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HATFIELD TOWNSHIP, PENNSYLVANIA ADVANCED WASTE TREATMENT PLANT

UNITED STATES · ENVIRONMENTAL PROTECTION AGENCY ·

Municipal Environmental Research Laboratory
Office of Research and Development
U.S. Environmental Protection Agency
Cincinnati, Ohio 45268

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HATFIELD TOWNSHIP, PENNSYLVANIA

ADVANCED WASTE TREATMENT PLANT

by

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DISCLAIMER

FOREWORD

Man and his environment must be protected from the adverse effects of pesticides, radiation, noise, and other forms of pollution, and the unwise management of solid waste. Efforts to protect the environment require a focus that recognizes the interplay between the components of our physical environment--air, water, and land. The Municipal Environmental Research Laboratory contributes to this multidisciplinary focus through programs engaged in

- studies on the effects of environmental contaminants on the biosphere, and
- a search for ways to prevent contamination and to recycle valuable resources.

This report details the engineering design, construction and operational considerations that are necessary to provide advanced wastewater treatment at a municipal facility.

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ABSTRACT

The Hatfield Township, Pennsylvania Water Pollution Control Plant was designed to encompass primary chemical treatment, secondary combined activated sludge and nitrification facilities, tertiary chemical tube clarification and mixed media filtration.

The operation of the facility demonstrated that the use of flow equalization facilities improves plant operations by reducing and standardizing chemical concentrations.

Phosphorus is removed efficiently in a combined primary-tertiary phase with operations personnel having the flexibility to optimize each process. Lime feed control by pH is easily accomplished although recirculation of primary sludges is not always necessary. Tube clarifiers and mixed media filters combine to produce a highly polished effluent.

Nitrification was observed to some extent in this modified facility, however, it was extremely difficult to control.

This report was submitted in fullfillment of project number 11060 FRQ by the Hatfield Township, Pennsylvania, Municipal Authority, under partial sponsorship of the Environmental Protection Agency.

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ACKNOWLEDGEMENTS

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Employees of the authority involved with the work included Roy B. Love, Executive Director, Stephen Weech, Plant Manager and Laboratory Technicians, Kenneth W. Mawson, Kathy Hughes, Cathy Love and Robert Lewis. Preparation of this report was aided by Margaret Salinger and Mary Ann Casper of Tracy Engineers and final typing was by Janice Gonczkowski. Figures and graphs were by Frank Salinger of Tracy Engineers Inc. and the authors.

SECTION I

CONCLUSIONS

The results of the demonstration grant performed at the Hatfield Township Advanced Waste Treatment facility led to a number of conclusions, some of which were positive, and some of which were negative.

1. The utilization of screening and grit removal ahead of the raw sewage pump station would have lessened considerably operating problems encountered in the significant grit accumulations in the surge storage tanks, and the clogging of sludge recirculation pumps by re-weaved, previously comminuted, rags and fibrous material.

2. The use of surge storage tanks for flow equalization upstream of the primary treatment system improves plant operations by insuring regulated flow throughout the plant. In addition to a regulated hydraulic flow, it also contributes towards a more homogeneous blend of waste characteristics.

3. The surge storage tanks should have a more positive mixing device than mechanical mixers, and the use of aerators would not only tend to promote more vigorous mixing, but would supply sufficient oxygen to retard septicity during the very warm summer period.

4. The addition of lime to a pH of 9.5 in the primary flocculating clarifiers resulted in primary phosphorus removals averaging 72.1%, an average overall phosphorus removal of 92.3%, and average residual phosphorus of 4.68 mg/l (primary effluent) and 0.54 mg/l (plant effluent).

5. The use of submersible electrodes in the primary flocculation chamber resulted in a daily operational problem requiring frequent acid washing and recalibration of the probes. Only one manufacturers probes were used, however, and new designs for anit-fouling probes may eliminate this problem.

6. The aeration units reduced BOD_5 and at certain times partially reduced nitrogenous oxygen demand concurrently, albeit not consistently.

7. The mixed-media filters removed approximately 50% of all suspended solids fed to them. The tertiary tube clarifiers, however, removed the bulk of the secondary solids, resulting in exceptionally low loadings of solids to the filters. Where low phosphorus residuals, i.e., 0.75 to 1.5 mg/l are all that

is required, there may not be a necessity to utilize tertiary filters after tertiary tube-type clarifiers. The presence of tertiary filters, however, can polish not only phosphorus, but suspended solids, and some BOD_5 as well, and offer a final protection to maintaining high effluent quality.

8. If tertiary filters are utilized after tertiary clarification, provision should be made to by-pass the filters. Unit by-passes throughout the process flowsheet are, and have been, most useful at times of single process malfunction.

9. The flowsheet utilized at Hatfield Township consistently produced a high quality effluent, even though in-plant recycle flows, particularly from periodically overloaded sludge thickeners, at times exceeded original design calculations by substantial amounts.

SECTION II

RECOMMENDATIONS

As a result of the demonstration grant, certain items are recommended as being immediately required to maintain the high level of operation required, while other items are recommended for inclusion with the next expansion stage, soon to be undertaken.

Those items suggested for immediate installation include:

1. The installation of floating aerators in the surge storage tank.

2. The utilization of different electrodes in the primary flocculation units, either more advanced anti-fouling units, or flow-through units.

3. The construction of a by-pass around the tertiary pressure filters, such that full utilization of tertiary chemical precipitation can be realized, even with a malfunction in the tertiary filtration system.

4. Modification or replacement of the existing vacuum filters to provide both additional dewatering and incineration capabilities through the development of a higher solids content in the dewatered sludge.

5. Installation of liquid alum feed capabilities in the tertiary chemical precipitation process.

Those items suggested for inclusion with the next expansion stage include:

1. The construction of raw sewage screening and grit removal ahead of the raw sewage pumping station.

2. An increase in aeration detention time, in order to more consistently achieve ammonia conversion in the aerators.

In addition to the above enumerated physical additions to the existing Hatfield Township Advanced Waste Treatment Facility, the following items are recommended:

1. Continuation of approximately the same level of laboratory analysis work as was maintained during the demonstration grant period as an operating tool. Effective laboratory control is fundamental to achieving advanced waste treatment.

2. Institution of a more detailed preventive maintenance program,

including adequate recording of pertinent data on equipment, and the development of systematic maintenance schedules.

3. Continued operator training sessions on a scheduled basis, to include not only the "what" of operation, but also the "why".

SECTION III

BACKGROUND

GENERAL

Hatfield Township is an Incorporated Municipality lying in Montgomery County, Pennsylvania.

Hatfield Township lies on the fringe of the urbanized Philadelphia area. In Figure 1, the general location of Hatfield Township with respect to the Philadelphia urbanized area, is indicated. It may be noted from this that Hatfield Township completely surrounds the small Borough of Hatfield, and is adjacent to Lansdale Borough, as well as to a number of surrounding Townships

According to the U. S. Census Bureau, the population of Hatfield Township in 1960 was 5,759. In the early 1960s, Hatfield Township was essentially a rural community with some scattered residential development, and virtually no industry, and only a very small amount of commercial activity. In 1960, there were no public sewer facilities and the entire Township utilized individual well supplies as a source of water.

The study of Hatfield Township during the period of 1960 through 1973 is typical of many of the communities lying on the fringes of highly urbanized areas throughout the United States, which have increased dramatically in population, and because of this population increase, have had to provide a wide variety of essential services in a very short period. One such service is that of sewage collection and treatment, which has grown from no facilities in 1962 to highly advanced waste treatment in 1973.

COMMUNITY STATUS IN 1962

A Municipal Authority is a public device which has been created in Pennsylvania, and in certain of the other states of the United States, for the purpose of financing public works projects where State and local limitations are placed on the borrowing capacity of local sub-divisions. In Pennsylvania, a Municipal Authority has no restriction on the amount of money it can borrow, providing that it can demonstrate adequate revenues from user charges and other sources to pay debt service costs as well as any operating expenses that are applicable to a project.

In 1961, the local governing body, the Commissioners of Hatfield Township, created the Hatfield Township Municipal Authority for the purpose of investigating, and subsequently financing and constructing a sewage collection system in Hatfield Township, and one or more sewage treatment plants. This action was taken by the local political sub-division in order to alleviate problems of malfunctioning septic tanks and other on-lot individual sewage disposal systems, which were creating unsightly conditions and health hazards in various built-up sections of the Township.

HATFIELD TOWNSHIP

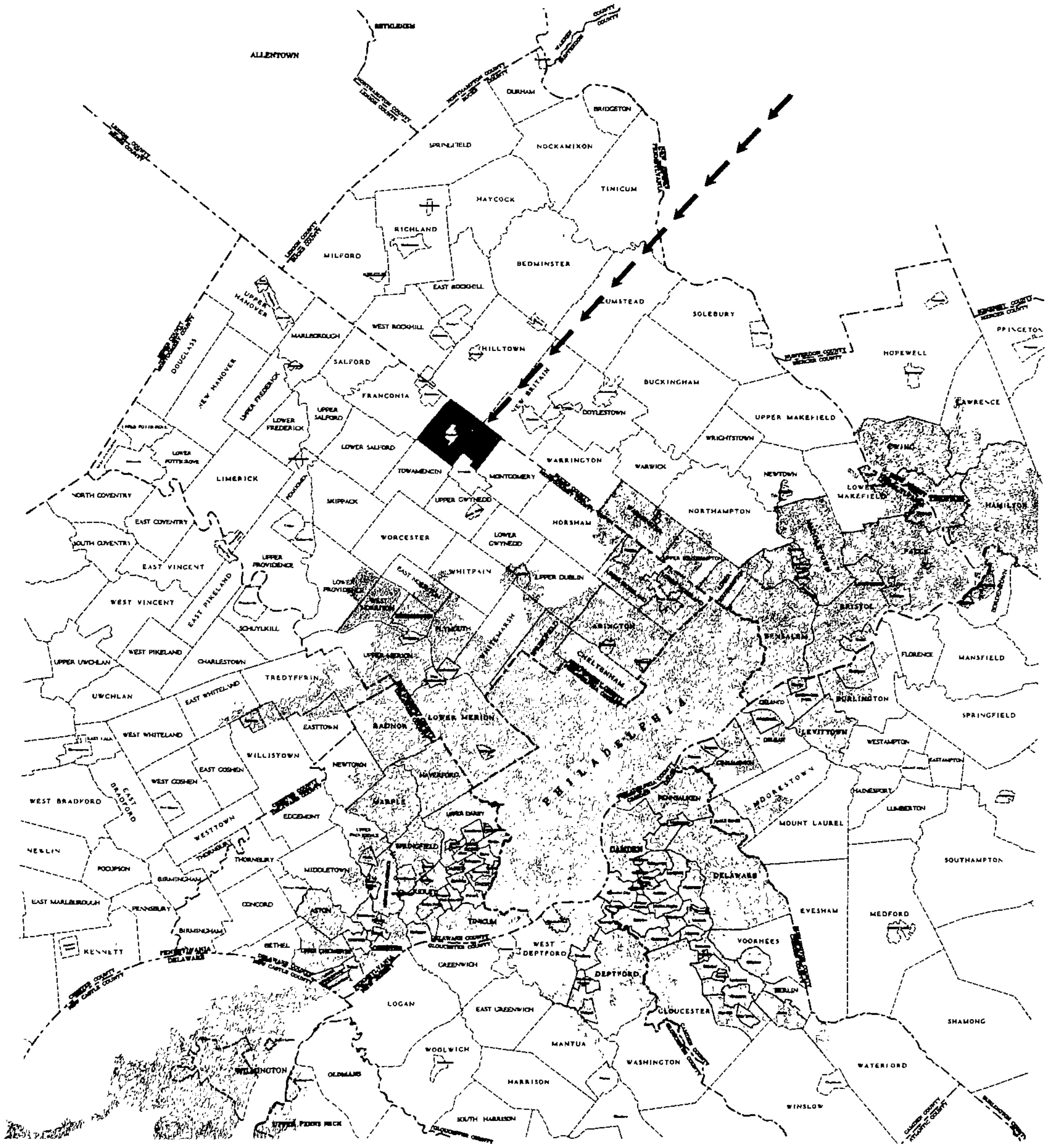


Figure 1. Regional Position of Hatfield Township

This action was taken in advance of any directives from the State regulatory bodies, and constituted an unusual situation, in that a community undertook to solve its sewerage problems without any direct requirements by the State regulatory bodies.

The Hatfield Township Municipal Authority was incorporated in 1961, and proceeded to explore the problem of sewage treatment in the Township by interviewing a number of engineers for the purpose of selecting a Consulting firm to develop their planning. In the fall of 1962, Tracy Engineers, Inc. was retained by the Hatfield Township Municipal Authority, to initiate its planning program with respect to sewerage.

This initial planning involved consideration of the then applicable zoning and land use planning in the Township; the attitudes of the local elected officials with respect to future growth and its impact on other municipal services; projections of growth developed by County and Regional Planning Commissions; and the requirements of the State regulatory agency with respect to the degree of waste treatment necessary.

PRELIMINARY REPORT

The initial thinking of the Hatfield Township Municipal Authority, shortly after its formation in 1961, centered around the possibilities of providing separate sewerage treatment plants for two built-up areas of the Township.

In Figure 2, a more detailed location plan of Hatfield Township is indicated together with the surrounding communities. In 1962 there were two existing sewage treatment plants in the general area; one servicing Hatfield Borough indicated by location "C" in Figure 2, and the other service Lansdale Borough, indicated by location "D". The Hatfield Township Municipal Authority originally felt that it desired to see construction of small seperate treatment facilities at location "B" and location "A", being at the points of the only significant areas of concentrated development within the Township at that time.

In a report presented to the Authority in January 1963, the consultant advanced the proposition that Hatfield Township offered significant potential for future development, even though areas of concentration were limited to two general locations at that time, and that the logical location of a single sewage treatment facility to service the entire Township would be at point "A", as shown on Figure 2. The Hatfield Borough plant (point "C") had been constructed some years earlier, and consisted of Imhoff tanks and standard rate trickling filters, and appeared to be adequate for servicing the needs of Hatfield Borough for a number of years in the future. The Lansdale Borough sewage treatment plant (point "D") was a primary treatment facility constructed in the 1930s, which was overloaded and could not accept any sewage flows from any portion of Hatfield Township.

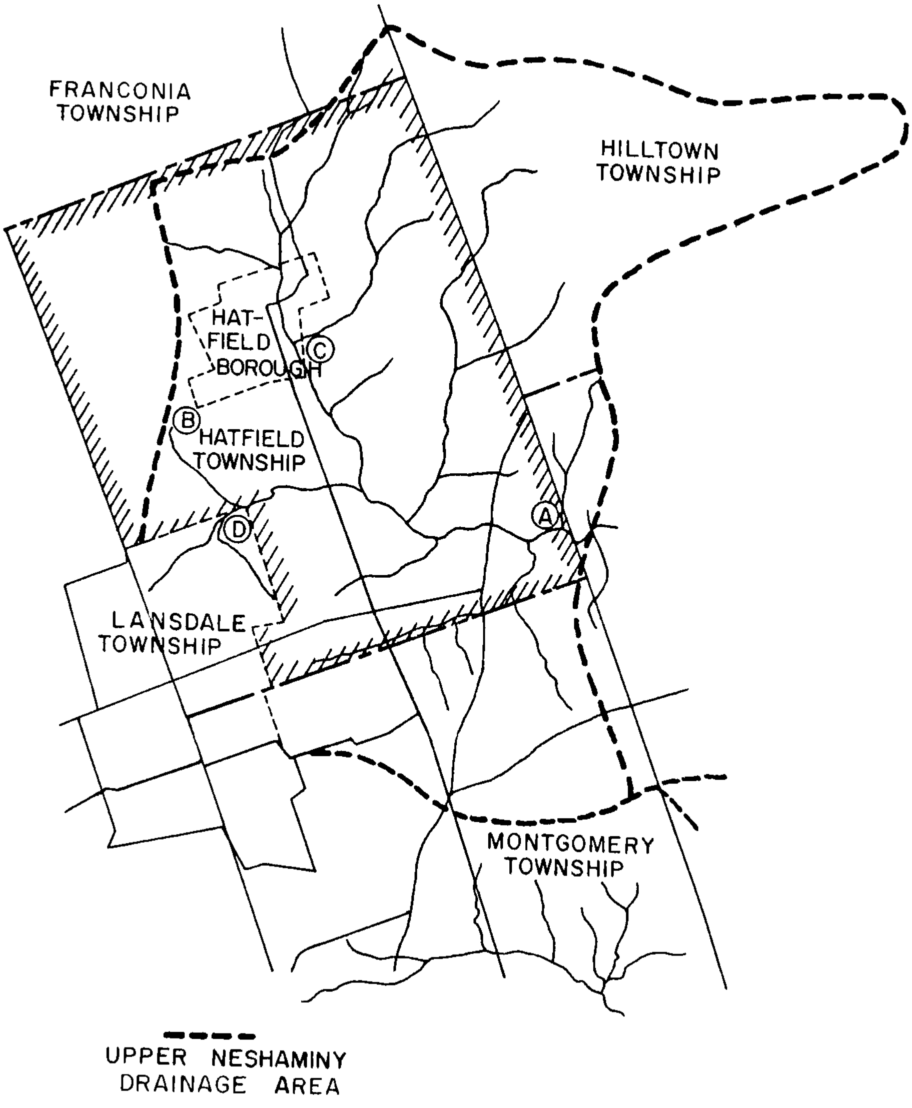


Figure 2. Area of Hatfield Township

While the Hatfield Borough plant (point "C") could, perhaps, have provided temporary treatment capacity to portions of Hatfield Township, north, and east and west of the community, local political consideration dictated that they did not desire to enter into any arrangement with the Township at that time. Therefore, the obvious conclusion of a single treatment facility for the entire Township at point "A" was finally accepted by the Authority as the basic premise in their sewerage planning program.

This location also would be ideally suited to providing future service to the total area which drained to the site, including portions of Hilltown Township, which lay in an adjacent County, and both Lansdale Borough and Hatfield Borough. The development of comprehensive regional waste treatment was not a consideration in 1962, but its potential in later years had to be one of the points considered in the selection of the site for the treatment facility.

LEGAL DIFFICULTIES

Following completion of the preliminary report and its approval by the Hatfield Township Municipal Authority, the detailed design of the project, including a secondary sewage treatment plant and interceptor and collector sewers, proceeded rapidly. The project was ready for advertisement and bidding in the late spring of 1963. The bidding actually took place, but the implementation of construction was stopped by legal action brought by a number of tax-payers against the local governing body and the Authority.

As was not uncommon in the early 1960s, citizens' groups agitated against sewage treatment and its imposition of user charges as a replacement for on-lot sewage disposal units of an individual nature. The legal problems delayed the project for two years, but the Supreme Court of Pennsylvania ultimately ruled in favor of the Authority, and the project proceeded to a second bidding stage in early 1965, and contracts were awarded and construction commenced in May 1965.

During this period between 1963 and 1965, certain sewer lines were constructed to service a new secondary school, and were temporarily connected to the Hatfield Borough system. These new sewer lines were then disconnected from the Borough system in 1966, and formed a part of the total collection system constructed as part of the 1965 project.

1965 PROJECT CONSIDERATIONS

At the time of the design of the 1965 project, consulting engineers were under strict requirements to conform to a "Sewerage Manual", published by the Pennsylvania Department of Health, which provided design parameters for all types of sewage treatment facilities. In those days, the procedure to be followed in the design of a project involved initial meetings with regional engineers for the Pennsylvania Department of Health having jurisdiction over a specific geographical area. In these initial meetings, design concepts were discussed and the general design details were developed. Strict adherance to the requirements of the Sewerage Manual was a very definite factor in the selection of the process, and in the sizing of units.

The original 1965 plant was a secondary treatment plant, utilizing a main pump station with three raw sewage pumps, two drag-chain type primary clarifiers, a two-compartment common wall aeration tank with diffused air, two three-phase separation secondary clarifiers, and a contact chlorine tank, as shown in Figure 3.

This original plant was sized for an average daily flow, according to the requirements of the Pennsylvania Department of Health in their Sewerage Manual, at 0.9 mgd (3,406.5 m^3/day). The maximum rate of flow was anticipated at 1.35 mgd (5,110 m^3/day, and a peak rate of flow of 1.8 mgd (6,813 m^3/day).

Based upon rational design criteria, however, this first plant project had an average daily flow capacity of 1.3 mgd (4,920.5 m^3/day).

This ability to absorb a hydraulic loading in excess of normal design parameters at that time was based upon a conservative design of overflow rates for the primary and secondary clarifiers, and the fact that the Sewerage Manual required a six hour detention time in the aeration tanks as the fundamental design parameter, without regard to the mode of operation of the unit, the mixed liquor suspended solids, or any other factor which would demonstrate a lower total detention time requirement.

The total loadings were 1,530 pounds per day of BOD_5 (694.6 kg/day), and 1,800 pounds per day of total suspended solids (817.2 kg/day).

In addition to the hydraulic units at the treatment facility, there was provided a complete solids handling system, which included a sludge holding tank, or sludge thickener, vacuum filtration equipment, and a multiple hearth incinerator. The solids handling equipment was provided with a capacity three times that of the average rational daily design flow, or 3.6 mgd (13,626 m^3/day). It was the intention that the solids handling would operate on a oneshift basis up to the average daily flow of the plant, and upon future hydraulic expansions, could begin to operate on a second, and even a third shift per day, thus precluding the necessity of additional capital construction in the solids handling area when growth requirements dictated hydraulic plant expansions.

Inasmuch as the initial sewage treatment plant, constructed in 1965, was constructed concurrently with a new sewage collection system for the community, the only mode of pretreatment provided ahead of the main raw sewage pumps, was comminution. No facilities were provided for grit removal, nor for screening of gross solids and rags. This was a quite common practice in the United States in the middle 1960s, when an

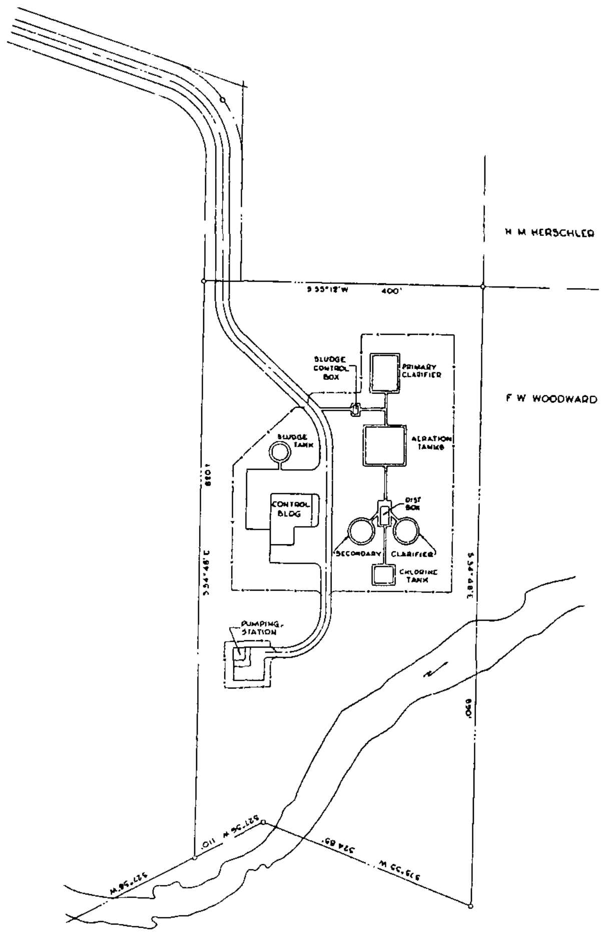


Figure 3. Plan of 1965 Sewage Plant

entirely new system was placed in operation, inasmuch as the new condition of sewage collection lines was assumed to preclude the entrance of large amounts of grit into the sewage treatment plant process.

In Figure 3. a general layout plan of the Hatfield Township sewage treatment plant as it was constructed in 1965, is shown.

CONSTRUCTION OF 1965 PROJECT

The original Hatfield Township sewage treatment plant, commonly known as "The 1965 Project", was started in the construction phase in June 1965, and was officially certified as completed in August 1966. Portions of the plant were placed in operation during the summer months of 1966, and by the first part of 1967 the plant was in full operation, with virtually all of the initial operating problems resolved.

This 1965 project, as it related to the construction of the sewage treatment plant, had a total construction cost of $848,767. In Table 1, which follows, the construction costs for the 1965 sewage treatment plant are shown in more detail.

The net project cost, after deduction of a Federal Grant, was financed by the Hatfield Township Municipal Authority through the sale of Authority Revenue Bonds to the general public, through a Bond Underwriting firm. Included in the total Authority Bond issue of $2,700,000, was the cost of providing interceptor sewers and collecting sewers throughout the entire township not sewered in 1965.

TABLE 1. HATFIELD TOWNSHIP MUNICIPAL AUTHORITY

CONSTRUCTION COSTS - 1965 PROJECT

SEWAGE TREATMENT PLANT

General Construction	$745,844
Plumbing Construction	26,289
Heating (Ventilating) Construction	13,300
Electrical Construction	63,334
TOTAL 1965 TREATMENT PLANT CONSTRUCTION COST	$848,767

OPERATION OF 1965 PROJECT

As indicated previously, the sewage treatment plant for which construction was commenced in the spring of 1965, was placed in full operation in September 1966. From the period September 1966 through the early part of 1967, connections were being made to the new sewage collection system throughout the Township, and full connection and subsequent development of total flows did not actually fully occur until early 1968.

The initial operation of the new facility in the fall of 1966 encountered the usual operating problems, including those occasioned by the hiring of a new work force, generally unfamiliar with this type of operation. However, by 1968, the initial operating difficulties had been resolved and the plant was in sound operating condition.

In Table 2, plant operating characteristics are indicated for the period of late 1967 to early 1970. The only parameters required for reporting to the State regulatory agency were BOD and Suspended Solids.

TABLE 2. PLANT OPERATING CHARACTERISTICS

HATFIELD TOWNSHIP SEWAGE TREATMENT PLANT

1967 - 1970

BOD_5 Raw, mg/1	227
BOD_5 Effluent, mg/1	13
% BOD Removed (19 samples)	94
Suspended Solids Raw, mg/1	171
Suspended Solids Effluent, mg/1	12
% Suspended Solids Removed (16 samples)	93
Average D.O. in Effluent, mg/1	9.6
Phosphate (PO_4) Raw, mg/1	23
Phosphate (PO_4)•Effluent, mg/1	10
% Phosphate (PO_4) Removal (11 samples)	56.5

The average daily flow during this period approximated 660,000 gpd (2,498 m^3/day), or approximately onehalf of the rational design capacity. As can be seen by an inspection of Table 2., the degrees of BOD removal and suspended solids removal exceeded 93%

While phosphorus was not a required parameter in the late 1960s, the data for the removal of phosphate is indicated in Table 2., as being approximately 56.5%. This represented an upper limit which could be theoretically achieved by the activated sludge process utilizing some form of solids handling other than sludge digestion.

At the time of the design of the advanced waste treatment facilities, the existing sewage treatment plant at Hatfield Township was producing a high degree of BOD and suspended solids removal, and a very commendable percentage of phosphate removal, but the facility was operating at about one-half of its capacity. Nevertheless, the operation of the facility represented an achievement of efficient operation ranking in the very upper portion of those treatment plants in operation at that time in the United States.

SECTION IV

ADVANCED WASTE TREATMENT

GENERAL

About the time of initiation of construction of the 1965 Hatfield project, the Federal Government began consideration of more stringent guidelines for effluent discharge from sewage treatment facilities. In particular, the Federal Water Quality Administration began to consider limiting effluent values versus the prior and common practice of establishing simply a percentage removal. Under the old manner of operation, primary treatment was generally considered to be that which provided 35% BOD removal and up to 60% suspended solids removal, and secondary treatment was considered to provide between 85% and 90% BOD removal and 90% to 95% suspended solids removal. Any other treatment beyond secondary treatment, such as the use of slow sand filters, polishing ponds, or the like, were given no special name or consideration, and were infrequently used.

In the middle and late 1960s, attention focused on the condition of many fresh water lakes throughout the United States, and in particular, the five Great Lakes. Of prime concern was the very advanced deterioration of Lake Eire, and the Federal Water Quality Administration conducted a number of seminars in the Great Lakes Basin, aimed at disseminating technology then available on nutrient removal from waste water which would lessen the impact on the lakes, and would permit them to reverse the trend of decay which had been accelerating for a number of years.

Concurrently with the Federal studies, the State of Pennsylvania began to classify each and every stream in the State, and among the very first to be so classified was the Neshaminy Creek, into which the effluent from the Hatfield Township sewage treatment plant discharges. New limiting effluent quality criteria were published in July 1967 for the Neshaminy Creek Basin, and constituted the first such criteria promulgated within the State of Pennsylvania. Because of the size of the stream into which the discharge is made, and because of the large number of communities utilizing this stream, the limitations which were established were extreme in nature and exceeded any of those previously published throughout the United States, excepting where actual physical discharge was completely eliminated from a surface water.

SANITARY WATER BOARD ORDER OF 1967

On July 1, 1967, the Sanitary Water Board issued an Order to the Hatfield Township Municipal Authority setting waste discharge limitations, and requiring that a preliminary study be completed and submitted to the State of Pennsylvania on or before July 1, 1968, indicating how the community intended to meet the new waste discharge limitations.

The new waste discharge limitations, as published in July 1967, are reproduced in Table 3.

The standards set in this Order were most stringent, and appeared, in 1967, to be somewhat contradictory. They were not in general keeping with other standards which had been published for other areas, notably in areas such as the Lake Michigan drainage basin.

TABLE 3. WASTE DISCHARGE LIMITATIONS

HATFIELD TOWNSHIP

	Monthly Average Value, Not More Than:	Maximum Single Value
5-Day BOD	4 mg/1	10 mg/1
Coliform Organisms:		
Date: 5/15 - 9/15 of Any Year	100/100 ml	2,400/100 ml
Date: 9/16 - 5/14 of Any Year	1,000/100 ml	20,000/100 ml
Suspended Solids	15 mg/1	50 mg/1
Total Soluble Phosphate (PO_4)	0.2 mg/1	1.0 mg/1
Dissolved Oxygen	Not Less Than 5 mg/1 at Any Time	

The specific criteria developed in the Order from the Sanitary Water Board, and reproduced in Table 3., requires some individual comment. The first criteria of a maximum of 4 mg/1 of 5day BOD as a monthly average, and a maximum of 10 mg/1 of 5day BOD at any one time, required that the effluent be virtually suspended solids free. It also required virtually total conversion of both carbonaceous and nitrogenous BOD, and most probably, the utilization of a tertiary filtration step, in order to capture as much of the suspended solids as possible. The limitation of a monthly average value of not more than 4 mg/1 appeared, at least in 1967, to be unrealistic, in that the accuracy of the 5day BOD test was not considered reliable at such low values.

The total suspended solids limitation of a 15 mg/1 maximum as a monthly average value, and a maximum of 50 mg/1 at any one time, appeared to be completely inconsistent with the BOD limitation. It appeared, theoretically, that a waste effluent containing 15 mg/1 suspended solids could not possible produce a 4 mg/1 BOD concentration.

The criteria for coliform organisms began its introduction in the United States with this Order, and relies primarily upon Post-chlorination as a means of achieving the limiting values. This is not a particularly difficult situation, inasmuch as post-chlorination is a widely utilized concept throughout the United States, and has been required by a number of the States for many years.

The criteria for total soluble phosphate (PO_4) was the most stringent requirement, and the one which appeared to be very much in need of modification. This limitation of 0.2 mg/1 of total soluble phosphate in the effluent as a monthly average, would be extremely difficult to achieve, as would even the upper limit of 1.0 mg/1 at any one time. It was pointed out to the State regulatory agency in 1968 that the then current standards in other areas of the country, and as suggested by the Federal Water Pollution Administration, provided for a minimum of 80% removal of phosphorus as P.

Investigators in sanitary engineering expressed the results of phosphorus analysis as P (phosphorus) and not as PO_4 (phosphate), or P_2O_5 (phosphorus pentoxide). The relationship between the various forms are shown in Table 4.

Reference to this table will indicate the relationship between PO_4 and P, to be PO_4 x 0.327. Thus, the requirement of the Sanitary Water Board, expressed as phosphorus (P) would be 0.065 mg/1 P as a monthly average value, and not more than 0.33 mg/1 P at any time.

TABLE 4. FORMS AND MEASUREMENT OF PHOSPHORUS,

CONVERSION FACTORS

In water analyses all measurements should be reported as mg/1 of P.

To Convert X to P Multiply By:	X	To Convert P to X Multiply By:
1.00	P	1.00
.451	P_2O_5	2.29
.327	PO_4	3.06
.316	H_3PO_4	3.16
.200	$Ca_3(PO_4)_2$	5.00
.181	$Ca_5OH(PO_4)_3$	5.51

In the report prepared at the request of the Hatfield Township Municipal Authority for presentation to the State regulatory agency in June 1968, it was indicated that the technology then prevailing would preclude meeting this effluent limitation. As a matter of evolvement over the past several years, the State regulatory and Federal regulatory agencies have informally agreed that this criteria cannot be consistently achieved although they have not modified the actual written requirements. When the 1970 project was designed and submitted for State and Federal approval, the statement was made that the phosphate limiting criteria could not be met, and it was indicated that the Authority would meet the 0.2 mg/l as PO_4 fifty percent of the time, 0.5 mg/l as PO_4 seventy-five percent of the time, and 1.0 mg/l as PO_4 one hundred percent of the time. Although this was never formally acknowledged by the State and Federal regulatory agencies, the permit to construct the new facility was issued on the basis of the information submitted.

1968 REVIEW OF CURRENT TECHNOLOGY ON PHOSPHORUS REMOVAL

As part of the report published in June 1968 for submission by the Hatfield Township Municipal Authority to the Pennsylvania Department of Health, a review of the then current technology on phosphorus removal was included. This review of the then current technology was the result of an intensive literature search, attendance at a number of Great Lakes Basin seminars conducted by the Federal Water Quality Administration, and discussions with the leading equipment manufacturers, as to the most current technology then being produced by them, or being considered for production through their research work.

In order that the reader may have some indication of the thinking which lay behind the development of the Hatfield Township advanced waste treatment plant design, there is included in this portion of the study, a complete reproduction of the data presented in 1968 to the State regulatory agency concerning the review of the then current technology on phosphorus removal. As the reader considers this information, he should be reminded that this information was written in 1968, and at that time the technology on phosphorus removal was severely limited, and was generally confined to laboratory and bench-scale studies, with the exception of the advanced waste treatment facility at South Lake Tahoe, California.

The following is a direct quotation from the 1968 report, as submitted by the Hatfield Township Municipal Authority to the State regulatory agency.

> *"A review of past thinking in the wastewater field reveals that intermittent attention has been paid, until the last three or four years, to any parameters for waste discharge acceptability other than the popular BOD and suspended solids criteria.*
>
> *That there is a need for additional parameters has been recently amply demonstrated.* [4] [5] *Sawyer,* [5] *has*

indicated that 'In any system of nutrient control, phosphorous removal is considered absolutely essential because of the ability of certain blue-green algae to fix nitrogen gas (atmospheric nitrogen) contained in the water.....The degree of (algae) control to be expected will depend in large measure on the percentage of total input to a lake that can be eliminated or removed'."

The attention focused on the need for nutrient removal has resulted in a current, rather large-scale investigation program by many individuals into the most effective means of removing phosphorus from waste discharges. Virtually all of the present studies are being conducted in the laboratory, with some pilot plant work having been completed in the field. Full-scale plant investigations of the problem are just now emerging.

One of the prime considerations in phosphorus removal is that phosphorus is conserved throughout the entire waste treatment process, that is, there is no net loss through treatment. Phosphorus enters a treatment process in its highest oxidized form, and no common biological systems can reduce phosphate. Phosphorus removal is probably a combination of cellular growth and inorganic solubility, and is associated with the sludges formed in the treatment process.[6]

The past approaches to the phosphorous problem centered mainly around attenpted control by biological means, and attempted control through adjustment of the mineral composition of the effluent to precipitate phosphorous. A more recent approach has been to blend chemical precipitation with active biological solids. Another recent approach has been chemical treatment or raw wastewater through the use of specifically lime.

Considering biological methods of phosphorus removal, one of the most detailed studies was conducted in 1964 by Levin & Shapiro.[7] They demonstrated that biological uptake of orthophosphate in excess of that required for cellular growth is possible, and termed this fact"luxury" uptake. Their studies, which included both laboratory and field plant studies, indicated that dissolved oxygen is essential to, and exerts profound control, over orthophosphate uptake, and that the rate at which oxygen is applied greatly affects the uptake capacity of the sludge organisms. They also found that in wastewater treatment plant operation, dissolved inorganic orthophosphate leaches out of sewage organisms when the dissolved oxygen level of the sewage is permitted to fall. If there is insufficient aeration, the leakage may occur in the aeration basin. If there is sufficient oxygen in the aeration basin, the leakage will occur in secondary settling, as the sluge blanket rapidly consumes available dissolved oxygen. It appears that the pH of the mixed liquor in the aeration basin vitally affects orthophosphate uptake, with a maximum uptake occurring in the pH range 7.0 - 8.0. A small amount of return sludge in raw sewage significantly improves the orthophosphate uptake.

An evaluation of a full-scale plant operation utilizing biological control has been reported in detail by Vacker, etal.[8] This evaluation of the San Antonio, Texas wastewater plant revealed that in excess of 87% of the phosphorus was removed through the biological process. Vacker reports that there are four requirements for effective phosphorus removal; first, the incorporation of the phosphates into the solids of the system; second, the production of enough solids of high phosphate uptake capacity; third, removal of solids from the system; and fourth, non-return of phosphate-containing degradation products of the solids removal. It was reported that an optimum rate BOD/aeration solids loading should be in the range of 50 Lb. BOD/100 Lb. of aeration solids, and that avoidance of long retention of solids in the secondary clarifiers during low flow, through the use of a sludge return surge tank, was important. Also reported, was the fact that rate of raw waste flow controlled to provide the optimum BOD/aeration solids loading, through the use of surge tanks, appeared to be vital. Vacker further indicated that it was necessary to maintain a DO in the aeration such that the DO does not drop below 2 mg/l in more than half the tank, and reaches a level of about 5 mg/l at the effluent end. This must be accomplished, however, while avoiding over-aeration, which would result in excessive nitrification and aerobic digestion of solids. The phosphate rich waste activated sludge must be disposed of completely apart from primary and secondary processes.

This study was an attempt to determine why the San Antonio Rilling Plant produced such a high phosphorus removal, when eight other plants in Texas, with approximately the same BOD and Suspended Solids efficiencies, were removing only 9% to 55% of the phosphorus. Bunch[2] indicates that anticipated phosphate removal through biological means in an activated sludge plant is in the range of 30-50%, with the latter a maximum. The operation at San Antonio, and a similar operation in Baltimore, Maryland, consistently produce higher phosphorus removals, for reasons that are not yet entirely clear.

Bunch[2] suggests that among the variables, aeration rates of 3 to 7 cfm/gal., and detention times of 4 to 6 hours appear to be desirable, with aeration rates being probably more critical. He further suggests avoidance of excessive phosphorus leakage in secondary clarifiers can be best accomplished if solids detention in these units is maintained at less than 30 minutes.

In another approach to phosphorus removal, that of adjustment of the mineral composition of the biological effluent to precipitate phosphorus, Brunner[9] reported on studies using lime and alum. This approach represents the tertiary approach, that of following conventional treatment with a third stage process intended specifically to reduce phosphorus content.

When lime is used as the precipitate, the equipment used is the same as that utilized in lime softening of water, i.e., up-flow clarifiers with recycle of sludge. Filtration following settling improves phosphate removal.

The lime dosage must be determined specifically for each application, since it is affected by concentrations of other materials in the water, especially bicarbonate alkalinity. Experimental results indicate phosphate removal improves with increasing pH. At pH 9.0, results have been obtained at 80% removals. The pH of the lime treated water is likely to be too high for direct discharge without recarbonation. In large plants, it may be practical to calcine the sludge and recover usable lime.

Where alum is used as the precipitate, the equipment should be the same as that used for alum clarification, i.e., a horizontal flocculator-settler arrangement. Use of filters after settling will increase the phosphate removal, but it appears that the 80% removal can be achieved without filtration.

The alum dose is difficult to predict, but may be on the order of two parts of aluminum by weight, per part of phosphorus. For very high degrees of phosphate removal, the ratio is four to one. For municipal secondary effluent, the required dose of commercial alum is likely to be 200 mg/l or more for 80% phosphate removal. The sludge formed is voluminous and of low density, and presents handling problems.

Another tertiary process of importance is that now in operation at Lake Tahoe, California, and reported by Culp, R. L. & Roderick.(10) This plant employs a patented process which includes two fundamental steps. The first is the removal of the particulate matter by chemical coagulation, and removal of the resulting precipitate by means of filtration. The filtration is accomplished on separation beds, utilizing filter media blended hydraulically to produce an overall media that decreases in void space from coarse to fine in the direction of filtration. The second step involves adsorption of dissolved matter on granular activated carbon, using carbon columns. Carbon regeneration equipment is provided at the plant to reactivate the exausted carbon for reuse. The initial operations utilized alum for the chemical precipitation, and at an average dosage, achieved a final phosphate content varying between 0.1 and 1.0 mg/l. BOD removals were accomplished to less than 1 mg/l, suspended solids to less than 0.5 mg/l, and the coliform removal was to less than 2.2 MPN/ 100 ml.

Additional data on the South Tahoe plant has been reported by Slechta and Culp, G. L.(11) Pilot studies on the use of lime as the chemical precipitant reveal similar material concentration percentage removals. In addition, the use of lime raises the pH to a point where pilot studies were run on ammonia stripping, in a stripping tower, as a combined means with the existing process, of phosphorus, nitrogen and organic treatment in one plant.

Another approach to the phosphorus removal problem has been studied by several persons. This method involves lime treatment of the raw wastewater ahead of the biological process. Buzzell & Sawyer(12) report that lime treatment of raw water can effect an 80% to 90% removal of total phosphorus with a better than 97% removal of soluble inorganic forms. This same treatment can remove 50% to 70% of the BOD, almost 25% of the nitrogen, and 99.9% of the coliform bacteria. The lime requirement is independent of the phosphorus content and can be estimated in terms of the alkalinity. Lime dosage can be controlled by pH measurement, and for each situation, the optimum pH must be determined. The volume of sludge produced in lime treatment is approximately 1% of the volume of wastewater treated, and may require chemical conditioning to improve the filterability of the sludge.

In connection with the removal of phosphorus by precipitating raw wastewater with lime, Albertson & Sherwood[13] have reported in detail. Their findings indicate that there are two fundamental recognized relationships; first, the chemical dose for phosphate removal is controllable, and depends upon the desired residual phosphate concentration, and not necessarily the initial phosphate concentration; and second, biological phosphate removal is difficult to control and depends upon the needs of the activated sludge cell, and is independent of the phosphorus concentration--as long as a phosphate deficiency does not exist. Tertiary treatment does not appear to allow for an optimum combination of these two fundamentals. It allows the activated sludge to extract the required amount of phosphorus, and then requires chemical removal to the desired residual phosphorus level. In a patented process, Albertson & Sherwood extract phosphorus to a specific concentration by lime precipitation of the raw sewage, then the activated sludge process is used to extract the remaining phosphorus to reach the desired level.

It is reported that this patented approach reduces chemical costs in the range of 60%-70%, over tertiary chemical precipitation. It was also reported that solids recirculation around the primary unit would enhance clarification, and that as much as 25% more phosphate is removed with recirculation than with straight chemical precipitation. A secondary effect of recirculation appeared to be a much clearer overflow liquor, which resulted in even lower BOD addition to the secondary system.

It is reported that the lime must be added to the flocculator-settling unit at a pH of 9.0 to 10.0, depending on waste characteristics. The majority of the phosphate, 85% of the suspended solids and 65% -75% of the raw BOD is removed in this stage. It has been estimated that in some cases, the requirement for recarbonation may be eliminated through the release of CO_2 in the activated sludge system.

It appears that sludge handling and dewatering may benefit from this approach, since it practically eliminates conventional chemical costs for sludge dewatering units because of the $CaCO_3$ in the raw sludge.

There are underway other investigations as to alternative methods of phosphorus removal. Cohen[14] has reported on utilization of activated alumina and lanthanum as precipitants. He has also reported on consideration of ion exchange, the use of soil systems, and utilization of reverse osmosis as potential means of achieving acceptable phosphorus removals. He also mentions up-flow clarification through a sludge blanket as an alternative device. His conclusions are that none of these alternatives are now ready for application, although research will almost certainly make some of those alternatives useful for application in specialized instances.

This brief review of current technology on phosphorus removal is intended to provide only a general indication of the various methods which might be applied to solving Hatfield Township's problem. It must be recognized that much of the data discussed is undergoing further analysis, and pilot and field sutdies, and that other methods of phosphorus removal will undoubtedly be investigated and may prove quite practical. It is, unfortunately, impossible for Hatfield Township, or any other community in the Neshaminy Basin, to sit by until the picture has clarified and the processes discussed herein have been fully proven beyond question. It is virtually impossible to assume that one single process will evolve as the answer to the problem. Almost all of the current technology is based on studies of aeration processes, with little work having been done on trickling filter plants. The problems of trickling filter operation most certainly will indicate that many of the processes reviewed will be unsuitable.

At a recent Seminar on phosphorus removal, conducted by the Federal Water Pollution Control Administration, it was repeatedly mentioned that individual factors, such as flow, waste composition, industrial waste volumes and character, the type of facility, and operating technology, must seriously affect the choice of method at each individual plant. There is not now just one answer, nor does it appear that there will be evolved a singular approach to the problem.

SELECTION OF PROCESS

The selection of the treatment process to achieve the effluent limitations previously presented in Table 3., involved a thorough analysis of the literature search, as presented in the previous section, the availability of equipment, and evaluation of the then current thinking of those engineers who were in the forefront at that time in advanced waste treatment technology.

It was finally decided that the most suitable approach to the problem would be that of chemical precipitation of the raw wastewater with lime, followed by a complete-mix activated sludge process, and the inclusion of a tertiary step, involving alum addition, and then filtration through a mixed-media filter bed.

The reasoning which resulted in this selection included the following:

1. An expanded hydraulic capacity at the new advanced waste

treatment facility would be required by reason of the rapid growth in the service area, and new primary units would be necessary. It was anticipated that significant savings could be realized by combining clarification and flocculation in the same unit.

2. The chemical precipitation of new wastewater would result in a high degree of phosphorus removal in the primary cycle, as well as a high percentage of organic removal.

3. The complete-mix aeration could be designed to function at a much lower organic loading, and the phosphorus could further be reduced by the action of biological up-take.

4. The tertiary step, including chemical addition, mixing, flocculation, settling and filtration, was required to effect the lowest possible phosphorus content, but providing for chemical precipitation in both the primary and tertiary stages, a maximum of flexibility in operation could be available, such that the optimum could be obtained between phosphorus removal and chemical costs.

Once the selection was made as to the processes, the further decision was made to provide surge storage, or flow equalization, and control the flow throughout the entire plant. A degree of controlled flow was a requirement for the filtration phase of the tertiary step, and it was apparent that since it was necessary at some point in the flowsheet, the primary and secondary stages would benefit considerably by its inclusion at the beginning of the initial process.

The total project, as finally evolved, resulted in the following:

1. Additional raw sewage pump station.

2. Primary surge storage or flow equalization in duplicate units, with the existing primary clarifiers converted to service as auxiliary surge storage tanks.

3. Duplicate circular primary clarifier-flocculator units.

4. An additional aeration basin of the same size as the existing unit, with the existing unit modified to permit complete-mix operation.

5. Duplicate rapid sludge removal secondary clarifiers.

6. The tertiary step, involving:

    a. Mixing, flocculation, and settling in duplicate units, with the settling being accomplished in tube-settlers.

b. Filtration through mixed-media filters.

7. Chlorine contact, accomplished by slightly modified existing secondary clarifiers, which would also serve as filter backwash storage tanks.

8. A new sludge thickener, with the existing sludge holding tank to be retained for stand-by use.

9. A chemical feed building, which would be combined with the surge storage tank and primary clarifier-flocculator construction, and would house the flash mixing facilities and the lime feed equipment, as well as sludge recirculation pumps and sludge draw-off pumps. A bulk lime storage silo, adjacent to this chemical feed building, was provided.

10. A filter building which would house the mixed-media pressure filters, as well as tertiary chemical feed equipment, secondary sludge return pumps, waste activated sludge pumps, and tertiary filter influent pumps.

11. Utilization of the existing sludge handling facilities, including vacuum filtration and incineration, without additional modification.

    In 1968, it was anticipated that when hydraulic flows reached 2.5 mgd (9,462.5 m^3/day) that additional sludge handling facilities would be required. This was anticipated because the character of the sludge would change with the use of lime in the primary cycle.

A plan of the 1970 advanced waste treatment plant is shown in Figure 4.

BENCH SCALE TESTING

Throughout the period when the selection of the process for the new Hatfield Township advanced waste treatment facility was under consideration, numerous testing of alternative means of phosphorus removal were simulated in the laboratory. Once the selection of the process was made, and it was determined that the major emphasis on phosphorus removal would be in the primary lime precipitation step, a series of tests were performed at the existing Hatfield Township plant in October 1968, to determine the chemical addition required to provide the optimum phosphorus removal in the primary cycle. These tests are summarized in Table 5.

The data developed in Table 5. proved to be most interesting. The Hatfield waste is high in alkalinity, averaging slightly in excess of 400 mg/l. The lime addition of 185 mg/l indicated a reduction in PO_4 to a 3-4 mg/l range, which it was anticipated would be further reduced by about 50% in the

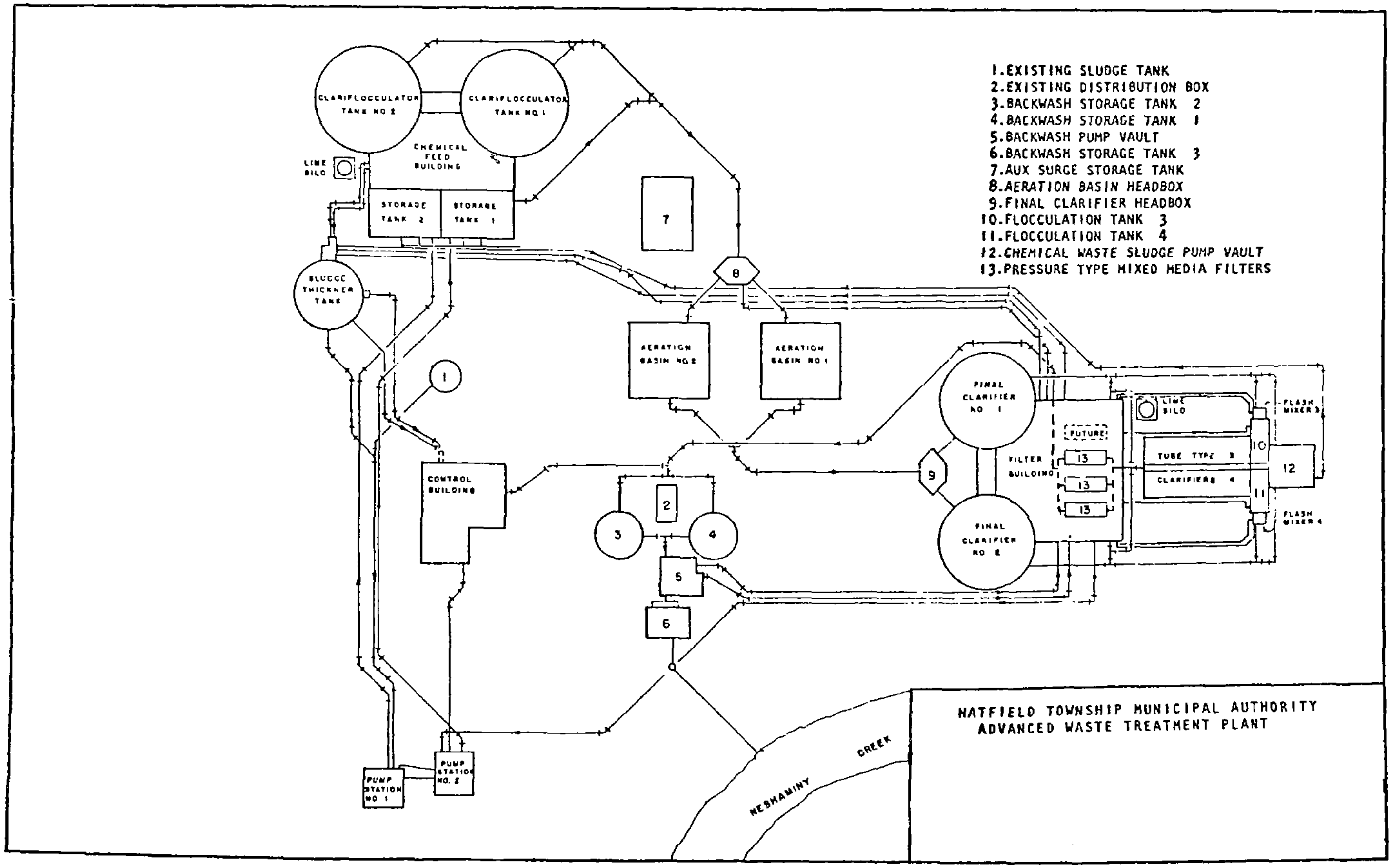


Figure 4. AWT Plant Flow Diagram

TABLE 5. PRIMARY PO_4 REMOVAL TESTS

Date	Jar Test No.	Sludge Recycle,%	pH	$Ca(OH)_2$ mg/l	Polyelectrolyte mg/l	Analytical Determination* BOD mg/l	COD mg/l	PO_4 mg/l
10/28/68	Raw	-	7.5	-	-	355	758	29.7
	Filter	-	7.5	-	-	240	557	25.0
	IA	0	9.5	185	0.5	160	448	5.86
	IB	0	9.5	185	-	-	390	5.42
	IC	20	9.45	185	0.5	-	378	4.05
	ID	20	9.45	185	-	-	454	2.68
	IE	50	9.40	185	-	-	393	3.26
	IF	100	9.45	185	-	160	417	3.92
	IIA	-	10.1	370	0.5	160	417	2.93
	IIB	-	10.1	370	-	-	431	2.61
	IIIA	-	11.65	740	0.5	-	370	0.39
	IIIB	-	7.5	0	-	-	463	32.0
	IVA	-	9.9	555	0.5	-	400	3.14
	IVB	-	9.95	555	-	-	393	2.52
10/28/68	Raw	-	7.4	-	-	365	526	25.0
	Filter	-	7.4	-	-	165	294	21.2
	IIC	20	9.45	185	-	-	424	6.54
	IID	20	9.45	185	-	-	431	5.50
	IIE	50	9.40	185	-	-	470	4.83
	IIF	100	9.45	185	-	130	232	5.21
10/29/68	Raw	-	7.95	-	-	335	624	23.0
	Filter	-	7.95	-	-	120	371	19.1
	VA	0	10.3	370	0.2	145	262	0.65
	VB	0	10.35	370	-	-	324	1.17
	VC	20	9.95	333	0.2	-	354	1.24
	VD	20	10.10	333	-	180	300	1.17
	VE	50	10.0	333	-	-	308	1.30
	VF	100	9.95	333	-	160	288	1.04
	VIA	-	9.36	185	0.2	130	254	1.83
	VIB	-	9.4	185	-	-	262	1.30
	VIIA	-	9.8	185	0.2	-	314	1.30
	VIIB	-	9.8	185	-	-	314	1.30
	VIIIA	-	10.9	444	0.2	150	254	0.52
	VIIIB	-	10.85	444	-	-	301	0.78
	VIC	-	10.05	333	0.2	-	331	1.17
	VID	-	10.10	333	-	170	300	1.30
	VIE	-	10.05	333	-	-	331	1.17
	VIF	-	10.05	333	-	170	292	1.40

*Raw = Raw Sewage Sample
Filter = Raw sewage sample filtered; no chemical addition.
Number of Tests = Raw sewage sample, with chemical additions and sludge recycle as indicated.

biological up-take phenomenon of the complete-mix aeration process. The results also indicated that a higher lime addition, sufficient to raise the pH to the range of 10, would not provide a significant enough PO_4 reduction in relation to the amount of chemical required.

The data further indicated that a pH increase in the range of 10.9 to 11.6 would have a striking effect on the phosphate residual, but at an extremely high cost in chemical addition. A considerable aid to the removal of the phosphorus by chemical precipitation was the recycling of the primary sludge. The data in Table 5. indicates this effect. From these results, and from a summary of the manufacturer's experience, the sludge recirculation ratio was chosen at 50%, with the chemical addition directly to the recycled sludge before it was to be mixed with the raw sewage.

The net result of the primary operation was anticipated to be a phosphorus reduction of 85%, and a BOD removal of 60%. This latter reduction was judged to have a significant effect upon the complete-mix process, in that the sizing of the aerators could be considerably less than with conventional primary treatment.

BASIS OF DESIGN

On completion of the bench scale testing, and with the processes having been selected, detailed design of this advanced waste treatment facility was commenced in the latter part of 1968. Due to the fact that in 1968 the type of design contemplated had not been previously developed as one total package, it was anticipated that there might be some variations from the design basis necessary in the development of the detailed drawings and specifications.

As the design developed, it was found that there were indeed a number of variations required, some occasioned by the availability of equipment, some due to the limitations of the then available control systems, and certain modifications by reason of political considerations involved in the future growth of the area that developed during the design stage.

As with the 1965 project, described in an earlier section of this study, the 1970 project required the development of a detailed basis of design, and submission of that basis to Federal and State regulatory agencies for their approval. The detailed basis of design and the component calculations for the 1970 AWT facility are included in this study as Appendix A. This data included in Appendix A is that which was submitted in early 1969 to the State and Federal regulatory agencies, and certain of these numbers were modified during the period between the completion of design and the initiation of construction, which spanned a period of some fourteen months. In actual practice, additional modifications continued during the construction period, and even now, with one year of operation completed, there are variations being made to the total facility as the method of operation utilized points to the necessity for such changes.

As mentioned earlier in this study, it was common in the 1960s for State regulatory agencies to publish design guidelines for sewage treatment plants, which set limiting factors with respect to unit sizing, detention times, overflow rates, and the like. Although these sewerage manuals were intended to be guidelines, as most often happens with regulatory agencies, they became, in many instances, inflexible requirements. One of the early problems with the design of the Hatfield Township advanced waste treatment facility was the necessity of convincing the State regulatory agency that many of the design parameters contained in their guidelines were not applicable to the type of treatment process to be utilized in the new facility.

In the development of the design of this project, it was recognized that the suburban character of this community, and other communities surrounding, which would eventually utilize this facility for treatment of their wastes, would require further expansion of the advanced waste treatment facility at some future date. In consideration of this fact, the hydraulic design capacity of 3.6 mgd (13,626 m^3/day) reflected the best estimate of capacity required by approximately 1980. In a suburban community, such as Hatfield Township, where there is ample growing room left, the development of adequate waste treatment facilities becomes a continuing process, as opposed to the more built-up and stabilized urban areas, where total flows are not likely to vary in future years.

In the development of the basis of design, peak instantaneous rates of flow were assumed at 200% of the average daily flow, or 7.2 mgd (27,252 m^3/day). This is equivalent to a flow rate of 5,000 gpm (18,925 l/minute), but the raw sewage pumping facilities and the yard piping were sized, with the addition of a third raw sewage pump, for an ultimate peak instantaneous flow of approximately 10,000 gpm or 14.4 mgd (54,504 m^3/day). Throughout the design of the individual facilities, which made up the total plant, wherever future expansion would be rendered particularly difficult, such as in underground piping or in building expansion, sufficient capacity was provided to enable the next expansion, perhaps in 1980, to be accomplished without the necessity of duplicating these costly types of construction.

Items included in the design of the project which will not require duplication at the next expansion stage, include the raw sewage pumping, the chemical feed and storage, the flow equalization, and the tertiary filtration.

The organic loadings utilized as a basis for design of this new facility reflected the commonly used parameters in the United States of 0.17 pounds per capita per day (0.077 kg/day) in both BOD and suspended solids. These loading rates are equivalent to 226 mg/l, but in 1969 the average BOD was approximately 192 mg/l, and the suspended solids 178 mg/l. The higher total BOD and total suspended solids was assumed to include a much greater use of such devices as home garbage grinders, which tend to increase significantly the total solids loading as well as the BOD. The industrial fraction of the 3.6 mgd (13,626 cum/day) design flow was assumed to have a future total BOD and suspended solids loading of 0.2 per equivalent capita per day (0.091 kg/cap./day),or the equivalent of approximately 264 mg/l.

The other basic parameter, that of phosphorus, is not indicated on the basis of design in Appendix A simply because the bench scale testing done, as well as the data available in the literature, indicated that the effluent phosphorus for a process using lime precipitation was essentially independent of the value in the raw waste. A normal figure for phosphorus in raw waste would be 10 mg/l as P, or 30 mg/l as PO_4.

The parameter for coliform bacteria does not require consideration of raw values, but instead, requires efficient chlorination of the final plant effluent.

PRE-TREATMENT AND RAW SEWAGE PUMPING

At the time of the construction of the 1965 project, pre-treatment of the sewage wastes prior to raw sewage pumping, was limited to comminution of the waste. Because the collection system was constructed new in 1965, utilization of grit facilities, and the utilization of coarse screening ahead of all unit processes, was not considered necessary. With the 1970 project, the subject of providing coarse screening and grit removal ahead of comminution was explored, but within the economic limitations placed on the consulting engineer by the community, it was felt that these desirable functions could not be considered as part of the 1970 project, but would have to be constructed at a subsequent time.

At the time of the 1965 construction, all sewage entering the sewage treatment plant reached the facility through a 24 inch (0.60 m) gravity sewer terminating at what was then the main pump station, and is now referred to as Pump Station No. 1. This station was equipped with a single comminutor with a capacity range of 0.3 mgd to 3.5 mgd (1,136 m^3/day to 13,248 m^3/day).

This Pump Station No. 1, the original pumping facility, was provided with three pumps; Pump No. 1 rated at 1,300 gpm (82.03 l/sec.)(4,914 l/min.), Pump No. 2 at 1,000 gpm (63.1 l/sec.)(3,785 l/min.), and Pump No. 3 at 500 gpm (31.55 l/sec.)(1,890 l/min.). The maximum capability of this original pumping station was 1,840 gpm (116 l/sec.) (6,964 l/min.).

The 1970 project provided for the construction of a second main pumping station, referred to as Pump Station No. 2, with a metering chamber and comminuting pit ahead of the pump wet well. This metering chamber was equipped with a comminutor of the same size as was originally installed in Pump Station No. 1, and a duplicate unit was installed in Pump Station No. 1 in what had previously been an auxiliary by-pass channel. There therefore exist at the present time, comminutors with a total capacity of 10.5 mgd (39,742 m^3/day), and there is a space reserved in the new metering chamber ahead of Pump Station No. 2, for the installation of a fourth comminutor of the same size, such that the total future capacity available for comminution is 14.0 mgd (52,990 m^3/day).

New raw sewage pumping facilities provided for Pump Station No. 2, included two constant speed pumps, each with a capacity of 3,500 gpm (220.8 l/sec.)(13,248 l/min.), with provision for the installation of a third pump of the same size. There was a 30 inch tie-in between the wet wells of Pump Stations No. 1 and 2 provided, so that the total effective capacity of both sections would be available. Upon the installation of the future third pump in Pump Station No. 2, the total available pumping capacity, with respect to the raw sewage, will be approximately 11,040 gpm (41,786 l/min.), or sufficient to handle a flow of approximately 15.9 mgd (60,180 m^3/day).

A schematic plan of Pump Stations No. 1 and 2 is shown in Figure 5.

FLOW EQUALIZATION

One of the factors which is a prime requisite for successful advanced waste treatment is the flow equalization phase of the process.

The design of a flow equalization, or surge storage, system requires the development of three factors. The first factor is the rate of total daily average flow anticipated to reach the plant in any one hour or one-half hour period throughout the 24-hour day. The second factor is the amount and the period of time during the day when recycled water from other plant processes will reach the influent point. In this category would be included filter backwash waters, thickener overflow wastes, centrifuge centrate, vacuum filter filtrate, and any other item of reasonable proportion which would find its way back into the flow stream. The third factor is the limiting flow factor for the most critical portion of the plant process.

In the case of the Hatfield Township advanced waste treatment facility design, the surge storage basins were sized as shown in Section "R" of Appendix A. The percentage of hourly flow factor was developed from a compilation of data taken from the flow charts of the 1965 plant operation during the year 1968 and the early part of 1969. In developing this factor for surge storage, it is most useful if historical data can be utilized, but in the absence of historical data, one must take into consideration unusual circumstances which might cause peaking of flows during periods of the day because of industrial operations or other local situations. This percentage factor is then applied to the total

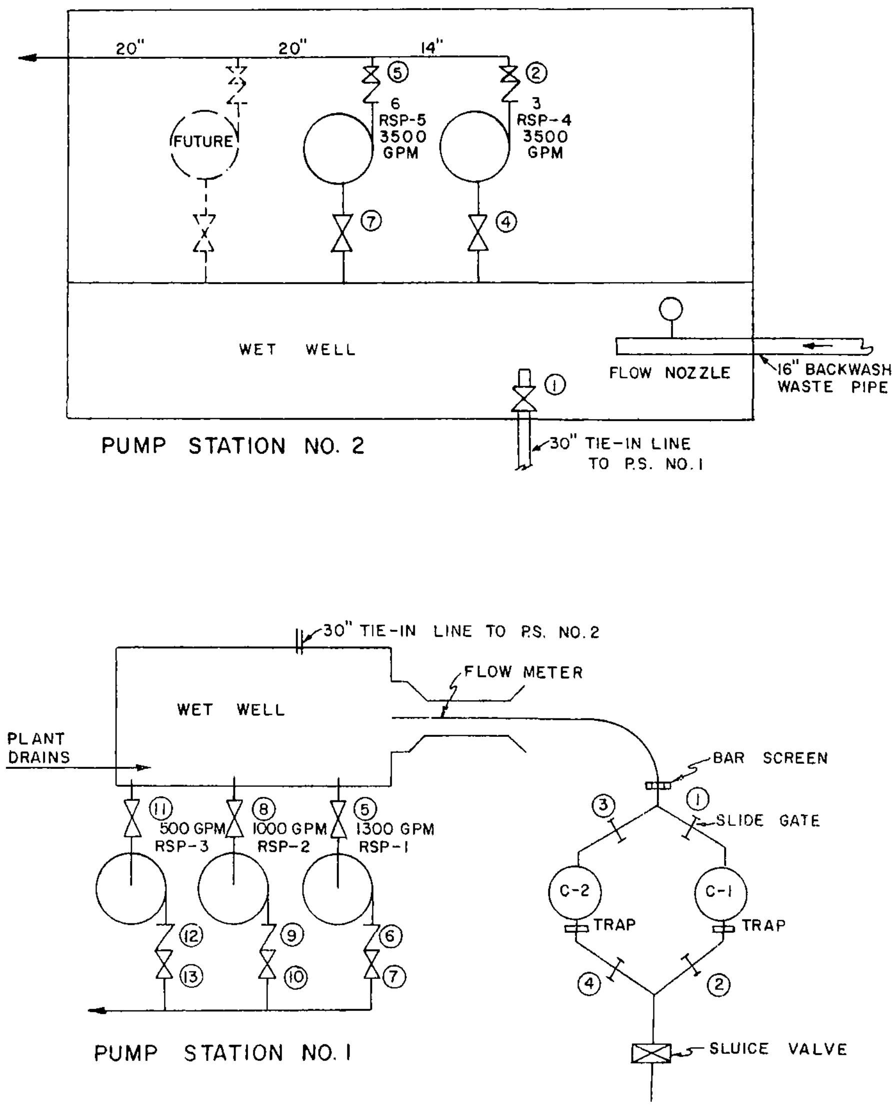


Figure 5. Schematic of Pump Station No. 1 and No. 2

average daily raw sewage flow to develop the flow in gallons for each one hour period. It is suggested that the one hour periods represent a reasonable basis for the development of surge storage requirements, although other periods of variation throughout the day might be utilized.

In the Hatfield operation, it was assumed that there would be a backwash return from the tertiary filter system, based upon backwashing each of the three filters four times per day. This would be the anticipated situation at design flow capacity. In the data shown in Appendix A, no values were assumed for the thickener overflow of the vacuum filter filtrate. In any calculation where the surge storage facilities are to be sized in close approximation to the surge buildup required to be handled, all of the flows which cycle into the influent stream should be accounted for. In the case of the Hatfield design, it was anticipated that surge storage would be provided far in excess of the requirement, and therefore, only the major sources of flow were calculated in the table.

In the design of the Hatfield surge storage system, the limiting factor in the entire process was the capacity of the tertiary mixed-media filters. In other systems not utilizing a filter as a limiting criteria, this value may be the overflow rate on a secondary clarifier, the detention time in an aeration system, or the like.

The hourly accumulation of surge storage in gallons is then the difference between the total flow from all sources in gallons in any given hourly period, and the limiting capacity of flow through the critical plant process in that period. The total buildup of surge storage required in gallons is then a cumulative summary of the hourly accumulations. Where the hourly accumulation shows a negative sign, that is the total flow through the plant is less than the limiting unit process capacity, the buildup is assigned a zero. Where a positive accumulation is indicated, it becomes a factor for that hourly period in the total buildup. Reference to the Table in Appendix A for the surge storage, indicates that at Hatfield Township, it was anticipated that the maximum surge capacity required would occur between 1:00 and 2:00 P.M., when it would amount to 206,400 gallons (781.2 m^3).

In the actual construction of the Hatfield AWT facility, duplicate surge storage tanks, each with a capacity of 268,000 gallons (1,014 m^3), were provided. Thus, the total capacity in the main surge storage tanks is 536,000 gallons (2,029 m^3). In addition to these main surge storage facilities, the existing primary clarifiers from the 1965 project were converted to auziliary surge storage use. These rectangular units have a total storage capacity of 122,040 gallons (462 m^3).

A schematic plan of the surge storage system is shown in Figure 6.

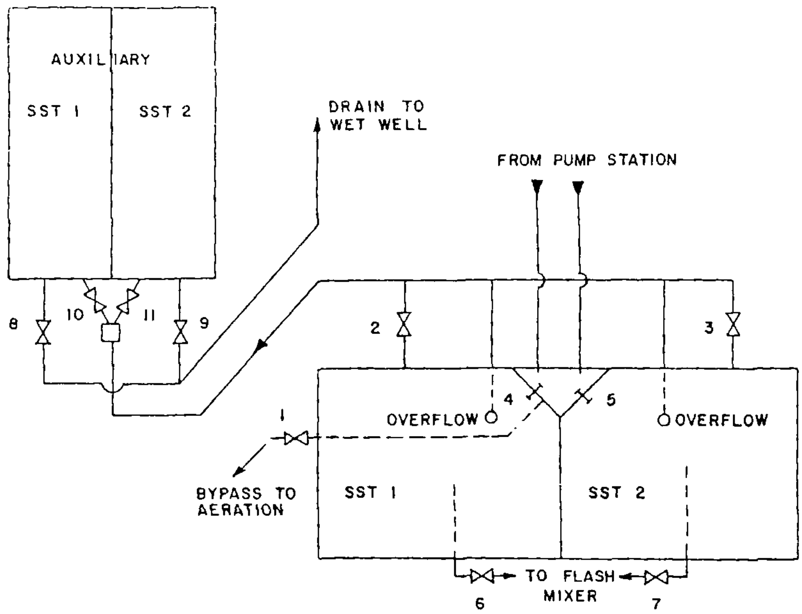


Figure 6. Surge Storage System

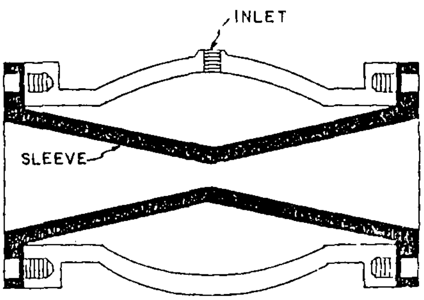


Figure 7. Red Valve Mftg. Co. Pinch Valve

Each of the main surge storage tanks is equipped with a propeller mixer for the purpose of keeping solids and grit in suspension, and these propeller mixers have proved to be only moderately successful. A temporary system has been employed in one of the surge tanks, utilizing a portable air compressor and an air line to provide more agitation to the surge tank contents, and it has been found that this scheme is superior to the use of the mixer alone, in that it keeps the solids in suspension far better. During the warm summer months, the addition of the air also helps to retard the approach of septicity.

One of the problems associated with the design of the surge storage system was the location of a suitable control device which could operate over a range of head conditions which would occur in the surge tanks as excess flows were stored. The control device finally selected is known as the "Red Valve", and a schematic representation is shown in Figure 7.

The theory of operation of the Red Valve is contained in the internal rubber sleeve, which is compressed by a combination of liquid and air to restrict the flow to permit a preset volume of flow through the unit. The initial control system utilized for the Red Valve was entirely an air pressure system, but, due to the head fluctuations in the surge tanks, one of the valves was ruptured due to an oscillating motion set up which could not be controlled. This was modified by the addition of a liquid control system, in addition to the air, which has provided a significant reduction in the tendency of the rubber sleeve to flutter.

Each of the Red Valves can be pre-set to any given flow rate, up to 2,500 gpm (9,462 l/min.).

PRIMARY TREATMENT SYSTEM

The key to the primary lime treatment system utilized at the Hatfield Township AWT facility, is the use of recycled primary sludge to provide seed for floc formation. A schematic plan of the flash mix system in the chemical feed building is shown in Figure 8.

The recycled primary sludge at the rate of one-half the average daily design flow rate, is introduced into Flash Mixer No. 2, where polymer and lime additions are made. This mixer was designed to provide 5.5 minutes detention at a flow of one-half the design flow rate. This mixed material then overflows into Flash Mixer No. 1, and is mixed with the raw sewage from the surge storage tanks, which then passes over weirs to the two primary clariflocculators. Mixer No. 1 was designed to provide a detention of approximately 0.6 of a minute, at the design average flow rate. The action in this smaller flash mixer should not be less than 30 seconds.

The primary lime feed system at Hatfield Township utilizes pebble grade

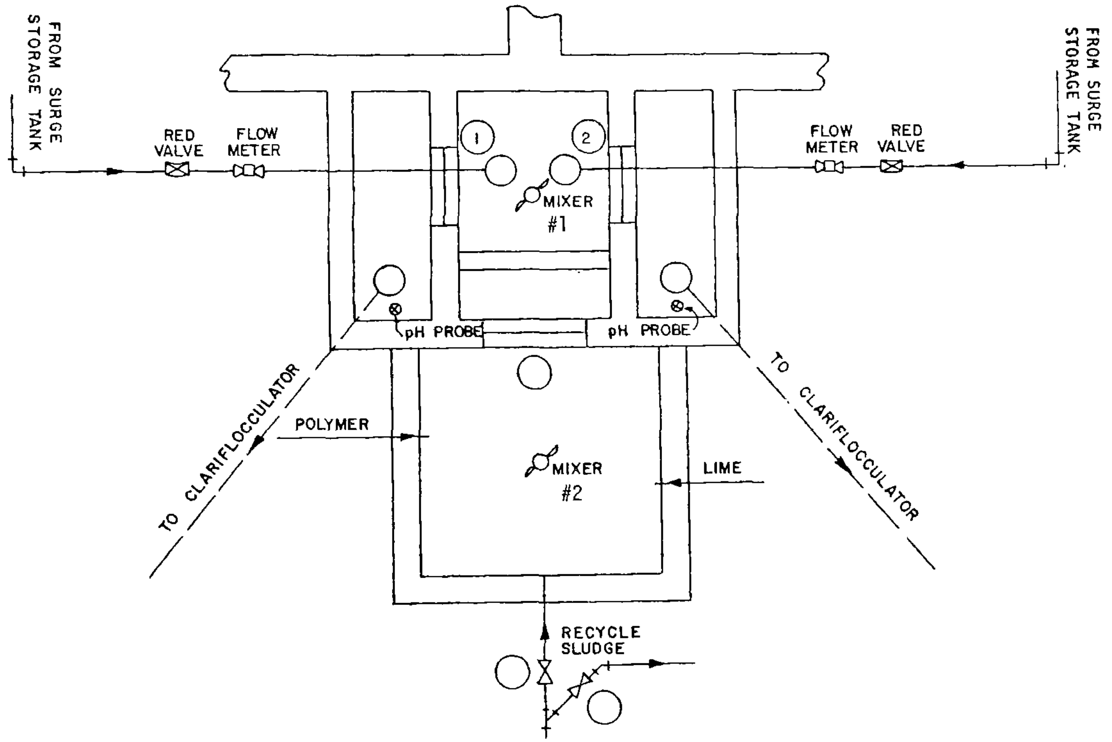


Figure 8. Chemical Feed Building-Primary Flash Mixer Station

quick-lime (calcium oxide), which is delivered to the plant by truck, and is unloaded pneumatically into a lime storage silo adjacent to the chemical feed building. Lime is then transferred as needed by an auger conveyor from the lime storage silo to the day lime storage bin inside the chemical feed building. The bulk storage silo has a total storage capacity of 970 cubic feet (27.5 m^3), or a capacity of 29 tons (26.3 metric tons). The day storage tank has a capacity of 151 cubic feet (4.3 m^3), or 4.5 tons (4.1 metric tons).

A schematic plan of the lime feed system for the primary treatment process is shown in Figure 9. The lime passes from the day tank, or lime storage bin inside the chemical feed building, into a lime slaker, where water is combined with the quick-lime to produce hydrated lime (calcium hydroxide). The slaked lime is stored in the lime slurry tank, which is located in the basement of the chemical feed building, until needed. It is then pumped to a liquid lime feeder, located just adjacent to the flash mixers on the main floor of the chemical feed building, from where it is then fed into Rapid Mix Tank No. 2. The operation of the slaker is automatically controlled by liquid level probes in the slurry tank. The slurry tank is also provided with a mixer to keep the lime in suspension. The lime is metered to the flash mixer by the use of a roto-dip volumetric feeder, and this feed is controlled by pH probes, located in the effluent chamber of Rapid Mix Tank No. 1. The pH recorder has set points, which automatically control the lime feed. Recently a flow-through pH device has been inserted into the system in lieu of the pH probes in the flash mix tank, and is found to be far more accurate and dependable. There is also a package polymer feed station, which has been provided in the chemical feed building, adjacent to the flash mixers, consisting of a polymer mix tank, a mixer, and a small diaphragm pump.

Separation of the bulk of the solids from the raw waste flow is accomplished in the primary clarifier-flocculators. There are two such units, each 60'-0" in diameter, with a 10'-0" side water depth (18.29 m diameter x 3.05 m SWD). The basic criteria for sizing a clarifier that is used in the United States is the surface settling rate, or the rate of total flow per day per square foot of surface area. A commonly used parameter for a quarter of a century has been a primary surface settling rate of between 600 to 650 gallons per square foot per day (24.4 m^3/m^2/day, or 24,432 l/m^2/day - 26.4 m^3/m^2/day, or 26,488 l/m^2/day). This surface settling rate of between 600 and 650 gal./$ft.^2$/day has been commonly utilized for normal primary treatment without chemical precipitation. With chemical precipitation, primary surface settling rates have been utilized as high as 1,000 gal./$ft.^2$/day (40.8 m^3/day sq. m).

The design of the primary clarifier-flocculators at Hatfield Township was based upon a surface settling rate at the design flow of 3.6 mgd (13,626 m^3/day) at 637 gal./$ft.^2$/day. This is an extremely conservative

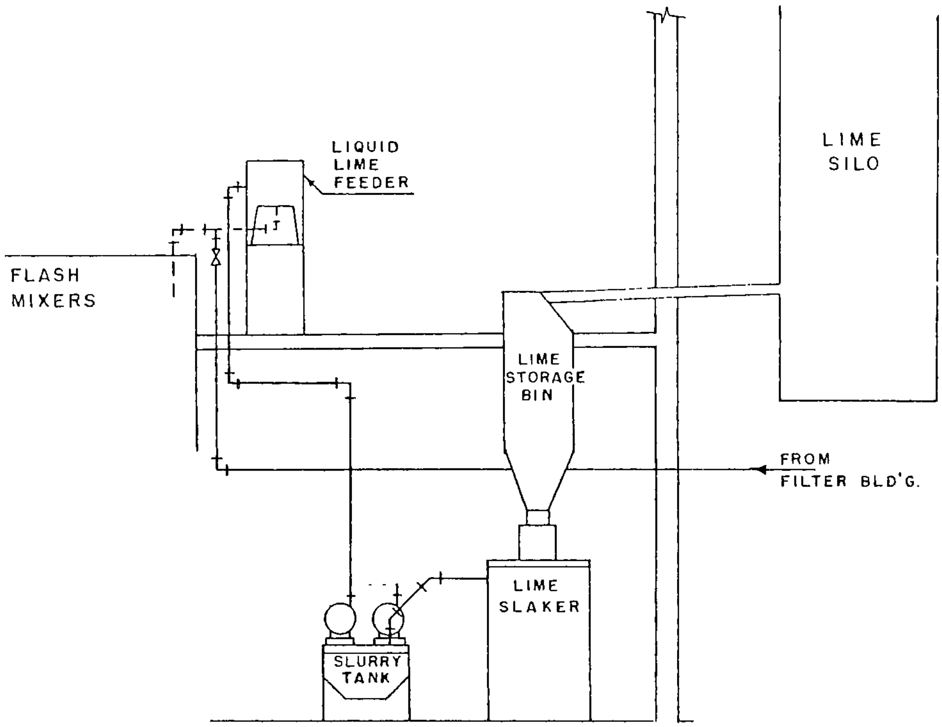


Figure 9. Lime Feed System

surface settling rate, and produces a detention time at the design flow of 2.82 hours. This design has been utilized in order that sewage flows in excess of 5.0 mgd (18,925 m^3/day) could be passed through the primary system without a serious loss of overall settling efficiency.

In the United States, is is common practice to determine the surface settling rate on the entire surface area of both the clarifier and flocculator sections, even though it may be technically argued that the net clarifier section is not equal to the total surface area. The flocculation of the mixed sewage sludge - raw waste - lime mixture is accomplished in a circular chamber in each of the clarifiers by means of small turbine mixers, which provide gentle agitation. When both clarifiers are in use, the total flocculation detention is approximately 41 minutes at the design flow, and if only one unit is in service, the flocculation detention is approximately 20 minutes at the design flow. These two detention periods include recirculation of 50% of the design average flow as recycled sludge.

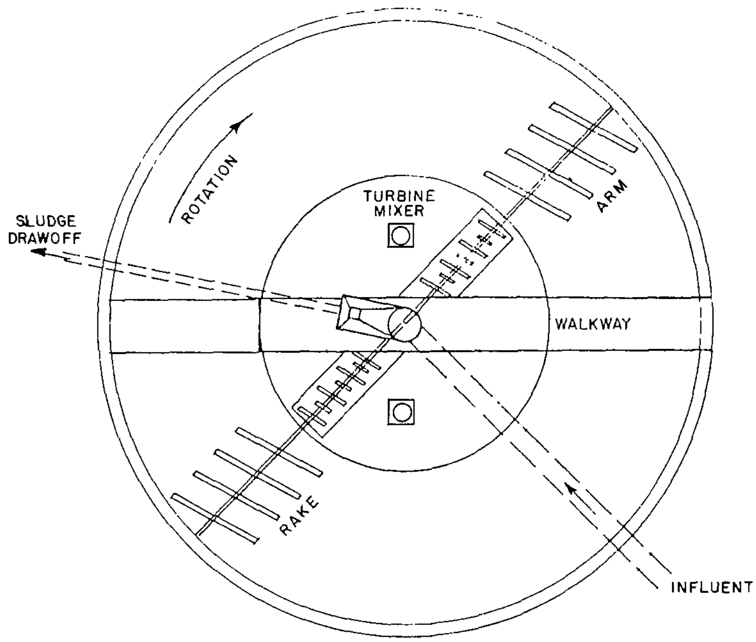
A plan of the clariflocculator tank, including a sectional drawing, is contained in Figure 10. The rake arms in the bottom of the clarifier-flocculator constantly move the sludge to the center of the tank, and the sludge is withdrawn continuously for either recycling or wasting to the sludge thickener.

This primary treatment system was designed to remove 60% of the BOD in the raw waste flow, or 4,230 pounds per day (1,920 kg/day).

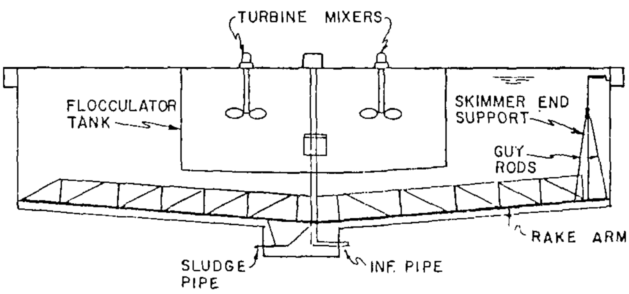
This system also removes between 75% and 80% of the suspended solids in the raw waste. The phosphorus removal is pH dependent. The pH is determined by lime addition, and phosphorus removals are independent of the raw phosphorus concentration. At low pH (9.0-9.5) approximately 2.0-2.5 mg/l of phosphorus carries over into the secondary system. The pH of the primary clarifier effluent will vary according to the pH level maintained in the flocculation zone of the clarifier-flocculators, but when this value in the flocculation zone is approximately 9.5, the pH of the clarifier effluent is approximately 9.0 to 9.1.

SECONDARY TREATMENT SYSTEM

The design of the aeration system at Hatfield Township involved, in 1969, a radical departure from the then accepted and required standards with relation to detention time in aeration tanks. Detention time was the fundamental criteria, and was usually required to be a minimum of six hours although a waste with an exceptionally high BOD loading might require a longer detention time. The design of the Hatfield Township aeration system was based upon the mixed-liquor suspended solids loading, which provided a minimum mixed-liquor suspended solids concentration of 1,716 mg/l. In support of the design basis utilized in the Hatfield facility for the aeration system, the design data supplied to the regulatory agencies for review included a discussion of the variation from



PLAN



SECTION

Figure 10. Clariflocculator

the regulatory manual design criteria involved in the Hatfield design. In order that the reader may follow the method of design utilized, the following data is quoted directly from this information which was supplied.

"The design criteria of Paragraph 72.1, beginning on Page 61 of the Sewerage Manual is aimed at producing a plant efficiency of at least 90% when dealing with a domestic waste of average strength, and when utilizing the conventional activated sludge process.

These design criteria generally assume a raw BOD in the range of 0.17 Lb./Capita, and a mixed liquor suspended solids concentration in the range of 2,000 mg/l.

For a mixed liquor suspended solids concentration of 2,000 mg/l, the BOD loading per pound of MLSS would be in the range of 0.27 to 0.28 Lb. BOD/Lb. MLSS.

In the design of any aeration system, the loading rate expressed in Lb. BOD/Lb. MLSS is now generally recognized as the dominant criteria. This criteria, together with the oxygen requirement expressed as Lb. O_2/ Lb. BOD Applied, and the temperature, determines removal efficiencies, and is considered the "rational" approach to the design of aeration systems.

The aeration tank sizing is now generally recognized as being dependent upon the quantity of mixed liquor suspended solids to be carried in the aeration tank.

The Pennsylvania Standard is in the range of 2,000 mg/l MLSS. Rational design considerations are now generally at a figure of 4,000 mg/l MLSS, and in those areas of the country where standards are not available, this figure is utilized for design purposes.

In the rational approach, the detention time in the aeration basin is generally not regarded as a critical factor so long as it is in excess of 2.0 to 2.5 hours. Sewage with a largely insoluble BOD is recognized to require a minimum detention period of 20 to 30 minutes to effect adsorption. A completely soluble BOD is recognized to require 2.0 hours detention to effect the adsorption. Since BOD adsorption generally proceeds at a slower rate than BOD stabilization, the limiting adsorption figures provide the basis for determining minimum detention time.

The reason that the Pennsylvania Standards utilize 6.0 hours as limiting minimum detention period is precisely based upon the two parameters of 0.27-0.28 Lb. BOD/Lb. MLSS, and a limiting 2,000 (+/-) mg/l MLSS. In any set of design conditions where either or both of these parameters are not valid, the limiting detention period of 6.0 hours likewise is not valid.

The aeration process for Hatfield Township utilizes the complete-mix activated sludge concept. This concept, as those of other aeration processes such as high rate activated sludge, biosorption, extended aeration and contact stabilization, is a modification of the conventional activated sludge process utilizing plug-flow aeration.

The complete-mix activated sludge system attempts to minimize the cycle of growth of the organisms in the waste by mixing raw wastes completely with the micro-organisms in the aeration tank. This puts a more uniform load on the aeration tank and gives a more uniform oxygen demand with a lower demand rate than traditional activated sludge. The same treatment results can be obtained in less time since complete mixing dilutes the waste.

Complete-mix process designs in use today are based upon the mathematical work of Professor Ross E. McKinney, University of Kansas, which is contained in a paper, "Mathematics of Complete-Mixing Activated Sludge", Journal of the Sanitary Engineering Division, ASCE, May 1962.

There are numerous equations presented by McKinney, but the key equations in determining the loading, oxygen requirements and quantity of mixed liquor suspended solids carried in a normal treatment system are equations noted in his paper as 7f, 12a and 21.

In McKinney's analysis, there is a relationship, as previously stated, between BOD loading, temperature, oxygen requirements, and removal efficiencies. If one of these factors is changed during operation, the effect on the other factors can be predicted as follows:

Decrease in temperature causes decrease in removal efficiency.

Decrease in BOD applied causes increase in O_2/BOD applied.

Decrease in BOD applied causes increase in removal efficiency.

Decrease in MLSS causes decrease in O_2/BOD applied.

Decrease in MLSS causes decrease in removal efficiency.

In general, the design procedures evolved from the McKinney analysis utilize the desired BOD removal and the lowest expected mixed liquor temperature for the plant to determine the loading, and then the aeration tank sizing. The oxygen requirements are found based upon this previously determined loading and the highest expected temperature. This procedure results in an inherent safety factor in the design since the design is based upon extreme conditions and the operation will be somewhere between these extremes.

From the McKinney analysis, various sources have determined design loadings as follows:

Type of Process	*Loading*
Extended Aeration	*0.1 Lb. BOD/Lb. MLSS*
Conventional	*0.3 Lb. BOD/Lb. MLSS*
Complete-Mix	*0.5 Lb. BOD/Lb. MLSS*
Modified Aeration	*1.0 Lb. BOD/Lb. MLSS*

From the above, it can be seen that the loading rate for a conventional activated sludge system closely approaches the Sewerage Manual, where the figure is 0.27 to 0.28 Lb. BOD/Lb. MLSS. In a complete-mix system, homogeneity is approached. As feed enters the tank, it is instantaneously distributed throughout the entire tank. Due to this instantaneous action, the aeration volume smooths out variations in the feed and maximizes the effectiveness of the mixed liquor in reducing the load. In a conventional system, more mixed liquor is required to reduce a similar quantity of BOD. Thus, a lower loading must be employed to get the same efficiency as a complete-mix system.

In any aerobic system, complete stabilization of the BOD will require 1.4 to 1.8 pounds of oxygen per pound of BOD applied, depending upon the temperature. If complete stabilization is to be effected in the aeration basins, then the total oxygen input to the aeration basins would have to be in the range indicated.

However, in neither a conventional activated sludge plant, nor in a complete-mix activated sludge plant, is total stabilization a function of the aeration process. Therefore, each pound of BOD will require some lesser amount of oxygen than 1.4 to 1.8 pound to provide the desired efficiency. Application of the McKinney equations, with the attendant consideration of loading and lowest mixed liquor temperature anticipated, and loading and the highest temperature anticipated, generally result in a range of 0.7 Lb. O_2 per Lb. MLSS, where an efficiency of 85% or better is desired.

The treatment process to be utilized for Hatfield Township provides for a primary settled and precipitated sewage feed to complete-mix activated sludge tanks, followed by settling, and with incineration of the waste activated sludge.

With reference to the loading rate of 0.35 Lb. BOD/Lb. MLSS for the domestic waste, this figure is actually conservative. Based upon the McKinney analysis, a complete-mix system can be loaded at 0.5 Lb. BOD/ Lb. MLSS. In fact, the loading chosen is only slightly above the McKinney value for conventional activated sludge.

With reference to the oxygen supplied, the 0.8 Lb. O_2/Lb. BOD Applied is in the upper range of the McKinney values determined from the application of his equations, in order to produce a plant efficiency of at least 90%.

The aerators in the aeration basins have been selected to carry a minimum of 2 mg/l of dissolved oxygen in the mixed liquor at an elevation of 500 feet, and a temperature of 25° C.

The design criteria chosen have been fixed to a certain extent by the capacity of the existing aeration basin. The resultant duplication of units, coupled with the chemical precipitation of the raw wastewater will result in a concentration of 1,716 mg/l of mixed liquor suspended solids. It would have been desirable to reduce the aeration volume to achieve a higher MLSS concentration, but the design will allow a 100% increase in sewage strength without affecting performance. Therefore, even though the 3.75 hours is probably a maximum detention period, it will be utilized to cope with possible variations in future sewage strength."

As indicated in the above data submitted to the regulatory agency, it would have been desirable to size the aeration basin to carry a much higher mixed liquor suspended solids concentration, thereby reducing the size. A decision was made, however, to construct the second aeration basin of the same size as the existing unit, which was modified from diffused air to mechanical aeration, and this provided the basis for the theoretical detention time of 3.75 hours.

This detention period of 3.75 hours, which varied significantly from the then commonly accepted minimum figure of 6 hours, caused the regulatory agency to request substantiation of the design. In the work done by Professor McKinney, he developed a theory that the activity in the aeration basin, or the active microbial fraction, is an inverse function of the sludge age, which is, in turn, a function of the mixed liquor suspended solids divided by the loading of the aeration system. McKinney's work also developed a theoretical required detention time based on BOD stabilization at a rate of 25 mg/l of BOD/hour. In Table 6, which follows, the data which was submitted to the regulatory agencies as additional substantiation of the design, is reproduced.

The information in Table 6. indicates that the loading rate could have been as high as 0.53 Lb. BOD/Lb. MLSS, hence the actual loading rate of 0.35 Lb. BOD/Lb. MLSS was conservative. It also indicates that the theoretical required detention time at the design MLSS of 1,716 mg/l would be 3.6 hours, or slightly less than the 3.7 hours provided. It further shows that the theoretical required detention time at a MLSS concentration of 3,500 mg/l, and a stabilization rate of the aeration basin at a much higher MLSS concentration actually reduces the required amount of detention time.

TABLE 6. HATFIELD TOWNSHIP MUNICIPAL AUTHORITY

AERATION TANKS @ 3.6 MGD

1. Sludge Age

$$S_A = \frac{\text{Lb. MLSS}}{\text{Lb. BOD/Day}} \qquad S_A = \frac{\text{8,050 Lb. MLSS}}{\text{2,820 Lb. BOD}} = \text{2.85 Days}$$

2. Active Microbial Fraction

$$AMF = \frac{1}{S_A} = \frac{1}{2.85} = 0.35$$

3. Loading Rate

Permissable Range for High-Rate System Between 1 and 2 Lb. BOD/ Lb. AMF/Day At Average Range of 1.5 Lb. BOD/Lb. AMF/Day:

$$\frac{\text{1.5 Lb. BOD}}{\text{(1 Lb. AMF + 1.85 Lb. AMF)}} = \text{0.53 Lb. BOD/Lb. MLSS}$$

Actual Loading rate Utilized = 0.35 Lb. BOD/Lb. MLSS

4. Theoretical Required Detention Time @ 1,176 mg/1 MLSS @ Stabilization Rate of 25 mg/1 BOD/Hour

$$\frac{\text{226 mg/1 Raw BOD x 0.40}}{\text{25 mg/1 per Hour}} = \frac{\text{90.5 mg/1 BOD to Aerator}}{\text{25 mg/1 per Hour}} = \text{3.6 Hours}$$

Actual Detention Time Provided = 3.75 Hours

5. Theoretical Required Detention Time @ 3,500 mg/1 MLSS @ Stabilization Rate of 35 mg/1 BOD/Hour

$$\frac{\text{90.5 mg/1 BOD to Aerator}}{\text{35 mg/1 per Hour}} = \text{2.6 Hours}$$

In the construction of the 1965 project, the activated sludge tanks were supplied with diffused air from blowers. In the 1970 project, it was determined that mechanical mixing was required in order to achieve the complete-mix result.

It was determined that the utilization of combination aerators, providing mechanical mixing at the surface, and surface transfer of oxygen as well as sparged air diffusion at the lower level of the tank, would be desirable. Inasmuch as the three existing blowers had adequate capacity to supply this air to be introduced at the bottom of the tank, it was felt that these combination aerators were the best choice for the introduction of oxygen into the aeration tanks. The calculations for the amount of air required are contained in Appendix A, and it was decided to provide 100% of the total air requirements in each aeration basin.

A schematic plan of an aeration basin is contained in Figure 11.

The utilization of the combination aerators is far more flexible than fixed mechanical surface aerators, or diffused air, by themselves. As it subsequently occurred at Hatfield Township, additional requirements were placed on nitrification, and even more stringent future additional requirements are anticipated. Since good nitrification in an aeration basin is, in large part, a function of the oxygen transfer, the ability to provide excess amounts of oxygen is greatly to be desired. Also, during colder weather, with the solubility of the oxygen in the waste at a higher level, it is possible to supply ample oxygen to the aeration basins by the use of the surface mixing alone, and therefore save in power costs the air that would normally be supplied from the blowers. This ability to vary the air input to an aeration basin is particularly of value in those situations where flows increase gradually over a period of time, thus requiring more air. It is possible with combination units to meet these increasing oxygen needs and at the same time conserve energy by regulating blowers to only supply the necessary air.

The second, and perhaps the most important component of the total secondary treatment system, are the final settling tanks. These final settling tanks serve a dual function of separating the solids fraction from the mixed liquor flow, and providing for a rapid return of this solids fraction as return activated sludge before it has an opportunity to deteriorate. This ability to return activated sludge to the aeration system in a relatively short period ot time, while it still maintains its activity, is fundamental to the operation of any secondary treatment system.

Therefore, the design of final, or secondary, clarifiers requires attention to surface settling rates and detention times, but it also requires particular attention to the method of sludge return, which can be provided in the unit itself.

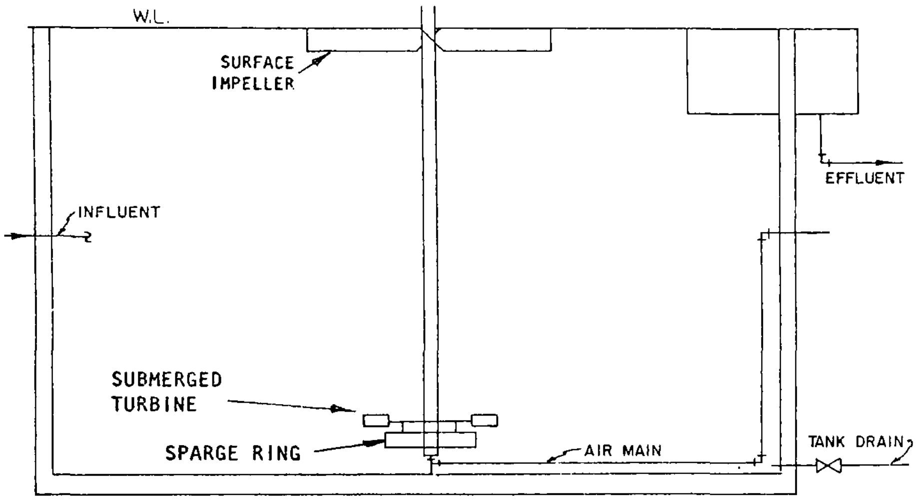


Figure 11. Aeration Tank

In Figure 12, a sectional plan of the final settling tanks used in Hatfield Township, is shown. As in a conventional settling tank, a rake mechanism is provided to channel sludge to a center hopper. In a rapid sludge removal clarifier, however, there are up-take pipes attached to the rake arms which discharge into a center well and column for rapid removal of the sludge as it settles over the entire tank bottom. The sludge withdrawal valves are located in this center section, and are provided with rings of uniform height, which can be removed so that the flow out of these pipes can be determined within relatively close limits by the variation in the height of the overflow pipe and the water level in the remaining portion of the tank.

This clarifier operates on a siphon principle. All sludges are raked toward the center of the unit with the inert, denser, sludge eventually being directly pumped from the sludge pocket. The sludge level in the sludge box is maintained at a lower water level than that of the clarifier by pumping. This creates a siphon effect which draws the lighter more volatile sludge up the withdrawal pipes mounted on the rake arms. This sludge is then recycled to maintain the reactor.

The maximum surface settling rate generally considered for a secondary clarifier unit, based upon regulatory agency criteria, is 1,000 gal/ft^2/day (40.69 m^3/m^2/day). Some designers have taken the rational approach that, with rapid sludge removal facilities, settling rates may be in the range of 1,400 gal/ft^2/day (56.97 m^3/m^2/day). The Hatfield Township design is based upon a surface settling rate of 758 gal/ft^2/day (30.88 m^3/m^2/day). The total detention time when both secondary clarifiers are in use, at the design flow, is 2.85 hours.

Based upon rational design criteria, the secondary treatment system is capable of operating at a level much higher than the design flow of 3.6 mgd (13,626 m^3/day). If the aeration system is operated at a mixed liquor suspended solids content in excess of 3,500 mg/l, and if it is assumed that the stabilization of 35 mg/l/BOD/hour is reasonable, then the aeration basins, theoretically, have a capacity, without consideration of any nitrification, of slightly in excess of 5.0 mgd (18,925 m^3/day). If a surface settling rate of 1,400 gal/ft^2/day is considered as rational for a secondary clarifier system, the secondary clarifiers then have a capacity to absorb a flow approximating 6.5 mgd (24,602 m^3/day). Thus, the limiting capacity of the secondary treatment system would appear to be approximately 5.0 mgd. With a similar rational maximum flow for the primary system, the plant process, including primary chemical precipitation, aeration, and secondary settling, can function at a much higher level than the average design capacity. This, however, is dependent upon the flow equalization eliminating peaks from the primary and secondary units of these high flow values.

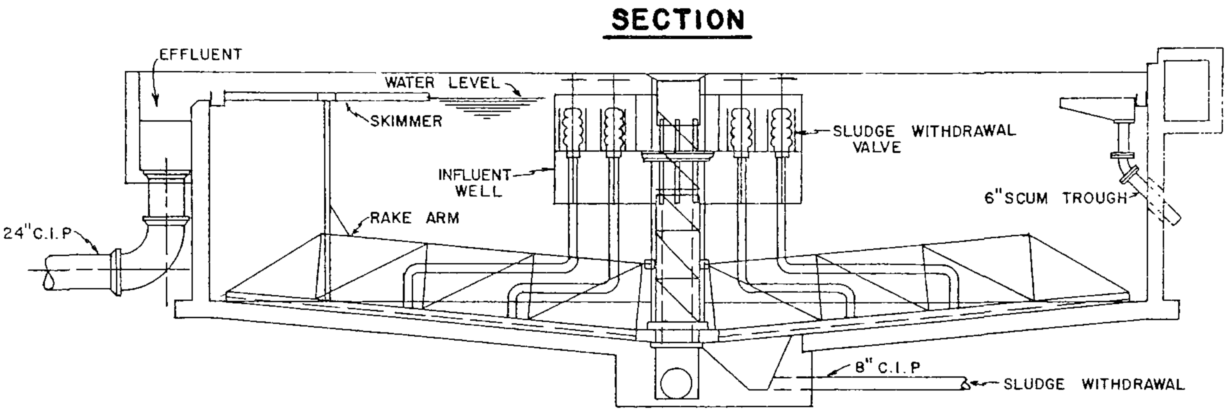


Figure 12. Secondary Clarifier

TERTIARY TREATMENT SYSTEM

At the time of the initial conceptual planning for the 1970 project, the original plan provided for a tertiary treatment system consisting solely of an alum feed system and the tertiary mixed-media filters. It was anticipated that an alum feed of approximately 40 mg/l, directly ahead of the filters, would be sufficient to polish the secondary effluent and that the utilization of the tertiary filtration system would reduce the phosphorus residual to an acceptable level.

During the development of the design drawings, however, intensive discussions with research personnel from the manufacturer who ultimately furnished the equipment, as well as other recognized experts on advanced waste treatment, led to the conclusion that a significantly higher alum feed might be required, and if that were the case, the tertiary filtration step would have to be preceded by a flash mix-flocculation-settling process. As the actual operation has shown, this was a correct assumption because the current alum feed is in excess of 120 mg/l. Operation of the system without the tertiary flocculation-settling step does not produce the quality of effluent attainable as when these units are utilized.

The design of the tertiary system at Hatfield Township includes duplicate flash mixing chambers, duplicate flocculation chambers, and duplicate tube settler units. The flash mixing chamber provides for a detention of 4.33 minutes at one-half the average design flow, or 2.17 minutes at average design flow. The capacity of each flash mix unit is 722 cubic feet (20.4 m^3). Each rectangular flocculation unit has a volume of 3,744 cubic feet (106 m^3), and the detention time in each flocculation unit is 22.5 minutes at one-half the design flow, or 11.2 minutes at the total average design flow. A sectional plan of the rapid mix-flocculator tank is shown in Figure 13.

As indicated previously, in 1969, consideration of tube settlers for tertiary clarification was a new concept, and even today, the use of tube settling clarifiers throughout the United States is not widely implemented.

The theory behind the development of tube settling modules is an extension of the long recognized principals of sedimentation, as developed early in this century with respect to the efficiency of shallow depth sedimentation. Instead of using wide, shallow trays to promote better settling characteristics, small dimension tubes have been utilized instead. These tubes offer optimum hydraulic conditions for sedimentation, with a large wetted perimeter relative to the wetted area. These tubes develop laminar flow conditions, and when inclined at a 60° angle, the tubes develop a solids build-up which reaches a certain level in the tube and then falls back down along the length of the tube, counter-current to the upward flow of the waste. In theory, these tubes

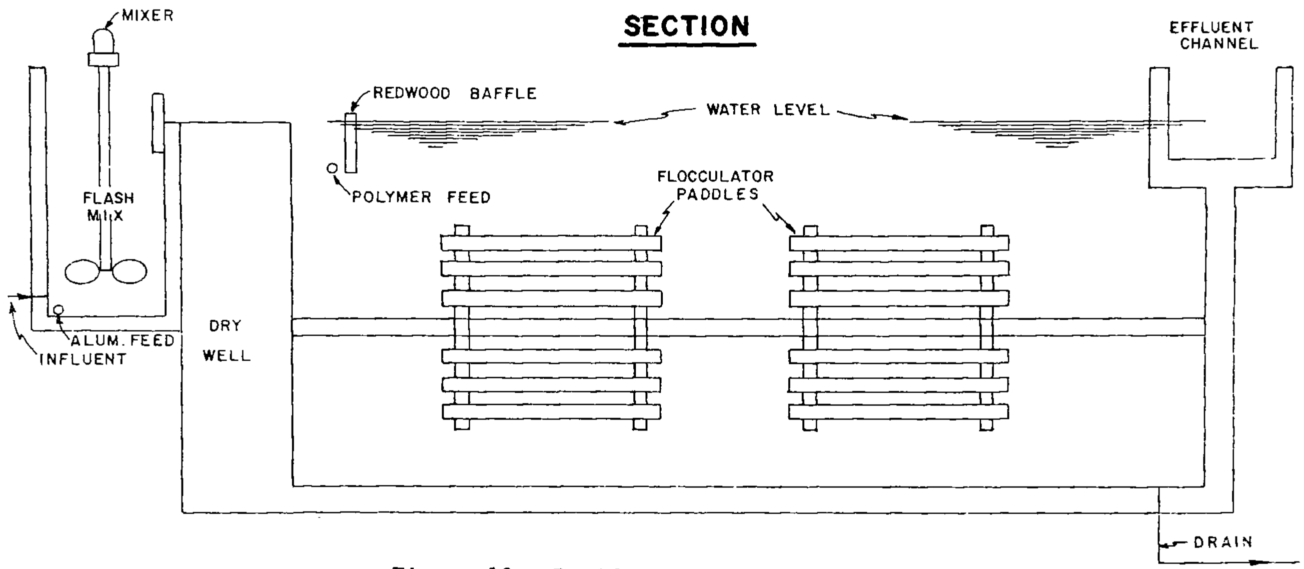


Figure 13. Rapid Mix-Flocculator Tank

are essentially self-cleaning in nature, providing a continuous sludge deposit is directed towards the bottom of the clarifier.

These tube modules are generally composed of bundles of 2" x 2" channels, lying at a 60° angle on alternate facings. These modules are normally constructed of Polyvinyl Chloride sheets, with Alkylbenzene Sulfonate channels, solvent welded together. They normally are provided in a bundle 10' in length, 30" in width, and 20" in depth (3.05 m x 0.76 m wide x 0.5 m deep). The 2" x 2" tube is equivalent to approximately 5.1 cm x 5.1 cm. A module bundle covering the standard size provides 300 ft^2 of tube area (27.9 m^2), but covers a water surface area of only 25 ft^2 (2.3 m^2).

The installation in Hatfield Township provides 880 ft^2 (81.8 m^2) of water surface area, under which tubes are located. This is equivalent to 10.560 ft^2 (981.6 m^2), which produces a surface settling rate equivalent to 170 gal/ft^2/day (0.48 m^3/m^2/day). These units have the ability to absorb flows of approximately 14.4 mgd (54,504 m^3/day) and still not exceed a surface settling rate of 1,000 gal/ft^2/day. In Figure 14, a representation of a standard inclined tube module bundle is shown.

The configuration of a tube settler, or a tube clarifier, varies from that of a normal settling unit in that it is relatively shallow in depth from the bottom of the clarifier to the bottom of the tubes, and there is a 2' depth of water over the top of the tubes to the overflow weir elevation. A sectional plan of the tube type settler is in Figure 14.

In the operation at Hatfield Township, it has been found that although the tubes are essentially self-cleansing, with continuous solids deposition to the bottom of the tank, periodically it is desirable to hose down the tubes from the top to completely clean any accumulated material which has not sluffed-off.

The chemical feed system in this tertiary portion of the treatment process involves mixing alum in an alum storage tank with water, and then discharge to liquid alum feeders for proportionong to the tertiary flash mixers. As indicated earlier, the original design envisioned an alum feed in the range of 40 mg/l, hence the utilization of an alum storage tank with batch bag mixing. In actual practice, the alum feed at times has been in excess of 140 mg/l, primarily to adjust the pH of the secondary effluent before the coagulant begins to work properly. Consideration is now being given to the installation of bulk liquid alum storage and, as an alternative, an acid feed system to control the pH.

The final portion of the tertiary treatment process is the three mixed-media filters. Effluent from the tertiary tube clarifiers is pumped through these pressure filters. Each filter is 28' long by 10' in diameter (8.53 m long x 3 m diameter). Each filter has an effective

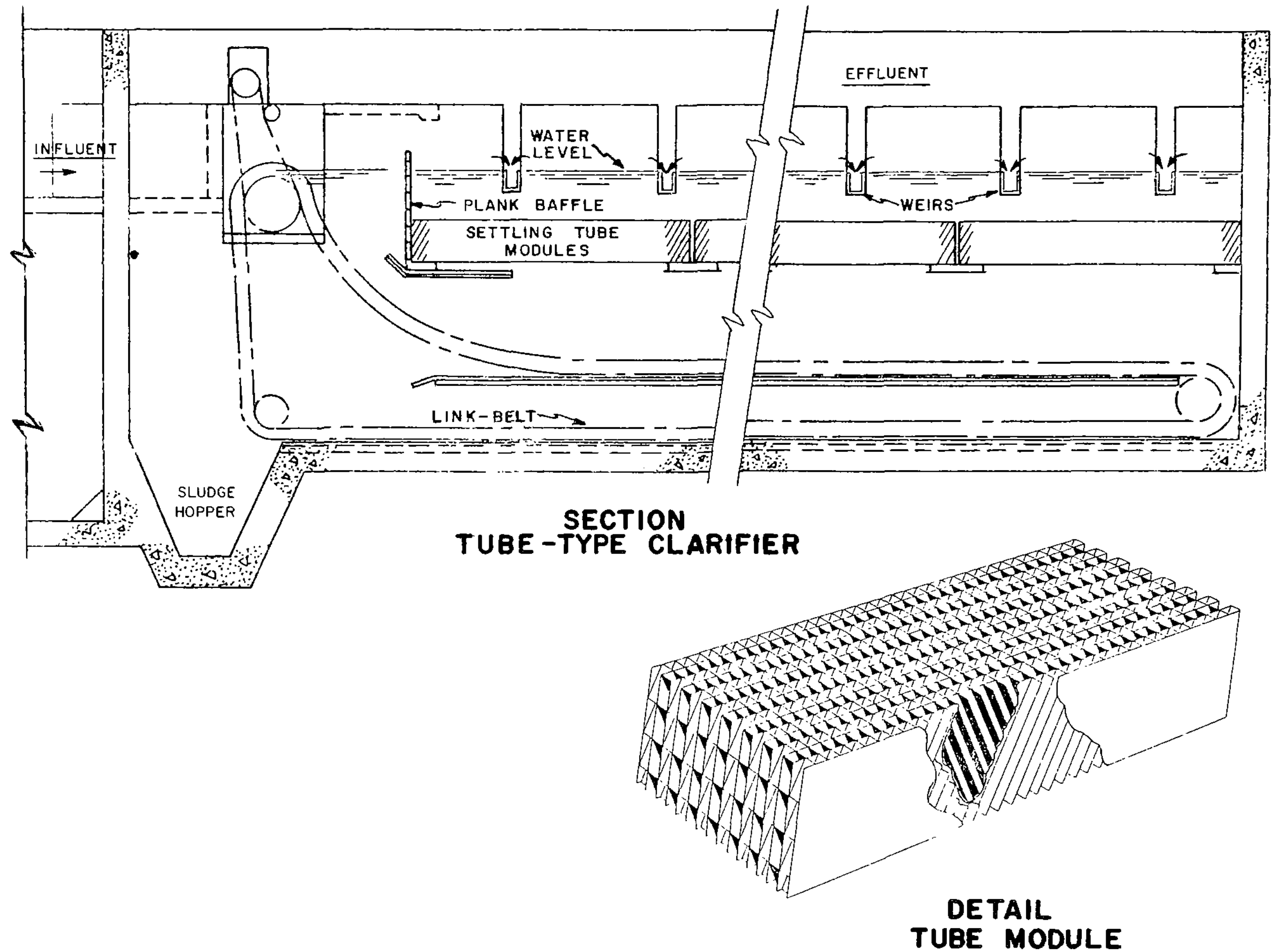


Figure 14. Inclined Tube Settler System Arrangement

surface area of 287 ft^2 (26.7 m^2), and the total available filter surface area provided is 861 ft^2 (80.1 m^2).

At the design flow rate of 3.6 mgd (13,626 m^3/day) the filtration rate is 2.9 gal/min/ft^2. Filters of this type have been successfully operated at a level of 7.5 gpm/ft^2, so that these filters have a maximum effective capacity of 9.28 mgd (35,125 m^3/day).

These pressure filters are known as mixed-media filters, because the filtering medium consists of anthracite, sand, and garnet, above a gravel support, with the filtering materials being graded inversely to a normal sand filter. The larger, more porous materials are at the top surface, and the gradation of the materials decreases with increasing depth. The structuring of these materials is maintained due to the specific gravity of each material. Each pressure filter is equipped with a surface washing system, and the total backwash and surface wash flow rate is just slightly in excess of 15 gpm/ft^2.

Three of these units have been installed, and provision has been made for the installation of a fourth unit. Although these units may be operated at levels as high as 7.5 gpm/ft^2 of filter area, the more common practice is to utilize a maximum filtering rate of 5 gpm/ft2. This means that the effective capacity of these units is approximately 6.2 mgd (23,429 m^2/day). A graphic representation of a pressure filter is contained in Figure 15.

The filter effluent proceeds to the backwash storage and chlorine contact system. In the design of the 1970 project, it developed that the existing secondary clarifiers from the 1965 project had sufficient capacity to store required volumes of backwash water, hence, these units were modified by removal of the secondary clarifier mechanisms, and were provided with chlorine solution feed lines. The flows enter these backwash storage tanks and then overflow to the chlorine contact tank, which was a part of the 1965 construction. At the time of the 1970 construction, the criteria for chlorine addition was an effluent residual of 0.3 mg/l, or greater. This has since been revised to a maximum of 0.1 mg/l, and requires more precise chlorine feed control. The backwash and chlorine contact system is diagrammed in Figure 16.

A visitor to the Hatfield Township facility can visually see the effect of the entire treatment process by observing the backwash storage tanks and the chlorine contact tank. It is possible, under normal operations, to clearly see an object, such as a coin, on the bottom, through ten feet of effluent water. This, of course, does not provide any clue as to the efficiency of the total process in removing nutrients, but it does demonstrate strikingly the ability of the process to remove solid material, and hence, insoluble BOD.

As indicated in the discussion of each portion of the treatment process

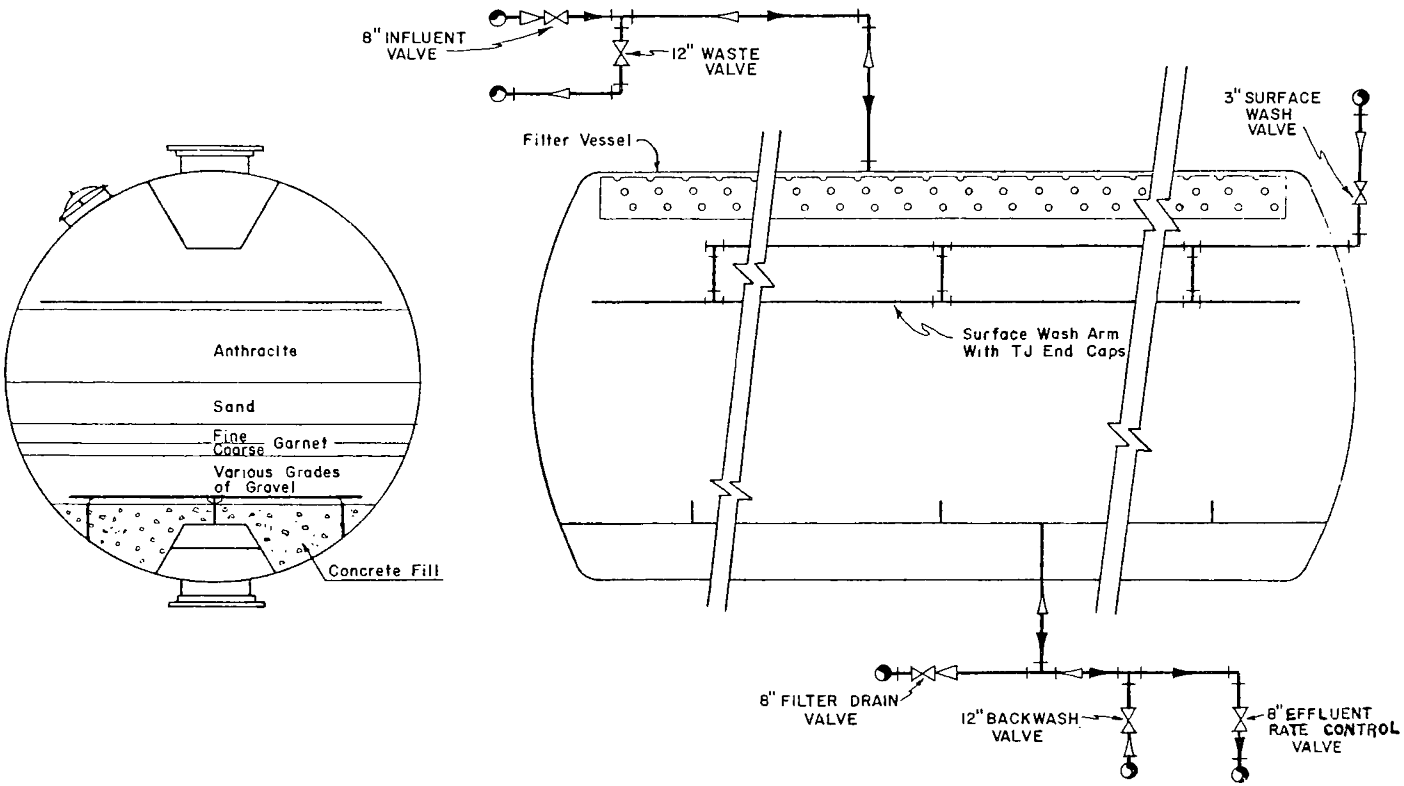


Figure 15. Pressure Filter Details

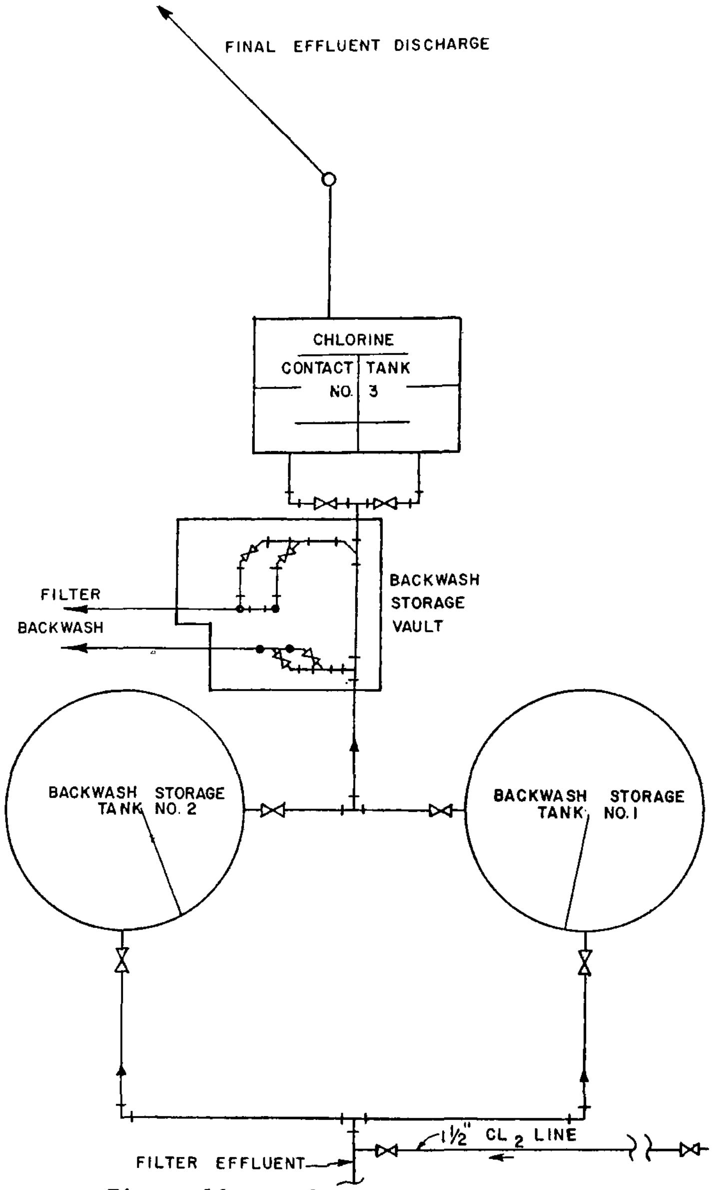


Figure 16. Backwash and Chlorine Tank

there is excess capacity available, above the design average flow of 3.6 mgd, and it is theoretically practical to load this entire treatment plant with flows in the range of 5.0 mgd (18,925 m^3/day) and still meet the same effluent characteristics as are now being produced. No matter how sound a design is, the effluent entering the receiving stream is the result of essentially the operation of the system, and the operation of the system is subject to human failures. In later chapters there will be a detailed presentation of the mode of operation and actual operating data during the last year. These operating data will show, on occasion, wide variations in effluent characteristics, some occasioned by the requirements of the Federal Demonstration Grant, but many others occasioned by the type of operation provided, and by the type of laboratory control exercised over the operation.

SECTION V

OPERATIONAL PROGRAMS

STAFFING

The design of a modern water pollution control facility must presuppose competent personnel, capable of operating and maintaining the highly sophisticated facilities and processes in an efficient, economic manner.

The 1965 plant was operated by a crew of five or six people, as seen in Figure 17, operating on a 5-1/2 day per week basis. The greatest emphasis was placed upon maintenance of building and grounds; a formal preventive maintenance program for operational equipment was not instituted until the construction of the new advanced waste treatment facility.

The operations personnel then, as now, had the responsibility of the plant and the collection system. On a daily basis, one or two people would visit the two metering pits which monitor the flow rates from an adjoining community. They also visited the five outlying pump stations, which provide flow to the plant. These trips were used for general housekeeping and minor repairs. Major repairs were coordinated through the plant manager.

Plant operations consisted of maintenance of equipment and grounds, and operation of the sludge processing and wastewater treatment equipment on a 5-day, 16-hour operation. Saturdays were used for clean-up and checking pump stations. All analytical testing was done when possible by the plant manager. No testing was done for quality control except for testing chlorine. The only tests required by the regulatory agencies were weekly BOD_5 and suspended solids.

The ultimate selection of the processes to meet the stream criteria and the 1965 design of the facility was predicated almost exclusively on reducing capital, chemical, and maintenance costs, since labor was of little significance. The average wage was $3.15 an hour for six men, plus $9,500 per year for the plant manager.

Upon completion of the new 1970 facility and the hiring of additional personnel, the plant was "organized" by a trade union. The average wage has now risen to $3.90 an hour and the total wages estimated in 1973 are $217,400, exclusive of fringe benefits. The salary schedule for present plant personnel is shown in Table 7.

The plant is presently manned on a 7-day, 24-hour basis with a minimum of two men per shift.

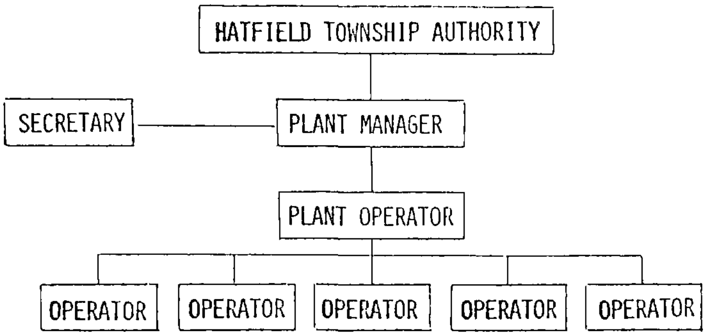


Figure 17. Organization Chart Original Plant

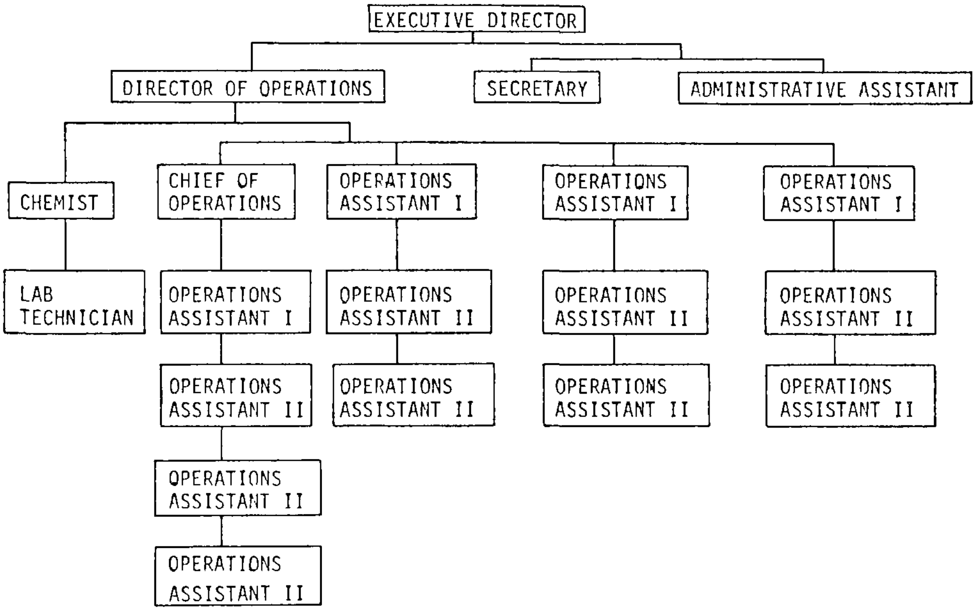


Figure 18. Organizational Chart AWT Facility

TABLE 7. SALARY SCHEDULES

	1973		1974	
Classification	Wage $/Hr	Number Personnel	Wage $/Hr	Number Personnel
Trainee	3.15-3.55	3	3.85	3
I	3.60-3.80	1	4.10	0
II	3.90	8	4.20	8
III	4.00	4	4.30	3
IV	4.10	2	4.40	3
V	4.30	2	4.60	1

Responsibilities include the collection system as outlined previously, as well as the maintenance and operation of the A.W.T. facility. This includes overseeing the operation of over 65 different pumps, 10 air compressors, various chemical feeders, and a sludge furnace. There are presently 22 full-time employees, including 2 secretaries, an executive director, a chemist, a laboratory technician and 17 multiple purpose operators. The operators are responsible for day-to-day maintenance of all operational equipment, as well as maintenance of effluent quality. Figure 18, on the previous page, shows present staffing.

Due to the nature of the facility, obtaining qualified personnel was a high priority item. The original staff provided the nucleus for the operations personnel of the A.W.T. facility. They profited by being present during the construction phase, and exposure to the various contractors' personnel. It is now a Federal Government requirement that a new facility have at least one operator in full-time attendance during the last half of the construction phase.

MANPOWER TRAINING

One of the major problems that has affected the United States environmental effort is the lack of trained wastewater treatment plant operations personnel. Many of the plants in the U. S. are operated by semi-skilled individuals with a high-school education and a minimum amount of additional formal training.

Manpower development is now subsidized in the United States by the Environmental Protection Agency, a division of the Federal Government. The Federal agency, in turn, works with the various State agencies charged with the training responsibility. In the State of Pennsylvania, this has been handled by the Public Service Institute, a division of the Pennsylvania Department of Education. The Public Service Institute has arranged for part-time instructors and classroom facilities, and conducts training sessions for prospective operators.

The operation of wastewater treatment plants in many States now requires that operators be certified by the States. The Pennsylvania Department of Environmental Resources requires that each operational plant have a Certified Operator, as well as a Certified Back-up Operator. Depending upon a plant's size, personnel are required to have a knowledge of mathematics, chemistry, biology, a mechanical aptitude and the ability to deal with the public.

Seven of the operators at the Hatfield plant are certified by the State. They were certified after taking courses offered through the Public Service Institute, and upon successful completion of an examination by the State government.

Training of operations personnel was accomplished by the following sources:

1. Manufacturers' Representatives

2. State Sponsored courses

3. Tracy Engineers, Inc. courses

4. On-the-job training

There is probably no person better qualified to instruct in the operation of a piece of equipment or a patented, proprietary process than the manufacturer or his agent. No one else has lived through the testing programs and seen the problems initially overcome. In addition, no one is in a better position to keep track of other consumers and correlate all problem areas. The vendors for the Hatfield project were all required to spend some time training operations personnel, and the shift schedule was often changed to provide the bulk of the operations personnel the opportunity to learn from thise instructors.

The State of Pennsylvania funded a training program at the Hatfield facility which was offered to the employees as well as those of nearby facilities. The program was held from 6:00 A.M. until 9:00 A.M., twice a week, for ten weeks. Operators were paid from 7:30 A.M., so the Authority subsidized this training course. The subjects included were the operation of the facility, equipment operation and maintenance, and in-plant laboratory control.

The State also sponsored several additional courses at the near-by Community College, in which personnel affiliated with the Hatfield facility participated. These included laboratory operations, a safety program, and elementary, intermediate and advanced wastewater treatment plant operator courses. Several hundred hours of instruction were received by Hatfield personnel, which led to a deeper understanding of the theory as well as the application of plant operations.

Tracy Engineers, Inc., as the consultants to the Authority, also had an obligation to train personnel in the system that the engineers designed. Their representatives met with plant personnel and the Authority many times during the end of the construction phase to outline operations. In addition, they wrote the Operations Manual for the facility, a 300 page manual of instructions specific towards meeting the needs of the plant. They provided an instructor to participate in the in-plant training course.

The writing of an operations manual for a facility such as Hatfield was a major undertaking. It had to meet newly issued Federal requirements and was very comprehensive. A sample page, Figure 19, is reproduced in this report, showing the alternate valving operations for different operating sequences.

Tracy Engineers, Inc., also offered short courses at its home office in Camp Hill, Pennsylvania, to further supplement operator training. A combination of well-trained instructor-engineers and visual aids made these courses very well received.

Management periodically holds meetings with personnel, at which time plant problems and solutions are aired, allowing personnel to share completely their common experiences.

PLANT START-UP

The many participants in the start-up of the A.W.T. facility included the following:

1. The Hatfield Township Municipal Authority - Owner

2. Tracy Engineers, Inc. - Design, Supervision of Construction

3. J. E. Brenneman Company - General Construction

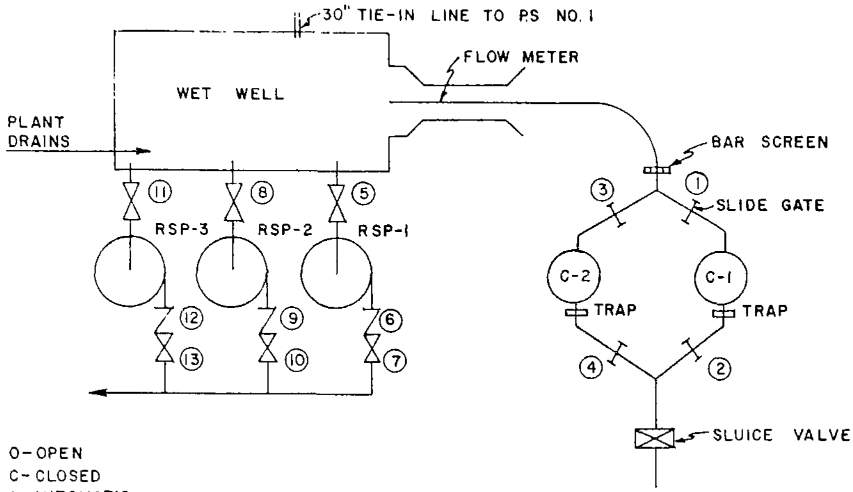
4. Coastal Construction Company - Electrical Contractor

5. Triangle Mechanical Company - Plumbing Contractor

6. Borden Company - Mechanical Sub-contractor

7. Major Manufacturers:

    a. Dorr-Oliver - Primary-Secondary System
    b. Neptune-Microfloc - Tertiary System
    c. Allis-Chalmers - Pumps
    d. BIF Corporation - Chemical Feed System
    e. Fischer & Porter - Flow Control System



O-OPEN
C-CLOSED
A-AUTOMATIC

PUMP STATION NO. I

	NORMAL	PUMP NO 1 DOWN	PUMP NO. 2 DOWN	PUMP NO. 3 DOWN	COMMINUTOR 1 DOWN	COMMINUTOR 2 DOWN	C-1 & C-2 REMOVED	STATION OFF LINE
V-1	O	O	O	O	C	O	O	O
V-2	O	O	O	O	C	O	O	O
V-3	O	O	O	O	C	O	O	O
V-4	O	O	O	O	C	O	O	O
V-5	O	C	O	O	O	O	O	C
V-6	O	C	O	O	O	O	O	C
V-7	O	C	O	O	O	O	O	C
V-8	O	O	C	O	O	O	O	C
V-9	O	O	C	O	O	O	O	C
V-10	O	O	C	O	O	O	O	C
V-11	O	O	O	C	O	O	O	C
V-12	O	O	O	C	O	O	O	C
V-13	O	O	O	C	O	O	O	C
RSP-1	A	OFF	A	A	A	A	A	OFF
RSP-2	A	A	OFF	A	A	A	A	OFF
RSP-3	A	A	A	OFF	A	A	A	OFF
C-1	ON	ON	ON	ON	OFF	ON	OFF	ON
C-2	ON	ON	ON	ON	ON	OFF	OFF	ON

Figure 19. Typical Operations Manual Page

The coordination of all these organizations and their representatives was the responsibility of the engineer. At the same time the new plant was being started up, the old plant had to be maintained in operation; no by-passing could take place, and certain units of operation had to be converted.

The Hatfield Township Municipal Authority, during the time of start-up, was undergoing problems of its own. The plant personnel were growing from six to sixteen operators, and they all required training. A new Executive Director had been employed, and he was familiarizing himself with operations of the plant and the Authority. Authority personnel, however, managed to provide the contractors and manufacturers with complete cooperation, and as a result, the startup only spanned September 1972 through January 1973. During this time, the contractor had to place the new aeration tank and secondary clarifier system in line, prior to converting the old ones. Since the aerator head box receives its flow from the primaries, these had to be serviceable also. The two wet wells had to be joined, which in itself was an interesting task, since at least one had to remain in service. In addition, the old secondary clarifiers had to be converted from secondaries to chlorine contact units.

The only by-passing possible was the primary system, or an individual duplicate unit. The methods used to meet these objectives were as follows:

1. Tying Together Wet Wells

   Early one morning, the source of flow was shut off to the plant and the flow stored in the collection system. Plant personnel pumped down Pump Station No. 1 wet well, and contractor's personnel erected a sand bagged wall within the wet well. Flow was then allowed to re-enter the wet well, except for the isolated section. The contractor proceeded to cut a hole into the isolated wet well section and install a 30" (0.762 m) by-pass line with a valve. The process was reversed to remove the sand bags.

2. Tying In New Aeration Tank

   In order to tie in the aeration tanks, the contractor ran a temporary line from the new aeration tanks to the old secondary clarifiers. This allowed him time to convert the old aeration tank from diffused aeration to a combination mechanical aeration system. The converted unit was then tied in to the new secondary clarifiers, and the plant was "in line".

   Upon completion of the conversion, both tanks were connected and the flows were transmitted to the new secondary clarifiers and then on to the tertiary system.

The existing secondary clarifiers were relatively easily converted to chlorine contact tanks and backwash tanks, although the secondary effluent had to be by-passed to the existing chlorine contact tank at the time.

No serious problems were encountered by the contractors during the start-up. Manufacturers' representatives were extremely cooperative, and the plant personnel were anxious to learn. Upon completion of the tie-ins of the major equipment and the de-bugging of the equipment, the plant operations personnel assumed responsibility for the equipment, the manufacturers' personnel provided brief training programs, but experience became the major teacher. The results of this experience, however, depended upon a well equipped, completely staffed laboratory.

LABORATORY REQUIREMENTS

Laboratory facilities, when constructed in the past, were designed to meet the minimum needs of the regulatory agencies. These requirements were often based upon the size of the plant, smaller facilities requiring less analysis, since it was assumed their effluent had less of an environmental impact than larger plants.

The State of Pennsylvania requires a minimal amount of analysis, and these were, at one time, limited to BOD_5, Suspended Solids, Settleable Solids, Dissolved Oxygen, MBAS and Residual Chlorine. The number and types of tests depended upon the size and type of plant. Now, in some cases, Ammonia-Nitrogen and Total Soluble Phosphorus are also required, but there is no stipulation as to how these analyses are to be run.

Laboratory analysis serves three general purposes:

1. It provides the operator with information on current plant operations.

2. It provides a record of data which is useful in future design.

3. It provides information to regulatory agencies.

Historically, the third purpose has received the greatest emphasis. Plant operators of smaller plants have, in the past, not usually received the training to accurately run analyses. Since these analyses were performed by people with limited training, or inferior procedures were used, the historical data was often questionable, and plants were designed using accepted national norms.

There are five basic steps involved in the correct use of a laboratory. These include:

1. Sample collection

2. Standard analysis

3. Recording and interpretation

4. Application to process

5. Report preparation

Two types of samples are generally collected at sewage treatment plants; grab samples and composite samples. Grab samples are not representative of average flows and are not normally used at the Hatfield facility. They are used when an operator suspects an industrial load, or during the daily operations for calibrating pH probes, checking SVI, and monitoring residual chlorine.

Automatic composite sampling is used throughout the Hatfield Township Municipal Authority advanced waste treatment facility. Samples are taken of the raw wastewater, as well as the effluents from the surge storage, primary clariflocculation, secondary clarification, tertiary filtered, and final effluent. The sampler gathers a 1-gallon (3.785 l) sample over a 24-hour period in a refrigerated sampler.

Composite samples are gathered daily and brought to the laboratory in the morning, where they are stored in a refrigerator prior to analysis.

The analyses performed at the Hatfield Township facility are as follows:

1. Biological Oxygen Demand, Five Day

2. Chemical Oxygen Demand, Dichromate

3. Nitrate Nitrogen

4. Nitrite Nitrogen

5. Ammonia Nitrogen

6. Total Kjeldahl Nitrogen

7. Total Phosphorus

8. Total Soluble Phosphorus

9. Total Solids

10. Total Volatile Solids

11. Total Suspended Solids

12. Volatile Solids

13. Fecal Coliform Organisms

14. Residual Chlorine

15. pH

16. Dissolved Oxygen

17. SVI (Sludge Volume Index)

18. Calcium

19. Alkalinity

20. Slaking Test

21. Jar Tests

All analyses are run in accordance with the 13th Edition of Standard Methods for the Examination of Water and Wastewater, 1971, or according to the Methods for Chemical Analysis for Water and Waste, 1971, published by the United States Environmental Protection Agency. This laboratory operation has proved itself to be a major item of expense to the Authority. The analyses require fairly sophisticated equipment not previously found in many sewage treatment plants. These tests are performed by a graduate chemist with the ability to both perform these analyses as well as interpret the results.

There is an emphasis in the United States on the BOD_5 analysis, as the method of control of waste treatment processes. The analysis is dependent upon accurate determination of dissolved oxygen, either by wet chemical analysis or by the membrane probe method. The BOD test is subject to toxic upset, and requires five days to complete. The advanced waste treatment facility controls the process with COD (Chemical Oxygen Demand) analysis. This test requires less than two hours to complete and is less subject to errors.

The biological transformations serve as an indication of plant efficiency. The ammonia nitrogen may be acted upon by bacteria for conversion to nitrates and is indicated by the following reactions:

$$\text{Protein (Organic Nitrogen)} + \text{Bacteria} \longrightarrow NH_4^+ \qquad (1)$$

$$NH_4^+ + 1.5\ O_2 \xrightarrow{\text{Nitrosomanas}} NO_2^- + 2H^+ + H_2O \qquad (2)$$

$$2NO_2^- + O_2 \xrightarrow{\text{Nitrobacter}} 2NO_3^- \qquad (3)$$

Analysis of the ammonia nitrogen provides plant operations personnel with information on the efficiency of secondary operations, as regards nitrification capability.

The most rigid criteria that the plant must meet is that of phosphorus removal. Phosphorus analysis in the low range is an extremely difficult analysis to perform. The glassware must be hot acid washed with hydrochloric acid at least twice to prevent contamination of the sample. All glassware must then be kept in closed drawers or plastic bags to prevent air contamination.

At the Hatfield facility the method used for phosphorus analysis is strong acid (nitric and sulphuric acid) digestion, followed by the stannous chloride method of analysis. The digestion process converts all of the phosphorus to the ortho form and then it is readily measurable, utilizing the stannous chloride method of analysis. The complete method appears as Appendix B of the report.

The methodology must be followed with the greatest care; contamination of the samples, or over-digestion and the loss of sample as phosphorus pentoxide will lead to gross errors.

Solids analysis are relatively simple to perform, but also depend upon completely following the prescribed procedures. These include utilizing tongs to hold Gooch crucibles and evaporating dishes to prevent moisture from the technician's hands creating a false reading.

Upon completion of the ashing for volatile analysis of the samples, precautions must be taken to prevent loss of sample due to air currents. In some cases, a "weighing agent", such as magnesium sulphate may be added to prevent sample loss.

Measurements of pH and dissolved oxygen are performed by operations personnel using portable instruments. The pH and D.O. probes are standardiced at least weekly. Operations personnel also periodically run jar tests to determine chemical dosages.

The Hatfield plant used approximately 360 tons of high calcium lime in 1973. Periodic tests are performed on size analysis of the lime, available calcium, and inerts.

Additional analyses include fecal coliform, using the membrane method. This method is an extremely simple method.

The analyses being performed at the Hatfield facility are compatible with those being run by Environmental Protection Agency installations, university research centers and other advanced waste treatment installations.

There is also an industrial waste treatment program, as well as a stream monitoring program in effect at the Hatfield facility. The ideal method for monitoring heavy metals, atomic adsorption, is beyond the present capability of the Hatfield laboratory.

The data received from the laboratory is recorded and analyzed using statistical analysis. All data is recorded and becomes a permanent record of the plant. Interpretation of all data is performed by the plant chemist, the Executive Director and the engineer, in order to improve plant operations.

The testing schedule being followed at the Hatfield plant appears as Table 8. All test work is done with the idea of improving plant processes.

Chemical dosing, frequency of instrument maintenance, pump settings, etc., are all dictated by the laboratory testing program results.

TABLE 8. MINIMUM ANALYSIS SCHEDULE - 1973

Tests	Sun.	Mon.	Tues.	Weds.	Thurs.	Fri.	Sat.	Sample Points
BOD_5		X		X		X		*
TSS		X		X		X		*
COD		X	X	X	X	X		*
P-ORTHO								*
P-TOTAL		X	X	X	X	X		*
P-SOL-O								*
P-SOL-T		X		X		X		*
$N-NH_3$			X		X			*

*See Appendix D and Table D-1

(continued...)

Table 8 (continued) MINIMUM ANALYSIS SCHEDULE - 1973

Tests	Sun.	Mon.	Tues.	Weds.	Thurs.	Fri.	Sat.	Sample Points
$N-NO_2$			X		X			*
$N-NO_3$			X		X			*
N-TK			X		X			*
TS			X		X			*
VS			X		X			*
VSS		X						*

The required physical facilities of a wastewater treatment plant laboratory have increased considerably with the requirements for better effluents. The Hatfield facility laboratory layout is shown in Figure 20. All the following items are necessities of the A.W.T. facility:

1. TKN Apparatus
2. High Capacity, All-glass Still
3. COD Apparatus
4. Steam Bath
5. Fume Hood
6. Wide Band-Pass Spectrophotometer
7. Large Area Hot Plate
8. 6-Gang Jar Stirrer
9. Dissolved Oxygen Meter
10. Fecal Coliform Test Apparatus

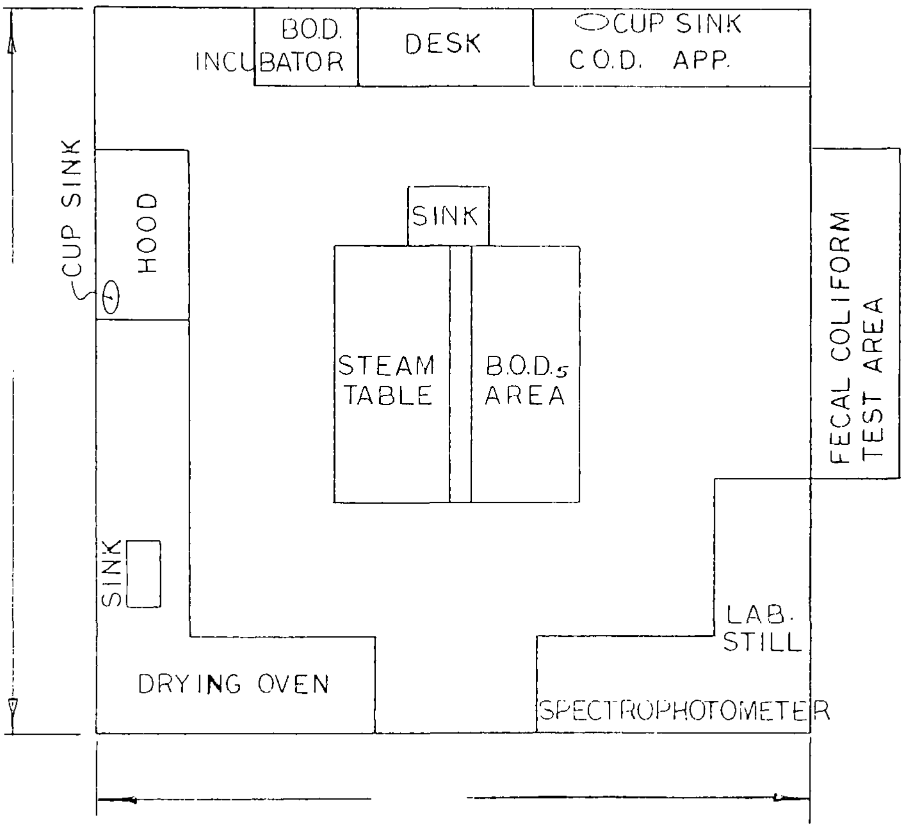


Figure 20. Laboratory Layout

11. Adequate Washing Area

12. Laboratory Refrigerator

13. BOD_5 Incubator

14. Extra Glassware

15. Analytical Balance

16. Moisture Tester

17. Muffle Furnace

Laboratory operations generally are dependent upon the availability of bench space. The provision of extra space generally results in greater efficiency, although unused space is subject to clutter.

An adequately staffed, efficiencly operated, and well supplied laboratory is the mainstay of the advanced waste treatment facility. The results produced from such a laboratory can yield a wealth of useful data.

INDUSTRIAL WASTE

One of the external factors which affects the plant's operations is that of the introduction of industrial wastes into the system. There are many small industrial facilities, as well as some large ones, which contribute to the Hatfield plant.

One of the larger facilities in the area served is that of a manufacturer of steel office furniture. The company utilizes an electrolytic paint process for coating its products. The paint is kept in a water based solution and the steel furniture acts as an electrode, allowing the paint to deposit evenly on the surface. Unfortunately, as the paint solution becomes contaminated with dust and metal fines, it starts to affect the finish. At that time the paint batch must be disposed of.

Historically, the manufacturers' personnel would add the batch of used paint to a small clarifier to which alum was added. The alum would clarify the liquid and the "paint sludge" would be drawn off for disposal on land fill. Unfortunately, the alum system periodically failed, and the wastewater treatment plant would get hit by a "slug" of grey, green, tan or black paint. The paint would coat the tanks and generate a tremendous housekeeping problem. In addition, it was toxic to the secondary system, and the plant effluent would suffer due to loss of secondary treatment for two to three weeks.

The one redeeming feature was that immediately after an accidental spill, the industrial plant's management would notify the treatment plant. Treatment personnel would prepare for clean-up and, if possible, contain the spill. On occasions, industrial personnel were sent to assist in clean-up operations.

In order to eliminate the periodic spills, the industry installed a membrane type filter on its paint spray system. The process filters the paint solution and removes all dust and metal particulate matter, allowing the paint to be reused without having to waste batches periodically.

The membrane filter system cost $150,000, yet the industrial plant personnel found that the new system could be paid off in less than nine months. This capital expenditure was also subject to a more rapid depreciation under existing Federal tax laws. There was some loss of revenue to the Authority due to the reduced flow, but this was countered by the ability of the system to handle additional flows. The system has been in effect for over a year, and since that time there have been no paint spills.

The other major industries in the Township include a meat packing plant, several plating facilities, a mushroom processor, and a dairy.

The meat packer is in the process of installing a pre-treatment system, which includes flotation skimming, sedimentation and screening in order to remove fats and other settleable material. Nearly all of the material recovered from the waste streams is reusable. The blood recovered from the kill floor is dehydrated and used as an animal feed supplement, due to its high amount of protein. The other solid material is rendered and used as a fertilizer supplement for fields in which cattle feed is grown.

Mushrooms in the United States are grown in beds of moist horse manure. The harvested mushrooms are processed by washing them and sizing them. The wash water is extremely rich in soluble BOD_5 , and pieces of mushromms. The firm has now installed a screening device with the intention of removing all screenings and other large particulate material.

The platers and the dairy are currently being surcharged for excessive waste strength.

The Authority has instituted a program of surcharging contributors of industrial wastes. This program is included in Appendix C. A surcharge is placed upon industries, based upon the amounts of BOD_5, suspended solids, total phosphorus, ammonia-nitrogen, pH, chlorine demand and flow. The analytical requirements to support this program are paid for by a permit fee program.

The objectives of the industrial waste program are to obtain an inventory of all possible industrial wastes and provide plant operators with this information in order to anticipate industrial surges and loads having a harmful effect upon the process. The program also provides each industry with both a recommendation as to its pre-treatment requirements, and an incentive to undertake, at a minimum, the requirements as a means of reducing its waste treatment charges. In some cases, the industry, by practicing sound pre-treatment, can also realize revenue from byproducts which can be sold, instead of being lost to the waste collection system.

INFILTRATION/INFLOW

Many of the sewerage systems in the United States are subject to excessive inflow and infiltration. The sources of this infiltration are generally storm water encroaching into the sanitary collection system. The sources of this storm water vary from illegal sources, such as sump connections and down spouts, broken sewer lines, improperly laid laterals, and low lying manholes. In an advanced waste treatment facility where costs are generally dependent upon the amount of flow treated, any flow reduction generally is reflected by a reduction in operating costs. Excessive infiltration is determined by the degree to which a community's collection system meets the State's standards relating to infiltration. In Pennsylvania, the standard is, "the infiltration should not exceed 500 gallons per inch of pipe diameter per mile per day for any section of the system". This amounts to 4,000 gallons/day for each mile of conventional 8" sewer line, an extremely conservative amount when one compares the actual experience of many operational collection systems with the theoretical values. The Hatfield Township plant has a normal dry weather flow of 1.8 mgd, and wet weather flow of 3.2 mgd, or 1.4 mgd in extraneous flows. The allowable rate, according to Pennsylvania Department of Environmental Resources standards, is less than 0.2 mgd, which indicates there is 1.2 mgd of excessive infiltration.

The elimination of infiltration is considered necessary to eliminate hazards caused by by-passing, reduce excessive costs of wastewater treatment, prevent damage to lines and plant, etc. The elimination of these extraneous flows has become as great a problem for municipalities as modern advanced waste treatment processes.

The Federal and State governments and consulting engineers are developing methods to eliminate these problem flows.

According to the United States Environmental Protection Agency, a phased program for sewer system evaluation should be followed. Such a program would include, but not be limited to, the following phases:

I. Infiltration/Inflow Analysis

   A. Patterned Interviews

   B. Sanitary and Storm Sewer Map Study

   C. System Flow Diagrams

   D. Dry Vs. Wet Weather Flow Determination

   E. Preliminary Field Study and Selective Flow Tests

   F. Determination of Excessive or Non-Excessive Infiltration/Inflow

   G. Establish a Plan of Action, Budget and Timetable for Execution

II. Field Investigation and Survey

   A. Physical Survey and Groundwater Analysis

   B. Rainfall Simulation

   C. Prepare Engineering Report and Analysis

   D. Preparatory Sewer Cleaning

   E. Television Inspection of Preselected Sewers

   F. Preparation of the Evaluation Survey Report and Analysis

   G. Preparation of the Proposed Rehabilitation Program

III. Rehabilitation

   A. Sewer Repair

   B. Pipe Relining

   C. Sewer Replacement

   D. Manhole Repair

The purpose of Phase I is to delineate the problem and determine a series of objectives that must be reached. The results would yield a program to reduce the problem gradually, first eliminating the major sources of extraneous flows, and then, step by step, correcting smaller problems. This would produce the best results as rapidly as possible and be the most economical. Charting these flows could show a diagram as indicated in Figure 21.

The cost of infiltration control is extremely difficult to estimate. One recent article indicated costs of approximately $1.08 per lineal foot of sewer system to seal joints, repair major manhole difficiencies, and T.V. inspect these lines. Each project, unfortunately, must be estimated on its own merits. Factors to be included are:

1. Age of Collection System

2. Materials of Construction of Collection System

3. Size of Various Pipe

4. Condition of Manholes

5. Non-sanitary Sources - eg: Downspouts, Cellar Drains, etc.

The cost of an infiltration study is quite high, and a community must be prepared to spend money to meet the State and Federal standards.

The Hatfield facility has an on-going infiltration program, which is exemplary. The program consists of several steps, including:

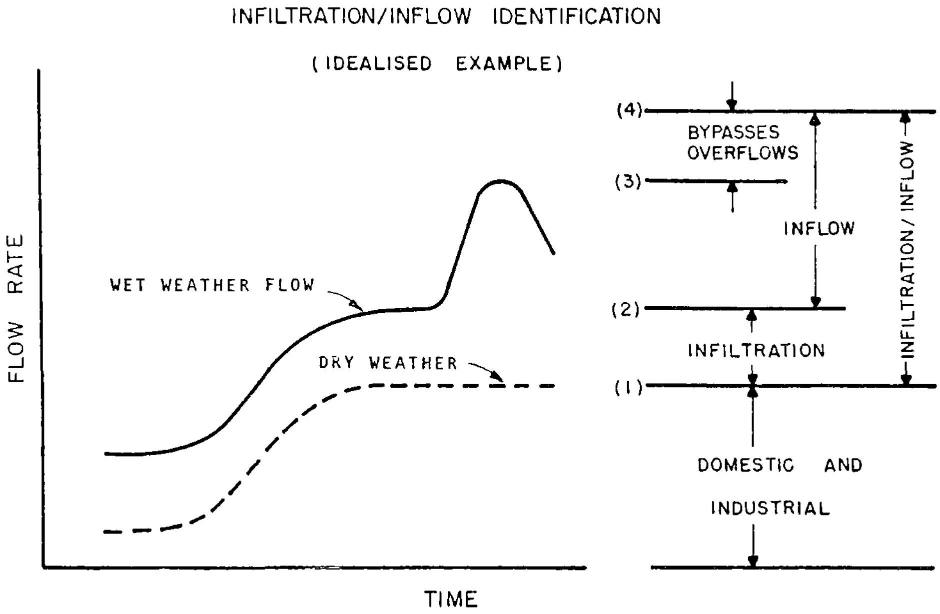
1. Raising Manholes

2. Visually Inspecting All Houses for Illegal Connections

3. Television Inspection of All Sewer Lines

4. Repairing All Suspected Leaks in Manholes and Sewer Lines

The Authority has three people on a nearly full-time program to curb infiltration. A house-to-house survey has been made of the community to determine illegal connections. The net result was the elimination of several illegal sump pump connections. In addition, during this survey, dwelling units were found which had been hooked up to the collection system and were not being charged rent. This portion of the program more than payed for itself. At the same time, an outside contractor was brought in by the Authority to raise manholes on an individual basis wherever the potential for drainage of storm water was found. The Authority has purchased a television inspection truck and a high-pressure sewer cleaning truck. These vehicles are utilized from three to five days per week to inspect and seal possible leaks in the collection system. The net result of an effective infiltration program is to reduce costs of operation of the facility.



(1) PEAK DOMESTIC AND INDUSTRIAL (NO INFILTRATION/INFLOW)

(2) PEAK (NONRAINFALL) DURING PERIODS OF HIGH GROUNDWATER

(3) PEAK FLOW

(4) TOTAL FLOW

SOURCE: EPA PROPOSED GUIDELINES

Figure 21. Infiltration/Inflow Identification

SECTION VI

OPERATIONAL RESULTS

GENERAL

Previous chapters have developed the design of the Hatfield Township AWT facility, and have discussed in detail the basis of design and unit sizing. The combination of the process units finally constructed, and the varying operating procedures carried out during the Demonstration Grant period, have provided a significant accumulation of operational data. Some of this data would have been anticipated as a product of the design theory, but other data has been generated which is at variance with predictable conclusions. Additional data is still being collected.

1972 AND 1973 OPERATION

The operation of the Hatfield Township Advanced Waste Treatment Facility varied greatly between 1972 and 1973. In 1972, the new units were under construction, and were phased into the operating sequence throughout the year as they became available for use, or as their use was required to permit modifications to existing units, such as the existing aeration tank.

By January 1973, all of the AWT units were in operation, although not all of the chemical additions were being accomplished in accordance with the design, and not all the units were being operated fully in the current mode.

1972 provided an operating situation reflecting essentially secondary treatment, but with modifications and interruptions occasioned by the construction program. 1973 reflected AWT operation, but, until March 1973, did not include total flow equalization, nor did it include, until late March 1973, a fully complete lime feed program.

A summary of average removal efficiencies for the year 1972, 1973 and the Grant period are shown in Table 9. Also values for 1974 are given.

TABLE 9. AVERAGE OVERALL REMOVAL EFFICIENCIES: PERCENT

	1972	1973	April 1973– March 1974	1974
COD	79	54.1	55.8	35.5
BOD_5	77	86.5	94.5	96.3
SS	75	93.3	95.2	96.0
Total Phosphorus	14	83.5	90.1	86.4
NH_3-N	-21	48.1	71.7	89.4

THEORETICAL COMPOSITION OF SEWAGE

Data on the composition of domestic sewage in the United States has been compiled by a number of different sources. One such compilation depicting norms of wastewater concentration is contained in Table 10.

TABLE 10. COMPOSITION OF DOMESTIC SEWAGE

(Source: Babbitt and Baumann, SEWERAGE AND SEWAGE TREATMENT, John Wiley & Sons, Copyright 1958, p. 341)

(All Values in Milligrams Per Liter)

Constituent	Strong	Medium	Weak
Solids, Total	1,000	500	200
Suspended, Total	500	300	100
Dissolved, Total	500	200	100
BOD (5-Day, 20° C)	300	200	100
Dissolved Oxygen	0	0	0
Nitrogen, Total	86	50	25
Organic Nitrogen	35	20	10
Free Ammonia	50	30	15
Nitrites (NO_2)	0.1	0.1	0
Nitrates (NO_3)	0.4	0.2	0.1
Alkalinity, $CaCO_3$	200	100	50
Total Phosphorus (P)*	10	8	6

* Concentration for Total Phosphorus has been assumed.

HATFIELD TOWNSHIP SEWAGE CHARACTERISTICS

A summary of raw sewage characteristics for the years 1972, 1973, 1974, and the Grant test period are contained in Table 11.

TABLE 11. SUMMARY OF RAW SEWAGE CHARACTERISTICS

(All Values in mg/l Unless Noted)

	1972	1973	April 1973-March 1974	1974
COD	329	338.7	389.7	315.2
BOD_5	124	74.2	86.7	111.9
Org-N	-	12.3	12.3	4.6
NH_3-N	58	42.6	52.6	53.1
NO_2-N	0.8	0.14	0.11	0.15
NO_3-N	1.4	1.23	0.39	0.02
TKN	-	4.92	67.2	60.1
pH*	7.6	7.7	8.0	7.8
Total Phosphorus	8.4	6.6	6.8	6.2
Total Soluble Phosphorus	6.8	6.0	5.1	4.8
SS	173	182.0	199.6	134.3
% VSS	-	70.9	71.5	71.7
TS	1,037	846.8	872.8	619.2
% TVS	-	31.2	30.0	30.1
Alk.	-	167.5	167.5	236.2

* Median Values

A comparison of these values with the average characteristic for "normal domestic sewage", as presented in Table 10, would indicate that total solids and ammonia-nitrogen are of strong to medium strength, while BOD_5 and suspended solids are relatively weak. The general conclusion derived from these raw sewage characteristics is that Hatfield has a relatively weak domestic waste, with an industrial contribution of ammonia and dissolved solids.

The data summarized in Table 11, for the 1972 raw sewage characteristics is derived from Table 12. Table 12 lists the average monthly characteristics of the raw waste for 1972, and the yearly agerage.

In Table 13, the average monthly characteristics for the final effluent are listed for 1972. Inspection of Table 13 will indicate that the efficiency of the 1972 operation started to increase significantly in June and July, as new units were added to the process operation.

In Table 14, the average monthly raw waste characteristics for 1973 are shown. In Table 15, the average monthly final effluent characteristics for 1973 are shown. A review of Table 15 will indicate a substantial improvement in the effluent quality commencing in April, after full flow equalization and lime addition had been instituted. Table 16 and Table 17 indicate the raw and final effluent characteristics for early 1974.

Analytical data for 1973 and 1974 are contained in Appendix D.

TABLE 12. RAW WASTEWATER CHARACTERISTICS

(All Values in mg/l Unless Noted)

Hatfield Township Municipal Authority

1972 Monthly Averages

	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Year Average
BOD_5	94	97	65	95	128	166	194	121	157	160	84	126	124
COD	163	132	92	130	190	208	383	1008	697	608	109	226	329
TS	-	610	678	582	-	-	1974	-	1339	-	-	-	1037
SS	142	145	110	133	160	418	193	212	132	208	166	55	173
D.O.	-	-	-	5.6	-	4.3	6.0	2.3	-	-	4.9	6.9	5.0
NH_3-N	29	35	39	45	63.1	99.5	71	120.5	91.4	77	9.8	14.2	58
NO_2-N	-	0.2	0.2	0.2	-	-	-	2.9	1.7	-	-	-	0.7
NO_3-N	0.3	0.7	0.7	0.8	0.4	1.0	1.0	0.6	7.8	0.6	1.1	1.6	1.4
P_O (1)	4.4	2.0	3.4	5.0	-	7.7	9.4	3.4	14.3	6.8	2.1	2.6	5.6
P_T (2)	9.2	2.6	3.8	5.6	-	11.1	9.8	18.5	16.3	9.3	2.9	3.6	8.4
P_S-O (3)	3.4	0.7	3.3	5.0	-	3.7	9.8	-	6.6	5.0	-	-	4.7
P_S-T (4)	7.0	2.5	4.1	5.5	-	5.4	9.8	-	13.1	-	-	-	6.8
pH Units (5)	7.8	7.9	-	8.1	-	6.9	6.8	7.6	7.8	-	7.7	7.5	-
Temp C°	13°	7.9°	-	12.8°	-	15.8°	19°	20.3°	-	-	9.8°	6.1°	13.1°

(1) P_O = Ortho Phosphorus as P
(2) P_T = Total Phosphorus as P
(3) P_S-O = Soluble Ortho Phosphorus as P
(4) P_S-T = Soluble Total Phosphorus as P
(5) Median Values of pH Units

TABLE 13. FINAL EFFLUENT VALUES

(All Values in mg/l Unless Noted)

Hatfield Township Municipal Authority

1972 Monthly Averages

	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Year Avg.	% Re-moval
BOD_5	63	55	46	52	35	10	17	5	13	8	6	12	27	77
COD	111	95	75	125	40	35	58	235	254	101	37	54	102	79
TS	-	592	507	554	-	-	404	-	795	-	-	-	570	45
SS	-	-	61	85	69	23	16	-	29	37	24	-	43	75
VS	-	-	-	-	-	-	-	-	-	-	-	-	-	-
VSS	-	-	-	-	-	-	-	-	-	-	-	-	-	-
D.O.	-	-	8.1	5.3	-	6.9	6.1	2.1	-	-	7.1	7.2	6.1	N/A
NH_3-N	23	28.2	20.5	31	73	73.5	69	77	60	54.2	21.6	20	46	21
NO_2-N	-	0.2	0.2	0.2	-	0.08	-	1.0	0.3	-	-	-	0.3	-
NO_3-N	0.4	0.8	0.5	0.9	0.2	0.6	0.6	0.6	0.8	0.4	0.9	1.0	0.6	-
P_O	3.2	2.2	3.6	3.5	-	8.4	6.1	4.8	13.4	10.5	2.1	1.1	5.4	4
P_T	7.0	2.6	3.9	4.1	-	10.1	6.4	20.5	-	12.1	2.9	1.2	7.2	14
P_S-O	3.1	0.8	3.6	2.3	-	4.4	5.9	-	12.6	6.3	-	-	4.9	0
P_S-T	5.2	2.1	4.2	3.9	-	5.1	6.1	-	9.0	-	-	-	5.1	2.5
pH Units*	7.9	-	7.6	-	7.2	7.0	7.2	7.3	-	7.7	8.6	-	-	-
Temp C°	12.5	7.9	-	11.5	-	16.2	22.5	21.5	-	-	8.6	-	14.4	-
Turb. Unt.	-	-	-	-	40	34	13	-	-	-	-	-	33	-
Color Unt.	-	-	-	-	99	118	48	-	-	-	-	-	121.3	-

* Median Values of pH Units

TABLE 14. RAW SEWAGE CHARACTERISTICS

1973 Monthly Averages

(All Values in mg/1 Unless Noted)

Test	Jan.	Feb.	Mar	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Avg. Jan.-Mar.	Avg. Apr.-Dec.
COD	231	257	196	195	284.7	366.1	601.2	720.8	324	407.8	223.8	256.7	228	375.6
BOD_5	248	74	84	76.8	101.2	104.7	111.0	65.0	94.6	51.3	72.2	109	135	87.3
Org-N	-	-	-	36.2	5.8	2.8	10.3	3.5	-	-	-	15.0	-	12.3
NH_3-N	47.4	30.6	17.7	48.2	39.5	38.5	60.7	-	-	50.7	54.9	31	31.9	46.2
NO_2-N	-	-	0.2	0.06	0.09	0.46	0.1	0.09	0.06	0.17	0.08	-	0.2	0.12
NO_3-N	1.3	7.0	1.9	0.08	0.15	0.17	0.07	0.04	0.29	0.15	3.0	0.6	3.4	0.51
TKN	-	-	-	84.4	41.6	41.3	71	11	-	-	-	46	-	49.2
pH*	-	-	-	7.1	7.1	7.35	7.1	7.7	8.1	8.1	8.4	8.0	-	7.7
P-Total	5.1	3.4	4.9	5.1	7.9	8.1	7.9	6.07	12.9	6.9	3.9	7.2	4.5	7.3
PST	-	-	-	3.24	7.33	5.9	6.4	6.0	6.0	5.3	3.2	7.5	-	5.99
S.S.	101	105	91.8	85.0	106.5	116.0	232	116.8	372.5	246.3	342	269	993	209.6
%VSS	-	-	-	71	46	83.7	87.1	82.4	82.2	67.4	54.5	64	-	70.9
T.S.	502	660	490	379	542	695	1252.0	938	542.4	1139	1630	1392	550.7	945.5
%TVS	-	-	-	35.9	26.3	37.3	43.3	38.8	17.8	16.6	48	17	-	31.2
Alk.	-	-	-	99.6	122.3	157.8	-	240.6	196.4	166.8	162.8	193.8	-	167.5

* Median Values

NOTE: January-March operating period involved phasing in new process units.
April-December operating period of new AWTF.

TABLE 15. FINAL EFFLUENT VALUES

1973 Monthly Averages

(All Values in mg/1, Unless Noted)

Test	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Avg. Jan.-Mar.	Avg. Apr.-Dec.
COD	114	113	120	78.3	51.7	223.4	280.7	338.8	197.8	109.6	97.6	143.0	115	169
BOD_5	73	21	20	2.9	3.0	4.3	6.5	2.6	5.8	2.0	3.1	16	38.3	5.1
Org-N	-	-	-	27.2	20.0	1.4	1.6	2.1	-	-	-	0.7	-	8.8
NH_3-N	31.9	47.3	31.3	52.6	41.2	18.1	30.7	0.5	0.4	5.6	4.0	1.7	36.8	17.2
NO_2-N	-	-	0.2	0.05	2.0	2.24	1.9	0.5	1.3	1.1	0.05	-	0.2	1.14
NH_3-N	1.4	-	2.3	0.06	0.32	1.51	0.95	1.0	0.6	1.1	12.8	0.1	1.9	2.05
TKN	-	-	-	79.8	42.4	19.5	37.3	2.4	-	-	-	2.4	-	28.1
pH*	-	-	-	7.9	-	7.08	5.9	6.8	6.7	-	7.3	7.7	-	7.05
P-Total	3.3	2.8	0.9	0.7	0.6	0.45	0.6	0.15	-	0.2	0.3	0.1	2.3	0.68
PST	-	-	-	0.3	0.4	0.45	-	0.5	-	0.3	0.32	0.1	-	0.39
S.S.	29	20.5	10.6	7.0	10.2	11.7	13.9	8.1	4.7	16.4	4.9	11.0	20	9.76*
%VSS	-	-	-	62.2	69.2	74.0	57.4	61.5	78.4	43.5	46.6	55.0	-	51.2 *
T.S.	400	523	530	535	525	715	850	806	856	891	800	735	486	746 *
%TVS	-	-	-	16.5	11.8	28	23.5	36.2	11.7	22.0	24.0	12.0	-	20.6 *
Alk.	-	-	-	115	103.9	124	130	101.7	132.6	165.6	181.9	132.1	-	120.6

* Median Value

NOTE: January-March operating period involved phasing in new process units.
April-December operating period of new AWTF.

TABLE 16. RAW SEWAGE CHARACTERISTICS

1974 Monthly Averages

(All Values in mg/l, Unless Noted)

Test	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Year
COD	-	-	432	136	136.3	388.8	384	-	578.6	-	-	150.7	315.2
BOD_5	81.1	99.7	73.7	110.9	102.7	138.1	144.7	138.7	145.4	140.2	99.8	67.5	111.9
Org-N	-	-	-	3.9	6.4	7.3	8.0	2.6	2.7	2.6	3.7	4.4	4.6
NH_3-N	44.1	82	89.3	111.9	31.3	96.3	80	24.8	22.14	25.4	18.9	11.0	53.1
NO_2-N	-	-	0.06	0.11	0.23	0.12	0.15	0.30	-	-	0.15	0.14	0.15
NO_3-N	-	-	0.01	0.02	0.04	0.02	0.02	0.02	0.03	-	0.022	0.02	0.022
TKN	-	-	121.3	115.8	58	102.5	88	25	24.2	28.0	21.9	15.4	60.1
pH*	-	8.1	8.0	7.8	7.5	7.4	7.5	7.7	7.9	7.8	-	-	7.8
P-Total	2.9	4.2	8.6	4.8	5.4	4.8	6.8	9.6	7.3	5.8	7.0	7.1	6.2
PST	2.3	0.47	4.2	4.0	4.8	4.8	6.7	(9.8)	6.5	-	-	-	4.8
S.S.	58.6	235.2	215.3	137.4	148.9	128.8	168.6	-	-	87	107.1	56.5	134.3
%VSS	76.1	70.1	73.2	59.9	59.7	66.2	87.2	75.1	-	-	-	77.9	71.7
TS	-	676.2	632.9	431	730	958	864	149.5	-	-	954	177.5	619.2
%TVS	-	28.2	24.2	15.5	19.9	23.9	10.1	12.5	37.3	-	50.9	78.2	30.1
Alk.	-	-	-	129.6	249.3	291.3	205	355	-	-	-	186.7	236.2

* Median Values

TABLE 17. FINAL EFFLUENT VALUES

1974 Monthly Averages

(All Values in mg/l, Unless Noted)

Test	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Avg. 1974
COD	-	-	181.3	166.5	220.4	435	368	151.5	-	-	16	345.6	235.5
BOD_5	3.0	3.8	5.1	2.4	4.6	2.3	2.1	5.6	6.5	3.8	4.6	1.3	3.8
Org-N	-	-	40.7	4.0	3.2	3.4	28	1.4	1.3	-	0.6	40.7	9.3
NH_3-N	1.2	10.1	12.7	17.0	7.7	2.1	6.0	2.6	0.5	0.83	1.09	4.0	5.5
NO_2-N	-	-	0.1	0.14	0.43	0.63	0.39	0.70	0.26	-	1.93	0.50	0.51
NO_3-N	-	-	0.2	0.04	0.19	0.07	0.06	0.11	0.11	-	0.28	0.121	0.11
TKN	-	-	14.6	21.1	10.9	5.7	8.6	2.5	1.5	1.5	1.6	5.3	7.3
pH*	7.1	7.7	7.5	7.4	7.3	6.7	6.3	6.1	7.3	7.0	-	-	7.1
P-Total	0.5	0.7	0.71	0.70	0.9	0.8	1.5	1.2	1.14	0.69	0.85	0.65	0.86
PST	0.01	0.13	0.7	0.6	0.47	0.70	1.3	1.05	0.91	-	-	0.60	0.65
S.S.	8.4	11.1	7.0	6.3	3.2	1.8	2.6	1.9	2.8	2.87	4.2	3.5	4.64
%VSS	68.1	54.9	40.3	42.4	54	60.2	55.9	55.7	52.4	-	-	63.8	55.8
TS	-	-	702.7	559	710	771	808	788	344	-	802	331.5	646.2
%TVS	-	11.0	15	17.4	15.9	23.9	16.0	50.8	36.6	-	58.4	81.3	32.6
Alk.	-	-	180	116.1	123.9	-	50	-	218.5	-	-	164	142.1

* Median Value

WASTEWATER VOLUMES PROCESSED

The average plant influent flow rates for 1972, 1973 and 1974 were 1.352, 2.036 and 1.741 MGD (5.117, 7.706 and 5.589 x 10^3 cu m/day) respectively, as summarized in Table 18, below, and as shown in detail in Tables 19, 20 and 21.

TABLE 18. SUMMARY OF PLANT INFLUENT FLOWS

All Values in Million Gallons*

	1972	1973	1974	April '73-March '74**
Total Flow	510.780	744.490	637.650	736.790
Average Daily Flow	1.352	2.036	1.741	2.004
Daily Maximum	5.880	8.000	6.280	8.000
Daily Minimum	0.390	0.640	0.500	0.500

* Million Gallons x 3785 = cubic meters
** Extracted From Tables 20 and 21

The plant flows, however, do not take into consideration the volume of liquid recycled through a facility from sources such as thickener overflow, vacuum filter filtrate, incinerator scrubber water and backwash water from the tertiary filters. Recycle data for a typical month, March 1973, is indicated in Table 22. The amount of liquid recycled, as well as the strength of that liquid, can result in a serious overloading of a treatment facility.

Total flow for the month of March 1973 was 69.04 MG (261.32 x 10^6 cu m), or 2.23 MGD (8.44 x 10^3 cu m/day). The connected population to the facility in that month was approximately 5,734 equivalent dwelling units, proportional to a population of 20,069, (EDU x 3.5 capita/EDU). With an average domestic water consumption in the area of 100 Gal/Capita/Day (0.38 cu m/Day/Dapita), this accounts for 2.01 MGD (7.61 x 10^3 cu m/Day) with the unaccounted flows due to infiltration and/or inflow in the sanitary sewerage collection system, as well as illegally connected sump pumps.

The extra hydraulic load created by infiltration/inflow has been recognized by regulatory agencies and Federal guidelines call for applicants for public funds to either eliminate these extraneous flows or design for them. Likewise, designers must provide for in-plant recycle loads which can equal a large percentage of raw sewage loads.

TABLE 19. SUMMARY OF INFLUENT PLANT FLOWS

Hatfield Township Municipal Authority

1972

Month - MG		Daily - MGD		
	Total	Average	Maximum	Minimum
January	32,02	1.30	1.78	1.67
February	40.87	1.41	1.82	0.91
March	41.59	1.34	1.64	0.81
April	51.33	1.66	5.88	0.59
May	70.54	2.52	4.40	0.39
June	50.04	1.67	3.67	0.78
July	35.77	1.15	2.20	0.63
August	23.83	0.77	1.00	0.62
September	22.17	0.74	0.90	0.60
October	27.74	0.90	1.86	0.75
November	54.64	0.82	3.50	0.88
December	60.24	1.94	3.66	1.25

1972 Total Flow: 510,780,000 Gallons (1,933,302 cu m)

Average Daily: 1,352,000 GPD (5,117 cu m/Day)

Maximum Daily: 5,880,000 GPD (15,553 cu m/Day)

Minimum Daily: 390,000 GPD (1,476 cu m/Day)

TABLE 20. SUMMARY OF INFLUENT PLANT FLOWS

Hatfield Township Municipal Authority

1973

Month - MG		Daily - MGD		
	Total	Average	Maximum	Minimum
January	52.89	1.71	-	-
February	61.32	2.19	-	-
March	69.04	2.23	4.05	1.07
April	99.00	3.30	6.08	1.48
May	65.71	2.12	6.04	1.35
June	65.71	2.19	6.04	1.30
July	57.09	1.84	4.20	1.00
August	50.01	1.61	5.75	0.95
September	38.84	1.29	4.01	0.94
October	48.48	1.56	4.84	0.99
November	34.21	1.10	3.09	0.64
December	102.19	3.30	8.00	1.01

1973 Total Flow: 744,490,000 Gallons (2,817,894 cu m)

Average Daily: 2,036,000 GPD (7,706 cu m/Day)

Maximum Daily; 8,000,000 GPD (30,286 cu m/Day)

Minimum Daily; 640,000 GPD (2,422 cu m/Day)

TABLE 21. SUMMARY OF INFLUENT PLANT FLOWS

Hatfield Township Municipal Authority

1974

Month - MG	Total	Daily - MGD Average	Maximum	Minimum
January	87.24	2.81	6.28	1.66
February	22.26	0.80	1.47	0.50
March	66.05	2.13	5.89	1.14
April	70.40	2.35	4.95	1.20
May	42.19	1.36	3.35	0.82
June	61.78	2.06	4.71	1.70
July	39.73	1.28	2.27	0.88
August	37.09	1.20	2.18	0.92
September	52.39	1.75	3.64	0.74
October	51.04	1.65	4.95	1.17
November	35.93	1.20	2.13	0.95
December	71.55	2.31	5.94	1.26

1974 Total Flow: 637,650,000 Gallons (2,413,505 cu m)

Average Daily: 1,741,000 GPD (6,589 cu m/Day)

Maximum Daily; 6,280,000 GPD (23,769 cu m/Day)

Minimum Daily; 500,000 GPD (1,892 cu m/Day)

TABLE 22. RECYCLE FLOWS - MARCH 1973

DAILY AVERAGE

Source	Volume - MGD
Pressure Filter Backwash Water	0.11 (1)
Thickener Overflow	0.57
Scrubber Water	0.16 (2)
Dewatering	0.01
Vacuum Pumps	0.08
Drain Lines	0.01
TOTAL	0.94 MGD

(1) Based on backwashing each filter once per day

(2) Based on 168 hours/week incinerator operation

The 0.94 MGD (3.56 x 10^6 cu m/Day) average daily recycle flor for March 1973 was equal to 42% of the daily raw flow to the facility. This increases the hydraulic loading on the plant and can reduce efficiency. The aeration tanks, for example, with a volume of 562,500 gallons would have a detention time of 6.05 hours at the average daily raw flow of 2.23 MGD. The addition of the 0.94 MGD recycle flow reduces this to 4.28 hours detention time, which could reduce BOD removal efficiency. In addition, the two primary units have a total area of 5,652 square feet, allowing for an overflow rate of 469 gallons/sq. ft./day. With the recycle flow added, the overflow rate reaches 662 gallons/sq. ft./day, potentially generating a solids carry-over problem.

OPERATING DATA REVIEW

The Federal Demonstration Grant served as both an advantage and a disadvantage to the Authority personnel in plant operations. The advantages included provision of funds and a resident engineer; the disadvantages included non-routine operation for collection of specific evaluation data. There was no lime feed during June 1973 and during a brief period in September 1973, the tube settlers were by-passed. In addition, during the 1972-1973 period, various units were removed from operation to check for wear, and to assist in training operations personnel. All factors weighed heavily on plant results.

Raw sewage flows for 1973 are shown in Figure 22. These flows do not account for the recycle flows mentioned previously. The equalization basins reduce the effects of the 8:00 A.M. and 8:00 P.M. daily peak flow, and the 3:00 A.M. daily minimum flow as well as the weekly flow variation shown in Figure 23 which relate more to the actual February 1973 data.

Plant operations personnel adjust the flow control system as necessary to maintain a steady flow through the plant. This is accomplished by adjusting a set point, which regulates the opening of the "Red Valves", the pinch valves which limit the flow from the equalization tanks. As the daily flows increase during the week, the settings are changed periodically to insure a gradual increase. Simultaneously, chemical feed rates are changed to pace chemical feed to flow.

The 5-Day Biological Oxygen Demand in the raw and final effluents are represented as monthly averages in Figure 24. As flow lessened in the fall months, the concentration of BOD_5 in the raw waste decreased slightly, but there was little overall effect on the final BOD_5.

Chemical Oxygen Demand values are represented in Figure 25. A comparison of these values, particularly final effluent values, will show no correlation with the BOD_5 values of Figure 24. This virtually complete lack of any type of correlation was not pursued during the Grant Test Period, and cannot be explained.

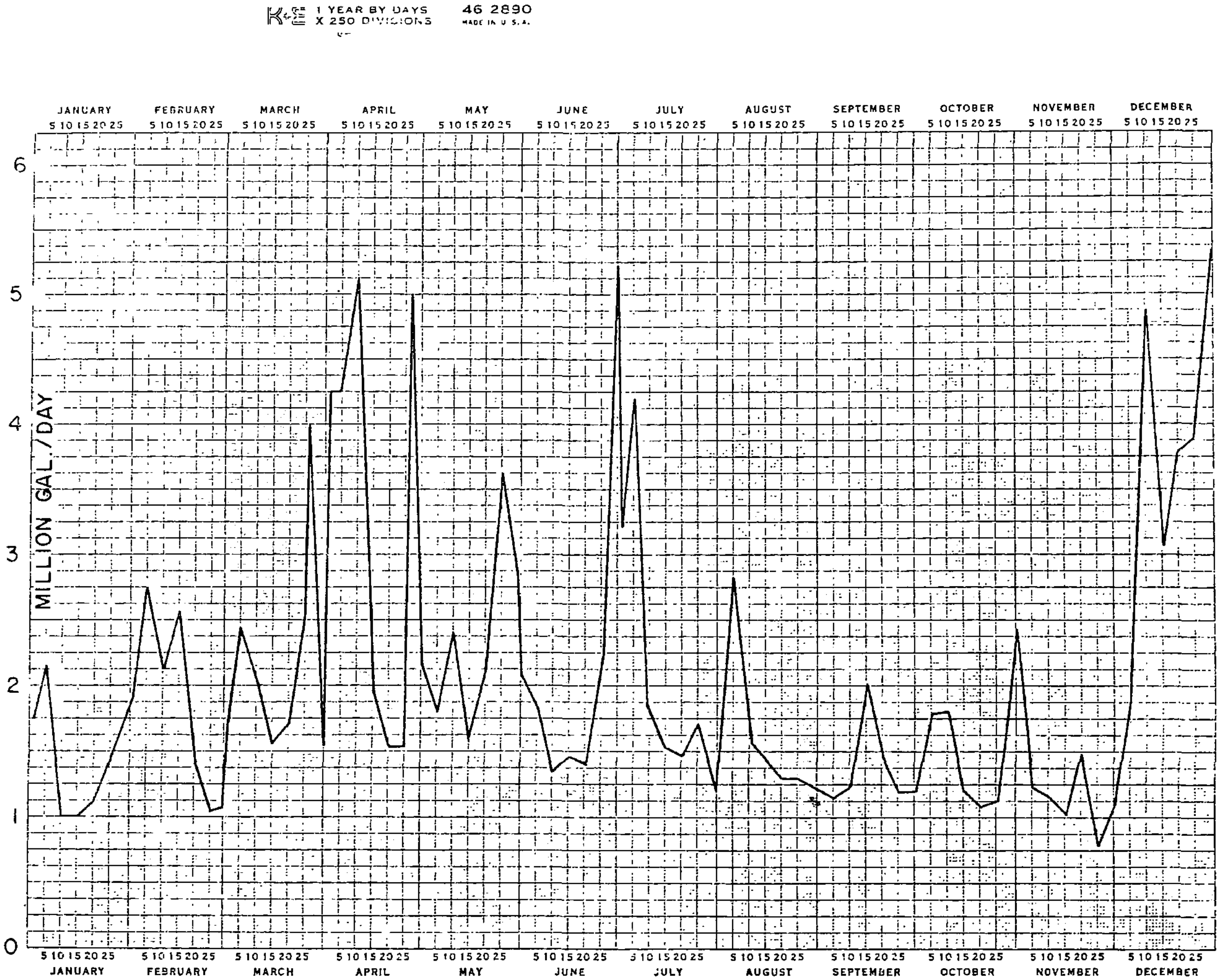


Figure 22. Raw Sewage Flows - 1973

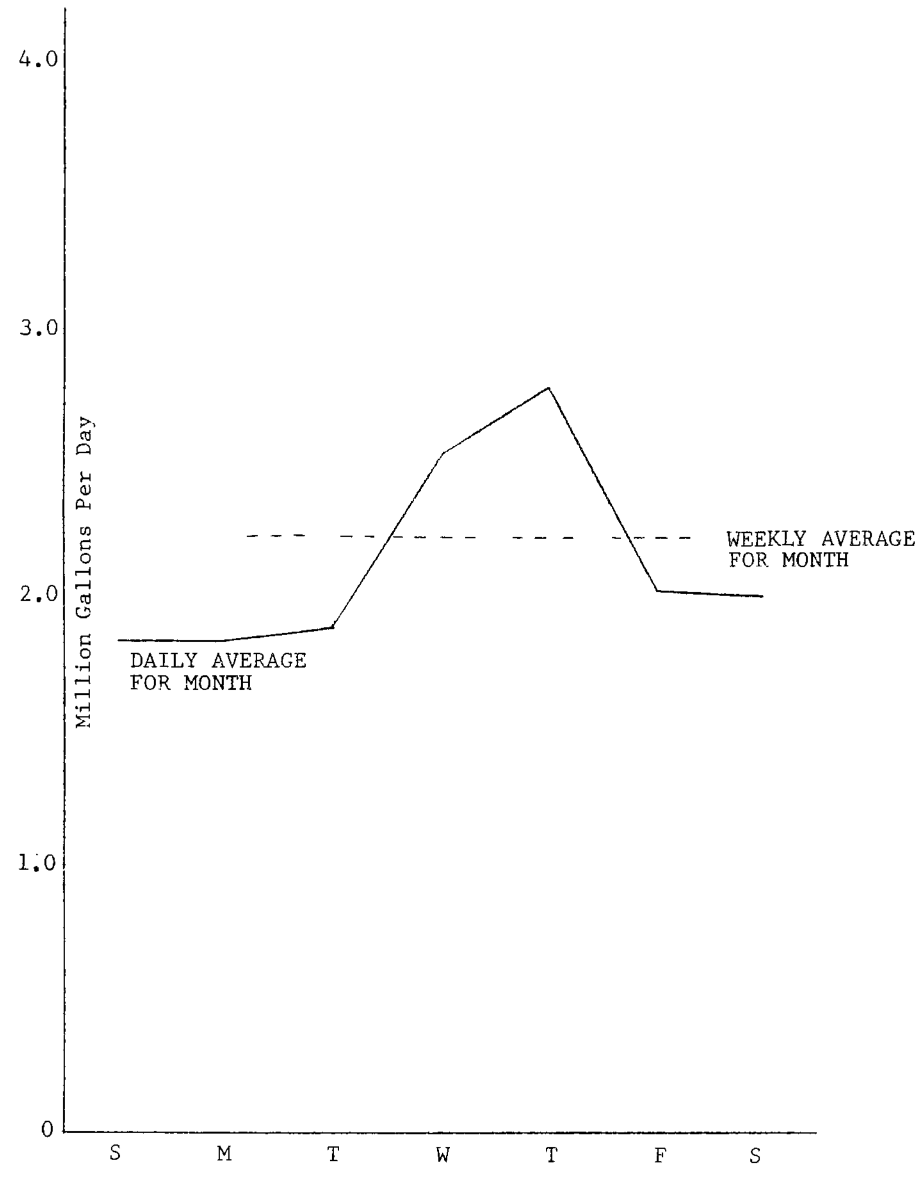


Figure 23. Flow variations by day of the week, February 1973

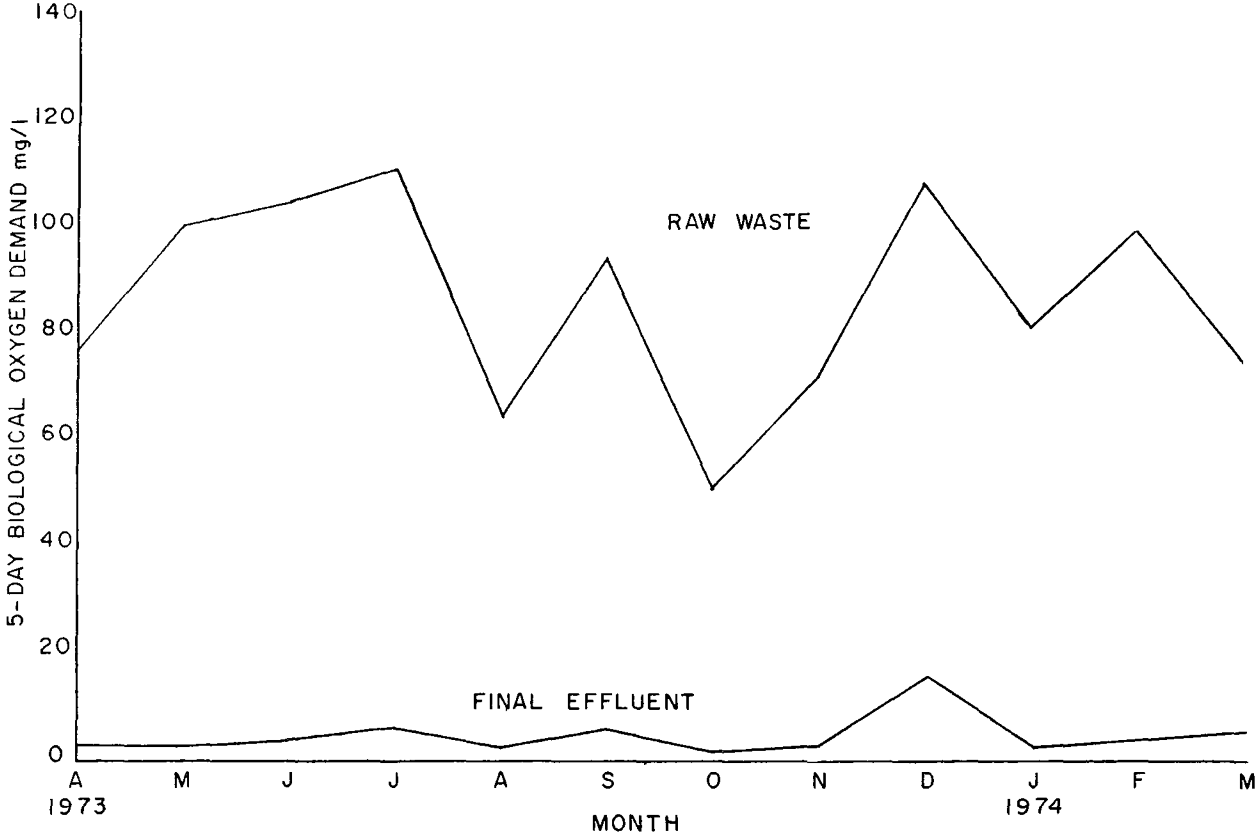


Figure 24. Raw and Final BOD5 Values - April 1973-March 1974 - Average Monthly Values

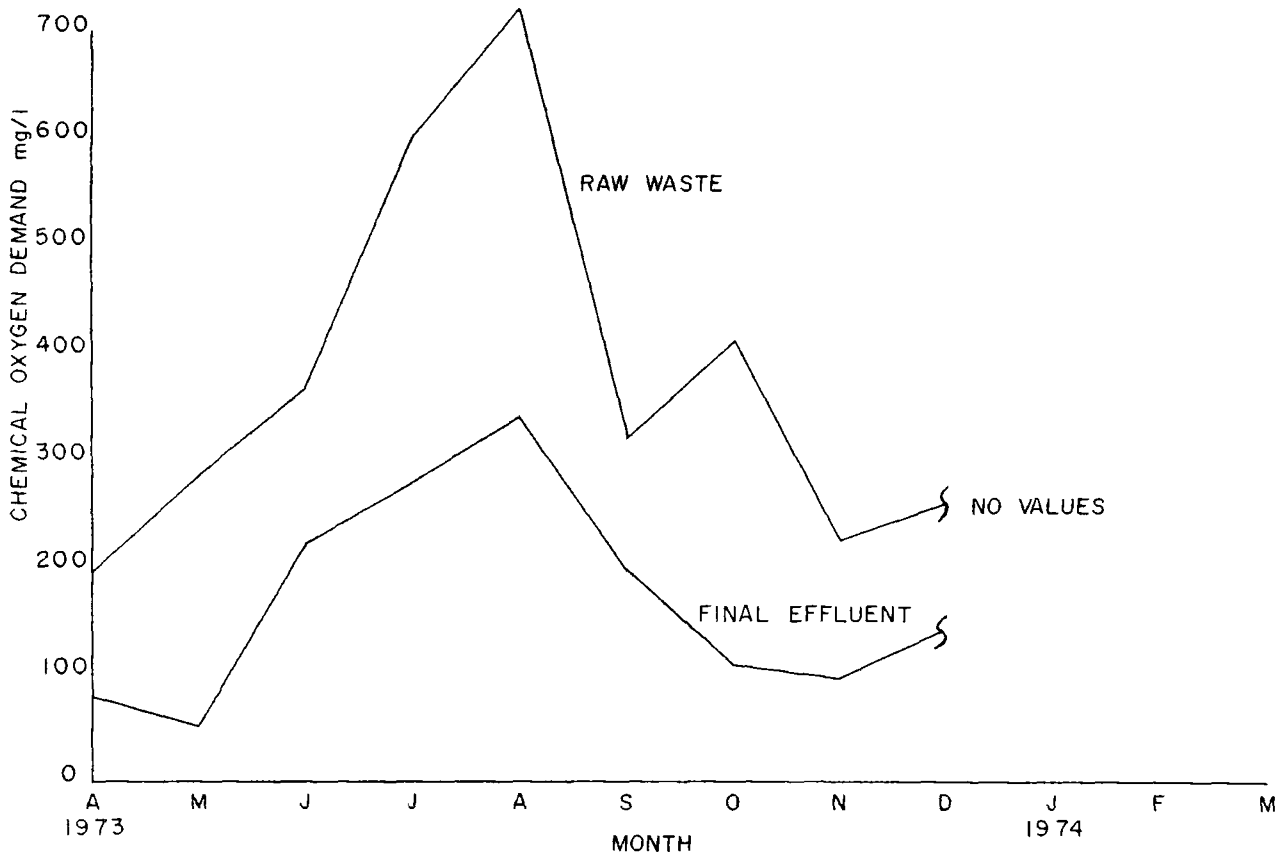


Figure 25. Chemical Oxygen Demand Values, April 1973-March 1974 - Average Monthly Values

Raw suspended solids values are indicated in Figure 26 and Table 23. In addition, the average raw suspended solids values for the preceding year, on a cumulative basis, have been tabulated and plotted. Indications are that the raw wastewater suspended solids concentrations are increasing. This may be due to an infiltration/inflow control program in effect. As the solids concentration increases, operations personnel must regulate sludge processing rates, since the proportion of dry weight of solids to volume of liquid will change.

TABLE 23. RAW WASTEWATER SUSPENDED SOLIDS - APRIL 1973 - MARCH 1974

Month	Value - mg/l	Average - mg/l (1)
April	85	160.6
May	160.5	156.1
June	116	130.9
July	232	134.2
August	116.8	126.3
September	372.5	146.3
October	246.3	149.5
November	342	156.2
December	269	180.1
January	58.6	176.6
February	235.2	187.4
March	215.3	197.7
Average	199.6	147.3
Range	58.6 - 372.5	126.3 - 197.7

(1) Average of month and last preceding 11 months (Cumulative average)

Phosphorus values are plotted in Figure 27. In March 1974 the system was operated for two weeks without tertiary clarification. Polymer and alum

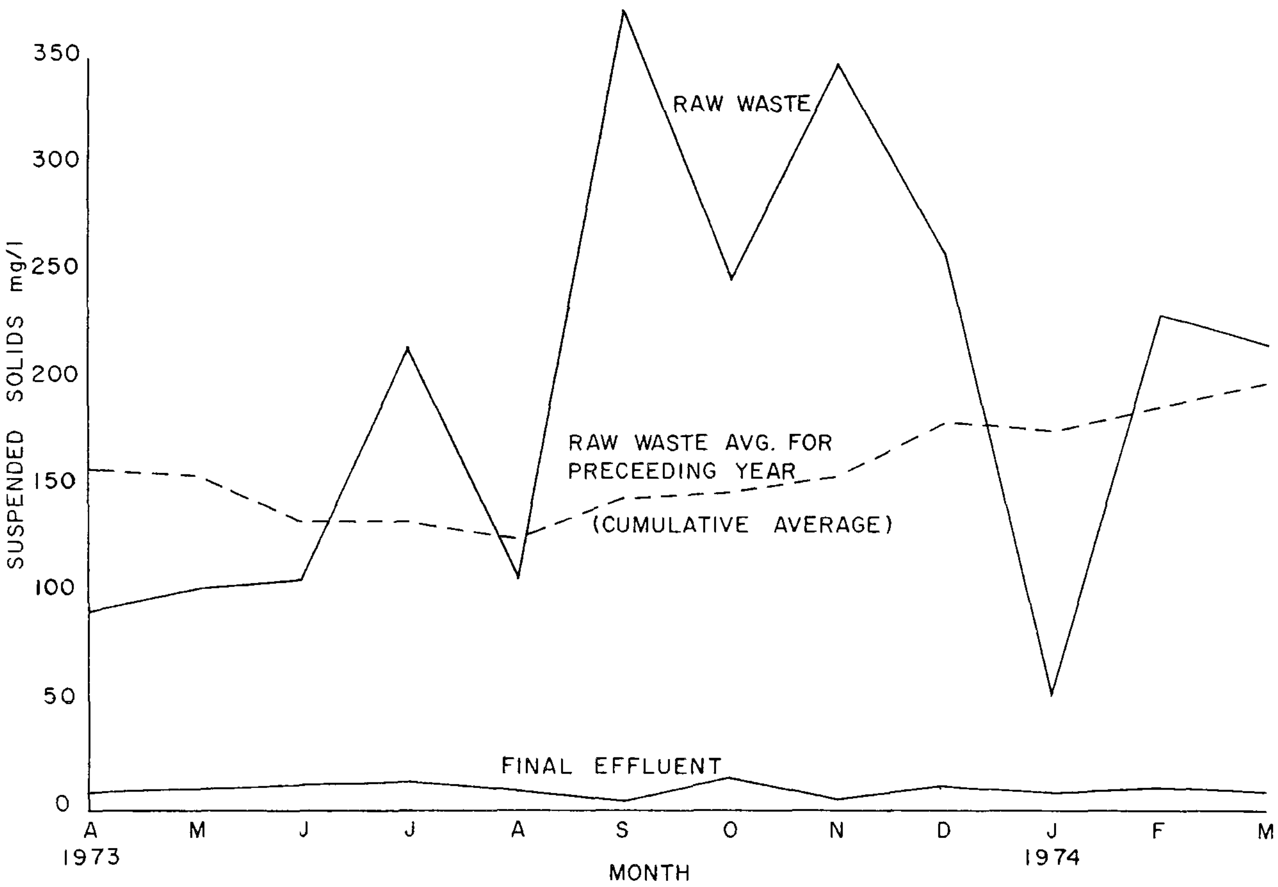


Figure 26. Suspended Solids Values - April 1973-March 1974 - Average Monthly Values Except Where Noted

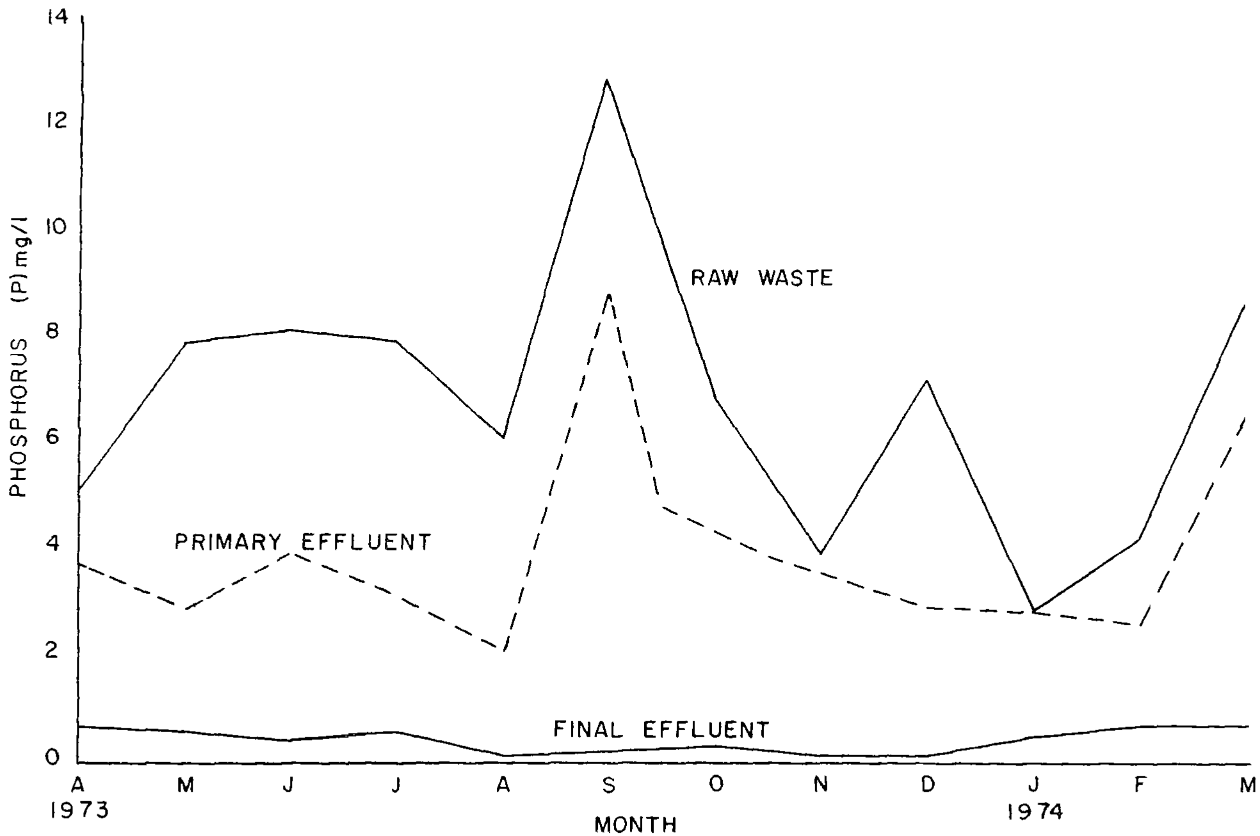


Figure 27. Phosphorus Values (As P) - March 1973-April 1974 - Average Monthly Values

were dosed directly to the tertiary filter pump wet wells, where there was no provision for adequate mixing. The phosphorus removals in the tertiary portion of the plant were poor at this time, 69.1% removal between the secondary and final stations versus an average 75.2% during the entire data collection phase.

In September 1973 the lime feed system was down for repairs. During this time the raw phosphorus was 12.9 mg/l and the primary effluent phosphorus reached 8.8 mg/l. The removal efficiency was 31.2% across the primary for this period, although removals during the entire data collection phase averaged 41.2%. Overall removals of total phosphorus averaged 90.1% during the data collection phase and 91.1% during September 1973, indicating that the extra phosphorus load and failure of primary lime system had adequate backup on the tertiary system, which maintained a high quality effluent.

The stream criteria are based upon meeting an average soluble total phosphate (PO_4) value of 0.2 mg/l with a maximum value of 1.0 mg/l. This compares to a value of 0.06 mg/l of total soluble phosphorus (as mg/l P) a value rarely achieved. The monthly averages of total soluble phosphorus are plotted in Figure 28.

Phosphorus is removed in two locations in the plant, primary and tertiary. The primary removals are obtained by adding lime to the effluent from the surge storage tank and precipitating out a calcium phosphate salt.

A typical reaction is shown below:

$$5\ Ca^{++} + 4\ OH^- + 3\ HPO_4^= \longrightarrow Ca_5OH(PO_4)_3 \downarrow + 3H_2O$$

In addition, the excess lime is converted to carbonate in a parallel reaction.

$$Ca(OH)_2 + Ca(HCO_3)_2 \longrightarrow 2\ CaCO_3 \downarrow + 2H_2O$$

The lime from the carbonate is recalaimed in some facilities by heating.

$$CaCo_3 \xrightarrow{\text{Temp.} > 1800\ F} CaO + CO_2$$

The average value of phosphorus is shown in Table 24. as mg/l Total Phosphorus.

The overall removal of phosphorus was 90.1% during the data collection phase. Various recycle sources added an additional 8.9 mg/l of Total Phosphorus, an increase of 128%. The primary units decreased this to 3.7 mg/l (Figure 29), a removal of 45.6% of the raw phosphorus, or 76.1% of the raw plus the recycled loading.

There was a reduction of 30.4%, from 2.7 mg/l to 0.69 mg/l, across the tube settlers where the alum was added (Figure 30). The secondary system

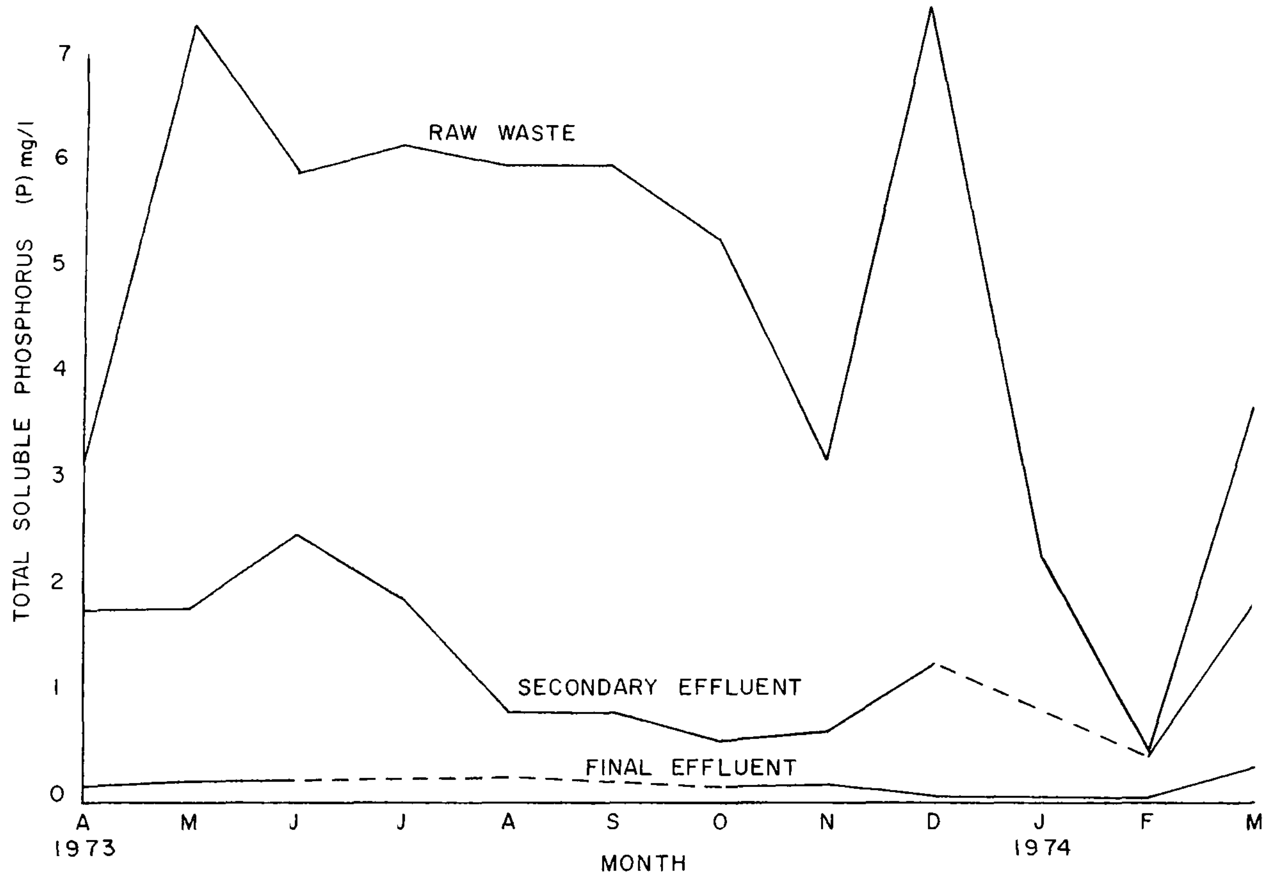


Figure 28. Total Soluble Phosphorus Values, April 1973-March 1974 - Average Monthly Values

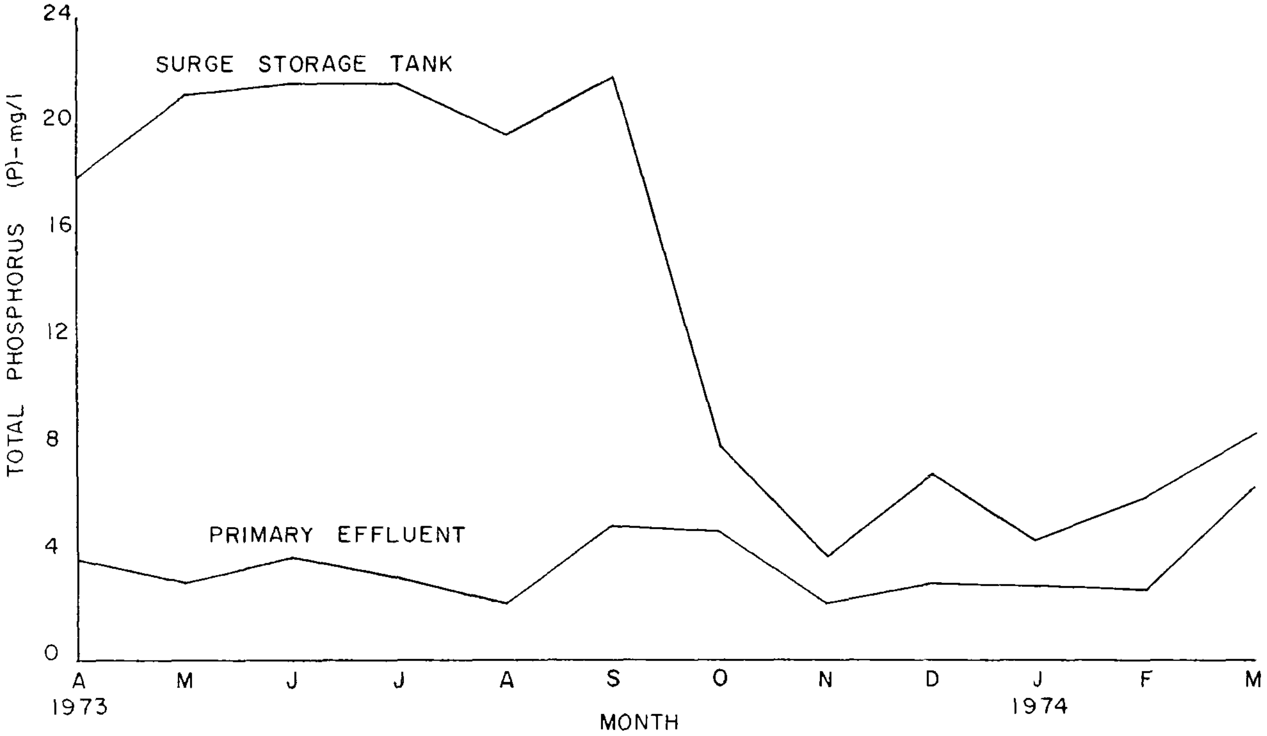


Figure 29. Primary Phosphorus Removals, April 1973-March 1974 - Average Monthly Values

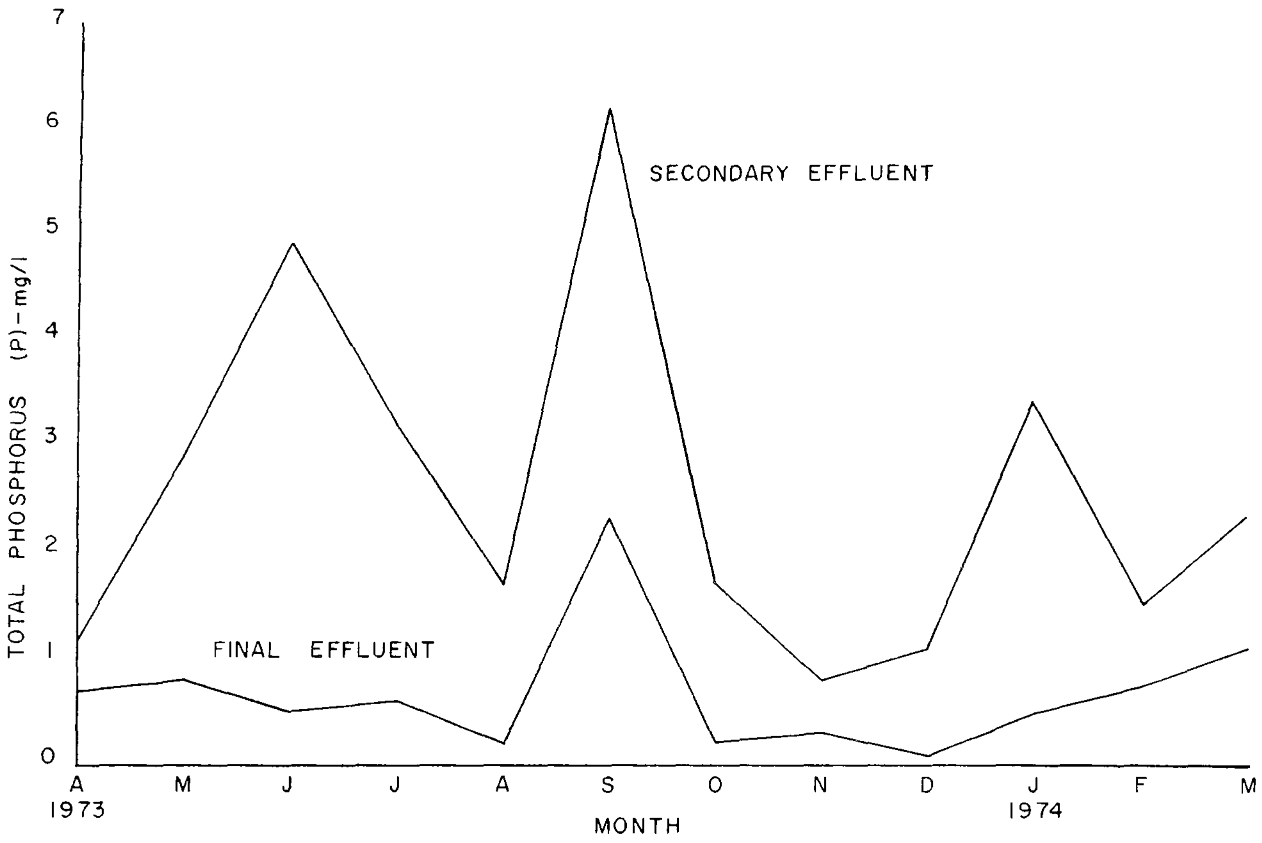


Figure 30. Tertiary Phosphorus Removal Values, April 1973-March 1974 - Average Monthly Values

removed 19.3% of the phosphorus fed to it, in either a biological mode (metabolic uptake) or by capturing small particles which had escaped the primaries.

TABLE 24. AVERAGE TOTAL PHOSPHORUS VALUES

APRIL 1973 - MARCH 1974

Process Stream	Average mg/l	% (1)
Raw Waste	6.8	-
Surge Storage	15.5	(130)
Primary Effluent	3.7	45.6
Secondary Effluent	2.7	60.3
Tube Settler Effluent	0.69	89.9
Pressure Filter Effluent	0.85	87.5
Chlorine Contact Tank Effluent	0.67	90.1

(1) % reduction from raw value

The tertiary treatment facility polishes the effluent by use of alum. The following reactions occur:

$$Al_2\ (SO_4)_3 \cdot 14\ H_2O + 2\ PO_4^{-3} \longrightarrow 2\ Al\ PO_4 \downarrow + 3\ SO_4^{-2} + 14\ H_2O$$

$$Al_2\ (SO_4)_3 \cdot 14\ H_2O + 6\ HCO_3 \longrightarrow 2\ Al\ (OH)_3 \downarrow + 6\ CO_2 + 14\ H_2O + 3SO_4^{-2}$$

Chemical consumption during the data collection phase included an average lime consumption of 248 mg/l as CaO and 105 mg/l as alum.

NITROGENOUS OXYGEN DEMAND

Nitrogen Compounds in Wastewater

Nitrogen is one of the fundamental nutrients required in the life cycle of all plants and animals. In sewage, the major source of the nitrogen compounds is from animals, specifically human beings. In some cases, industrial wastes will contain various nitrogen compounds due to the type of products being manufactured. Frequently, however, industrial wastes will be deficient in nitrogen and will require the addition of nitrogen compounds in order to grow and maintain the biological cultures necessary to stabilize the wastes.

The human body utilizes nitrogen available from plants and other animals in the form of protein. Within the body, protein is used largely for growth and repair of muscle tissue. Some may be used for production of energy. The waste products of the body are released in the form of feces and urine, which contain excess nitrogen compounds.

The feces contain major amounts of unassimilated protein (organic nitrogen). During transmission to the sewage treatment plant some of the protein is converted to ammonia by saprophytic bacteria. This can be described as follows:

$$\text{Protein (Organic Nitrogen)} + \text{Bacteria} \longrightarrow NH_3$$

In urine, the nitrogen exists primarily as urea, which is hydrolyzed to form ammonium carbonate. The enzyme urease causes the reaction, as follows:

$$C\begin{matrix} \diagup NH_2 \\ = \\ \diagdown NH_2 \end{matrix} O + 2H_2O \xrightarrow[\text{Urease}]{\text{enzyme}} (NH_4)_2\ CO_3$$

Nitrites and nitrates found in raw sewage are generally in concentrations of less than 1 mg/l because, under anaerobic conditions, nitrites and nitrates tend to be reduced to free nitrogen gas. Since nitrites are not stable, the concentration of nitrites in wastewater has little significance on the nitrogenous oxygen demand.

The relationship between carbon and nitrogen compounds is shown in Table 25, for a conventional secondary treatment process.

As indicated in Table 25, 90% of the Carbonaceous Oxygen Demand has been satisfied. The oxygen demand of the organic matter, however, in only 1.5 mg/l per mg/l organic matter. The oxygen demand of the NH_3 is 4.5 mg/l per mg/l NH_3, therefore, requiring a higher degree of treatment. Even though 90% of the carbonaceous oxygen demand has been satisfied, only 74% of the total oxygen demand has been satisfied.

TABLE 25. RELATIONSHIP OF CARBON OXIDATION AND NITROGEN OXIDATION IN WASTEWATER TREATMENT

	mg/l			
	Wastewater		Final Effluent	
Organic Matter	250		25	
Oxygen Demand		375		37
NH_3	25		20	
Oxygen Demand		112		90
Total Oxygen Demand		487		127
Percent Oxygen Demand Due to NH_3	22%		71%	

Nitrogen Compounds and the Receiving Waters

Nitrogen compounds in sewage treatment plant effluents are more or less undesirable, depending upon the compound discharges.

Organic Nitrogen

The majority of the organic nitrogen entering the facility appears to be broken down during cell metabolism or used in all growth. As a result, the effluent organic nitrogen is low.

Ammonia

The major effect of ammonia discharge is to reduce or deplete the oxygen available for aquatic life in the stream. Once the carbonaceous materials are removed or reduced, there is oxygen available to nitrifying bacteria in the stream to oxidize the ammonia to the nitrate form.

Four atoms of oxygen are required on a stoichiometric basis to oxidize one molecule of ammonia. On a weight basis, slightly less than 4.6 pounds of oxygen are required to convert 1 pound of ammonia to the nitrate form.

It becomes obvious that, with the substantial amount of oxygen required to convert ammonia to the nitrate form, the nitrogenous load on a stream can cause serious reductions, and in some cases complete depletion of all dissolved oxygen.

The disadvantages of ammonia nitrogen in effluents can be summarized as follows:

1. Ammonia consumes dissolved oxygen in the receiving water;
2. Ammonia reacts with chlorine to form chloramines which are less effective disinfectants than free chlorine;
3. Ammonia is toxic to fish life;
4. Ammonia is corrosive to copper fittings;
5. Ammonia increases the chlorine demand at waterworks downstream from the point of discharge of the sewage treatment plant.

Nitrates

High concentrations of nitrates in drinking waters can cause problems for some young babies. Nitrates in food and water are normally denitrified to nitrogen gas by bacteria in the intestines. However, some newborn infants do not have a complete intestinal flora. The bacteria which are present reduce the nitrates to nitrites, which then combine with hemoglobin to produce methemoglobinemia. This problem is characterized by the morbid condition in which the surface of the body appears blue. The 1962 U.S. Public Health Service Drinking Water Standards limited nitrates to 10 mg/l NO_3-N as a maximum concentration.

1. Nitrates are a source of oxygen for certain bacteria and help prevent septic conditions;

2. Nitrified effluents are more effectively and efficiently disinfected by chlorine treatment;

3. A nitrified effluent usually contains less soluble organic matter than the same effluent before nitrification.

It is apparent that the overall effect of nitrates on the receiving waters depends upon the amount of dilution water available in the receiving stream and upon the downstream usage.

Nitrogenous Oxygen Demand in the Commonwealth of Pennsylvania

The Commonwealth has required the satisfaction of the oxygen demands which can be exerted by nitrogen compounds, in some plants since 1970.

This policy, although perhaps not clearly enunciated, can be determined by examination of the equation used by the Commonwealth to calculate the Total Biochemical Oxygen Demand (BOD_T). The equation used is:

$$BOD_T = 1.5\ (BOD_5) + 4.6\ (NH_3 \cdot N)$$

The first term in the BOD_T equation, 1.5 (BOD_5), is an approximation used to convert the 5-day oxygen demand of the carbonaceous BOD to its ultimate oxygen demand. Although the factor will vary from waste to waste, it is a reasonable approximation.

The second term in the BOD_T equation, 4.6 ($NH_3 \cdot N$), is a reasonable approximation of the oxygen demand which could be exerted by the ammonia in the sewage treatment effluent. Essentially, this term says that 4.6 pounds of oxygen are used to stabilize 1 pound of ammonia, as discussed previously.

Although the requirements of the Commonwealth will vary from stream to stream, the criteria specified can be achieved, except in cold wastewaters, by good BOD_5 removal and by conversion of the ammonia to the nitrate form (biological nitrification).

Biological Nitrification at the Hatfield Township Waste Treatment Plant

The construction permits issued to the Hatfield Township Municipal Authority were revised after the completion of design, to require satisfaction of the oxygen demand due to the nitrogen compounds in the effluent. This can be accomplished by biological nitrification.

The conditions for biological nitrification of municipal effluents have been worked out by Downing and Hopwood in the article, "Some Observations on the Kinetics of Nitrifying Activated Sludge Plants", who showed that:

> *"...to achieve nitrification consistently, the period of aeration must exceed a minimum value which is a function of the concentration of activated sludge, the temperature, and the strength of the sewage. When conditions are favourable for nitrification, the rate of nitrification and rate of consumption of oxygen due to this process will tend to an equilibrium level which is proportional to the concentration of activated sludge under aeration."*

The original parameters used in the design of the Hatfield Township advanced waste treatment plant provided for each aerator to be capable of transferring 100% of the theoretical oxygen needed to achieve the specified BOD reduction. This was done in order to provide backup for mechanical failure, and also to have additional oxygen available to achieve biological nitrification when required.

Appendix E consists of calculations which demonstrate that, with the oxygen transfer capability and detention times provided in the advanced waste treatment plant, as constructed, it should be possible to achieve:

1. 78% nitrification of the influent ammonia at a winter wastewater temperature of 51° F (10.75° C); and

2. 94% nitrification of the influent ammonia at a summer wastewater temperature of 77° F (25° C).

It should be noted in analyzing Appendix E that the following factors affect the ability of the plant to achieve nitrification:

1. At the winter condition, no beneficial credit was given to the fact that the pH of the mixed liquor will be in the optimum range of 7.5 to 9.3. The bacteria which oxidize ammonia and nitrites function most efficiently in this range. Test work has demonstrated that at a pH of 8.4, the rate of oxidation of ammonia by the nitrifying organisms is maximum.

2. During the winter the stream will generally contain more dissolved oxygen than in the summer due to the lower degree of activity of all organisms (particularly the nitrifiers) at colder temperatures, and the increasing solubility of oxygen in water with decreasing water temperature.

3. The assumed BOD_5 removal across the primary treatment units was estimated at 60%. Better BOD_5 removals across the primary treatment units will make more oxygen available to the nitrifying organisms in the aeration tanks.

4. Lime precipitation at the primary treatment step removes more of the elements which are toxic to the nitrifying organisms than would occur in a plant without lime precipitation.

Analytical Problems

All analyses were performed at the Hatfield Township Municipal Authority laboratory by employees working under normal operating conditions for a small municipal wastewater treatment plant. Tests were performed in accordance with standard methods except for the use of the membrane method for Fecal Coliform.

The nitrogen analytical procedures were especially difficult. The use of the Nesslarazation nethod for Ammonia-Nitrogen was replaced by the Kjeldaht method, which is more reliable, however, more expensive. This also produced organic-nitrogen values. The results in the early data collection phase are questionable.

PLANT PERFORMANCE

The ammonia-nitrogen values varied greatly during the test period. (see Figure 31) There was consistent reduction in the ammonia, however, reducing the oxygen demand of the effluent. At those times when there were neither human nor mechanical problems, the Nitrosomes and Nitrobacter enjoyed a stable environment and a nitrified effluent was produced.

The combination of BOD_5 satisfaction with biological nitrification in a single reactor, while theoretically possible, is dependent upon many factors, including:

1. No mechanical failures

2. Consistant flow rates

3. Constant NH_3 feed

4. Minimal sludge wastage

At certain periods the nitrification in Hatfield was extremely satisfactory. At other times it was a victim of operator error, equipment malfunction, or infiltration/inflow.

The secondary clarifiers developed a build-up of floating solids during those times that nitrification was occurring. It was theorized that this was the result of denitrification occurring in the secondary clarifiers. During denitrification, nitrogen gas if formed, and the gas bubbles tend to "float" the sludge. The sludge sinks rapidly when hit with a heavy spray of water.

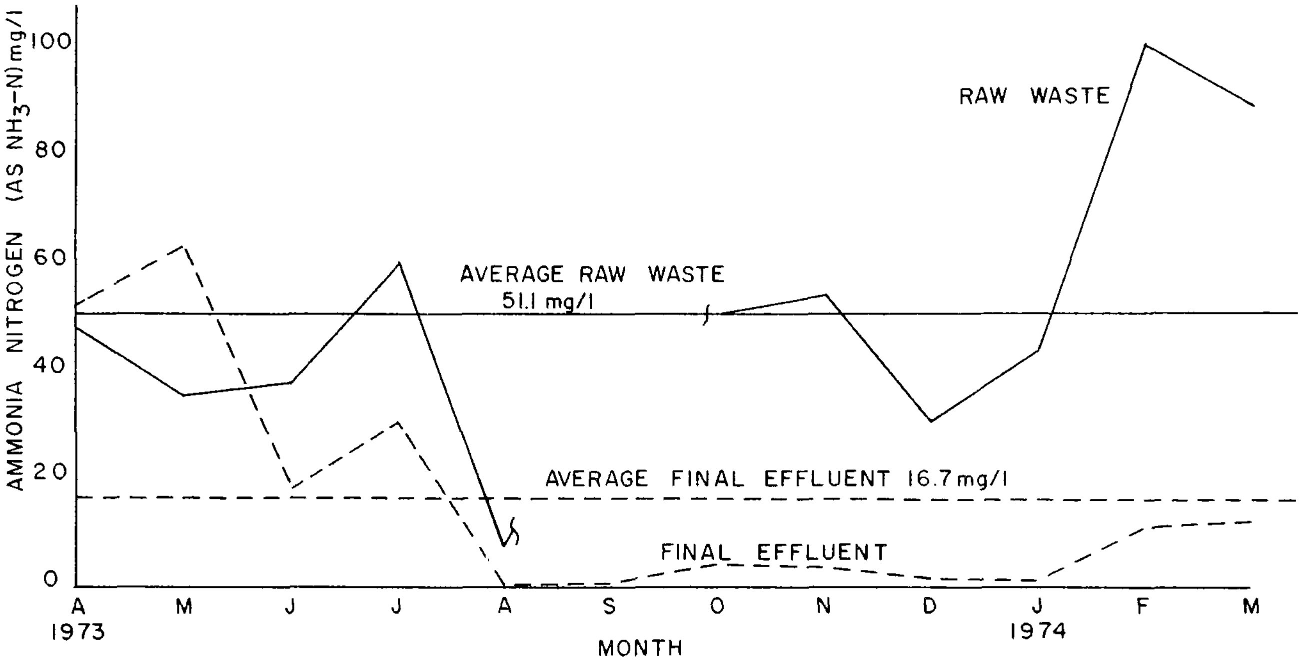


Figure 31. Ammonia-Nitrogen Values, April 1973-March 1974 - Average Monthly Values

FECAL COLIFORM

Fecal coliform values are given in Table 26. As indicated during the first three months of the grant, the effluent was not in compliance with State coliform criteria. Once plant operations personnel determined the correct operations of the flow control and chlorination systems, however, they instituted a program to maintain the minimum chlorine residual of 1.0 mg/l and have produced a fecal coliform-free effluent. No attempt has been made to determine potential side effects caused by excess chlorination.

TABLE 26. FECAL COLIFORM ANALYSIS

April 1973 - March 1974

All Values in Colonies/100 ml x 10^6

	Raw	Final
April	0.96	0.57
May	3.001	0.002
June	2.174	0.002
July	-	-
August	1.853	0
September	1.513	0
October	0.392	0
November	0.181	0
December	0.08	0
January	0.18	0
February	0.04	0
March	0.07	0

UNIT PROCESS EFFICIENCIES

On the following pages a series of curves demonstrating the removal efficiencies of the unit process operations, during the period April 1973 through March 1974, for various contaminants are presented.

BOD_5 removal efficiencies are plotted in Figure 32. There is an increase in BOD_5 through the surge storage tank due to internal recycles and resolubilization of organic sludges. The greatest net change is 64.4% removal across the secondary system. There is continued BOD_5 removal across the entire treatment facility.

Suspended solids removals are presented in Figure 33. The greatest net change is in the primary clarifier where the reduction is from 1251.9 mg/l to 98.9 mg/l, a 92.1% removal. Unfortunately, this is based upon raw suspended solids and recycle solids. The percent removal from the raw waste alone was 50.5%. The secondary system, on the other hand, reduced the 74.0 mg/l suspended solids fed to 14.9 mg/l, a reduction of 79.9%. The recycled solids do not clarify out easily in the primary clarifiers. When the relatively high solids primary effluent comes in contact with the activated sludge the biological floc may act as a polymerizing agent and cause it to settle out in the secondary clarifiers.

The recycling of the primary sludges tended to add to the overflow suspended solids, while it aided in the prevention of plugging of sludge withdrawal pipelines caused by the rapidly settling chemical, primary sludge.

Chemical oxygen demand decreases through the treatment facility are shown in Figure 34. There is an increase of COD in the surge tank with a sharp decrease in the primary. There is a gradual decrease, indicating a marginal degree of removal of the COD, in the secondary and tertiary systems.

The efficiency of removal of phosphorus is shown in Figure 35. As in the other cases, there is a marked increase from recycle streams, in this case an additional 128%. The primary units removed 45.6% of the total phosphorus in the raw waste with lime. When one considers the recycle streams increased the total phosphorus to 15.5 mg/l, the unit removed 76.1% of the phosphorus while operating at a pH of 9.5. The secondary system removed an additional 27% of the phosphorus in the primary effluent for an overall primary-secondary reduction of 49.7% of the raw, or 82.6% of the primary influent. The tertiary-filter system removed 75.2% of the secondary effluent phosphorus with a slight pick-up through the filters. Overall reduction was 90.1%.

From a process efficiency viewpoint, the advantages of flow equalization were somewhat offset by the increases in solids, organics and phosphorus. It is possible that many plants have a similar problem with digester supernatent, or incinerator scrubber water, and/or filtrate or centrate. The problems may be so difficult to detect that plant personnel might not recognize their existence until the facility begins to produce a poor quality effluent.

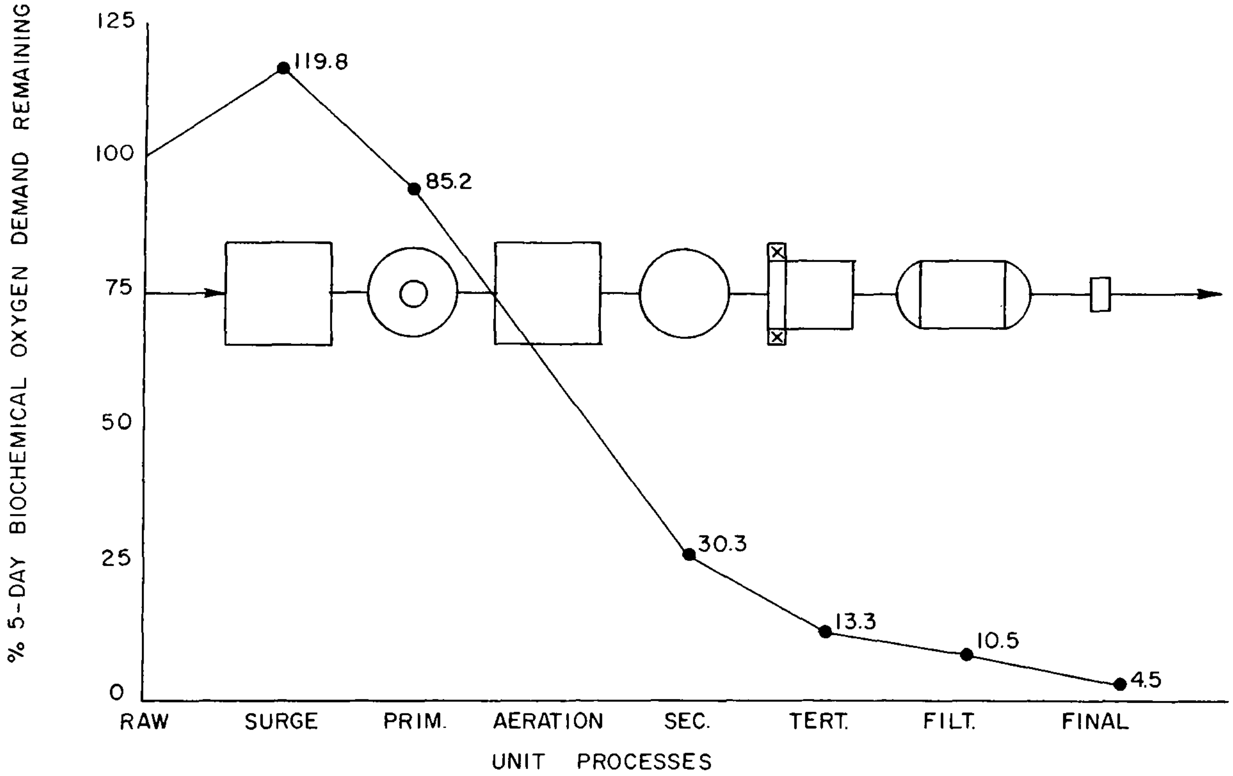


Figure 32. Percent BOD5 Remaining, April 1973-March 1974 - Average Values

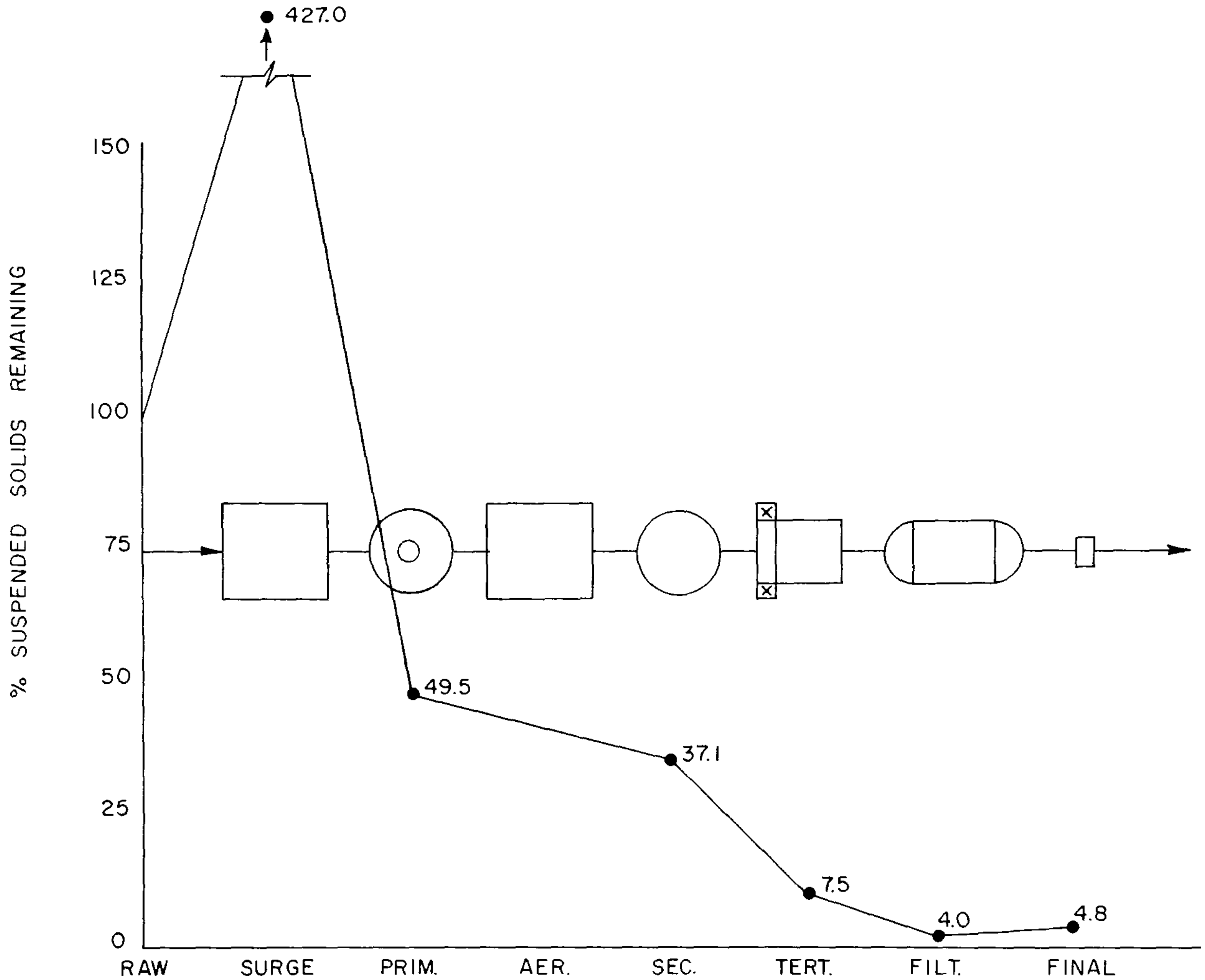


Figure 33. Percent Suspended Solids Remaining, April 1973-March 1974

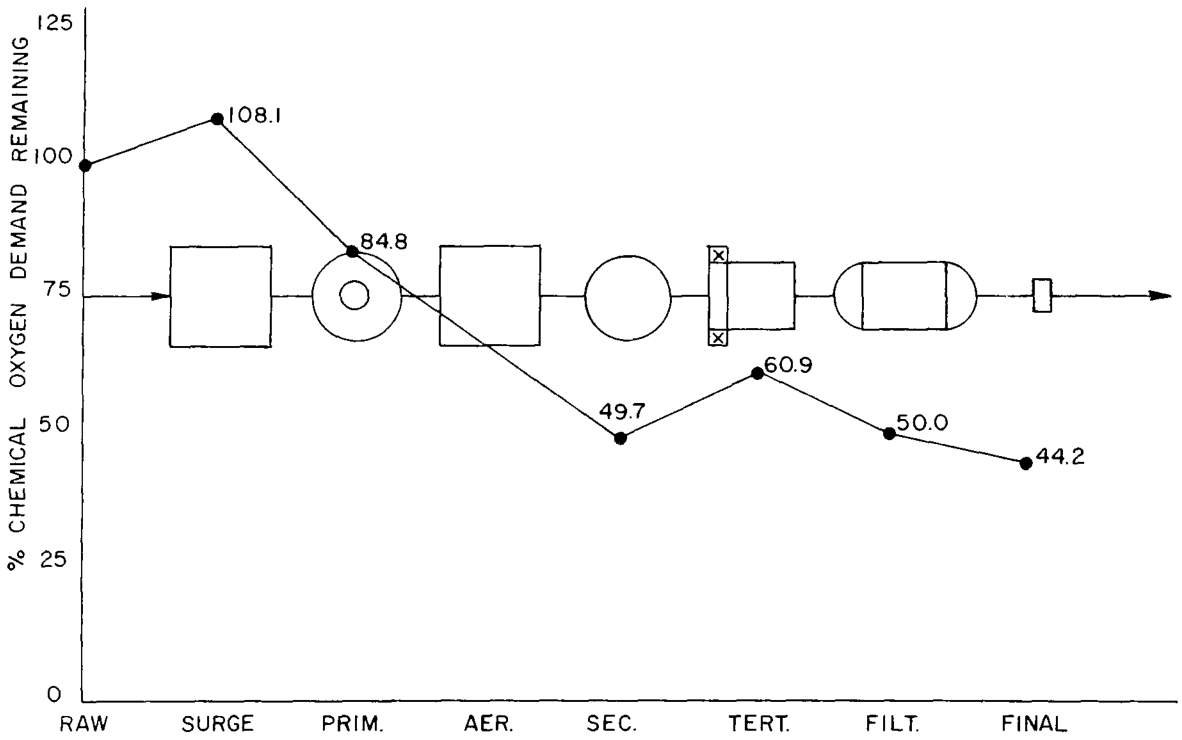


Figure 34. Percent COD Remaining, April 1973-March 1974 - Average Monthly Values

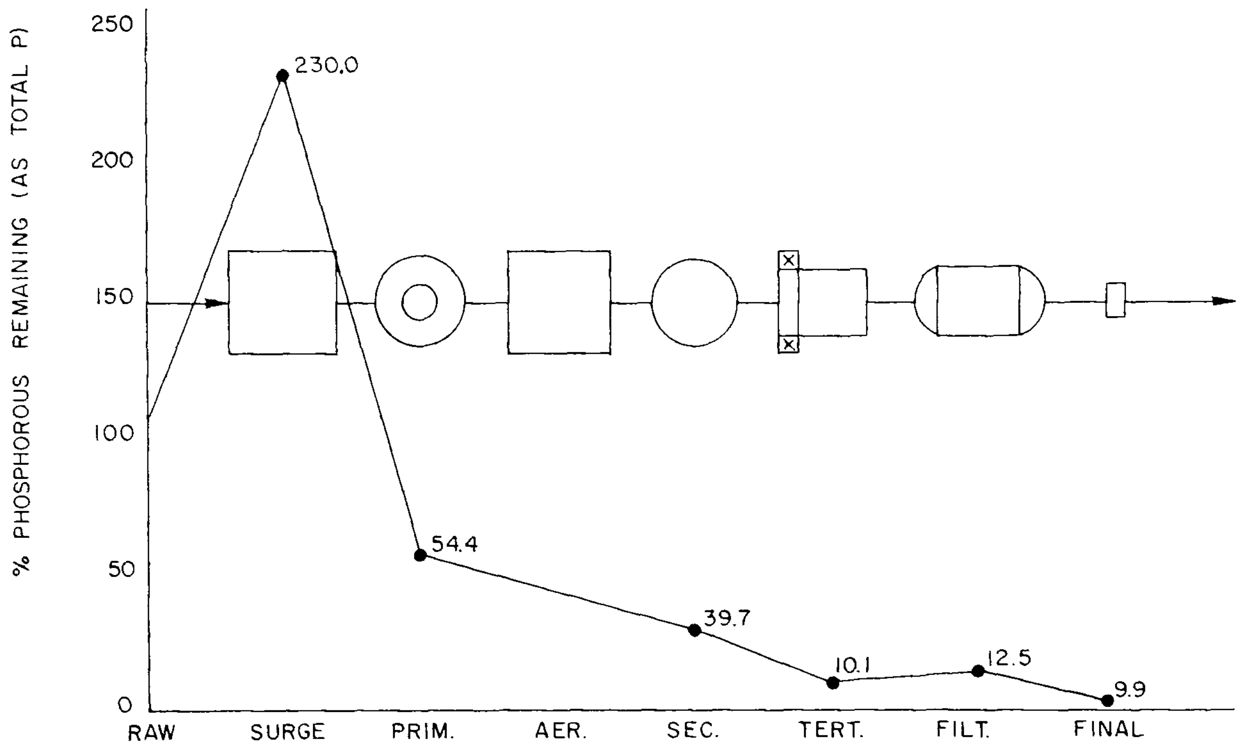


Figure 35. Percent Phosphorus Remaining, Average Values - April 1973-March 1974

OPERATING COSTS

A major factor in process selection is the Cost/Benefit ratio. By this is meant, the greatest benefit for the lowest cost. The costs incurred in operating the Hatfield Township facility are shown in Table 27.

TABLE 27. ANNUAL OPERATING COST x $1000

	1973	1974
Salaries and Fringe Benefits	188.2	286.1
Sludge Hauling	-	15.8
Utilities	65.9	90.4
Ash Disposal	4.3	5.3
Chemicals	33.7	43.0
Administration	10.0	12.8
Maintenance, Plant	36.6	31.1
Maintenance, Collection System	8.9	6.2
Maintenance, Vehicles	2.2	5.4
Laboratory Supplies	2.0	6.9
TOTAL OPERATING COSTS	351.8	504.0

The total operating costs for 1973 were $351,800, and for 1974, $504,000. Plant flows for these periods totaled 745.0 and 637.5 million gallons respectively for the average operating costs of $472.21 and $790.59 per million gallons. The largest single cost was labor at $188,200 and $286,100, or $256.62 and $448.78 per million gallons. Utilities represented the next major expenditure at $88.46 and $141.30 per million gallons. The chemical costs for 1973 and 1974, in turn, are broken down in Table 28.

TABLE 28. CHEMICAL COSTS $/MILLION GALLONS

	1973	1974	April '73 - March '74
Lime as CaO	$11.45	$20.26	$13.65
Alum	16.17	24.49	18.25
Polymer	2.69	6.37	3.61
Chlorine	3.37	2.55	3.17
	$33.68	$53.67	$38.68

The removal of phosphorus is one of the major design objectives at the Hatfield plant. During the test period it is estimated that 22,746 pounds of phosphorus were removed from the raw waste in the primaries with lime and 12,480 pounds in the tertiary system using alum and polymer. Chemical costs for phosphorus removal in the primaries with lime was approximately \$0.44 per pound of phosphorus removed. The cost of chemicals for tertiary treatment amounted to \$1.29 per pound of phosphorus removed.

In January 1974 there was a general increase on chemicals. Lime increased to \$24.07 per ton, and alum in 100 pound bags to \$86.02 per ton, whereas the cost in 1973 was \$22.50 and \$70.20 per ton respectively

SECTION VII

EXPERIENCE WITH EXISTING SLUDGE HANDLING AT THE HATFIELD AWT FACILITY

BACKGROUND

The sludge handling facilities at the Hatfield Township plant were designed to handle the sludges produced by the original 1.0 mgd (3,785 cu m/Day) conventional activated sludge process plant. They consist of duplicate 4' x 4' (1.22 m x 1.22 m) rotary vacuum filters and a 5-hearth 10'-9" (3.28 m) I.D. multiple hearth furnace with 235 square feet (21.83 sq. m) of area. The furnace was designed to handle 1,750 lbs/hour (794.5 kg/hour) of wet sludge at 30% T.S. Each vacuum filter is capable of producing one ton of wet sludge with provision for conditioning with lime (as $Ca(OH)_2$) and $FeCl_3$. The furnace design rating was 7.5 Lbs/SF/hour (86.4 kg/sq m/hour) of wet sludge, although at times this was exceeded with conventional sludges with no problem. When operating at 1,750 lbs/hour (794.5 kg/hour) of wet sludge, the 50 ft^2 (4.65 sq m) of vacuum filter area resulted in a filtration rate of 35 lb/ft^2/hour (170.9 kg/sq m/hour) of wet sludge.

It should also be pointed out that there was provided an 18' Ø x 10'-6" SWD (5.49 m Ø 3.20 m SWD) gravity thickener to thicken the mixed primary and secondary sludges. The design of the hydraulic expansion with AWT processes was begun in 1968, and the knowledge of sludge production of the various units of operation was uncertain. It was, therefore, decided to provide an extra gravity thickener and not design the expanded sludge processing facilities until such time as actual operational data on sludge production would be generated. An arbitrary rating of 2.5 mgd (9,462.5 cu m/day) was assigned to the facility due to the limited solids facilities.

SLUDGE PRODUCTION

A review of late 1973 operating conditions at the facility, as well as chemical purchased, indicates the following chemical doses were used, based on total plant flow.

Lime - 248 mg/l as 90% available CaO

Alum - 110 mg/l as $Al_2(SO_4)_3 \cdot 16\ H_2O$

As a result, the sludge production is summarized in Table 29.

TABLE 29. SLUDGE PRODUCTION - ESTIMATED, JUNE 1974

	lbs/mg	g/cu m
Primary Sludge	1,668	200.16
WAS	330	39.60
$CaCO_3$	3,345	401.40
$Ca_5OH(PO_4)_3$	360	43.20
Aluminum Hydroxide and Aluminum Phosphate	271	32.52
TOTAL	5,974	716.88

These quantities differ with the theoretical figures used in a June 1973 report on sludge disposal, since that report was based upon lime dose of 420 mg/l as $Ca(OH)_2$, alum dose of 150 mg/l, 8 mg/l P removed in the primary, and 2 mg/l removed in the tertiary. They compare with an average sludge processed of 7,033 lbs/mg (844 kgs/cu m) for the period July 1973-June 1974, as shown in Table 30.

TABLE 30. SLUDGE PRODUCTION

(Includes Recycles)

Month	Plant Flow mgd	Plant Flow 1,000 cu m day	Sludge Processed tons/day	Sludge Processed kgs/d	Sludge Processed lb/mg	Sludge Processed g cu m
July '73	1.8	6.81	5.25	4,767	5,833	700
Aug.	1.6	6.06	7.72	7,010	9,650	1,158
Sept.	1.3	4.92	5.79	5,257	8,908	1,069
Oct.	1.56	5.90	6.77	6,147	8,679	1,042
Nov.	1.14	4.31	4.49	4,077	7,877	945
Dec.	3.30	12.49	4.63	4,204	2,806	337
Jan. '74	2.83	10.71	5.05	4,585	3,560	427
Feb.	2.09	7.91	4.00	3,632	3,820	458
March	2.13	8.06	7.94	7,210	7,440	893
April	2.35	8.89	7.14	6,483	6,080	730
May	1.88	7.11	8.56	7,772	9,100	1,092
June	2.06	7.80	10.95	9,943	10,640	1,277
Average	2.00	7.50	6.52	5,924	7,033	844

GRAVITY THICKENING

Reviewing the gravity thickener requirements, for a 1 mg (3,785 cu m) flow, 512 square feet (47.56 sq. m) is required on a floor loading basis. With an average flow of 2.0 mgd (7,570 cu m/day), a minimum area of 1,024 square feet (95.13 sq. m) is required. The existing gravity thickener is 40' Ø x 10' SWD (12.2 m Ø x 3.05 m SWD), with 1,256 square feet (116.68 sq. m) which indicates that the thickener has enough capacity. Limitations of the furnace, however, have led to an overloaded situation. This has resulted in a fairly consistent operational problem, with the finer, more difficult to dewater, solids not settling and thickening.

In the early stages of operation, prior to WAS production and alum usage, high lime doses were fed in the primary system. Sludge production was estimated as shown in Table 31.

TABLE 31. SLUDGE PRODUCTION, JANUARY-MARCH 1973

	Lb/Day	Kg/Day
Primary Solids	3,400	1,543.6
$CaCO_3$	8,800	3,995.2
$Ca_5OH(PO_4)_3$	720	326.9
	12,920	5,865.7

At the same time the waste primary sludge flow rate was 250 gpm (5.8 l/sec) at 4,500-5,000 mg/l SS, or 6,432 Lb/d (2,916 kg/d). During the time only primary sludges were being processed, the thickened sludge averaged 8-14% TS. At the same time the vacuum filter was loaded at 11.8 Lb/ft^2/hr. (57.62 kg/sq m/hr) and produced a cake of 30-40% TS.

The maximum allowable overflow rate on the gravity thickener is 800 gal/sq. ft/day (32.64 cu m/day/sq m) or 1.009 mgd (3,851 cu m/day). Primary sludges are wasted at 250 gal/min. (15.8 l/sec); secondary at 50-100 gal/min. (3.16-6.31 l/sec); the aluminum waste sludge pumps at 60-1,200 gal/min. (37.86-75.72 l/sec); scum at 90-100 gal/min. (5.68-6.31 l/sec) (4 pumps); and the thickener dilution water at 150-300 gal/min. (9.47-18.93 l/sec). The result is a potential hydraulic load of 2,230 gal/min. (140.71 l/sec), which would overload the facility by a factor of 3. The operators, therefore, either operate the waste pumps at reduced rates or stagger the pump operations. Neither of these methods of operations is ideal. The provision of a second gravity thickener with 50' Ø (15.24 m Ø) would allow for future anticipated solids loadings to 4.0 mgd (15,140 cu m/day) and permit an additional 1,090 gpm (68.78 l/sec) overflow rate, if necessary.

The present operating conditions result in a thickened sludge to dewatering of 7.5-8.5% TS and an overflow of 2,000-6,000 mg/l suspended solids.

In general the flows to the thickenera are now:

	Volume/Day
Primary:	
300 gal/min x 1,440 min/day =	432,000 Gallons
18.93 l/sec x 86,400 sec/day =	1.64×10^6 Liters
WAS:	
1,001 gal/min x 3 shifts x 90 min/shift x 2 =	54,000 Gallons
6.31 l/sec x 3 x 90 x 2 x 60 sec/min =	0.204×10^6 Liters
CWS*:	
900 gal/min x 4 pockets x 3 shifts x 15 min/shift	162,000 Gallons
56.8 l/sec x 4 x 3 x 15 x 60 sec/min =	0.613 Liters
Scum:	
380 gal/min x 15 min/shift x 3 shifts/day =	17,000 Gallons
24 l/sec x 15 x 3 x 60 sec/min =	0.065×10^6 Liters

*CWS - Chemical (Alum) Waste Sludge

The thickener overflow usually is turbid and at best has an overflow suspended solids of 2,000-6,000 mg/l. The addition of 0.5-1.0 mg/l anionic polymer to the thickener feed resulted in a clearer liquor.

SLUDGE DEWATERING - EXISTING VACUUM FILTERS

In order to incorporate the best available technology on the dewatering of the various sludges, a program was entered into to compare the existing drum type vacuum filters with a filter press and a centrifuge.

As indicated in Table 32, the average monthly sludge loadings were 237.6 tons/month of sludge dewatered to 30.1% TS and 48.4% VS.

The average wet sludge filter loadings for the first six months in 1974 were 30.9 lbs/sq. ft/hr (150.9 kg/sq. m/hr), with an average cake concentration of 24-28% TS, after conditioning with 0.3% $FeCl_3$ and 8.7% $Ca(OH)_2$. Operating hours for the solids handling units are shown in Table 33.

TABLE 32. SUMMARY OF SLUDGE OPERATIONS

MARCH 1973 - JUNE 1974

Month	Tons/D.S.	% T.S.	% V.S.	Lbs. $FeCl_3$	Lbs. $Ca(OH)_2$	Adusted[1] Tons D.S.	# $FeCl_3$ 100 Lb DS	# $Ca(OH)_2$ 100 Lb DS
Mar. '73	258.6	40.5	55.4	765	21,000	247.4	0.20	4.34
Apr.	277.5	40.5	59.7	969	21,490	267.4	0.27	3.53
May	336.3	40.5	60.5	1,428	18,890	323.5	0.20	3.76
June	290.6	43.0	56.0	1,275	24,340	279.0	0.17	3.97
July	176.8	24.0	49.0	969	22,140	162.8	0.13	8.59
Aug.	253.9	26.5	36.5	408	27,920	239.2	0.13	6.03[2]
Sept.	191.3	23.0	40.0	612	28.820	173.8	0.10	10.00
Oct.	218.5	28.5	-	357	34,680	209.8	0.10	4.06
Nov.	146.7	30.0	-	408	17,015	134.7	0.21	9.39
Dec.	156.0	27.5	-	561	25,315	143.6	0.25	8.42
Jan. '74	177.5	28.8	-	714	24,175	156.6	1.06	12.26
Feb.	131.0	25.2	35.8	3,315	38,420	112.1	0.43	16.42
Mar.	282.3	25.2	43.6	969	36,800	246.2	0.36	14.32
Apr.	238.5	24.2	53.0	1,785	70,500	214.1	0.55	10.87
May	296.4	27.8	-	2,396	46,560	265.4	0.39	11.29
June	369.3	26.5	-	2,091	59,910	328.6	0.35	12.05
TOTAL	3,801.2			19,022	517.975	3,504.0		
AVG.	237.6	30.1	48.4			219.06	0.30	8.70

(1) Adjusted for 8.7% $Ca(OH)_2$ and 0.3% $FeCl_3$
(2) Switched to Dolimitic Limo

TABLE 33. SLUDGE OPERATIONS - 1974

Hours / Operations

Month	Filtration (2 units)	Multiple Hearth Incinerator
Jan.	984	744
Feb.	888	504
March	1,464	744
April	1,296	648
May	1,248	624
June	1,344	672

FILTER PRESSING SLUDGES

The filter press used was an Edwards and Jones unit with 12' x 12' x 1" (0.305 m x 0.305 m x 2.54 cu) plates. The results are summarized in Table 34. With a feed of 15% $Ca(OH)_2$, or 15 lb. (6.81 kg) of lime to 100 lb. (45.4 kg) of wet sludge, the unit produced a cake of 41% solids and a bulk density of 78 lbs/ft^3 (1,248 kg/cu m). This compares to operation at the rate of 3.25 lbs/hr (1.48 kg/hr). The filtrate averaged 19 mg/l suspended solids.

CENTRIFUGATION OF SLUDGES

A series of tests were run utilizing a Sharples Super-D-Canter on various sludges at the Hatfield AWT facility. Hercules 814.3 Poly-electrolyte was used on all cases. The results of the tests on the primary sludges are shown in Table 35. As indicated, when maximum recovery was sought the cake concentration varied from 19.6-39.2% T.S., with recoveries ranging from 99%-96%. When maximum cake concentration was sought values ranged from 39.2% to 28.2% T.S., with recoveries falling off to between 96-81%.

It was noted that the chemical primary sludges were much finer than normal primary sludges. Polymer was needed to obtain a continuous solids discharge at low rates. Process results did not vary significantly with change in speed differential or pond depth.

TABLE 34. FILTER PRESS TEST DATA - NOVEMBER 1973

Chemical Addition $Ca(OH)_2$ %	Slurry Concen. %	Cycle Time Hours	Cake Solids %	pH Uncond.	pH Cond.	Bulk Density Lbs/Ft^3	Cake Thickness Inch	Cake Thickness cm
20	8.0	1.0	47	7.6	11.6	75	1.0	2.54
15	7.6	1.0	48			75	1.0	2.54
10	8.0	1.0	41	7.5	11.4	77	1.0	2.54
5	7.6	1.75	35	7.6	11.2	76	1.0	2.54
10	5.5	1.75	40	7.1	11.4	78	1.0	2.54
15	5.0	1.3	41	7.5	11.8	78	1.0	2.54
15	5.3	2.0	41	7.0	11.4	81	1.25	3.175
10	5.0	2.5	40	6.9	11.0	79	1.25	3.175

$Lbs/Ft^3 \times 16 = Kg/cu\ m$

TABLE 35. CENTRIFUGE TESTS DECEMBER 1973

PRIMARY SLUDGE

Gallons Per Minute	Thickened Sludge % S.S.	Cent. Cake % T.S.	Recovery %	Polymer #/T	gm/kg
1.0	0.49	19.6	99	12.0	6.0
5.3	1.07	22.8	97	8.4	4.2
1.2	1.05	39.2	96	8.0	4.0
1.2	1.05	39.2	96	8.0	4.0
3.3	1.12	30.0	81	2.6	1.3
1.2	1.05	28.2	91	4.6	2.3

Machine Rated at 60 gal/min (3,786 l/sec) maximum.

Gallons/minute x .0631 = l/sec.

#/T x 0.05 - %

Possible reasons for the fine sludge characteristics are recycle of fines from the filtrate, pressure filter backwash and thickener overflow. In addition, the surge mixers and flash mixers could shear the natural floc particles and yield a fine floc.

The secondary sludge was also tested. The average S.V.I. was 70, and at no time did it exceed 100. This resulted in an easy to handle cake of 11-12% T.S., as shown in Table 36.

TABLE 36. CENTRIFUGE STUDY DECEMBER 1973

SECONDARY SLUDGE

Gallons Per Minute	Thickened Sludge % S.S.	Cent. Cake % T.S.	Recovery %	Polymer #/T	gm/kg
4.6	0.74	12.1	100	6.2	3.1
4.5	0.74	11.6	100	3.8	1.9
4.1	0.74	11.9	99	5.7	2.85
-	0.84	23.5	95	0.0	0.0
2.2	0.73	15.2	82	5.5	2.75
2.8	0.73	14.3	72	4.7	2.35

It must be pointed out, however, that the secondary system removed a large percentage of the suspended solids which escaped the primary clarifiers. It was assumed that much of this was caught up in the WAS, and since it was predominantly $CaCO_3$, it served as an excellent weighting agent. The WAS, in turn, had a polymerizing effect and aided in the secondary clarification, producing a cake with better dewatering characteristics than conventional sludges.

The tertiary alum sludge was centrifuged and the results are shown on Table 37. The alum sludge values of 12-21% T.S. with 99-100% recovery, or 32-35% T.S. with 60-80% recovery, is relatively high according to the Sharples personnel. There was a problem of obtaining a representative sample of the alum sludge, since the wasting of the chemical sludges is accomplished on a "batch-basis" rather than continuous. There is probably carryover from the primaries all the way through to the tertiary system.

TABLE 37. CENTRIFUGE STUDY DECEMBER 1973

TERTIARY (ALUM) SLUDGE

Gallons Per Minute	Liters Per Second	Thickened Sludge % S.S.	Cent. Cake % T.S.	Recovery %	Polymer #/T	gm/kg
4.6	0.29	0.7	12.1	100	6.2	3.1
4.5	0.28	0.7	11.6	100	3.8	1.9
2.3	0.15	2.4	21.3	99	2.1	1.05
3.3	0.21	2.4	18.2	99	1.7	0.85
3.0	0.19	0.2	35.2	80	14.0	7.0
1.7	0.11	0.2	35.0	72	240.0	120.0
1.7	0.11	0.2	31.8	60	14.0	7.0

There were two additional series of tests run, one with combined Secondary/Tertiary and one with all sludge combined. The results are reported in Table 38 and Table 39, respectively.

TABLE 38. CENTRIFUGE TESTS DECEMBER 1973

SECONDARY/TERTIARY

Gallons Per Minute	Liters Per Second	Thickened Sludge % S.S.	Cent. Cake % T.S.	Recovery %	Polymer Utilization #/T	gm/kg
1.1	0.07	2.2	19.4	99	2.4	1.2
2.4	0.15	2.3	10.4	99	2.9	1.45
3.5	0.22	2.3	10.3	98	3.6	1.8
2.2	0.14	2.3	10.2	99	3.0	1.5
1.1	0.07	2.2	19.4	99	2.4	1.2
2.0	0.13	2.2	13.9	86	0.6	0.3
2.0	0.13	2.2	13.3	77	2.7	1.35

TABLE 38 (continued). CENTRIFUGE TESTS DECEMBER 1973
SECONDARY/TERTIARY

	OPTIMUM	
GPM	1.1 - 2.2	1.1 - 2.0
L/Sec	0.07 - 0.14	0.07 - 0.13
% S.S.	2.2 - 2.3	2.2
% T.S.	10.4 - 19.4	13.3 - 19.4
Rec.	98 - 99	77 - 99
#/T	2.4 - 3.6	0.6 - 2.7
gm/kg	1.2 - 1.8	0.3 - 1.35

TABLE 39. CENTRIFUGE TESTING DECEMBER 1973
COMBINED SLUDGES

Gallons Per Minute	Liters Per Second	Thickened Sludge % S.S.	Cent. Cake % T.S.	Recovery %	Polymer Utilization #/T	gm/kg
2.3	0.15	x	17.7	100	1.5	0.75
3.0	0.19	x	15.4	99	0.88	0.44
1.3	0.08	4.4	13.8	99	2.7	1.35
2.3	0.15	x	13.3	99	2.3	1.15
2.3	0.15	x	17.7	100	1.5	0.75
3.0	0.19	x	15.4	99	0.88	0.44
2.5	0.15	x	15.2	98	2.3	1.15
2.6	0.164	4.3	15.2	94	2.2	1.10

	OPTIMUM	
GPM	1.3 - 3.0	2.3 - 3.0
L/Sec	0.08 - 0.19	0.15 - 0.19
% S.S.	4.4	4.3
% T.S.	13.3 - 17.7	15.2 - 17.7
Rec.	99 - 100	94 - 100
#/T	0.9 - 2.7	0.9 - 2.3
gm/kg	0.45 - 1.35	0.45 - 1.15

A comparison of the three dewatering devices tested at the Hatfield facility under similar operating conditions indicated that the filter press produced a drier cake with greater recovery of solids fed than either the vacuum filter or the solid bowl centrifuge. The press operates on a batch process principal, as opposed to the continuous operation of the centrifuge and vacuum filter.

While these tests proved significant in determining the relative performance of each of the three types of mechanical dewatering devices on the same sludge, a total cost evaluation, including both amortized capital and operating costs, would be required before a final selection is made.

ASH ANALYSIS

The relationship between advanced wastewater treatment processes and the sludges that they produced were often neglected in the past. In the Hatfield Township operation, analyses were performed to determine the characteristics of the incinerator ash, primarily to ascertain what options were available to the Authority for final disposal, under current Pennsylvania regulations.

The analysis of June 14, 1973 (Table 40) indicates a high % volatiles. This is determined by weight loss, however, the weight loss was probably due to calcination of the $CaCO_3$ and loss of CO_2.

The leaching analysis (Table 41) indicates a stable, inert material, with a possibility of generating heavy metals in the leachate. This is a significant factor in limiting the final disposal to generally State approved sanitary landfills with adequate facilities to capture, and treat, leachates.

TABLE 40. ASH ANALYSIS - JUNE 1973

Filter Cake Sample - Taken June 14, 1973

Moisture - 67.6%

TS - 32.4%

Analysis	%
$Ca(OH)_2$	8
$Mg(OH)_2$	3
$CaCO_3$	30
Ca_xP_y	8
Inerts	11
Volatiles	43
	103

TABLE 41. CHEMICAL COMPOSITION OF INCINERATOR ASH FROM HATFIELD TOWNSHIP MUNICIPAL AUTHORITY AWT FACILITY

DECEMBER 1973

Mineral	% (1)	Lb. Mineral Per Ton Ash	Lb. Material Leached Per Ton Ash	% Material Leached (2)
Calcium	53.040	1060.8	256.20	13.81
Magnesium	29.465	589.3	1.00	0.05
Aluminum	9.000	180.0	1.00	0.05
Silicon	5.000	100.0	1.80	0.09
Baron	0.005	0.1	1.00	0.05
Copper	0.500	10.0	0.10	0.005
Iron	0.900	18.0	-	-
Lead	0.090	1.8	-	-
Phosphorus	0.900	18.0	-	-
Sodium	0.500	10.0	-	-
Tin	0.050	1.0	0.50	0.025
Titanium	0.500	10.0	-	-
Zinc	0.050	1.0	-	-
	100.50	2000.0	261.60	-

(1) Tests done by spectrophotometer and given in relative concentrations.

(2) In estimating percentages, maximum amounts were used on all parameters except calcium and magnesium. On these two, a ratio of 9:5 was presupposed and the percentages arrived at analytically.

Maximum amounts were used except for Tin (Sn) which was assumed to be partially leached out.

SLUDGE INCINERATION

The existing furnace is a 10'-9" I.D. x 5 Hearth (3.28 m I.D. x 5 Hearth) Multiple Hearth furnace rated for combusting 1,750 lb/hour of 60% V.S.S at 30% T.S. On this basis the furnace rate is 4.82 lb/ft^2/hour of wet cake. Present technology generally rates furnaces at 8.0 lb/ft^2 (39.06 kgs/sq m). Attempts to exceed 2,000 lb/hour, or 5.5 lb/ft^2/day, reduce furnace capacity and form clinkers, or lumps of improperly combusted sludges.

Furnace operational temperatures vary, but are generally maintained at 300° F on #1 Hearth, 600° F on #2 Hearth, 1,600-1,700° F on #3 Hearth, 1,300-1,400° F on #4 Hearth, and 700° F on #5 Hearth. There are adequate air pollution control devices to eliminate potential problems.

As indicated in Table 42, the average furnace loading has been 3.25 lbs/ft^2/hour (15.93 g/sq m/hour) of dry solids, or 12.43 lbs/ft^2/hour (60.91 g/sq m/hour) of wet sludge.

TABLE 42. SLUDGE LOADINGS

JANUARY - JUNE 1974

	WET SLUDGE		FILTER LOADINGS		INCINERATOR LOADINGS	
Month	10^3kg/month	tons/month	kg/m^2/hr	lb/ft^2/hr	kg/m^2/hr	lb/ft^2/hr
Jan.	559.1	616.3	112.6	25.1	34.4	7.05
Feb.	471.6	519.8	114.3	23.4	42.9	8.78
March	1,016.2	1,120.2	149.4	30.6	62.6	12.81
April	894.0	985.5	148.4	30.4	63.2	12.94
May	967.2	1,066.2	167.0	34.2	71.0	14.54
June	1,264.3	1,393.6	202.6	41.5	86.2	17.65
AVG.	768.9	950.3	150.9	30.9	60.1	12.30

SECTION VIII

CONSTRUCTION OF WASTE TREATMENT PROJECTS

GENERAL

The construction of both the 1965 and 1970 projects were carried out in the manner which has prevailed for decades in the United States. In the United States, municipal construction work has always been awarded on the basis of competitive bidding, and, in the State of Pennsylvania, has always required that separate bidding occur for the prime contract functions of general construction, plumbing construction, heating construction, and electrical construction. The obvious advantage of competitive bidding under one entire construction program, with one designated contractor, is not currently permitted in the State of Pennsylvania under existing laws.

Turnkey construction, that is, development, design, construction, and operation, has not been, except in very isolated instances, utilized in municipal work in the United States. This situation, utilized quite widely throughout other parts of the world, has, in many cases, distinct advantages over the method employed in the United States, but resistance is still very high to this approach, and even though the Federal regulatory agencies allow turnkey applications, it appears that it may be some years before a significant movement in this direction can be achieved.

The 1970 construction project differed in many respects from the 1965 construction project in the approach as utilized by the contractors. The 1970 construction project was far more complicated, in that the units and processes were far more sophisticated, and also the 1970 project had to be constructed around the existing facility and the existing facility had to be maintained, insofar as possible, at its full treatment capability during the entire construction period. As indicated previously, the 1970 construction required the integration of the units which were in service in 1970, primarily because of effluent requirements, and because the previous construction had occurred only five years earlier. Experience with this project, and other projects where remodelling of existing facilities has been undertaken, has led to a general conclusion that if the value of any existing unit is even the slightest bit questionable, it should be abandoned rather than to spend money to integrate it into a new facility. This general conclusion has severe limitations, but the experience at Hatfield Township would indicate it might well have been far better to construct the 1970 project adjacent to the existing facility, and then to abandon the existing facility upon completion of the new plant. The problems associated with working around existing piping and interconnecting existing units with new units, undoubtedly raised the cost of construction considerably, and there is a strong question whether there was really any savings in total dollars by utilizing the existing facilities.

1965 CONSTRUCTION

The 1965 construction was generally straight-forward. There were no existing utilities on the plant site, and the contractor was free to undertake the work without any restrictions whatsoever.

The total time of construction of the 1965 project was fifteen months, and the work proceeded with only minor difficulties. In 1965, the availability of equipment was well established, and the time period which elapsed from the placing of an equipment order to the delivery on site was quite reasonable.

The general contractor, that is the contractor who did all of the building construction as well as the mechanical installations, was responsible for coordinating all of the various trades at the project site. There were some minor difficulties with scheduling the plumbing and heating and electrical construction concurrently with the general construction, but by and large, the project proceeded in good harmony. As was quite common in the middle 1960s, the general contractor was competent in all aspects of the construction, and did little, if any, sub-contracting to specialized trades.

In most respects, the 1965 construction project was typical of that era, and caused the Hatfield Township Municipal Authority a minimum of problems.

1970 CONSTRUCTION

The contrast between the 1970 construction and that which had occurred previously in 1965 was marked. Among other things, the trend of construction by 1970 had undergone a change in that many of the older contractors who specialized in waste treatment projects, were no longer capable of bidding larger sized projects, and an entirely new grouping of contractors were entering the scene, many with little or no experience in the waste treatment plant construction area.

Another interesting aspect of the 1970 construction, and one which was not untypical at that time, nor at the present time, was the tendency for contractors to act as a broker, that is, to sub-contract virtually every specialized trade, and perform little, if any, services with their own personnel. As in 1965, the general contractor was responsible for coordination of all of the trades, but with the tendency of the general contractor and the other specialized contractors to sub-contract much of their work, the problems of coordination among the trades became, at times, severe.

The problem of providing construction around, and connecting to, existing facilities has been mentioned, but it provided some particularly difficult periods during the construction of the 1970 project. Prior to the construction, detailed operational procedures were developed to maintain the operation of the existing facility, but due to the method of construction undertaken by the contractor, it was found impossible to maintain full operational service at all times, and the service was interrupted on a number of occasions, sometimes for periods as long as several days. This caused a number of problems to occur with the State regulatory agency, which in turn, by reason of their concern, compounded the construction problems in some instances.

One of the striking factors, which became apparent during the 1970 construction, was that the availability of equipment and materials was far less than that which had occurred in prior years. Delivery times on even minor items of equipment were considerably longer, and this situation is now at the present time, even more acute. The construction period for the 1970 project was twenty-two and a half months, and it is currently estimated that if the 1970 project were to be placed into the construction phase at the present time, that the time of construction would probably extend to twenty-eight or thirty months, occasioned almost entirely by delays in procuring equipment and materials. This, much longer time for construction, translates to a higher cost for construction because all public bidding work in the United States must utilize minimum wage rates as published by either the U. S. Department of Labor or the local State Department of Labor. Normally these minimum wage rates have yearly escalation clauses, hence, the longer a project consumes the higher the labor costs will be at various stages throughout the project.

One requirement placed upon the contractor in the 1970 project, which was far greater than that which had been required in 1965, was the provision for detailed and extended instruction in the operation and maintenance of individual items of equipment. It was the contractor's responsibility to provide technical instruction from the equipment manufacturer on the job site, in order that the operators could become familiar with all facets of each item of major equipment before it was placed in actual operation.

Actual practice has indicated that this requirement is absolutely essential in advanced waste treatment construction, and further, that it would probably be well to require a minimum of two detailed technical instructions sessions, with perhaps a six month interval.

SECTION IX

PROJECT COSTS

GENERAL

In the estimating the cost of waste treatment, it has become quite popular to attempt to reduce, for comparative purposes, the cost of construction of specific types of treatment processes to a cost per gallon of capacity provided. These figures, then, are often mistakenly utilized by governmental agencies, and local authorities, in determining a general course of action to follow, completely neglecting the individual characteristics which should and must ultimately influence the decision in any specific case.

If a development of a cost per gallon treated can be developed with a full understanding of the major factors which contributed to that particular cost figure, then the value, or values, become more meaningful in cost projections for other projects with similar overall characteristics.

The reader is cautioned, that in the data which is developed in this chapter, the ultimate values are greatly affected by the following:

1. The three construction programs: the 1965 project, the 1970 project, and a 1975-76 project, have a construction interval of five years, and that the value of the dollars spent at each period of construction varies considerably.

2. The 1970 project represented a radical departure from the then prevailing technical thoughts with respect to waste treatment, and greatly altered the types of units required to meet the increased effluent criteria. The 1965 construction was integrated into the 1970 project by converting these 1965 units into other uses, with the exception of the aeration basin and the solids handling facilities. In 1975-76 construction will, however, supplement the 1970 construction rather than eliminate portions of the process or modify them to other uses.

3. If there•had been no 1965 project, the construction of the 1970 project could have been more efficiently accomplished in terms of dollars, inasmuch as there would have been no need to construct around operating units.

4. In the construction of the 1970 project, it was decided to provide extra capacity in those units not readily expandable in the future so that succeeding hydraulic expansions would not require additional units in these areas. The

ultimate Hatfield facility, as it may exist beyond 1976, will represent stepped or phased construction, with varying degrees of construction in each step or phsse, and will not represent a straight-line relationship as to cost of the facilities provided per gallon of capacity.

5. In projecting costs for the 1975-76 construction, an escalation factor has been included, but it is impossible to predict with even a moderate degree of accuracy, the conditions that will prevail at that time.

1965 COSTS

The 1965 costs were developed previously in this report and shown in Table 1. The total 1965 treatment plant construction cost was $848,767. This sum of money was expended to provide a rational hydraulic capacity of 1.3 mgd, and a solids handling capability for a flow of 3.6 mgd.

The 1965 construction consisted of a secondary treatment plant, providing primary clarification, aeration, secondary settling, and chlorination, with sludge thickening, vacuum filtration and multiple hearth incineration of the sewage solids. There was, in addition, provided as part of the sludge control building, office space, meeting room space, and garage and maintenance facilities.

The unit cost of the 1965 project was $0.65/gallon of treatment plant hydraulic capacity.

It was anticipated in 1965 that subsequent hydraulic expansions would increase the plant capacity of 3.6 mgd, and that the subsequent hydraulic expansion, when required, would probably necessitate the expenditure of an additional $850,000, with the exact figure dependent upon the inflationary situation which prevailed at the time of the expansion.

Assuming that this reasoning would have followed, the total cost of expanding to a 3.6 mgd secondary treatment plant, with incineration of the sewage solids, would have been approximately $1,700,000, or, on the basis of capacity, $0.47/gallon of hydraulic capacity provided.

1970 COSTS

The construction cost for the 1970 project is summarized in Table 43.

The total construction costs expended for both the 1965 and the 1970 projects, at unadjusted dollars actually spent, was $4,789,197.

On the basis of dollars actually spent, unadjusted for inflation, the Hatfield Township advanced waste treatment facility at the theoretical design capacity of 3.6 mgd, cost $1.33/gallon of hydraulic capacity provided. On the basis of a 5.0 mgd rational design capacity, that is the capacity to which the plant could ultimately be loaded and still, theoretically, maintain a comparable effluent quality, the actual dollar cost was $0.95/gallon of hydraulic capacity provided.

TABLE 43. HATFIELD TOWNSHIP MUNICIPAL AUTHORITY

CONSTRUCTION COSTS - 1970 PROJECT

ADVANCED WASTE TREATMENT FACILITY

General Construction	$3,538,603
Plumbing Construction	72,958
Electrical Construction	328,859
TOTAL 1970 TREATMENT PLANT CONSTRUCTION COST	$3,940,420
Total 1965 and 1970 Treatment Plant Construction Cost at Unadjusted Dollars	$4,789,187
Total 1965 and 1970 Treatment Plant Construction Cost at June 1973 Adjusted Dollars (Based on ENR Indicies)	$6,028,805

3.6 mgd Theoretical Capacity

5.0 mgd Rational Capacity

On a rational design basis, and considering both the 1965 and the 1970 projects, one conclusion might be drawn, subject to the limitations described in the previous section, that advanced waste treatment in the size range of 3.5 mgd to 5.0 mgd capacity, costs approximately twice as much as secondary treatment per gallon of hydraulic capacity provided. This relates to conventional secondary treatment, as opposed to advanced waste treatment consisting of primary chemical precipitation-complete mix aeration-tertiary chemical precipitation followed by tertiary filtration. The reader is cautioned that this very general conclusion should not be utilized for any other process comparison, such as secondary treatment with physical-chemical advanced waste treatment, or secondary treatment with advanced waste treatment where the total advanced process is concentrated in a tertiary step.

The most recent estimate for expansion of the hydraulic component and the solids handling component at the Hatfield Township AWT facility was made in June 1973.

As of June 1973, assuming there had been no prior construction, and the total facilities now present were constructed with dollars of the 1973 value, the value of the work which now exists is estimated at $6,028,805. On the basis of a 5.0 mgd (18,925 cum/day) rational capacity, this figure translates to $1.20/gallon ($0.317/liter) of capacity provided.

Comparison of this figure, adjusted to June 1973 dollar value, as opposed to actual dollars spent, indicates an increase of $0.30/gallon ($0.079/liter) of capacity provided, and represents the inflationary increase between 1965, 1970 and 1973. This fact would emphasize the extremely limited value of utilizing a cost/gallon of capacity for projecting future costs of similar type installations.

FUTURE COSTS

Estimated construction costs for the next expansion program at the Hatfield Township AWT facility, assumed at providing a total hydraulic capacity of 8.0 mgd (30,280 cum/day) on a two-shift operation basis per day, or an ultimate solids handling capacity of 16.0 mgd (60,560 cum/day) are summarized in Table 44.

The total estimated construction cost, if the program is undertaken prior to 1976 is $5,024,500.

The total value of the 1965, 1970 and future treatment plant construction costs, at actual dollars spent or estimated, and unadjusted, is $9,823,687. Based upon the theoretical design capacity of 7.5 mgd (28,388 cum/day), it is estimated that the total cost will be $1.30/gallon ($0.343/liter) of capacity provided. On the basis of a rational design capacity for the future expansion of 10.5 mgd (39,743 cum/day), the cost is estimated at $0.93/gallon ($0.245/liter) capacity provided.

The figures in Table 45 compare closely to those estimates provided for the existing facility at a 3.6 mgd (13,626 cum/day) theoretical capacity, and a 5.0 mgd (18,925 cum/day) rational capacity, and they include the inflationary costs anticipated to occur over the next fifteen to eighteen months.

If there had been no 1965 and 1970 construction, but the entire project was to be constructed within the next fifteen to eighteen months, the total estimated cost at 1973 adjusted dollars would be $15,842,500. This translates to a cost per gallon of rational capacity of 10.5 mgd (39,743 cum/day) of $1.50/gallon ($0.396/liter) of capacity provided.

It is suggested that if the reader desires to draw conclusions as to cost for the type of process utilized at Hatfield Township, on the basis of dollars/gallon capacity provided, that as an initial starting figure he consider a value of $1.80/gallon ($0.476/liter) to $2.00/gallon ($0.528/liter) of capacity provided. This figure, if utilized at all, should only be in the very initial analyses of the potential costs for a facility of this type, and should be refined in further detail as soon as sufficient information is available to permit a revision.

TABLE 44. HATFIELD TOWNSHIP MUNICIPAL AUTHORITY

ESTIMATED CONSTRUCTION COSTS - 1973 BASE

SOLIDS HANDLING AND HYDRAULIC EXPANSION

General Construction - Hudraulic	$1,878,200
General Construction - Solids Handling	2,476,300
Plumbing Construction	95,000
Electrical Construction	575,000
TOTAL ESTIMATED FUTURE CONSTRUCTION COSTS	$5,024,500
Total 1965, 1970 and Future Treatment Plant Construction Cost at Unadjusted Dollars	$9,813,687
Total 1965, 1970 and Future Treatment Plant Construction Cost at June 1973 Adjusted Dollars (Based on ENR Indices)	$15,842,500

7.5 mgd Theoretical Capacity

10.5 mgd Rational Capacity

SECTION X

FUTURE REQUIREMENTS

GENERAL

At the present time, the Hatfield Township advanced waste treatment facility is precessing the waste flows from Hatfield Township and a portion of a neighboring community, Montgomery Township. These two communities are operating under an agreement established first in 1965, and modified in 1970 with the initiation of that project. In the 1970 project, provision was made for acceptance of waste flows from portions of other communities which lie in the drainage area of the AWT facility.

Since the 1970 project, new regulations have been published by the Environmental Protection Agency on a Federal level, and the Pennsylvania Department of Environmental Resources, which require regional planning for not only wastewater treatment, but also for wastewater management. These discussions in the Upper Neshaminy area are currently nearing their final stages, and it appears that the Hatfield Township facility will have to be expanded to accommodate at least two, and perhaps as many as four, of the adjacent communities. This fact, together with the growth patterns prevailing in both Hatfield Township and Montgomery Township, will dictate an early expansion of existing facilities.

Even prior to this planning, however, the need for expansion of the solids handling portion of the AWT facility has been known. In the design of the 1970 Hatfield Township advanced waste treatment project, only one addition was made to the existing solids handling system, and that was the construction of a new sludge thickener to receive the primary waste activated and tertiary waste sludge volumes.

At the time of the design of the 1970 project, the exact characteristics of the future chemical sludges was now known. Parameters for this sludge concentration were approximated in bench scale laboratory studies, but the exact character of these waste sludges ultimately had to be dependent upon the actual chemical feeds and their combinations once actual operation commenced.

The new sludge thickener was designed with a 40' diameter and a 10' side water depth (12.2 m diameter x 3 m sidewater depth), and it was estimated that at a 3.6 mgd (13.63 cum/day) flow this unit would handle a sludge load totalling 12,960 lbs/day (5,891 kg/day). This estimated sludge load was composed of 6,000 lbs/day of primary sludge, 4,800 lbs/day of lime sludge, and 2,160 lbs/day of waste activated sludge. At the time of the design, the use of alum at 40 mg/l was anticipated, or perhaps even a lesser amount, for polishing purposes prior to the tertiary filtration step, and the sludge generated from this volume of chemical use was neglected in the total sludge volume calculation. The design of the thickener assumed a feed concentration of 3,500 mg/1, and a 700 gpd/ft^2 overflow rate. The thickener overflow was assumed to be composed of 310 gpm of thickener flow, plus 300 gpm of dilution water.

The existing dewatering facilities consisted of two vacuum filters of a rotary type, with plastic surfaces, each having a capacity of 450 pounds of dry solids per hour (204.5 kg d.s./hour).

The existing sludge incinerator from the 1965 project was a multiple hearth furnace with a rated capacity of 1,900 pounds of filtered sludge cake per hour (863.6 kg/hour).

The existing sludge dewatering and sludge incineration facilities were, obviously, inadequate to handle the anticipated sludge to be developed at the design average flow of 3.6 mgd, and in 1969, the estimate was made that the ultimate limit of these items of solids handling would be reached at a hydraulic flow in the range of 2.5 mgd (9,462 m^3/day). Due to economic limitations imposed by the local governing authorities in 1969 and 1970, it was decided to defer any additions to the solids handling system until the operation began to approach 140 hours per week, or 20 hours per day. This decision, in retrospect, was not completely sound because the character of the sludge developed has resulted in nearly continuous operation at flows of 2.0 mgd or less. The present solids handling system is the weakest portion of the total Hatfield Township AWT operation, and is in need of immediate expansion. Initial studies have already been developed for the expansion of these facilities.

A section plan of the one addition to the solids handling units provided in the 1970 project, the new sludge thickener, is contained in Figure 36.

The sludge handling program, a hold-over from the original 1965 project, consists of two 4' x 4' (1.22m x 1.22m) Drum-Type Vacuum Filters, with chemical conditioning, and a 10' x 9" O.D. (3.28 m) Multiple Hearth Furnace. The system operates at capacity on a 24-hour, 7 day per week schedule. See Figure 37 for a diagram of the system. The expansion of the facility to include advanced waste treatment deferred the expansion of the sludge handling facilities until enough information could be generated to design an adequate system. Additional information on sludge handling appears in later chapters.

The need for the expansion of the solids handling facilities has become so critical that it appears, unfortunately, that a crash program must be undertaken to find some alternate temporary means of disposal of accumulated sludge volumes in advance of any new construction, in order to relieve the load placed on the existing multiple hearth incinerator. The problem appears to be most critical during the summer months when the solids in the sludge thickener tend to initiate a condition of septicity, and thus become less easily dewatered. This reduces the solids content of the vacuum filtered sludge, which results in the incinerator burning larger volumes of sludge with a smaller dry solids content. During the spring of 1973, mixed liquid sludge was removed by tank truck

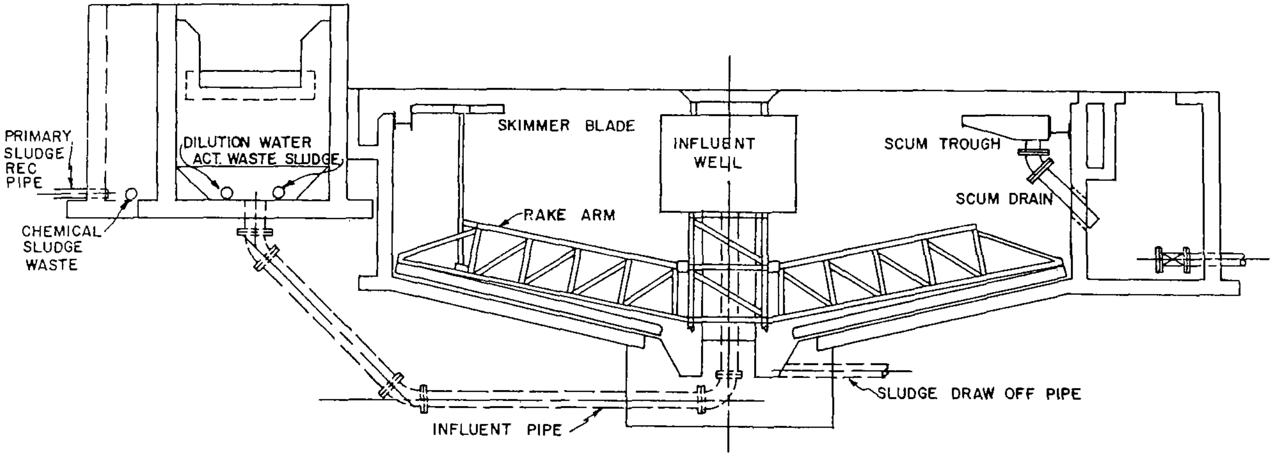


Figure 36. Sludge Thickener Tank

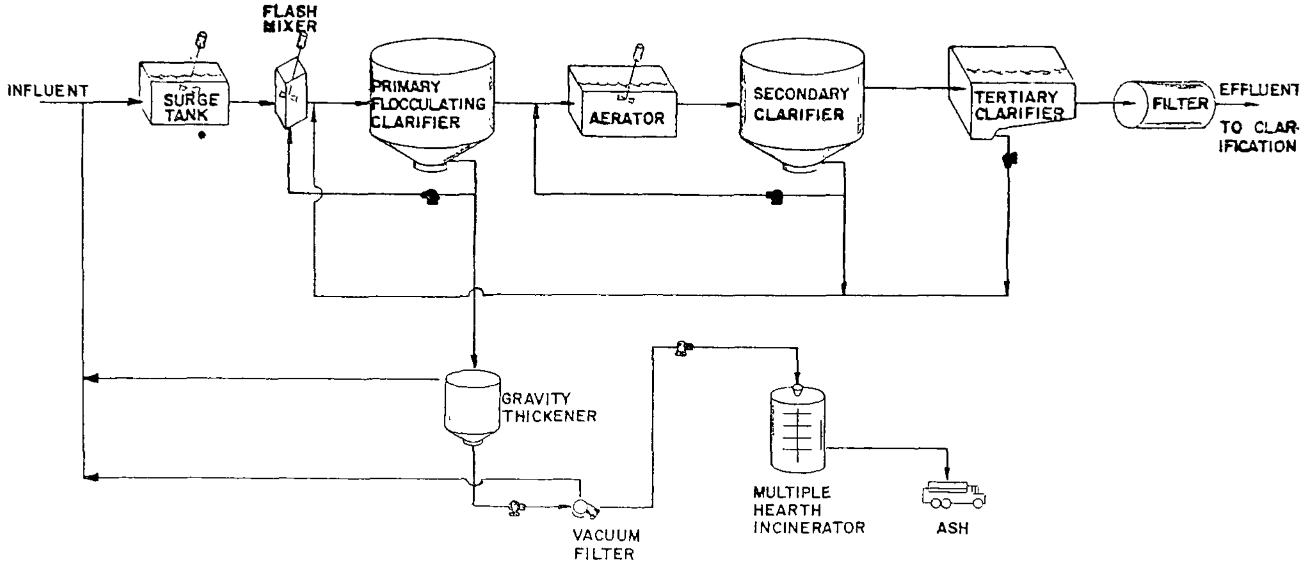


Figure 37. Existing Sludge disposal system - 1974

on a fairly constant basis, for a two and a half month period, for land disposal. Investigations are under way to locate a source where vacuum filtered sludge can be disposed of in a land-fill operation, which would be far more efficient than removing the sludge by tank truck in a liquid form.

This thickener condition also affects hydraulic performance by directing a deteriorated overflow of greater volume to the raw waste pump station, thus building up the inventory of sludge in the recycle process.

SOLIDS HANDLING

This obvious need for additional solids handling facilities prompted the Hatfield Township Municipal Authority to authorize the developement of a report on solids handling expansion, early in 1973. This report was completed and submitted to the Authority in June 1973. This report explored in depth ten alternative programs for solids handling expansion, based upon various combinations of treating chemical primary sludge, waste activated sludge and tertiary chemical sludge, and considering the possibility for the various programs of recalcining the lime for reuse in the process.

As part of the development of this solids handling report, the total weight of dry solids generated per day per million gallons was developed for both the conventional treatment facility, as constructed under the 1965 project, and for AWT sludges, as produced in the current operation. The calculation of these sludge volumes is given Appendix F of this study, and a summary of the sludge production is shown in Table 29.

The summary of sludge production, as shown in Table 29, indicates the sludge produced on a dry weight basis. It was recommended in the report to the Authority that the design of the solids handling expansion consider not only total anticipated future sludge production, but also the optimum number of hours per week of operation. With 75% Federal funding, the operating costs may well be in excess of the amortized community share of capital costs. Hence, the optimization of operating costs sometimes becomes the dominant economic criteria in process selection.

It may be noted from Table 29 that the conventional sludge generated from a secondary treatment facility is approximately 1/3 of that which is generated per million gallons of flow from an AWT facility utilizing the Hatfield Township process. The reader is cautioned, in reviewing this data, that the generation of the AWT sludges is dependent on a number of variables, and will not be identical for each separate facility which may utilize the Hatfield type process. For instance, the amount of calcium carbonate and other calcium sludges generated will be dependent upon the amount of lime necessary to achieve a pH of 9.5, which in turn, is dependent upon the alkalinity of the raw wastewater. In a similar manner, the generation of alum sludges is a function of the amount of alum necessary to achieve proper coagulation, which in turn, is in part a function of the pH of the secondary effluent as it enters the tertiary process.

It may be generally said, however, that the ratio of AWT sludge to conventional secondary treatment sludge will be in an approximate ratio of 3.5-4:1 for a facility utilizing primary lime precipitation, activated sludge, and tertiary alum precipitation followed by filtration.

In any waste treatment facility where large amounts of lime are used in either the primary or tertiary phase, consideration must be given to the possibility of recalcination of this lime. When lime is used, calcium carbonate and calcium hydroxyapatite are the major sludges produced. The hydroxyapatite is fairly stable, but the calcium carbonate can be recalcined to recover lime. It has been reported by various sources that there is no change in the recovered lime capability for phosphorus removal, but the make-up demand will vary considerably. It has been reported as being as low as 13%, or as high as an average of 25% to 35%. The classification of the sludge in order to separate $CaCO_3$ from inerts in a classifying centrifuge, or the air classification of this material, is a determining factor in the efficiency of the process.

A decision on recalcination of lime is essentially an economic study considering the cost of lime, the capital costs necessary to achieve the recalcination, the additional operating costs necessary to achieve the recalcination, and the costs of disposing of the incinerated ash. The latter item is one which is very often not considered in such an economic analysis, but the reuse of 65% to 85% of the lime can reduce significantly the volume of ash generated from the incineration process, and, if the costs associated with this final ash disposal are significant, it can have a marked effect upon the total economic picture.

In the solids handling report, analyses of ten separate programs were made. The recommendation to the Authority was the adoption of a program, the flow diagram for which is contained in Figure 38. The initial recommendation included incineration of primary sludges and the secondary-tertiary sludges in dual train incinerators. It also included provision for recalcination of the primary lime sludges, and utilization of two stage centrification. Reference to Figure 38 will indicate that the primary lime sludges would be proposed to be thickened in a gravity thickener, and would then be passed through the first stage centrifuge. The cake from the first stage centrifuge would be directed to the multiple hearth incinerator, which would be heated to a recalcining temperature. The material from the incineration of the primary sludge would be sent to a classifier, where the recovered lime would be conveyed to the bulk lime storage silo, and the rejects from the ash classifier would be combined with the ash from the secondary furnace and hauled to a landfill. The centrate from the first stage centrification of the primary sludge would be submitted to a second stage centrifuge for dewatering, and the cake would be discharged directly to the top part of the WAS-tertiary multiple hearth furnace.

The WAS-tertiary sludges, being much harder to dewater, would be subjected to flotation or mechanical thickening, then heat treatment and decantation, and finally centrification and incineration.

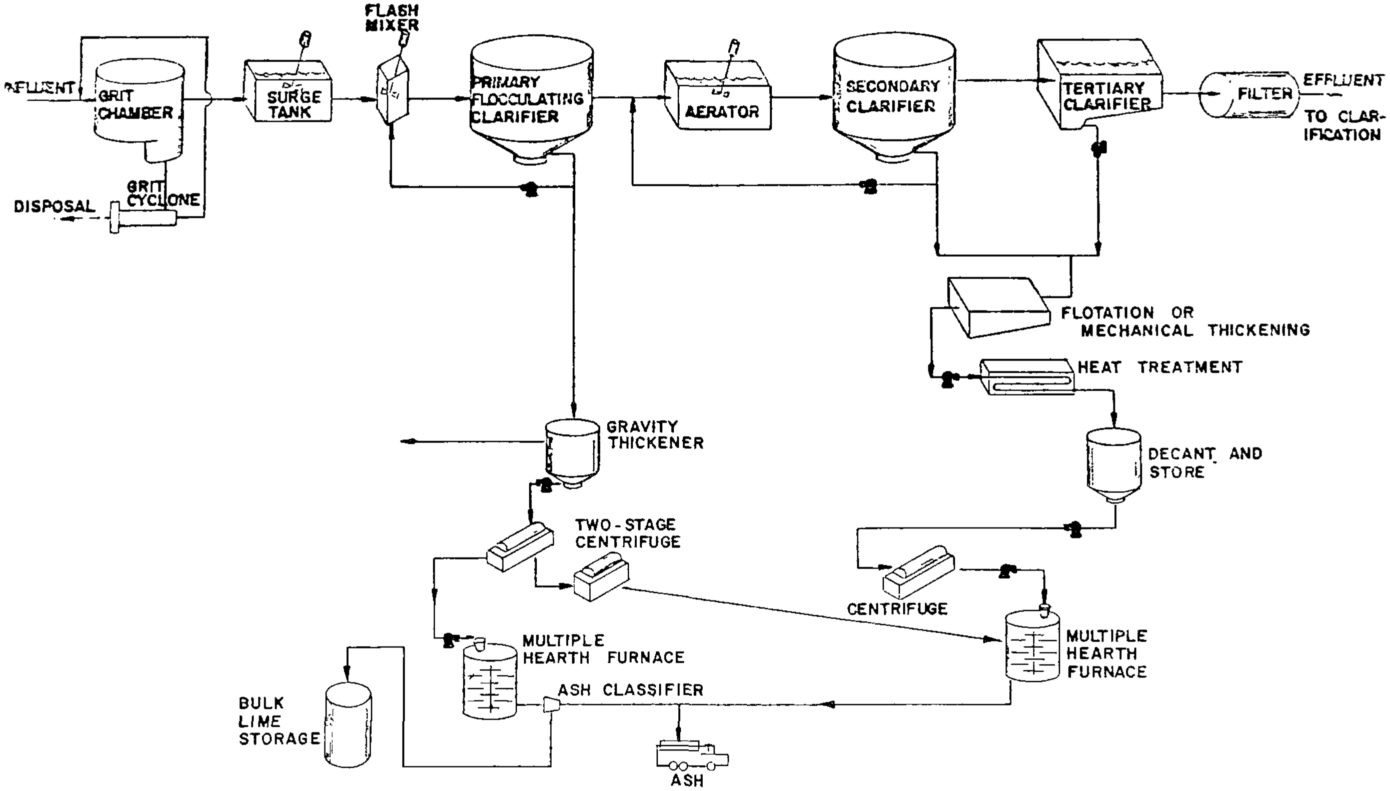


Figure 38. Diagram of Recommended Sludge Disposal System - 1973

This recommendation to the Authority was deemed to be the most suitable, long-range method of providing for complete solids handling, utilizing the most efficient process combinations for each type of sludge. The Authority, however, deemed that recalcination of the lime could result in operating problems, and further, felt that the utilization of heat treatment in the secondary-tertiary sludge processing would create objectionable odors and other operating problems, and they indicated their desire to adopt a different program.

The program which the Authority finally selected is shown in flow diagram form in Figure 39. This program utilizes the separate treatment of the primary and the secondary-tertiary sludges, and eliminates the recalcination step, as well as the heat treatment of the secondary-tertiary sludges. A summary of the solids handling units required for this Program VII is contained in Table 45.

TABLE 45. SUMMARY OF SOLIDS HANDLING UNITS, PROGRAM VII

1 - Gravity Thickener	65' Ø x 10' swd (19.8 m Ø x 3 m swd)
2 - Centrifuges	10' x 12' (3 m x 3.65 m)
2 - Multiple-Hearth Furnaces	1 - 22.25' x 10 Hearth (6.8 m) 1 - 16.75' x 7 Hearth (5.1 m)
1 - Flotation Unit	25' Ø x 10' swd (7.6 m Ø x 3 m swd)
1 - Blend-Mix Unit	25' Ø x 10' swd (7.6 m Ø x 3 m swd)

The Authority decided that, in lieu of providing two separate multiple hearth furnaces with different diameters and hearth arrangements, two furnaces would be provided, each of identical size, and each of the largest capacity required. This will be done in order that a malfunction of one of the furnaces will permit utilization of the remaining furnace for combined incineration of the total sludge on an emergency or interim basis.

The solids dewatering facilities investigated were based upon a 12-hour/day, 5 days/week operation, plus an additional 35-hour/week of start-up and take-down time, including maintenance, for a total operation of approximately 95-hours/week at 8.0 mgd flow. This appears to be an optimum balance between capital costs in sizing units, and operating costs. This would involve, at 8.0 mgd, essentially a two shift per day operation, and the facility would have an ultimate capacity of 16.0 mgd, with 120-hours/week of actual burning time, plus 35-hours/week of start-up and shut-down. In Figure 40 a graph is presented indicating total annual costs versus million gallons per day treated. This figure is presented as a means of indicating the requirements for operation at various flows. For instance, up to 5.2 mgd would require only one shift

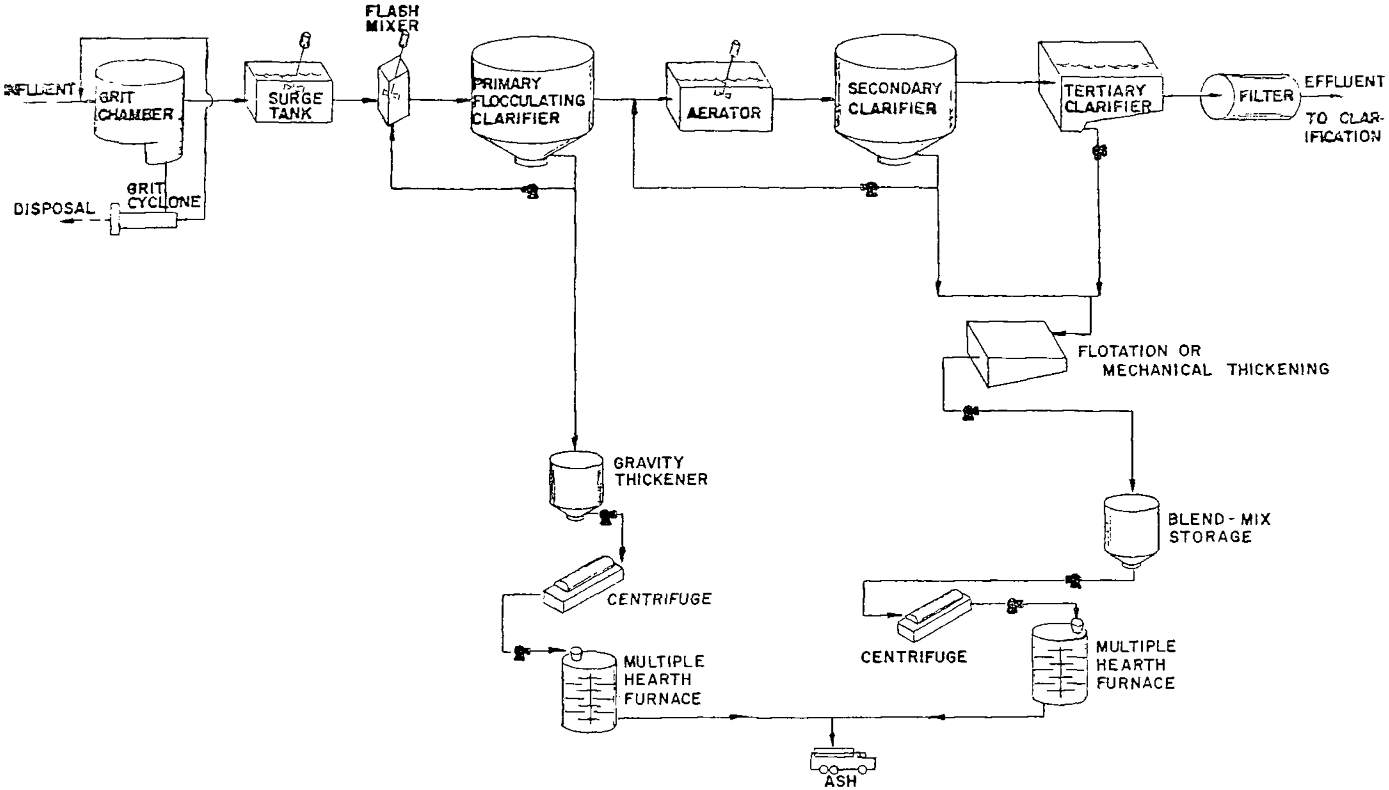


Figure 39. Diagram of Sludge Disposal System Under Design-1974

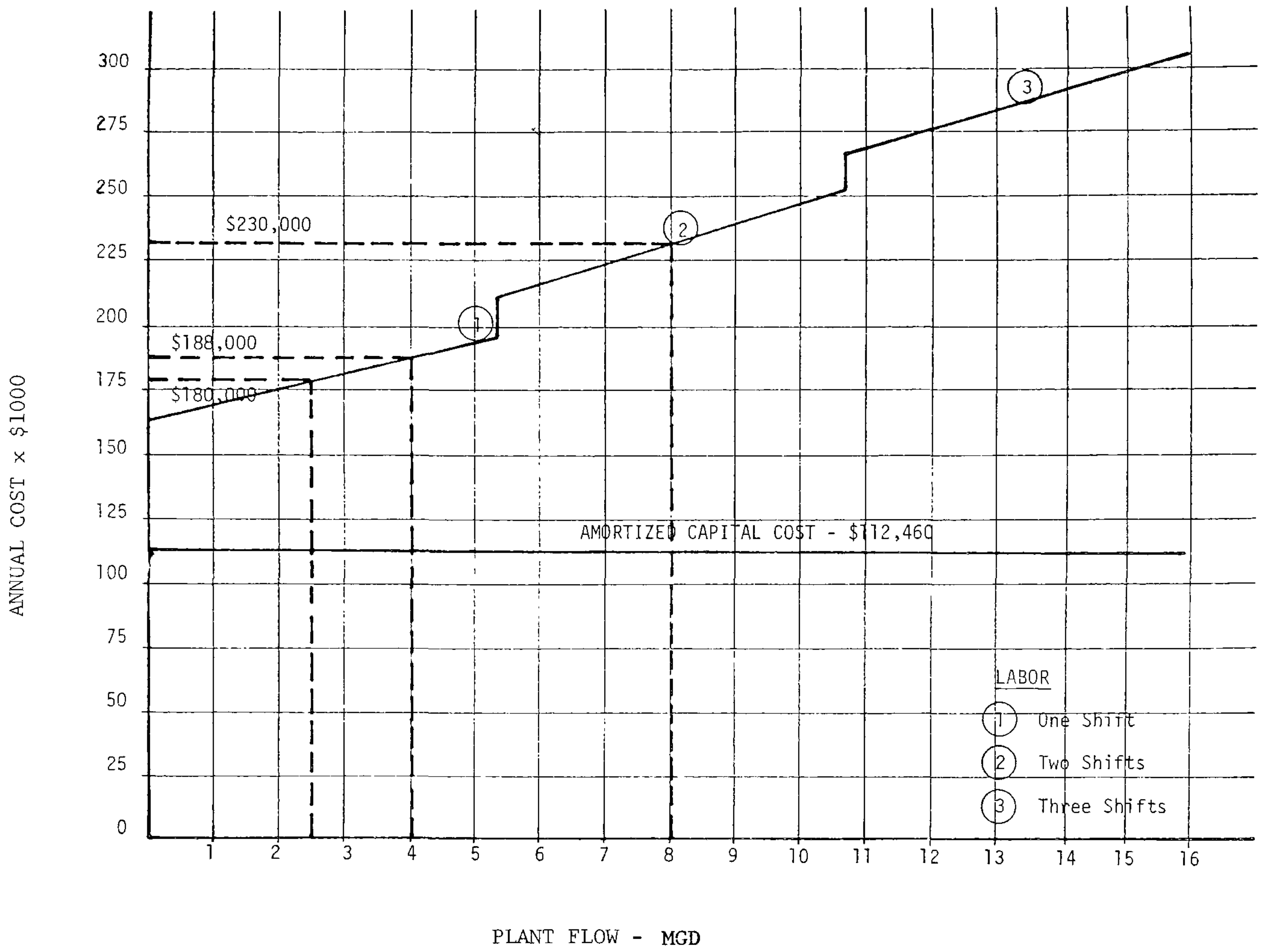


Figure 40. Annual Cost of Sludge Processing

operation each day, and two shifts would be required between approximately 5.3 and 10.7 mgd.

Thus, the proposed solids handling expansion can accommodate flows in excess of 10.0 mgd with a two shift operation, which would appear to be the most optimum balance between capital and operating costs. With the Federal participation of the United States at a 75% level of construction costs, the desirability of providing sufficient future capacity in this expansion for the ultimate foreseeable development of the entire drainage area, appears warranted. Once the solids handling capacity is provided, hydraulic expansions can take place at a pace required by growth in the area at a greatly reduced cost per additional capacity provided.

HYDRAULIC EXPANSION

The future hydraulic expansion of the existing Hatfield Township advanced waste treatment facility is subject to the finalization of the regional wastewater treatment program now being discussed among the various communities and the State government. If all of the communities tributary to the advanced waste treatment facility are included, an expansion program of an additional 8.5 mgd (32,172 m^3/day) to a total capacity of 12.5 mgd will be required. It appears more likely, however, that the required expansion will more nearly approximate 3.65 mgd (13,815 m^3/day) to a total hydraulic capacity of about 7.5 mgd (28,388 m^3/day).

Based upon the expansion of the facility to approximately 7.5 mgd, it is anticipated that the total program will include the addition of the following units.

1. Screening and grit removal ahead of the main raw sewage pumping stations.

2. The addition of a third raw sewage pump in Pump Station No. 2.

3. Possible additional flow equalization, dependent upon the success of removing inflow/infiltration from all of the collection systems utilizing the facility.

4. Duplicate additional primary clari-flocculators.

5. Additional primary sludge pumping facilities.

6. Duplicate additional aeration tanks.

7. Duplicate additional secondary clarifiers.

8. Additional back-wash storage facilities, and back-wash pumps.

9. Additional chlorination facilities.

In addition to these units, it is anticipated that the expansion program would include the installation of an automated biological control system to control the mixed liquor solids content in the aeration basins and to control the return of activated sludge and the wasting of activated sludge. It is also anticipated that this program would include the addition of more sophisticated laboratory equipment, probably including a Technicon analyzer, and the addition of an electrical consol to centralize, at the very least, a monitoring operation in a new administrative and office building.

Consideration is being given to the possibility of refining the operation by the utilization of more automatic controls, and possibly computers, in an effort to reduce the labor costs and provide more precise control of the process. This, however, appears to be a situation that will not practically occur within this next expansion program. At a recent I.A.W.P.R. workshop, held in London in September 1974, on the very subject of automation of wastewater treatment facilities, it was evident that there are significant problems in the total automation of a facility, occasioned primarily by the lack of either accurate enough, or in some cases, any sensor for a particular controlling function. The present thinking with respect to expansion of the Hatfield Township advanced waste treatment facility is to provide in the next expansion as much automation as is practical.

ADDITIONAL EFFLUENT REQUIREMENTS

The current limiting effluent quality characteristics, as originally published by the State government in 1967, require a degree of treatment in excess of 99% efficiency. The State government has now imposed a limiting criteria on the nitrogen component discharged into the receiving stream. It is a maximum total nitrogen content of 8.0 mg/1.

In the planning for the next hydraulic expansion, emphasis will be placed on the nitrogen removal with further investigation of the ability to nitrify in the aeration basins, with the tentative thought that denitrification would be accomplished by introducing a carbon source into the influent to the tertiary pressure filter system. Work has been done in other areas on denitrification by the addition of methanol to the flow stream, just ahead of the filtration step, and from the data reported thusfar, it appears that this may be a suitable means of achieving total denitrification. The utilization of such a step would fit the Hatfield flowsheet with very minor modifications.

One major factor to be reckoned with is the high cost of methanol (CH_3OH) at $0.37/gallon (August 1974). Various other alternatives are being investigated, none of which are completely satisfactory from an economic standpoint.

Beyond the addition of a limiting nitrogen effluent criteria, there does not appear to be on the horizon any other effluent limitation which might be imposed short of the prohibition of any discharge at all directly to the surface waters. This is quite commonly referred to as "zero" discharge, and there is considerable discussion being generated as to the merits of this program. Certainly, the characteristics of the effluent being produced currently at the Hatfield Township waste treatment facility are more than sufficient for the utilization of this effluent for industrial process purposes, for cooling towers, and, in areas where public water supplies are not available, this water could be utilized in a separate fire protection system.

SECTION XI

CRITICAL EVALUATION OF HATFIELD TOWNSHIP ADVANCED WASTE TREATMENT FACILITY

GENERAL

The Hatfield Township treatment facility has been operational for fourteen months. The initial operation of the plant benefitted by the existence of an Environmental Protection Agency sponsored Demonstration Grant, which was intended to provide funds for the analysis of operations to meet certain design parameters. One of the fringe benefits of the grant was the presence at the plant of an engineer, who was able to aid in operation and at the same time, critique the design of the plant.

As is always the case, in addition to this critique by the engineer, operations personnel have offered many suggestions and criticisms, which would enhance the operational practices. These latter suggestions most often pertain to operator ease, and operator safety, but in some cases, provided insight into process modifications which would be beneficial.

Finally, the consulting engineer recognized a number of situations that required, or still require, modification to acheive an optimum blend of excellent operation, coupled with maximum operator convenience and safety, and reasonable capital and operating costs.

PRELIMINARY TREATMENT

All flows entering the Hatfield Township facility are comminuted. The liquid passes through moving screens, which chop up the rags and other large solid matter to prevent clogging of the pumps. There is no grit removal, since there was no evidence of grit prior to the design of the expanded facility.

One for the first tasks accomplished by the general contractor in the 1970 construction project, was the elimination of plant by-passes. In the months that followed, it became evident that there was a good deal of infiltration into the collection system during periods of heavy rainfall. Flows which previously had been by-passed into the Neshaminy Creek were allowed to enter the plant for treatment. These peak flows also tended to scour the sewer lines and carried with them large volumes of grit. The grit eventually settled-out on the floor of the surge storage tanks, and periodically must be removed by plant operations personnel.

Even when the comminutors are working, fibrous material seems to extrude itself through the comminution devices. Chopped up rags then mix with this fiber material and reweave themselves into mats, resulting in a maintenance problem. These mats serve as a point for solids to collect, and will periodically clog pipes and pumps. It is proposed, in the future, to use complete initial screening to fully eliminate grit and rags from the process flow stream.

FLOW EQUALIZATION BASINS

The tank configuration of the flow equalization basins at the Hatfield facility are rectangular. Solids entering the vessel tend to agglomerate in the corners. Large volumes of organic material in the solids eventually become putrescible and lead to process, as well as esthetic, problems. During extremely low flow conditions, the dissolved oxygen in the flow equalization basins drops off considerably. The mechanical mixers utilized do not have the capability of preventing these problems.

It is proposed to resolve the problems of the flow equalization basins by the addition of air mixing, which should not only keep the solids in suspension, but should also eliminate problems of septicity. The units at Hatfield, supplied with air and protected by adequate grit removal and screening, should not experience the difficulties presently faced.

As a result of the Hatfield Township flow equalization problems, a circular flow equalization basin with three compartments, each equipped with a floating aerator would be recommended. This method appears to be far more satisfactory than that utilized in the Hatfield facility. Unfortunately, future additional equalization basins at Hatfield will, if required, probably necessitate expansion by construction of additional rectangular basins, due to the space available in the area where such construction would be required. However, adequate mixing and dissolved oxygen would be provided.

CHEMICAL FEEDS

One of the problems, not anticipated at the Hatfield facility, was the volume of inert material in the pebble grade lime. At least 2% of the pebble lime is inert. As a result, for every ton of lime consumed, there is produced at least forty pounds of gritty material. Since the lime slakers are located in the basement of the building, this means that periodically operators must manually haul the grit out of the basement for disposal on landfill. This is a laborious task, and consumes many man-hours of the operators' time. The slaked lime is contained in a slurry tank and must be pumped up to the volumetric lime feeder. The inert material creates a good deal of wear on the slurry pumps, and has a tendency to deposit itself in the valves and in the piping. This has led to a maintenance problem, in that periodically the slurry tank, pumps, and lines must be cleaned of this material.

The problem of wear and tear has been solved by the use of cast iron pump impellers, water seals on the pumps and a regular maintenance program. In addition, an industrial elevator or electric hoist will be utilized to bring the collected inerts to the surface.

It is proposed that, in new plants, the lime feed equipment will be located above the point of introduction. The slurry type slaking system operates fairly well, and will operate much better with the solids problem eliminated. Ball valves will be eliminated from the lime slurry piping system to prevent plugging up with inert material.

The alum fed into the tertiary system is dry, granular, aluminum sulfate. It is packed in 100-pound (45.4 kg) bags, and the unexpectedly large amounts consumed result in the operators having to carry a 1,200-1,500 pound (544.8-681 kg) daily supply of alum up a flight of stairs. This has met with the dissatisfaction of operations personnel, and has become a subject of grievance with the labor union. It is proposed to convert to a liquid alum feed system. Liquid alum is less expensive on a dry-weight basis, and the cost savings in chemicals alone are expected to amortize the tankage and piping required in less than three years.

In the tertiary system, alum can be fed to the flash mixer, or directly to the filter influent clear well. Polymer can be fed to the tertiary flocculator, or also the the filter influent clear well. This arrangement does not give the operations personnel the flexibility to feed polymer and alum prior to the tertiary filters in the best possible manner. Any floc that may form may be destroyed by its passage through the high rpm centrifugal pumps feeding the filters. Plant personnel are in the process of relocating these feed points downstream of the filter influent pumps.

In a wastewater treatment facility, it sometimes becomes necessary to feed chlorine in order to reduce septic conditions. Chlorine feed lines should be included into the gravity thickener, the head-box of the aeration tank, and to the secondary clarifiers.

AUTOMATION

The Hatfield facility was designed utilizing probe type pH sensing elements. These have proven to be a major operational problem. The pH probes in the surge storage tanks were never installed because, periodically, they would be out in the air and they would be impossible to maintain. The pH probes in the flocculation zone of the clari-flocculator, which serves as the major point of control, have a tendency to coat up with calcium carbonate scale. This scale increases the resistence across the probe, calling for more lime to be fed. With more lime being fed, more scale is formed, and eventually the pH of the primary effluent increases beyond the limits of biological secondary treatment. As a result, the pH control of lime feed has been converted to a manual basis. At the same time, flow-through probes are being experimented with, and the initial response is an extremely good one. Indications are that, except for plugging of the feed line into the pH probes, the flow-through probe requires minimal maintenance and is fairly consistent with manual laboratory testing.

The flow through probes have been in operation for over six months, and they are checked at least twice per week against a standardized laboratory pH meter. The flow-through probes correlate very well with the lab meter (± 0.2 pH units). There is, however, a need to clean the inlet hose of accumulated solids to prevent plugging. This is done at least once per day.

There are various types of flow measuring devices in use in the Hatfield Township facility. There are magnetic flow meters which send signals to the Red Valves. One of the problems with these meters is that it is extremely difficult to calibrate them. Raw sewage into one pump station is measured by a Parshall flume, and into the other station, by the use of a flow nozzle. Both units depend upon a float-type action to sense a level in the measuring device. During high flow rates, the flows exceed the limits of these devices, and the net result is loss of accurate flow measurement. The flow of all sludges is measured by the use of Venturi type meters. The centers in the Venturi meters tend to plug up with solids, and as a result, false readings are obtained. Grit removal, and adequate screening should resolve this problem.

One of the most basic methods of monitoring a plant is that of measuring flow. The heterogeneous nature of sewage and sewage sludges has pointed out the problems inherent with operation of conventional flow meters. Until such time as something as basic as flow measurement can be resolved, additional automation of treatment facilities is questionable.

AUTOMATIC SAMPLERS

The data used in the design of the 1970 facility was based upon manually composited grab samples. The expanded facility includes the utilization of composite samplers throughout the plant. While there are many benefits to be derived from the use of automatic, refrigerated, composite samplers, they too result in a maintenance problem. The lack of maintenance of a composit sampler can result in more misleading information than obtained from a grab sample.

Automatic samplers are to be found in the Hatfield Township facility, as indicated in Table 46.

TABLE 46. AUTOMATIC SAMPLER LOCATIONS

Raw Wastewater	Secondary Effluent
Surge Storage Tank	Filtered Effluent
Primary Effluent	Final Effluent

PUMPS

The Hatfield Township facility utilizes centrifugal pumps throughout. One of the problems, not yet investigated is the potential problem of degradation of the sludges by shearing through the high-speed centrifugal machine.

The design of the Hatfield Township facility requires the use of portable pumps for dewatering the majority of treatment vessels. Where possible, it is recommended that all tanks be capable of draining by gravity. This will facilitate flushing operations. The use of large gate valves with gear reducers results in a good deal of manual effort in opening or closing these valves. The cost of motorizing these valves was found to be prohibitive on the initial design of the project. Plant operations personnel are looking into the purchase of a portable electric valve operator. Ideally, all of the valves should have been equipped with electric operators, controllable from a central control point.

It is presently impossible to by-pass the tertiary filters. During extremely high flow conditions, the solids carry-over from the secondary and tertiary cycle tends to plug up the filters, resulting in continuous back-wash and more recycle. Eventually, the plant operates on just treating its own recycle. To eliminate this problem would require the inclusion of a by-pass around the tertiary filters in the plant flow scheme. A by-pass was eliminated during the construction plan review by Federal Regulatory Agency personnel.

Where there are no prohibitions on in-plant by-passes around unit processes, their availability should be encouraged.

PROCESS OBSERVATIONS

The benefit of flow equalization basins has been proven by the operation at the Hatfield Township facility. The sudden surges from industrial loads, filter back-washes within the plant itself, or heavy rain which normally washes out the biological reactor, or upsets the chemical treatment processes in the facility is relieved. The flow equalization basins tend to dampen the effect of these surges and give the operators enough time to compensate for any problems incurred. The actual location of these units, upstream of the primary, helps provide steady state condition throughout the plant. The large amount of recycle within the plant however, results in a considerable expenditure of chemical to treat the recycled waste through the facility. In order to meet effluent standards, however, this is a must.

The plant is equipped with two aeration tanks, 50' x 50' x 15'. These provide 3.75-hours of total detention, enough detention for bio-oxidation, but not enough for complete oxidation of the ammonia form nitrogen. Any additional expansion will include enough aeration capacity for the completion of nitrification.

The Hatfield facility utilizes pressure filters, furnished with a mixed-media. Since the majority of the suspended solids have been removed by clarification in the tertiary tube-type clarifiers, the necessity of this mixed-media has not been unequivocally proven in operations to date.

The use of lime in the primary treatment vessels tends to break down the fatty acid material and generally forms a scum of calcium sterate precipitate, which collects within the flocculation basin, which is sometimes difficult to remove without manually hosing it out. Scum is gathered in collectors in the primary and secondary treatment vessels and pumped to the thickener. Any scum floating on the surface of the thickener is gathered in a collector, and pumped to the vacuum filter and then to incineration. During periods of nitrification, a good deal of the aeration sludge tends to float in the secondary clarifiers, and is extremely difficult to remove except by allowing it to wash out into the tertiary units, where it is clarified out. Provisions should be made to pump the secondary sludge back to the head-box of the aeration basin in order to maintain a nitrifying capability.

The Hatfield Township Municipal Authority advanced waste treatment facility was a prototype. As such, it serves as a basis to build upon. A firm knowledge of the problems encountered in the plant are as important as the knowleddge of those units of operation and processes which operate adequately. It is hoped that the data and discussions presented in this report will enable future projects to benefit from the deficiencies observed during the development of this project, and to benefit from the demonstrated success of the total operation.

SECTION XII

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SECTION XIV

APPENDICES

APPENDIX A

BASIS OF DESIGN
AND
COMPONENT CALCULATIONS

1970 AWT FACILITY

1. Basis of Design

Average 24 Hour Domestic Sewage Flow		2,850,000 GPD (10,787 M^3/D)
Allowance for Industrial Flows - Present & Future		750,000 GPD (2,839 M^3/D)
Design Average Hour Flow = Q =		3,600,000 GPD (13,626 M^3/D)
	or	2,500 GPM (9,462 L/M)
16 Hour Rate of Flow = 1.5Q =		5,400,000 GPD (20,439 M^3/D)
	or	3,750 GPM (14,194 L/M)
Peak Rate of Flow = 2.0Q =		7,200,000 GPD (27,252 M^3/D)
	or	5,000 GPM (18,925 L/M)

Equivalent Design Population @ 90 GPC (342 L/C)

Domestic	31,700 Persons
Industrial	8,300 Persons

BOD Loading - Domestic	0.17 Lb/Capita/Day* (0.077 Kg/D/C)
Suspended Solids Loading - Domestic	0.17 Lb/Capita/Day* (0.077 Kg/D/C)
Total BOD	7,050 Lb/Day** (3,204 Kg/D)
Total Suspended Solids	7,050 Lb/Day** (3,204 Kg/D)

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* Equivalent to 226 mg/1. Present BOD average 192 mg/1. Present suspended solids average 178 mg/1.

** Future Industrial flows anticipated to have BOD and suspended solids loadings equivalent to 0.20 Lb/Equiv. Capita/Day (0.091 Kg/D/C)

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2. Plant Components

A. Comminutors

(1) Number of Units - Two (One Existing)

(2) Type, Size and Capacity

Oscillating type, 15" size, 0.30 mgd to 3.5 mgd capacity.

(3) Auxiliary Screen - None

Second comminutor to be located in existing hand-cleaned bar screen location.

B. Flash Mixing - Primary

(1) Number of Units - Two

(2) Main Dimensions

Mixer No. 1 (Bottom Feed) - 5'-0" x 5'-0" x 7'-8" SWD

Mixer No. 2 (Bottom Feed) - 9'-0" x 9'-0" x 11'-4" SWD

(3) Capacity

Mixer No. 1 - 191.8 CF = 1,438 Gallons

Mixer No. 2 - 917.7 CF = 6,883 Gallons

(4) Detention

Mixer No. 1 - @ Q = $\frac{1{,}438 \text{ Gal.}}{2{,}500 \text{ GPM}}$ = 0.58 Minutes

Mixer No. 2 - @ 0.5 Q = $\frac{6{,}883 \text{ Gal.}}{1{,}250 \text{ GPM}}$ = 5.5 Minutes

NOTE: Lime added to 50% recirculated sludge in Mixer No. 2, then mixed with raw sewage in Mixer No. 1.

C. Clarifier - Flocculators

(1) Number of Units - Two

(2) Main Dimensions

Each: 60'-0" Diameter x 10'-0" SWD

(3) Capacity of Units

2 x 2,825 SF x 10 Ft. = 56,500 Cu. Ft. = 423,750 Gallons

(4) Detention Period at Q

$$\frac{423,750 \text{ Gal.}}{2,500 \text{ GPM}} = 169.5 \text{ Minutes} = 2.82 \text{ Hours}$$

(5) Surface Settling Rate at Q

$$\frac{2,600,000 \text{ GPD}}{5,650 \text{ SF}} = 637 \text{ Gal/SF/Day}$$

(6) Effluent Weir Overflow Rate at Q

$$\frac{3,600,000 \text{ GPD}}{2 \times 188.5 \text{ LF}} = 9,549 \text{ Gal/LF/Day}$$

(7) Flocculation

(a) Main Dimensions

Each: 31'-0" Diameter x 6'-9" SWD

(b) Capacity of Units

2 x 755 SF x 6.75 Ft. = 10,192 Cu. Ft. = 76,440 Gallons

(c) Detention

$$\frac{76,440 \text{ Gal.}}{2,500 \text{ GPM}} = 30.6 \text{ Minutes}$$

(8) Sludge Recirculation

50% Maximum, 1,250 GPM

D. Aeration Tanks

(1) Number of Units - Two (One Existing)

(2) Main Dimensions

Each: 50'-0" x 50'-0" x 15'-0" SWD

(3) Capacity of Units

2 x 50' x 50' x 15' = 75,000 Cu. Ft. = 562.500 Gal. = 4.69 (10^6) Lb.

(4) Detention Period at Average Flow (Not Including Return Sludge)

$$\frac{562{,}500 \text{ Gal.}}{2{,}500 \text{ GPM}} = 225 \text{ Minutes} = 3.75 \text{ Hours}$$

(5) Loading of Tanks

BOD to Aerators: - 7,050 Lb. x 0.40 = 2,820 Lb/Day

MLSS: - $\frac{2{,}820 \text{ Lb/Day}}{0.35 \text{ Lb. BOD/Lb. MLSS}} = 8{,}050 \text{ Lb. MLSS}$

MLSS Concentration: - $\frac{8{,}050 \text{ Lb. MLSS}}{4.69\ (10^6) \text{ Lb. Vol.}} = 1716 \text{ mg/1 MLSS}$

(6) *Type of Aeration Facilities*

(a) Classification

Combination Aerators, providing mechanical mixing and surface transfer of oxygen as well as sparged air diffusion.

(b) Number of Units - Two (One Each Tank)

(c) Capacity Required

0.80 Lb. O_2/Lb. BOD x 2,820 Lb. BOD = 2,260 Lb/Day O_2
= 94.2 Lb/Hour O_2

Provide Total O_2 Required in Each Basin

Assume 2 Lb. O_2 Transfer per HP Hour

HP Required, Each Basin = $\frac{94.2 \text{ Lb/Hour}}{2 \text{ Lb/Hr/HP}}$ =

47.1 HP Use 50 HP Each Basin

E. Secondary Clarifiers

(1) Number of Units - Two

(2) Main Dimensions

Each: 55'-0" Diameter x 12'-0" SWD

(3) Capacity of Units

2 x 2,376 SF x 12 Ft. = 57,024 Cu. Ft. = 427,680 Gal.

(4) Detention Period at Q (Not Including Return Sludge)

$$\frac{427{,}680 \text{ Gal.}}{2{,}500 \text{ GPM}} = 171.1 \text{ Minutes} = 2.85 \text{ Hours}$$

(5) Surface Settling Rate at Q

$$\frac{3{,}600{,}000 \text{ GPD}}{4{,}752 \text{ SF}} = 758 \text{ Gal/SF/Day}$$

(6) Effluent Weir Overflow Rate

$$\frac{3{,}600{,}000 \text{ GPD}}{2 \times 173 \text{ LF}} = 10{,}405 \text{ Gal/LF/Day}$$

F. Flash Mixing - Tertiary

(1) Number of Units - Two

(2) Main Dimensions

Each: 9'-6" x 8'-0" x 9'-6" SWD

(3) Capacity of Units

Each: 722 Cu. Ft. = 5,415 Gallons

(4) Detention - Each

$$\frac{5{,}415 \text{ Gal.}}{1{,}250 \text{ GPM}} = 4.33 \text{ Minutes @ 1/2 Q Each}$$

$$\frac{5{,}415 \text{ Gal.}}{2{,}500 \text{ GPM}} = 2.17 \text{ Minutes @ Q Each}$$

G. Flocculation

(1) Number of Units - Two

(2) Main Dimensions

Each: 26'-0" x 12'-0" x 12'-0" SWD

(3) Capacity of Units

Each: 3,744 Cu. Ft. = 28,080 Gallons

(4) Detention - Each

$$\frac{28{,}080 \text{ Gal.}}{1{,}250 \text{ GPM}} = 22.5 \text{ Minutes @ 1/2 Q Each}$$

$$\frac{28{,}080 \text{ Gal.}}{2{,}500 \text{ GPM}} = 11.23 \text{ Minutes @ Q Each}$$

H. Tube Settling - Tertiary

(1) Number of Units - Two

(2) Main Dimensions

Each: 67'-0" x 16'-0" x 10'-6" SWD

(3) Capacity of Units

Each: 11,256 Cu. Ft. = 84,420 Gallons

(4) Detention - Each

$$\frac{84{,}420 \text{ Gal.}}{1{,}250 \text{ GPM}} = 67.5 \text{ Minutes @ 1/2 Q Each}$$

$$\frac{84{,}420 \text{ Gal.}}{2{,}500 \text{ GPM}} = 33.8 \text{ Minutes @ Q Each}$$

(5) Surface Settling Rate @ Q

Tube - Module Design - Angle tube - settling modules, 2" Square, inclined at 60^{o} to the horizontal, 24" long.

Tube Overflow Rate: - 1.9 GPM/Ft.

Equivalent Tube - Settling Overflow Rate - 240 Gal/SF/Day

(6) Effluent Weir Overflow Rate

$$\frac{3{,}600{,}000 \text{ GPD}}{640 \text{ Lin. Ft.}} = 5{,}625 \text{ Gal/Lin. Ft./Day}$$

I. Mixed-Media Filters - Tertiary

(1) Number of Units - Three

(2) Type

Pressure Filters, Mixed-Media

(3) Main Dimensions

Each: 28'-0" L x 10'-0" Diameter

(4) Filter Area

Each: 287 Sq. Ft.

(5) Filter Rate - Each

$$\frac{833 \text{ GPM}}{287 \text{ Sq. Ft.}} = 2.9 \text{ GPM/Sq. Ft. @ } 1/3 \text{ Q}$$

$$\frac{1{,}250 \text{ GPM}}{287 \text{ Sq. Ft.}} = 4.35 \text{ GPM/Sq. Ft. @ } 1/2 \text{ Q}$$

$$\frac{1{,}875 \text{ GPM}}{287 \text{ Sq. Ft.}} = 6.53 \text{ GPM/Sq. Ft. @ } 3/4 \text{ Q}$$

Maximum Filter Rate: - 7.5 GOM/Sq. Ft.

Maximum Flow/Filter = 2,200 GPM = 9.5 MGD*

*Gross, Not including down time for backwash.

(6) Backwash Flow Rate

14.6 GPM/Sq. Ft.

(7) Surface Wash Flow Rate

0.7 GPM/Sq. Ft.

J. Chlorine Contact Tanks

(1) Number of Units - Four (All Existing)

(2) Main Dimensions

Existing Cl_2 Tanks - 2 @ 20'-0" long x 13'-0" wide x 6'-6" SWD

Existing Sec. Clarifiers - 2 @ 28'-0" diameter x 5'-6" SWD

(3) Capacity of Units

Existing Cl_2 Tanks =	2,800 CF = 21,000 Gal.
Existing Sec. Clarifiers =	6,776 CF = 50,820 Gal.
	9,576 CF = 71,820 Gal.

(4) Contact Time @ Maximum Pumping Rate

$$\frac{71{,}820 \text{ Gal.}}{5{,}000 \text{ GPM}} = 14.4 \text{ Minutes}$$

K. Chlorination

(1) Number of Units - Two

(2) Capacity

Each: 100 Lb/Day, Total 200 Lb/Day

(3) Capacity Required

6 mg/l @ Q Ave. = 6 mg/l x 8.34 Lb/MG/mg/l x 3.6 MGD = 180 Lb/Day

NOTE: Upon completion of Phase II, dosage required should be 4-5 mg/l at Q Ave.

L. Sludge Thickener

(1) Number of Units - One (Abandon existing unit)

(2) Main Dimensions

40'-0" diameter x 10'0" SWD

(3) Capacity of Unit

1,260 SF x 10 Ft. = 12,600 CF = 94,500 Gallons

(4) Estimated Sludge Volume

Primary - 7.050 Lb. SS/Day x 0.85 Removal =	6,000 Lb/D
Lime - 6,000 Lb/Day x 0.80 Recovery =	4,800 Lb/D
WAS - $\frac{160 \text{ mg/l BOD}}{10^6 \text{ Lb}} \times \frac{3.6(10)^6 \text{Gal}}{\text{Day}} \times \frac{8.34 \text{ Lb}}{\text{Gal}} \times \frac{0.45 \text{ Lb DS}}{\text{Lb BOD}} =$	2,160 Lb/D
Total Sludge Volume	12,960 Lb/D

(5) Thickener Flow

Feed Concentration = 3,500 mg/1

Total Solids = 12,960 Lb/Day

$$\frac{12,960 \text{ Lb/D}}{3,500\ (10^6)} = 3.7\ (10^6) \text{ Lb. Liquid/Day} = 445,000 \text{ Gal/Day} = 310 \text{ GPM}$$

(6) Overflow

@ 700 GPD/Ft.^{2} Overflow Rate, Inflow =
700 GPD/Ft.^{2} x 1,260 SF = 882,500 GPD = 610 GPM

Provide 300 GPM Dilution Water

M. Sludge Dewatering

(1) Classification - Vacuum Filters

(2) Number of Units - Two (Existing)

(3) Main Dimensions

Each: 4'-0" diameter x 4'-0" long

(4) Capacity

50 Sq. Ft., Total 100 Sq. Ft.

(5) Filtering Rate

4.5 Lb. DS/SF/Hour, Total = 450 Lb. DS/Hour

(6) Hours Operation/Week

From E. above - Total Lb. DS = 12,960 Lb/Day

12,960 Lb/Day x 7 D/Wk. = 90,720 Lb. DS/Wk.

$$\frac{90,720 \text{ Lb. DS/Wk.}}{450 \text{ Lb. DS/Hour}} = 201.6 \text{ Hours/Week}$$

NOTE: At such time as solids loading approaches 140 Hrs./Wk. operation, an additional vacuum filter will be installed.

N. Sludge Incineration

(1) Number of Units - One (Existing)

(2) Type - Multiple Hearth Furnace

(3) Main Dimensions

10'-9" diameter x 17'-6" high

(4) Capacity

1,500 Lb. Filtered Sludge Cake/Hour @ 45% Volatile Content

(5) Hours Operation/Week

From E. above - Total Lb. DS = 12,960 Lb/Day = 90,720 Lb/Week

@ Filtered Cake Moisture Content of 75%, amount of Sludge Lb. Cake/Week

$$\text{Cake/Week} = \frac{90{,}720 \text{ Lb/Wk}}{0.25} = 362{,}880 \text{ Lb. Cake/Week}$$

$$\frac{362{,}880 \text{ Lb. Cake/Week}}{1{,}500 \text{ Lb. Cake/Hour}} = 241.9 \text{ Hours/Week}$$

NOTE: At such time as furnace operations approaches 140 hours/week, provision will be made to increase furnace capacity. Present furnace operations approximately 24 Hours/Week.

O. Pumping

(1) Raw Sewage Pumps

(a) Number - Two

(b) Type

Vertical, extended, open-shaft, dry-pit, non-clog, constant speed.

(c) Capacity, Each - 3,500 GPM @ 51.75' TDH

(2) Activated Sludge Return Pumps

(a) Number - Three

(b) Type

Horizontal, ball-bearing, non-clog, variable speed.

(c) Capacities

Pump No. 1 - 625 to 2,500 GPM @ 31.3' TDH

Pump No. 2 - 625 to 2,500 GPM @ 31.3' TDH

Pump No. 3 - 0 to 625 GPM @ 32.3' TDH

(3) Waste Activated Sludge Pumps

(a) Number - Two

(b) Type

Horizontal, ballbearing, non-clog, variable speed.

(c) Capacity, Each - 50 to 100 GPM @ 30.8' TDH

(4) Waste Primary Sludge Pumps

(a) Number - Two

(b) Type

Horizontal ball-bearing, non-clog, variable speed.

(c) Capacity, Each - 160 to 320 GPM @ 42' TDH

(5) Primary Sludge Recirculation Pumps

(a) Number - Two

(b) Type

Horizontal ball-bearing, non-clog, variable speed.

(c) Capacity, Each - 625 to 1,250 GPM @ 22' TDH

(6) Filter Influent Pumps (Secondary Clarifier Effluent Pumps)

(a) Number - Three

(b) Type

Horizontal ball-bearing, non-clog, variable speed.

(c) Capacity, Each - 1,100 to 2,200 GPM @ 49.4' TDH

(7) Dilution Water Pumps

(a) Number - Two

(b) Type

Horizontal ball-bearing, non-clog, variable speed.

(c) Capacity, Each 150 to 300 GPM @ 43.4' TDH

(8) Scum Pumps

(a) Number - Four

(b) Type

Vertical, extended, open-shaft, dry pit, non-clog, constant speed.

(c) Capacities

Pump Nos. 1 and 2 - 100 GPD @ 61.6' TDH

Pump Nos. 3 and 4 - 90 GPM @ 84' TDH

(9) Tertiary Sludge Pumps

(a) Number - Two

(b) Type

Horizontal ball-bearing, non-clog, variable speed.

(c) Capacity, Each - 600 to 1,200 GPM @ 69.5' TDH

P. Chemical Feed Equipment - Primary

(1) Number of Units - One

(2) Type - Dry

(3) Feed Rate - 350 Lb/Hour

(4) Dosage Rate of Lime - 200 mg/1

Q. Chemical Feed Equipment - Tertiary

(1) Number of Units - One

(2) Type - Dry

(3) Feed Rate - 350 Lb/Day

(4) Dosage Rate of Alum - 40 mg/1

R. Surge Storage Tank Capacity Determination

Daily Raw Sewage Flow:	3.60 MGD	Ave. 24 Hr. Flow
	0.73 MGD	Infiltration
	4.32 MGD	Total Raw Sewage
	18,000 GPH	
	3,000 GPM	

1% Daily Flow: 43,200 Gallons

Daily Filter Flow:	2,500 GPM	Ave. 24 Hr. Flow
	1,250 GPM	Recirculation Flow
	3,750 GPM	Total Flow
	225,000 GPM	
	5.40 MGD	

Filter Capacity:

28.0' long x 10.0' = 280 SF x 5 GPM/SF =

1,400 GPM Filter Rate Per Filter

3 Filters x 1,400 GPM = 4,200 GPM Rate = 252,000 GPH = 6.05 MGD

Backwash Rate:

15 GPM/SF Filter x 280 SF = 4,200 GPM

Backwash Time:		
8 Minutes x 4,200 GPM	=	33,600 Gallons
Backwash Volume Only:	=	33,600 Gallons
Surface Wash Rate:		
0.71 GPM/SF Filter x 280 SF	=	200 GPM
Surface Wash Time:		
4 Minutes x 200 GPM	=	800 Gallons
Surface Wash Volume Only:	=	800 Gallons
Backwash Volume:		33,600 Gallons
Surface Wash Volume:		800 Gallons
Total Backwash per Filter		34,400 Gallons
Filter Plant Flow:		252,000 GPM*
Filter Plant Flow During Backwash		240,800 GPM**

* 3 Filters Operating - Full Capacity

** 2 Filters on Plus 1 On For 52 Minutes

DETERMINATION OF SURGE STORAGE REQUIRED

Time	Per Cent Flow Factor	Raw Sewage Gallons	B/W Return Gallons	Total Flow Gallons	Filter Capacity Gallons	Accumulation Surge Storage Gallons	Total Build-Up Surge Storage Gallons
6- 7 A.M.	3.0	129,600	+ 34,400	164,000	240,000**	- 76,800	0
7- 8	3.0	129,600	-	129,600	252,000*	- 122,400	0
8- 9	4.0	172,800	+ 34,400	207,200	240,800	- 33,600	0
9-10	4.5	194,400	-	194,400	252,000	- 57,600	0
10-11	6.5	280,800	+ 34,400	315,200	240,800	+ 74,400	74,400
11-12 Noon	6.5	280,800	-	280,800	252,000	+ 28,800	103,200
12- 1 P.M.	6.5	280,800	+ 34,400	315,200	240,800	+ 74,400	177,600
1- 2	6.5	280,800	-	280,800	252,000	+ 28,800	206,400***
2- 3	4.5	194,400	+ 34,400	228,800	240,800	- 12,000	194,400
3- 4	4.5	194,400	-	194,400	252,000	- 57,600	136,800
4- 5	4.5	194,400	+ 34,400	228,800	240,800	- 12,000	124,800
5- 6	4.5	194,000	-	194,400	252,000	- 57,600	67,200
6- 7	5.0	216,000	+ 34,400	250,400	240,800	+ 9,600	76,800
7- 8	5.0	216,000	-	216,000	252,000	- 36,000	40,800
8- 9	5.0	216,000	+ 34,400	250,400	240,800	+ 9,600	50,400
9-10	5.5	237,600	-	237,600	252,000	- 14,400	35,000
10-11	5.0	216,000	+ 34,400	250,400	240,800	+ 9,600	45,600
11-12 Midnight	5.0	216,000	-	216,000	252,000	- 36,000	9,600
12- 1 A.M.	1.5	64,800	+ 34,400	99,200	240,800	- 141,600	0
1- 2	1.5	64,800	-	64,800	252,000	- 187,200	0
2- 3	1.5	64,800	+ 34,400	99,200	240,800	- 141,600	0
3- 4	1.5	64,800	-	64,800	252,000	- 187,200	0
4- 5	2.5	108,000	+ 34,400	142,400	240,800	- 98,400	0
5- 6	2.5	108,000	-	108,000	252,000	- 144,000	0

* 3 Filters Operating - Full Capacity

** 2 Filters on Plus 1 on For 52 Minutes

*** Surge Storage Required

APPENDIX B

METHOD FOR TOTAL PHOSPHORUS ANALYSIS

Phosphorus, Total and Total Soluable

The sanitary significance of the various phosphorus compounds has been discussed in Section III. Since the permit for the Hatfield plant specifies an effluent phosphorus limit, the sewage must be analyzed routinely for phosphorus content.

The procedure which follows is for nitric acid-sulfuric acid sample digestion, followed by the stannous chloride method of phosphorus determination. This method is suitable for work on domestic wastewater. Alternate procedures can be found in "Standard Methods" and "FWPCA Methods".

1. Scope and Application

   1.1 The following procudure can be utilized to determine the total phosphorus and total filterable (soluble) phosphorus content in domestic wastes, industrial wastes and natural waters.

   1.2 The procedure can be used to measure phosphorus in the range of 0.01 to 6 mg P/l. An extraction procedure is used to determine low range phosphorus concentrations. For values above 6 mg P/l, the vanadomolybdic acid method, found in "Standard Methods" must be used.

2. Summary of Method

   2.1 Poly and organic forms of phosphorus are converted to orthophosphate by digesting the sample with nitric and sulfuric acids. The orthophosphate then reacts with ammonium molybdate under acid conditions to form a complex compound known as ammonium phosphomolybdate.

$$PO_4^{-3} + 12(NH_4)_2\ MoO_4 + 24H^+ \longrightarrow (NH_4)_3PO_4 \cdot 12MoO_3 + 21\ NH_4^+ + 12H_2O$$

Stannous chloride is then added as a reducing agent. It reacts with the ammonium phosphomolybdate to form a blue colored compound known as molybdenum blue. The formula for molybedenum blue is not known. The blue color resulting from this reaction is proportional to the amount of ammonium phosphomolybdate present. The stannous chloride does not react with any excess ammonium molybdate.

The intensity of the blue color is measured using a spectrophotometer. Phosphorus concentration can then be determined from this data.

3. Sampling and Preservation

3.1 If the sample cannot be analyzed immediately, the portion intended for the total soluable phosphorus determination should be filtered before being stored. The samples should not be stored in plastic bottles because there is the possibility that phosphorus compounds can be adsorbed onto the walls of the plastic containers.

4. Interferences

4.1 Gross positive errors can come from using glassware cleaned with phosphate detergents.

5. Apparatus

5.1 Hot plate

5.2 Spectrophotometer with curvettes

If the analyst is not familiar with the limitations and idiosyncrasies of his spectrophotometer, he should refer to pages 9 - 12 of "Standard Methods" and to the manufacturers instructtions.

5.3 Acid - cleaned glassware

All glassware should be cleaned with hot dilute HCl and rinsed several times with distilled water. The glassware should be reserved solely for phosphorus determinations and after each use, it should be washed and filled with distilled water until needed. If this procedure is used, the acid treatment is only required occasionally. Phosphate detergents should never be used on glassware used for phosphorus determination.

Preliminary Measures:

A. Filtration

a.1 Separation of filterable (soluble) phosphorus from non-filterable (particulate) phosphorus is made using a .45 membrane filter. This procedure is purely an analytical technique which is convenient and easily reproduceable. It is not a true separation of soluble and suspended phosphorus compounds.

a.2 The filters must be washed before use because they can contain significant amounts of phosphorus. They can be washed by either soaking filters (50 per 20 ml water) in distilled water for 24 hours or soaking filters (50 per 20 ml water) in distilled water for one (1) hour, changing water and soaking filters an additional three (3) hours.

B. Preparation of Calibration Chart

b.1 Reagents

<u>Stock Solution</u>

Dissolve 0.4393 g of pre-dried KH_2PO_4 in distilled water and dilute to 1 liter 1 ml = 0.1 mg P.

<u>Standard Solution A</u>

Dilute 100 ml of stock solution to 1 liter. 1 ml = 0.01 mg P.

<u>Standard Solution B</u>

Dilute 100 ml of Standard Solution A to 1 liter 1 ml = 0.001 mg P.

b.2 Prepare a series of standards by diluting suitable volumes of Standard Solution A and B to 100 ml with distilled water. The following dilutions are suggested.

ml of S. Sol B	Conc. mg P/1
0.0	0.00
2.0	0.02
5.0	0.05
10.0	0.10

ml of S. Sol A	Conc. mg P/l
2.0	0.20
5.0	0.50
8.0	0.80
10.0	1.00
20.0	2.00
30.0	3.00

Carry the standard solutions through the digestion and the stannous chloride stages.

Allow color to develope over 10 minutes but less than 12 minutes, then measure the color photometrically at 690 mu. Plot a calibration curve on rectangular graph paper. The calibration curve may deviate from a straight line at the upper range.

C. Digestion

c.1 The analyst will have to have a rough idea of the amount of phosphorus to expect at the various sampling locations in order to be able to select an appropriate sample size. If 100 ml of sample will contain more than 0.2 mg of phosphorus, an aliquot will have to be used. The following table can be used to determine sample size.

Sample Size	Phosphorus Concentration
100 ml	2 mg/l P or less
50 ml	4 mg/l P

Sample Size	Phosphorus Concentration
25 ml	8 mg/1 P
10 ml	20 mg/1 P
5 ml	40 mg/1 P
2.5 ml	80 mg/1 P

c.2 Procedure

An appropriately sized sample, 1 ml conc.H_2SO_4 and 5 ml concentrated HNO_3 are added to a flask.

Digest the sample to a volume of 1 ml, then continue digesting until the solution becomes colorless.

Cool and add approximately 20 ml distilled water, 1 drop phenolphthalein indicator and as much 1N NaOH as necessary to produce a faint pink color. Transfer the solution to a 100 ml volumetric flask, filtering with washed filters if the sample contains particulate matter, If the sample had to be filtered, wash the filter with distilled water and add these washings to the volumetric flask. Adjust the sample volume to 100 ml and proceed with the colorimetric determination.

Phosphorus Determination

6. Reagents

6.1 Phenolphthalein Indicator Solution

6.2 Strong Acid Solution

Slowly add 300 ml concentrated H_2SO_4 to 600 ml distilled water.

Cool and add 4 ml concentrated HNO_3 and dilute to 1 liter.

6.3 Ammonium Molybdate Reagent I

Dissolve 25 g $(NH_4)_6MoO_{24} \cdot 4 H_2O$ in 175 ml distilled water. cautiously add 280 ml concentrated H_2SO_4 to 400 ml distilled water. Cool, add the molybdate solution and dilute to 1 liter.

6.4 Stannous Chloride Reagent I

Dissolve 2.5 of $SnCl_2 \cdot 2 H_2O$ in 100 ml glycerol. Heat in a water bath and stir until all the crystals are dissolved. This reagent is realitively stable. If any turbidity developes when the reagent is added to the sample, the quality of the stannous chloride is questionable.

7. Procedure

7.1 If the sample is pink from the phenophthalein addition in the digestion stage, add strong acid solution dropwise until the sample turns colorless.

7.2 Add 4 ml molydbate reagent I and 0.5 ml stannous chloride reagent I and mix well. Color development is dependant on the temperature of the solution, therefore all samples, reagents, and standards should be within 2° C. of each other and be between 20° C. and 30° C.

7.3 After 10 minutes, but before 12 minutes, measure the color using a spectrophotometer set at 690 mu. Use calibration chart to determine phosphorus determination by direct readout from absorbance data. Use a distilled water blank and run at least one standard to check the calibration curve. A blank on the reagents should also be run.

8. Extraction

At phosphorus concentrations below 0.1 mg P/1 an extraction procedure can be used for increased sensitivity and more accurate results. This procedure can be found in "Standard Methods."

The following table of conversion factors can be used to compare the various forms of phosphorus.

To Convert X to P Multiply by:	X	To Convert P to X Multiply by:
1.000	P	1.00
.451	P_2O_5	3.29
.327	PO_4	3.06
.316	H_3PO_4	3.16
.256	$AlPO_4$	3.90
.200	$Ca_3(PO_4)_2$	5.00
.181	$Ca_5OH(PO_4)_3$	5.51

APPENDIX C

INDUSTRIAL SURCHARGE PROGRAM

The Hatfield Authority has instituted a program whereby industry must pay a "fair share" for any extra operating costs imposed upon the facility above environmental waste. A portion of the permit application has been included herein.

HATFIELD TOWNSHIP MUNICIPAL AUTHORITY

APPLICATION FOR INITIAL INDUSTRIAL WASTE DISCHARGE PERMIT

I/WE, the undersigned, hereby make application to the Hatfield Township Municipal Authority for an initial permit to discharge industrial waste to the Hatfield Township sewerage system, in accordance with section 7, Ordinance No. 114, of the Township of Hatfield, Montgomery County, Pennsylvania.

I/WE understand that the Authority has imposed regulations and requirements for the discharge of industrial waste to the Hatfield Township sewerage system, as permitted under said Section 7, Ordinance 114, and that the following general regulations and requirements apply:

A. *All industrial waste flows shall be metered by a separate sewage meter, which meter shall be satisfactory to the Authority, and shall provide for a separate remote recording device at an accessible location which shall have a totalizer, and a 30-day recording chart.*

B. *The meter described in A. above shall be installed and fully operable within 90-days after receipt of the Initial Industrial Waste Discharge Permit. All costs incident to the furnishing, installation, initial calibration, and maintenance on a continuing basis of said meter is and shall be my/our responsibility, and the I/we will provide to the Authority or its designated representative within 15-days of receipt of the Initial Industrial Waste Discharge Permit full details on the proposed meter installation, and that such installation will commence only upon the written authorization of the Authority, which shall not be more than 10-days after receipt by the Authority of the details of the proposed meter installation.*

C. *The meter installation shall be such that samples of the industrial waste flow shall be readily obtainable at the meter location, and I/we understand that the Authority will collect samples three times yearly for the analysis of the waste flow, one of which shall be analyzed by an independent certified laboratory, and two of which will be analyzed at the Hatfield Township Advanced Waste Treatment Facility laboratory. I/we further understand and agree that a charge for these industrial waste flow sample analyzation will be $_________, and that a check for $________ is attached to this application.*

D. *I/we understand that the Authority may at any time request that the meter be calibrated, and that the cost of such calibration* *shall be mine/ours if the meter is found to be out of calibration, or shall be the Authority's if the meter is found to be correct.*

E. *The charge for metered industrial waste flow shall be at a single rate per 1000 gallons, regardless of volume, which rate is currently \$________ per 1000 gallons.*

F. *There shall be a surcharge for industrial waste flow which shall be computed as follows:*

$$F = 1 + R\ (S + B + P + N + A) + C$$

Where F = *Facter to multiply the basic metered industrial process charge for a surcharge for strengths in excess of normal domestic sewage strengths.*

$R = 0.5$ = *Ratio of the estimated cost of treatment for quality, and the total sewerage cost.*

S = *Strength factor for total suspended solids computed at*

$$S = 0.40\ \left(\frac{S_1 - 200\ mg/l}{200\ mg/l}\right)$$

Where S_1 *is the industrial total suspended solids in mg/l.*

B = *Strength factor for* BOD_5 *computed at*

$$B = 0.30\ \left(\frac{B_1 - 200\ mg/l}{200\ mg/l}\right)$$

Where B_1 *is the industrial* BOD_5 *in mg/l.*

P = *Strength factor for Phosphorus, computed at*

$$P = 0.15\ \left(\frac{P_1 - 8\ mg/l}{8\ mg/l}\right)$$

Where P_1 *is the industrial P in mg/l.*

N = *Strength factor for Nitrogen, computed at*

$$N = 0.10\ \left(\frac{N_1 - 25\ mg/l}{25\ mg/l}\right)$$

Where N_1 *is the sum of the industrial* NH_3, NO_3 *and* NO_2 *in mg/l.*

A = Strength factor for Acid/Alkali, computed at

$$A = 0.02\ (7.0 - A_1) \text{ when } A_1 < 7.0$$

$$A = 0.02\ (A_1 - 7.0) \text{ when } A_1 > 7.0$$

Where A_1 *is the industrial pH.*

C = Strength factor for Chlorine Demand, computed at

$$C = 0.03 \times 8.33\ P_C\ (C_D - 5 \text{ mg/l})$$

Where P_C *= Cost of Chlorine per Pound*

C_D *= Industrial Chlorine Demand in mg/l.*

G. *I/we understand the because the Hatfield Township Advanced Waste Treatment Facility will not realize any benefit if my/our industrial process flow has any strength factor with a negative value, that any such negative strength factor value shall be considered as being zero.*

H. *The term of the Initial Industrial Waste Discharge Permit is one (1) year from the date of issue.*

I. *An application for a Renewed Industrial Waste Discharge Permit shall be filed thirty (30) days prior to the expiration date of the Initial Industrial Waste Discharge Permit.*

J. *I/we understand that the Initial Industrial Waste Discharge Permit will contain Special Conditions for the discharge of my/our industrial waste.*

K. *I/we understand that items prohibited from discharge to the sewage collection system as defined in Section 6 of Ordinance 114, and the limitations of items a. through n. of Section 7 of Ordinance 114 of the Township of Hatfield will be strictly enforced.*

L. *I/we understand the should I/we be in violation of the prohibitions of Section 6 of Ordinance 114, the limitations of Section 7 of Ordinance 114, or limitations imposed under Special Conditions as indicated in K. above, my/our Initial Industrial Waste Discharge Permit will be revoked and I/we must cease discharge to the Hatfield Township Sewage Collection System, and that I/we will be responsible for all costs incurred by the Hatfield Township Municipal Authority for damage or repair to the Sewage Collection System and the Advanced Waste Treatment Facility, and for all costs incurred to re-*

establish the correct operating conditions at the Advanced Waste Treatment Facility, by virtue of my/our failure to conform to the limitations and prohibitions.

M. I/we understand that if my/our waste flow consists only of sanitary sewage flows, and that I/we have no industrial process flow, items A. through G., and item J. above shall not apply, and that my/our sewage rate shall be established in accordance with Section 3, Ordinance 114 of the Township of Hatfield.

I/WE hereby declare that the following information furnished is true and accurate:

1. Number of permanent employees____________________.

2. Number of permanent employees working full-time on premises ____________________.

3. Average number of hours per day spend on premises by employees not working full-time on premises____________________.

4. Do you provide showers for your employees?___________.

5. Indicate length of operating day____________________.

   How many days per week?____________________.

   How many shifts per week?____________________.

   Number of employees per shift____________________.

6. Do you meter your water consumption?____________________.

   If so, attach water consumption data for preceeding 4 quarters. If not, estimate water consumption, and industrial waste flow

   ____

   ____.

7. Describe your operation and the nature of your industrial process waste:____

   ____

   ____

   ____

8. *Attach an analysis of your waste from an independent certified laboratory indicating Total Suspended Solids, BOD_5, Phosphorus as P, Ammonia Nitrogen, Nitrate Nitrogen, Nitrite Nitrogen, pH, and Chlorine Demand. Include also any items which may, by virtue of your process conceivably have an effect upon the operation of the Advanced Waste Treatment Facility, such as, color, heavy metals, oil, grease, and toxic materials.*

9. *Refer to paragraphs a. through n. Section 7, Ordinance 114 attached. Indicate if your waste is within the limits prescribed.*

   ____

   ____.

   If not, indicate variances from conditions set forth in Ordinance 114. ____

   ____

10. *Remarks:* ____

   ____

   ____

   ____

   ____

   ____

DATE: ____________________________

____
(Name of Sole Proprietorship/Partnership /Corporation)

BY: ______________________________________
(Signature)

____
(Title)

____
(Address of Organization)

____

____

Submit Application with supporting data in duplicate to:
Director of Operations
Hatfield Township Municipal Authority
Advance Lane, Colmar, Pennsylvania 18915

APPENDIX D

ANALYTICAL DATA

The following pages summarize analytical data for the period April 1973-March 1974.

1. The location of each sample point is as follows:

   Raw - Sample is taken from wet well of Pump Station #1 and Wet Well of Pump Station No. 2. Combined raw is mixed sample based upon flows from Montgomeryville and Hatfield.

   Surge - Effluent sample is taken just prior to flash mixer.

   Primary - Sample is taken from overflow trough of Clariflocculator.

   Secondary - Sample from overflow trough.

   Tertiary - Sample is obtained from effluent chamber of tertiary tube clarifiers.

   Filter - Sample point is on filtered water effluent line.

   Final - Samples are taken after chlorination, just prior to discharge to stream.

   Upstream - Grab Samples are taken at least 100 feet upstream of plant outfall on West Branch of the Neshaminy Creek.

   Downstream - Grab samples are taken approximately 150 feet downstream of plant outfall along the Neshaminy Creek.

2. All values are reported as average values for the test period, except pH values, which are median values.

3. A testing schedule for laboratory sampling is contained in Table D-1.

4. Data for the period April 1974 through December 1974 is included, although it is beyond, and not a part of, the test period.

5. With respect particularly to upstream and downstream COD, at many times of the year, the only flow upstream is the effluent from the Lansdale Sewage Treatment Plant, which operates as a secondary treatment plant sometimes, but mostly as a primary treatment plant. Therefore, although the values appear rather varied, they were reported as entered.

TABLE D-1. GRANT PERIOD LABORATORY TESTING SCHEDULE

TEST \ Sample Point	Montgomeryv'l Raw Influent	Hatfield Raw Influent	Combined Raw Influent	Surge Storage #1 Effluent	Surge Storage #2 Effluent	Primary #1 Effluent	Primary #2 Effluent	Secondary #1 Effluent	Secondary #2 Effluent	Tertiary Effluent	Filter Effluent	Final Effluent	Upstream	Downstream
BOD_5	W	W	WRF	WRF	WRF	WRF	WRF	WRF	WRF	WRF	WRF	WRF	WRF	WRF
COD			MWF	MWF	MWF	MWF	MWF	MWF	MWF	MWF	MWF	MWF		
P-Total	W	W	M-F	M-F	M-F	M-F	M-F	M-F	M-F	M-F	M-F	M-F	MWF	MWF
P-Ortho	W	W	M-F	M-F	M-F	M-F	M-F	M-F	M-F	M-F	M-F	M-F	MWF	MWF
P-Total Soluble			T-R	T-R	T-R	T-R	T-R	T-R	T-R	T-R	T-R	T-R		
P-Soluble Ortho			T-R	T-R	T-R	T-R	T-R	T-R	T-R	T-R	T-R	T-R		
NH_3-N			M-F			M-F		M-F				M-F		
TKN			M-F			M-F		M-F				M-F		
NO_2-N			M-F			M-F		M-F				M-F		
NO_3-N			M-F			M-F		M-F				M-F		
T.S.S.	W	W	M-F	M-F	M-F	M-F	M-F	M-F	M-F	M-F	M-F	M-F	M-F	M-F
T.S.			R	R		R		R		R	R	R		
V.S.S.	W	W	M-F	M-F	M-F	M-F	M-F	M-F	M-F	M-F	M-F	M-F	M-F	M-F
V.S.			R	R		R		R		R	R	R		
Fecal Coliform			R									R		

M = Monday, T = Tuesday, W = Wednesday, R = Thursday, F = Friday

APRIL 1973 DATA

Parameter	Raw	Surge	Prim.	Sec.	Tert.	Filt.	Final	Upstream	Downstream
COD	195.0	574.4	316.2	175.7	-	67.8	78.3	29.3	10.2
BOD_5	76.8	115.7	74.5	34.6	14.9	7.7	2.9	3.9	4.0
Org-N	36.2	68.1	40.0	-	-	-	27.2	-	-
NH_3-N	48.2	11.5	48.4	24.4	33.1	-	52.6	-	-
NO_2-N	0.06	0.18	0.14	0.11	0.06	0.08	0.05	0.13	0.10
NO_3-N	0.08	0.24	0.02	0.06	0.09	0.18	0.06	0.38	0.26
TKN	84.4	79.6	88.4	-	-	-	79.8	-	-
pH*	6.9	7.5	8.7	8.7	7.8	7.9	7.9	7.1	7.5
P Total	5.1	9.0	3.7	2.4	1.2	1.2	0.7	1.6	1.5
P St	3.24	4.57	1.6	1.81	1.1	0.4	0.3	1.60	1.1
SS	85.0	1447.0	112.0	101.0	16.0	5.0	7.0	13.0	19.0
VSS%	71	65.5	56.8	54.8	-	64.5	62.2	41.4	39.0
TS	379.0	805.0	490.0	465.0	363.0	430.0	535.0	-	-
TVS%	35.9	29	15.7	24.7	17.4	13.7	16.5	-	-
Alk.	99.6	-	-	-	-	-	115.0	-	-

*Median Value

MAY 1973 DATA

May 1 - May 14, 1973

Parameter	Raw	Surge	Prim.	Sec.	Tert.	Filt.	Final	Upstream	Downstream
COD	252.9	558.5	178.9	13.8	72.8	59.9	51.7	-	-
BOD_5	93.1	139.6	60.7	19.4	5.2	6.1	1.3	4.6	1.7
Org-N	7.2	10.8	6.4	-	-	-	7.2	-	-
NH_3-N	37.8	47.8	59.9	-	-	-	97.4	-	-
NO_2-N	0.9	0.21	0.31	1.74	1.20	1.28	0.8	0.23	1.05
NO_3-N	0.2	0.12	0.16	0.28	0.19	0.26	0.3	0.69	0.34
TKN	45.0	58.6	66.3	-	-	-	-	-	-
pH*	7.1	7.8	8.8	7.7	7.6	7.2	7.0	7.0	7.2
P Total	6.5	22.3	2.3	2.2	0.6	0.4	0.7	2.0	1.1
P St	6.0	9.0	1.9	1.8	0.3	0.3	0.3	1.1	0.3
SS	89.1	653.7	46.7	18.3	5.0	6.5	52.1	33.3	54.3
VSS%	43.0	60.0	36.4	46.9	50.0	25.0	68.0	57.7	61.5
TS	583.5	1071.5	611.5	628.0	607.5	590.5	609.0	-	-
TVS%	32.0	27.2	14.2	14.3	11.3	11.2	13.0	-	-
Alk.	122.3	181.4	172.8	134.6	117.0	110.0	103.9	63.4	95.4

*Median Value

MAY 1973 DATA

May 15 - May 30, 1973

Parameter	Raw	Surge	Prim.	Sec.	Tert.	Filt.	Final	Upstream	Downstream
COD	346.4	600.5	149.98	89.9	-	51.5	29.5	-	-
BOD_5	109.3	124.0	60.3	27.3	10.5	14.8	4.9	4.5	4.7
Org-N	4.2	8.5	4.2	8.5	-	-	12.7	-	-
NH_3-N	34.0	59.4	72.2	67.9	-	-	29.7	-	-
NO_2-N	0.09	0.14	1.0	2.5	2.5	4.1	3.5	0.3	1.4
NO_3-N	0.15	0.04	0.3	0.5	1.5	1.0	0.2	0.8	0.4
TKN	38.2	67.9	76.4	76.4	-	-	42.4	-	-
pH*	7.0	7.8	8.8	7.7	7.6	7.1	6.9	7.0	7.1
P Total	9.9	34.4	3.9	3.7	0.8	0.6	0.8	3.2	2.3
P St	8.7	10.9	2.6	2.5	0.5	0.6	0.5	1.1	0.3
SS	123.8	6336.3	110.8	32.1	7.2	6.6	13.3	25.8	12.0
VSS%	48.9	48.2	51.3	42.4	54.9	90.0	76.1	71.2	60.0
TS	501.5	2061.5	594.0	568.5	552.0	562.5	515.0	-	-
TVS%	21.6	19.7	37.5	17.8	22.5	8.6	10.5	-	-
Alk.	-	-	-	-	-	-	-	-	-

*Median Value

JUNE 1973 DATA

Parameter	Raw	Surge	Prime.	Sec.	Tert.	Filt.	Final	Upstream	Downstream
COD	336.1	-	781.8	269.9	244.8	289.2	223.4	270.7	494.5
BOD_5	104.7	146.2	124.9	25.0	9.2	9.1	4.3	5.4	5.6
Org-N	2.8	-	-	-	-	-	1.4	-	-
NH_3-N	38.5	-	-	-	-	-	18.1	-	-
NO_2-N	0.5	0.8	1.9	2.2	2.0	2.4	2.4	0.3	1.2
NO_3-N	0.2	0.1	0.4	1.4	1.4	1.7	1.5	0.4	0.5
TKN	41.3	-	-	-	-	-	19.5	-	-
pH*	7.4	7.8	8.5	7.6	7.5	7.0	7.1	-	-
P Total	8.1	21.5	3.9	4.9	1.1	0.6	0.5	3.4	3.7
P St	5.9	28.0	5.9	2.5	0.5	0.5	0.4	1.3	1.6
SS	116.0	-	-	118.0	18.6	9.4	11.7	31.0	34.5
VSS%	83.7	35.9	48.4	52.1	58.3	82.0	74.0	58.9	65.5
TS	695.0	-	1019.0	840.0	673.0	699.0	715.0	352.0	518.0
TVS%	37.3	35.3	38.8	30.2	26.7	27.9	28.0	26.4	24.2
Alk.	157.8	466.9	253.5	161.9	120.8	113.9	124.0	69.2	83.4

*Median Value

JULY 1973 DATA

Parameter	Raw	Surge	Prim.	Sec.	Tert.	Filt.	Final	Upstream	Downstream
COD	601.2	445.4	220.7	264.1	234.1	-	280.7	-	-
BOD_5	111.0	124.8	66.0	8.5	2.3	10.9	6.5	11.0	9.5
Org-N	10.3	-	-	-	-	-	1.6	-	-
NH_3-N	60.7	-	-	-	-	-	30.7	-	-
NO_2-N	0.1	0.2	0.2	1.9	2.5	2.1	1.9	1.1	1.2
NO_3-N	0.7	0.2	0.1	1.8	0.9	1.1	0.9	0.2	0.3
TKN	71.0	-	-	-	-	-	32.3	-	-
pH*	7.1	6.7	8.1	6.9	6.1	6.1	5.9	-	-
P Total	7.9	21.5	3.1	3.2	0.3	1.2	0.6	3.5	2.0
P St	6.4	26.6	2.2	1.9	0.0	0.0	0.0	2.2	2.2
SS	232.0	1058.0	105.4	77.0	17.5	12.3	13.9	35.5	35.8
VSS%	87.1	49.7	69.2	64.7	59.5	71.9	57.4	36.7	55.4
TS	1252.0	-	810.0	835.0	775.0	789.0	850.0	514.0	600.0
TVS%	43.3	35.9	27.0	24.5	26.5	21.2	23.5	27.5	23.5
Alk.	-	-	-	-	-	15.9	130.0	-	-

*Median Value

AUGUST 1973 DATA

Parameter	Raw	Surge	Prim.	Sec.	Tert.	Filt.	Final	Upstream	Downstream
COD	720.8	444.6	441.7	226.4	434.5	374.0	338.8	336.5	309.8
BOD_5	65.0	78.4	59.3	8.5	2.3	2.4	2.6	3.4	2.6
Org-N	3.5	-	-	-	-	-	2.1	-	-
NH_3-N	-	-	28.7	24.7	-	-	0.5	-	-
NO_2-N	0.09	0.09	0.15	0.90	0.77	0.64	0.50	0.43	0.60
NO_3-N	0.0	0.0	0.0	0.7	0.5	0.8	1.0	0.2	0.3
TKN	11.2	-	-	-	-	-	2.4	-	-
pH*	7.7	7.6	8.3	8.1	7.2	7.1	6.8	-	-
P Total	6.1	19.6	2.1	1.7	0.3	0.4	0.2	2.2	1.6
P St	6.0	9.1	2.1	0.8	0.1	0.3	0.5	2.4	1.6
SS	116.8	688.2	68.5	32.9	9.9	8.1	8.1	13.8	10.0
VSS%	82.4	48.8	77.1	66.1	69.9	54.0	61.5	-	-
TS	938.0	2866.0	2494.0	495.0	960.0	880.0	806.0	534.0	679.0
TVS%	38.8	40.4	15.0	40.5	32.7	35.9	36.2	-	-
Alk.	240.6	221.5	253.5	122.1	130.3	97.3	101.7	167.4	121.1

*Median Value

SEPTEMBER 1973 DATA

Parameter	Raw	Surge	Prim.	Sec.	Tert.	Filt.	Final	Upstream	Downstream
COD	324.0	368.8	276.1	222.7	232.6	198.7	197.8	179.7	160.1
BOD_5	94.6	120.9	66.1	14.1	17.0	9.7	5.8	13.6	10.4
Org-N	-	-	-	-	-	-	-	-	-
NH_3-N	-	0.47	0.31	-	-	0.23	-	-	-
NO_2-N	0.1	0.0	0.3	0.1	1.4	1.3	1.3	0.3	4.9
NO_3-N	0.3	1.0	0.6	0.7	0.7	1.1	0.6	0.3	0.3
TKN	-	-	-	-	-	-	-	-	-
pH*	8.1	8.0	8.4	8.1	7.2	7.4	6.7	-	-
P Total	12.9	21.8	5.0	6.2	0.5	2.9	-	5.3	3.6
P St	6.0	9.1	2.1	0.9	2.4	0.9	-	2.4	1.6
SS	372.5	1579.3	-	204.5	-	12.8	4.7	4.1	2.7
VSS%	82.2	65.8	70.9	79.3	62.8	66.2	78.4	62.5	57.4
TS	542.5	953.5	804.5	841.0	825.0	744.0	856.0	930.0	854.0
TVS%	17.8	26.3	14.0	18.6	35.9	11.7	11.7	65.4	11.2
Alk.	196.4	154.5	136.0	101.9	121.2	84.2	132.6	112.9	134.2

*Median Value

OCTOBER 1973 DATA

Parameter	Raw	Surge	Prim.	Sec.	Tert.	Filt.	Final	Upstream	Downstream
COD	407.8	299.4	187.8	137.6	310.3	118.4	109.6	138.7	96.9
BOD_5	51.3	54.6	43.0	11.2	5.2	4.3	2.0	6.0	3.3
Org-N	-	-	-	-	-	-	-	-	-
NH_3-N	50.7	47.0	33.8	14.1	4.4	4.4	5.6	5.6	5.4
NO_2-N	0.2	0.1	0.5	0.4	0.6	0.3	0.3	1.1	1.2
NO_3-N	0.2	0.2	0.6	1.7	1.2	1.3	1.2	1.1	1.2
TKN	-	-	-	-	-	-	-	-	-
pH*	8.1	8.3	9.3	7.9	7.1	7.3	7.1	-	-
P Total	6.9	8.0	4.8	1.7	0.9	0.8	0.2	4.7	4.6
P St	5.3	5.8	3.2	0.6	0.3	0.9	0.3	1.8	4.4
SS	246.3	671.0	93.2	50.5	12.2	10.0	16.4	15.2	19.2
VSS%	67.4	61.6	61.5	55.5	39.2	53.3	43.5	54.9	61.8
TS	1139.0	1490.0	918.0	860.0	973.0	958.0	891.0	319.0	550.0
TVS%	16.6	21.7	19.0	39.4	27.9	22.0	22.0	20.7	13.5
Alk.	166.8	195.3	214.0	192.3	165.0	160.5	205.3	165.6	136.3

*Median Value

NOVEMBER 1973 DATA

Parameter	Raw	Surge	Prim.	Sec.	Tert.	Filt.	Final	Upstream	Downstream
COD	223.8	297.8	219.0	160.5	195.6	116.6	97.6	138.4	132.0
BOD_5	72.2	72.2	52.5	7.7	5.5	4.0	3.1	7.3	5.0
Org-N	-	-	-	-	-	-	-	-	-
NH_3-N	54.9	-	35.6	17.1	6.2	5.0	4.0	4.8	5.1
NO_2-N	0.08	0.07	0.10	0.11	0.09	0.09	0.05	0.13	0.12
NO_3-N	3.0	3.0	1.08	23.2	24.3	19.9	12.8	29.4	31.5
TKN	-	-	-	-	-	-	-	-	-
pH*	8.4	8.7	9.3	9.1	7.2	7.6	7.3	7.8	7.9
P Total	3.9	4.11	3.9	0.8	0.4	0.4	0.3	1.0	0.8
P St	3.24	3.20	2.0	0.68	0.40	0.38	0.32	0.77	0.70
SS	342.0	685.1	50.7	49.9	12.9	7.0	4.9	12.2	12.5
VSS%	54.5	56.9	41.3	63.8	48.3	49.1	46.6	47.8	40.5
TS	1630.0	1344.0	712.0	931.0	847.0	809.0	800.0	652.0	654.0
TVS%	48.0	36.0	17.0	22.0	14.0	24.0	24.0	15.0	35.0
Alk.	162.8	241.7	172.6	111.9	119.8	152.8	181.9	167.0	142.8

*Median Value

DECEMBER 1973 DATA

Parameter	Raw	Surge	Prim.	Sec.	Tert.	Filt.	Final	Upstream	Downstream
COD	256.7	395.4	241.1	140.0	173.1	183.0	143.0	178.4	198.1
BOD_5	109.0	134.0	118.0	92.0	22.0	19.0	16.0	15.0	21.0
Org-N	15.0	-	13.0	-	-	-	0.7	-	-
NH_3-N	31.0	60.0	31.0	30.0	5.8	5.7	1.7	-	-
NO_2-N	-	-	-	-	-	-	-	-	-
NO_3-N	0.6	0.9	0.1	0.6	0.0	0.1	0.1	0.0	0.0
TKN	46.0	-	44.0	-	-	-	2.4	-	-
pH*	8.0	7.9	8.4	8.3	7.3	7.5	7.7	-	-
P Total	7.2	7.0	2.9	1.1	0.3	0.2	0.1	1.9	1.8
P St	7.5	7.1	3.4	1.3	0.1	0.1	0.1	2.3	2.2
SS	269.0	1409.0	156.0	25.0	15.0	50.0	11.0	14.0	61.0
VSS%	64.0	64.0	59.0	71.0	61.0	55.0	55.0	41.0	44.0
TS	1392.0	5493.0	1283.0	841.0	894.0	866.0	735.0	513.0	671.0
TVS%	17.0	46.0	31.0	13.0	12.0	12.0	12.0	12.0	10.0
Alk.	193.8	238.8	146.6	115.9	97.5	151.3	132.1	120.9	104.4

*Median Value

JANUARY 1974 DATA

Parameter	Raw	Surge	Prim.	Sec.	Tert.	Filt.	Final	Upstream	Downstream
COD	-	-	-	-	-	-	-	-	-
BOD_5	81.1	64.6	59.6	51.4	24.1	9.9	3.0	5.8	5.3
Org-N	-	-	-	-	-	-	-	-	-
NH_3-N	44.1	-	-	-	-	-	1.2	-	-
NO_2-N	-	-	-	-	-	-	-	-	-
NO_3-N	-	-	0.05	-	-	-	-	-	-
TKN	-	-	-	-	-	-	-	-	-
pH*	7.4	7.4	8.3	7.4	7.2	7.3	7.1	7.5	7.4
P Total	2.9	4.5	2.8	3.4	0.6	-	0.5	0.7	0.8
P St	2.3	-	-	-	-	-	0.01	-	-
SS	58.6	149.0	56.3	60.2	24.7	5.0	8.4	9.9	14.1
VSS%	76.1	69.7	68.9	58.7	63.3	52.0	68.1	77.6	62.8
TS	-	-	-	-	-	-	-	-	-
TVS%	-	-	-	-	-	-	-	-	-
Alk.	-	-	-	-	-	-	-	-	-

*Median Value

FEBRUARY 1974 DATA

Parameter	Raw	Surge	Prim.	Sec.	Tert.	Filt.	Final	Upstream	Downstream
COD	-	-	-	-	-	-	-	-	-
BOD_5	99.7	112.6	100.5	25.4	17.4	13.1	3.8	9.7	8.2
Org-N	-	-	-	-	-	-	-	-	-
NH_3-N	82.0	-	-	-	-	7.6	10.1	-	-
NO_2-N	-	-	-	-	-	-	-	-	-
NO_3-N	-	-	-	-	-	-	-	-	-
TKN	-	-	-	-	-	-	-	-	-
pH*	8.1	8.4	9.1	7.9	7.7	7.7	7.7	8.2	8.1
P Total	4.2	6.1	2.6	1.5	0.8	0.7	0.7	0.5	0.7
P St	0.47	2.20	1.40	0.45	0.35	0.25	0.13	0.35	0.25
SS	235.2	1791.0	-	96.9	23.3	7.2	11.1	22.9	19.3
VSS%	70.1	54.6	63.1	64.0	49.9	63.4	54.9	50.6	40.9
TS	676.2	2931.0	-	857.3	795.3	739.0	-	-	-
TVS%	28.2	87.7	59.6	21.2	11.7	11.5	11.0	-	-
Alk.	-	-	-	-	-	-	-	-	-

*Median Value

MARCH 1974 DATA

Parameter	Raw	Surge	Prim.	Sec.	Tert.	Filt.	Final	Upstream	Downstream
COD	432.0	408.0	441.6	327.4	239.1	345.6	181.3	374.4	345.6
BOD_5	73.7	90.4	61.4	14.2	10.2	8.1	5.1	10.9	6.5
Org-N	-	-	-	25.2	-	-	40.7	-	-
NH_3-N	89.3	-	65.2	66.1	-	-	12.7	-	-
NO_2-N	0.06	0.06	0.07	0.02	0.06	0.06	0.10	0.07	0.07
NO_3-N	0.01	0.01	0.02	0.01	0.04	0.04	0.02	0.05	0.05
TKN	121.3	-	94.0	91.3	-	-	14.6	-	-
pH*	8.0	8.2	9.3	8.1	7.8	7.6	7.5	7.7	7.7
P Total	8.6	8.5	6.5	2.3	1.1	1.1	0.7	1.3	1.6
P St	4.2	4.7	4.3	1.9	0.8	0.9	0.7	1.4	1.6
SS	215.3	263.8	175.6	46.9	17.6	7.4	7.0	101.9	12.6
VSS%	73.2	64.2	63.7	66.5	57.4	61.7	50.3	48.6	55.4
TS	632.9	1255.0	684.1	565.9	660.1	602.1	702.7	-	-
TVS%	24.2	39.0	21.6	23.1	21.7	18.3	15.0	-	-
Alk.	-	-	340.0	100.0	-	-	180.0	-	-

*Median Value

APRIL 1974 DATA

Parameter	Raw	Surge	Prim.	Sec.	Tert.	Filt.	Final	Upstream	Downstream
COD	136.0	140.4	140.4	129.6	162.5	-	166.5	-	-
BOD_5	110.9	114.6	95.8	19.0	12.8	11.6	2.4	16.3	9.4
Org-N	3.9	-	17.9	41.4	-	-	4.0	-	-
NH_3-N	111.9	-	103.8	80.4	-	-	17.0	-	-
NO_2-N	0.11	-	0.11	0.29	-	-	0.14	-	-
NO_3-N	0.02	-	0.02	0.13	-	-	0.04	-	-
TKN	115.8	-	121.7	121.9	-	-	21.1	-	-
pH*	7.8	7.4	8.2	7.5	7.4	7.4	7.4	-	-
P Total	4.8	5.1	3.8	1.5	1.0	0.9	0.7	1.5	1.6
P St	4.0	4.0	3.3	3.1	0.8	0.7	0.6	1.6	1.9
SS	137.4	2626.0	110.5	35.6	9.7	7.0	6.3	6.3	12.8
VSS%	59.9	70.0	51.3	60.9	49.8	45.7	42.4	49.2	49.3
TS	431.0	1245.0	908.0	551.0	549.0	534.0	559.0	-	-
TVS%	15.5	39.0	34.9	13.3	15.5	17.4	17.4	-	-
Alk.	129.6	-	198.0	156.8	123.3	122.8	116.1	-	-

*Median Value

MAY 1974 DATA

Parameter	Raw	Surge	Prim.	Sec.	Tert.	Filt.	Final	Upstream	Downstream
COD	136.3	145.7	145.7	169.0	206.0	-	220.4	-	-
BOD_5	102.7	123.1	90.9	18.1	7.3	5.8	4.6	12.4	11.1
Org-N	6.4	-	6.9	23.7	-	-	3.2	-	-
NH_3-N	31.3	-	36.3	27.2	-	-	7.7	-	-
NO_2-N	0.23	-	0.23	0.43	-	-	0.43	-	-
NO_3-N	0.04	-	0.03	0.20	-	-	0.19	-	-
TKN	58.0	-	43.2	50.8	-	-	10.9	-	-
pH*	7.5	7.8	8.9	7.6	7.3	7.3	7.3	-	-
P Total	5.4	5.3	4.1	1.6	1.1	1.2	0.9	1.3	1.3
P St	4.8	4.7	3.5	3.0	0.8	0.7	0.5	0.9	1.0
SS	148.9	-	179.0	29.5	6.2	4.3	3.2	3.2	12.9
VSS%	59.7	85.8	66.8	59.9	54.3	49.8	54.0	62.5	50.9
TS	730.0	1245.0	733.0	743.0	710.0	696.0	710.0	-	-
TVS%	19.9	55.0	17.6	18.2	19.9	15.9	15.9	-	-
Alk.	249.3	138.9	406.2	331.4	135.6	129.8	123.9	-	-

*Median Value

JUNE 1974 DATA

Parameter	Raw	Surge	Prim.	Sec.	Tert.	Filt.	Final	Upstream	Downstream
COD	388.8	429.8	429.8	430.4	340.0	-	435.0	-	-
BOD_5	138.1	137.6	96.8	18.0	8.4	4.7	2.3	14.1	14.4
Org-N	7.3	-	10.2	25.9	-	-	3.4	-	-
NH_3-N	96.3	-	72.4	49.4	-	-	2.1	-	-
NO_2-N	0.12	0.02	0.18	1.37	0.04	-	0.63	0.04	0.04
NO_3-N	0.02	0.00	0.03	0.18	0.02	0.20	0.07	-	-
TKN	103.5	-	82.6	75.3	-	-	5.7	-	-
pH*	7.4	7.6	8.3	6.9	6.7	7.1	6.7	-	-
P Total	4.8	4.8	4.1	1.6	0.7	0.9	0.8	1.6	1.6
P St	4.8	4.6	3.5	1.2	1.0	0.8	0.7	1.6	1.6
SS	128.8	3938.0	179.0	17.2	4.4	2.6	1.8	1.8	38.8
VSS%	66.2	75.6	66.8	69.4	58.8	68.5	60.2	59.7	51.9
TS	958.0	1520.0	733.0	1006.0	792.0	784.0	771.0	300.0	446.0
TVS%	23.9	35.1	17.6	24.1	23.9	23.9	23.9	34.3	28.9
Alk.	291.3	390.0	406.2	-	-	-	-	-	-

*Median Value

JULY 1974 DATA

Parameter	Raw	Surge	Prim.	Sec.	Tert.	Filt.	Final	Upstream	Downstream
COD	384.0	364.0	337.0	386.0	335.0	-	368.0	-	-
BOD_5	144.7	137.6	102.2	14.1	2.8	2.7	2.1	1.3	2.5
Org-N	8.0	-	14.3	19.3	-	-	2.8	-	-
NH_3-N	80.0	-	64.0	55.0	-	-	6.0	-	-
NO_2-N	0.15	0.02	0.30	0.69	0.59	0.45	0.39	0.35	0.38
NO_3-N	0.02	0.00	0.04	0.21	0.11	0.07	0.06	0.01	0.01
TKN	88.0	-	78.0	79.0	-	-	5.7	-	-
pH*	7.5	7.6	7.8	7.2	6.8	7.1	6.7	-	-
P Total	6.8	4.8	6.1	6.4	1.5	1.1	0.8	1.3	1.4
P St	6.7	4.6	5.9	2.2	1.5	1.5	0.7	1.1	1.2
SS	168.6	3938.0	30.9	16.0	6.6	4.9	1.8	2.6	21.5
VSS%	87.2	75.6	76.0	58.7	54.8	40.1	60.2	54.9	54.3
TS	864.0	1520.0	765.0	823.0	813.0	781.0	771.0	-	-
TVS%	10.1	35.1	18.8	13.7	10.1	11.0	23.9	-	-
Alk.	205.0	390.0	218.0	222.0	183.0	171.0	-	-	-

*Median Value

AUGUST 1974 DATA

Parameter	Raw	Surge	Prim.	Sec.	Tert.	Filt.	Final	Upstream	Downstream
COD	-	-	-	100.4	152.0	-	151.5	-	-
BOD_5	138.7	138.3	131.4	18.6	8.1	6.2	5.6	6.9	6.7
Org-N	2.6	-	2.4	8.2	-	-	1.4	-	-
NH_3-N	24.8	-	20.1	15.8	-	-	2.6	-	-
NO_2-N	0.30	0.28	0.37	1.05	0.80	0.60	0.70	0.48	0.48
NO_3-N	0.02	0.04	0.06	0.20	0.21	0.12	0.11	0.03	0.03
TKN	25.0	-	-	-	-	-	2.5	-	-
pH*	7.7	7.8	8.1	7.1	6.9	6.7	6.1	-	-
P Total	9.6	9.5	8.9	2.4	1.4	1.2	1.2	1.6	1.7
P St	9.8	9.8	9.1	2.3	1.5	1.2	1.1	1.3	1.5
SS	-	2253.0	44.3	24.9	4.7	2.1	1.9	2.1	11.0
VSS%	75.1	76.7	64.7	67.1	55.5	55.0	55.7	66.3	69.3
TS	149.5	-	928.0	937.0	906.0	833.0	788.0	-	-
TVS%	12.5	-	22.8	17.4	10.5	50.8	50.8	-	-
Alk.	355.0	-	350.0	90.0	80.0	40.0	-	-	-

*Median Value

SEPTEMBER 1974 DATA

Parameter	Raw	Surge	Prim.	Sec.	Tert.	Filt.	Final	Upstream	Downstream
COD	578.6	624.0	587.0	303.2	224.0	-	-	-	-
BOD_5	145.4	162.2	131.0	23.4	9.8	7.1	6.5	5.9	6.3
Org-N	2.7	-	2.0	5.6	-	-	1.3	-	-
NH_3-N	22.1	-	16.8	13.8	-	-	0.5	-	-
NO_2-N	-	0.11	0.09	0.77	0.49	0.70	0.26	-	-
NO_3-N	0.03	0.03	0.03	0.36	0.19	0.11	0.11	0.08	0.08
TKN	24.8	-	18.8	18.1	-	-	1.5	-	-
pH*	7.9	7.3	8.2	7.4	7.2	7.2	7.3	-	-
P Total	7.3	8.6	6.7	2.3	1.4	0.94	1.14	1.70	1.75
P St	6.5	7.6	5.8	1.8	1.3	1.15	0.91	-	-
SS	-	1975.0	33.8	30.8	4.2	3.4	2.8	2.77	-
VSS%	-	62.5	60.4	66.7	58.9	53.6	52.4	57.3	43.1
TS	-	4338.0	699.5	676.0	388.0	800.0	344.0	-	565.0
TVS%	37.3	46.4	41.3	22.8	37.3	36.6	36.6	14.9	-
Alk.	-	-	286.0	280.0	239.5	229.0	218.5	-	-

*Median Value

OCTOBER 1974 DATA

Parameter	Raw	Surge	Prim.	Sec.	Tert.	Filt.	Final	Upstream	Downstream
COD	-	-	-	-	-	-	-	-	-
BOD_5	140.2	147.7	120.5	20.4	5.5	5.4	3.8	5.2	3.0
Org-N	2.6	-	-	-	-	-	-	-	-
NH_3-N	25.4	-	19.2	17.1	10.0	-	0.83	-	-
NO_2-N	-	-	-	-	-	-	-	-	-
NO_3-N	-	-	-	-	-	-	-	-	-
TKN	28.0	-	22.8	21.5	11.0	-	1.5	-	-
pH*	7.8	7.7	8.8	7.4	7.0	7.0	7.0	7.1	7.0
P Total	5.8	5.6	4.3	1.6	1.6	1.0	0.69	0.3	1.42
P St	-	-	-	-	-	-	-	-	-
SS	87.0	78.9	63.4	-	8.4	4.0	2.87	5.3	9.2
VSS%	-	-	-	-	-	-	-	-	-
TS	-	-	-	-	-	-	-	-	-
TVS%	-	-	-	-	-	-	-	-	-
Alk.	-	-	-	-	-	-	-	-	-

*Median Value

NOVEMBER 1974 DATA

Parameter	Raw	Surge	Prim.	Sec.	Tert.	Filt.	Final	Upstream	Downstream
COD	128.0	-	432.0	464.0	48.0	-	16.0	-	-
BOD_5	99.8	-	-	-	-	-	4.6	-	-
Org-N	3.7	-	-	-	-	-	0.6	-	-
NH_3-N	18.9	-	-	-	-	-	1.09	-	-
NO_2-N	0.15	0.13	0.12	0.72	0.56	0.32	1.93	-	-
NO_3-N	0.02	0.03	0.03	0.13	0.27	0.22	0.28	-	-
TKN	21.9	-	-	-	-	-	1.6	-	-
pH*	-	-	-	-	-	-	-	-	-
P Total	7.0	7.5	6.5	1.1	1.05	1.15	0.85	1.6	1.6
P St	-	-	-	-	-	-	-	-	-
SS	107.1	4339.0	79.7	24.5	8.2	3.5	4.2	4.0	6.0
VSS%	-	-	-	-	-	-	-	-	-
TS	954.0	-	862.0	809.0	820.0	799.0	802.0	-	-
TVS%	50.9	-	73.8	66.7	79.9	66.1	58.4	-	-
Alk.	-	-	-	-	-	-	-	-	-

*Median Value

DECEMBER 1974 DATA

Parameter	Raw	Surge	Prim.	Sec.	Tert.	Filt.	Final	Upstream	Downstream
COD	150.7	112.0	181.3	241.3	315.6	-	345.6	-	-
BOD_5	67.5	123.8	87.8	39.3	11.9	8.1	1.3	-	-
Org-N	4.4	-	2.9	2.9	-	-	1.3	-	-
NH_3-N	11.0	-	8.8	5.1	-	-	4.0	-	-
NO_2-N	0.14	0.07	0.8	0.41	0.52	-	0.50	-	-
NO_3-N	0.02	0.11	0.02	0.06	0.03	0.03	0.12	-	-
TKN	15.4	-	11.7	8.0	-	-	5.3	-	-
pH*	-	-	-	-	-	-	-	-	-
P Total	7.1	7.0	6.3	4.4	4.4	1.3	0.65	-	-
P St	6.8	7.0	6.3	3.1	1.0	0.7	0.60	-	-
SS	56.5	547.9	95.1	45.6	11.7	10.3	3.5	-	-
VSS%	77.9	58.7	63.4	71.1	73.5	69.0	63.8	-	-
TS	117.5	1844.0	556.0	353.0	306.5	530.0	331.5	-	-
TVS%	78.2	65.2	74.4	62.8	46.8	79.4	81.3	-	-
Alk.	186.7	195.0	238.3	243.3	188.3	174.4	164.0	-	-

*Median Value

APPENDIX E

HATFIELD TOWNSHIP MUNICIPAL AUTHORITY

REVIEW PRESENT DESIGN FOR ABILITY TO ACHIEVE

ADDITIONAL NITROGENOUS BOD REMOVAL

DESIGN BASIS

Q_{AVE}	=	3.6 mgd		13.626×10^3 cu m/day
BOD_5	=	7050 Lb/Day	(235 mg/1)	3200.7 Kg/Day
SS	=	7050 Lb/Day	(235 mg/1)	3200.7 Kg/Day
$NH_3 \cdot N$	=	900 Lb/Day	(30 mg/1)	408.6 Kg/Day

LOAD TO AERATION SYSTEM

BOD Removal Across Primary Treatment Units With Lime Addition Estimated To Be 60%

∴ BOD_5 To Aeration

0.4 (3201) = 1280 Kg
0.40 (7050) = 2820 Lb. BOD_5/Day

$NH_3 \cdot N$ To Aeration Estimated to be 67% of Influent $NH_3 \cdot N$

(See: Systems for Phosphate and Nitrogen Removal, by O.E. Albertson, presented at the 41st Annual Conference of Water Pollution Control Association of Pennsylvania, University Park, Pennsylvania, August 6, 1969)

0.67 (409 = 274 kg
0.67 (900) = Lb. $NH_3 \cdot N$/Day

Aeration Tanks - Two (2) 15.2 m sq x 4.6 m
50' sq x 15' SWD
(1060 cu m each)

Detention Time @ Q_{AVE}

$$\frac{2120 \text{ cum}}{13.6 \text{ cum/day/24 hr}} = 3.73 \text{ Hr.}$$

Aeration System Has Been Sized to Provide:

2052 kg O_2/Day @

α = 0.8
B = 0.95
T = 25° C
DO = 2 mg/1
Elev = 300'

which is equivalent to:

3564 kg O_2/Day @

α = 1.0
B = 1.0
T = 20° C
DO = Zero
Elev = Sea Level

Calculate Nitrification Completed at Winter Wastewater Temperature if 10.75° C (~ 51° F) (After Downing)

BOD Loading Rate @ 4000 mg/1 MLSS

$$\frac{4000}{10^6} \times 0.56(10^6) \times 8.34 = \text{MLSS} \times 0.454 \frac{\text{kg}}{\text{lb}} = 8489 \text{ kg MLSS}$$

$$\frac{1280 \text{ kg } BOD_5}{8489 \text{ kg MLSS}} = 0.151 \text{ kg BOD/kg MLSS}$$

Assume Use of 0.95 kg O_2/kg BOD Applied (@ 10.75° C) For Metabolism (Into The Mixed Liquor)

For Nitrification Use 46 kg O_2/kg $NH_3 \cdot N$ Applied (Into Mixed Liquor)

∴ Total O_2 Required

BOD_5 = 0.93 (1280) = 1190 kg O_2/Day

$NH_3 \cdot N$ = 4.6 (274) = 1260 kg O_2/Day

Σ = 2450 kg O_2/Day

Into The Mixed Liquor

which is equivalent to:

4022 kg O_2/Day @

α = 1.0
B = 0.95
T = 20° C
DO = Zero
Elev = Sea Level

Apparent O_2 Deficit

$$\begin{array}{r} 4022 \\ -3564 \\ \hline 458 \end{array}$$ kg O_2/Day @ Standard Conditions

Or 279 kg O_2/Day @ Design Conditions (10.75° C)

$$\begin{array}{r} 1260 \\ -\ 279 \\ \hline 981 \end{array}$$ kg O_2/Day Available For Nitrification

$$\frac{981}{1260} = \sim 78\% \text{ Nitrification}$$

Calculate Nitrification Completed at Summer Wastewater Temperature of 20° C (~77° F) (After Sawyer)

Use 0.175 gm NH_3 Nitrified/Day per gm MLVSS

$$\frac{274 \text{ kg } NH_3 \cdot N}{0.175 \text{ kg } NH_3 \cdot N/\text{kg MLVSS}} = 1566 \text{ kg MLVSS}$$

Assume MLVSS = 0.75 MLSS

$$\text{MLSS} = \frac{1566}{0.75}$$

$$= 2088 \text{ kg MLSS}$$

BOD Loading Rate

$$\frac{1280}{2088} = 0.614 \text{ kg BOD/kg MLSS}$$

Use 0.77 kg O_2/kg BOD Applied For Metabolism (Into Mixed Liquor)

$$\begin{array}{lllr} \therefore \text{ Total } O_2 \text{ Required} = & BOD_5 - 0.77\ (1280) & = & 985.8 \text{ kg } O_2\text{/Day} \\ & NH_3 \cdot N - 4.6\ (274) & = & \underline{1260.0 \text{ kg } O_2\text{/Day}} \\ & & & 2245.8 \text{ kg } O_2\text{/Day} \end{array}$$

Into The Mixed Liquor

which is equivalent to:

3895 kg O_2/Day @

α = 1.0

B = 0.95

T = 20° C

DO = Zero

Elev = Sea Level

Apparent O_2

$$\begin{array}{r} 4022 \\ \underline{-3895} \\ 127 \end{array}$$ kg O_2/Day @ Standard Conditions

Or 72.2 kg O_2/Day @ Design Conditions (25° C)

$$\begin{array}{r} 1260 \\ \underline{-\quad 72} \end{array}$$

1188 kg O_2/Day Available for Nitrification

$$\frac{1188}{1260} = \sim 94.2\% \text{ Nitrification}$$

APPENDIX F

GENERATION OF SLUDGES

A. CONVENTIONAL TREATMENT

Given:	Flow =	1 mgd
	BOD_5 =	200 mg/l
	TSS =	250 mg/l
	0.5 #WAS/#BOD_5	
Primary Efficiency	60% TSS Removal	
	40% BOD_5 Removal	

250 mg/l x 8.34 x 1.0 x 0.6 = 1,251 Lb/Day Primary

$200\ \text{mg/l} \times 8.34 \times 1.0 \times 0.60 \times 0.5\ \frac{\#\text{WAS}}{\#\text{BOD}_5} =$ 500 Lb/Day WAS

1,751 D.S./mg Flow

B. GENERATION OF SLUDGES FROM THE HATFIELD AWT FACILITY

Assumptions:	Flow=	1 mgd
	BOD_5 =	200 mg/l
	TSS =	250 mg/l
	P Total	10 mg/l
	0.5 #WAS/#BOD_5	
	420 mg/l $Ca(OH)_2$ to achieve pH 9.5	
	150 mg/l Alum	
Primary Efficiency:	80% TSS Removal	
	80% P Removal	
	60% BOD_5 Removal	

250 mg/l x 8.34 x 1 mgd x 0.8 = 1668 lb/day

Chemical Sludges:

420 mg/1 $Ca(OH)_2$ x 8.34 x 1.0 mgd = 3503 lb/mg

3503 lb/mg x 0.54 lb. Ca++/Lb. $Ca(OH)_2$ = 1892 lb/mg Ca++ fed

P Removed = 8 mg/1

8 mg/1 x 5.4 Lb. $Ca_5OH(PO_4)_3$/Lb. P x 8.34 x 1 mg = 360 lb/mg $Ca_5OH(PO_4)_3$

360 lb/mg x 0.398 $\frac{\text{Lb. Ca++}}{\text{Lb. } Ca_xP_y}$ = 133 lb. Ca++ consumed/mg

1892-133 = 1759 lb/mg x 2.5 = 4400 lb/mg $CaCO_3$

Secondary Treatment:

200 mg/1 BOD_5 x 0.4 x 0.5 $\frac{\text{Lb. WAS}}{\text{Lb. } BOD_5}$ x 8.34 = 330 lb/day WAS

Tertiary Treatment:

Alum Fed: 150 mg/1 x 8.34 x 1.0 mgd = 1251 lb/day

1251 Lb. $Al_2SO_4 \cdot 16\ H_2O$ x $\frac{54 \text{ Lb. } Al^{+3}}{\text{Lb. } Al_2(SO_4)_3 \cdot 14\ H_2O}$ =101.3 Lb. Al^{+3} Fed

P Removed = 2.0 mg/1

2.0 mg/1 P_x x 8.34 x 1.0 mgd - 16.7 lb/mg

16.7 lb/mg x 0.87 Lb. Al/Lb.P = 14.5 Lb. Al^{+3} Consumed

101.3-14.5 = 86.8 Lb. Al^{+3}/mg x 78/27 = 250 lb/mg as $Al(OH)_3$

14.5 Lb. Al^{+3} x 188/27 = 101 lb/mg $Al(PO_4) \cdot 2H_2O$

Sludge Summary:

Primary Sludge	1,668 lb/day
$CaCO_3$	4,400
$Ca_5OH(PO_4)_3$	360
WAS	330
$Al(OH)_3$ plus Al PO_4	351
	7,109 lb/day D.S./mg flow

TECHNICAL REPORT DATA
(Please read Instructions on the reverse before completing)

1. REPORT NO.	2.	3. RECIPIENT'S ACCESSION NO.
EPA-600/2-75-030		

4. TITLE AND SUBTITLE HATFIELD TOWNSHIP, PENNSYLVANIA, ADVANCED WASTE TREATMENT PLANT	5. REPORT DATE September 1975 (Issuing Date) 6. PERFORMING ORGANIZATION CODE
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16. ABSTRACT

The Hatfield Township, Pennsylvania, Water Pollution Control Plant was designed to encompass primary chemical treatment, secondary combined activated sludge and nitrification facilities, tertiary chemical tube clarification and mixed media filtration. The operation of the facility demonstrated that the use of flow equalization facilities improves plant operations by reducing and standardizing chemical concentrations. Phosphorus is removed efficiently in a combined primary-tertiary phase with operations personnel having the flexibility to optimize each process. Lime feed control by pH is easily accomplished, although recirculation of primary sludges is not always necessary. Tube clarifiers and mixed media filters combine to produce a highly polished effluent. Nitrification was observed to some extent in this modified facility, however, it was extremely difficult to control.

17. KEY WORDS AND DOCUMENT ANALYSIS

a. DESCRIPTORS	b. IDENTIFIERS/OPEN ENDED TERMS	c. COSATI Field/Group
Waste water*, Activated sludge process, Nitrification, Filtration*, Phosphorus	Phosphorus control, Effluent standards, Lime coagulation, Alum precipitation, Tertiary treatment*, Flow equalization	13B

18. DISTRIBUTION STATEMENT RELEASE TO PUBLIC	19. SECURITY CLASS *(This Report)* Unclassified	21. NO. OF PAGES 236
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