Sludge volume index

7. Rate of sludge return
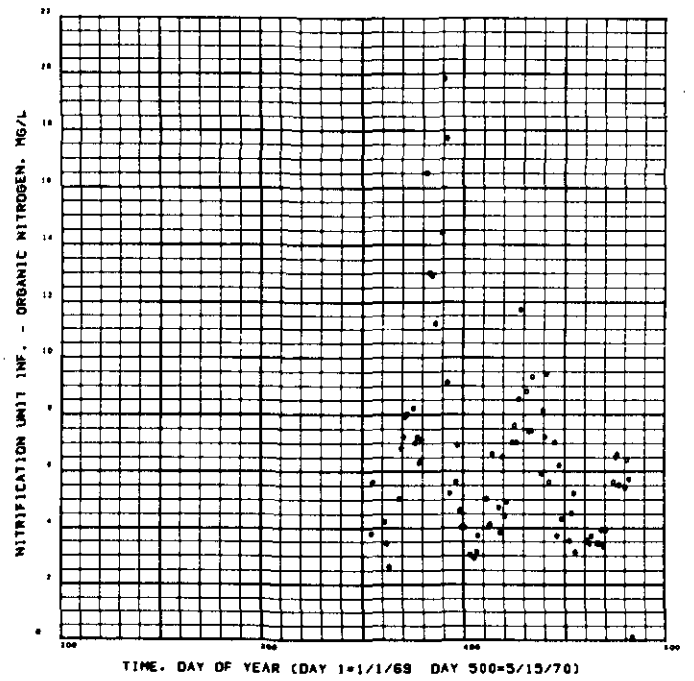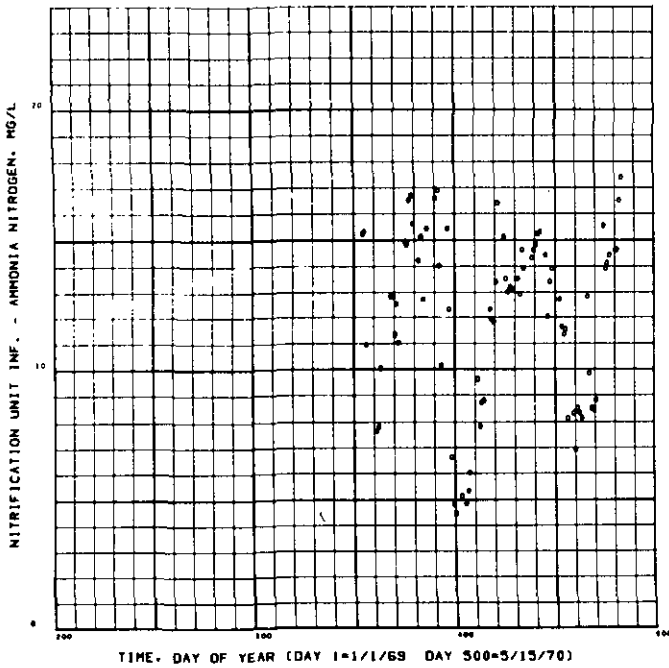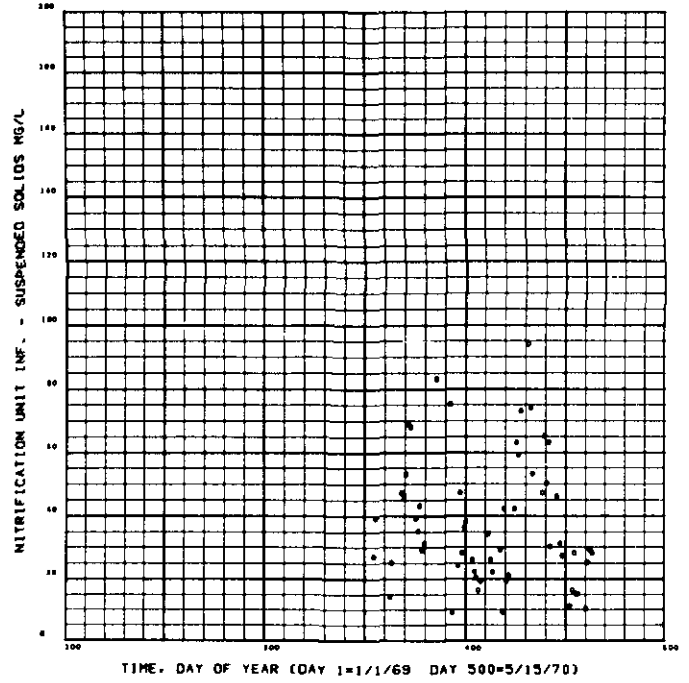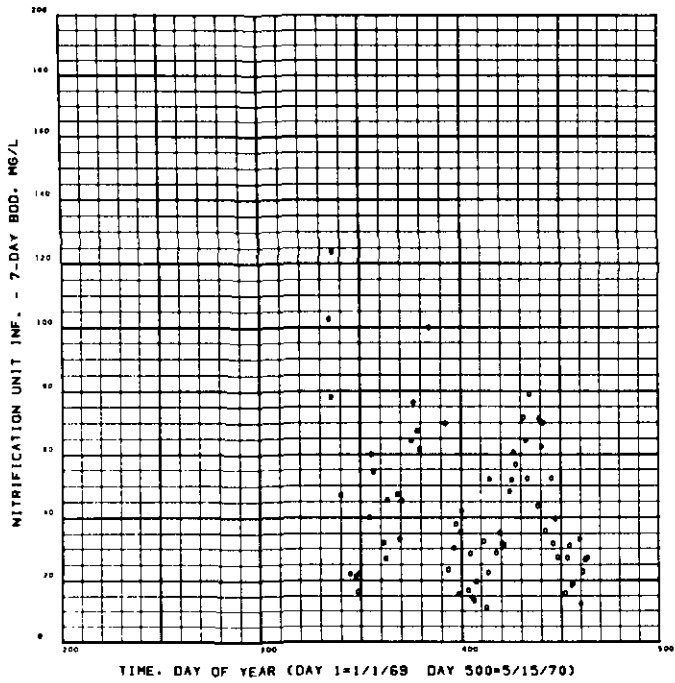
Fig. 27 Characteristics of Settled Trickling Filter Effluent Used to

Feed Nitrification System

4-7

A summary of the above variables as they occurred chronologically during the course of the studies on nitrification is given on Figures 28 and 29.

Temperatures in the nitrification unit varied from a high of 18 deg C (64 F) at the start of the studies on October 7 to lows of 4 deg C (39 F) on several days between January 15 and March 28, 1970. By late April temperatures of 10 deg C (50 F) or more were common. The great day-to-day variations were a function of weather conditions, particularly rainfall, and its effect on wastewater temperatures.

During the early days of the nitrification study, some trouble was encountered in keeping the pH in a satisfactory range for nitrification. This was corrected by adding sodium hydroxide on a drip-feed basis. Following the 330th day, November 25, the pH was kept in the range of 7 to 9, except for a few days. A notable instance was from day 390 to 400, January 24 to February 4, when too much hydroxide was added and pH levels above 9 occurred.

Dissolved oxygen conditions were normally above 4 mg/L but on several occasions they were less. The values below 4 were a result of a limited air supply due to one of the two air compressors being out of service and varying demand for compressed air at other locations

At the start of the nitrification studies, the hydraulic load on the system was adjusted to provide a 15.5 hour detention time in the aeration tanks. This was done purposely to facilitate the development of a nitrifying sludge. Early attempts to decrease the detention time resulted in erratic operation but by December 15 a 12-hour period was proven successful, even with temperatures of 6 deg C and less in the aeration tank. Detention times as low as 5 hours were used from March 15 to 25 with good results being obtained.

From day 300 to 460, October 25 to April 5, mixed liquor settleable solids were kept in the range of 2,500 to 4,000 mg/L. The decrease to 2,000 mg/L on day 461 was due to a diversion of sludge to another project for seeding purposes.

The SVI of the sludge generally ranged between 90 and 140, during the major part of the time, and is quite satisfactory.

The rate of sludge return normally ranged from 100 to 200 percent of the settled TFE entering the aeration tanks. These rates were abnormally
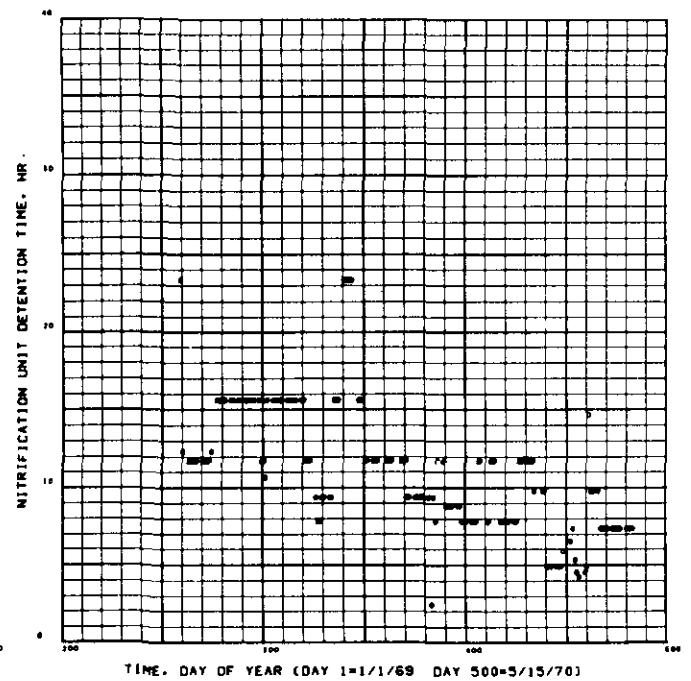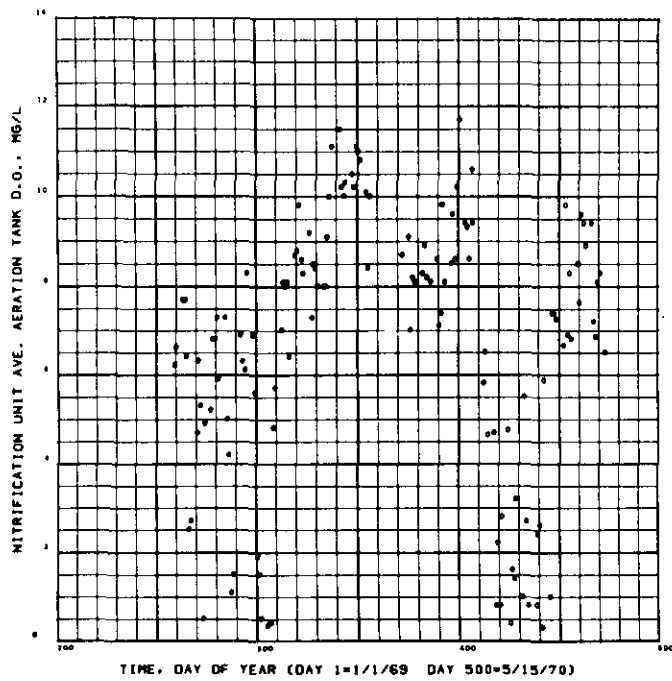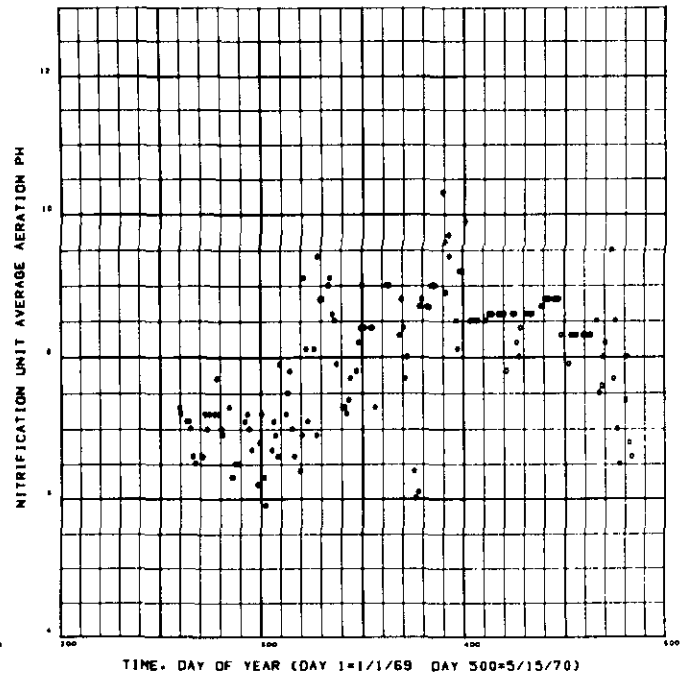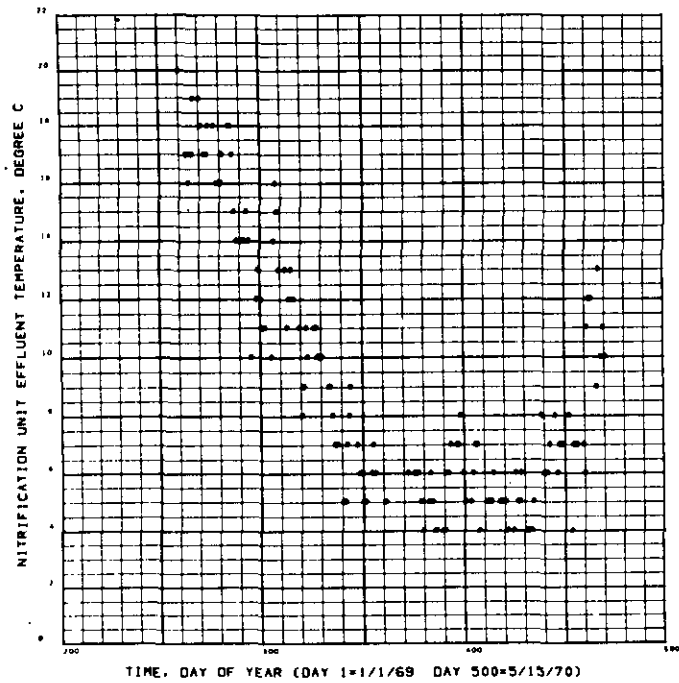
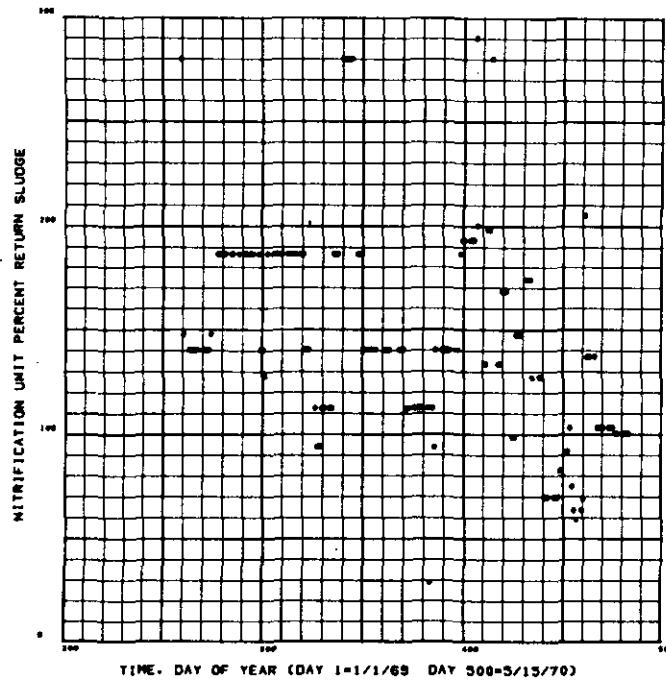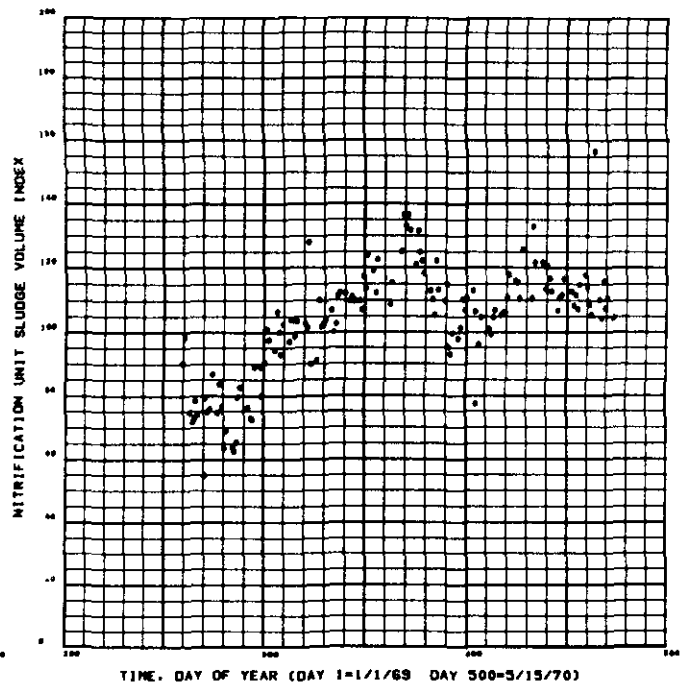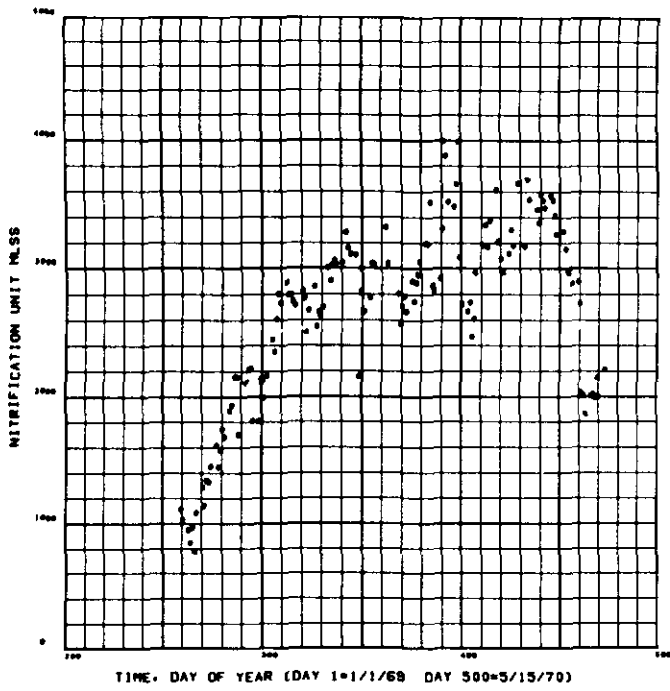Fig. 28　Operational Variables of Importance in Control of Biological Systems

4-9

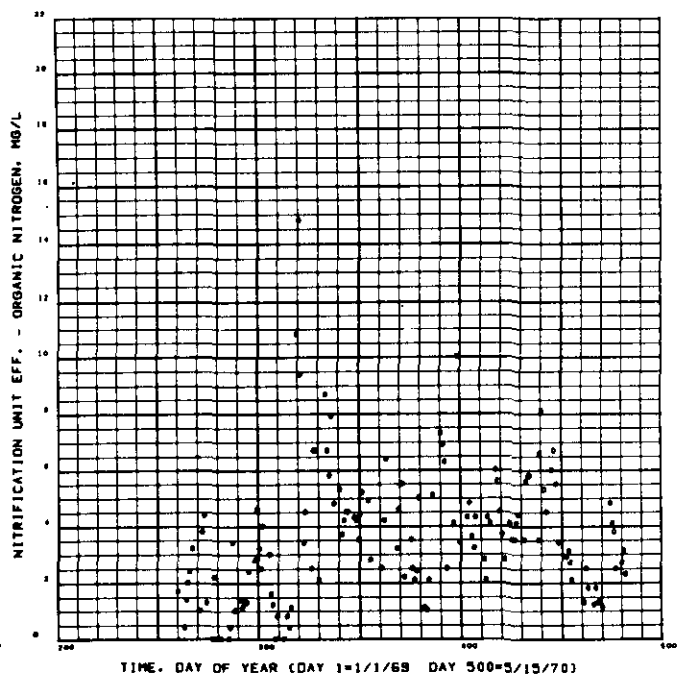Fig. 29 Operational Variables of Importance in Control of Biological Systems
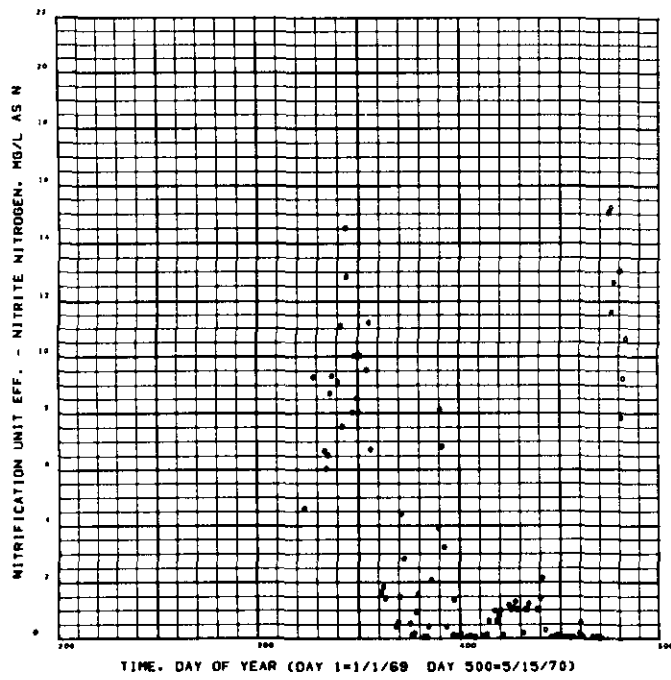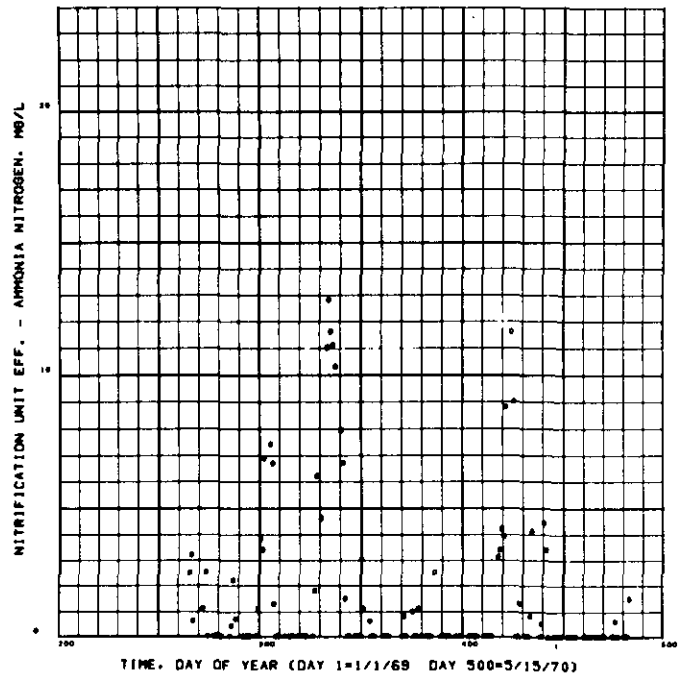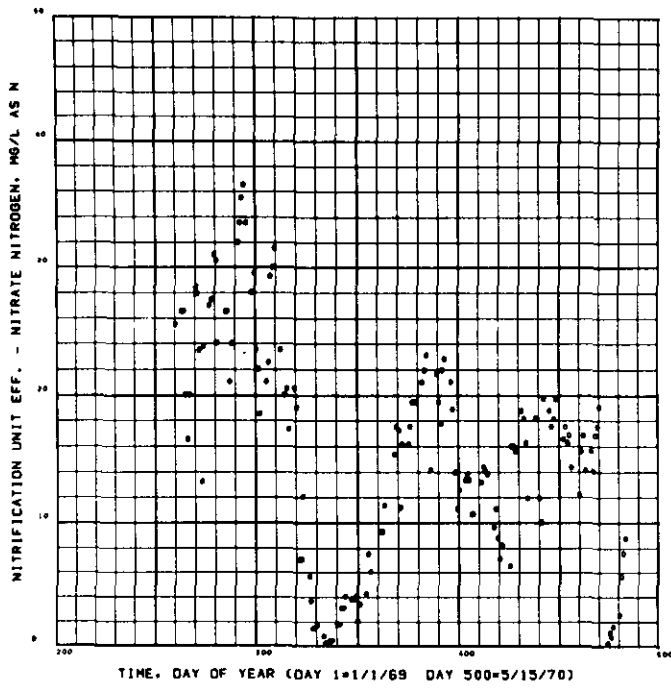
4-10

Fig. 30 Nitrogenous Compounds in Nitrification System Effluent

4-11

high but were necessary because of limitations of the air-lift system used. Attempts were made to use a pumped return but freezing problems made its use unreliable.

The effectiveness of the nitrification system in accomplishing its purpose is shown by the data on Figure 30, with respect to nitrogenous compounds in the effluent.

A high degree of nitrate formation occurred at all times during the study, except for the period from day 320 to 360 and after 471 and on days when low DO conditions prevailed. The first lapse occurred as a result of a drop in detention time from 15.5 to 9.5 hours. The temperature also fell rapidly during this period. Although nitrate formation suffered greatly, formation of nitrites was still considerable and effluent ammonia nitrogen exceeded 5 mg/L on only 8 days of the 40. The failure to produce normal levels of nitrates after day 472 did not result in a breakdown of the process, as nitrites continued to be formed and very little ammonia escaped in the effluent.

During the course of the nitrification studies, ammonia nitrogen was reduced to less than 2 mg/L in the effluent on all but 25 days. On at least 8 days the high ammonia levels were related to low dissolved oxygen conditions that limited oxidation of ammonia to nitrites and nitrates.

Organic nitrogen varied considerably in the effluent and was undoubtedly related to a great extent upon the considerable amounts of suspended solids that escaped as shown on Figure 31. A reduction in loss of suspended solids, or their removal in the subsequent denitrification unit, will, undoubtedly reduce organic nitrogen to very low levels. Other investigators have shown values of less than 1 mg/L on filtered samples.

The BOD data given on Figure 31 indicate that values of less than 10 mg/L will normally occur. Values in excess of 20 mg/L were correlated with high suspended solids in the effluent.

Figure 32 presents data to show the influence of dissolved oxygen and temperature upon nitrification as measured by residual ammonia nitrogen in the effluent. When the data from day 320 to 360 and other abnormal days are excluded, the influence of dissolved oxygen appears to be as indicated by the curve shown. This demonstrates that dissolved oxygen levels of 1 mg/L or more are essential to ensure essentially complete nitrification. The data on temperatures indicate that the process becomes relatively unreliable at
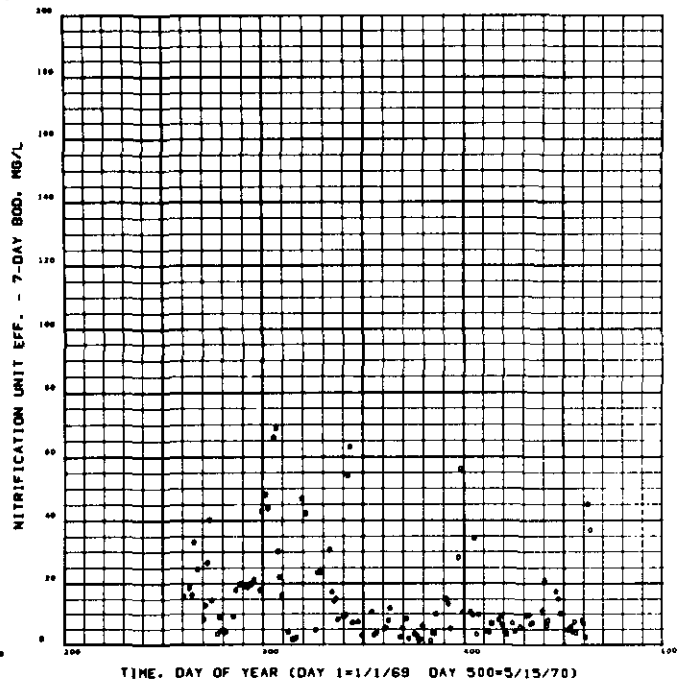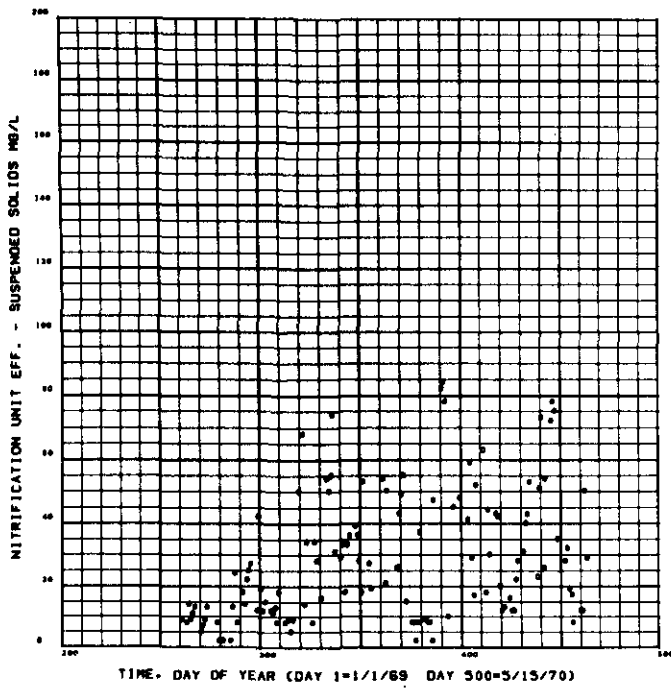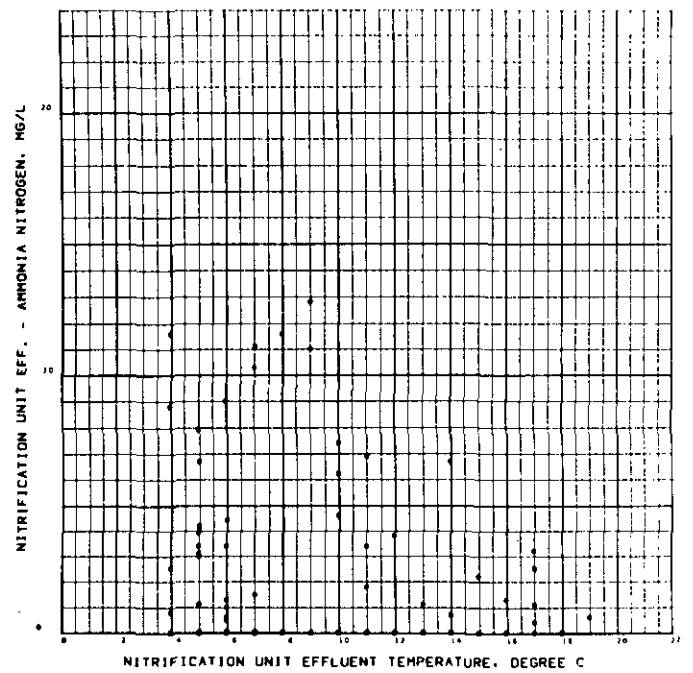
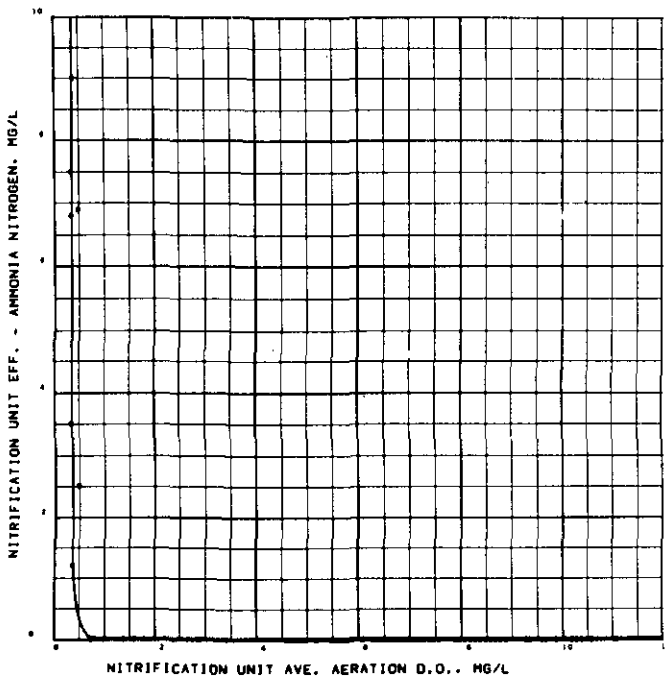**Fig. 31 Effluent Quality of Nitrification System**

Fig. 32 Influence of Dissolved Oxygen and Temperature Upon Nitrification

4-13

temperatures of 6 deg C (43 deg F) or less. Fortunately, very few wastewaters approach such low temperatures.

Laboratory Studies of Nitrification Kinetics — The laboratory studies were concerned with determining the effect of temperature and pH under carefully controlled conditions. The procedure used involved, collection of samples of return sludge from the nitrification pilot plant and of settled trickling filter effluent, determination of suspended and volatile suspended solids in the return sludge, and the adjustment of portions of each to definite pH and temperatures before making the desired mixtures in the aeration units. In most instances, the trickling filter effluent was supplemented with a dilute aqueous solution of ammonium chloride in order to give runs of sufficient duration to obtain three or more experimental values.

The rate of nitrification was determined by measuring residual ammonia nitrogen on grab samples of mixed liquor which were filtered immediately after collection. Dissolved oxygen, pH, and temperature were monitored continuously during the course of each study. Dilute sodium hydroxide was added to control pH as needed. The system of study may have involved some slight loss of ammonia at pH levels above 8.5 but such *losses were too insignificant to be detected from a plotting of the data.* The laboratory aeration unit is shown on Figure 33.
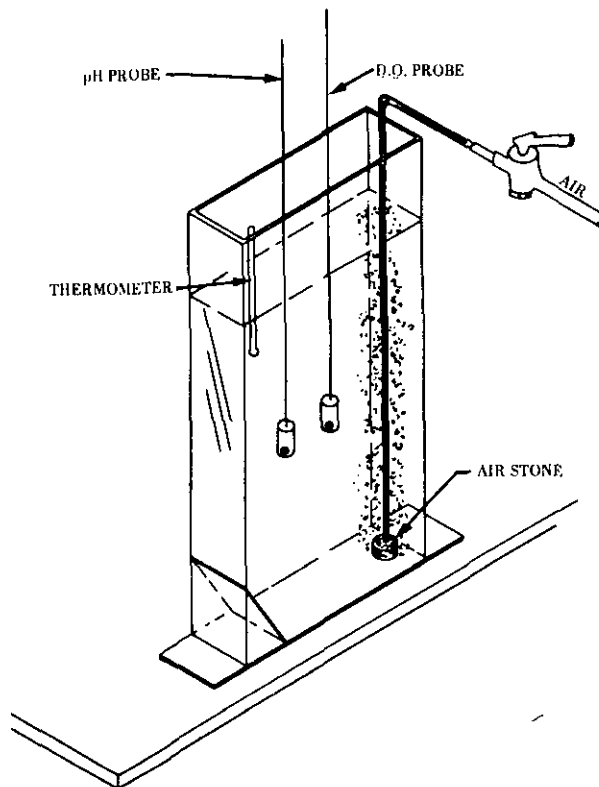

Fig. 33 Laboratory Aeration Unit

It was assumed that the relative population of nitrifiers in the total MLVSS concentration for the duration of the study remained constant. It is felt that this assumption was justified because of the long duration of the pilot studies run under the same conditions, employing settled trickling filter effluent as feed stock.

The nitrification studies were conducted with MLVSS concentrations within the range of 800 to 6,000 mg/L.

A sample of two of the experiments run at the same pH and temperature conditions but with two different mixed liquor volatile suspended solids is shown on Figure 34. It was observed that the time to completely nitrify the same amount of ammonia per gram of MLVSS was constant given the same environmental conditions, allowing direct comparisons to be made for different MLVSS concentrations in the study and permitting subsequent data to be expressed in terms of mg of ammonia nitrogen oxidized per mg of MLVSS.
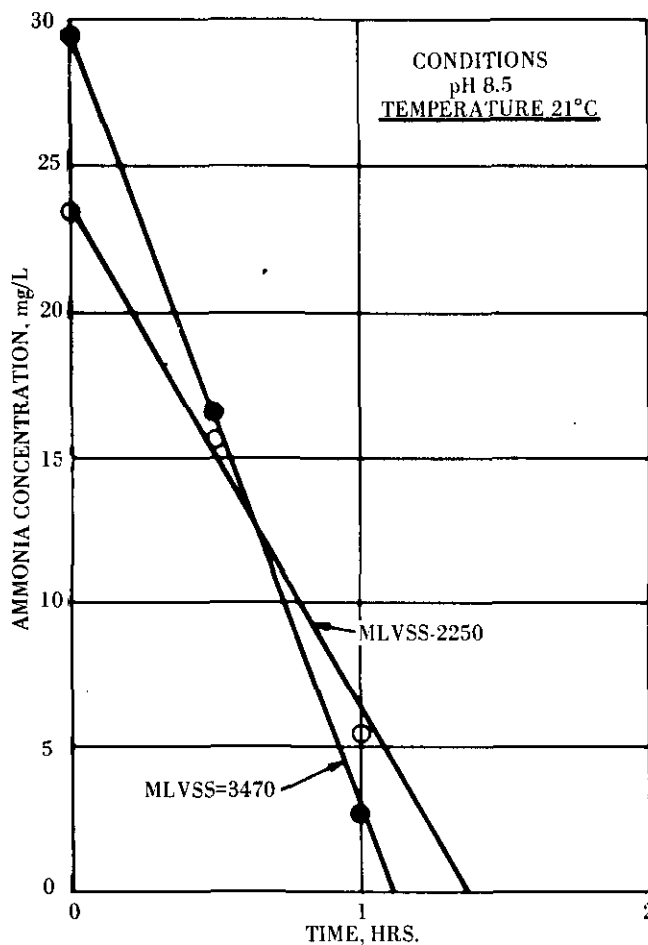
Fig. 34   Effect of Variation in
Mixed Liquor Volatile Suspended Solids

4-15

The augmented ammonia concentrations for the studies varied from 6 to 60 mg/L. The ammonia concentration had to be augmented on many occasions because the time required for complete nitrification of low levels of ammonia was so short that only one or two samples could be analyzed prior to attaining the zero ammonia level. Two sample results are shown on Figure 35. Both of the experiments were conducted at the same pH and temperature conditions. As can be seen from the figure, the slopes of the lines are parallel and constant for all residual concentrations of ammonia regardless of the initial concentration, indicating that nitrification is not inhibited at ammonia concentrations normally found in a domestic wastewater system. Also, adjustment of other data for different ammonia concentrations may be made by constructing a line parallel to the experimental line at the desired ammonia concentration.
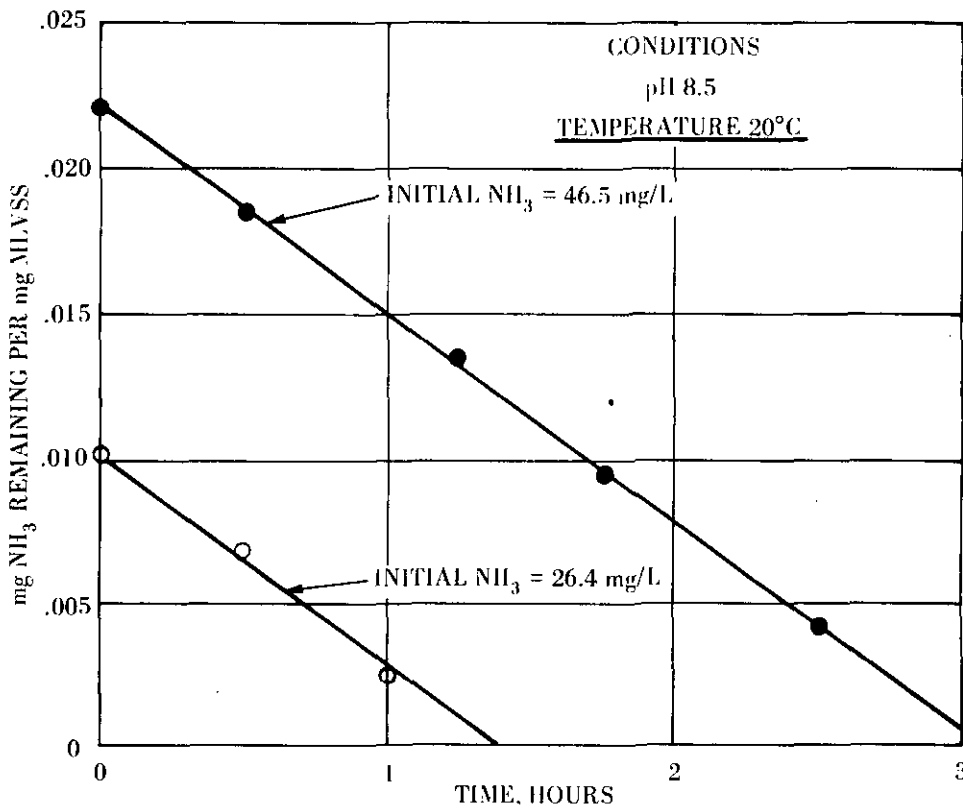
Fig. 35 Effect of Variation in Ammonia Concentration

A special study was made to determine the effect of variable BOD upon the rate of nitrification by nitrifying three different samples. The temperature and pH for all three units were the same. The wastewater in the first unit was primary effluent with a BOD of 110 mg/L, the second unit contained settled trickling filter effluent with a BOD of 45 mg/L, and the

4-16

third unit contained nitrification effluent from the pilot plant with a BOD of 5 mg/L. All samples were augmented with enough ammonium chloride to give a reasonable duration for the test.

Figure 36 shows the results of this special study. Within the limits of the study, there was no apparent inhibition of nitrification for the various BOD concentrations. It should be realized that this study was undertaken to determine the reaction of the nitrifiers to a shock loading of BOD and that any sustained high BOD loading would eventually cause nitrification to cease. Wasting of sludge would have a washing-out effect on the nitrifiers because of the low growth rate exhibited by the nitrifiers as compared to those organisms utilizing carbonaceous BOD and the established fact that an increased BOD loading in a conventional system leads to greater sludge production.
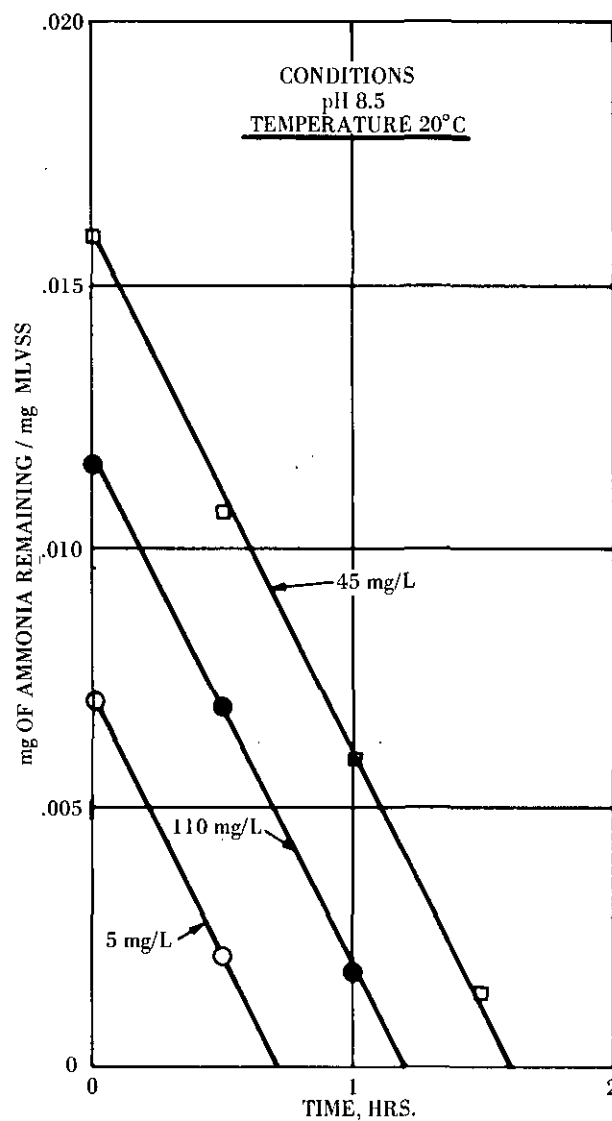
Fig. 36   Effect of Variation in BOD

4-17

The pH range investigated in these studies was from 6.0 to 10.5. The samples were adjusted to the desired pH level and maintained at that level for the duration of the experiment. The ammonia weight per MLVSS weight ratio of the grab samples was plotted against time and all the other variables were noted. The time plot allowed calculation of the exact time of complete nitrification, i.e., complete oxidation of ammonia. A sample graph is presented on Figure 37. This graph shows two sample results both of which were obtained at a temperature of 20 deg C. The pH of one sample was 8.5 and the other 6.5. The figure also shows an adjusted line to compensate for different initial concentrations of ammonia.

Three factors are immediately evident from the preceding figure.

1. There was no apparent initial uptake of ammonia by the nitrifiers.

2. There was no lag time involved in the rate of nitrification.

3. The rate was uniform and constant for the entire length of the experiment. This indicates that the nitrifiers work at maximum efficiency at all times independent of the residual concentration of ammonia.
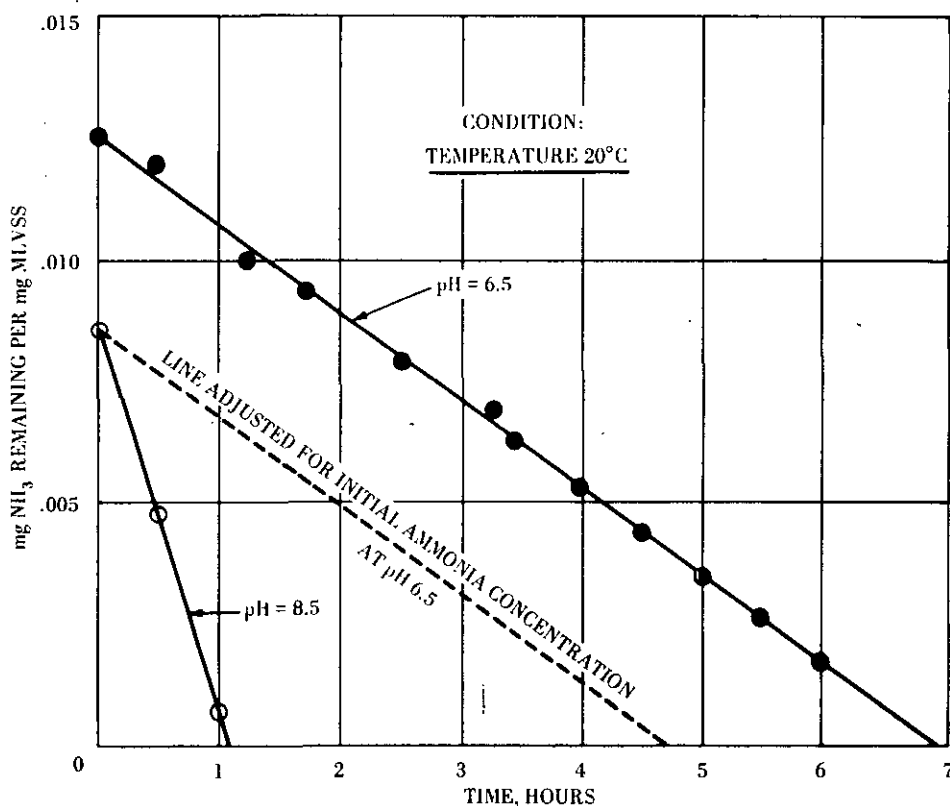
Fig. 37   Effect of Variation in pH

Our studies indicate an optimum pH for nitrification to be 8.4. Figure 38 indicates that 90 percent of the maximum rate occurs in the range of 7.8 to 8.9 and that outside the ranges of 7.0 to 9.8 less than 50 percent of the optimum rate occurs.
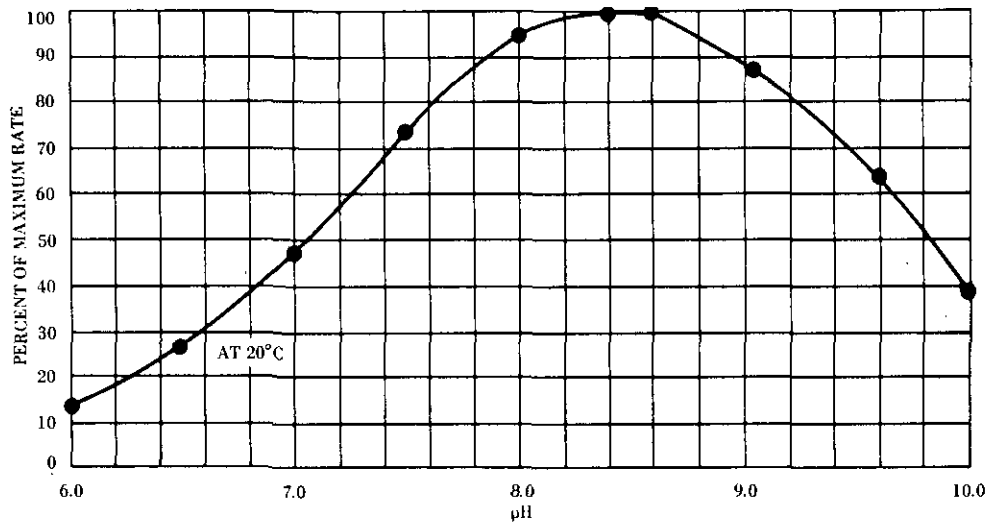
Fig. 38   Percent of Maximum Rate of Nitrification at
Constant Temperature Versus pH

The temperature studies covered the range from 5 deg C to 30 deg C, and nitrification occurred at all temperatures investigated. The rate of nitrification increased with temperature throughout the full range. Figure 39 indicates the straight-line relationships for two sample experiments run at different temperatures but the same pH. One adjusted line is shown to offset
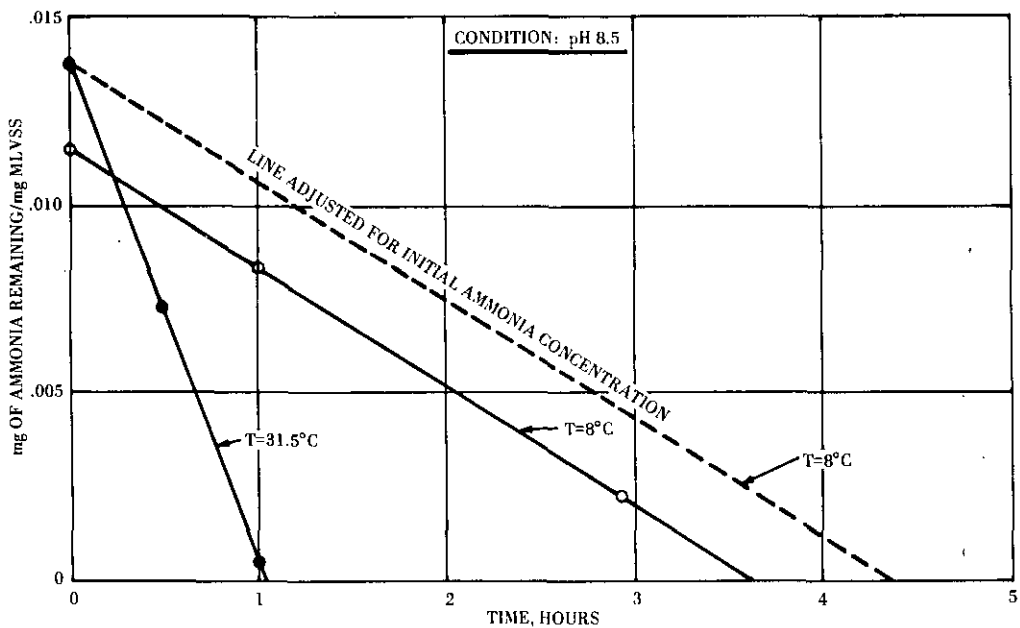
Fig. 39   Effect of Variation in Temperature

4-19

the initial ammonia concentration difference. There was no lag period observed nor any decrease in the rate of nitrification as the residual ammonia concentration decreased.

The relationship of the rate of nitrification at all temperatures studied to the rate at 30 deg C is indicated on Figure 40. Since 30 deg C is a very high sewage temperature for all but the most southerly states in the United States, a summary of relative rates in terms of other maximum temperatures is as shown in Table 2, based upon the data of Figure 40.
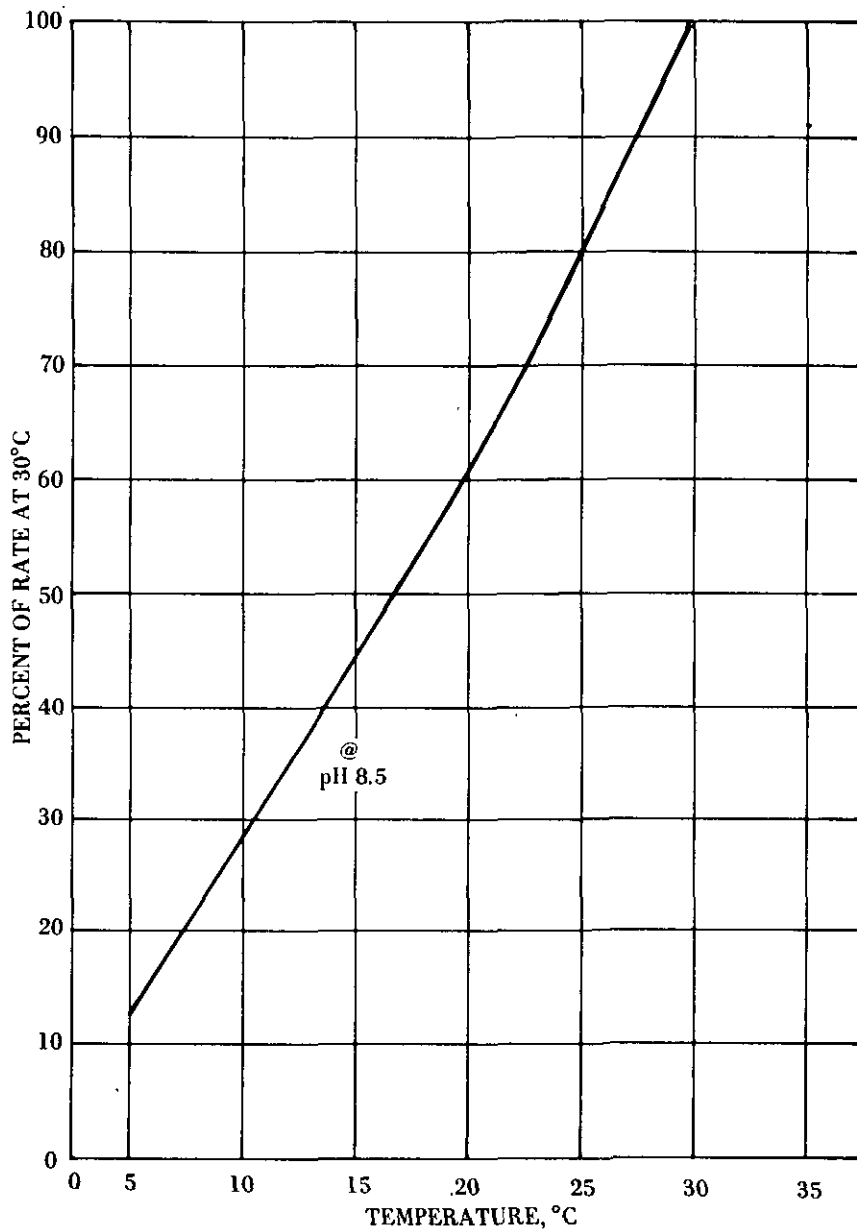
**Fig. 40   Rate of Nitrification at All Temperatures
Compared to the Rate at 30 Degrees C**

Table 2. Relative Rates of Nitrification at Various Temperatures

Sewage Temperature deg C					
30	25	20	15	10	5
100	80	60	48	27[1]	12[1]
	100	75	60	34	16[1]
		100	80	45	21

1. Abnormal temperatures for maximums stated.

These data indicate, on the basis of temperature alone and the most adverse conditions considered possible, that up to five times the detention time may be needed to accomplish complete nitrification in the winter than in the summer. However, temperature effects can be overcome to a considerable degree by increasing mixed liquor suspended solids and adjustment of pH to more favorable levels. Optimum design for complete nitrification will depend upon the best combination of aeration tank capacity, mixed liquor suspended solids, and pH for winter operating conditions. Under summer conditions, operation will be possible at less favorable pH levels and lower mixed liquor solids.

Discussion — When all of the above information is evaluated, rates of nitrification can be computed. The rate of nitrification has been defined as the weight ratio of ammonia nitrogen oxidized per day to the mixed liquor volatile suspended solids.

The rates for any pH within the range of 6.0 to 10.5 are shown on Figure 41. All of the rates are for a temperature of 20 deg C. As can be seen from the figure, the rate varies from a maximum of 0.185 g NH_3 nitrified per day per g MLVSS at a pH of 8.4 to a minimum of 0.020 g NH_3 nitrified per day per g MLVSS at a pH of 6.0.
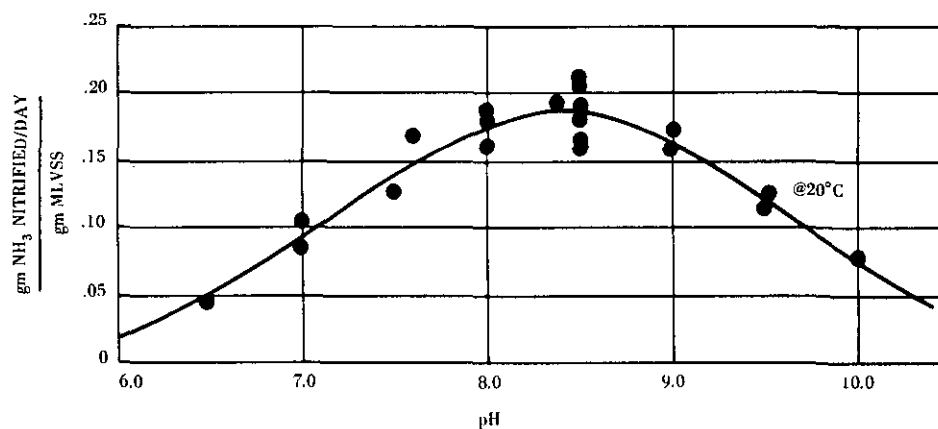
Fig. 41 Rate of Nitrification Versus pH at Constant Temperature

4-21

The results obtained in this study with respect to pH show good correlation with the results on work performed by others.

Our results on temperature effects are opposed to what was observed at Ann Arbor, Michigan, and reported by Borchardt* as shown on Figure 42. Borchardt's low temperature observations were made in extended aeration studies by measuring the ammonia and nitrate nitrogen in the effluent of the units. It is our opinion that the apparent effects of temperature were observed because the units were not being stressed to their limit of nitrification at the higher temperatures and complete nitrification was being obtained in less time than the detention time of the units. As the temperature decreased, the time required to obtain complete nitrification approached the detention time of the units and when the temperatures dropped low enough, the time required for complete nitrification exceeded the detention time and this lesser percent of nitrification was noted. Our results are from units which were stressed to their capacity at all times, and indicated an immediate drop in efficiency as the temperatures decreased, which is in agreement with results reported by Sawyer and Rohlich.**

Figure 43 gives the expected rate of nitrification compared to temperature for various selected pH conditions. This figure allows the computation of the time required for complete nitrification at any MLVSS concentration, ammonia concentration, temperature, and pH. For example, complete nitrification can be obtained in 1.8 hours contact with a MLVSS of 1,450 mg/L, average BOD feed of 50 mg/L, pH of 8.4, 20 deg C temperature, and an ammonia feed concentration of 20 mg/L. If the temperature were reduced to 10 deg C and the pH reduced to 7.0, the required time would increase to 9.45 hours.

The curve at the optimum pH of 8.4 was determined from information gathered during the course of the study. The curves for 75 percent and 50 percent of the optimum rates were computed from the rates experienced at pH 8.4. The pH values for the 75 percent and 50 percent curves were obtained from Figure 38.

From a practical standpoint, Figure 43 indicates that if the nitrification system were run at 50 percent of the optimum conditions, the time required

*Borchardt, J. A., "Nitrification in the Activated Sludge Process", Bull. "The Activated Sludge Process". The University of Michigan, Division of Sanitary and Water Resources Engineering, Ann Arbor, Michigan.

**Sawyer, C. N. and Rohlich, G. A., "The Influence of Temperature upon the Rate of Oxygen Utilization by Activated Sludge". Sewage Works J., 11, 946 (1939).
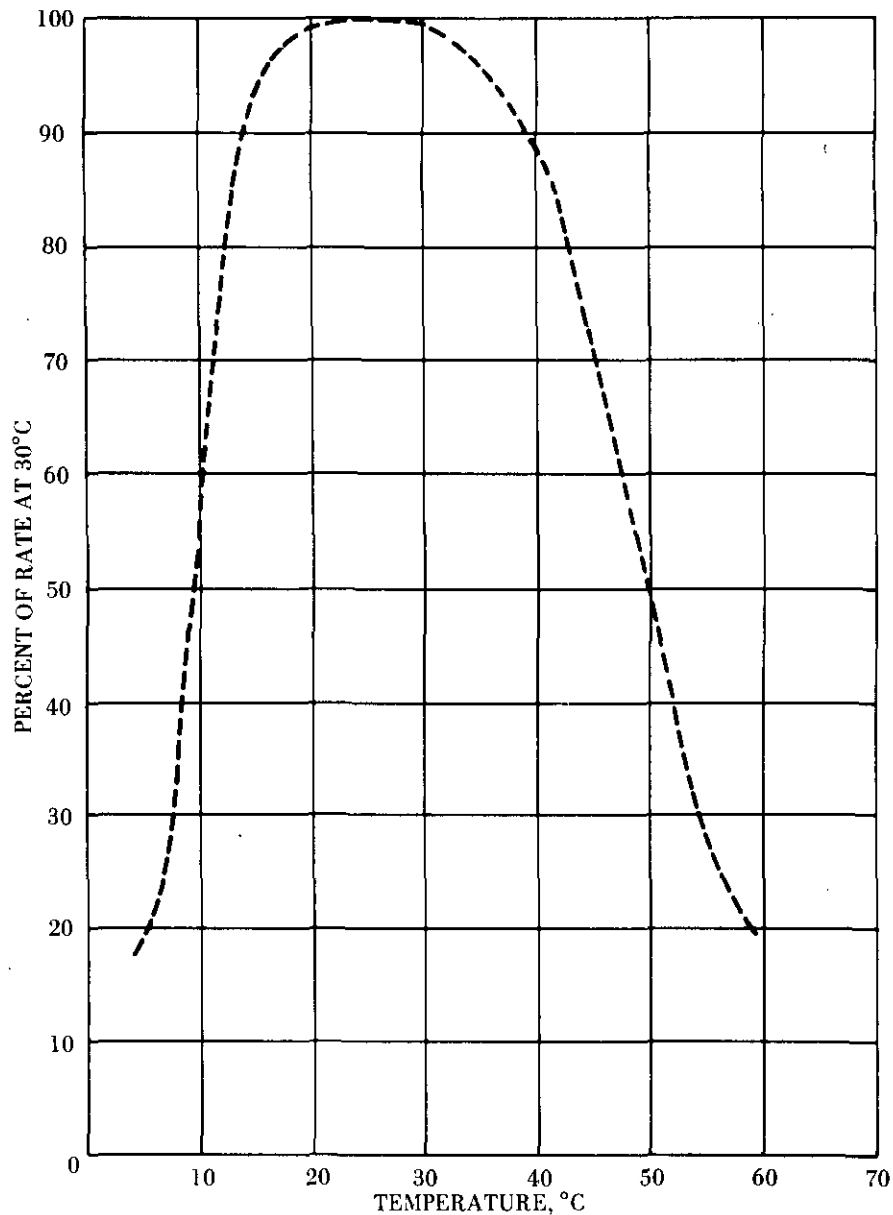
Fig. 42 Effect of Temperature on Nitrification as Reported by Borchardt

to completely oxidize the ammonia would double or the MLVSS would have to be carried at twice the level necessary for complete nitrification under optimum conditions.

Summary — TFE can be effectively nitrified in a separate system at temperatures as low as 6 deg C (43 deg F), provided dissolved oxygen levels are kept above 1 mg/L and other operational variables such as pH, mixed liquor suspended solids, and detention time are controlled in favorable ranges.

4-23

Denitrification — Denitrification may be accomplished in tanks wherein the biological sludge is kept in a fluidized condition by mechanical mixing, in upflow rock filters, or by downflow on fine media filters. Because studies employing the first and last systems were being conducted at other locations, we chose to investigate the use of upflow rock filters. Plastic media were
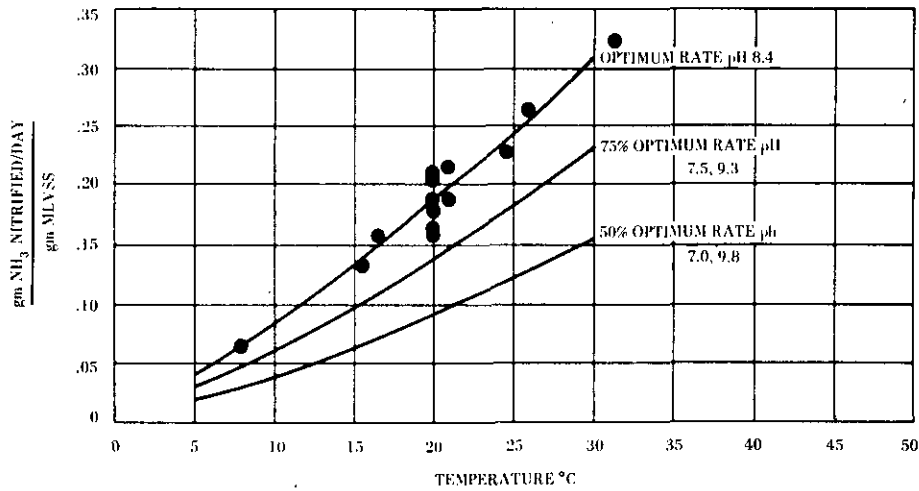
Fig. 43   Rate of Nitrification Versus Temperature at Various pH Levels

considered as an alternate to rock, so the study included a unit packed with a newly developed material* that seemed quite promising.

The facilities used in the denitrification studies are shown as Figure 44. Both the rock and plastic filters contained 5 feet of media depth. Effluent from the nitrification unit served as feed to the two units and had the

Fig. 44   Upflow Filters Used in the Denitrification Studies

Product of Koch.

characteristics shown on Figures 30 and 31. Methyl alcohol was added to the influent to serve as a source of carbon (BOD) to reduce the nitrates and nitrites present to nitrogen gas according to equation 4-4.

$$2 \, CH_3OH + 2 \, NO_3^- \xrightarrow{\text{Bact.}} 2 \, CO_2 + 4 \, H_2O + N_2 \uparrow \qquad (4\text{-}4)$$
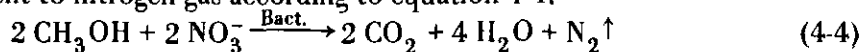
Additional methyl alcohol was added to compensate for the dissolved oxygen in the influent. An attempt was made to maintain a methanol to nitrate nitrogen ratio of 5 to 1.

Figure 45 presents information on operational variables during the course of the denitrification studies that were considered of importance. Temperature, of course, dropped radically from the high values of October to occasional lows of 4 deg C (39 deg F) during January through March. pH also varied from 6 to 10, depending principally upon the pH of the influent. Detention time was held reasonably close to 2.6 hours for the first 120 days and then fluctuated considerably during the rest of the study. There were numerous days when the methanol to nitrate nitrogen ratio exceeded 5 to 1.

The quality of the effluent produced by the denitrification unit charged with rock is shown on Figures 46 and 47. Corresponding data obtained with the plastic media are not shown but, in general, were inferior. In fact, the special plastic medium was replaced with a corrugated sheet type without improving performances.

The effectiveness of the rock filter in removing nitrate and nitrite nitrogen is shown on Figure 46. In general, removal of both was quite erratic. The data from day 320 to 360 show very low nitrate nitrogen in the effluent, from which it might be concluded that excellent results were produced. Such a conclusion is not justified as the influent contained very low nitrate levels, and the nitrite removals during this period were poor. Ammonia nitrogen in the effluents was generally zero which provides good evidence to show that denitrification does not yield ammonia. On several days significant amounts of ammonia appeared in the effluent. This condition was well correlated with days when the nitrification unit failed to produce a completely nitrified effluent.

The organic nitrogen content of the effluent from the denitrification unit normally ranged from 2 to 4 mg/L, with an average of about 3 mg/L. The suspended solids were less than 10 mg/L on most of the days up to day 380. Following that the quality decreased in this regard, indicating channeling and need for backwash. The decreased quality indicates that a clarifier would probably be necessary following the upflow unit.
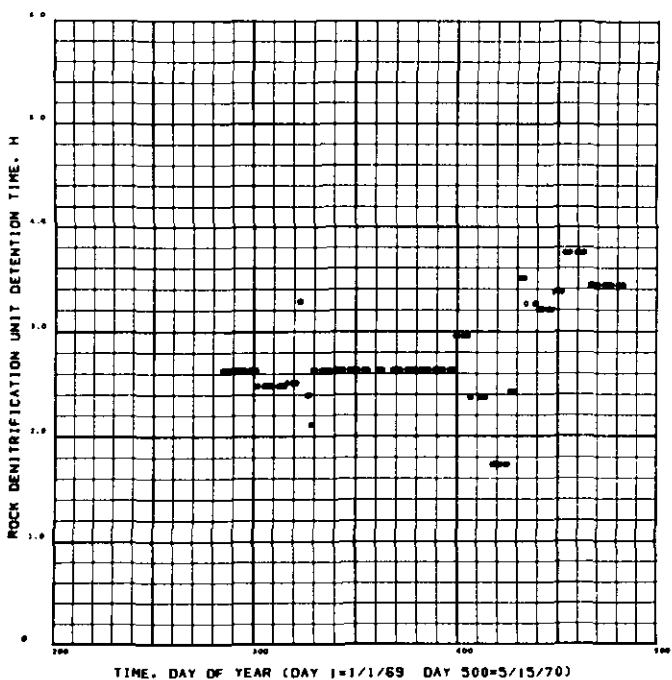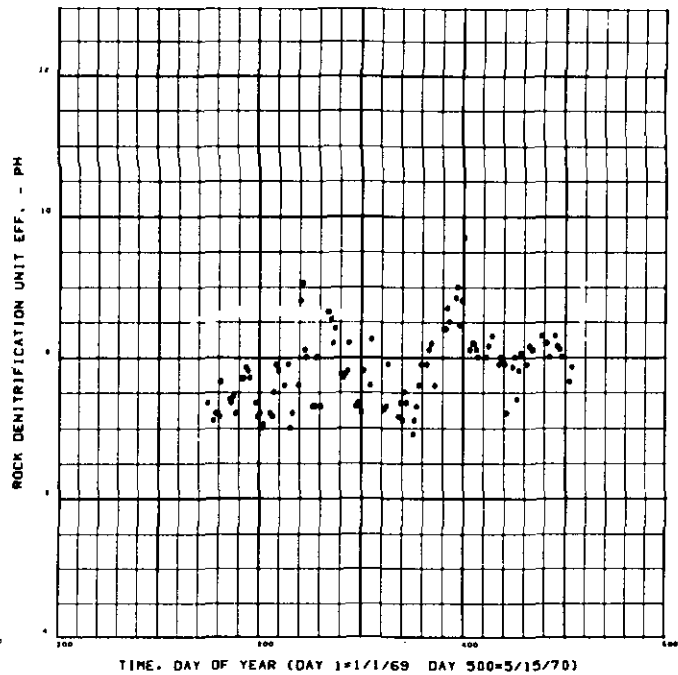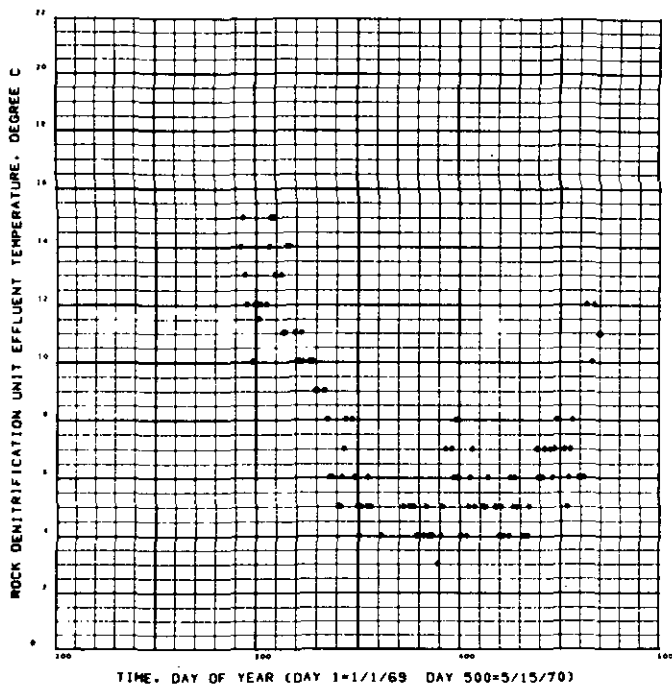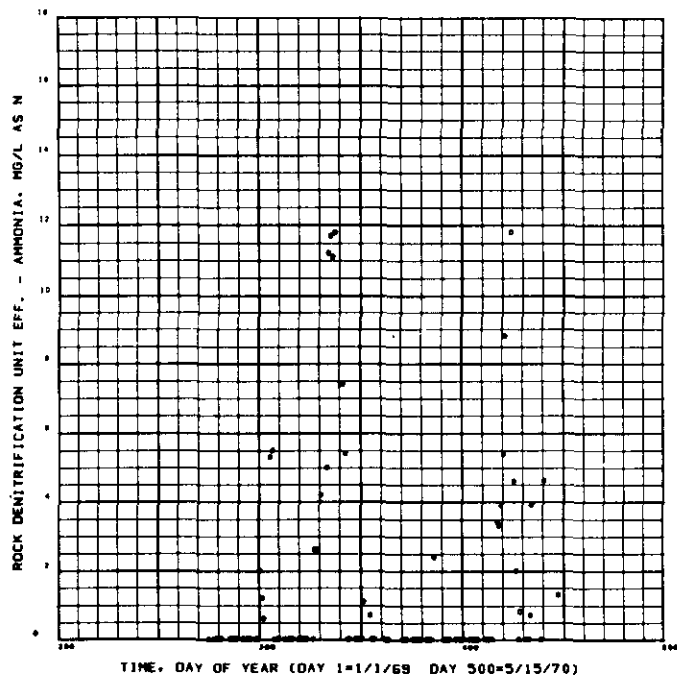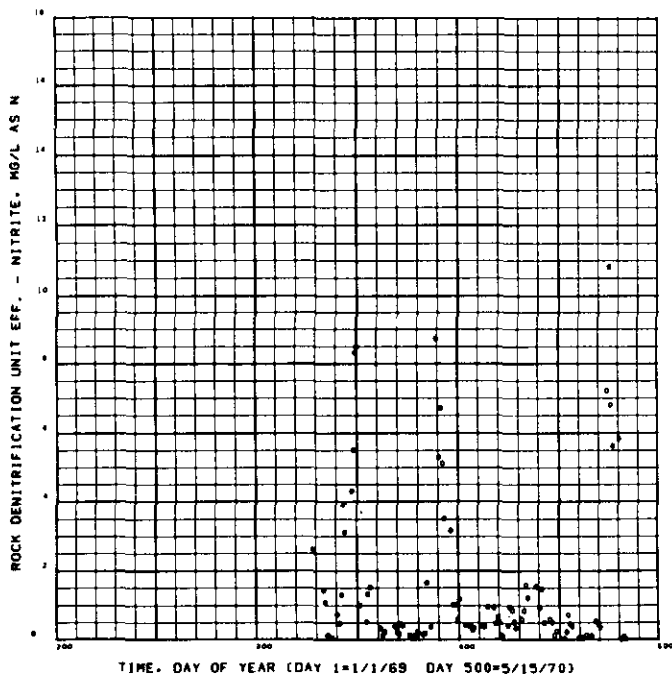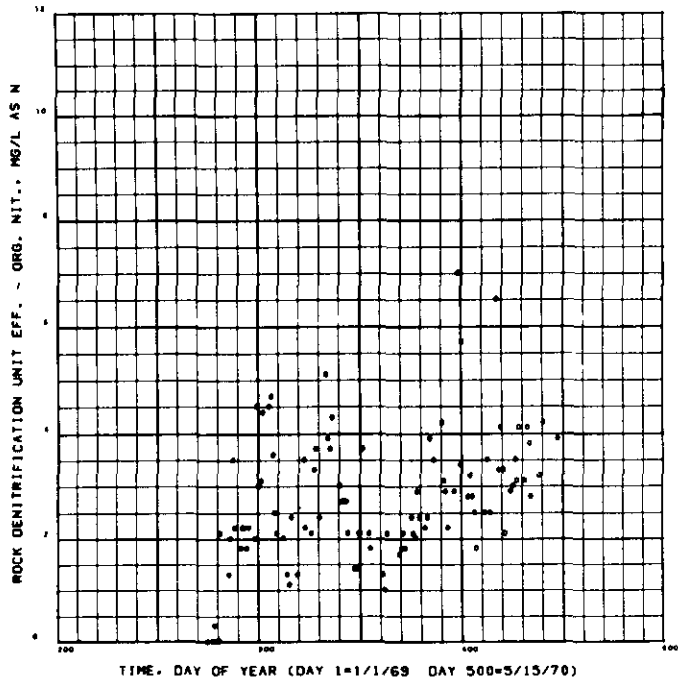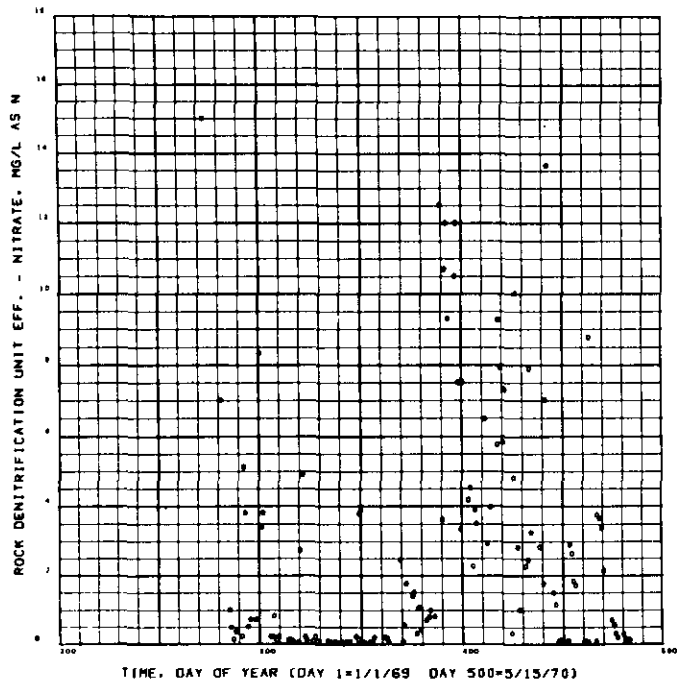
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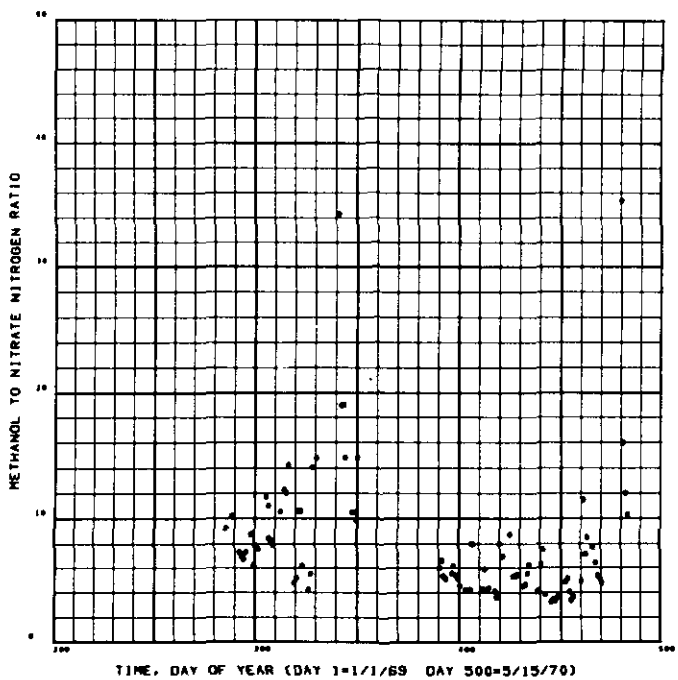
Fig. 45   Operational Variables During Denitrification Studies
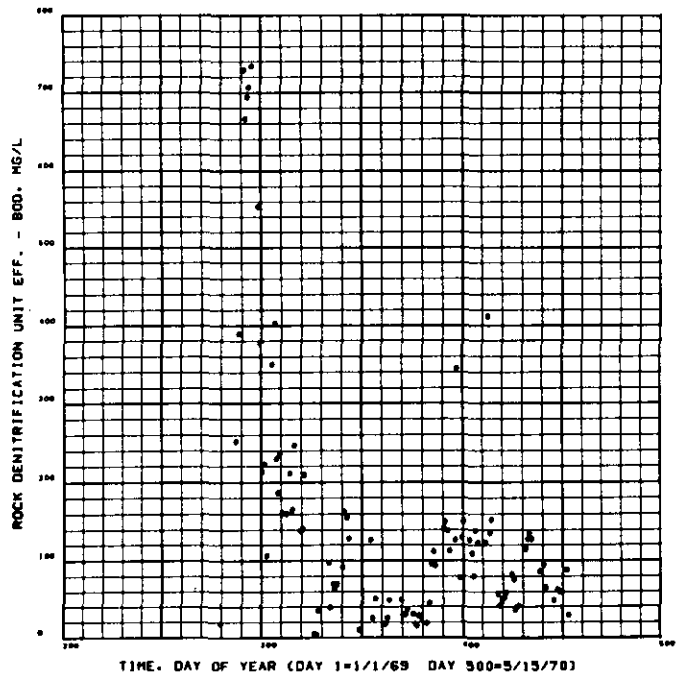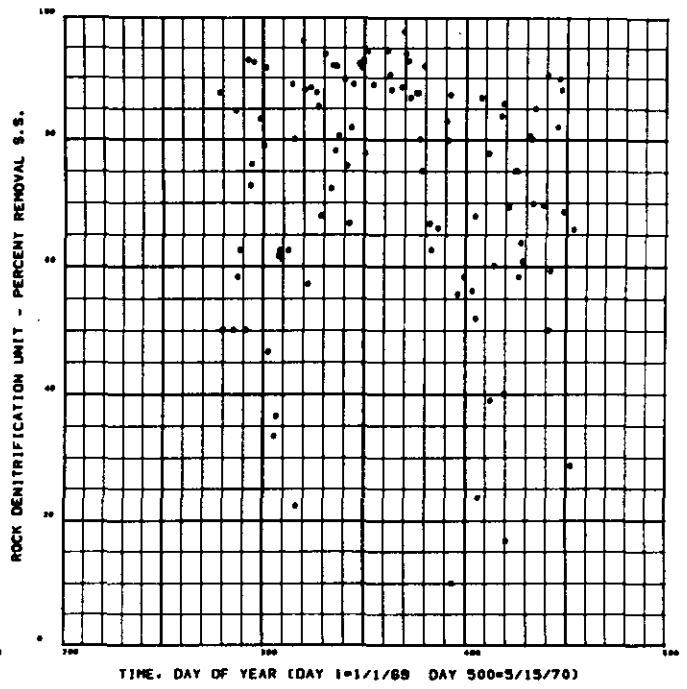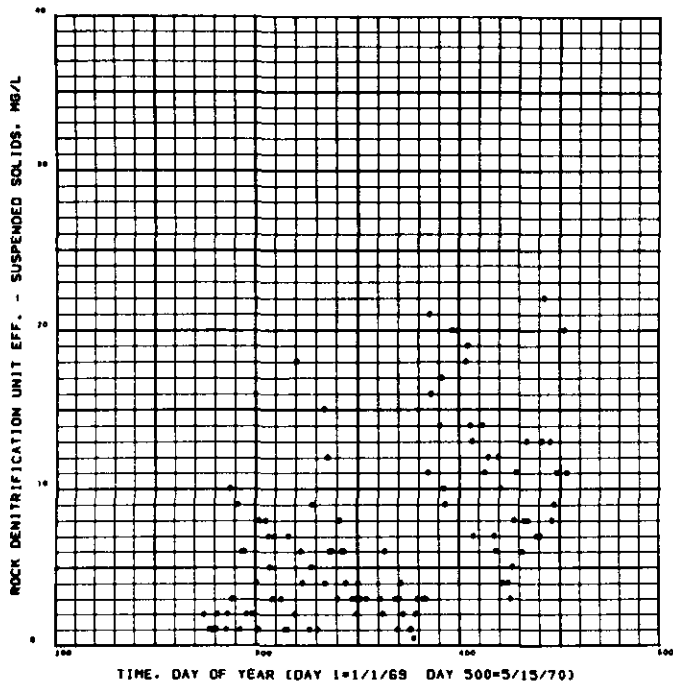
4-26

Fig. 46 Quality of Effluent from Denitrification Unit

4-27

Fig. 47   Quality of Effluent from Denitrification Unit

4-28

The BOD of the denitrified effluent ranged over wide limits, often exceeding 100 mg/L and seldom less than 20. Many, but not all, of the high values were a result of excessive ratios of methanol to nitrate. For example, during the period of day 400 to 420, the methanol to nitrogen ratio exceeded 5 on only 3 occasions, yet BOD of the effluent exceeded 100 mg/L on 9 days and never fell below 40 mg/L. These data indicate that either the control of methanol to nitrate ratios was not precisely controlled or that upflow rock filters are not acceptable units for denitrification.

In Figure 48, performance of the rock filter denitrification unit is analyzed in terms of operational variables. These data indicate that ratios of methanol to nitrate nitrogen of 10 or more to 1 were needed to produce denitrified effluent. Such ratios, of course, are exhorbitant. Neither detention time in the range of 2 to 4 hours nor pH in the range of 7 to 8.6 appeared to have a decided effect. Temperatures below 8 deg C (45 deg F) were definitely detrimental.

Summary — Denitrification in the upflow filters was not as successful as would have been expected. Control of methanol must be perfected and more studies need to be performed in the areas of rate kinetics and effects of variables such as pH, temperature, methanol dose, time, and sludge concentrations on the rate of denitrification.

On the basis of our studies it would have to be concluded that either the units were channeling, which reduced the detention time and should have been backwashed, or four hours detention at the temperatures and pH's experienced were not sufficient to accomplish complete denitrification. These experiences are probably illustrative of problems which would occur in any full scale unit.

Our data indicate a need for clarifiers following the upflow units and on this basis, the upflow unit appears to be inferior to a completely mixed denitrification system.

Conclusions

The removal of nitrogen as ammonia by means of air stripping is not practical in the cold climates because of temperature problems. Severe scaling and air pollution would probably make this process unacceptable at any location.

Nitrification has been shown to be a controllable process when conducted in a separate system. Ammonia concentrations of 0.3 ± 0.3 mg/L
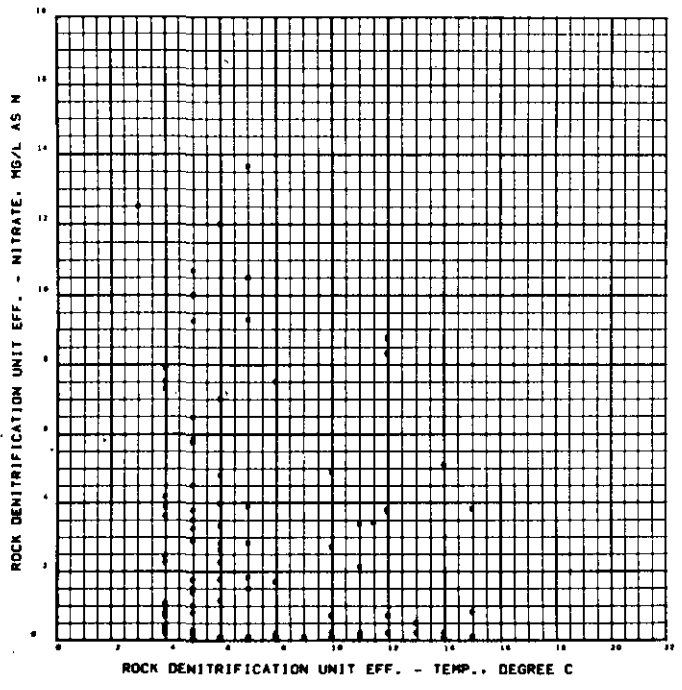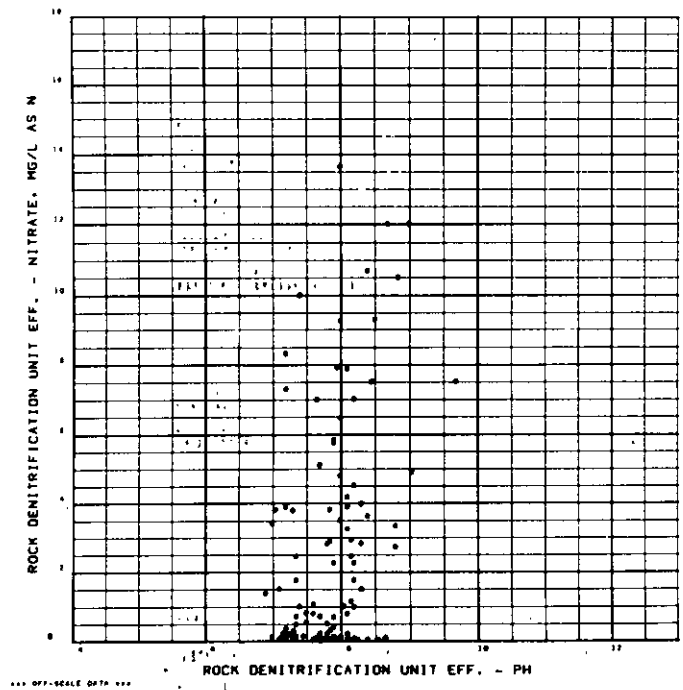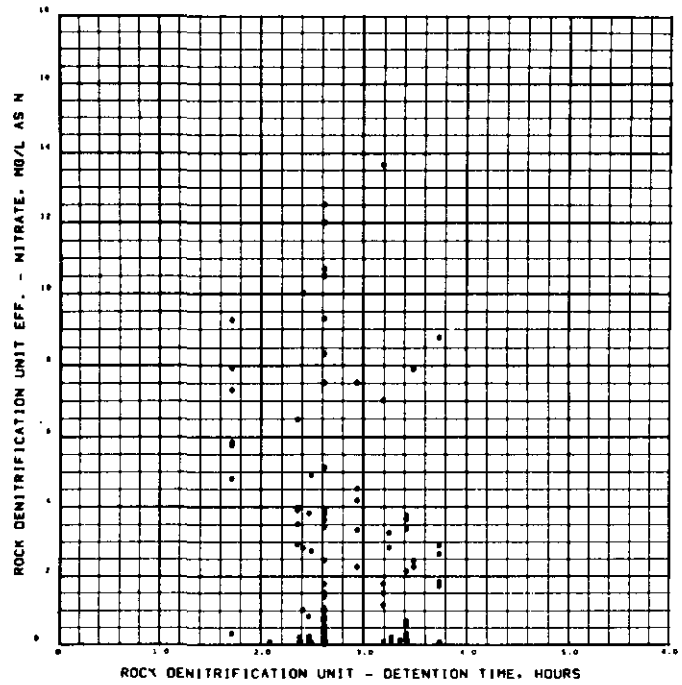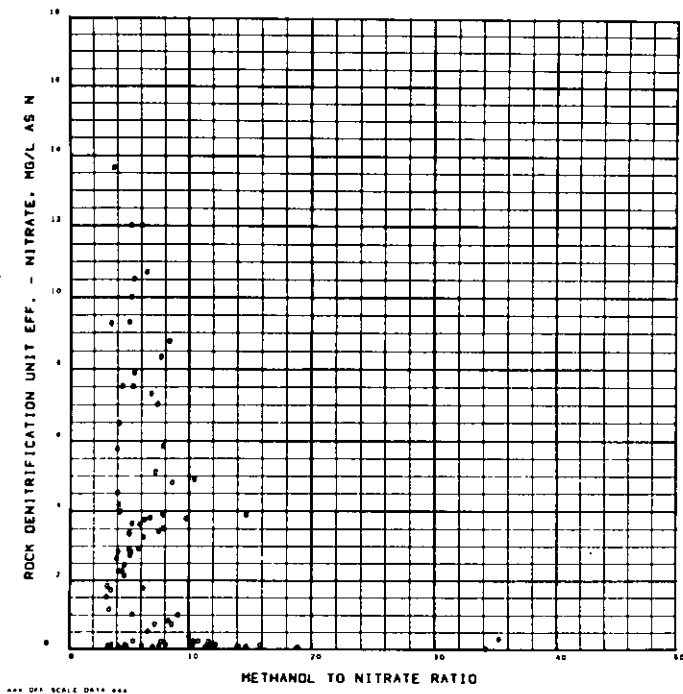
Fig. 48 Performance of Rock Denitrification Unit Versus Operational Variables

in the effluent can be attained if the factors of pH, temperature, time, and MLVSS are considered in the design of the units and DO levels are kept in excess of 1.0 mg/L. pH and temperature are especially important in determining the kinetics of the nitrification reaction.

Denitrification in upflow filters was not found to be a controllable process under the conditions existing at Marlborough.

ACKNOWLEDGMENTS

Respectfully submitted,

METCALF & EDDY, INC.

Clair N. Sawyer
Vice President
Director of Research

Approved:

David A. Duncan
Vice President
Registered Professional Engineer
Massachusetts license No. 6951

APPENDIX

APPENDIX A

MARLBOROUGH WASTEWATER TREATMENT PLANT

In order to better evaluate the operation of the pilot units, a study was made of the various components of the Marlborough wastewater treatment plant.

A schematic flow diagram of the existing Marlborough Easterly treatment plant is shown on Figure A-1 and the basic design data are given in Table A-1. The raw sewage enters the plant through a surcharged inlet and passes through a grit chamber to remove the grit and other heavy solids. The flow then passes through comminutors and into the Imhoff tanks. The Imhoff tanks provide an area for both sedimentation and sludge digestion. Rock media trickling filters, with provision for recirculation of either final tank influent or effluent, follow and are followed by final sedimentation. The final sedimentation tank effluent then passes to lagoons for polishing before being discharged to the receiving stream.
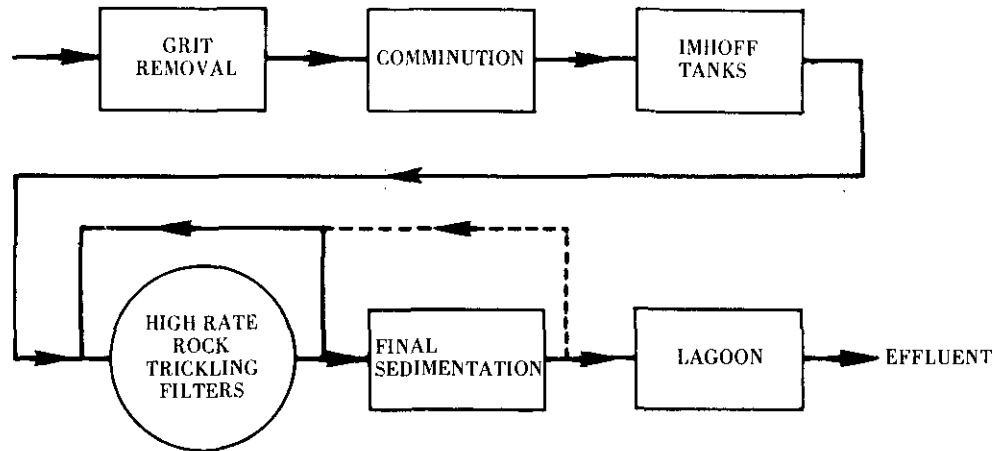
Fig. A-1 Marlborough Wastewater Treatment Plant

Table A-1. Marlborough-Sewage Treatment Plant Basic Design Data

	Design	Present
Population	20,000	21,000 (est.)
Sewage flow		
Average, mgd	1.4	2.33
Maximum, mgd	4.4	3.1
Minimum, mgd	0.55	0.8

A-1

Table A-1 (Continued). Marlborough-Sewage Treatment Plant
Basic Design Data

	Design	Present
Grit Chambers		
No. of units	2	2
Width of channel, ft	5	5
Length, ft	45	45
Parshall Flume		
Width of throat, in.	12	12
H_a at 4.4 mgd ft	1.41	1.41
Comminutors		
No. of units	2	2
Unit capacity, mgd	4.4	4.4
Size of unit	25M	25M
Max. head loss, ft	0.63	0.63
By-pass Rack		
Width of channel, ft	2.5	2.5
Clear spacing between bars, in.	2	2
Imhoff Tanks		
No. of units	2	2
Sedimentation compartments		
Length, ft	80	80
Width, ft	12	12
Depth, water ft	11.1	11.1
No. of channels per tank	2	2
Sludge compartments		
Capacity 18 in. below slots, cu ft per capita	2.46	2.46
Width, ft	31	31
Length, ft	80	80
Total water depth including sedimentation, ft	27	27
Scum vents		
Gas vent area, percent of tank	21	21

	Design	Present
Sludge Drying Beds	Existing beds to be used	
High Rate Trickling Filters		
Number of units	2	2
Diameter, ft	64	64
Area provided, sq ft	6,440	6,440
Area provided, acre	0.148	0.148
Depth of stone, ft	6	6
Acre feet of stone	0.88	0.88
Final Settling Tanks		
No. of units	2	2
Area, sq ft	2,540	2,540
No. of tanks	2	2
Width, ft	16	16
Length, ft	80	80
Water capacity, cu ft	20,300	20,300
Effective water depth, ft	8	8
Sludge removal	scraper conveyor	
Freeboard, ft	1.0	1.0
Pumping Station		
Recirculation, mgd required	3.0	3.0

Wastewater Flows

Maximum, minimum, and average daily flows for the period of January 1, 1969 (day 1) to April 29, 1970 (day 484) are shown on Figure A-2. Although the data shown on Figure A-2 are informative and give outside limits, a study of daily data during high flow periods is more revealing. For example, for the period of March 23 to April 10, 1969 (day 82-100), maximum flows ranged from 2.85 to 3.10 mgd while minimum flows ranged from 2.6 to 2.8 mgd. Average flows did not drop below 2.8 mgd during this 18-day period. These data show that there is presently very little variation in diurnal flows during wet periods at Marlborough.
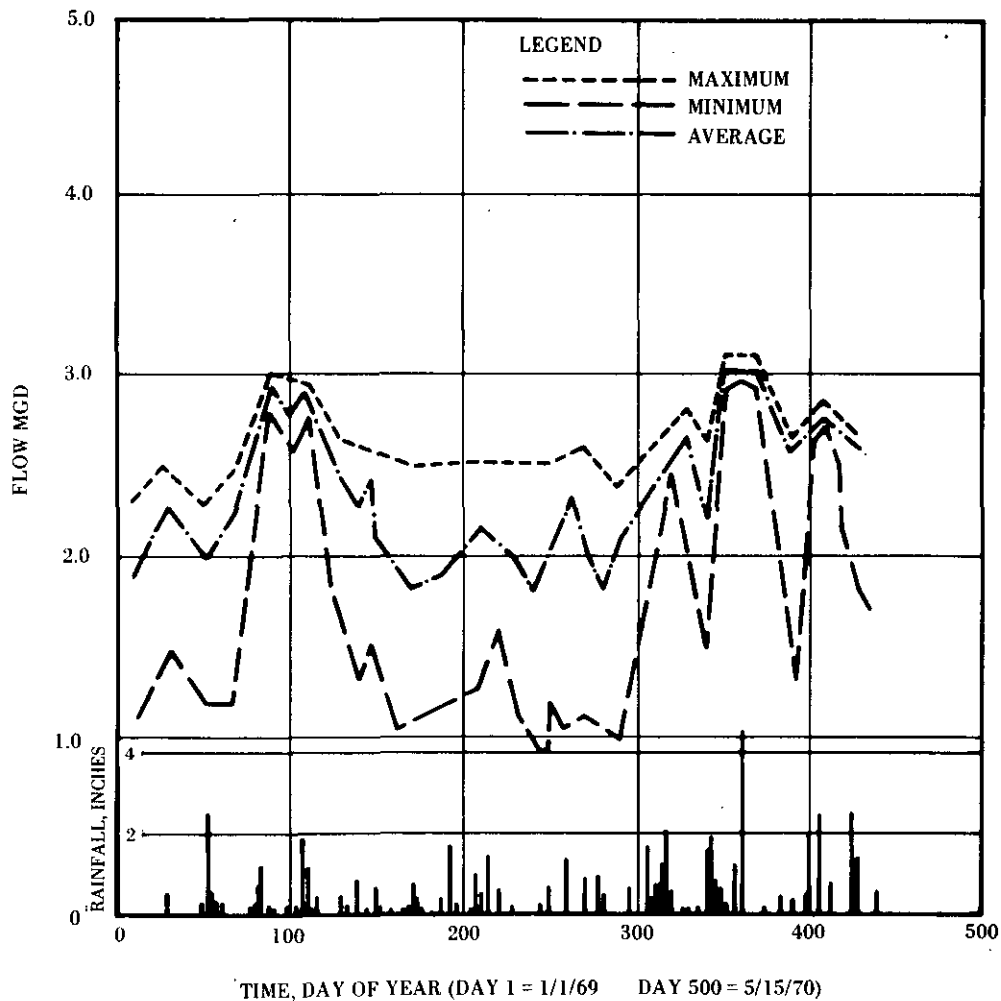
Fig. A-2 Plant Flows and Rainfall

To better indicate the relationship between plant flow and rainfall, the rainfall is plotted below the data on Figure A-2. During low groundwater table periods, February 9-March 11, 1969 and April 30-October 27, 1969 (day 40-70 and day 120-300) the rain had little effect upon the minimum plant flow. However, when the groundwater was high because of rains, March 20 - April 20, 1969, December 6 - December 26, 1969, and January 25 - February 13, 1970 (day 79-110, day 340-360, and day 390-410), the additional rainfall exerted a major effect upon the flow.

During periods of high groundwater, the infiltration into the collection system causes the inadequate main trunk sewer to overflow at various locations. Contract documents for the replacement of the main trunk sewer have been completed and are now under review by the Division of Water Pollution Control of the Water Resources Commission.

A-4

Temperature

The maximum and minimum air and sewage temperatures are shown on Figure A-3. It should be noted that the wintertime sewage temperatures are much lower than in most sewerage systems because of the influence of groundwater entering the Marlborough collection system. These low temperatures adversely affect the treatment plant performance.

TIME, DAY OF YEAR (DAY 1 = 1/1/69   DAY 500 = 5/15/70)
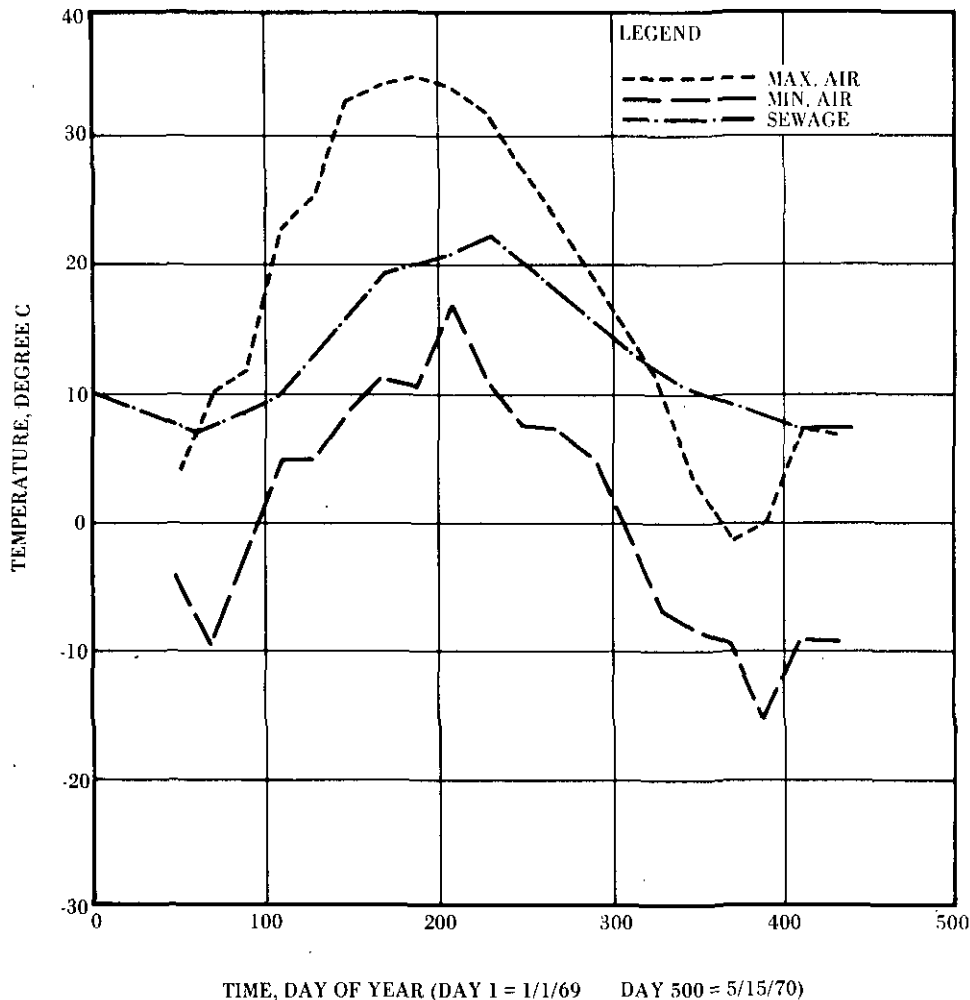
Fig. A-3   Maximum and Minimum Air and Sewage Temperatures

Raw Wastewater Characteristics

Data showing the BOD, suspended solids, total phosphorus and nitrogen content of the raw wastes for the period February 1, 1969 to April 9, 1970, are presented on Figure A-4. As would be expected from the flow data previously discussed, the concentrations of these constituents were highly variable and, in general, varied inversely with the flow. A summary of daily ranges of the constituents and also ammonia, pH, and alkalinity in the raw wastewater is given in Table A-2.

A-5
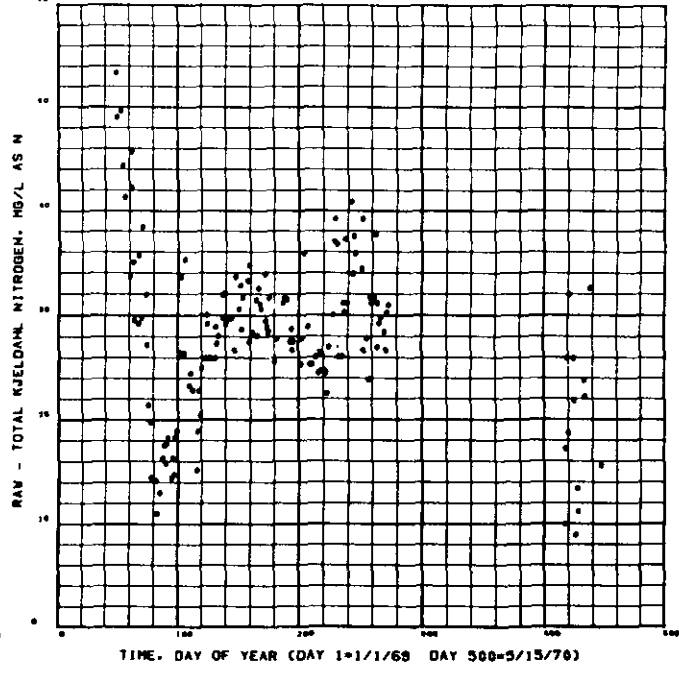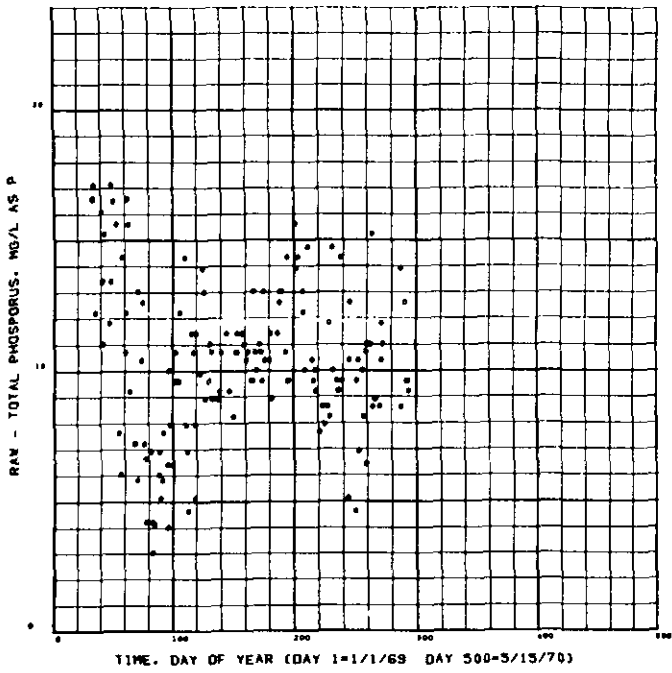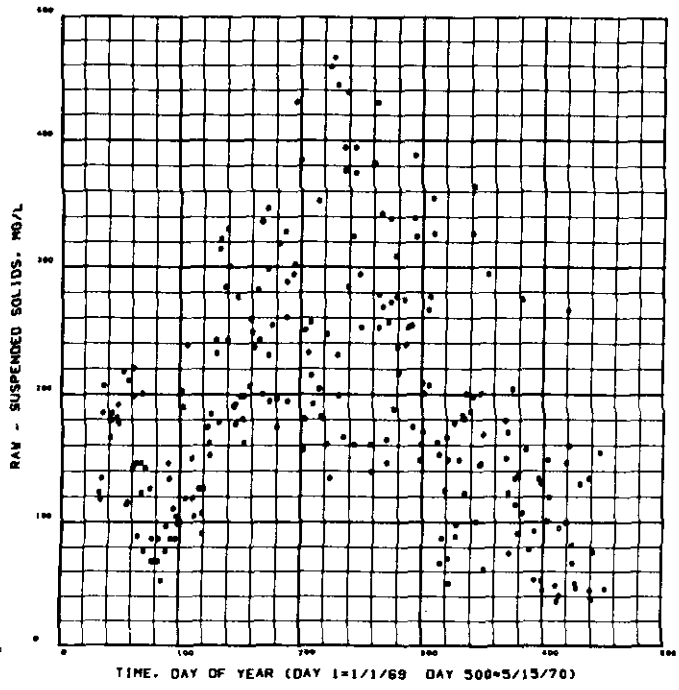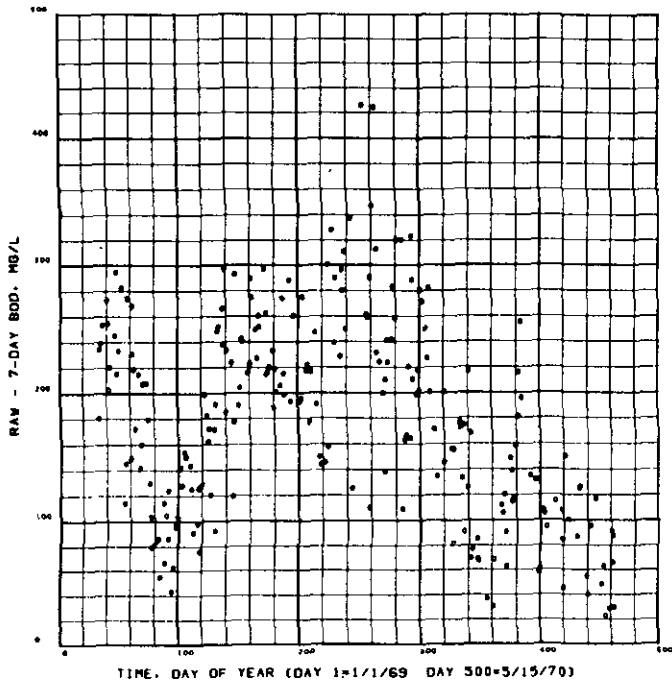
Fig. A-4 Raw Wastewater Characteristics

Table A-2. Summary of Variations in Raw Wastewater Characteristics
February 1, 1969 to April 9, 1970

	Range — daily basis, mg/L
BOD$_7$	30 — 425
Suspended solids	35 — 466
Total phosphorus	3 — 17
Total Kjeldahl nitrogen (NH$_3$ + Organic N)	9 — 53
NH$_3$	4 — 29
pH	6.3 — 7.8
Alkalinity	37 — 160

These data indicate, based upon dilution ratios, that if the main trunk sewer had the capacity, flows to the plant can approach 12 mgd rather than the much lower peak flows arriving at the plant.

Imhoff Tank

The quality of the Imhoff tank effluent in terms of BOD$_7$, suspended solids, total phosphorus, and nitrogen are shown on Figure A-5.

Because the Imhoff tank did not perform in the same way a primary sedimentation tank is expected to perform, special attention was given to this unit. The Imhoff tank would remove nutrients from the sewage and then later release these nutrients from the digestion zone back into the flow. For this reason the Imhoff effluent bore little relationship to the raw influent for the constituents of BOD$_7$, phosphorus, and nitrogen. In fact many times the Imhoff effluent concentrations exceeded the influent concentrations.

Final Sedimentation Tank Effluent

Figure A-6 shows the quality of the effluent from the final sedimentation tank in terms of BOD$_7$, suspended solids, total phosphorus, and nitrogen. The effluent quality was not as good as would normally be expected. The rock media trickling filters were fractured and ponding
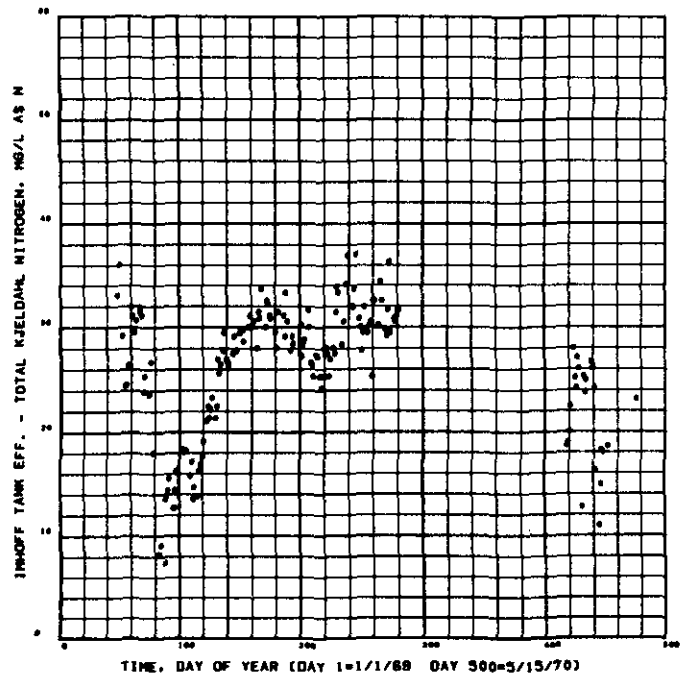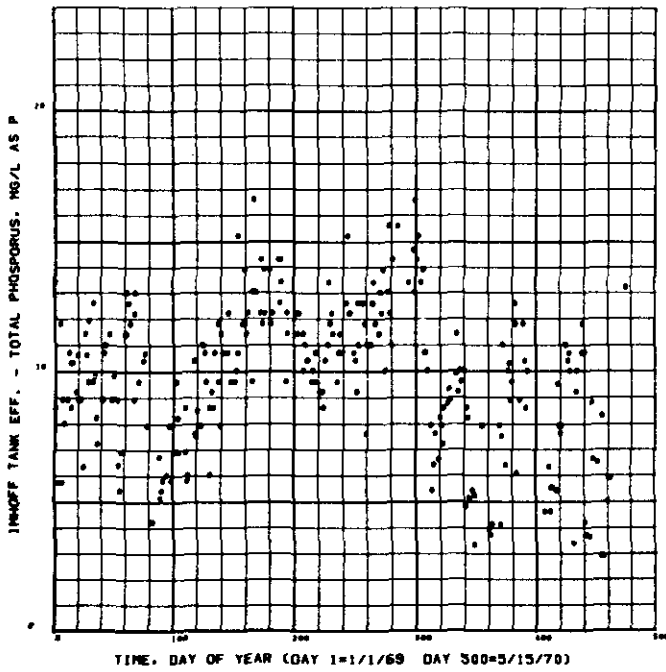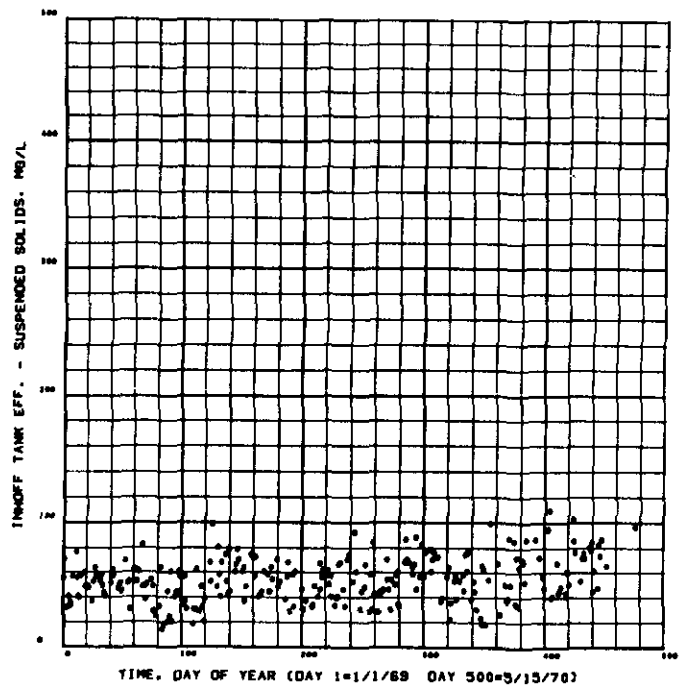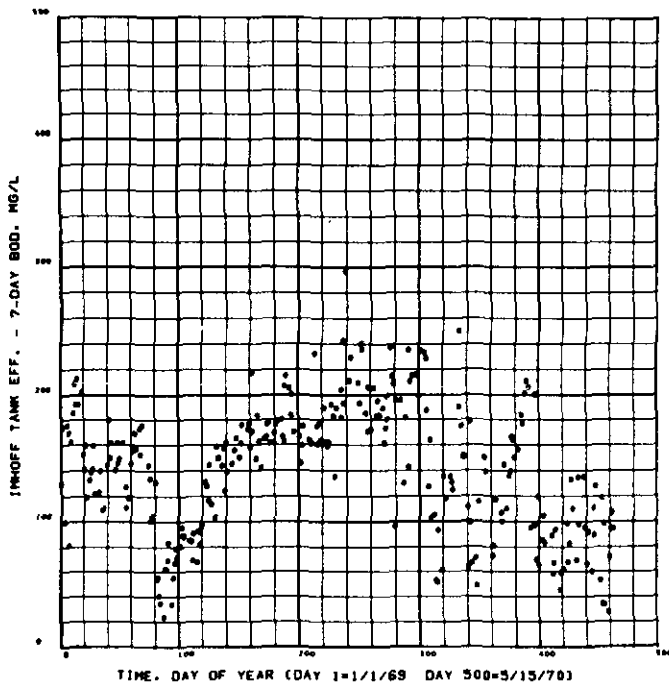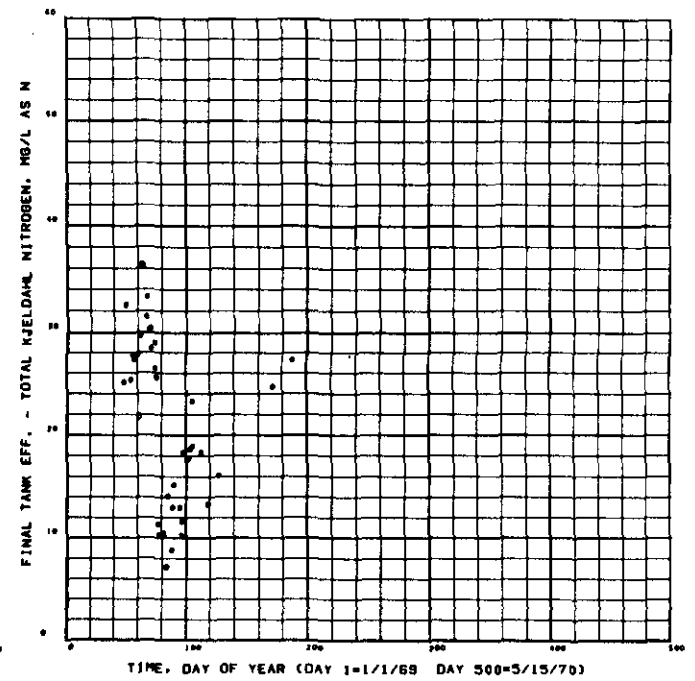
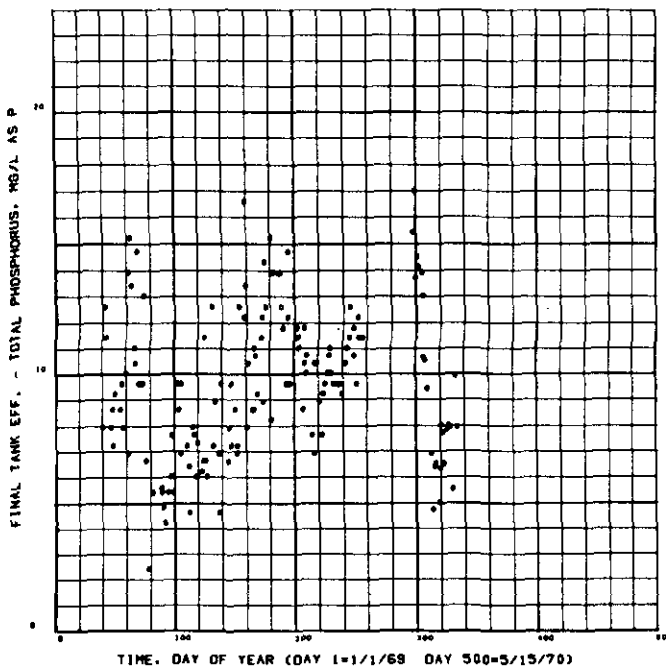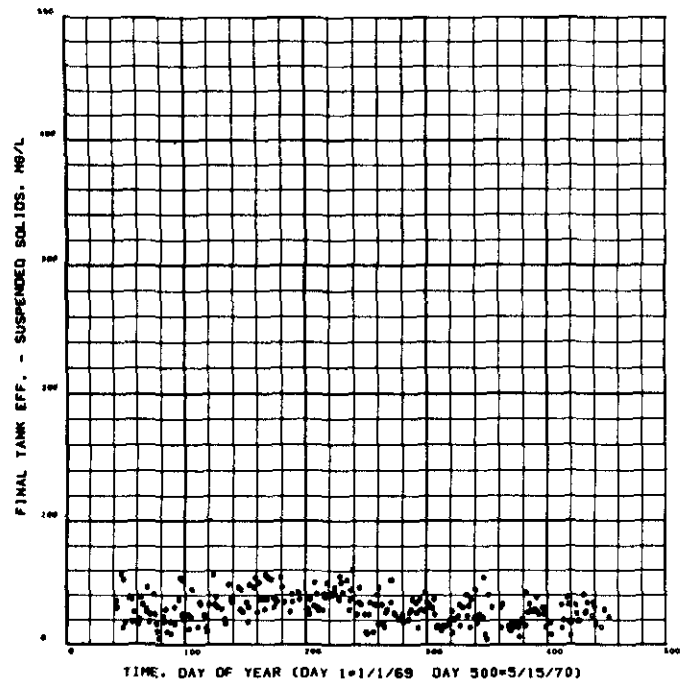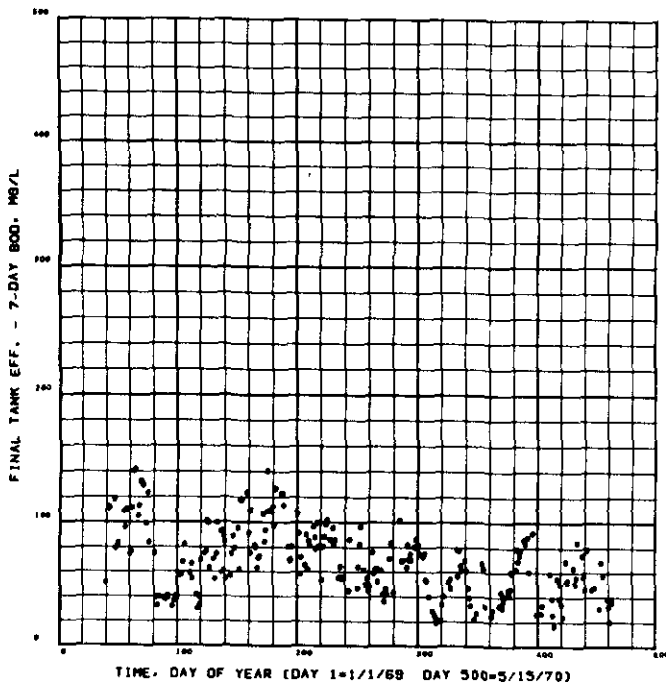Fig. A-5 Imhoff Tank Effluent Characteristics

A-8

Fig. A-6   Final Sedimentation Tank Effluent Characteristics

A-9

occurred during periods of the year. Because of the high flow to the plant, recirculation was not practiced on a continuous basis.

The flow through the existing treatment plant sedimentation tanks approached and sometimes suprassed the design flows and the removal of suspended solids suffered because of this.

Final Lagoon Effluent

In the final analysis, the quality of the final effluent as discharged to the receiving stream is the most important consideration. Limited data were obtained on the effluent discharged from the plant to the receiving stream. Figure A-7 shows the effluent quality in terms of suspended solids and BOD_7.

In general, the best quality effluent was produced during the high flow periods (low influent concentration periods) when the trickling filter was producing effluents with the lowest BOD_7.

The percentage removals of both suspended solids and BOD_7 from the raw waste to the oxidation pond effluent are presented on Figure A-7.

The best overall percentage removals were during times of low or normal flow conditions (high influent concentrations), and the removals deteriorated during periods of high flow and low influent concentrations.
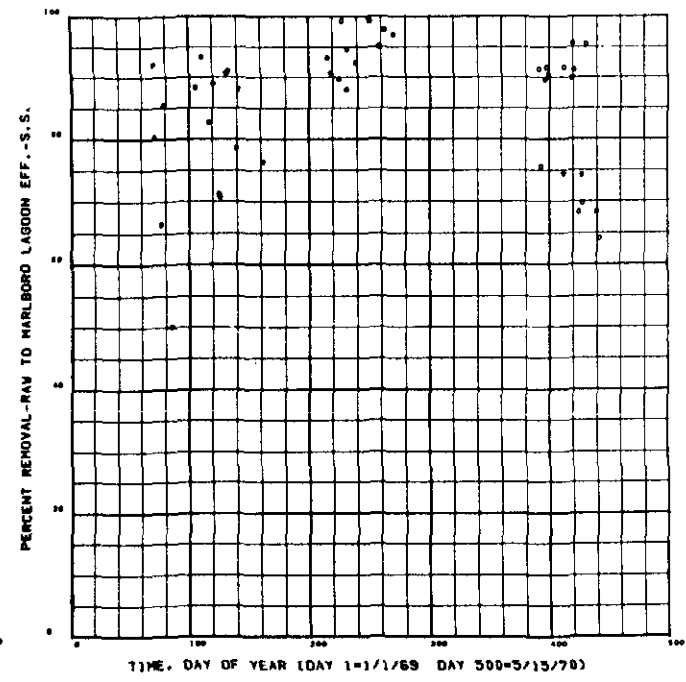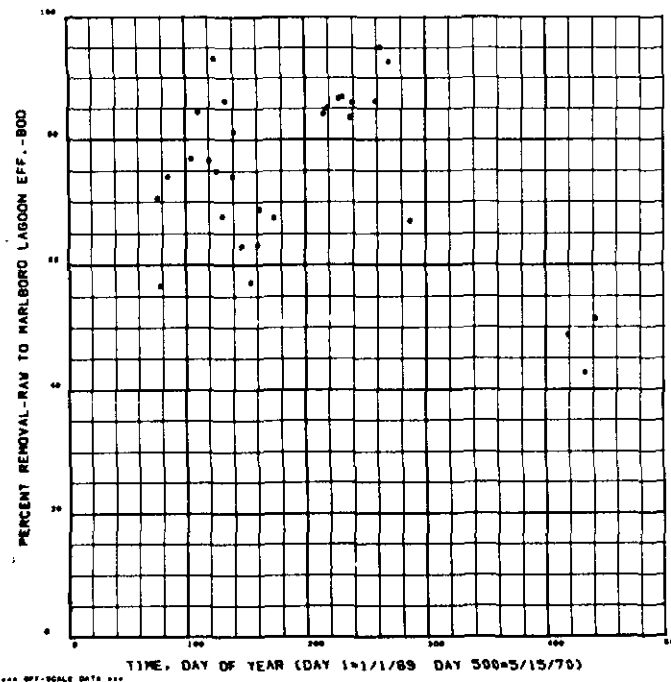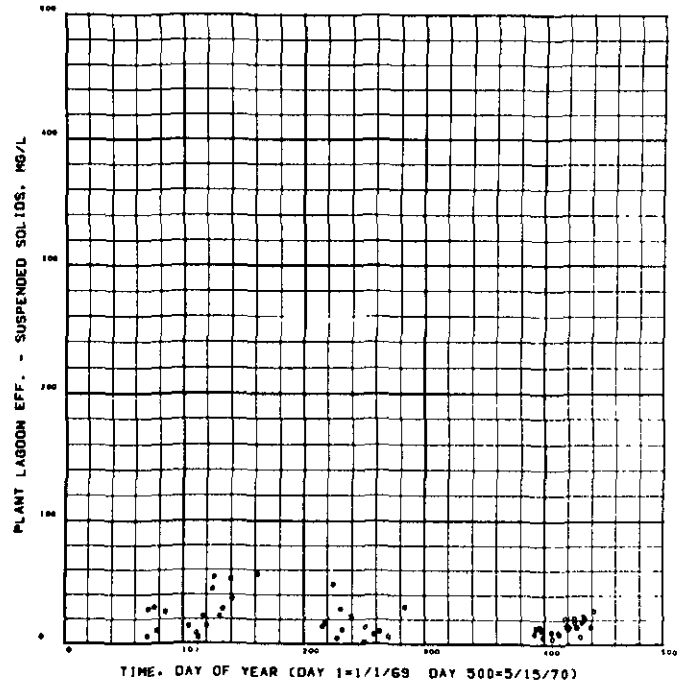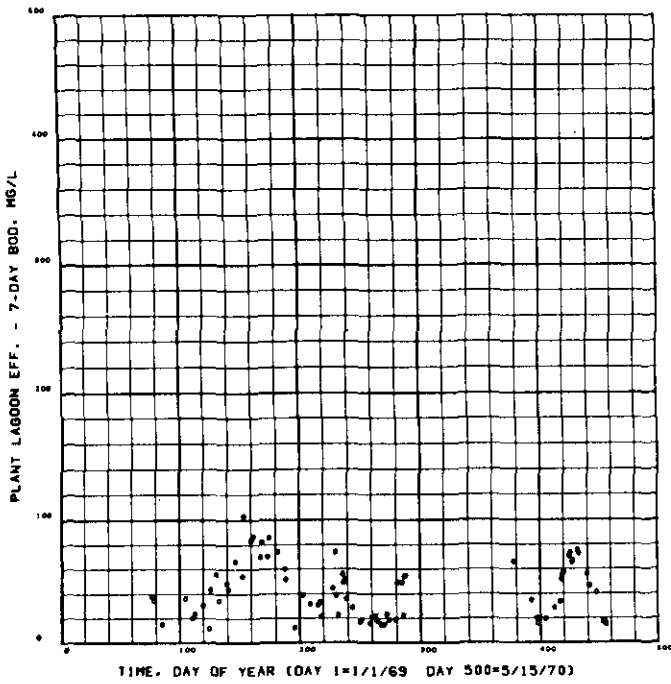
Fig. A-7 Final Plant Effluent Characteristics and Performance

A-11