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Proceedings: Ninth United States/ Japan Conference on Sewage Treatment Technology

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PROCEEDINGS

NINTH UNITED STATES/JAPAN CONFERENCE ON SEWAGE TREATMENT TECHNOLOGY

September 19 - 21, 1983
Tokyo, Japan

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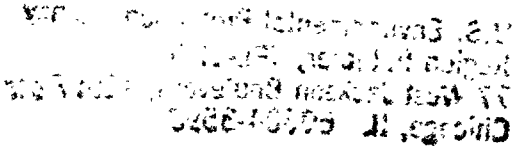

FOREWORD

The maintenance of clean water supplies and the management of municipal and industrial wastes are vital elements in the protection of the environment.

The participants in the United States-Japan cooperative project on sewage treatment technology have completed their Ninth Conference. These conferences, held at 18-month intervals, give the scientists and engineers of the cooperating agencies an opportunity to study and compare the latest practices and developments in the United States and Japan. These Proceedings of the Ninth Conference comprise a useful body of knowledge on sewage treatment, which will be available not only to Japan and the United States but also to all nations of the world who desire it.

William D. Ruckelshaus
Administrator

Washington, D.C.

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Tsutomu Tamaki Team Leader-Director Department of Sewerage and Sewage Purification City Bureau, Ministry of Construction	Current Topics on Sewage Works in Japan
Hiroshi Kurokawa Head, Planning Division Department of Sewerage and Sewage Purification City Bureau, Ministry of Construction	
Itaru Nakamoto Head, Sewage Works Division Department of Sewerage and Sewage Purification City Bureau, Ministry of Construction	Construction Technology Assessment System and Development of Mechanical Aerators for the Oxidation Ditch Process
Keiichi Fukui Head, Water Quality Control Division Public Works Research Institute Ministry of Construction	
Dr. Ken Murakami Chief, Water Quality Section Water Quality Control Division Public Works Research Institute Ministry of Construction	Current Status of Development in Automatic Water Quality Monitoring Equipment for Wastewater Treatment
Shunsoku Kyosai Chief, Ultimate Disposal Section Water Quality Control Division Public Works Research Institute Ministry of Construction	Upgrading of Anaerobic Digestion Process
Daisaku Sugito Head, Water Pollution Control Division Water Quality Bureau Environment Agency	Recent Developments in Japanese Water Quality Management Policy

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Dr. Kazuhiro Tanaka Chief, Section of Research and Technology Development Division of Research and Technology Development Japan Sewage Works Agency	Nitrogen and Phosphorus Biological Removal Systems Research
Dr. Nagaharu Okuno Head, Section of Engineering Research and Development Sewage Works Bureau Tokyo Metropolitan Government	Manufacturing Artificial Light Weight Aggregates from Sewage Sludge
Hiroshi Ouchi Senior Technical Advisor Construction Division Sewage Works Bureau Yokohama City Office	Study on Fluidized Bed Incineration with a Drying System
Kazunari Matsunaga Director, Construction Division Sewage Works Bureau Osaka City Office	Secondary Sewage Treatment Using Contact Aeration Process
Hiroo Nakagawa Deputy Director, Sewage Works Bureau Kobe City Office	Pilot Plant Study on Treatment of Supernatant from Thermal Sludge Treatment Process

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Delegates	Presentation Topics
John J. Convery - Delegation Leader Director Wastewater Research Division Municipal Environmental Research Laboratory U.S. Environmental Protection Agency Cincinnati, Ohio 45268	Removal and Partitioning of Volatile Organic Priority Pollutants in Wastewater Treatment
Henry L. Longest Director Office of Water Program Operations U.S. Environmental Protection Agency Washington, D.C. 20460	History of the Federal Construction Grants Program
Paul N. Guthrie Director, Office of Intergovernmental Programs Department of Natural Resources Madison, Wisconsin 53707	Clean Water in Wisconsin
Cecil Lue Hing Director Research and Development Metropolitan Sanitary District Chicago, Illinois 60611	Energy Recovery, Reuse and Conservation at Metro Chicago
Paul F. Gilbert Plant Engineer Metropolitan District Commission Hartford, Connecticut 06101	Coarse Bubble to Fine Bubble Aeration Retrofit
Gary R. Lubin Environmental Engineer Wastewater Research Division Municipal Environmental Research Laboratory U.S. Environmental Protection Agency Cincinnati, Ohio 45268	Energy Use at Municipal Wastewater Treatment Plants - Overview and Case Studies

*Addresses are correct as of September 21, 1983.

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Henry L. Longest Director Office of Water Program Operations U.S. Environmental Protection Agency Washington, D.C. 20460	History of the Federal Construction Grants Program
Paul N. Guthrie Director Office of Intergovernmental Programs Department of Natural Resources Madison, Wisconsin 53707	Clean Water in Wisconsin
Cecil Lue-Hing Director Research and Development Metropolitan Sanitary District Chicago, Illinois 60611	Sludge Management Plans and Practices at the Metropolitan Sanitary District of Greater Chicago
Paul F. Gilbert Plant Engineer Metropolitan District Commission Hartford, Connecticut 06101	Sludge Incinerator Fuel Reduction Program
Gary R. Lubin Environmental Engineer Wastewater Research Division Municipal Environmental Research Laboratory U.S. Environmental Protection Agency Cincinnati, Ohio 45268	The Biological Aerated Filter - Progress of Development in the United States

*Addresses are correct as of September 21, 1983

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BEING GREETED UPON ARRIVAL CHUO WASTEWATER TREATMENT PLANT
AIGAWA REGIONAL SEWERAGE SYSTEM - OSAKA

VISIT OF U.S. DELEGATION TO A SLUDGE SMELTER
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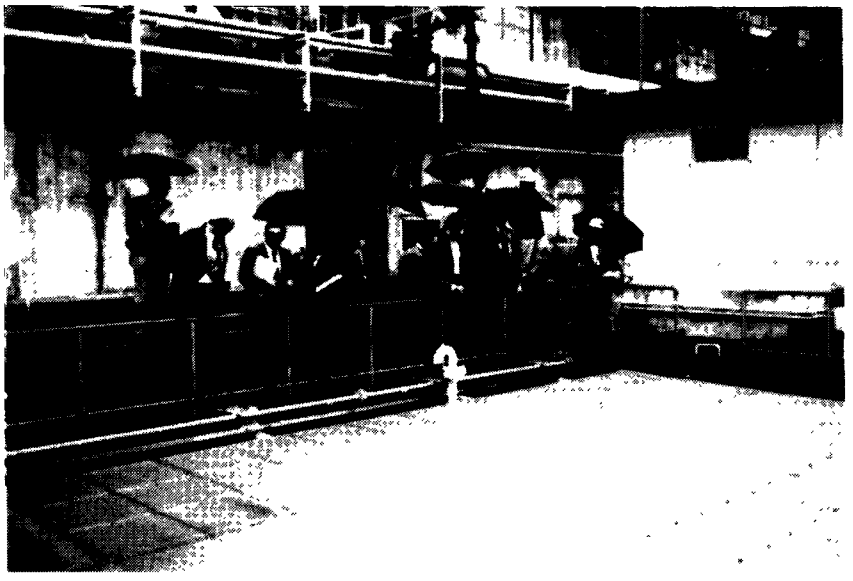
SITE VISIT TO PORT ISLAND NORTH PARK - KOBE



TOUR OF A CONTACT AERATION FACILITY
SUZURANDAI WASTEWATER TREATMENT PLANT - KOBE

JOINT COMMUNIQUE

Ninth United States/Japan Conference on
Sewage Treatment Technology

Wednesday, September 21, 1983, Tokyo, Japan

1. The ninth United States/Japan Conference on Sewage Treatment Technology was held in Tokyo from September 19 to 21, 1983.

2. The United States Delegation, headed by Mr. John J. Convery, Director, Wastewater Research Division, Municipal Environmental Research Laboratory, U.S. Environmental Protection Agency (USEPA), Cincinnati, Ohio, was composed of three representatives from the USEPA, one representative from the State of Wisconsin, and two representatives from local authorities (Metropolitan Sanitary District of Greater Chicago and Metropolitan District Commission of Hartford).

3. Mr. T. Tamaki, Director, Department of Sewerage and Sewage Purification, Ministry of Construction, was Head of the Japanese Delegation, which consisted of seven National Government representatives (six from the Ministry of Construction and one from the Environment Agency), two Japan Sewage Works Agency representatives, and four local government officials (Tokyo, Yokohama, Osaka, and Kobe).

4. The Chairmanship for the Conference was shared jointly by Mr. John Convery and Mr. T. Tamaki. Dr. T. Kubo, President of the Japan Sewage Works Agency, attended as a special advisor and accepted the delegations from both sides. Dr. Kubo served as the Japanese Chairman for the First through Eighth Conferences.

5. During the Conference, both sides presented papers relating to joint research projects on biological removal of specific contaminants, municipal sludge disposal, instrumentation and automation, energy conservation, operation and maintenance practices, and small flow treatment.

6. Principal topics of the Conference were (1) institutional structures and policies on sewerage and water pollution control and (2) technological developments on energy conservation, toxic wastes, contact aeration, sludge treatment, manufacturing construction materials from sludge, infiltration problems in sewer systems, etc. The discussions following each presentation were mutually productive and useful to both countries.

7. The Conference was followed by the Technical Seminar, which was organized by the Japan Sewage Works Association. Some 300 active members of the Japan Sewage Works Association attended the Seminar, which involved presentations by the U.S. Conference delegates and floor discussions.

8. An inspection visit was made to a metropolitan Tokyo facility producing light-weight aggregates from sludge. Also planned for the U.S. Delegation were field visits to wastewater treatment facilities in Fukuchiyama City, Nara Prefecture, Osaka Prefecture, Osaka City, and Kobe City, and an inspection tour of the Public Works Research Institute (Ministry of Construction) and the National Institute for Environmental Studies (Environment Agency) in Tsukuba Science City.

9. Recent exchanges of engineers between the two countries include a 3-week visit to Japan by Dr. James A. Heidman, Municipal Environmental Research Laboratory, USEPA, in 1982. Mr. A. Oshima, Japan Sewage Works Agency, plans a year's stay in the United States beginning in October 1983. The United States intends to send two engineers to Japan during the next 2 years. Both countries agreed to continue the engineer exchange programs.

10. A decision was made to continue exchanging summaries of important research reports and design guidelines on new technologies.

11. The United States proposed that the Tenth Conference be held in the United States about October 1985.

12. A proceedings of the Conference will be printed in English and Japanese.

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Ninth US/Japan Conference
on
Sewage Treatment Technology

CURRENT TOPICS ON SEWAGE WORKS IN JAPAN

September 19–21, 1983
Tokyo, Japan

The work described in this paper was not funded by the U.S. Environmental Protection Agency. The contents do not necessarily reflect the views of the Agency and no official endorsement should be inferred.

Tsutomu Tamaki
Director
Department of Sewerage & Sewage Purification
City Bureau
Ministry of Construction

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1. INTRODUCTION

"The socio-economic environment of Japan has changed drastically. Japan now is facing severely limiting constraints on energy and resources, financial difficulties, and other various national and international problems, and at the same time is being urged to play a more active international role as a developed and advanced country. The Japanese government is determined to do its best for the benefits of the world while solving domestic problems through administrative reform. For the purpose of drastic reform and streamlining of its administration, the Japanese government is also determined to organize a responsible body consisting of the greatest minds of the country."

This is the Japanese government's expressed commitment which led to the adoption at an extraordinary session of the Diet held in the fall of 1980 of a law concerning the establishment of an ad hoc administrative reform commission.

According to the law, the 2nd Ad Hoc Commission on Administrative Reform was started on March 16, 1981. On July 10, about four months after its inauguration, the commission submitted to then Prime Minister Zenko Suzuki its first proposal for administrative reform. The proposal provided guidelines for the government agencies to prepare their budgets for the fiscal year 1982. Specifically, the proposal directed the government to formulate its 1982 budget without increasing taxes or the amount of special government bond issues.

As a result, spending on public projects was cut below the level of the previous year. The fiscal 1980 budget for public works had already remained on the same level as that for the preceding year. For the four years from that time up to the present, investments on public projects have been pegged. The Ad Hoc Commission on Administrative Reform touched upon the matter of the sewage projects in its report titled "Concerning the Consolidation of Government Subsidies and Other Matters" prepared by its Third Technical Committee. The report recommended that the government should review the criteria for granting subsidies for public works, including sewerage construction, establish a system of determining the subvention ratios, scrutinize public works projects rigorously, tighten eligibility requirements, and rationalize subvention services.

It was also proposed that the beneficiaries-will-pay principle should be more rigorously applied to the construction and operation of sewage projects, industry-related public facilities and various other public works with due consideration given to equity. In fact, we cannot speak of present-day sewage works in Japan without speaking of Japanese government policies or public opinion in favor of administrative reform. The following shows the problems now confronting Japan's sewage works under financial difficulties, and the measures taken by public enterprises engaged in sewage treatment against various problems, together with relevant remarks.

2. THE PROGRESS OF FIFTH FIVE-YEAR PROGRAM FOR SEWERAGE CONSTRUCTION

Mr. Tokuji Annaka reported on the antecedents and particulars of the Fifth Five-Year Program for Sewerage Construction at the eighth US/Japan Conference. As shown in Tables 1 and 2, the Fifth Five-Year Program is based on the New Seven-Year Socio-economic Plan, and is designed to increase the ratio of the sewer-served pupulation from 30% at the end of fiscal 1980 to 44% by the end of fiscal 1985. The total investment planned for the program is ¥11,800,000 million (inclusive of adjustment program expenses of ¥590,000 million). As it turned out, the economic situation had so deteriorated that the New Seven-Year Socio-economic Plan had to be reviewed while the Fifth Five-Year Program for Sewerage Construction was being formulated. In any event, the Fifth Five-Year Program for Sewerage Construction was decided upon by the cabinet in November 1981.

Table 1 Five-year program for sewerage construction

(Unit: million yen)

Classification	New program (1981 - 1985)	Preceding program (1976 - 1980)
Public sewerage	8,391,000	5,455,000
with government grant	5,161,500	3,276,400
by local municipalities	3,229,500	2,178,600
Regional sewerages	2,230,000	1,250,000
with government grant	2,073,900	1,162,500
by local municipalities	156,100	87,500
Urban storm water conduit	460,000	270,000
Specified public sewerages for industries	27,000	61,000
with government grant	18,100	44,600
by local municipalities	8,900	16,400
Specified public sewerages for conservation of the environment	102,000	64,000
with government grant	76,500	48,000
by local municipalities	25,500	16,000
Adjustment program expenses	590,000	-
Contingency allowances	-	400,000
Total	11,800,000	7,500,000
Total expenses for projects with government grants	7,790,000	4,801,500
Total expenses for projects by local municipalities	3,420,000	2,298,500

Table 2 Forecast of sewer service by the five-year program for sewerage construction

Item	End of fiscal 1980	End of fiscal 1985
Total population ($\times 10^4$)	11,706	12,286
Sewer-served population ($\times 10^4$)	3,454	5,390
Coverage (%)	30	44

Note: including projects by local governments

Table 3 shows the progress of the Fifth Five-Year Program for Sewerage Construction. Every public undertaking is subject to tight no-growth ceiling budgeting, not excepting the area of sewerage construction. The budget for every sewerage construction project has been cut every year; although more than half the period of the Fifth Five-Year Program for Sewerage Construction has passed, the amount of investments made so far is far less than half the planned value.

In order to complete the Fifth Five-Year Program for Sewerage Construction, it will be necessary to secure funds equal to 55% of the total planned costs in the remaining two years and to increase working capital at an annual growth rate of more than 60% from 1984 ahead.

But an improvement in the public financial situation within the next few years is a remote possibility (Fig. 1), and the Fifth Five-Year Program for Sewerage Construction appears to have no chance of being fulfilled.

Table 3 Progress of five-year program for sewerage construction (Total expenses)

(Unit: million yen)

Classification	Planned five-year amount	fiscal 1981		fiscal 1982		fiscal 1983	
		Amount invested	Progress ratio (%)	Amount invested	Cumulative progress ratio (%)	Budget	Cumulative progress ratio (%)
Public sewerage	8,391,000	1,460,900	17.4	1,393,721	34.0	1,183,892	48.1
Regional sewerage	2,230,000	294,739	13.2	248,302	24.4	235,618	34.9
Urban storm water conduit	460,000	65,047	14.1	55,417	26.2	49,629	37.0
Specified public sewerage for industries	27,000	3,152	11.7	2,634	21.4	2,542	30.8
Specified public sewerage for conservation of environment	102,000	11,096	10.9	9,653	20.3	9,351	29.5
Total	11,210,000	1,834,934	16.4	1,709,727	31.6	1,481,032	44.8
Adjustment program expenses	590,000						
Grand total	11,800,000						

Note: The amounts invested in and before fiscal 1982 include the comprehensive national land development adjustment program expenses, etc.

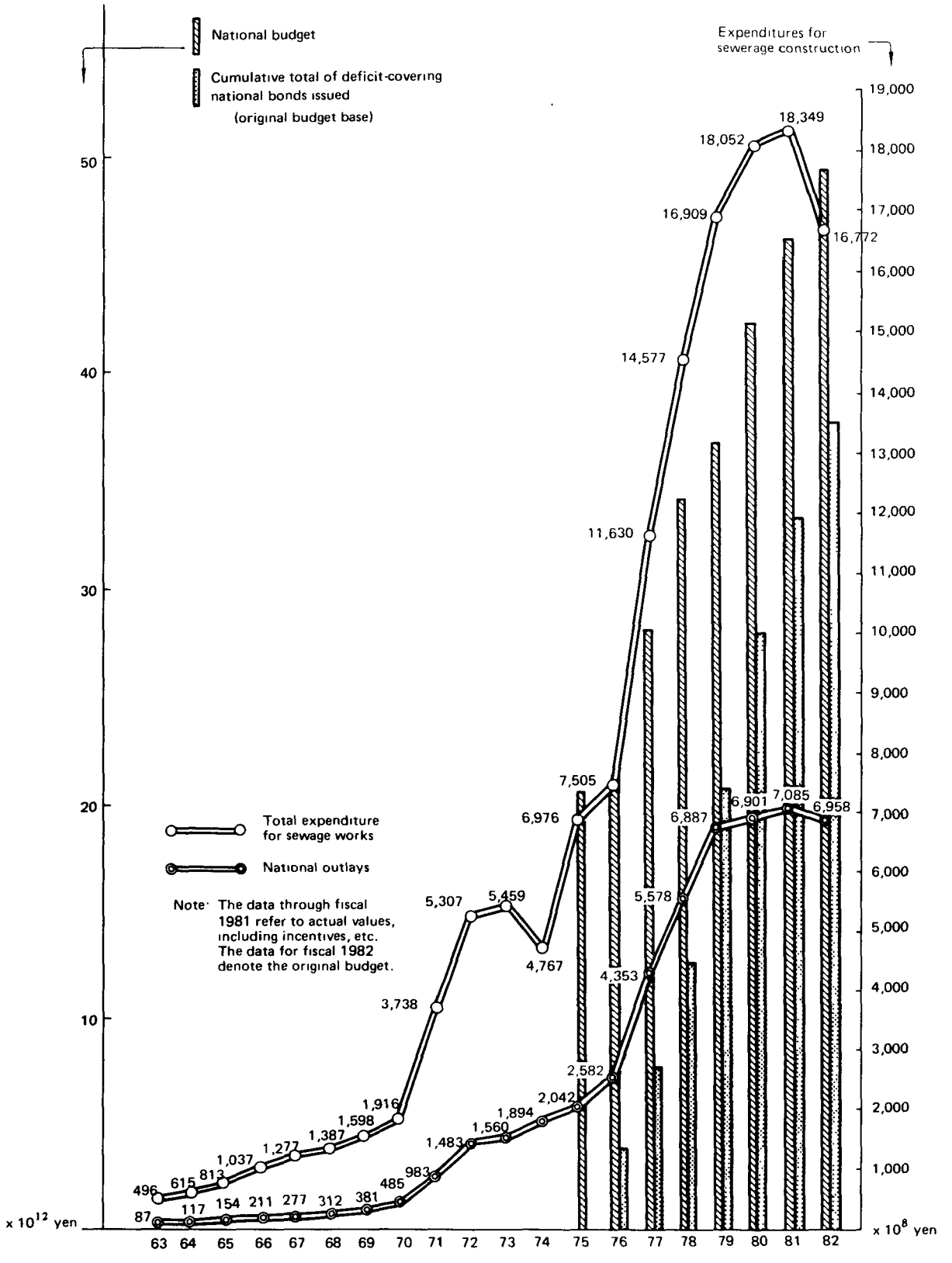


Fig. 1 Japan's financial status, and the flow of expenditures for sewerage construction

Under these bleak financial circumstances, the municipalities are feeling hardships as regards how to attain their objectives in sewage works within a limited budget. The following is a list of major policies proposed for fiscal 1983 sewerage construction projects.

(1) To promote the sewage projects in line with the Regional Environmental Pollution Control Program and The Area-wide Pollutant Load Abatement Plan.

(2) To promote the sewerage construction for the purpose of promoting regional housing development. (To construct efficient sewerages to meet the sizes and characteristics of local cities.)

(3) To promote the construction of sewers in developed areas for the purpose of preventing inundation and improving urban amenities.

(4) To promote the construction of sewer systems in advance of new urban development, and to promote the construction of sewer systems for developed areas in connection with further large-scale housing development.

(5) To promote the construction of sewerages in rural communities for the purpose of protecting the water environment.

(6) To promote advanced wastewater treatment in order to achieve and maintain environmental water quality standards and to prevent the eutrophication of landlocked waters; to start model urban storm water conduit projects for the treatment of domestic effluents by "in-sewer treatment projects"; and to promote the improvement of combined sewers.

(7) To promote the agricultural utilization of sewage sludge and electric power generation with sludge digester gas for the purpose of saving energy and resources.

(8) To continue model projects for the purpose of turning treated wastewater to good use.

(9) To conduct investigations for the promotion of wide-area treatment and disposal of sewage sludge in the Tokyo metropolitan area, Kinki area, etc.

3. SEWAGE PROJECTS FOR THE WATER QUALITY CONSERVATION IN LAKES AND OTHER SEMIENCLOSED WATER AREAS

Public water supplies in Japan are generally improving in quality in recent years, though there still remain many waters where the quality is far from acceptable. Especially as regards landlocked waters such as inland seas, semienclosed bays, lakes and reservoirs, eutrophication with nutrient salts such as nitrogen and phosphorus and the resultant proliferation of algae are exacerbating the water quality if the basins have large pollutant sources. Lake Biwa, Lake Kasumigaura, Lake Suwa, Tokyo Bay, Ise Bay, and the Seto Inland Sea are among the most notorious landlocked waters in Japan. The pollution in these waters due to eutrophication is almost past remedy. Unless proper government measures are taken, their value as water sources or tourist resources will soon be lost.

In order to fight this problem, the Seto Inland Sea Conservation Law was passed in June 1978, and the Water Pollution Control Law was amended to introduce the so-called total load control concerning COD in the basins draining into Tokyo Bay, Ise Bay and the Seto Inland Sea. Along with this, sewerage construction in these basins was given top priority; in fiscal 1981, for example, about ¥521,300 million or about 34.4% of the total national budget for sewage projects (¥1,513,800 million) was invested into the Public sewerage systems, Regional sewerage systems, Specified public sewerage systems for industries, and Specified public sewerage systems for conservation of the environment in the 13 prefectures surrounding the Seto Inland Sea.

To prevent eutrophication, however, not only COD, but also nutrient salts such as nitrogen and phosphorus must be stopped from entering into the waters. In March 1980, the Environment Agency made public its basic policy for eutrophication prevention. Since then, studies have been made to set limits on nitrogen and phosphorus discharges into lakes and reservoirs. In December 1982, Environmental water quality standards concerning nitrogen and phosphorus in lakes and reservoirs were announced as shown in Table 4, and the studies on effluent standards commenced.

Table 4 Environmental water quality standards concerning nitrogen and phosphorus in lakes and reservoirs

Item / Type	Water uses	Standard values	
		Total nitrogen	Total phosphorus
I	Natural environment protection, and types II through V	less than 0.1 mg/lit.	less than 0.005 mg/lit.
II	Tap water Classes 1, 2, 3 (excluding special uses) Pisciculture Class 1 Bathing and types III through V	less than 0.2 mg/lit.	less than 0.01 mg/lit.
III	Tap water Class 3 (special uses) and types IV and V	less than 0.4 mg/lit.	less than 0.03 mg/lit.
IV	Pisciculture Class 2 and type V	less than 0.6 mg/lit.	less than 0.05 mg/lit.
V	Pisciculture Class 3 Industrial water Irrigation water Environmental protection	less than 1 mg/lit.	less than 0.1 mg/lit.
	Measuring methods	As per specified in Table 7a	As per specified in Table 8a

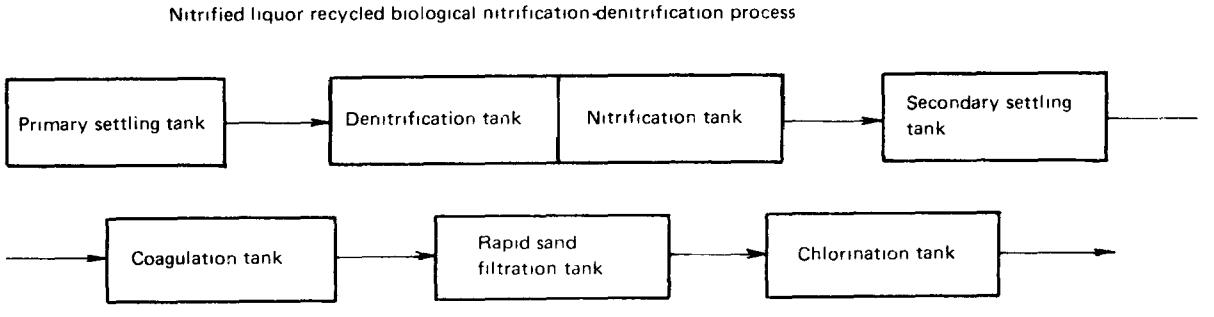
Remarks:
1. The standard values refer to annual average values.
2. As regards the irrigation water, the standard values for total phosphorus do not apply.

Notes:
1. Natural environmental protection: Environmental protection in resort areas etc.
2. Tap water Class 1: Water requiring simple purification as by filtration.
   Tap water Class 2: Water requiring ordinary purification as by settling and filtration.
   Tap water Class 3: Water requiring advanced treatment including pretreatment. "Special" means special purification operations capable of removing odors.
3. Pisciculture Class 1: For aquatic life such as salmon and fresh river trout, etc. and for aquatic life coming under the category of Pisciculture Classes 2 and 3.
   Pisciculture Class 2: For aquatic life such as pond smelts, etc. and for aquatic life coming under Pisciculture Class 3.
   Pisciculture Class 3: For aquatics life like carp, gibel, etc.
4. Environmental protection: Water not arousing discomfort in daily life (including promenading along shoreline etc.).
(Tables 7a and 8a are omitted here)

It goes without saying that sewerage is a viable measure against water pollution in landlocked waters. While the conventional secondary treatment plants are designed primarily for the removal of BOD, they can also remove 20 to 40% of the nitrogen and phosphorus which are the major culprits of eutrophication. In Japan, the sewer coverage is quite low, and it is planned to push forward the construction of secondary treatment plants for various landlocked waters for some time now. In some waters which are desperately plagued with eutrophication, it will be necessary to install advanced treatment systems capable of removing nitrogen and phosphorus as well as BOD and SS. In fact, advanced treatment facilities are already in use at the Konan Purification Center of the Lake Biwa Regional Sewerage System and the Kohoku Purification Center of the Kasumigaura Regional Sewerage System as shown in Fig. 2.

As regards Lake Biwa, the construction of advanced treatment facilities for the removal of nitrogen and phosphorus has been under way at the Kosei Purification Center since fiscal 1982.

Konan Purification Center, Lake Biwa Regional Sewerage System



Kohoku Purification Center, Kasumigaura Regional Sewerage System

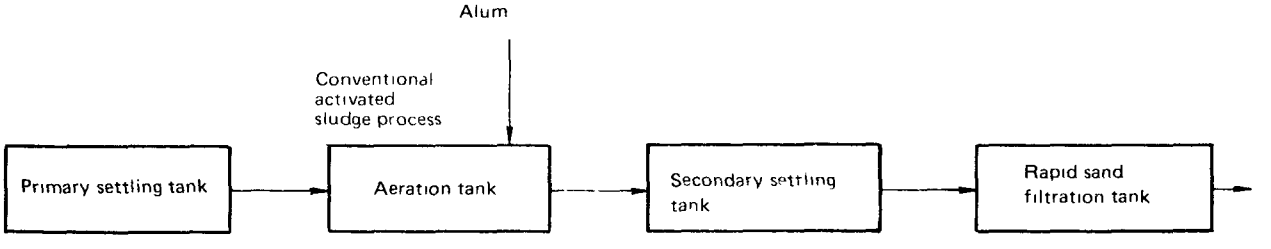


Fig. 2 Sewage treatment process flow employed for the Konan Purification Center, Lake Biwa Regional Sewerage System, and the Kohoku Purification Center, Kasumigaura Regional Sewerage System

The demand for advanced treatment for the control of water pollution due to eutrophication will grow in future, but the implementation of advanced treatment technologies entails the following problems.

(1) A system for quantitative evaluation of the effects of advanced treatment on the water quality improvement has not completely be established. Specifically, it is difficult to work out an optimum advanced treatment plan for a given water quality target.
(2) Sewer service is recognized as a minimum national requirement to which every citizen is entitled for healthy and comfortable living. In view of this, there are opinions arguing that increasing the sewer coverage be given top priority. There are controversies over whether the budgets be appropriated for the construction of advanced treatment plants in specific areas or for extension of sewer services to unsewered areas under the current situation of financial difficulties. A general rule for the logical distribution of budgets for sewerage construction has not been established yet.
(3) If the costs for maintenance and operation of advanced treatment plants are to be counterbalanced by billing the users, such users will be put at a disadvantage compared with those in other areas. If only those living in lower basins can enjoy the benefits of water quality improvement in the form of, say, water resources, by advanced treatment in the upper basin, this will give rise to a serious problem of who should bear the cost for water quality improvement.

As regards (1), the Basic Study Commission on the Construction of Sewerage for Eutrophication Prevention has been making studies for the formulation of a basic plan for eutrophication preventive sewerage systems since fiscal 1980.

So far, sewerage systems for the prevention of water pollution due to BOD, COD and SS have been designed and engineered within the framework of the Comprehensive Basin-wide Planning of Sewerage System. But the basic plan for eutrophication preventive sewerage systems aims at including nitrogen and phosphorus as pollutants to be controlled together with BOD, COD, and SS.

Fig. 3 shows a presumed procedure for formulating a basic plan for an eutrophication preventive sewerage system. Compared with the analysis of BOD, COD, and SS, pollution analysis relating to eutrophication has differences or difficulties on the following points.

(1) It is said that compared with BOD, COD and SS, much of nitrogen and phosphorus loads is generated by farmland, forests and other non-point sources rather than by point sources.
However, the best methods of estimating the generated loads of nitrogen and phosphorus from non-point sources have not necessarily been established.

(2) Although nitrogen and phosphorus are considered less susceptible to dissipation during transportation from sources to the receiving waters compared with BOD, COD, and SS, not all of them yet reach the landlocked waters because part of nitrogen will be lost by denitrification or other processes, while phosphorus is accumulated in the soil of the basin by adsorption, etc. The load run-off ratio, which means the ratio of the pollutant load carried off into the landlocked waters to the total pullutant load generated in the basin, is hard to estimate logically so far as nitrogen and phosphorus are concerned.

(3) Nitrogen and phosphorus running into the landlocked waters invigorate the growth of algae, increasing the primary productivity in a given body of water only to invite organic pollution. But little is known about the quantitative relationship between the influent loads of nitrogen and phosphorus and the level of organic pollution. Among the currently available methods are statistical methods including the critical nutrient loading model proposed by Vollenweider, and the dynamic ecosystem model first tried by Di Toro, et al. The models to be applied should naturally be selected depending on the data available and the nature of the water quality information to be forecast, but the criteria for such selection still remain in nebulous.

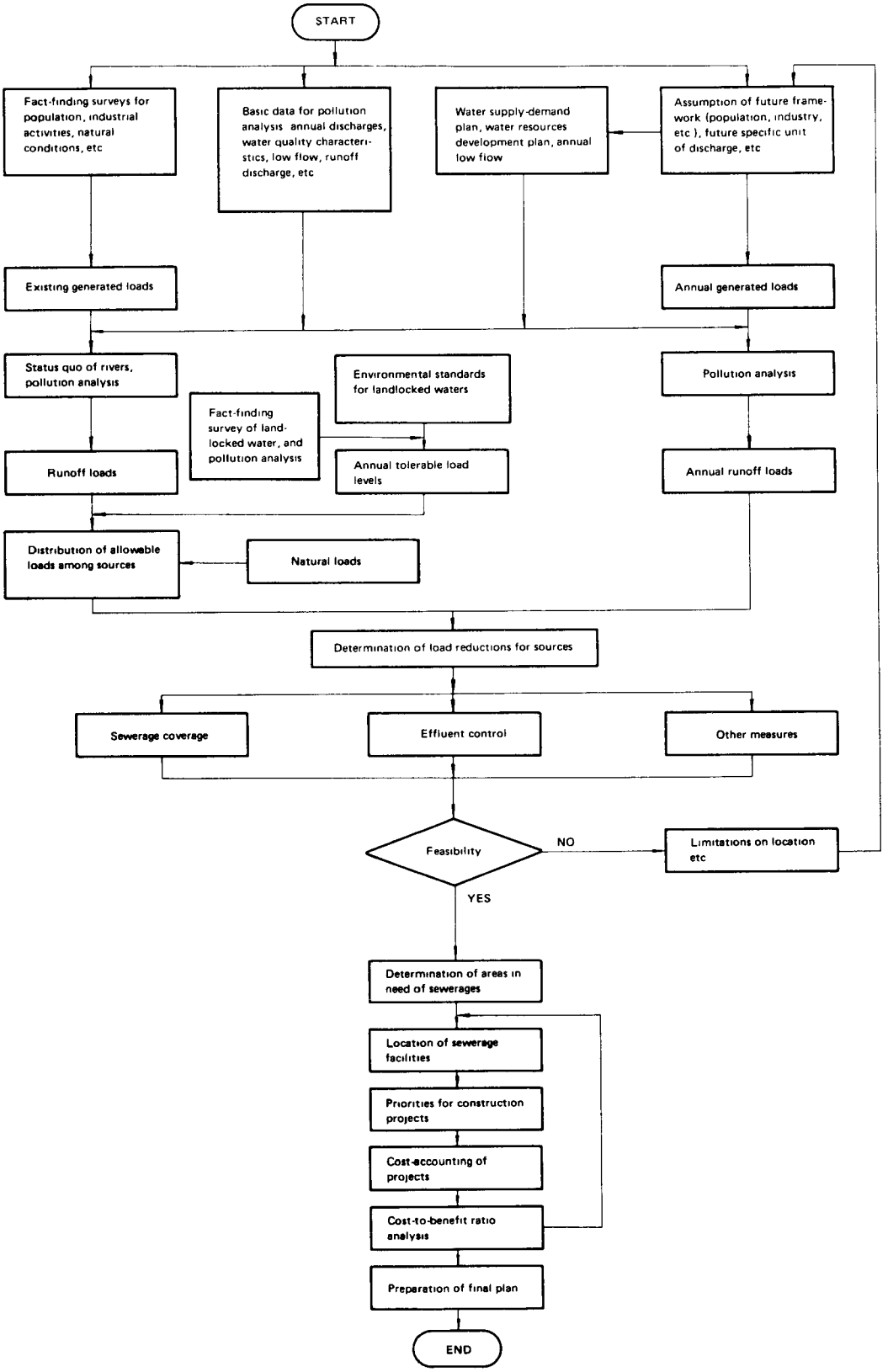


Fig. 3 Survey flowchart for master planning of sewerage construction

The Basic Study Commission on the Construction of Sewerage for Eutrophication Prevention has been making progress with the study of all these problems based on scientific data, and is expected to formulate a manual for the Comprehensive Basin-wide Planning of Sewerage System to fight off eutrophication problems.

As regards (2) and (3), it will be necessary to work out measures that will be accepted to all. At present, the utility of sewerages is being studied from the various angles including the axiology of water quality conservation, for the purpose of formulating logical guidelines.

4. SMALL-SCALE SEWERAGE SYSTEMS

The sewer-served population in Japan was about 32% of the total population at the end of fiscal 1982. But there are great regional differences in sewer service between the local municipalities. The sewer service in towns and villages is quite low compared with the cities. For example, the sewer-served population in the 11 cities designated by the Cabinet order, including Tokyo and Osaka is 73%, while that in the other municipalities is a mere 20%.

Table 5 shows the frequency distribution of municipalities with public sewerage in relation to population size. The rate of municipalities with sewerage is 100% in the cities with a population of more than 300,000, 53.4% in the cities with a population of 30,000 to 50,000 and 4.8% in the cities with a population of up to 10,000.

In the past sewer service in comparatively small rural communities with a small population has remained quite low. The factors explaining this are as follows.

(1) Such local communities could not afford to invest in sewerages because they lacked the ways and means.
(2) The government promoted the construction of sewerages with emphasis on the increase of sewer service, and was only ready to invest in cost-efficient highly populated areas.
(3) Sewerage systems were designed and engineered for use for metropolises, and sewerage technology for small cities was not well developed.

In recent years, however, people in rural communities all over the country are beginning to voice their strong desire for the construction of sewerages, which possibly will fit in with the movements of the regional domiciliation scheme (a plan in which the country is divided into several regions for domiciliation, and living infrastructural facilities are developed to meet regional characteristics for the purpose of encouraging people to live in these regions in order to use the national land in a balanced way without overpopulation or depopulation problems) extolled in the Third Comprehensive National Development Plan.

Since the mid-1970s, the so-called U-turn phenomenon (in which the youths after graduation from universities in large cities go back home and find jobs in their birthplaces) has become conspicuous. In addition, urbanites who prefer country life are increasing in number. Living standards in the country side have also improved. All these have combined to increase the demand for flush toilets. As lifestyles in the rural communities have changed, domestic wastewater and effluents from septic tanks are beginning to pollute the public waters in the country.

While septic tank-type flush toilets have become increasingly popular in the country in the last few years to the extent that their population coverage surpasses the sewer-served population, there are many cases in which the septic tanks are maintained and managed poorly.

The quality of the discharges from the septic tanks is poor under the effluent standard of as high a BOD value as 90 mg/lit. The pollution of public waters is seriously concerned, accordingly.

In view of this, the Ministry of Construction started a specific public sewerage system for conservation of the environment in fiscal 1975. It is classified by purposes into natural environment protection sewerage and rural community sewerage.

Table 5 Number of municipalities having sewerage as classified by population size

Population size	Total number of minici-palities	Public sewerages		Specified public sewer-age for conservation of the environment		Total		Ratio	
		Number of municipali-ties going on sewage projects	Number of municipali-ties with sewerage services	Number of municipali-ties going on sewage projects	Number of municipali-ties with sewerage services	Number of municipali-ties going on sewage projects	Number of municipali-ties with sewerage services	Ongoing projects	Sewerages in commi-ssion
1 million plus	10	10	10	0	0	10	10	100	100
500,000 to 1 million	9	9	9	0	0	9	9	100	100
300,000 to 500,000	36	36	35	3	2	36	35	100	97.2
100,000 to 300,000	136	128	116	3	4	128	116	92.8	84.1
50,000 to 100,000	213	173	97	8	3	174	97	81.7	45.5
30,000 to 50,000	251	133	50	8	2	134	50	53.4	19.9
10,000 to 30,000	1,089	196	54	25	2	216	55	19.8	5.1
up to 10,000	1,510	42	11	32	6	72	15	4.8	1.0
Total	3,256	727	382	79	19	779	387	23.9	11.9

Notes: 1. Population data from the Oct. 1, 1980 national census.
2. Some municipalities are counted for both public sewerages and specified public sewerages for conservation of the environment, and the footings are adjusted by subtracting those which are counted twice.

Natural environment protection sewerage is designed for tourist resorts, such as Lake Akan and Lake Ashinoko, where much celebrated water quality is beginning to be degraded with the prosperity of tourism. Some lakes have lost their transparency, and some others have been so vitiated even to send off offensive odors. In fiscal 1982, natural environment protection sewerage was constructed at 35 places to improve local water environments.

Rural community sewerages are designed to protect the public waters due to domestic wastewater from farming communities, fishing communities, etc., where the demand for flush toilets has been increasing in keeping with the urbanization of lifestyles. In fiscal 1982, rural community sewerages were constructed at 49 places.

So far, sewerage systems have progressed in the urban areas, and their design and engineering know-how reflect urban conditions which are quite different from those in rural communities. The size of a sewerage system for a rural community is generally small, and existing sanitary engineering know-how cannot always be applied to it directly. For example, the road density, traffic volume, underground structures, conditions for the acquisition of right-of-ways, and influent loads and their changes in the rural communities differ greatly from those in urban areas. The planning and construction of sewerages change from community to community depending on location, environmental conditions, and the nature of the communities, even when the population size is almost the same.

The Ministry of Construction has been studying the methods of planning and designing small sewerage systems since fiscal 1981, for the purpose of effective planning and construction of sewerages for rural communities. The achievements so far were compiled in the Small-scale Sewerage Planning Guidelines (draft) published last March.

This draft specifies what should be taken into account in working out a basic plant for a small-scale sewerage system for a community size of 2 households to 5,000 people. Compared with the conventional guidelines for sewerage system planning, the draft has the following features:

(1) The area for which a small-scale sewerage system is to be constructed usually is rural, and the population there is unlikely to grow fast. Thus, the planned target year is basically set at the present time.

(2) In a conventional sewerage system, the minimum size of a sanitary sewer is set at 200 mm in inside diameter. In a small-scale sewerage system, this size will not produce enough tractive force. For this reason, the service sanitary sewer pipe is set at 100 mm in principle.

(3) As regards sewage treatment, not only the conventional activated sludge process which has been used most widely for the conventional medium-size and large-size treatment plants, but also the sequential batch reactor process, semi-batch oxidation ditch process, oxidation ditch process, aerated lagoon process, extended aeration process, high-rate aeration process, rotating bio-disc contactor process, contact aeration process, trickling filter process, single-stage nitrification-denitrification process, metal-salt coagulation process, filtration process, and other various physico-chemical processes are taken up, and their selection criteria are stated in consideration of local conditions.

(4) There is a great possiblity that farms and green lots abound around small-scale sewage plants for the easy disposal of sludge. In view of this and from the viewpoint of energy economy and sludge recycling, it is planned to promote the use of sludge for agricultural purposes. Specifically, the draft shows several standard sludge processing flows applicable to small-scale sewage treatment plants for these purposes.

As logical design techniques and design and engineering parameters have been established by the Small-Scale Sewerage Planning Guidelines (draft), the Ministry of Construction is now planning to start small-scale sewerage model projects from the coming fiscal year. In these projects, the Ministry of Construction will select as models projects in which such Specified public sewerage systems for the conservation of the environment or small-scale Public sewerage systems that are designed with efficient processes. The Ministry of Construction will extend technical and financial assistance to such projects.

5. MODEL PROJECTS FOR SEWERAGE CONSTRUCTION

As shown in Fig. 1, the sewage works in Japan have fallen prey to the influence of austerity budgeting programs for two consecutive years. In spite of financial retrenchment, the municipalities are called upon to step their efforts to promote the construction of such sewerage systems as are viable both economically and technically.

The Ministry of Construction set afloat the "impressive sewerage project" and "good-idea sewerage project" in fiscal 1982 and the "in-sewer treatment project" in fiscal 1983.

5.1 IMPRESSIVE SEWERAGE PROJECT AND GOOD-IDEA SEWERAGE PROJECT

These projects were designed to obtain a deep understanding and the strong support of the general public concerning sewage projects. Specifically, sewage projects which have an appealing effect on the local inhabitants or public opinion or which have been designed with new technologies or the people's ideas are taken up as models for demonstration of effects. Special budgetary credit is given to these projects.

In fiscal 1982, 20 model projects got started with a total ¥3,000 million. They included the "Save-the-King-Crab Sewerage Project (Okayama City, Okayama)," "Sewerage Project for Protection of Yanagawa Riverside - the Hometown of Poet Hakushu (Mitsuhashi and Yanagawa Official Cooperation, Fukuoka)," "Metropolis Inundation Protection Operations Bypass Sewerage Project (Utsunomiya, Tochigi)," "Batch-type Sewage Treatment Project for Promotion of Sewage Treatment Technology (Maizuru, Kyoto)," and "Farewell-to-Boots Operations Project (Hamamatsu, Shizuoka)."

In fiscal 1983, it is planned to evaluate creative projects worked out by local municipalities according to the following criteria and target of selected projects as coming under the impressive sewerage or the good-idea sewerage category.

(1) Impressive sewerage projects

A project which is effective in accelerating the construction of sewerage and is appealing to the minds of the local inhabitants or the general public.

- Eligibility requirements -

1. Having a quick-fix effect.
2. To be located for the protection of well-known waters.
3. A project which will encourage a citizens' movement for the construction of sewerages.

(2) Good-idea sewerage project

A project based on innovative technologies for the reduction of costs for construction, operation, and maintenance of sewerages.

- Eligibility requirements -

1. To reduce construction costs.
2. To reduce installation space.
3. To save maintenance and management expenses through the reduction of needs for resources, energy and labor.
4. To improve treatment efficiency.
5. To appeal to the local populace from the viewpoint of environmental protection.

In fiscal 1983, 12 impressive sewerage projects and 7 good-idea sewerage projects were newly listed. Some of them are as outlined in Table 6.

Table 6 Examples of impressive sewerage systems and good-idea sewerage systems newly selected for fiscal 1983

Impressive sewerage		
(1) Clean Otaru Canal Operations for Protection of What It was in the Pioneering Age (Otaru City, Hokkaido) To protect the traditional water-scape of the canals from domestic waste-water by quick buildup of sewerage systems.	(2) Sewerage for Conser-vation of Traditio-nal Beauty of Yodo Castle (Kyoto City, Kyoto) To install well-equipped sewerage systems for the pur-pose of recouping the water quality in the moats and canals around the remains of the Yodo Castle which are endangered by domestic effluents.	(3) Agricultural Waste-water Reclamation Operations (Yamate Village, Okayama) Yamate Village is destitute of water resources such as rivers and is always feeling a water shortage because annual rainfalls are short. The project is to recover wastewater treatment plant effluent for irrigation and industrial purposes.
Good-idea sewerage		
(1) All-Participatory Wealth-from-Waste Purification Center (Tazawako Town, Akita) To reduce sewage plant construction costs and to promote the recycling of refuse by using waste plastics as biological contact filter materials in the biological contact oxidation process.	(2) Energy-efficient Sludge Treatment Plants working on Waste Heat of Refuse Incinerator (Zushi City, Kanagawa) A sludge drying plant is annexed to a municipal refuse incineration plant. Waste heat available from the incineration plant is used to dry sludge cake to reduce sludge volume, stabilize sludge quality, and at the same time to save on fuel, operation, and maintenance costs.	(3) Air-economized Sewage Treatment Process (aerobic/ anaerobic biological filtra-tion process) (Takane Town, Yamanashi) A treatment process designed for a small-scale sewerage system which economizes on air injection by making use of anaerobic microbes for treatment.

5.2 IN-SEWER TREATMENT MODEL PROJECTS

The urban storm water conduit is one of the urban facilities used for draining stormwater quickly out of the urban area. In those areas where sewage plants are not installed, a storm water conduit is likely to be used for collecting and draining not only stormwater but also domestic wastewater. What is carried by the storm water conduit includes slop and other domestic washings (excluding raw sewage), effluents from septic tanks and industrial effluents, causing the receiving public waters to be polluted.

As a matter of course, this problem should be solved by new construction and amplification or improvement of sewerage. However, in those areas which cannot wait, it is necessary to take a jury-rigging measure like a utility purification device installed in a sewer.

In view of this, the Ministry of Construction decides on an in-sewer treatment model project for the storm water conduit under its care.

In an in-sewer treatment system, contact materials such as net, ropes and pebbles, which offer a good habitat for microbes are installed in the storm water conduit for treatment of wastewater through biological oxidation. As shown in Fig. 4, in-sewer treatment systems are in general classified into two types:

In fiscal 1983, in-sewer treatment model projects will be started at several places in the Kasumigaura Basin and the Lake Biwa Basin.

(A) In-line system

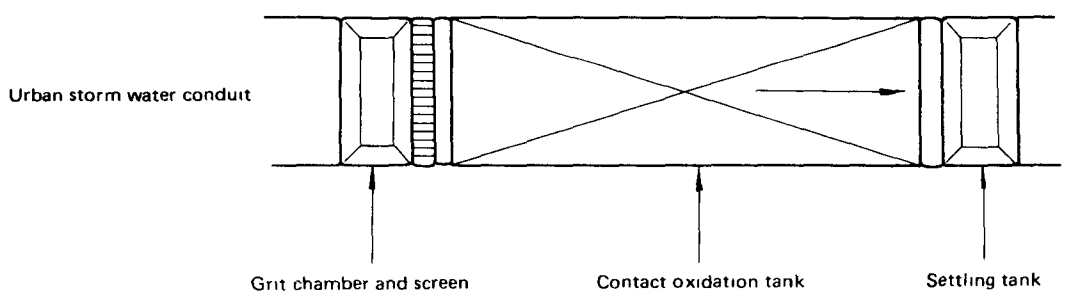


Fig. 4 A schematic view of in-sewer treatment system

(B) Bypass system

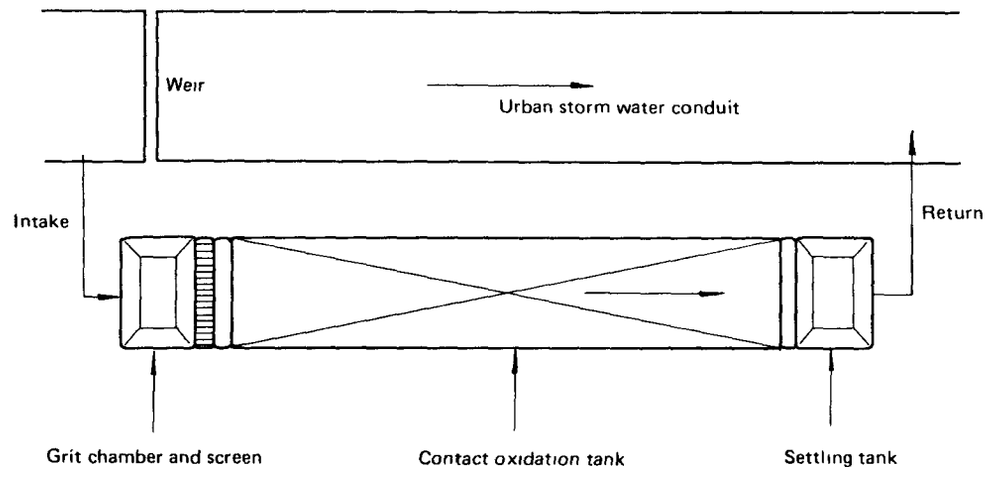


Fig. 4 A schematic view of in-sewer treatment system (Cont'd)

6. TREATMENT, DISPOSAL AND EFFECTIVE UTILIZATION OF SEWAGE SLUDGE

At present, the sewage treatment plants in Japan are treating about 6,700 million m^3 of wastewater a year, turning out large volumes of sludge. Infact, the sludge poses many problems to every municipality, and efforts are being made to find a way out for proper treatment and disposal of sewage sludge.

6.1 THE STATUS QUO OF SEWAGE SLUDGE TREATMENT AND DISPOSAL

Table 7 shows the outlets of sewage sludge during the period from November 1981 to October 1982. The total volume of sludge disposed is about 2,390,000 m^3, of which about 78% was dumped in the form of dewatered sludge cake, about 12% in the form of incinerated ash, about 8% in the form of digested sludge, and about 3% in the form of dry sludge. It should be noted that the volume shown refers to the sludge when carried out of plants. Sludge which is turned into compost for agricultural use is counted as dry sludge.

Sludge which is dumped at landfills leads the rest, accounting for some 47% of the total. This is followed by land reclamation with about 30%, utilization with about 15% and others with about 8%. The 8% portion mainly includes disposal in the ocean. The land reclamation is widely practiced by large municipalities where sludge production is prohibitively voluminous. Of the total sludge cake producted in Japan, some 49% undergoes incineration; in the cities designated by the Cabinet order, the incineration average ratio is about 57%, and in other ordinary cities, the average ratio is about 35%, suggesting that every large city is making efforts to extend the service life of fast-dwindling disposal sites by reducing the quantity of sludge.

Table 7 Disposal of sewage sludge
(Nov. 1, 1981 - Oct. 31, 1982)
(unit: x 10^3 m^3)

Disposal method / Sludge condition	Landfill	Land reclamation	Utilization	Others	Total (%)
Dewatered sludge	871	726	260	13	1,870 (78)
Incinerated ash	140	102	35	0	277 (12)
Dried sludge	6	0	54	0	60 (3)
Digested sludge	7	0	0	173	180 (8)
Total (%)	1,024 (43)	828 (35)	349 (15)	186 (8)	2,387 (100)

6.2 TRANSITION OF TREATMENT AND DISPOSAL OF SEWAGE SLUDGE

Fig. 5 shows the annual transition of sewage sludge treatment and disposal. It is found that the sludge production has been rising year after year. On a sludge cake basis, the sludge volume rose from about 3,640,000 m^3 in 1981 to about 3,840,000 m^3 in 1982, but the volume of disposed sludge has been running almost constant at about 2,400,000 m^3 for the past five years, owing to efforts made to increase the incineration rate and to improve the dewatering efficiency for the reduction of sludge volume. The increase of the incineration rate is demonstrated in Fig. 5 which shows a steady increase in the volume of incinerated ash disposal. A look into the transition of disposal methods over long periods shows that the volume of sludge recovered for useful purposes has been rising. This upward trend reflects the promotion of sludge utilization for farmland and green areas.

Disposed volume by type of sludge

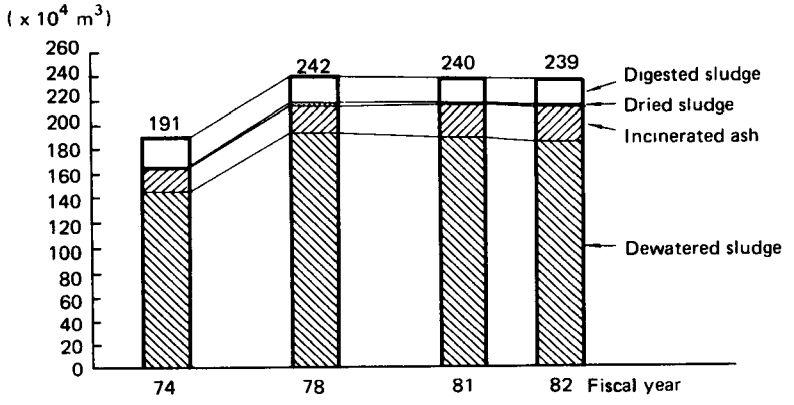


Fig. 5 Annual transition of sewage sludge disposal

Disposed volume by uses

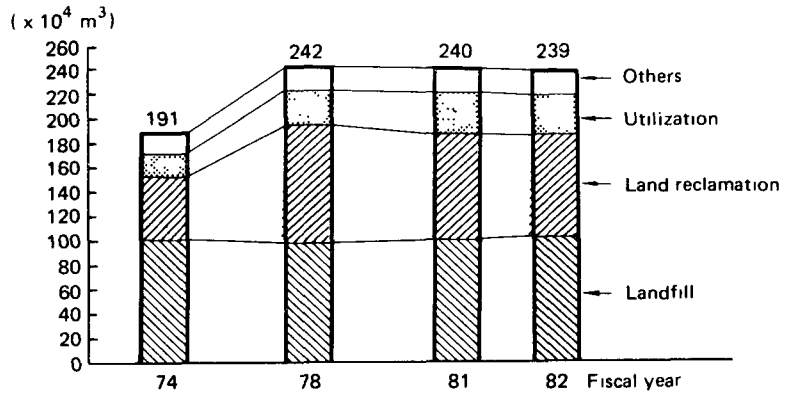


Fig. 5 Annual transition of sewage sludge disposal (Cont'd)

6.3 SLUDGE UTILIZATION

The recovery of sludge for use is shown in Table 8, and its annual transition, in Fig. 6. In fiscal 1982, the recovered volume dropped from the fiscal 1981 level. This is because the recovery of digested sludge decreased.

Instead, sludge was recovered in forms with lower water content; on a sludge cake basis, the recovery rate of sludge has been increasing steadily.

Table 8 Utilization of sewage sludge

(x 10^3 m^3)

Uses \ Sludge condition		De-watered sludge	Incinerated ash	Dried sludge	Compost	Digested sludge	Total
Farmland and green lots	by municipal government	205	1	5	49	0	260
	by fertilizer companies, etc.	55	9	0	0	0	64
	Subtotal	260	10	5	49	0	324
Construction materials		0	25	0	0	0	25
Total		260	35	5	49	0	349

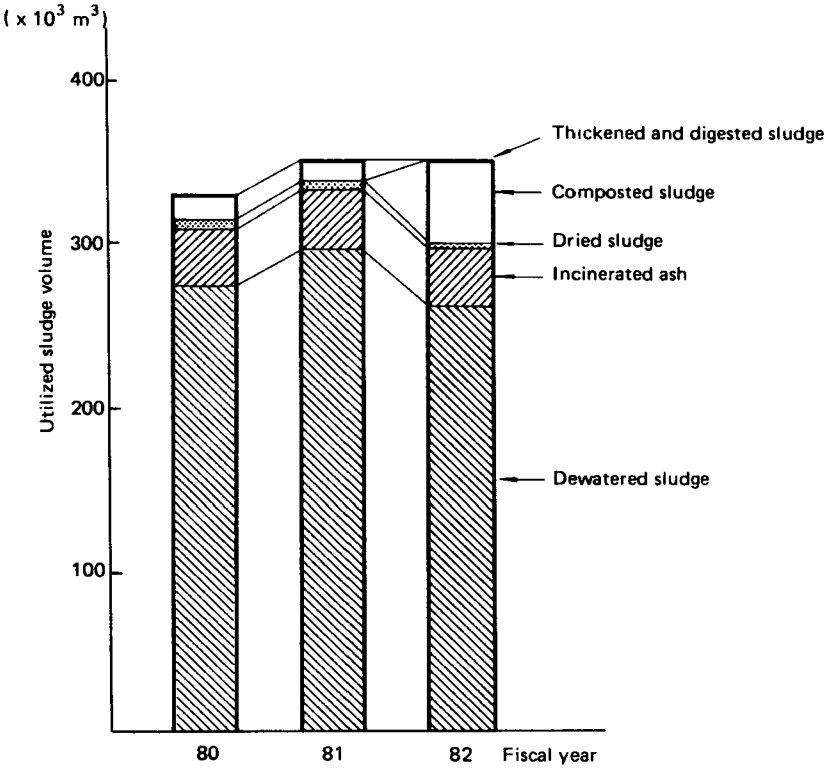


Fig. 6 Annual transition of sewage sludge utilization

The greater part of the sludge recovered is used for soil conditioning or fertilizing croplands and green lots. For these agricultural purposes, sludge cake is most widely used. Some sludge cake is delivered to fertilizer manufacturers, who process it into useful forms like compost or dry sludge. In recent years, those municipalities which have entered the composting business are growing in number.

The volume of sludge which is turned into construction materials remains low, accounting for a mere 1% of the total. In Sapporo, sludge ash is used as a fill for land leveling and banking. In Tokyo, sludge ash is used for a roadbed aggregate. All these processes are being conducted experimentally.

6.4 FUTURE STRATEGIES AND TACTICS FOR SLUDGE DISPOSAL

Whether sludge will be treated and disposed of without trouble in future is a matter of critical importance governing the future of the entire sewage project field. Although sludge is a threat to our future well-being, it can also be a rich vein of resources. Sludge is, so to speak, a modern version of the ugly duckling.

The following are the strategies and tactics on hand to solve sewage sludge problems:

Strategy I: Reduction of sludge volume

The most widely accepted method of reducing the volume of sludge is incineration. But incineration processes consumes a great deal of energy and call for labor for operation and maintenance. In some areas, it is hard to procure sites for incinerators. To provide against these problems, the following tactics are being considered:

Tactic 1: Wide-area joint-use incineration plants

These are intermural incineration plants for joint use by a number of municipalities for high-efficiency continuous operation. Where there are a number of sewage treatment plants operated by a single municipality, as in a large city, it is practical to haul dewatered sludge cake from sewage treatment plants to an incineration plant for intensive incineration.

Tactic 2: Combustion with refuse

Combustion of sludge and refuse together is advantageous in two ways: first, the refuse produces the heat required for the incineration of sludge, and second, it is possible to moderate the incineration temperature when burning refuse of high calorific value. In Japan, the disposal of city refuse is under the jurisdiction of the Ministry of Health and Welfare. Studies are now being pushed forward to formulate a system for distributing the costs, duties, and responsibilities between the Ministry of Construction and the Ministry of Health and Welfare for the construction, operation, and maintenance of sludge/refuse incinerators. In Japan, there is only one sludge/refuse incineration plant in Kanazawa.

Tactic 3: Development of technology for self-sustainable combustion

Usually, the lower calorific value of dewatered sludge cake is positive. It is theoretically possible to sustain the combustion of dewatered sludge cake if an ideal incinerator of a high thermal efficiency is used. In support of this process, there are two self-sustained combustion incinerators, one each at Takaoka City and Nagoya City. If the technology for self-sustainable combustion of sewage sludge is well established and popularized, auxiliary fuel, which now is consumed at 50 to 70 lit. of heavy oil per ton of sludge cake, can be dispensed with, and at the same time, excess energy will be available for more productive purposes.

Strategy II: Promotion of sludge recycling for farmland and green lots

The use of sludge for farming and similar purposes is slow to progress, because the heavy metals and coagulants contained in it are much dreaded. A market has yet to be developed for it. The social consensus on the utilization of sludge for farming purposes is far from firm.

Tactic 1: Establishment of application standards

It is necessary to formulate standards for the utilization of sludge for agricultural purposes based on surveys of the fertilizing effect of sludge and the effects of heavy metals, etc. The Ministry of Construction and the Japan Sewage Works Agency have been studying this problem for more than 10 years. Recently the Ministry of Construction and the Japan Sewage Works Agency have jointly compiled the Guidelines for Application of Sewage Sludge Compost to Farmland (draft). According to the draft guidelines, the following limitations are imposed.

(a) Feedstocks should be limited to sludge cake available from domestic wastewater only from the viewpoint of safety.
(b) Composting should be carried out at temperatures above 60^{o}C for hygiene.
(c) The compost should be subjected to aerobic fermentation and 1 to 3 months of open yard storage and turnover for sufficient post-fermentation.

(d) The compost should be enriched with potassium and other chemical fertilizers to offer a well-balanced fertilizer formula.

Continued efforts are being made in this respect to establish a standard for agricultural applications of sewage sludge in the long run.

Tactic 2: Development of distribution channels for sludge-turned compost

So far the sewage bureaus of the manicipal governments usually have distributed sludge-turned compost almost free of charge, only to dumpen the entrepreneurship of those who are willing to commercialize the sludge-turned compost. To avoid this farce, studies will be made to go hand in hand with these entrepreneurs for promoting sludge-turned compost on the existing fertilizer distribution channels.

In the future, measures will be studied for enlisting the existing fertilizer channeling routes for marketing sludge-turned compost.

Strategy III: Utilization of sludge for construction materials

The recycling of sewage sludge for construction purposes is considered most ideal and promising not only for safety but also from the viewpoint of market potential. At present, the government, local municipalities, and businesses are making progress with respective projects for the development of sludge ash calcining and smelting technologies, finding the sludge ash outlets as artificial lightweight aggregates, tiles, and bricks.

Tactic 1: Development and improvement of technologies for costs reduction

By making use of the energy recovered from sludge (Strategy IV), the cost can be reduced. It is expected that the lightweight aggregates will be produced at a lower cost than the conventional lightweight aggregates.

However, the demand for gravel aggregates is higher, and it is urged to develop technologies for the reduction of costs for the productions of gravel aggregates through sludge smelting.

Tactic 2: Study and evaluation of quality

At present, the study and evaluation of sludge-turned construction materials is carried out by sanitary engineers. But sludge-turned construction materials should undergo study and evaluation by users such as concrete engineers, road construction engineers, and soil mechanics engineers. The Ministry of Construction has been playing up the ministry-wide surveys for the utilization of sludge-turned construction materials according to the Comprehensive Project for Technological Development for Recovery of Wastes for Civil Engineering and Construction, which was started in fiscal 1981. At present, the Ministry of Construction is conducting experiments and studies on the physical properties of sludge-turned gravel aggregates, cost analysis of smelting operations, and construction and civil engineering applications.

Tractic 3: Demonstration of sewage projects using sludge turned construction materials

If the sum of the cost for the ordinary treatment and disposal of sewage sludge and the cost for the ordinary construction materials is larger than the cost for sludge-truned construction materials, it will require administrative guidance in order for sludge-turned construction materials such as artificial lightweight aggregates be used for the construction of sewage projects. It is therefore planned to raise substantive results in the construction of sewage projects to establish a strong toehold for the promotion of sludge-turned construction materials.

Tactic 4: Streamlined management of sludge-turned construction materials production department

For the purpose of opening up a market for sludge-turned construction materials and improving the management efficiency of the sludge-turned construction materials production department, it will be necessary to provide measures such as the establishment of a streamlined organization, the introduction of private funds, etc.

Strategy IV: Recovery of energy from sludge

As regards the recovery of energy from sludge, digester gas-fired electric power generation is most popular. In addition, the various other ways of recovering energy from sludge are under study. The recovery of energy from sludge is important as it supports other strategies in terms of energy.

Tactic 1: Promotion of digester gas-fired electric power generation
In Japan, power generation systems working on anaerobic digester gas are still at an elementary stage. The digester gas-fired power generation plants now in commission or under construction number only seven throughout Japan. Their capacities range from 130 kW to 500 kW to share 15% to 40% of the electric power required by respective sewage treatment plants.

The Ministry of Construction is hastening the preparation of a design manual for digester gas-fired power generation plants. It is expected that the manual will be published within this fiscal year.

Tactic 2: Development of other methods for recovery of energy
At present, the harnessing of excess heat available from self-sustained incineration process, briquetting of organic materials from sludge, etc., are being contemplated. There are some municipalities conducting pilot plant studies for new energy recovery technologies.

Strategy V: Planning of wide-area sewage sludge treatment and disposal system

At the eighth US/Japan Conference on Wastewater Treatment Technology, Dr. Kubo, head of the Japanese Delegation, announced a scheme for wide-area treatment and disposal of sewage sludge in the catchment area of Tokyo Bay, giving a warning that an integrated system of sewage sludge treatment and disposal be established in good time as it is becoming increasingly difficult to find sites for sewage treatment and sludge disposal.

But in June 1981, the Law of Environmental Centers for Waste Reclamation was passed. Since then, the circumstances surrounding sewage sludge treatment and disposal have changed slightly. This law specifies a principal organization which should undertake the development of a large-scale reclamation site for wastes turned out in megalopolises. Naturally, the wastes to be handled by such an organization include sewage sludge. This means that sewage projects need not always make an effort for the procurement of dump sites. However, the disposal amounts and the costs of disposal are dependent on the location of dump sites, the amounts of other wastes, and disposal methods, etc. At this time, however, there are many unsettled factors as to the specifics of sludge disposal.

In 1982, the Osaka Bay Environmental Center for Waste Reclamation was chartered according to the Law of Environmental Centers for Waste Reclamation, and the sewerage operators in the greater Osaka Bay area are being urged to establish a proper system for sewage sludge disposal. In view of this, the Ministry of Construction is studying a total system for sewage sludge treatment and disposal. The study is concerned more with the integration of tactics or methodology than with independent tactics. The study flow is as shown in Fig. 7. Fig. 8 shows an example of a case study concerning the sludge transportation system proposed for the greater Osaka Bay area.

According to this example, it is proposed that a raw sewage collecting pipeline network interconnecting treatment plants located within 20 to 30 km from the joint dump site be provided for purpose of economy.

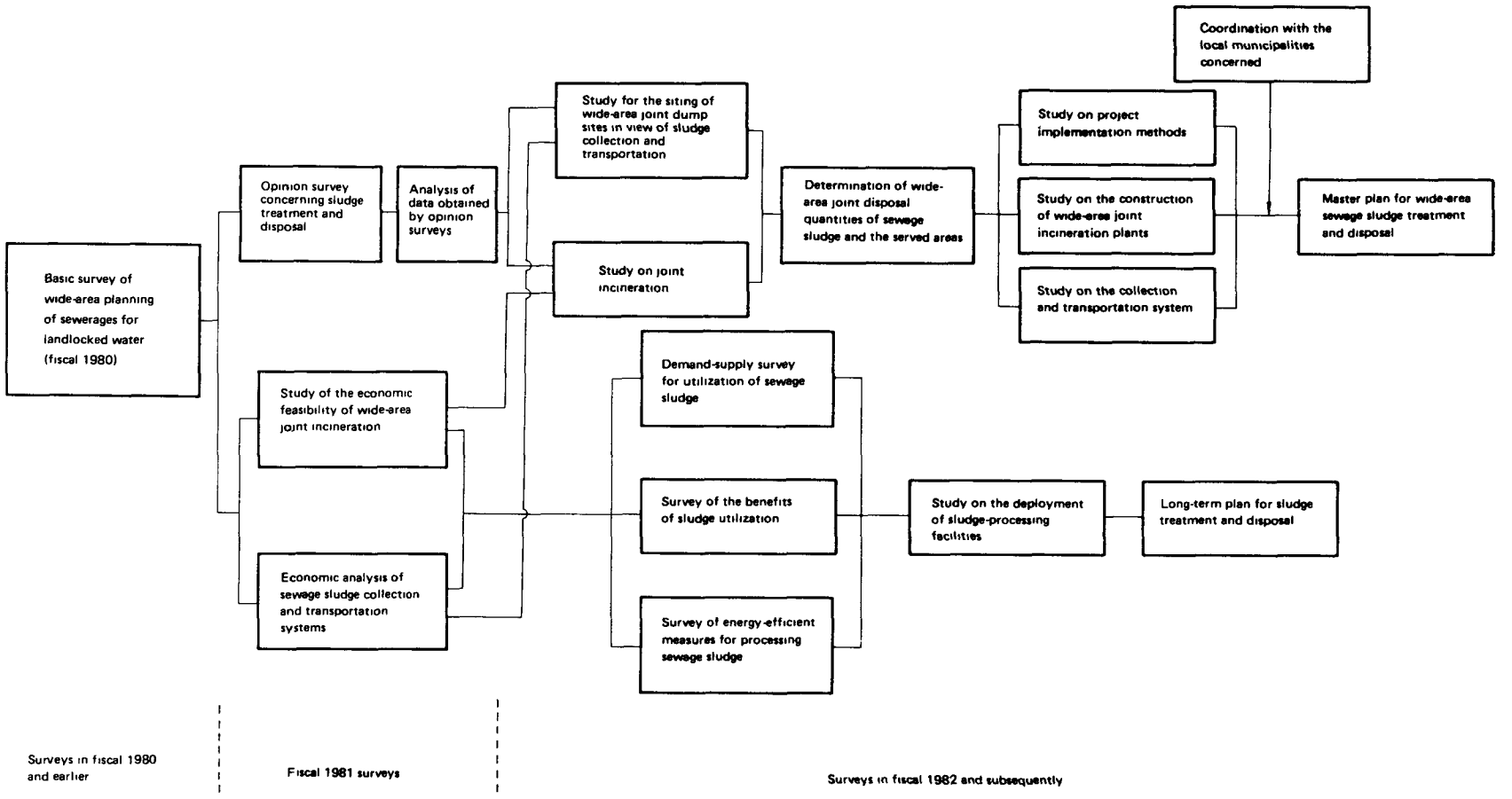


Fig. 7 Survey flow for the planning of wide-area sewage sludge treatment and disposal systems

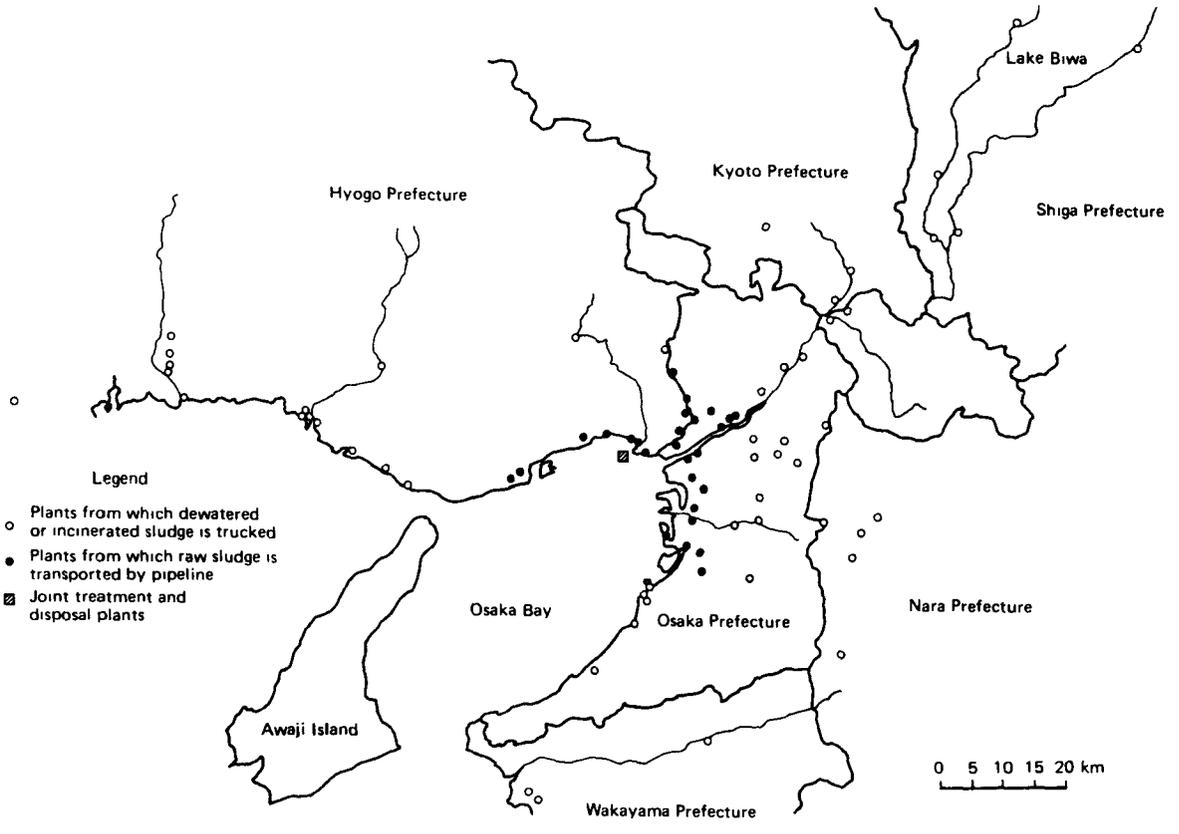


Fig. 8 A case study on a sludge transportation system for the Greater Osaka Bay Joint Sludge Treatment and Disposal System

7. CONCLUSIONS

Some of the current topics concerning sewage projects in Japan have been discussed. As stated in Chap. 1, the current socio-economic conditions in Japan are not always in favor of sewage works. Under these conditions, the sewage works operators are being called upon to tackle various problems highly crucial to the future of the disposal of Japan's ever-increasing domestic and industrial wastes.

The first priority is to find a cost-effective way to dispose of wastes within a limited budget. Investment should therefore be made selectively for maximum effects and quick returns. Specifically, we are asked to rack our brains for the selection of projects, control of capital investment, employment of economic processes, etc. In the future, we will be required to extend sewer services to small cities and rural communities. In these areas, the scale of sewerages will be small, and the investment efficiency will be low, as well. For this reason, sewerage systems will have to be changed to best meet local characteristics for the purpose of maximizing investment effects. The Small-scale Sewerage Planning Guidelines (draft) referred to in Chap. 4 has been formulated with these future needs in mind.

Secondly, the importance of sewerage systems must be emphasized to the people to get a strong support from the general public for the promotion of sewage projects. To this end, it will be necessary to pick model projects appealing to local inhabitants or the general public and to develop such techniques as are instrumental to playing up quantitatively and with cogency what sewerage brings about for the common good. The model projects touched upon in Chap. 5 have been implemented as part of this public enlightenment program.

Thirdly, financial policies for sewerage systems should be studied. The existing financial system should be reviewed to identify the problems, and the opinions of the public, government authorities, and academic and industrial circles should be polled in pursuit of an effective, rewarding, and truly workable financial system. In view of this, the Fifth Sewege Works Financial Policy Study Committee is addressing itself to study the future of sewage works financial policies.

Ninth US/Japan Conference
on
Sewage Treatment Technology

RECENT DEVELOPMENTS IN JAPANESE WATER QUALITY MANAGEMENT POLICY

Sept. 1983
Tokyo, Japan

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1. CURRENT STATE OF POLLUTION IN JAPANESE WATERS

The situation regarding water pollution in Japan has on the whole improved in the last few years, although there still remain many water bodies which stand far from "desirable" levels prescribed by the environmental water quality standards. According to the results of the nationwide water quality monitoring conducted in 1981, 0.05% of the samples analyzed contained cadmium or other toxic substances in concentrations exceeding the water quality standards related to the protection of human health. This figure represents a remarkable drop from 1.4% in 1970 and shows that the situation has continued to improve over the years. It is particularly noteworthy that in no places total mercury was found to exceed the standard, nor was there any single sample containing alkyl mercury or organic phosphorus. (Table 1-1)

Table 1-1. Ratio of Samples Exceeding Environmental Water Quality Standards in Terms of Toxic Substances (1981)

Item	Number of samples (A)	Number of samples exceeding environmental water quality standards (B)	Ratio (%) (B)/(A)
Cadmium	29,231	34	0.12
Cyanide	24,410	2	0.01
Organic phosphorus	9,022	0	0.00
Lead	29,339	8	0.03
Chromium (VI)	25,082	0	0.00
Arsenic	26,749	30	0.11
Alkyl mercury	7,933	0	0.00
PCB	4,126	0	0.00
Total*	155,892	74	0.05

Total mercury	Number of samples	Number of samples exceeding 0.0005 mg/ℓ	Number of places where readings exceed environmental quality standards
1981	31,023	29	0

* Excepting total mercury

Regarding BOD, COD, and other indices related to the preservation of the living environment, Tables 1-2 and 1-3 illustrate that of the total of 2,935 water areas (2,279 rivers, 103 lakes and reservoirs, and 553 sea areas) which had been classified by 1979, 1,938 water areas (1,443 rivers, 44 lakes and reservoirs, and 451 sea areas) accounting for 66.0% (68.7% for the previous year) have been found to meet the standard for BOD or COD, both typical indicators of water quality. Thus, on the whole, 34.0% of the total water areas have not attained the standards (Table 1-2).

It is true that among the water bodies surveyed there were some which had not yet reached their "target dates" for the attainment of the water quality standards, but even if they were excluded, many water bodies which had been classified up to 1976, with target periods falling under either "a" (immediately) or "b" (as early as possible within five years), only 70.8% have been found to meet the standards. This indicates that there is need for further strengthened efforts for water pollution control in the coming years. (Table 1-3)

Table 1-2. How Environmental Water Quality Standards are Met in Terms of BOD or COD

Water areas classified	Number of water areas (A)	Number of water areas meeting environmental quality standards (B)	Ratio % (B)/(A)
Total water areas	2,935	1,938	66.0
Water areas classified at the end of 1976	2,634 ["a" or "b" 2,296]	1,722 ["a" or "b" 1,625]	65.4 ["a" or "b" 70.8]
Water areas classified for 1977	123 ["a" or "b" 116]	98 ["a" or "b" 97]	79.7 ["a" or "b" 83.6]

Note: "a" and "b" refer to the target periods within which the environmental water quality standards are to be attained as follows:
a: immediately, b: as early as possible within five years.

The achievement rates for environmental standards in different water bodies are — rivers: 63.3% (previous year 67.2%), lakes and reservoirs: 42.7% (41.6%), and seas: 81.6% (79.8%). The achievement rates for small and medium-size rivers, lakes and reservoirs within major city areas, inland bays and inland seas, and other inland waterways remain unsatisfactory, and this fact has become one of the main characteristics of the pollution of public waters in recent years.

Table 1-3. How Environmental Water Quality Standards are Met in Terms of BOD or COD — By Type of Water Area

Category of water areas	Target period	Rivers (BOD)				Lakes and reservoirs (COD)				Sea areas (COD)			
		Number of water areas (A)	Number of water areas attaining environmental quality standards (B)	Ratio (B)/(A) (%)		Number of water areas (A)	Number of water areas attaining environmental quality standards (B)	Ratio (B)/(A) (%)		Number of water areas (A)	Number of water areas attaining environmental quality standards (B)	Ratio (B)/(A) (%)	
				1981	1980			1981	1980			1981	1980
AA	a	266	184	64	75	15	4	27	40				
	b	16	7	44	50	3	0	0	0				
	c	5	0	0	0	4	1	25	25				
A	a	773	609	79	83	37	26	70	65	189	134	71	74
	b	198	118	60	61	22	11	50	48	40	20	50	40
	c	55	12	22	22	7	1	14	14	7	1	14	14
B	a	238	172	72	79	7	1	14	14	133	125	94	89
	b	199	100	50	53	3	0	0	0	59	50	85	76
	c	68	16	24	33	4	0	0	0	7	4	57	71
C	a	77	59	77	76	1	0	0	0	81	81	100	100
	b	95	44	46	49	—	—	—	—	34	34	100	97
	c	67	13	19	22	—	—	—	—	3	2	67	67
D	a	14	12	86	93								
	b	33	19	58	66								
	c	32	16	50	47								
E	a	12	9	75	82								
	b	31	19	61	56								
	c	100	34	34	36								
Total	a	1,380	1,045	76	81	60	31	52	51	403	340	84	84
	b	572	307	54	56	28	11	39	38	133	104	78	70
	c	327	91	28	31	15	2	13	13	17	7	41	47
Total		2,279	1,443	63.3	67.2	103	44	42.7	41.6	553	451	81.6	79.8

2. WATER QUALITY IN ENCLOSED WATER AREAS

Enclosed water areas such as inland seas, inner bays, lakes and reservoirs are prone to suffer from a large influx of pollutants, particularly organic ones from various sources. The pollutants in turn tend to accumulate in these enclosed water bodies. Thus, they attain environmental quality standards to a lesser extent than other water areas and show progressive eutrophication or gradual degradation of water quality resulting from an influx of substances containing nitrogen and phosphorus as well as from an excessive algae and other aquatic plants.

Such enclosed water areas attained environmental water quality standards to the following extent in 1981: Tokyo Bay, Ise Bay and the Seto Inland Sea achieved 61%, 59% and 81% respectively, which are lower than the 84% for other sea areas, while lakes and reservoirs achieved only 42.7%. (Tables 2-1, 2-2, 2-3)

As for the number of red tides observed in 1981, 5 were in Tokyo Bay, 20 in Ise Bay and 55 in Seto Inland Sea, while fresh water red tides (in Lake Biwa, etc.) and 'aoko' (water-bloom caused by green algae) (in Kasumigaura, Lake Suwa, etc.) were seen in lakes and reservoirs.

Table 2-1. Trends in Water Pollution: Rate of achievement Environmental Water Quality Standards Relating to the Living Environment (BOD or COD)

	'74	'75	'76	'77	'78	'79	'80	'81
Rivers	51.3	57.1	57.6	58.5	59.5	65.0	67.2	63.3
Lakes and reservoirs	41.9	38.6	40.7	35.2	37.5	41.8	41.6	42.7
Coastal water areas	70.7	72.4	76.4	76.9	75.3	78.2	79.8	81.6
Tokyo Bay	44	44	67	61	61	61	61	61
Ise Bay	47	53	47	47	53	53	53	59
Seto Inland sea	67	69	72	73	75	76	72	81
Others	77	77	81	81	77	82	85	84
Total	54.9	59.6	60.6	61.2	61.7	66.7	68.7	66.0

Notes: 1. Rivers: BOD lakes and reservoirs, and coastal water areas: COD

2. The achievement rate (%):

$$\frac{\text{No. of water areas complying with environmental quality standards}}{\text{No. of water areas to which environmental quality standards are applied}} \times 100\ (\%)$$

Table 2-2. Water Quality of Major Coastal Water Areas (COD)

Unit: mg/ℓ

	Area	Categories of water quality standards	Standard value	'70	'76	'77	'78	'79	'80	'81
Tokyo Bay	Chiba Port	C	8	2.2	2.6	2.9	3.9	4.2	3.7	3.9
	Tokyo Port	C	8	3.8	3.4	3.4	3.3	3.1	3.1	3.8
	Yokohama Port	C	8	*3.4	1.8	2.4	3.1	3.1	3.2	3.5
Ise Bay	Nagoya Port	C	8	6.5	3.6	3.8	3.9	3.5	3.4	3.3
	Kinuura Port	C	8	*2.4	2.5	2.7	3.0	3.2	2.8	2.8
	Yokkaichi Port	B	3	3.9	2.7	2.8	3.1	4.3	3.3	3.4
Seto Inland Sea	Osaka Port	C	8	4.8	4.4	3.5	4.4	4.7	3.8	4.9
	Mizushima Port	C	8	2.7	2.0	2.1	2.0	2.6	2.4	2.2
	Iwakuni Port	B	3	3.0	2.6	2.7	2.0	2.5	1.8	2.1
	Tokuyama Port	B	3	*2.3	2.8	2.4	2.4	3.2	2.5	2.8
	Mishima-Kawanoe Port	B	3	2.9	2.3	2.0	2.1	2.5	2.5	1.8

Notes: 1. Figures in annual average

2. *1971 data

Table 2-3. Water Quality of Major Lakes (COD)

Unit: mg/ℓ

Lake	Categories of water quality standards	Standard value	'70	'76	'77	'78	'79	'80	'81
Lake Akan	AA	1	2.6	2.7	2.6	2.3	2.8	2.5	2.9
Lake Towada	AA	1	0.6	0.8	0.9	0.9	0.9	0.8	0.8
Lake Shinji	A	3	3.5	3.4	3.6	4.0	4.0	4.3	4.0
Kasumigaura	A	3	3.8	6.2	7.1	9.1	11.8	9.2	7.9
Lake Biwa (North)	AA	1	—	1.6	1.8	2.3	2.3	2.2	2.2
Lake Biwa (South)	AA	1	—	2.7	3.0	3.6	3.5	3.0	3.1
Lake Suwa	A	3	3.4	6.8	9.1	11	6.3	6.2	5.1
Teganuma	B	5	11	17	18	22	26	23	22

Note: Annual average.

3. MEASURES FOR EUTROPHICATION CONTROL

Eutrophication basically denotes the phenomenon of lakes and reservoirs being gradually enriched by nutrients such as nitrogen, phosphorus, etc. supplied from streams running into them. In recent years, lakes and reservoirs, in addition to inland seas and bays have begun to suffer from increasing levels of nutrients allowing algae and other aquatic life to propagate. This is resulting in progressive degradation of water quality, presenting problems of water quality management.

The degradation of water quality by eutrophication has given rise to various problems: losses in the beauty of lakes and reservoirs due to decreases in their transparency or changes in their colour, troubles in filtration of water purifying plants and offensive odor or bad taste of drinking water, in fish species and so on. Also, sea areas have long been troubled, by the red tides in fisheries.

In order to enable local governments as well as other organizations concerned to take measures for controlling the progress of eutrophication in lakes and reservoirs, the Environment Agency established, on December 25, 1982, environmental water quality standards for nitrogen and phosphorus, based on the recommendation of the Central Council for Environmental Pollution Control. A summary of the recommendation of the Council follows.

Table 3-1. Environmental Water Quality Standards on Nitrogen and Phosphorus in Lakes and Reservoirs

Category	Purpose of water use	Standard values	
		Total Nitrogen	Total Phosphorus
I	Conservation of natural environment, and uses listed in II-V	0.1 mg/ℓ or less	0.005 mg/ℓ or less
II	Water supply classes 1, 2 and 3 (excluding special types); Fishery type 1, bathing; and uses listed in III-V	0.2 mg/ℓ or less	0.01 mg/ℓ or less
III	Water supply class 3 (special types), and uses listed in IV-V	0.4 mg/ℓ or less	0.03 mg/ℓ or less
IV	Fishery type 2, and uses listed in V	0.6 mg/ℓ or less	0.05 mg/ℓ or less
V	Fishery type 3; industrial water; agricultural water; conservation of the living environment	1 mg/ℓ or less	0.1 mg/ℓ or less

Notes: 1. The standards are measured in terms of annual averages.

2. The standards for total phosphorus are not applicable to agricultural water uses.

3.1 Basic Views

3.1.1 Problems in various water uses arise when a body of water becomes eutrophic as a result of increased inflow and concentration of nitrogen and phosphorus. Diminished transparency of water impairs the scenic value of the water body. Eutrophication also gives tap water a foul smell and taste or filtration problems at water purification plants and causes fish and shellfish to die.

3.1.2 These problems result mainly from excessive growth of planktons, or "algal blooms."

3.1.3 The growth of algae and other plants is basically controlled by the levels of nitrogen and phosphorus present in the water.

On the basis of the elemental views cited above, environmental standards for nitrogen and phosphorus in lakes and reservoirs should be established in order to prevent nitrogen and phosphorus in lake water from causing excessive algal growth and thus maintain the desirable state of the living environment.

3.2 Water Uses and Environmental Conditions

3.2.1 Nature Conservation

The eutrophication-caused multiplication of algae reduces the transparency of water and turns the color of water green or brown. This results in a deteriorated scenery and other adverse effects on the natural environment.

With a view towards maintaining the quality of water in the lakes and reservoirs (such as Lakes Mashu and Shikotsu in Hokkaido), which ensures sufficient transparency nitrogen (meaning total nitrogen) should be held below 0.1 mg/ℓ and phosphorus (meaning total phosphorus) below 0.005 mg/ℓ.

3.2.2 Drinking Water

Among the problems in water supply, which are caused by eutrophication, are smelling or foul taste of water and problems in water purification operations, such as clogging in the filtration system, which results from multiplied algae.

Since the patterns of the problems vary, different approaches were taken in the deliberation of the Council so as to take into account such specific patterns corresponding to the categories of water use as well as to respective water treatment processes adopted.

(a) Category 1 Drinking Water Supply

In the case of the category 1 drinking water supply (which requires simple purification processes such as filtration), smelling substances and others are decomposed and removed in the course of low speed filtering. While this seems to prevent the problem of smelling or foul taste of water, the filtering basin sometimes gets clogged because of multiplied algae. When this happens, the basin's filtering capacity sharply diminishes.

Considering the quality of water in those water-supplying lakes and reservoirs which have faced with filtering problems (such as the southern part of Lake Biwa, the Murayama Reservoir, and Lake Nojiri), and that of other trouble-free lakes, nitrogen should be kept below 0.2 mg/ℓ and phosphorus below 0.01 mg/ℓ in setting the standard values for the category 1 water supply.

(b) Category 2 and 3 Drinking Water Supply

The category 2 drinking water supply involves sedimentation, filtering, and other normal purification processes. The category 3 drinking water supply involves more sophisticated purification processes, which include preparatory treatment.

Various kinds of purification trouble arise in their cases. The presence of multiplied algae in water sometimes requires an increase in the amount of drugs consumed in the coagulation and sedimentation pond and shortens the life time of the rapid-filtering basin.

Considering the water quality in water-supplying lakes and reservoirs which have experienced these problems (such as Lakes Kasumigaura and Sagami and the Hatake Reservoir) and that of other trouble-free lakes, nitrogen should be held below 0.4 mg/ℓ and phosphorus below 0.03 mg/ℓ if the normal functions of water purification facilities are to be maintained.

Since the removal of smelling substances is difficult through the normal purification processes, their presence sometimes causes foul odor or taste bad in tapped water. Problems of smelling or foul taste of drinking water have recently arisen in various parts of the country. Considering the situations of such problems, as reviewed by a Countermeasures Panel of Japan Water Supply Association and other bodies, and the quality of water in trouble-free water-supplying lakes and reservoirs, nitrogen should be kept below 0.2 mg/ℓ and phosphorus below 0.01 mg/ℓ so as to prevent smelling or foul taste of tap water, such as mold odor.

In the case of some of the category 3 where removal of smelling substances is ensured through normal purification operations, nitrogen should be held below 0.4 mg/ℓ and phosphorus below 0.03 mg/ℓ.

3.2.3 Bathing

A representative inland water body used as a bathing resort is Lake Biwa (northern section). Although this section is generally in a satisfactory condition, some of the bathing shores are not. Considering the good quality of water in the same section in the mid-1960s, nitrogen should be held below 0.2 mg/ℓ and phosphorus below 0.01 mg/ℓ in setting the standard values for bathing areas.

3.2.4 Fisheries

Trout and other species of the salmon family and "ayu" (sweetfish) flourish in lakes and reservoirs with low levels of nitrogen and phosphorus. "Wakasagi" (smelt) live in water bodies with medium levels of nitrogen and phosphorus. Species with strong resistance against pollution, such as carp and "funa" (crucian carp) dominate where concentrations of these substances are high.

Accordingly, the above groups of species were used as representative species in the deliberations on environmental quality standards to protect fishery resources. The underlying concept was to set the standards at levels low enough for each species to live and permit natural reproduction and growth.

(a) Category 1 (species of the salmon family and "ayu")

Species of the salmon family, such as "hime masu" (*Oncorhynchus nerka*) and "ayu" live where there is clean water bodies such as Lakes Chuzenji and Biwa.

The principal species in Lake Chuzenji is changing from "hime masu" to "wakasagi" as eutrophication is deteriorating the quality of water. The quality of water in the northern section of Lake Biwa remains generally fit for "ayu".

In view of these facts, nitrogen should be held below 0.2 mg/ℓ and phosphorus below 0.01 mg/ℓ when the standard values for category 1 are set.

(b) Category 2 ("wakasagi"-type species)

Among the bodies of water with high "wakasagi" output are Lakes Suwa and

Hachiro. But production in Lake Hachiro has been falling in the recent years as the quality of water is no longer fit for "wakasagi." Production in Lake Suwa has been leveling off. But existing levels have been maintained by the release of fries. If conditions for natural reproduction and growth are to be secured, improvement in the quality of water is warranted.

Considering these points, nitrogen should be kept below 0.6 mg/ℓ and phosphorus below 0.05 mg/ℓ when the standard values for category 2 are set.

(c) Category 3 (carp and "funa")

The proportion of carp and "funa" in fishery resources is tending to increase both in percentage terms and quantitatively in keeping with the rise in nitrogen and phosphorus concentrations. But problems arise when nitrogen exceeds 1 mg/ℓ and phosphorus 0.1 mg/ℓ.

Lake Kojima and two marshes — Teganuma and Inbanuma — have these substances in excess of these levels. Over the years, production in these bodies of water has been leveling off or falling. In Teganuma and Inbanuma there have been such problems as smelling fish flesh and dead fish.

In view of these facts, nitrogen should be held below 1 mg/ℓ and phosphorus below 0.1 mg/ℓ when the standard values for category 3 are set.

3.2.5 Water for Agricultural Use

Paddy rice plants are apt to overgrow during their principal growth period (about 40 days after seedlings are transplanted in paddies from their beds) and thus become liable to disease when there is too much nitrogen, particularly ammonia nitrogen, in water for agricultural use. The presence of excessive nitrogen in the stage of ear differentiation causes plants to grow too long between the lower joints. When plants fall down or end up being immature as a result, the yield suffers.

Considering these points, nitrogen should be held below 1 mg/ℓ in setting the standard values for irrigation water.

3.2.6 Industrial Water

The multiplication of algae, which results from eutrophication, makes water turbid, and this gives rise to problems in the use of industrial water.

The levels of nitrogen and phosphorus in the major lakes and reservoirs from which water is drawn for industrial purposes, such as Lakes Kasumigaura and Biwa, are generally below 1 mg/ℓ and 0.1 mg/ℓ respectively. No problems will arise when they are at these levels.

In view of this, the standard values for industrial water should be set below 1 mg/ℓ for nitrogen and below 0.1 mg/ℓ for phosphorus.

3.2.7 Environmental Conservation

The progress of eutrophication leads to the abnormal multiplication of algae and causes tall water grasses to flourish. The offensive odor, given off by them when they are decomposed, causes people discomfort. Among the bodies of water in this condition are Inbanuma and Lake Kojima.

Considering the quality of water in these lakes and reservoirs, the standard values for the conservation of the environment should be set below 1 mg/ℓ for nitrogen and below 0.1 mg/ℓ for phosphorus.

3.3 Effluent Standards for Nitrogen and Phosphorus

In an effort to attain the environmental water quality standards, the Water Pollution Control Law lays down the national effluent standards for specified facilities which discharge wastewater into public water areas. For water areas where the application of the uniform national effluent standards is judged insufficient to attain the environmental water quality standards, the Law provides that stricter effluent standards may be set by an ordinance of the prefecture having jurisdiction over that perticular water body.

At present, the possibility and scientific basis for establishing effluent standards for nitrogen and phosphorus are being discussed deliberately by the Central Council for Environmental Pollution Control.

4. A NEW LEGISLATION FOR LAKE WATER PROTECTION

On May 13, 1983, after more than two years of arduous inter-departmental negotiation within the government, a draft bill for the preservation of water quality in lakes and reservoirs was decided upon by the Cabinet and was presented on the same day to the Diet (Japanese National Parliament). The Government-proposed bill was then sent to the Environment Committee of the House of Representatives (Lower House), but, due to shortage of time before its closure, the bill was not debated upon to any extent during the current session. The Committee instead allowed the Director-General of the Environment Agency to give a brief introductory statement before it, and decided to "continue its delicerations" (e.e. postpone its decision) until the next session of the Diet.

In light of the fact that the Environment Committee itself and the Special Committee on Environmental Pollution and Traffic Safety of the House of Councillors (Upper House) had in the previous year both passed almost idential resolutions calling for the Government's strengthened efforts to protect the natural environment as well as water quality of lakes and reservoirs around Japan and for the promotion of a new legislation toward that end, the Government-proposed bill is likely to meet little opposition in both Houses, and, if all goes well, is expected to become a law during the next extraordinary session of the Diet to be opened in the fall of this year.

An outline of the proposed legislation follows:

Under the provisions of the proposed bill,

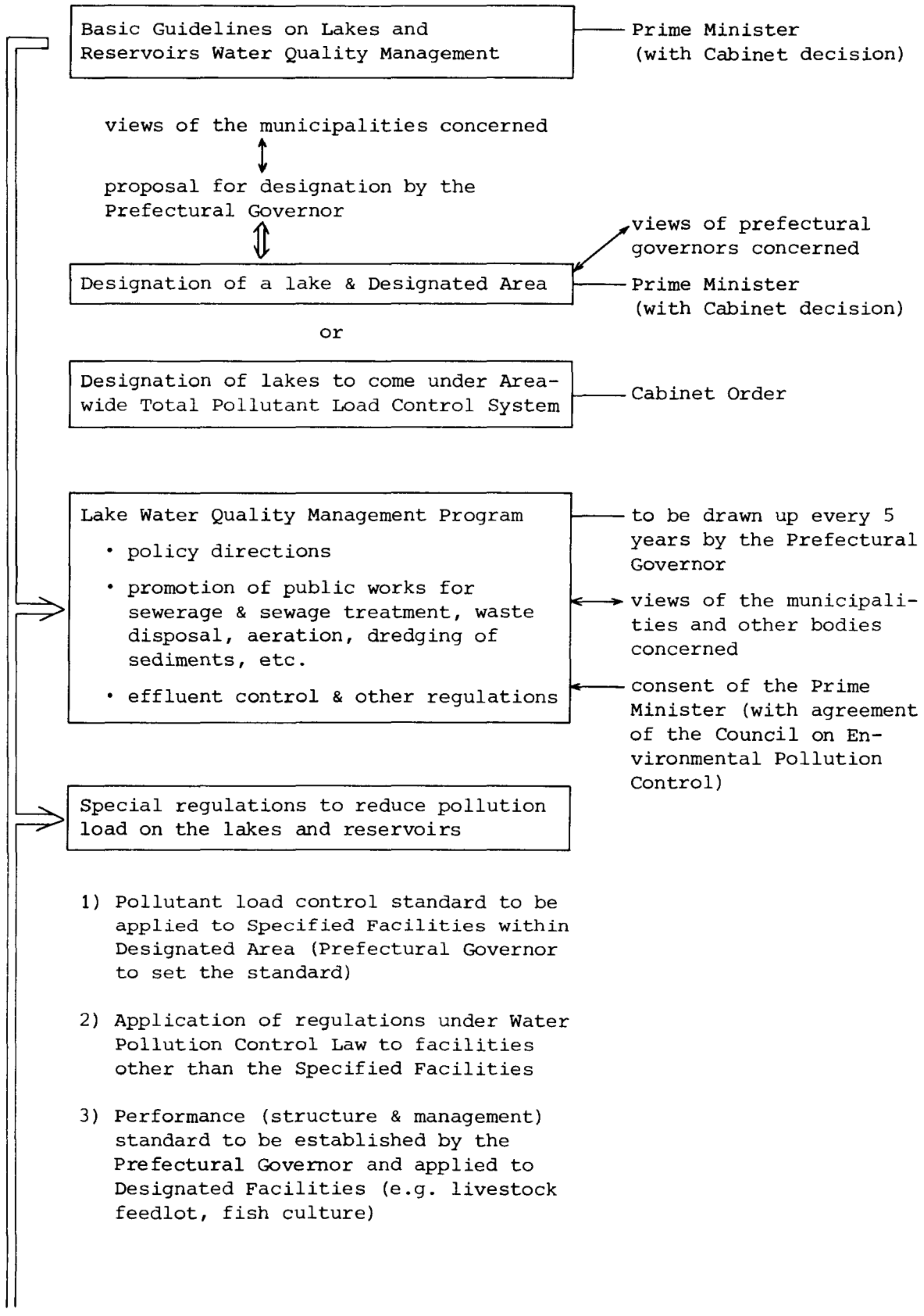
(1) The Government shall provide basic policy guidelines for the protection of lakes and reservoirs.

(2) The prefectural governors shall draw up environmental protection plans regarding lakes and reservoirs which are threatened with degradation of its water quality and considered to require comprehensive and systematic measures of protection to overcome the problem. The national and municipal authorities should carry out the plans accordingly.

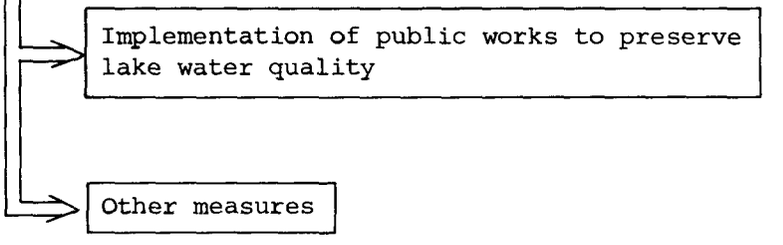
(3) Various measures for water quality protection, such as establishment of sewerage, should be taken steadily according to the characteristics of lakes and reservoirs under a long-term and comprehensive plan so that the measures as a whole will achieve the desired results. Such plans should include the strengthening of effluent control on factories and other business premises, permit for the establishment of specified facilities, prevention of pollution caused by small-scale livestock feedlots, fish culture grounds, domestic waste water, agricultural drainage, etc., and introduction of area-wide total pollutant load control system.

(4) The State shall give maximum possible financial aid to municipalities, while both the State and municipalities should strive to provide factory operators and others with necessary assistance regarding financing and taxation.

< Outline of the Proposed Legislation >



4) Introduction of Areawide Total Pollutant Load Control System to the lakes designated by Cabinet Order



- administrative guidance, advice, recommendation to domestic and other sources of pollution
- preservation of the lakeside natural environment
- financial assistance and aid-in-grant to prefectures and municipalities, cooperation between governmental organizations concerned, etc.

Ninth US/Japan Conference
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Sewage Treatment Technology

UPGRADING OF ANAEROBIC DIGESTION PROCESS

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1. INTRODUCTION

Japan is the second largest oil-consuming country in the free world, and its energy sources are almost entirely dependent on imports. Securing a stable source of energy supply, promoting energy-saving measures, and at the same time developing the alternatives to oil to minimize the dependency on oil are of critical importance to Japan. Fully realizing the significance of the matter, we investigated a way of saving energy in the wastewater treatment plants which are now increasingly growing in number and also of utilizing potential energy resources available there.

As of 1980, there were some 480 wastewater treatment plants in Japan, and their total annual energy consumption was something like 2,000 million kWh in electric power and about 460,000 kl in heavy oil. As the number of wastewater treatment plants will continue to grow in the future, their energy consumption will increase in proportion.

In the wastewater treatment plant, the anaerobic digestion process is operated mainly for the purpose of reducing the volume of sludge produced and at the same time stabilizing its quality. It gives off digester gas, including methane, with a heat value of 5,000 to 6,000 kcal per Nm^3. So far the digester gas has been used in Japan only for heating the digestion tank. If we could increase the output of digester gas, the anaerobic digestion process would be turned into a useful energy source.

This paper is primarily concerned with the following.

(1) Present status of anaerobic digestion in Japan
(2) Gas yields of primary sludge and excess sludge
(3) Effects of high-concentrated feed sludge on gas yield
(4) Increase of gas yield by the thermal conditioning of feed sludge

2. PRESENT STATUS OF ANAEROBIC DIGESTION IN JAPAN

As touched upon earlier, there were some 480 wastewater treatment plants in Japan as of 1980, of which about 180 were equipped with digestion tanks. As shown in Table 1, 156 out of these 180 wastewater treatment plants were operated on a system involving a thickening process, anaerobic digestion process, and dewatering process and disposal process.

The sludge treated in this way amounted to about 240,000 tons a year in terms of solids of thickened sludge, or about 26% of about 930,000 tons produced by all the systems. Of the 180 wastewater treatment plants, 17 were operated on a thickening-anaerobic digestion-dewatering-incineration system, accounting for about 24% of the total amount of sludge generated. All told, about a half of the sludge was treated by the digestion process.

From Table 1, it can also be said that:

(1) incinerated ash (56%) and dewatered sludge (43%) accounts for nearly 99% of the total quantity of sludge disposed of and that

(2) the incinerated ash is largely classified into two sludge treatment systems - one consisting of thickening, dewatering and incineration and the other consisting of thickening, anaerobic digestion, dewatering and incineration. The plants operating on these two systems were 55 and 17 respectively, turning out sludge at 290,000 tons/yr. and 220,000 tons/yr. In most large-scale wastewater treatment plants, incineration was preceded by an anaerobic digestion process.

The majority of those wastewater treatment plants with an anaerobic digestion tank were operated as follows.

- Two-stage digestion with 30 days of solids retention,
- mesophitic digestion at a temperature of $30^{\circ}C$ to $37^{\circ}C$,
- steam injection heating,
- agitation by gas mixing

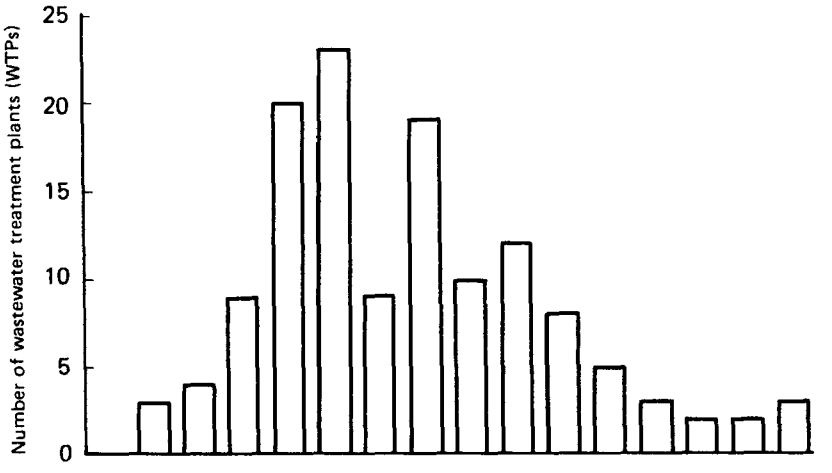
Table 1 Applied sludge treatment processes

Form of disposed sludge	Sludge treatment processes	Number of WTPs		Treated solids based on thickened sludge	
		Having the processes	Disposed by the processes	x1,000 tons/year	%
Liquid	Withdrawal	5(0)	5	-	0
	Thickening	19(16)	3	3.88	0.41
	Thickening-Anaerobic Digestion	3(2)	1	1.82	0.19
	Thickening-Aerobic Digestion	11(8)	3	0.02	0.0
	TOTAL	38(26)	12(3)	5.72	0.61
Dewatered	Thickening-Dewatering	116(12)	104	138.07	14.77
	Thickening-Anaerobic Digestion-Dewatering	156(7)	163	243.46	26.04
	Thickening-Aerobic Digestion-Dewatering	17(1)	17	2.31	0.24
	Thickening-Wet Oxidation-Dewatering	1(0)	1	9.16	0.98
	Thickening-Thermal Conditioning-Dewatering	6(0)	6	7.43	0.79
	TOTAL	296(20)	291(65)	400.43	42.84
Dried	Thickening-Drying	6(0)	5	0.00	0.0
	Thickening-Anaerobic Digestion-Drying	1(0)	1	-	0
	Thickening-Aerobic Digestion-Drying	1(0)	1	0.03	0.0
	Thickening-Dewatering-Drying	2(0)	2	1.88	0.20
	Thickening-Anaerobic Digestion-Dewatering-Drying	7(1)	6	4.86	0.51
	TOTAL	17(1)	15(3)	6.77	0.72
Composted	Thickening-Dewatering-Composting	1(0)	1	0.37	0.03
	Thickening-Anaerobic Digestion-Dewatering-Composting	3(0)	3	2.08	0.22
	TOTAL	4(0)	4(1)	2.45	0.26

Table 1 Applied sludge treatment processes1 (Cont'd)

Form of disposed sludge	Sludge treatment processes	Number of WTPs		Treated solids based on thickened sludge	
		Having the processes	Disposed by the processes	x1,000 tons/year	%
Incinerated Ash	Thickening-Dewatering-Incineration	55(0)	82	286.72	30.67
	Thickening-Anaerobic Digestion-Dewatering-Incineration	17(0)	34	221.33	23.67
	Thickening-Aerobic Digestion-Dewatering-Incineration	0(0)	8	0.68	0.07
	Thickening-Thermal Conditioning-Dewatering-Incineration	4(0)	4	10.58	1.13
	TOTAL	76(0)	128(100)	519.31	55.56
TOTAL		431(47)	450(100)	934.68	100
WTPs having sludge treatment facilities and transporting sludge to other WTPs.		40	-		
Unknown and no sludge production		6	27		
TOTAL		477	477	934.68	100

As shown in Fig. 1, about 50% of the wastewater treatment plants were operating at a gas yield (gas production/feed sludge volume) not exceeding 5:6, reportedly lower than similar treatment plants in the west.



Gas yield	0 ~ 1	1 ~ 2	2 ~ 3	3 ~ 4	4 ~ 5	5 ~ 6	6 ~ 7	7 ~ 8	8 ~ 9	9 ~ 10	10 ~ 11	11 ~ 12	12 ~ 13	13 ~ 14	14 ~ 15	15 ~
Number of WTPs	0	3	4	9	20	23	9	19	10	12	8	5	3	2	2	3

Fig. 1 Frequency distribution of gas yield (gas production/feed sludge) (1980)

A low gas yield is disadvantageous from the viewpoint of energy recovery. The following are cited as reasons for the low gas yield.

(1) Difference in eating habits

The Japanese take animal proteins and fats far less than the peoples in the West.

(2) Difference in water temperature (ambient temperature)

The ambient temperatures are higher in Japan than the countries at high latitudes, and the sludge is more liable to decompose during transportation and treatment process, reducing the gas production at the digestion task.

(3) Difference in secondary treatment

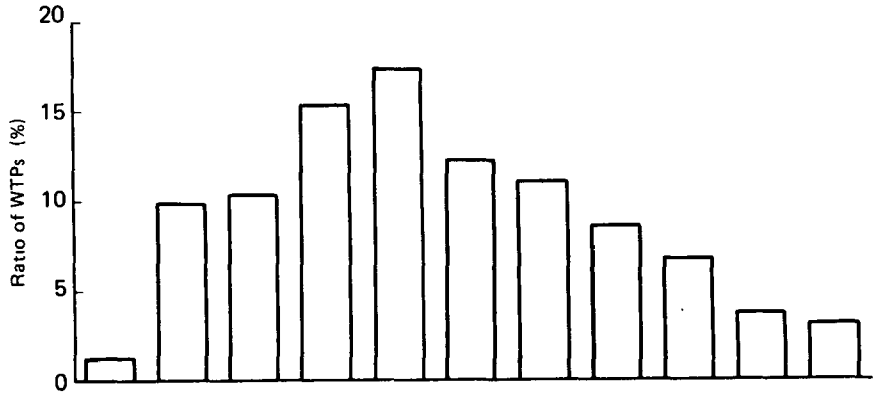
In Japan, the aeration time for secondary treatment is generally longer than that in Europe and the U.S.A. Namely, the sludge itself has already been reducing the gas production when it enters the digestion tank.

(4) Difference in sidestreams from sludge treatment processes to water treatment process

The flow of the sidestreams and SS loadings in them are said to be larger than in Europe and the U.S.A. As the sludge that has little potential to generate gas is circulated, the apparent gas yield is reduced.

While the four factors are conceivably responsible for the low gas yield, we cannot be categorical about which is the principal cause. Now we shall discuss the effects of sidestreams.

Fig. 2 is a histogram showing the frequency distribution relationship between the supernatant ratio (supernatant of digested liquor/feed sludge volume) and the number of digestion tanks as of 1980.



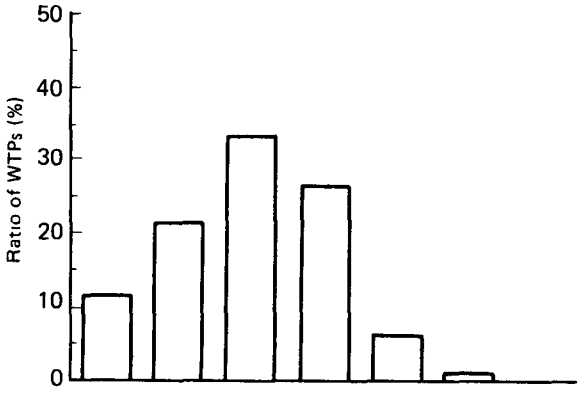
Supernatant ratio	100	100 ≳ 90	90 ≳ 80	80 ≳ 70	70 ≳ 60	60 ≳ 50	50 ≳ 40	40 ≳ 30	30 ≳ 20	20 ≳ 10	10 ≳ 0
Number of WTPs	2	16	17	25	28	20	18	14	11	6	5

Fig. 2 Frequency distribution of supernatant ratios

It is found from this figure that more than a half of the national total of the digestion tanks were operated at a supernatant ratio of 60% or higher. The SS concentration in the supernatant is extremely high, and is one of the causes of the reduction of gas yield as discussed in item (4) above.

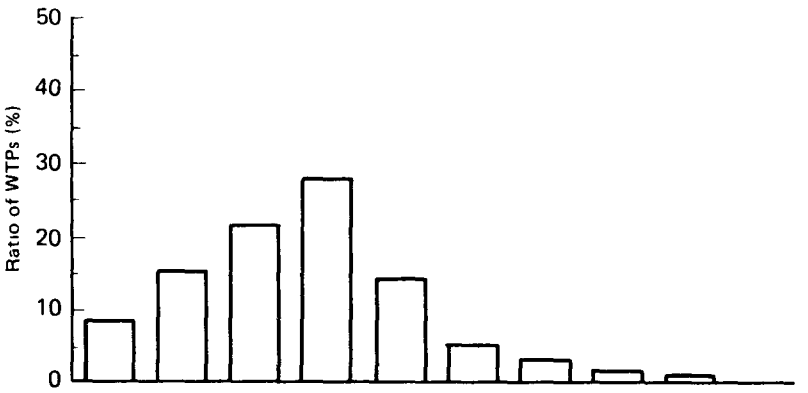
The next point for consideration is the concentration of digestion tank feed sludge and draw-off sludge.

Figs. 3 and 4 show the frequency distribution of the concentrations of digestion tank feed sludge and draw-off sludge; the concentrations of feed sludge or thickened sludge were less than 2% in about 30% of the wastewater treatment plants in Japan and less than 3% in 70% of the wastewater treatment plants.



Solids concentration	0 ~ 1	1 ~ 2	2 ~ 3	3 ~ 4	4 ~ 5	5 ~ 6	6 ~ 7
Number of WTPs	26	47	74	59	14	2	

Fig. 3 Frequency distribution of feed sludge concentrations

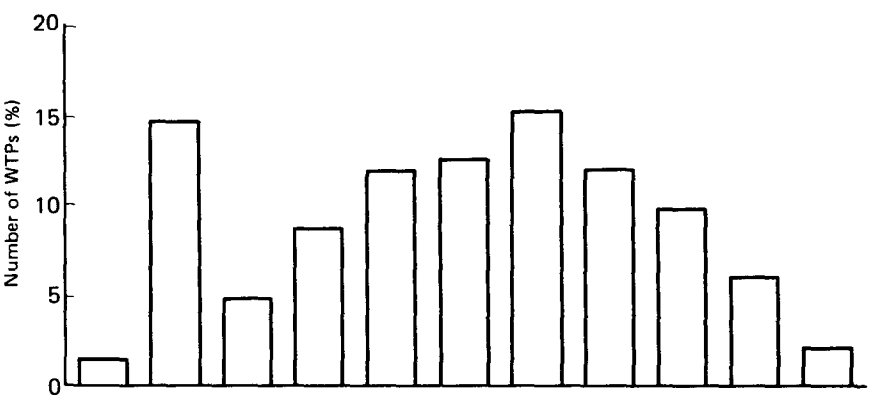


Solids concentration	0 ≀ 1	1 ≀ 2	2 ≀ 3	3 ≀ 4	4 ≀ 5	5 ≀ 6	6 ≀ 7	7 ≀ 8	8 ≀ 9	9 ≀ 10
Number of WTPs	16	29	41	53	27	10	6	3	2	

Fig. 4 Frequency distribution of draw-off sludge concentrations

In Japan, the concentrations of the thickened sludge are generally low. This may be attributed to the decomposition of sludge in the thickener due to high mean temperatures and also to the reduction of solids-liquid separation performance due to decomposition. From Figs. 3 and 4, it is found that the concentration of solids in the sludge increases after passing through the digestion tank.

Fig. 5 shows the mass balance in the digestion tank in terms of draw-off solids ratio (solids in draw-off sludge/solids in feed sludge).

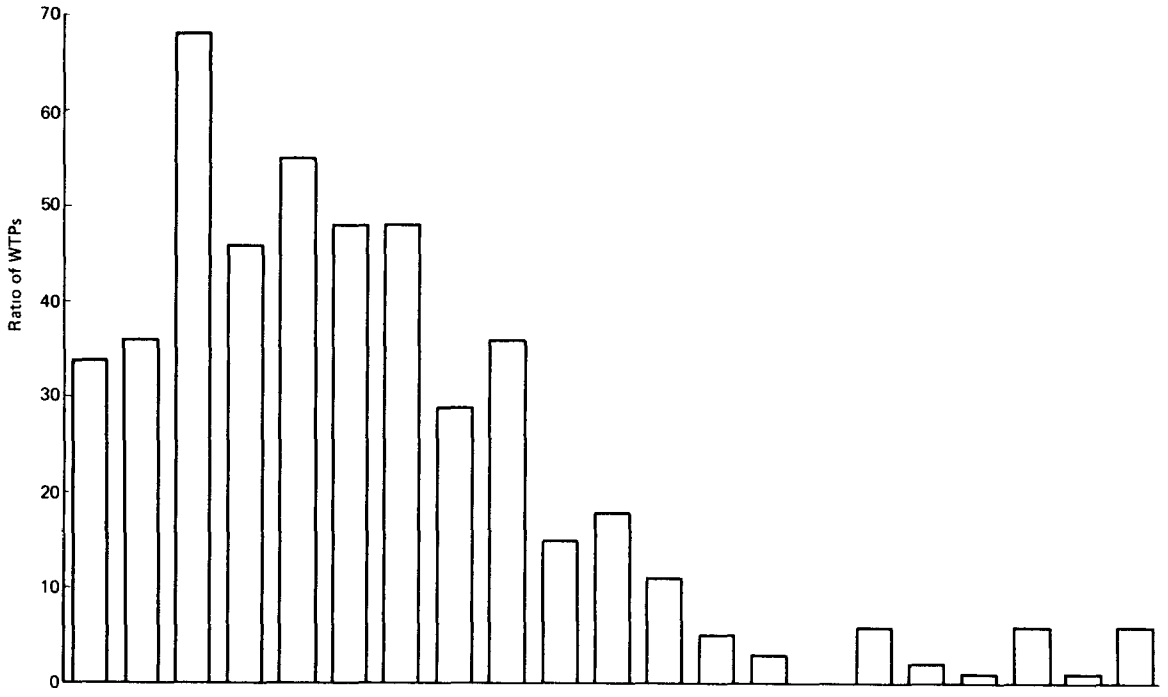


Draw-off solids ratio	100	100 ≀ 90	90 ≀ 80	80 ≀ 70	70 ≀ 60	60 ≀ 50	50 ≀ 40	40 ≀ 30	30 ≀ 20	20 ≀ 10	10 ≀ 0
Number of WTPs	3	27	9	16	22	23	28	22	16	11	4

Fig. 5 Frequency distribution of draw-off solids ratios

Those wastewater treatments which showed a draw-off solids ratio of more than 90% account for about 15%. In these plants, 90% of the solids in the feed sludge was sent as digested sludge to the succeeding processes, including dewatering process, and the remaining 10% was turned into digestion gas or returned in the form of supernatant.

As shown in Fig. 6, the unit capacity of digestion tank varies over a wide range from several hundred cubic meters to 5,000 cubic meters.



Digestion tank capacity (m3)	0 ~ 500	500 ~ 1,000	1,000 ~ 1,500	1,500 ~ 2,000	2,000 ~ 2,500	2,500 ~ 3,000	3,000 ~ 3,500	3,500 ~ 4,000	4,000 ~ 4,500	4,500 ~ 5,000	5,000 ~ 5,500	5,500 ~ 6,000	6,000 ~ 6,500	6,500 ~ 7,000	7,000 ~ 7,500	7,500 ~ 8,000	8,000 ~ 8,500	8,500 ~ 9,000	9,000 ~ 9,500	9,500 ~ 10,000	10,000 ~
Number of WTPs	34	36	68	46	55	48	48	29	36	15	18	11	5	3		6	2	1	6	1	6

Fig. 6 Frequency distribution of digestion tank unit capacities

Namely, the size of the digestion tank is determined to meet the sludge processing needs, and there is no size that is identified as optimal. Most of digestion tanks are designed to be cylindrical.

3. GAS YIELD OF PRIMARY SLUDGE AND EXCESS SLUDGE

This chapter is primarily concerned with the gas production by anaerobic digestion of primary sludge, excess sludge and mixed sludge and the causes of the differences in gas yields. To investigate these, laboratory tests were conducted as reported here.

3.1 TEST METHOD

Using an arrangement shown in Fig. 7, we conducted a test for 8 weeks at a digestion temperature of 30°C with the solids retention time set at 30 days.

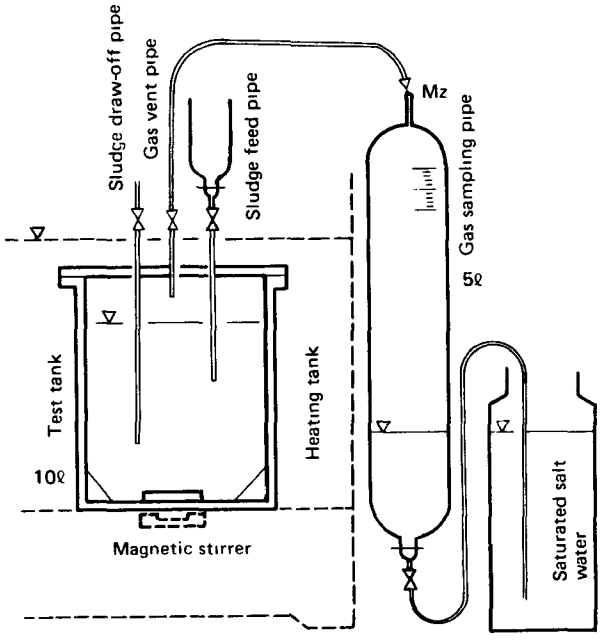


Fig. 7 Digestion test arrangement

During the test period, sludge was fed and drawn off once a day. The sludge used for the test is classified into six types as shown in Table 2.

Table 2 Characteristics of feed sludge

Sludge	TS mg/ℓ	VS mg/ℓ	T-C mg/ℓ	T-N mg/ℓ	T-P mg/ℓ	C/N	Heat value cal/g	WTPs from which sludge was sampled
1-Primary sludge	40,100	(84.6) 33,900	(42.1) 16,890	(4.2) 1,690	(0.8) 304	10.0	4,310	Separated sewer system 26,000 m^3/D Conventional activated sludge process No sludge treatment facility
1-Excess sludge	37,100	(85.0) 31,500	(48.2) 17,870	(9.8) 3,640	(1.4) 519	4.9	4,900	
2-Primary sludge	38,400	(61.8) 23,700	(31.0) 13,060	(5.2) 2,000	(1.1) 440	6.5	3,580	Separated but partially combined sewer system 88,000 m^3/D Conventional activated sludge process Thickening-Dewatering
2-Excess sludge	42,600	(69.2) 29,500	(33.4) 14,220	(6.7) 2,850	(1.1) 465	5.0	3,660	
3-Mixed sludge	37,900	(67.6) 25,600	(36.8) 13,950	(4.7) 1,770	(1.2) 449	7.9	3,730	Combined sewer system 90,000 m^7/D Conventional activated sludge process Thickening-Anaerobic Digestion-Dewatering
3-Excess sludge	41,000	(77.9) 31,900	(41.7) 17,080	(8.6) 3,520	(1.5) 619	4.9	4,370	

Note: Values in parentheses are percentages to TS.

From three wastewater treatment plants, primary sludge and excess sludge were sampled. At No. 3 wastewater treatment plant, mixed sludge was sampled instead of primary sludge. The sampled sludge was pulverized and screened to remove large blocks. Then, it was adjusted to a solids concentration of about 4% for test.

Before test, 7 lit. of the digested sludge was charged into each test tank as seed sludge, and 1 lit. of the test sludge was injected in about 2 weeks for acclimation of the seed sludge.

3.2 RESULTS

During the 8-week test period, the weekly production of total gas volume and methane gas volume per gram of organics (VS) in the feed sludge was measured as shown in Figs. 8 and 9. The gas production was almost constant over the entire test period.

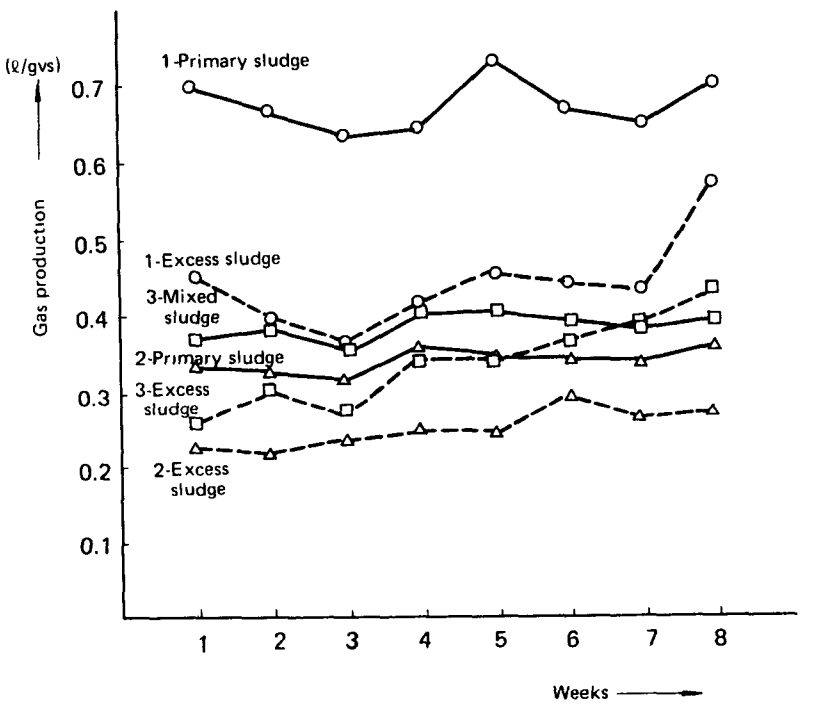


Fig. 8 Gas production per gram of feed VS

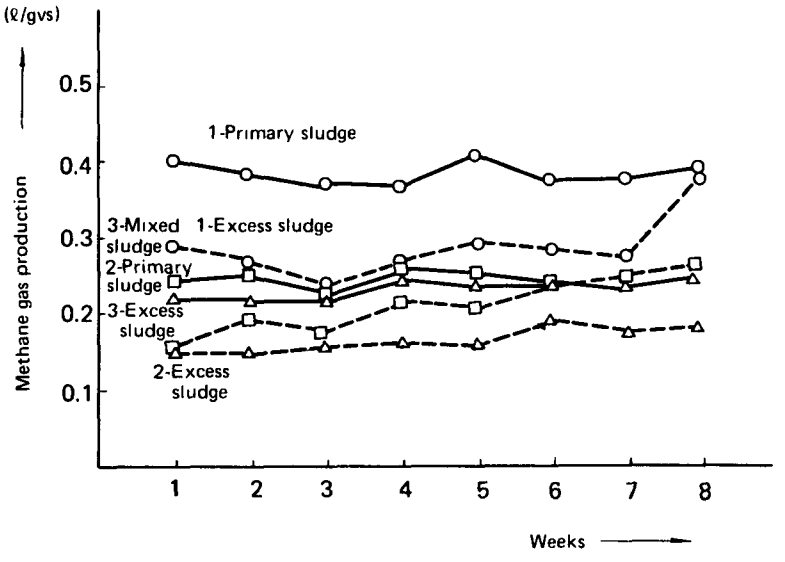


Fig. 9 Methane gas production per gram of feed VS

Table 3 compares the six types of sludge with respect to the mean value of gas production in the latter half (4 weeks) of the test period.

Table 3 Gas production

	Gas yield (ℓ/g.VS)	Methane yield (ℓ/g.VS)	Methane ratio, %
1-Primary sludge	0.688	0.389	56.5
1-Excess sludge	0.478	0.306	64.0
2-Primary sludge	0.349	0.238	68.2
2-Excess sludge	0.274	0.179	65.3
3-Mixed sludge	0.393	0.243	61.8
3-Excess sludge	0.385	0.238	61.8

Fig. 10 shows the relationship between the gas production per gram of VS and the organics ratio (VS/TS).

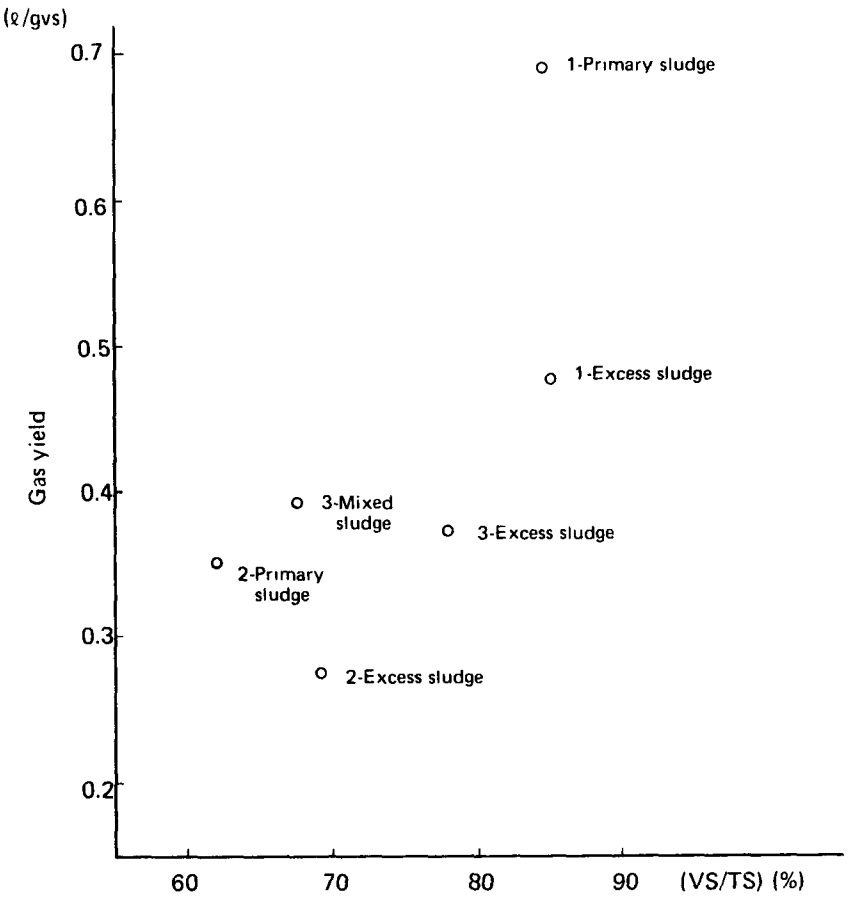


Fig. 10 Content of organics in sludge, and gas yield

Tables 2 and 3 and Fig. 10 show the following.

(1) For each specific wastewater treatment, the gas yield per gram of VS is greater in the primary sludge than in the excess sludge, and the gas production in the mixed sludge comes in between.

(2) For either excess sludge or primary sludge the greater the organics ratio, the larger the gas production per gram of VS.

(3) In the separated sewer wastewater treatment plant where there are no sidestreams from the sludge treatment process, the organics ratio in the sludge is higher than in other plants.

(4) The methane gas ratio in the produced gas is less influenced by the type of sludge, and is in the range of 55 to 70%, or about 60% on the average.

In order to investigate why the primary sludge produces more gas than the excess sludge, the composition of organics was analyzed.

Tables 4 and 5 show the analysis of carbohydrates, fats, and proteins in the feed sludge and digested sludge tested.

Table 4 An analysis of organics in feed sludge

Feed sludge	TS mg/ℓ	VS %	Total carbohydrate, %	Total fat, %	Total protein, %
1-Primary sludge	40,100	84.6	49.2	7.0	11.9
1-Excess sludge	37,100	85.0	13.2	26.9	27.6
2-Primary sludge	38,400	61.8	6.9	12.2	26.3
2-Excess sludge	42,600	69.2	6.0	12.1	26.3
3-Mixed sludge	37,900	67.6	19.5	8.1	26.3
3-Excess sludge	41,000	77.9	9.0	20.3	27.5

Table 5 An analysis of organics in digested sludge

Feed sludge	TS mg/ℓ	VS %	Total carbohydrate, %	Total fat, %	Total protein, %
1-Primary sludge	19,100	59.1	7.6	9.8	20.0
1-Excess sludge	22,000	68.8	6.8	14.4	31.9
2-Primary sludge	32,500	51.3	7.0	8.3	21.3
2-Excess sludge	23,200	54.7	7.5	7.3	24.4
3-Mixed sludge	27,700	53.7	7.9	8.3	20.0
3-Excess sludge	27,000	64.7	6.9	12.4	28.1

These three components account for 70 to 80% of the total organics in the sludge. As regards the feed sludge, the primary sludge shows a high content of carbohydrates, whereas the excess sludge a high content of fats and proteins.

The component ratios of the organics in the feed sludge are compared with those in the digested sludge below. In the digested sludge, the content of carbohydrates falls to a specific level irrespective of the content of feed sludge. The residual quantity of fats in the digested sludge has some correlation with the content of fats in the feed sludge. The reduction of proteins is not so high as compared with the two components referred to above.

From the above, it is inferred that carbohydrates and proteins in the feed sludge govern the gas production, and that the carbohydrates are particularly influential.

Fig. 11 shows the correlation between the total carbohydrate ratio and fat ratio and the gas production per unit quantity of feed VS.

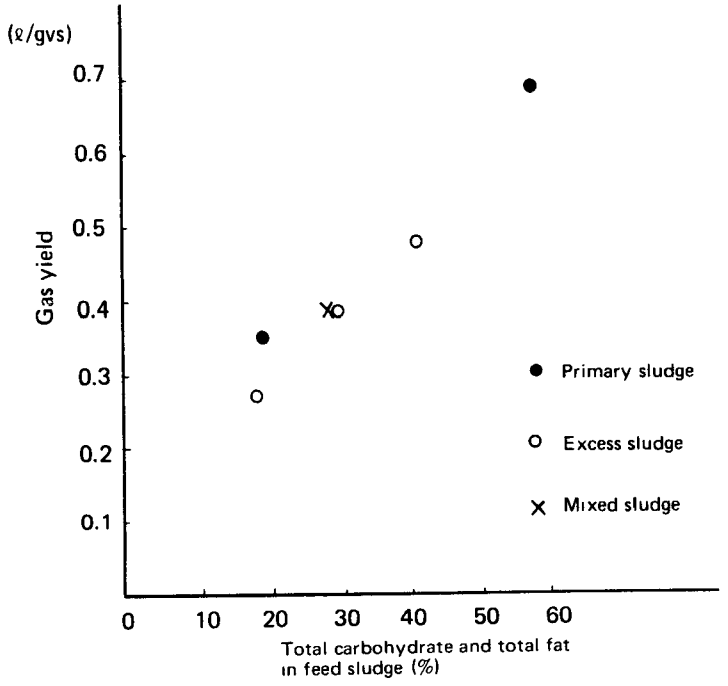


Fig. 11

As is clear from Fig. 11, the correlation is highly significant, and has little to do with the types of sludge. One can therefore conclude that the primary sludge produces more gas than the excess sludge because of the higher content of carbohydrates and fats contained in the former.

4. EFFECTS OF HIGH-CONCENTRATED FEED SLUDGE ON GAS YIELD

As discussed in Chapter 2, the concentration of feed sludge to the digestion tank is generally lower in Japan than in Europe and the U.S.A. In Japan, the capacity of the digestion tank is determined according to the following formula.

$$V = \left(\frac{Q_1 + Q_2}{2}\right) \cdot T$$

where V is the capacity of digestion tank, Q_1 is the feed sludge volume, Q_2 is the draw-off sludge volume, and T is the solids retention time (usually 30 days).

Namely, the lower the concentration of feed sludge, the greater becomes the capacity of the digestion tank required. The increase in the capacity of digestion tank leads to the increase in energy required for heating the digestion tank. In order to minimize the energy demand, it is imperative to increase the feed sludge concentration. In view of this, the wastewater treatment plants in Japan are beginning to employ the flotation thickening process or centrifugal thickening process. The adoption of mechanical thickening processes in Japan is intended not only for the reduction of heating energy requirements, but also for the improvement of thickening process and the reduction of overall cost for the processes following it. The main interests of this chapter are the effects of an increase in the feed sludge concentration on the performance of the existing digestion tanks and on the gas production.

4.1 TEST METHOD

For the test, the same arrangement (Fig. 7) as explained in Chapter 3 was used. The number of test tanks was six. They were paired off, and the digestion temperature was set at 30°C for one pair, 33°C for another pair, and 36°C for the remaining pair. The sludge adjusted at a concentration of 4% and 6% was digested for a solids retention time of 40 days, 30 days, 20 days and 15 days. For each solids retention time, the test was conducted for 4 weeks. That is to say the test was conducted for an total period of 16 weeks.

During the test period, the sludge was fed and drawn off once a day. The sludge used for the test was the mixed sludge sampled at a wastewater treatment plant, pulverized, screened, and adjusted to 4% and 6% in concentration by a centrifugal thickener. Table 6 shows an analysis of the sludge tested.

Table 6 An analysis of feed sludge

	TS mg/ℓ	VS mg/ℓ	T-C mg/ℓ	T-N mg/ℓ	T-P mg/ℓ	VS/TS %	CN ratio
4% feed sludge	41,280	32,130	17,050	1,960	372	77.8	8.7
6% feed sludge	60,840	47,440	25,310	2,860	483	78.0	8.8

For the test, each of the six test tanks was filled with 8 lit. of the same sludge, and was left to stand for 2 weeks at a specified digestion temperature. Then, the test tanks were run on a one daily feed and discharge basis with respective solids retention times.

4.2 RESULTS

Figs. 12 and 13 show the weekly gas production and weekly average methane ratio per unit quantity of feed organics over a total test period of 16 weeks.

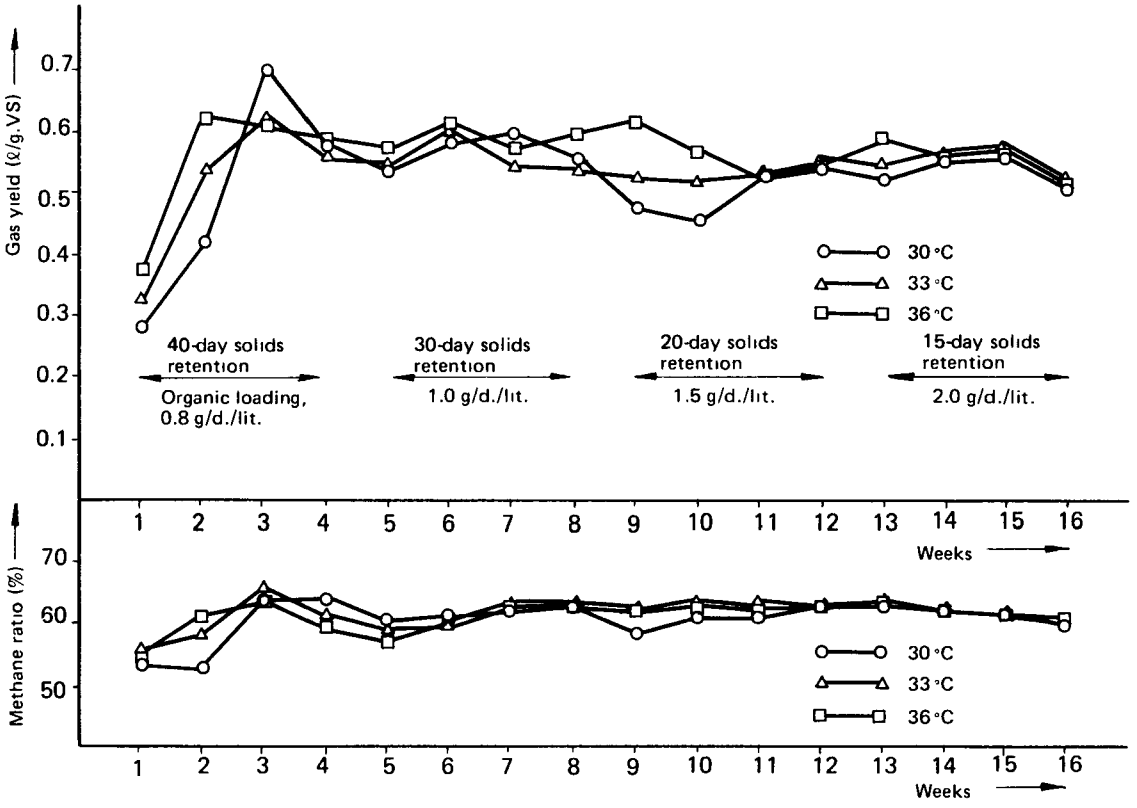


Fig. 12 4% sludge-gas yield and methane ratio

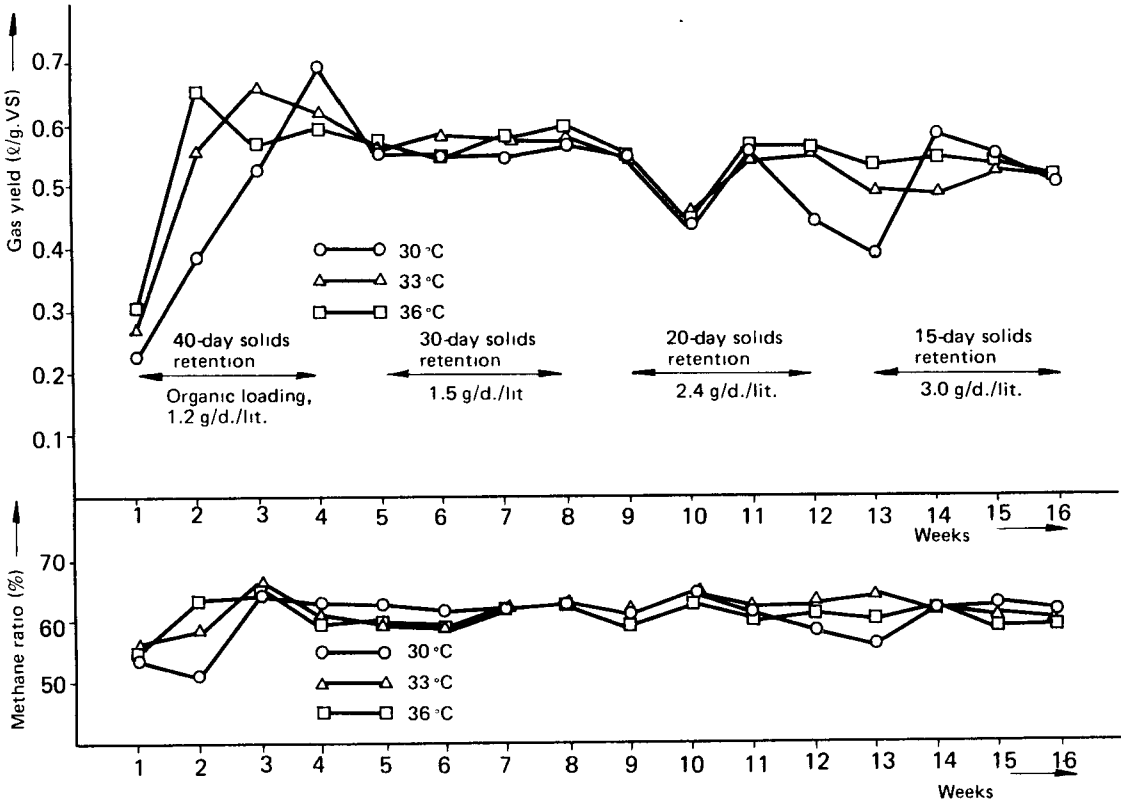


Fig. 13 6% sludge-gas yield and methane ratio

In the 40-day solids retention time in the first 4 weeks, methane bacteria in the sludge would have proliferated. The gas yield increased week after week during the period. The increase in gas production shows a significant difference depending on the digestion temperature. The time required for the gas production to level off was 2 weeks at a digestion temperature of 36°C, 3 weeks at 33°C and 4 weeks at 30°C.

It was found that the 6% sludge took a somewhat longer time for stabilized gas production than the 4% sludge.

In the 5th week, the loading was increased a step for a 30-day solids retention. The test tanks showed almost the same gas production; the gas yield was held at a level of about 0.55 lit./gVS.

Figs. 14 and 15 show the gas yield vs. solids retention time relationship for the 4% and 6% sludge.

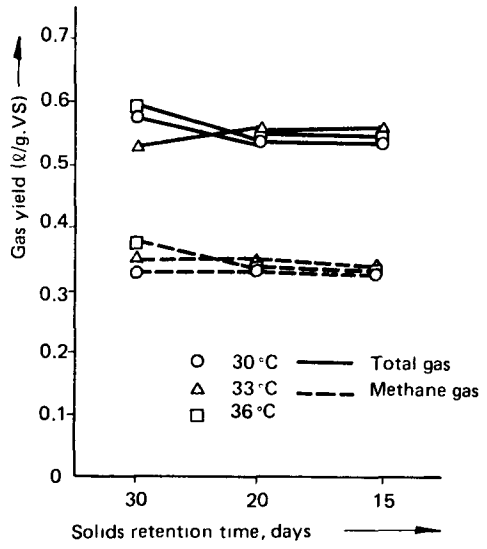


Fig. 14 4% sludge-gas yield

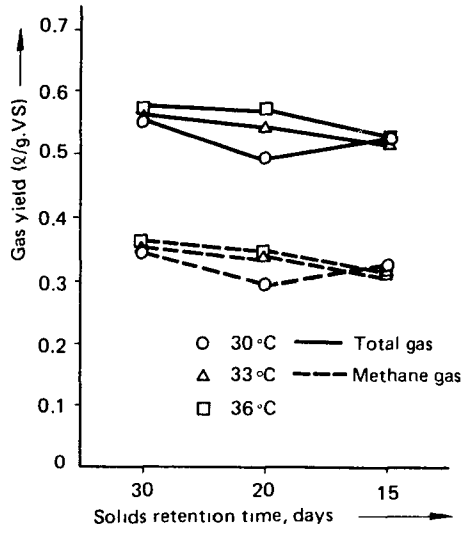


Fig. 15 6% sludge-gas yield

Plotted are the mean values of the data obtained in the latter 2 weeks in each solids retention period.

From these figures, the following are found.

(1) No noticeable change in gas yield with change in digestion temperature and solids retention time.

(2) No significant difference in gas yield between 4% and 6% sludge; the gas production remained at 0.55 lit./gVS on the average.
(3) Likewise, the methane gas yield was held almost constant at 0.33 lit./gVS.

From the above, it is concluded that the sludge with a high concentration of about 6% can be processed successfully under usual digestion conditions.

5. INCREASE OF GAS YIELD BY THE THERMAL CONDITIONING OF FEED SLUDGE

As discussed in Chapter 2, the gas yield in the wastewater treatment plants in Japan is low. For the sake of energy utility, it is desirable to develop an economical method of improving the gas yield.

A method of sludge pretreatment, called thermal conditioning, is already being studied. It is recognized that thermal conditioned sewage sludge increases gas production in the digestion tank. It has been reported that the effect of thermal conditioning on the improvement of gas yield is considerable particularly in the case of excess sludge.

This chapter is concerned with the thermal conditioning process investigated in relation to the following.

(1) Effects of thermal conditioning on the excess sludge in Japan.
(2) Optimal temperature for thermal conditioning of excess sludge.
(3) The effects of thermal conditioning on the primary sludge.
(4) Solids-liquid separation performance and dewaterability of thermally conditioned digested sludge.
(5) Quality of supernatant separated from the thermally conditioned digested sludge.

5.1 THERMAL CONDITIONING PROCESS

Usually, the thermal conditioning of sludge is carried out for 30 to 120 min. at a temperature of 180 to 210^{o}C in an anoxic, pressurized state, for the purpose of altering colloidal or gel substances in the sludge in order to produce a sludge with a high thickenability and a high dewaterability.

The operating principles of the thermal conditioning process are argued as follows. The sewage sludge forms hydrophilic colloid consisting of organics such as proteins, carbohydrates and fibrous residues and inorganics. The colloidal particles are encapsulated with hydrated layers. When the sludge is heated, water molecules are invigorated, increasing the collision and combination of sludge particles. The hydrated layers are broken up, and coagulation takes place (thermal coagulation phenomenon). The colloidal gel structure is destroyed at heat, and the water contained in it is separated (bleeding phenomenon). In addition, the organics in the sludge are subjected to hydrolysis at heat, and are dissolved (liquefaction of sludge). These phenomena are considered to take place not in stages, but concurrently during the thermal conditioning. As a result, the sewage sludge is turned into one which is high in settlability and dewaterability. The degree to which the sewage sludge is reformed is governed by the time and temperature of reaction. While the thermal conditioning improves settlability and dewaterability, it degrades the concentration, color and odor of the supernatant produced by the liquefaction of the sludge.

This chapter deals with an experiment in which the thermal conditioning process was used to precede the digestion process.

5.2 120°C THERMAL CONDITIONING OF EXCESS SLUDGE

The test was designed to investigate whether the thermal conditioning would improve the gas yield of excess sludge in Japan.

5.2.1 Test Method

For the test, two digesters with an effective capacity of 20 lit. were used. The excess sludge pretreated in an autoclave for 30 min. at 120°C was put into one digester, and the control sludge was put into another digester. These two digesters were run in parallel in order to compare respective gas outputs. The digestion was conducted over a period of 8 weeks at a temperature of 35°C with the solids retention time set at 30 days.

The sludge used was taken from the return sludge pit of a wastewater treatment plant, and was thickened to about 2.5% in TS concentration. The wastewater treatment plant from which the sludge was sampled was designated for the treating of domestic wastewater only, and the sample was considered to be the highly stabilized excess sludge.

Table 7 shows an analysis of the excess sludge used for test. Prior to the test, seed sludge was acclimated with the test sludge for 2 weeks.

Table 7 An analysis of feed sludge

Sludge	TS mg/ℓ	VTS mg/ℓ	T-C mg/ℓ	T-N mg/ℓ	T-P mg/ℓ	VTS/TS %	C/N
Control	25,000	17,800	9,750 (428)	1,560	350	71.2	6.3
120°C thermally conditioned	26,100	18,500	(2,110)	1,700	370	70.9	

( ): Supernatant

5.2.2 Results

Fig. 16 shows the weekly gas production over the 8-week period. Fig. 17 shows the weekly average methane ratio.

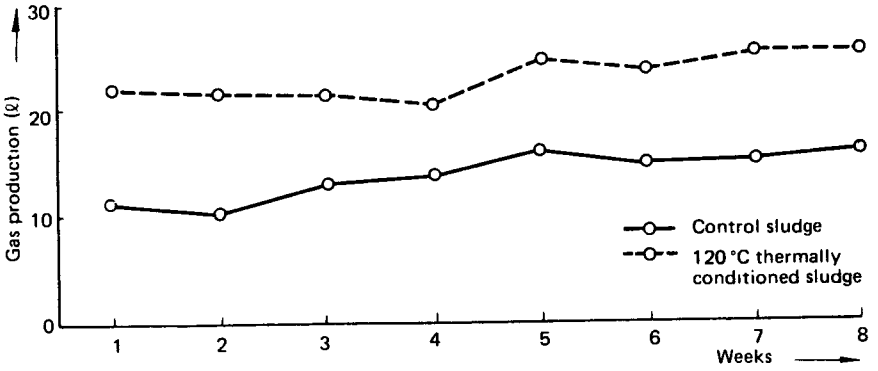


Fig. 16 Weekly gas production

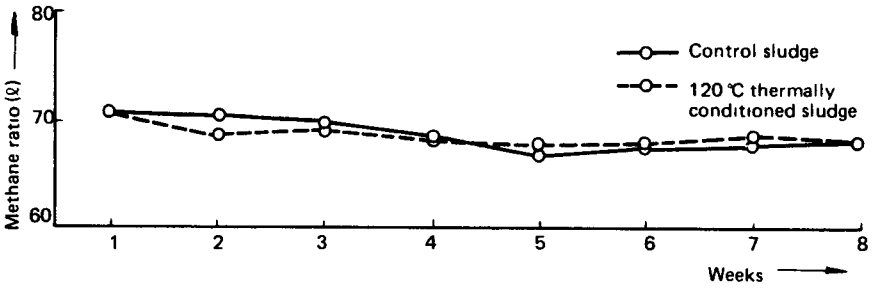


Fig. 17 Weekly average methane ratio

As shown, the gas production from the 120°C thermally conditioned sludge was higher than from the control sludge. In the latter 4 weeks, the gas production from the thermally conditioned sludge was 58% more than that from the control sludge. The gas yield was 0.19 lit./gVS for the control sludge and 0.29 lit./gVS for the thermally conditioned sludge, showing that the latter was up 53% from the former. Table 8 shows an analysis of the final draw-off digested sludge.

Table 8 An analysis of digested sludge

Sludge	TS mg/ℓ	VTS mg/ℓ	T-C mg/ℓ	T-N mg/ℓ	T-P mg/ℓ	VTS/TS %	C/N
Control	21,100	13,500	7,470 (606)	1,510	360	64.0	4.9
120°C thermally conditioned	19,600	11,700	(1,110)	1,660	380	59.7	

( ): Supernatant

The organics in the thermally conditioned sludge were reduced more than 58% than those in the control sludge, substantiating the gas production increase by the thermal conditioning.

It has thus been corroborated that the thermal conditioning process is quite effective for the improvement of gas yield from the excess sludge in Japan, and that the liquifaction of sludge by thermal conditioning has a close relationship with the increase in gas production. In the next section, discussions are made about the optimum temperature that maximizes the gas yield and the effects of thermal conditioning on the primary sludge.

5.3 BATCH DIGESTION TESTS ON SLUDGE SUBJECTED TO THERMAL CONDITIONING AT VARIOUS TEMPERATURES

The tests conducted were the same as in the foregoing, except that the thermal conditioning temperatures were changed. The tests were carried out with respect to three cases to compare the gas yields. The conditions for each case were as follows.

(1) Case I

Batch digestion test using thermally conditioned excess sludge and control sludge. Heating conditions: 70°C, 87°C and 120°C

(2) Case II

Batch digestion test using thermally conditioned excess sludge and control sludge. Heating conditions: 120°C, 150°C and 180°C

(3) Case III

Batch digestion test using thermally conditioned primary sludge and control sludge. Heating conditions: 120°C, 150°C and 200°C

5.3.1 Test Method

For all cases I, II and III, the digestion was carried out in the batch process. For each case, four digesters with an effective capacity of 10 lit. were used. The digestion temperature was set at 35°C. Prior to the test, 6 lit. of seed sludge and 4 lit. of test sludge were loaded into each digester.

5.3.2 Results

Here, the findings are discussed case by case, and then summarized.

(a) Case I

Fig. 18 shows the cumulative total of gas production with time over a test period of 56 days.

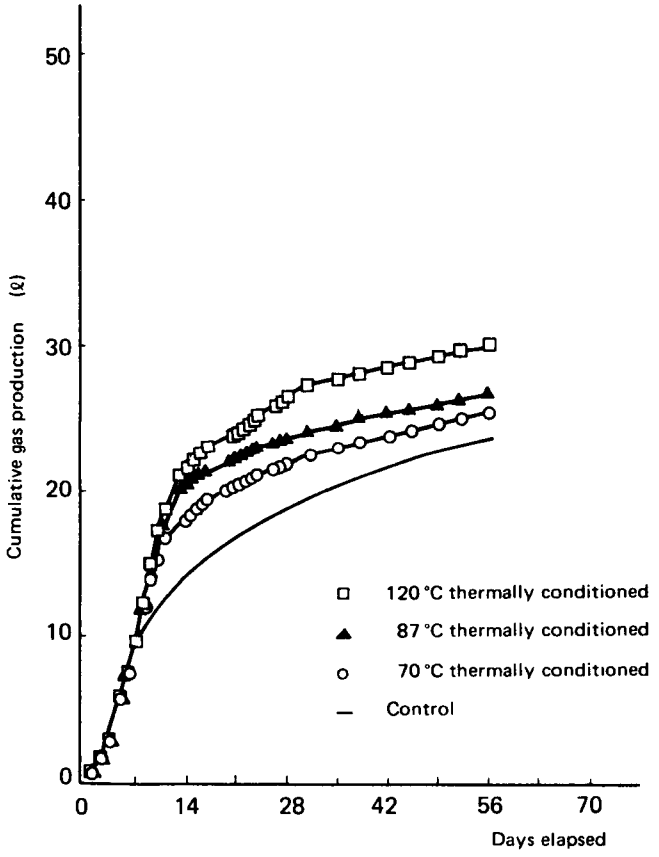


Fig. 18 Case I: Cumulative gas production

The total volume of digestion gas produced from the control sludge was about 24 lit. To produce the same amount of digestion gas, the 70°C thermally conditioned sludge took 44 days, the 87°C sludge took 30 days, and the 120°C sludge took 20 days. It is found that for an ordinary solids retention time of about 30 days, the thermal conditioning has a significant bearing on the improvement of gas yield. For up to 120°C, the higher the thermal conditioning temperature is, the greater the amount of gas production results.

Table 9 shows an analysis of the test sludge, and Table 10 an analysis of the digested sludge.

Table 9 An analysis of feed excess sludge (Case I)

Sludge	TS mg/ℓ	VTS mg/ℓ	T-C mg/ℓ	T-N mg/ℓ	T-P mg/ℓ	VTS/TS %	C/N
Control	26,500	20,400	11,100 (420)	1,860	380	77.0	6.0
70°C thermally conditioned	27,200	20,800	(2,110)	2,250	420	76.5	-
87°C thermally conditioned	25,500	19,500	(2,500)	1,770	390	76.5	-
120°C thermally conditioned	26,900	20,600	(2,900)	1,790	390	76.6	-

( ): Supernatant

Table 10 An analysis of digested sludge (Case I)

Sludge	TS mg/ℓ	VTS mg/ℓ	T-C mg/ℓ	T-N mg/ℓ	T-P mg/ℓ	VTS/TS %	C/N
Control	18,300	11,800	7,560 (1,440)	1,860	361	64.5	4.1
70°C thermally conditioned	18,200	11,700	(1,450)	1,850	368	64.3	-
87°C thermally conditioned	17,600	11,100	(1,520)	1,810	361	63.1	-
120°C thermally conditioned	17,200	10,800	(1,540)	1,820	365	62.8	-

( ): Supernatant

The control sludge and thermally conditioned sludges were compared in terms of the amount of residual organics in the digested sludge. So far as the temperatures used for thermal conditioning are concerned, the higher the temperature, the smaller the residual amount of organics, explaining why the gas production was increased at high thermal conditioning temperatures.

(b) Case II

Fig. 19 shows the cumulative total volume of the gas produced with time.

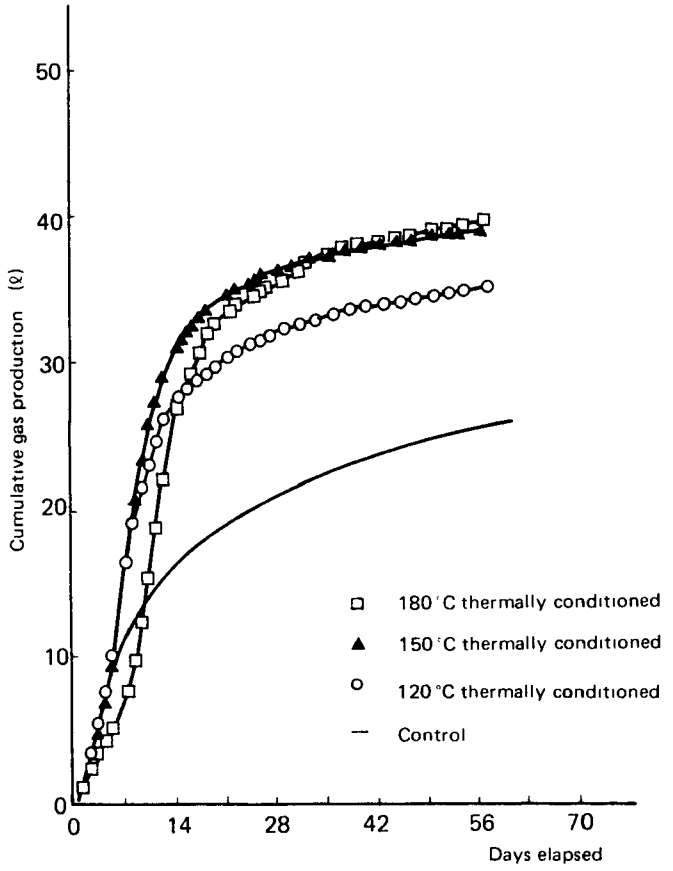


Fig. 19 Case II: Cumulative gas production

The total volume of gas produced over a solids retention time of 56 days was up 36% for the 120°C thermally conditioned sludge, up 52% for the 150°C sludge and up 54% for the 180°C sludge as against 26 lit. marked by the control sludge. In the 180°C sludge, gas production rate was retarded for the first 10 days. With regard to total gas volume it is inferred that the best thermal conditioning temperature is somewhere between 150°C and 180°C.

Table 11 shows an analysis of the test sludge, and Table 12 an analysis of the digested sludge.

Table 11 An analysis of feed excess sludge (Case II)

Sludge	TS mg/ℓ	VTS mg/ℓ	T-C mg/ℓ	T-N mg/ℓ	T-P mg/ℓ	VTS/TS %	C/N
Control	24,200	19,100	10,300 (380)	1,930	434	78.9	5.3
120°C thermally conditioned	25,000	19,900	(2,660)	2,060	436	79.6	-
150°C thermally conditioned	24,400	19,100	(4,510)	1,970	429	79.3	-
180°C thermally conditioned	24,500	18,800	(5,870)	2,040	420	76.7	-

( ): Supernatant

Table 12 An analysis of digested sludge (Case II)

Sludge	TS mg/ℓ	VTS mg/ℓ	T-C mg/ℓ	T-N mg/ℓ	T-P mg/ℓ	VTS/TS %	C/N
Control	17,500	11,700	6,930 (1,230)	1,920	389	66.9	3.6
120°C thermally conditioned	15,800	9,940	(1,500)	1,920	408	62.9	-
150°C thermally conditioned	15,700	9,900	(1,550)	1,920	401	63.1	-
180°C thermally conditioned	15,500	9,640	(1,830)	1,930	401	62.2	-

( ): Supernatant

Referring to Table 11 and Table 12, it is found from the change in total carbon in the supernatant that the liquefaction of thermally conditioned sludge progressed, and that the total carbon in the supernatant of the thermally conditioned sludge reduced greatly compared with the control sludge. It is considered that the high BOD component in the supernatant resulting from thermal conditioning got reduced sufficiently by the anaerobic digestion process.

Table 13 shows an analysis of organics in the supernatant of the test sludge.

Table 13 An analysis of organics in supernatant (mg/lit.)
Case II

Sludge	Carbohydrate	Fat	Protein
Control	57.9	32.1	240
120°C thermally conditioned	231	179	2,670
150°C thermally conditioned	447	186	5,080
180°C thermally conditioned	546	273	6,630

Table 14 An analysis of organics in digested sludge (mg/lit.)
Case II

Sludge	Carbohydrate	Fat	Protein
Control	992	1,060	4,660
120°C thermally conditioned	787	1,400	3,590
150°C thermally conditioned	741	1,030	3,360
180°C thermally conditioned	650	1,200	3,470

The higher the thermal conditioning temperature the more the liquefaction of organics in the sludge was promoted, and this was particularly the case with proteins. Table 14 shows an analysis of the organics in the digested sludge. A marked difference between the control sludge and thermally conditioned sludge suggests that the proteins in the thermally conditioned sludge may have been subjected to violent gasification.

(c) Case III

Fig. 20 is a cumulative graph of the volume of gas generated from the primary sludge.

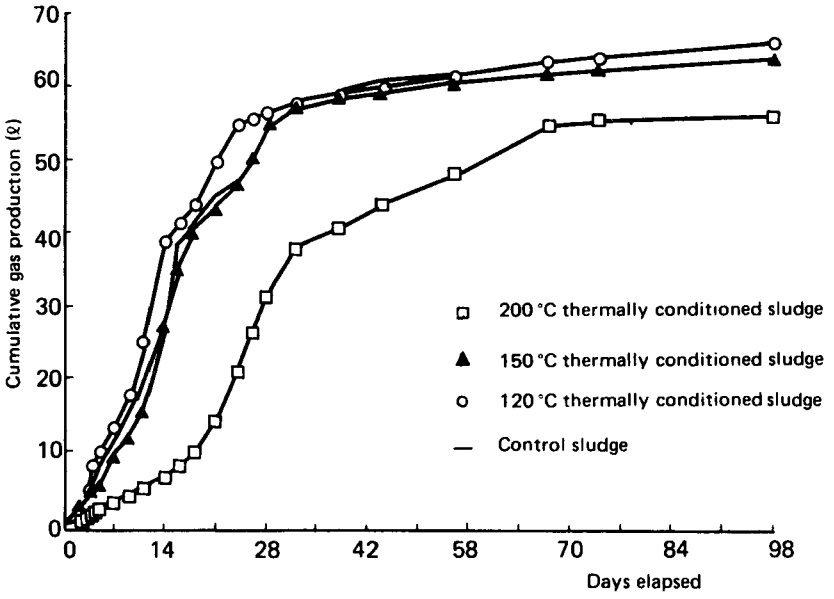


Fig. 20 Case III: Cumulative gas production

The 120°C sludge made a quick start in giving off gas, while the 200°C sludge was far slower. The cumulative total of gas volume after 12 weeks of operation was 65.0 lit. for the control sludge, 66.1 lit. for the 120°C sludge, 64.0 lit. for the 150°C and 56.2 lit. for the 200°C sludge. The following conclusions are reached accordingly.

(1) The gas production is impeded by heating up to 200°C.

(2) The thermal conditioning has little or no effect on the improvement of gas yield for the primary sludge.

Tables 15 and 16 shows the concentrations of the organics in the test sludge and digested sludge.

Table 15 An analysis of organics in feed sludge (mg/lit.)
Case III

Sludge	Carbohydrate	Fat	Protein
Control	6,159 (170)	3,812 (250)	9,629 (1,064)
120°C thermally conditioned	7,051 (510)	3,896 (130)	9,142 (1,410)
150°C thermally conditioned	6,324 (1,128)	4,248 (314)	8,702 (4,849)
200°C thermally conditioned	4,877 (354)	5,598 (1,462)	8,218 (6,676)

( ): Supernatant

Table 16 An analysis of organics in digested sludge (mg/lit.)
CASE III

Sludge	Carbohydrate	Fat	Protein
Control	1,121 (217)	892 (336)	4,169 (1,242)
120°C thermally conditioned	1,060 (221)	732 (322)	4,528 (1,159)
150°C thermally conditioned	995 (202)	876 (326)	4,493 (1,377)
200°C thermally conditioned	962 (237)	1,516 (796)	4,216 (1,836)

( ): Supernatant

(d) A summary of Cases I, II and III

The finding of Cases I, II and III are summarized as follows.

(1) Although the excess sludge increases gas production by thermal conditioning, the primary sludge does not.
(2) The thermal conditioning temperature at which the gas production from excess sludge is maximized lies in the range of 150°C to 180°C.
(3) The high BOD load in the supernatant due to thermal conditioning poses no problem because it is disposed of during the digestion process.
(4) The total carbon liquefaction ratio (decomposition ratio) is shown in Fig. 21 with respect to excess sludge and primary sludge. As shown, the primary sludge shows a much smaller ratio.

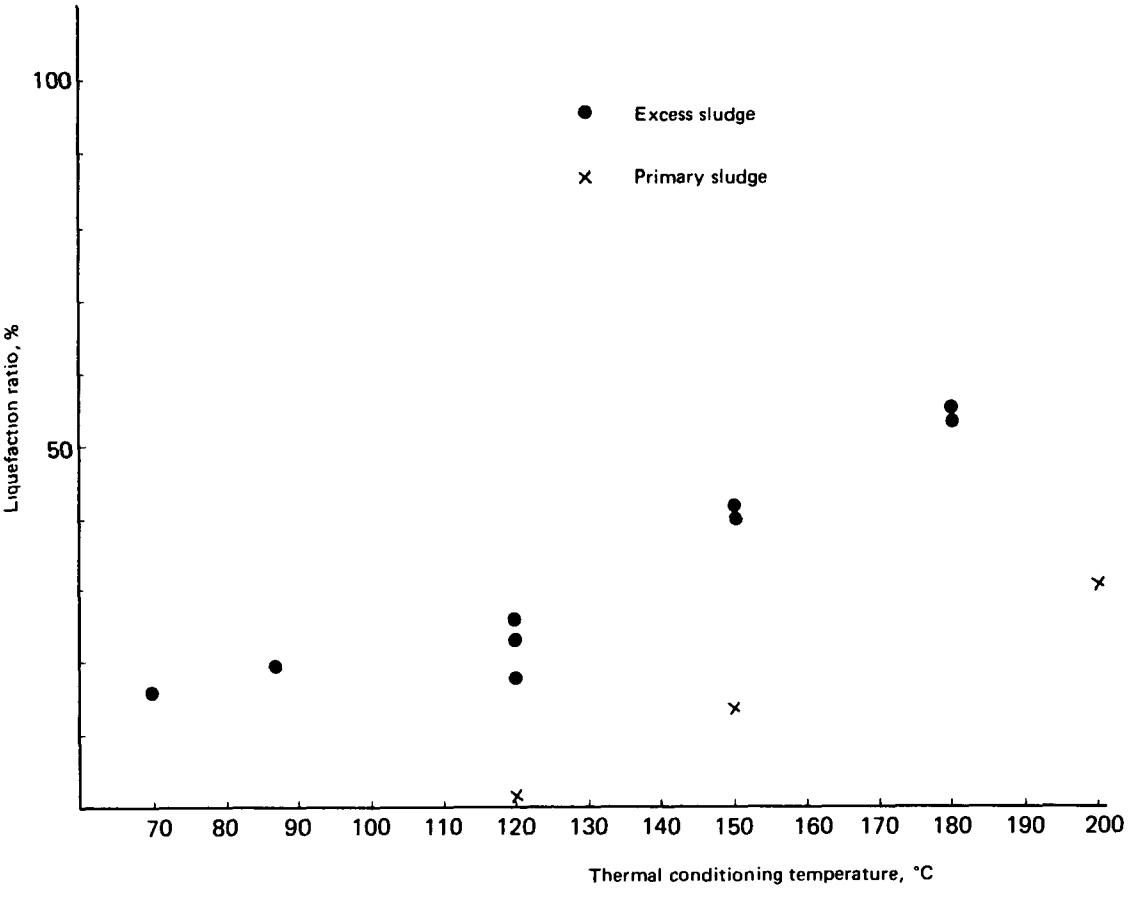


Fig. 21 Sludge liquefaction and thermal conditioning temperature

(5) The thermal conditioning is effective for the excess sludge, but not for the primary sludge. This is due primarily to the ratio of carbohydrates + fats to the total organics. As shown in Fig. 11 in Chapter 2, carbohydrates and fats are easy to gasify, and the primary sludge containing large quantities of these substances generates large volumes of digested gas. The thermal conditioning of the primary sludge to produce gasifiable and dissolvable proteins has an immaterial effect on the increase of gas production. On the contrary, the excess sludge usually has a small ratio of carbohydrates + fats to the total organics, and thus the effects of the dissolved proteins are relatively significant to increase in gas production through thermal conditioning. In addition to this, noteworthy is the fact cited under item (4) above.

5.4 CONTINUOUS DIGESTION TEST OF SLUDGE WITH THERMAL CONDITIONING TEMPERATURE AS A PARAMETER

At the beginning of Chapter 5, the study subjects (1) through (5) were listed. Items (1) through (3) and (5) have already been discussed, and here we will discuss item (4) concerning the solids-liquid separation and dewaterability of thermally conditioned sludge.

5.4.1 Test Method

The test arrangement used was almost the same as referred to in the foregoing. The test sludge used was excess sludge. The excess sludge sample was thickened by a centrifuge to a concentration of 2.5%.

The test was conducted concurrently for the control sludge, 120°C sludge, 150°C sludge and 180°C sludge.

The digestion was carried out at a temperature of 35°C for 8 weeks for the 20-day solids retention and for 4 weeks for the 10-day solids retention.

5.4.2 Results

Table 17 compares the test sludges and control sludge with respect to gas production and composition.

Table 17 Gas yield and gas analysis

Feed sludge	Sludge retention time	Gas analysis			Gas yield (ℓ/ℓ• feed sludge)	Methane yield (ℓ/ℓ• feed sludge)	Gas yield (ℓ/g.VS)	Methane yield (ℓ/g.VS)
		CH_4	CO_2	Others				
CONTROL	20-day	71.3	27.4	1.3	2.87	2.05	0.16	0.11
	10-day	70.3	27.6	2.1	1.68	1.18	0.10	0.07
120°C	20-day	70.3	28.8	1.7	5.58	3.95	0.30	0.21
	10-day	70.8	28.9	0.5	1.81	3.25	0.28	0.20
150°C	20-day	68.6	30.6	0.8	8.58	4.51	0.36	0.25
	10-day	68.5	31.2	0.3	5.21	3.57	0.32	0.22
180°C	20-day	68.8	30.1	1.3	6.85	4.70	0.38	0.28
	10-day	68.3	31.1	0.3	5.25	3.59	0.34	0.23

Fig. 22 shows the time-dependent changes of gas production. All these confirm the effectiveness of thermal conditioning.

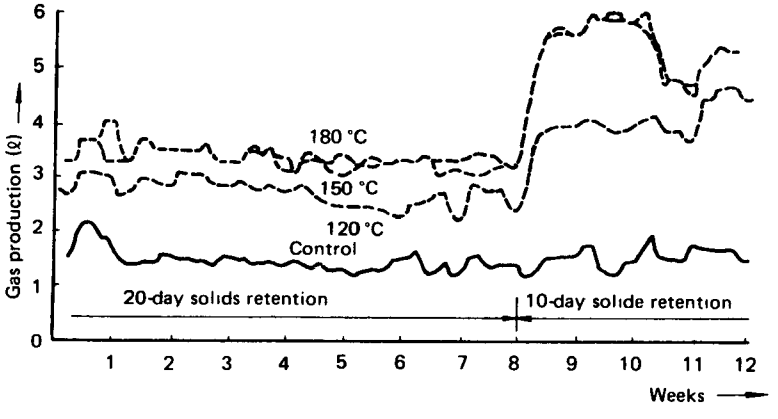
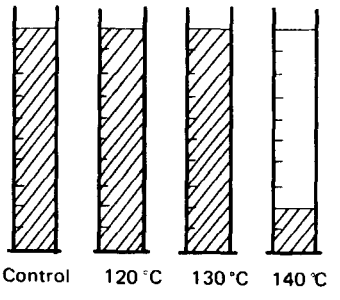


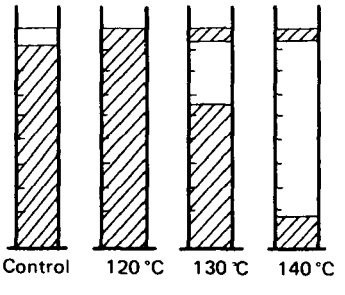
Fig. 22 Gas production change

For both the control and thermally conditioned sludges, the gas production per lit. of feed sludge or gram of feed VS is slightly larger by the 20-day solids retention than by the 10-day solids retention.

Fig. 23 shows the settlability of sludge measured by making use of a 1 lit. measuring cylinder.



Thermally conditioned feed sludge (24 hrs. later)



20-day solids retention digested sludge (24 hrs. later)

Fig. 23 Settlability of thermally conditioned sludge and digested sludge

The samples of the digested sludge were taken in the final week of the 20-day solids retention digestion test. The control sludge, the 120°C thermally conditioned sludge and their digested sludge hardly settled. The settlability of the 150°C thermally conditioned sludge varied over a wide range from a supernatantless state to a heavy thickening of about 50% in volume. On the other hand, the digested sludge always separated into sediment and supernatant, producing a supernatant of 30 to 50 vol.% in 24 hrs.

The 180°C digested sludge was thickened to 40% in an hour, and was reduced to about 20% in volume in 24 hrs. along a slow-decreasing curve. To sum up, it can be said that the settlability and thickenability of the thermally conditioned sludge are more or less inherited by the anaerobically digested sludge.

The 150°C and 180°C thermally conditioned sludge and their digested sludge were compared by a filter leaf test. For the test, the sludge was thickened for 8 hrs. and cleared of supernatant. The digested sludge used was sampled from what was subjected to a 10-day solids retention digestion. At first, we presumed that the comparison would be made without chemical conditioning. But even the 180°C thermally conditioned sludge showed a extremely poor dewaterability, and necessitated the injection of ferric chloride by 10% and slaked lime by 25% per unit weight of solids in order to attain a filtration velocity of about 10 kg/m^2h. On the other hand, the 150°C sludge called for twice as much chemical injection to attain the same filtration velocity. The thermally conditioned sludge and its digested sludge showed no substantial difference in dewaterability, though the digested sludge was a little inferior. On the other hand, the comparison between the thermally conditioned digested sludge and the control digested sludge showed that the former gave a better dewaterability than the latter.

All these are summarized as follows.

- Digestion of thermally conditioned sludge does not improve dewaterability.
- Thermally conditioned sludge is better in dewaterability than the sludge without thermal conditioning.

Regarding the odor of digested sludge, the measurements are shown in Table 18.

Table 18 Sludge odors

Sludge	Odor concentration	Hydrogen sulfide, ppm	Methyl mercaptan, ppm	Acet-aldehyde, ppm	Remarks
Feed, control	4,100	0.58	0.11	0.14	Odor intensity 5
Feed, thermally conditioned at 120°C	73,000	0.25	2.91	7.83	Odor intensity 5
Feed, thermally conditioned at 150°C	230,000	0.20	2.84	2.24	Odor intensity 5
Feed, thermally conditioned at 180°C	1,300,000	0.02	0.09	12.4	Odor intensity 5
Digested, control	1,700	0.33	0.25	0.12	Odor intensity 3-4
Digested, thermally conditioned at 120°C	550	ND	0.04	0.10	Odor intensity 4
Digested, thermally conditioned at 150°C	980	ND	0.06	0.08	Odor intensity 4-5
Digested, thermally conditioned at 180°C	980	ND	0.04	0.13	Odor intensity 4

Nitrogen gas was blown into 200 ml of sample sludge at a rate of 10 lit./hr., and the odors of entrained gases were measured. The measurements show that the odors of the sludge can be reduced drastically by digestion. In support of this, the irritating odor peculiar to aldehyde resulting from thermal conditioning was lowered in the digested sludge.

The colorimetric results of supernatant are shown in Table 19.

Table 19 Color of supernatant

Origin of supernatant	Chromaticity
Feed sludge, control	16
Feed sludge, 120°C thermally conditioned	130
Feed sludge, 150°C thermally conditioned	200
Feed sludge, 180°C thermally conditioned	600
Digested sludge, control	50
Digested sludge, 120°C thermally conditioned	130
Digested sludge, 150°C thermally conditioned	260
Digested sludge, 180°C thermally conditioned	700

It is found that the colors produced by thermal conditioning cannot be removed by anaerobic digestion.

6. SUMMARY

The discussions in the foregoing chapters are summarized as follows.

(1) In Japan, as of 1980, there were some 480 wastewater treatment plants of which about 180 were equipped with a digestion process. Of the 180 wastewater treatment plants, 156 were operated on a system consisting of thickening, anaerobic digestion, dewatering and disposal processes, and 17 were operated on a system consisting of thickening, anaerobic digestion, incineration and disposal processes. The wastewater treatment plants operating on the former system accounted for 26% of the national total of sludge generated, and those on the latter system accounted for 24%, on the basis of solids in thickened sludge.

(2) In Japan, the digestion tank typically is operated on the following conditions.

- Solids retention time, 30 days
- Mesophitic digestion (30°C to 37°C)
- Heating by steam injection

(3) About 50% of the wastewater treatment plants with digestion tank show a gas yield not exceeding to 5 or 6 ℓ/ℓ•feed sludge.

(4) Those wastewater treatment plants where more than 50% of digestion tank influent was returned to the water treatment process in the form of supernatant accounted for about two thirds of the total of wastewater treatment plants with digestion process.

(5) If the organics content is fixed, the primary sludge gives off more digestion gas per unit quantity of feed VS than the excess sludge.

(6) The probable cause of the phenomenon stated in (5) above is that the primary sludge contains more carbohydrates and fats in the total organics than the excess sludge.

(7) The ratio of methane to the total volume of digestion gas is about 60% on the average.

(8) Highly thickened sludge with a concentration of about 6% can be digested satisfactorily under usual digestion conditions.

(9) The effects of thermal conditioning are significant on the excess sludge, but are little on the primary sludge.

(10) By thermal conditioning, the excess sludge has its organics more liquefied than the primary sludge.

(11) The carbohydrates and fats are considered susceptible to gasification even under usual digestion conditions.

(12) Thermal conditioning works very effectively on the excess sludge because it steps up the gasification potential of the excess sludge from an essentially low to quite a high level.

(13) The optimum thermal conditioning temperature for excess sludge is within the range of 150°C to 180°C. The high level of dissolve BOD which is unavoidably developed by thermal conditioning can be reduced to a satisfactory low level in the digestion process.

(14) The high thickenability imparted by thermal conditioning to the sludge is retained even after digestion.

(15) The dewaterability of sludge digested after thermal conditioning is a little higher than that of control sludge which is not subjected to thermal conditioning.

(16) Odors of the thermally conditioned sludge are reduced by digestion.

(17) The colors caused by thermal conditioning cannot be removed by digestion.

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PILOT PLANT STUDY ON TREATMENT OF SUPERNATANT FROM THERMAL SLUDGE TREATMENT PROCESS

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PILOT PLANT STUDY ON TREATMENT OF SUPERNATANT FROM THERMAL SLUDGE TREATMENT PROCESS

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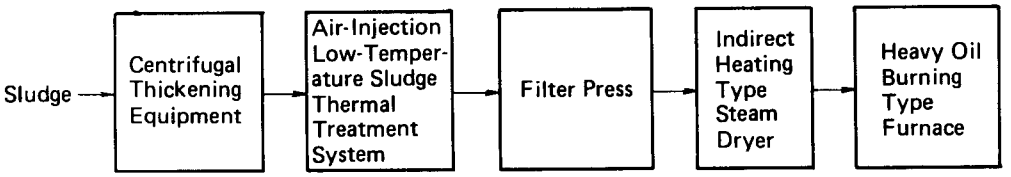
1. INTRODUCTION

There has been an annual increase in volume of sludge discharged from a sewage treatment process with the diffusion of a sewage system. The present situation is such that major cities, in particular, are burdened with increased cost required for sludge treatment and securing of the site for ultimate disposal.

Recently, a high rise in energy price and a sense of insecurity for energy supply have required review of sludge treatment. Thus, the selection of a proper treatment process allowing reduction of volume for ultimate disposal and biochemical stabilization of sludge economically has become an important theme.

Kobe city has conducted investigations to study a new method for sludge treatment using a small-scale sludge treatment plant combining the following processes, established in the West Sewage Treatment Plant in fiscal 1979. This method consists of mechanical concentration of sludge, followed by air-injection low-temperature sludge thermal treatment to improve the dewaterability of raw sludge, which is subjected to pressure filtration without dosing to produce dehydrated cake, which is then converted, using a steam dryer, into low-moisture dryed cake to be fed to a heavy oil burning type furnace so that organic matter in the cake is burned and, at the same time, ash or inorganic matter is melted to be slagged.



Kobe city entrusted the technological assessment on the sludge treatment performed using this small-scale plant to Japan Sewage Works Agency, which investigated this sludge treatment system in relation to treatment performance, operation ease, and economical advantage, and reported the results. This investigation, however, left behind the subject of treatment of supernate produced from thermal sludge treatment process of this system. Supernate from the thermal sludge treatment is brown in color with high concentrations of organic matter and nutrient salt.

Therefore, Kobe city constructed a pilot plant for treatment of supernatant liquid from thermal sludge treatment in the West Sewage Treatment Plant in fiscal 1981 to undertake investigations on system for treatment of supernate and system for maintenance and management, and entrusted analysis of the results and report preparation to Japan Sewage Works Agency. Very great pollutional potential of supernate from the thermal sludge treatment requires studies on treatment performance and economical advantage to be completely performed with consideration given to the effluent standard for public water area.

During the process of operation of, and investigation and experiment on this pilot plant, the staff from Kobe municipal Sewage Works Bureau and Japan Sewage Works Agency cooperated with each other through full consultation and much discussion so that the investigation and experiment would be successful with significant results obtained.

2. GENERAL DESCRIPTION

2.1 Outline of Small-Scale Pilot Plant for Thermal Treatment and Melting System Sludge Treatment

A plant for thermal treatment and melting treatment, built in the Kobe municipal West Sewage Treatment Plant, is a small-scale one having a capacity of treating approximately $6m^3$/day of sludge with a solid content of 2%. This plant is composed of a centrifugally sludge thickening equipment, air-injection low-temperature thermal treatment equipment, dewatering equipment, steam drying equipment, and melting equipment. Its flow sheet is given in Fig.-1.

Photo, Thermal Treatment Equipment

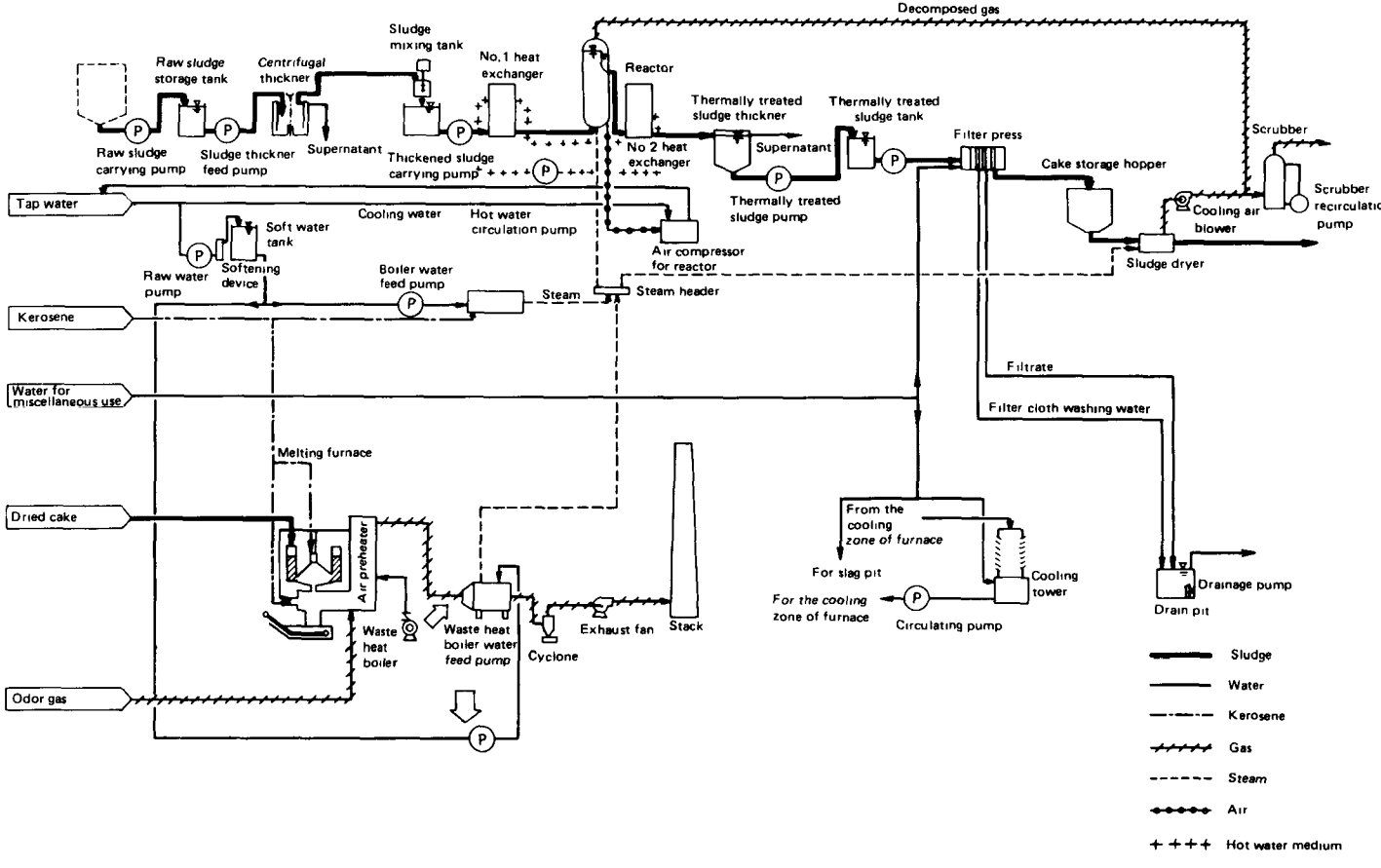


Figure-1 Flow Sheet of Small-Scale Pilot Plant for Thermal Treatment and Melting System Sludge Treatment

Photo; Centrifugally Sludge Thickening Equipment

In this system, mixed raw sludge is first concentrated to about 4% by means of a centrifugal thickening equipment to be fed to a thermal treatment equipment, in which the resultant concentrated sludge is subjected to low-temperature thermal treatment for 30–90 minutes at the reaction temperature of 155–165°C, pressure of 7–9kg/cm^2, and air injection of 10–15Nm^3/m^3 of sludge. The thermal-treated sludge is concentrated to about 8% in a gravitational thickener before being dewatered by a filter press at the filtration rate of about 6kg–DS/m^2·hr to obtain dehydrated cake with the water content of 40–50%. This cake is further reduced in water content to the extent of 20% by a steam dryer, and then fed to a heavy oil burning type melting furnace, in which combustibles are burned with ash discharged as slag. Thermally decomposed gas, discharged from a low-temperature thermal treatment reactor and then cooled to about 85°C using a cooling apparatus, and exhaust gas (about 85°C) from the steam dryer are cooled in a scrubber before being fed to a secondary combustion room of the melting furnace. Exhaust gas from the secondary combustion room is allowed to pass through a waste heat boiler, and scavenged before being released into the atmosphere. In addition, malodorous gases present in a sludge treatment building is deodorized by using sodium hypochlorite, sodium thiosulphate, etc. to be discharged into the air.

2.2 Specifications of Each Equipment

2.2.1 Centrifugally Sludge Thickening Equipment

Basket Type, Interior Volume: 1.1m^3, Centrifugal Effect: 200–500G, Treatment Capacity: 10m^3/H.

This equipment, developed to concentrate sludge difficultly subjected to gravity concentration, feeds sludge to a revolving cylindrical vessel, and, through skimming, withdraws concentrated sludge collected in the circumferential area by giving centrifugal force horizontally with feed and withdrawal of sludge performed at intermittent operation. The time required for one cycle is about 1–2 hours when the raw sludge SS is 0.3–0.5%, and about 20–30 minites when it exceeds 1%.

2.2.2 Air-Injection Low-Temperature Thermal Treatment Equipment

Reactor Type: Perforated-Plate Gas-Liquid Contactor
Reactor Volume: 2.4m^3
Capacity: Reaction Temperature; 165°C, Reaction Pressure; 7.5kg/cm^2,
Air Injection; 15m^3/m^3 of sludge, Treatment Capacity; 2.3m^3/H

The air-injection low-temperature thermal treatment equipment is mainly composed of a primary heat exchanger, reactor, secondary exchanger, thermally treated sludge thickening tank.

Sludge, concentrated to 4–5% in a sludge thickening tank, is heated to about 120°C in a primary heat exchanger, and is then pressure-fed to a reactor.

The heat exchanger, which is of double cylinder type, supplies heat water medium, subjected to heat recovery in the secondary heat exchanger, to the external cylinder to heat untreated sludge passing through the internal cylinder. The reactor interior is heated, using steam generated by a boiler to reach temperature (165°C), at which sludge is thermally treated. In addition, the inside is kept in oxidizing atmosphere by injecting air from the lower part of the reactor to improve the dewaterability of sludge, and, at the same time, allow partial oxidative decomposition of odorous components generated from sludge. Thermally treated sludge, which has completed reaction, is subjected to heat exchange, and cooled to be fed from a sludge drain valve to the thermally treated sludge thickening tank, in which the sludge is concentrated for about 8 hours, and the supernate is discharged to the outside of the system as an supernatant liquid from the thermal sludge treatment.

2.2.3 Sludge Dewatering Equipment

Type: Horizontal Plate Filter Press
No. of Filtration Rooms: 10
Filtration Area: 25m^2
Capacity: Filtration Rate; 6kg DS/m^2·H(cake moisture; 50%)

The dewatering equipment, a filter cloth travelling type filter press, allows thermally-treated and concentrated sludge with moisture of 92% to be converted into cake with moisture of below 50%. Sludge is fed by means of a feed pump at filtration pressure of 3–5kg/cm^2, and the resultant cake is then squeezed by expanding a diaphragm using water with a high pressure of about 15kg/cm^2 for dewatering. The cycle time is about 20 minutes.

2.2.4 Sludge Steam-Dryer

Type: Dual Paddle-Screw Type Steam Dryer
Capacity: Treatment Capacity; 70kgDS/H(cake moistre; 50% to 20%)
Heat Transfer Area: 8.47m^2

The dryer, a indirect heating type steam dryer, feeds pressure steam, obtained by heat recovery from a melting furnace, to the inside of paddles, and, using this as a heat source, dries cake in contact with the paddle metal face.

2.2.5 Sludge Melting Furnace

Type: Vertical Double Cylinder and Heavy Oil Burning type Melting Furnace

Capacity: Cake Treating Capacity; 90kg/H (moisture; 20%)

This heavy oil burning type sludge melting furnace melts dried cake for its stabilization and reduction, and encloses hazardous heavy metals, contained in sludge, in the crystal structure of an inorganic component to make such a safety slag as allowing no elution.

The furnace consists of furnace outer cylinder and inner cylinder, which are concentric with each other. A main combustion burner has been installed at the upper center of the inner cylinder, while, at the bottom center of the outer cylinder, a small-diameter outlet has been mounted, below which a secondary combustion room with auxiliary burners on the side is located, which is furnished with a slag watertank in its bottom. Dried cake with the moisture content of 20% is charged from a feed port into a ring-shape space in the furnace. The furnace outer cylinder revolves at a slow speed so that, as the charge of the dehydrated cake into the ring-shape space advances, the cake is fed from the lower end of the outer cylinder toward the outlet, forming a reverse conical side wall. The space formed by the side wall of dehydrated cake and the ceiling of the inner cylinder, which is referred to as a flame chamber, is maintained at temperatures as high as 1,300–1,400°C. Combustibles are burned into waste gas, while ash is discharged as melt slag into the outside of the furnace. (Fig.-2)

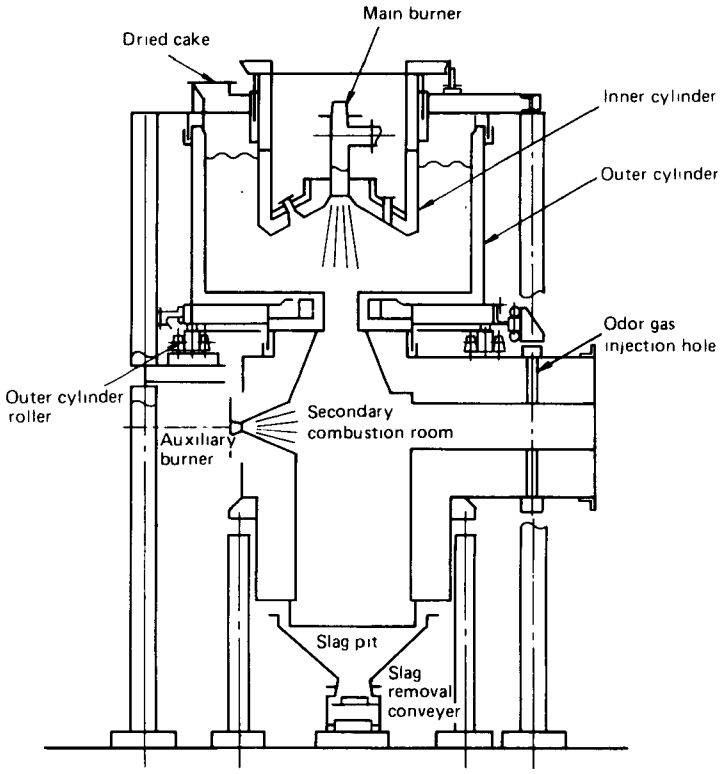


Figure-2 Cross Section of the Melting Furnace

2.3 Properties of Supernate from Thermal Sludge Treatment

2.3.1 Changes in Properties of Sludge by Thermal Treatment

Table-1 gives the properties of sludge fed to the thermal treatment and melting plant. According to the table, the raw sludge fed to the centrifugal thickener is sludge with a high content of organic components, showing 1.6–2.3% for TS concentration, 1.5–2.0% for SS concentration, and 80–90% for the content of organic matter. Table-2 indicates the properties of concentrated sludge obtained by subjecting the raw sludge to centrifugal concentration, properties of thermally treated sludge produced by the subsequent air-injection low-temperature thermal treatment of the concentrated sludge, and properties of supernatent liquid resulting from gravity concentration of the thermally treated sludge, followed by solid-liquid separation.

Table-1 Characteristics of Mixed Raw Sludge in West Wastewater Treatment Plant

Items	The first research (summer) Aug. 22–31, 1979				The 3rd research (winter) Jan. 17–Feb. 9, 1980			
	Sample size n	Max.	Min.	Average x	Sample size n	Max.	Min.	Average x
Temperature °C	37	325	290	305	38	135	80	11.6
pH	〃	58	51	–	〃	665	600	–
TSmg/ℓ	〃	20,500	14,000	16,300	〃	41,700	16,800	22,800
SSmg/ℓ	〃	19,200	13,000	15,100	〃	37,000	13,700	20,700
VSS%	〃	872	801	833	〃	924	809	881

Table-2 Results of Thermal Sludge Treatment Experiment

Aug. 22–31, 1979

Items		Concentrated sludge		Thermally treated sludge		Supernate from thermal treatment process	
		Sample size	Average	Sample size	Average	Sample size	Average
pH	–	10	–	10	–	10	–
TS	mg/ℓ	〃	50,500	〃	39,200	〃	12,700
VTS	%	〃	776	〃	799	–	–
SS	mg/ℓ	〃	43,900	〃	28,000	〃	949
VSS	%	〃	815	〃	78.8	–	–
BOD	mg/ℓ	4	20,900	4	15,700	〃	7,690
S-BOD	mg/ℓ	〃	5,260	〃	7,720	–	–
CODcr	mg/ℓ	〃	70,200	〃	67,300	〃	15,900
S-CODcr	mg/ℓ	〃	6,150	〃	15,200	–	–
CODMn	mg/ℓ	〃	11,400	〃	10,200	〃	4,730
S-CODMn	mg/ℓ	〃	782	〃	4,200	–	–
TOC	mg/ℓ	〃	18,900	〃	25,300	〃	6,290
S-TOC	mg/ℓ	〃	2,390	〃	6,290	–	–

S; soluble, BOD: 5-days BOD

Table-2 (Continued)

Jan. 17 – Feb. 9, 1980

Items		Concentrated sludge		Thermally treated sludge		Supernate from thermal treatment process	
		Sample size	Average	Sample size	Average	Sample size	Average
pH	–	10	–	10	–	10	–
TS	mg/ℓ	〃	48,000	〃	33,900	〃	10,900
VTS	%	〃	83 8	〃	81 7	–	
SS	mg/ℓ	〃	41,500	〃	26,000	〃	984
VSS	%	〃	86 2	〃	83 7	–	
BOD	mg/ℓ	4	31,500	4	21,200	〃	7,740
S-BOD	mg/ℓ	〃	6,820	〃	7,610	–	
CODcr	mg/ℓ	〃	62,400	〃	52,700	〃	13,100
S-CODcr	mg/ℓ	〃	9,030	〃	12,700	–	
CODMn	mg/ℓ	〃	10,900	〃	9,810	〃	3,840
S-CODMn	mg/ℓ	〃	1,300	〃	4,060	–	
TOC	mg/ℓ	〃	18,600	〃	16,400	4	5,600
S-TOC	mg/ℓ	〃	3,700	〃	5,300	–	
T-Nmg/	mg/ℓ	〃	2,140	〃	1,700	4	1,060
PO_4-Pmg/	mg/ℓ	〃	525	〃	442	〃	158

2.3.2 Water Quality of Raw Wastewater for the Experiment

Among supernatent liquid discharged from this sludge treatment system, supernate from thermal sludge treatment, filtrate from press filter, filter media washing water, condensed water from thermal treatment reactor decomposed gas, decomposed-gas scrubber water, and steam dryer waste gas scrubber water were decided to be treated. Scrubber water for waste gas from melting furnace air pre-heater was decided not to be treated regarding it as wastewater possible to be discharged in terms of water quality. The water quality of mixture of these wastewater was confirmed to be almost similar to that of doubly diluted supernate from thermal sludge treatment. Thus, in the pilot plant study on treatment of supernatant liquid, the supernate from thermal sludge treatment was diluted with tap water so that its water quality might approach to that of mixed wastewater to be used as raw wastewater for the experiment. Table-3 gives the water quality of this raw wastewater. With regard to each water quality-related item of BOD, COD, Kj-N, and T-P, the percentage of soluble portion was 70–90%.

Table-3 Characteristic Qualities of Raw Wastewater for the Experiment

Items		Average	Standard deviation
pH		6.00	0.72
BOD	(mg/ℓ)	3,154	907
CODMn	(mg/ℓ)	2,080	428
CODcr	(mg/ℓ)	5,959	1,222
SS	(mg/ℓ)	663	298
T-N	(mg/ℓ)	641	138
Kj-N	(mg/ℓ)	623	132
NH_4-N	(mg/ℓ)	180	72
NOn-N	(mg/ℓ)	17.8	6.6
T-P	(mg/ℓ)	83.3	33.6
Color	(degree)	1,656	268
M-alkalinity	(mg/ℓ)	276	191

NOn; $NO_2 + NO_3$

2.4 Target of Treated Water Quality

Kobe city, in which treated water is discharged into Seto Inland Sea, when taking into consideration the control by immutable weight of CODMn based on the Law on Special Measures for Seto Inland Sea Environmental Protection (fiscal 1979), and further the administrative guidance on regulation of phosphorus to be expected in future, requires the target of treated water quality to far exceed the technical standard for effluent specified in the government ordinance of The Sewerage Law. Therefore, the target of treated water quality was established making reference to the effluent standard for nitrogen and phosphorus based on the Lake Biwa Eutrofication Control Ordinance, and experimental results and literature so far obtained.

The target of treated water quality thus established is below 5mg/L for BOD, below 30mg/L for COD, below 5mg/L for SS, below 10mg/L for total nitrogen, and below 5mg/L for total phosphorus.

2.5 Description of Plant for Treatment of Supernate from Thermal Treatment

The combination of biological, chemical, and physical treatment processes is required to eliminate each pollutant from raw wastewater colored with high concentrations of pollutant. First, to remove BOD, COD, and nitrogen, a two-stage biological treatment, in which nitrified liquor recycled biological nitrification-denitrification process by suspended microbe using activated sludge is employed, followed by nitrification and denitrification of low-concentration nitrogen by adhering microbial film sticking to filling-up media, was performed. After the biological treatment, a physico-chemical treatment combining coagulative precipitation, high-rate filtration, and activated carbon adsorption was carried out. Fig.-3 provides a flow sheet of the pilot plant.

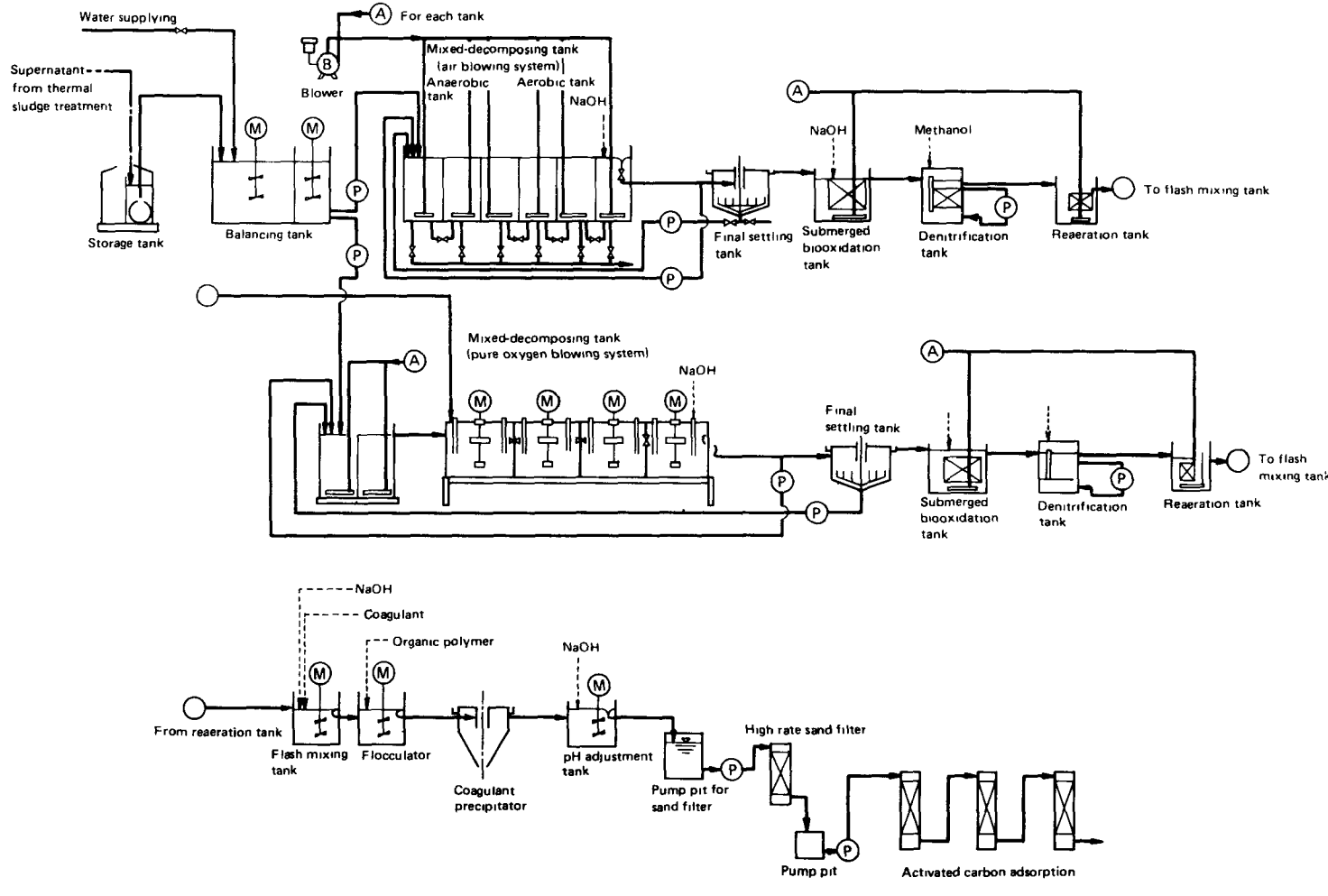
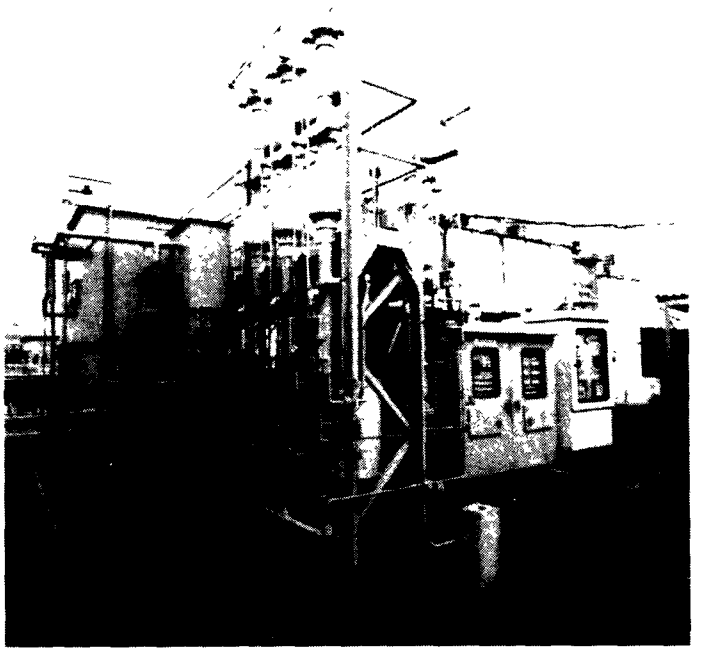


Figure-3 Treatment of Supernate from Thermal Sludge Treatment
– Pilot Plant Flow Sheet –



Photo; Treatment of Supernate from Thermal Sludge Treatment
– Pilot Plant –

A mixed-decomposing tank, given in this figure, means treatment vessels for performing nitrified liquor recycled nitrification-denitrification treatment by suspended microbe, which are divided into the front-stage anaerobic tanks and rear-stage aerobic tanks. The mixed-decomposing tank is composed of an air blowing system and a pure oxygen blowing system so that both may be compared with each other. Wastewater treated by both treatment methods is further treated in a submerged biooxidation tank and a denitrification tank by air blowing. Coagulative precipitation, sand filtration, and activated carbon treatment are provided to further treat wastewater treated in the previous process in such a manner as loading wastewater treated either by the air blowing method or oxygen blowing method..

3. MODIFIED MIXED LIQUOR RECIRCULATION ACTIVATED SLUDGE PROCESS BY AIR BLOWING

3-1 Experimental Procedure

A mixed-decompsing tank treats sludge using biological nitrification-denitrification reaction based on a nitrified liquor recycling system. Fig.-4 gives its flow sheet, while Table 4 the specifications of experimental equipment.

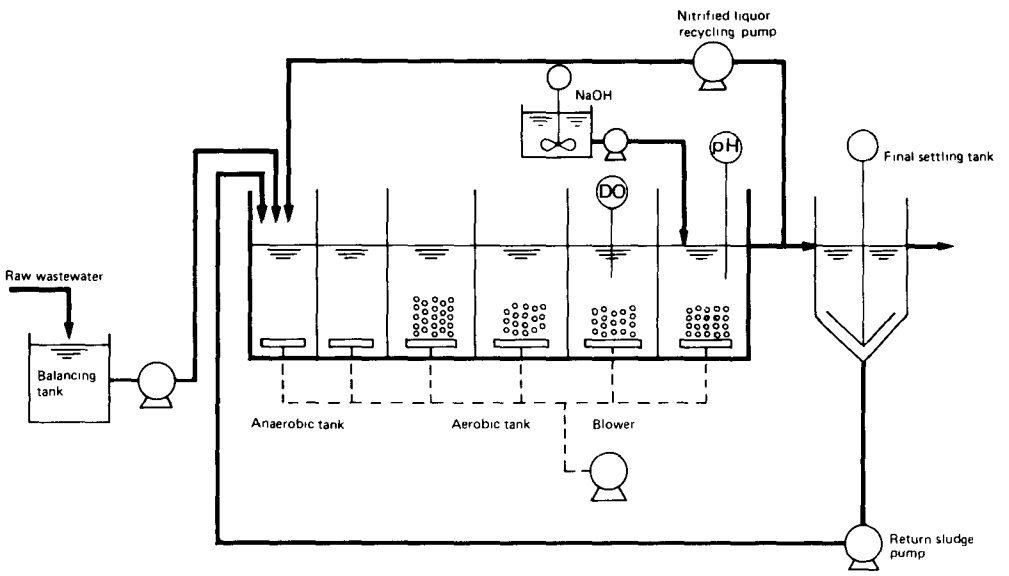


Figure-4 Flow-Diagram of Mixed-Decomposing Tank

Table-4 Specification of Experimental Apparatus

Facility		Items	Tank capacity (m^3)		Size	Specification
Balancing tank		1st stage	8.0	9.6	$1{,}600^{W} \times 3{,}100^{L} \times 2{,}000^{H}$	Mechanical agitation with propeller(0.4KW x 2 units)
		2nd stage	1.6			
Mixed-decomposing tank	Anaerobic tank	1st stage	1.4 (0.7)	2.8 (1.4)	$600^{W} \times 1{,}100^{L}$ (550) $\times 2{,}200^{H} \times 2$ tanks	Diffuser (pore size 250μ, void 40%)
		2nd stage	1.4 (0.7)			
	Aerobic tank	3rd stage	2.2 (1.1)	8.8 (4.4)	$800^{W} \times 1{,}100^{L}$ (550) $\times 2{,}500^{H} \times 4$ tanks	Diffuser (pore size 250μ, void 40%)
		4th stage	2.2 (1.1)			
		5th stage	2.2 (1.1)			
		6th stage	2.2 (1.1)			
Final settling tank		total	1.2		$800^{\phi} \times 2{,}400^{H}$	Rectangular clarifier (center-drive, 0.2KW) (with picket fence)
Instrument		(1) Air flowmeter	Rotor meter			
		(2) pH meter	Glass cell type indicating controller			
		(3) DO meter	Diaphragm galvanometer (Fixed type) Portable meter, Type EIL 1520 (Field measuring type)			

Remark) The numbers indicated in parenthesis for the items, volume and dimension apply to Run 4 and the following Runs.

The nitrified liquor recycling rate and return sludge volume were, respectively, 200% and 100% in relation to the inflow volume with the total recycling rate of 300%. The volume ratio of anaerobic tank to aerobic tank was 1:3. The anaerobic tank (denitrification tank) has been equally divided in volume by a partition plate into two tanks with the total volume of 2.8m (2 x 1.4m). The aerobic tank (nitrification tank) has also been equally divided in volume by partition plate with a 10cm x 10cm "opening" into four tanks with the total volume of 8.8m (4 x 2.2m). With the design inflow volume (Q) of 2.4m^3/day for the mixed-decomposing tank, in this experiment, an attempt was made to increase the inflow load varying it from 0.5Q to 2.5Q in sequence. Fig.-5 indicates water temperature and load condition for each Run.

The biological reaction is greatly affected by temperature. The temperature control was performed only in the final Run 6 at water temperature of 22–23°C. No temperature control was carried out in the other Runs.

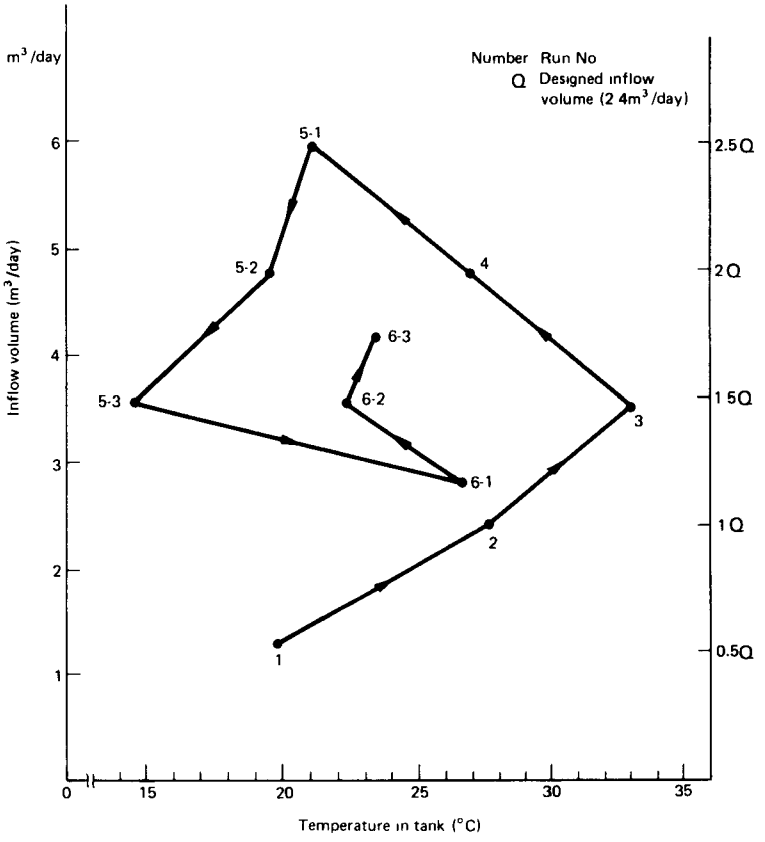


Figure-5 Inflow Volume and Temperature in Mixed-Decomposing Tank of Each Run

In Table-5 experimental conditions used in each Run are indicated. In Run 4 and the following Runs, the capacity of the mixed-decomposing tank was halved due to surface loading and solid loading become too much in a final settling tank.

Also in this case, partition plate was newly added to make an attempt at maintaining the flow condition in the tank the same.

To the aerobic tank, in which alkali is consumed with the reaction of nitrification, caustic soda (0.8% solution) was added as alkali source to compensate for its consumption.

In Run 3 and the following Runs, a pH meter was used to control pH in the fianl chamber of the aerobic tank at 6.5–7.0. Run 1 and Run 2 show cases, in which pH control was not performed, and Run 2 was subjected to no alkali supply. These Runs were carried out to know the feasibility of treatment of supernate from thermal sludge treatment with no addition of alkali, because the solution sometimes contains alkali to a considerable extent.

In the denitrification tank (anaerobic tank), in which air stirring was performed, MLSS was so high at 5,500–8,500 that its inside became anaerobic atmosphere with DO of ave. 0.13ppm and ORP of ave. 200mv. For each Run, two weeks were taken as an acclimatizing period due to the modification of conditions, and experimental data was obtained by four-week steady operation after the acclimatizing period. Excess sludge, by measuring the interfacial level of the final settling tank once a day, was withdrawn by timer operation so that the level might be in the range of about 15–20cm below the water level. The excess sludge volume was determined from measurements of withdrawn and stored excess sludge volume in pit and sludge concentration.

3.2 Experimental Results

Tables 5, 6, and 7 show the operating conditions and experimental results relating to the modified mixied liquor recirculation activated sludge process. The water temperature of raw wastewater was 13°–40°C, while the water temperature of the tank interior was in the range of 13°–35°C. The treated wastewater volume was in the range of 1.28m^3/day–5.94m^3/day (the treated wastewater volume for Run 4 and the following Runs, in which the tank capacity was halved, is compared with that for Run 1 – Run 3 by doubling the actual inflow volume). The detention time was in the range of 1.95 days – 8.9 days in relation to the inflow volume.

Table-5 Results of Modified Mixed Liquor Recirculation Activated Sludge Process by Air Blowing (1)

Items \ Run No	1	2	3	4	5−1	5−2	5−3	6−1	6−2	6−3
Period	4/15～5/15	6/2～6/29	7/23～8/24	9/7～10/5	10/22～10/29	11/2～11/5	11/9～11/19	11/24～11/26	11/27～12/10	12/14～12/25
Temperature of raw wastewater (°C)	20.2 (16.5～25.4)	32.3 (21.5～38.0)	36.1 (34.2～39.5)	27.4 (24.5～31.0)	22.3 (20.0～24.5)	21.0 (21.0～21.0)	16.6 (13.0～19.5)	26.4 (26.3～26.5)	23.5 (18.5～27.1)	23.2 (19.0～26.8)
Temperature in tank (°C)	19.7 (16.2～25.0)	27.6 (23.1～30.0)	33.0 (29.5～35.0)	26.9 (25.0～29.3)	21.2 (19.0～22.7)	19.6 (19.0～20.2)	14.6 (12.5～17.0)	26.5 (24.5～28.5)	22.4 (19.3～24.5)	23.4 (20.2～25.0)
Treated volume (m^3/day)	1.28	2.43	3.55	2.40	2.97	2.38	1.78	1.39	1.77	2.09
Recycled nitrified liquor volume (m^3/day)	1.63 (125%)	4.79 (197%)	6.81 (192%)	4.80 (200%)	6.0 (202%)	4.8 (202%)	3.6 (202%)	3.0 (216%)	3.61 (204%)	4.19 (200%)
Return sludge (m^3/day)	1.11 (86%)	2.43 (99.8%)	3.96 (111.6%)	2.43 (101.1%)	2.98 (100.3%)	2.36 (99.0%)	1.79 (100.5%)	1.62 (116.5%)	1.81 (102.2%)	2.09 (100.1%)
Total liquid volume (m^3/day)	4.02	9.65	14.32	9.63	11.95	9.54	7.17	6.01	7.19	8.37
Recycling ratio	211%	297%	304%	302%	302%	301%	305%	333%	303%	300%
Tank volume (m^3) — Anaerobic tank	2.8	2.8	2.8	1.4	1.4	1.4	1.4	1.4	1.4	1.4
Tank volume (m^3) — Aerobic tank	8.8	8.8	8.8	4.4	4.4	4.4	4.4	4.4	4.4	4.4
Tank volume (m^3) — Total	11.6	11.6	11.6	5.8	5.8	5.8	5.8	5.8	5.8	5.8
Tank volume (m^3) — Final settling tank	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2
Detention time (hr) (day) — Anaerobic tank	5.17 (2.2)	2.76 (1.2)	1.90 (0.8)	1.40 (0.6)	1.13 (0.5)	1.41 (0.6)	1.89 (0.8)	2.43 (1.0)	1.89 (0.8)	1.61 (0.7)
Detention time (hr) (day) — Aerobic tank	16.25 (6.8)	8.69 (3.6)	5.47 (2.5)	4.40 (1.8)	3.56 (1.5)	4.44 (1.9)	5.93 (2.5)	7.64 (3.2)	5.97 (2.5)	5.05 (2.1)
Detention time (hr) (day) — Total	21.42 (8.9)	11.45 (4.8)	7.87 (3.3)	5.80 (2.4)	4.69 (1.95)	5.85 (2.4)	7.82 (3.3)	10.07 (4.2)	7.86 (3.3)	6.66 (2.8)
Settling time (hr)	1.27	5.9	4.0	6.0	4.8	6.1	8.1	9.6	8.0	6.9
Air volume (m^3/hr) MLDO (mg/ℓ) Anaerobic tank 1st stage	5.1 0.17	4.1 0.06	5.0 0.06	− −	− −	− −	− −	− −	− −	− −
Air volume (m^3/hr) MLDO (mg/ℓ) Anaerobic tank 2nd stage	5.8 0.29	4.0 0.09	5.0 0.09	6.0 0.21	8.04 0.11	7.26 0.06	6.40 0.16	5.48 0.09	6.28 0.10	6.38 0.05
Air volume (m^3/hr) MLDO (mg/ℓ) Aerobic tank 3rd stage	9.8 1.3	19.0 2.6	21.5 1.65	− −	− −	− −	− −	− −	− −	− −
Air volume (m^3/hr) MLDO (mg/ℓ) Aerobic tank 4th stage	8.1 2.4	15.7 1.9	21.2 2.39	28.1 3.26	19.4 4.18	17.4 4.10	13.1 5.11	11.7 3.98	11.7 3.86	12.2 2.13
Air volume (m^3/hr) MLDO (mg/ℓ) Aerobic tank 5th stage	6.8 2.2	15.2 3.3	21.0 3.29	− −	− −	− −	− −	− −	− −	− −
Air volume (m^3/hr) MLDO (mg/ℓ) Aerobic tank 6th stage	5.1 2.0	15.4 5.1	21.5 4.11	28.8 4.51	20.1 5.52	18.6 5.95	13.5 6.75	11.4 5.07	11.6 4.82	12.6 3.61
Air volume (m^3/day) Air-wastewater ratio (Qair/Q) Anaerobic tank	262 204	194 80	240 68	144 60	192 65	174 73	154 86	132 95	151 85	153 73
Air volume (m^3/day) Air-wastewater ratio (Qair/Q) Aerobic tank	715 559	1,586 653	2,045 576	1,366 569	948 319	864 363	638 359	554 399	559 316	595 285
Air volume (m^3/day) Air-wastewater ratio (Qair/Q) Total	977 763	1,780 733	2,285 644	1,510 629	1,140 384	1,039 436	792 445	586 494	710 401	748 358
Air-tank volume ratio (m^3/m^3/hr) Anaerobic tank	3.9	2.9	3.6	4.3	5.7	3.1	4.6	3.9	4.5	4.6
Air-tank volume ratio (m^3/m^3/hr) Aerobic tank	3.4	7.5	9.7	12.9	9.0	8.2	6.1	5.3	5.3	5.6
Air-tank volume ratio (m^3/m^3/hr) Total	7.3	10.4	13.3	17.2	14.7	11.2	10.6	9.2	9.8	11.4
Air volume per BOD removed Air-m^3/kg-BOD	282	235	203	245	135	157	153	179	152	87
MLSS (mg/ℓ)	5,769	5,526	6,330	6,792	6,915	7,234	8,167	8,426	6,332	7,044
MLVSS (%)	78.7	81.2	84.9	88.3	90.3	90.0	90.2	88.3	86.6	85.8
SRT (day) Total volume	36	42	23	18	11	10	23	26	21	15
SRT (day) Mixed-decomposing tank	33	38	21	15	9	8	19	21	18	13
SRT (day) Aerobic tank	25	29	16	12	7	6	14	16	13	10

Table-6 Results of Modified Mixed Liquor Recirculation Activated Sludge Process by Air Blowing (2)

Items \ Run No		1	2	3	4	5 1	5 2	5 3	6 - 1	6 - 2	6 - 3
Surface loading	(m^3/m^2/day)	48	97	150	97	119	95	71	60	72	84
Solid loading	SSkg/m^3/day	27	53	94	65	82	68	58	50	45	59
SV_{30}		855	759	748	779	903	960	955	900	954	969
SVI		150	139	119	115	131	133	118	107	153	137
RSS	(mg/ℓ)	11,720	10,020	12,085	12,660	11,620	12,250	13,590	13,245	10,990	11,470
Sludge production	(kg/day)	209	196	360	261	451	526	261	228	208	334
	(kg/m^3)	1675	0816	1015	1084	1524	2215	1469	1649	1175	1607
	kg/BOD-kg	055	025	032	042	053	078	049	058	040	038
	kg/SS-kg	332	140	256	125	304	418	227	580	305	161
BOD-SS loading	(kg/kg/day) (Total)	0058	0128	0158	0161	0215	0165	0114	0081	0144	0217
BOD-capacity loading	(kg/m^3/day) (Total)	0335	0706	0991	1088	1488	1192	0921	0678	0900	1526
COD_{Mn}-SS loading	(kg/kg/day) (Total)	0034	0077	0103	0124	0169	0136	0093	0054	0110	0128
COD_{Mn}-capacity loading	(kg/m^3/day) (Total)	0195	0424	0644	0836	1165	0979	0747	0456	0685	0888
T-N-SS loading	(kg/kg/day) (Total)	0013	0025	0028	0036	0049	0036	0025	0014	0030	0043
T-N-capacity loading	(kg/m^3/day) (Total)	0072	0130	0180	0243	0338	0259	0208	0116	0187	0300
K_J-N-SS loading	(kg/kg/day) (Total)	0012	0023	0028	0035	0048	0.035	0025	0013	0030	0041
K_J-N-capacity loading	(kg/m^3/day) (Total)	0068	0126	0174	0237	0239	0252	0202	0112	0195	0290
K_J-N-SS loading	(kg/kg/day) (Aerobic tank)	0016	0030	0036	0046	0063	0046	0033	0018	0039	0054
K_J-N-capacity loading	(kg/m^3/day) (Aerobic tank)	0090	0166	0230	0312	0434	0333	0267	0148	0257	0382
NOn-N-SS loading	(kg/kg/day) (Anaerobic tank)	0021	0.037	0051	0080	0038	0041	0145	0038	0052	0060
NOn-N-capacity loading	(kg/m^3/day) (Anaerobic tank)	0116	0200	0320	0538	0260	0298	0115	0329	0339	0437
BOD removal rate (%)	Total	981	987	989	991	982	979	988	993	984	989
COD_{Mn} removal rate (%)	〃	785	768	815	841	823	836	827	877	859	843
T-N removal rate (%)	〃	718	772	782	763	621	610	551	744	814	802
K_J-N removal rate (%)	〃	875	885	911	930	666	687	576	943	941	903
NH_4-N removal rate(%)	〃	861	932	937	964	△	△	△	984	987	891
T-N removal rate (%)	Anaerobic tank	512	507	481	362	210	205	102	577	567	564
K_J-N removal rate (%)	〃	610	393	400	101	152	107	88	148	515	458
NOn-N removal rate (%)	〃	222	905	712	940	814	872	686	964	711	941
T-N removal rate (%)	Aerobic tank	△	△	△	132	103	. 96	128	△	△	△
K_J-N removal rate (%)	〃	228	359	523	743	212	277	181	68.5	582	440
Effluent, NOn-N/T-N (%)	〃	569	511	602	712	140	215	79	782	692	527
NH_4-N removal rate (%)	〃	235	682	773	919	144	218	67	953	951	649

△, minus value

Table-7 Results of Modified Mixed Liquor Recirculation Activated Sludge Process by Air Blowing (3)

Items	Run No	Plan	1	2	3	4	5-1	5-2	5-3	6-1	6-2	6-3
pH	Raw wastewater		6.7	6.1	5.8	6.6	5.6	5.1	5.1	5.1	5.2	5.7
	Anaerobic tank outlet		6.7	7.4	7.3	7.6	7.6	7.5	7.6	7.5	7.4	7.7
	Aerobic tank outlet		5.5	5.5	6.7	6.7	7.6	7.4	7.8	6.6	6.9	7.0
M-alkalinity mg/ℓ	Raw wastewater		469	352	224	319	124	27	74	37	85	241
	Anaerobic tank inlet		163	98	101	135	398	254	555	60	110	294
	Anaerobic tank outlet		74	295	283	549	653	604	807	476	403	746
	Dosing		162	0	140	212	0	0	0	70	144	231
	Final tank effluent		18	14	60	74	489	330	716	67	110	311
BOD mg/ℓ	Raw wastewater	4,200	3,070	3,356	3,240	2,633	2,910	2,910	3,009	2,841	2,930	4,235
	Anaerobic tank inlet	1,087	1,042	880	923	675	761	792	780	671	74	1,091
	Anaerobic tank outlet	-	89	126	114	219	331	284	293	62	87	367
	Mixed-D tank effluent	50	58(10)	42(4)	36(2)	24(3)	51(15)	62(12)	37(10)	19(3)	47(10)	46(9)
	Removal rate (%)	98.8	98.1	98.7	98.9	99.1	98.2	97.9	98.8	99.3	98.4	98.9
CODMn mg/ℓ	Raw wastewater	2,200	1,775	2,011	2,100	2,015	2,278	2,390	2,437	1,913	2,222	2,467
	Anaerobic tank inlet	925	831	856	814	743	869	890	922	622	783	906
	Anaerobic tank outlet		-	583	493	571	643	604	638	322	387	648
	Mixed-D tank effluent	500	382	467	388	321	403	391	421	235	313	387
	Removal rate (%)	77.3	78.5	76.8	81.5	84.1	82.3	83.6	82.7	87.7	85.9	84.3
CODcr mg/ℓ	Raw wastewater		5,954	6,483	5,819	5,652	6,652	6,400	6,412	4,386	4,911	6,645
	Anaerobic tank inlet		2,555	2,422	1,957	1,849	2,273	2,146	2,166	1,387	1,615	2,734
	Anaerobic tank outlet		975	1,068	938	1,263	1,480	1,393	1,448	590	746	1,431
	Mixed-D tank effluent		944	844	670	581	813	728	750	388	516	645
	Removal rate (%)		84.1	87.0	88.5	89.7	87.8	88.6	88.3	91.2	89.5	90.3
SS mg/ℓ	Raw wastewater	350	699	726	544	671	552	615	705	313	452	1,035
	Anaerobic tank inlet		365	288	247	232	177	218	221	101	163	291
	Anaerobic tank outlet		151	209	214	253	88	176	154	41	94	98
	Mixed-D tank effluent	50	207	142	148	85	52	86	60	30	67	43
	Removal rate (%)	85.7	70.4	80.4	72.8	87.3	90.6	86.0	91.5	90.4	85.2	95.8
T-N mg/ℓ	Raw wastewater	650	642	618	587	587	660	631	677	484	645	835
	Anaerobic tank inlet	343	334	261	241	251	353	343	398	206	250	333
	Anaerobic tank outlet		163	129	125	160	279	273	349	87	108	145
	Mixed-D tank effluent	240	181	141	128	139	250	246	304	124	120	165
	Removal rate (%)	63	71.8	77.2	78.2	76.3	62.1	61.0	55.1	74.4	81.4	80.2
Kj-N mg/ℓ	Raw wastewater	605	625	602	570	571	643	616	661	471	629	804
	Anaerobic tank inlet	177	259	203	178	172	322	299	375	129	184	260
	Anaerobic tank outlet		101	123	107	155	273	267	342	84	89	141
	Mixed-D tank effluent	34	78	69	51	40	215	193	280	27	37	78
	Removal rate (%)	94.4	87.5	88.5	91.1	93.0	66.6	68.7	57.6	94.3	94.1	90.3
NH_4-N mg/ℓ	Raw wastewater		201	237	159	149	132	122	153	123	151	230
	Anaerobic tank inlet		83	72	46	41	149	130	205	30	38.5	76
	Anaerobic tank outlet		36	52	42	65	181	170	238	41	35	71
	Mixed-D tank effluent		28	16	10	5.3	155	133	221	2	2	25
	Removal rate (%)		86.1	93.2	93.7	96.4	△	△	△	98.4	98.7	89.1
NO_2-N NO_3-N mg/ℓ	Raw wastewater	45	16	16	18	16	17	15	17	13	16	31
	Anaerobic tank inlet	166	79	58	63	78	31	44	23	77	67	73
	Anaerobic tank outlet		62	55	18	50	57	56	71	28	19	43
	Mixed-D tank effluent	206	103	72	77	99	35	53	24	97	83	87
	Effluent, NOn-N/T-N(%)	85.8	56.9	51.1	60.2	71.2	14.0	21.5	7.9	78.2	69.2	52.7
T-P mg/ℓ	Raw wastewater	200	80	87	53	59	117	97	114	67	115	144
	Anaerobic tank inlet	168	69	78	40	48	75	70	72	54	72	116
	Anaerobic tank outlet		-	-	-	-	-	-	-	-	-	-
	Mixed-D tank effluent	158	64	73	35	44	61	61	58	50	57	106
	Removal rate (%)	21	20	16	34	25	48	37	49	25	50	26
Color degree	Raw wastewater	-	-	1,610	1,550	1,620	1,800	1,600	1,720	1,640	1,740	1,910
	Effluent	-	-	1,010	990	1,050	1,190	1,280	1,490	960	1,200	1,400

( ) soluble, △, minus value

3.2.1 Characteristic of Raw Wastewater for The Experiment

BOD and CODMn of raw wastewater used for the experiment is considerably high in comparison with those for ordinary sewage. The BOD-to-CODMn ratio is about 1.4 from Fig.-6. Ordinary municipal sewage contains night-soil and domestic miscellaneous wastewater, the former of which contains a large amount of organic acid, thus being difficult to be measured as CODMn. Organism-derived high molecular organic matter, such as starch contained in domestic miscellaneous wastewater is also lower in CODMn than in BOD. Accordingly, for sewage, which mainly consists of night-soil and domestic miscellaneous wastewater, BOD/CODMn is larger than 1. Ordinary sewage has the BOD/CODMn of about 1.8, which, after biological treatment allowing removal of biologically decomposable matter, decreases to about 1.0. In light of the fact that raw wastewater used in the experiment is 1.4 in BOD/CODMn, this raw wastewater is considered to be slightly difficult to be subjected to biological treatment as compared with common municipal sewage.

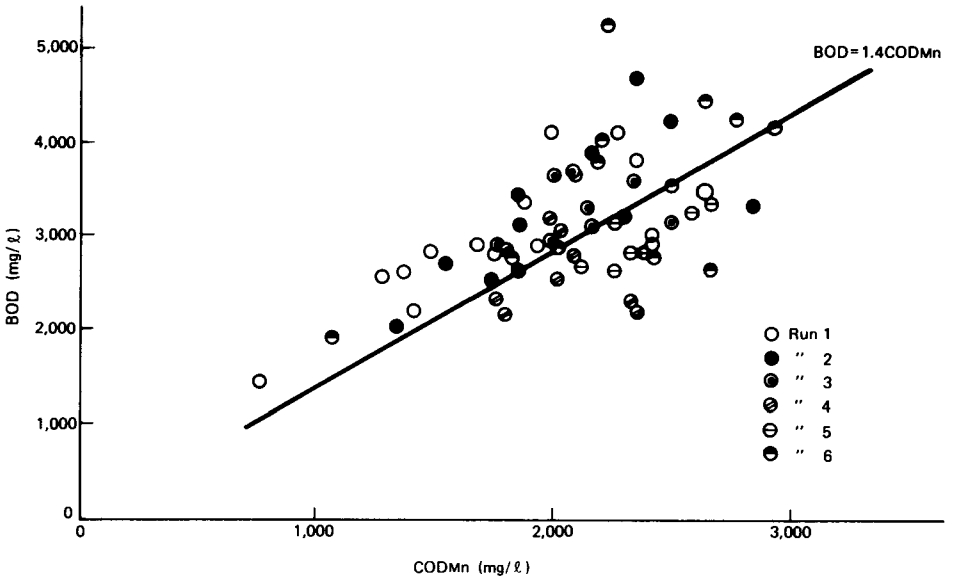


Figure-6 Relation between BOD and CODMn of Raw Wastewater

With regard to CODMn and CODcr, the relationship; CODcr = 2.3CODMn + 1,300 was obtained. Of total BOD, SS (suspended solid) – BOD is small, while S-BOD (soluble-BOD) much at almost 80%. With ordinary sewage, S-BOD holds 50%. This suggests that BOD is satisfactorily available as organic carbon source to heterotrophic denitrifying organisms.

T-N of raw wastewater for treatment is 640mg/L, about 20 times that of ordinary sewage, which is about 30mg/L. Kj-N occupies 97% of T-N in treatment raw wastewater, and 99% for ordinary sewage, showing that both little contain NO_2-N and NO_3-N forms. Org-N accounts for about 70% of T-N with about 450mg/L, while NH_4 N about 30% of T-N. Alubuminoid-nitrogen accounts for 45% of Org-N with about 200mg, indicating that organic nitrogen in other forms than alubuminoid form is relatively much contained. The BOD/T-N ratio for raw wastewater is about 4.4, while 3.7–4.4 for primary tank effluent of ordinary municipal sewage on the average.

3.2.2 BOD Removal

BOD-SS load was 0.058–0.217kgBOD/kgSS·day, and BOD hydraulic loading 0.335–1.488kgBOD/m^3 day, when MLSS was 5,500–8,400mg/L. This treatment condition allowed reduction in BOD from 2,600–4,200mg/L for inflow wastewater to 19–62mg/L for treated wastewater, giving the removal percentage of 98–99%. The soluble-BOD was 2–15mg/L. Fig.-7 shows the relationship between T-BOD and S-BOD for treated wastewater.

$$\text{S-BOD} = 0.233\ (\text{T-BOD}) - 2.08$$

Fig.-8 gives the relationship between BOD-SS loading and BOD removal level per unit SS.

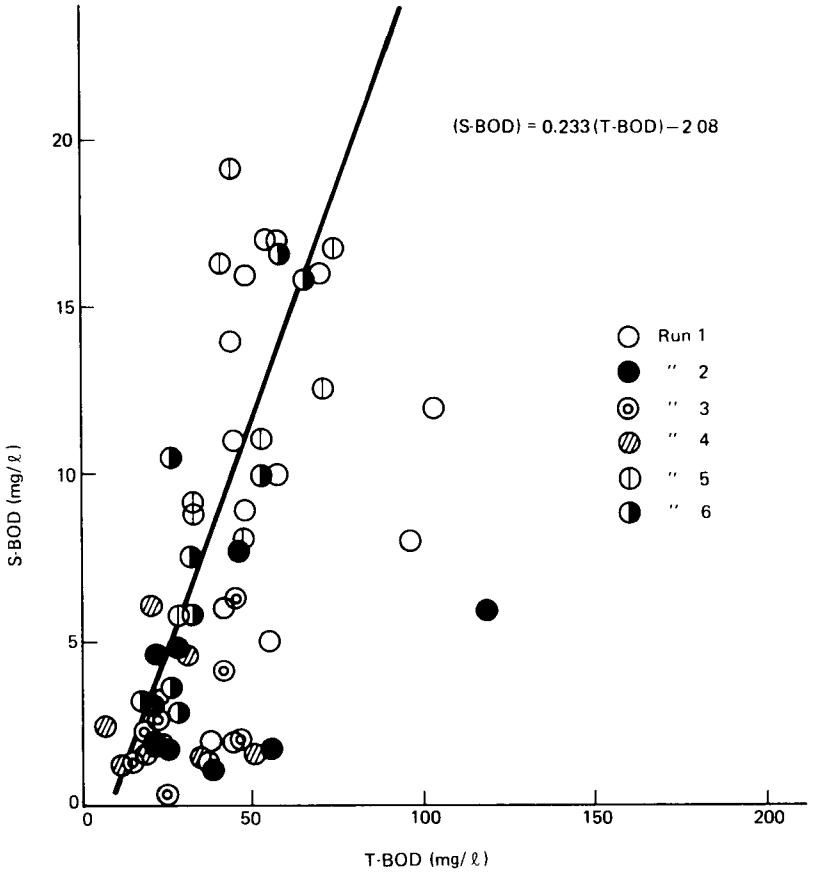


Figure-7 Relation between T-BOD and S-BOD
(effluents from mixed-decomposing tank)

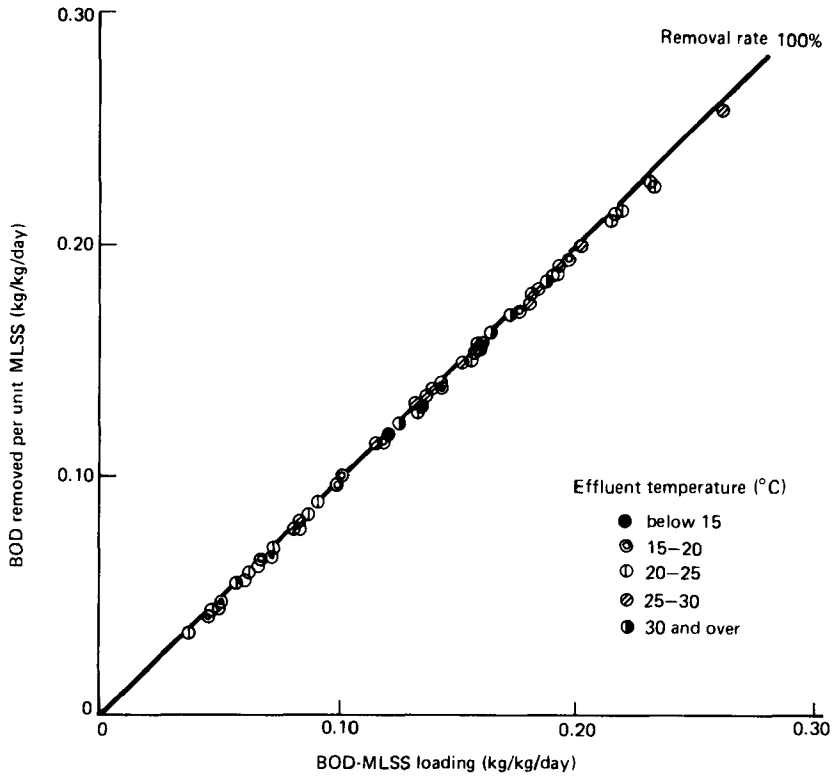


Figure-8 Relation between BOD-MLSS Loading and BOD Removed

BOD removal is performed in an anaerobic tank and aerobic tank of a mixed-decomposing tank. Fig.-9 plots BOD changes in each tank with the ave. values of each Run. This figure shows that, for all Runs, BOD was removed in the anaerobic tank to a considerable extent. In terms of BOD removal level, 70–90% (Run 3 and Run 4) of the total was removed in the anaerobic tank. Fig.-10 gives the transition in BOD removal for each chamber of the anaerobic tank and aerobic tank in relation to the data on Run 3, revealing that BOD removal rate is particularly large in the first chamber, and that there is a tendency of BOD removal rate being lowered with decreasing BOD concentration.

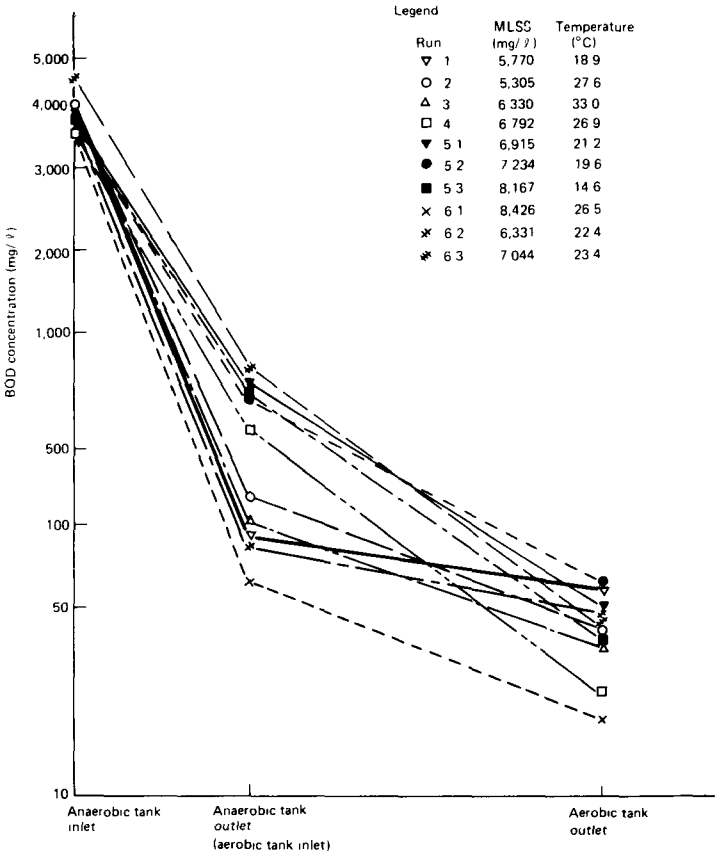


Figure-9 Variation of BOD in Mixed-Decomposing Tank

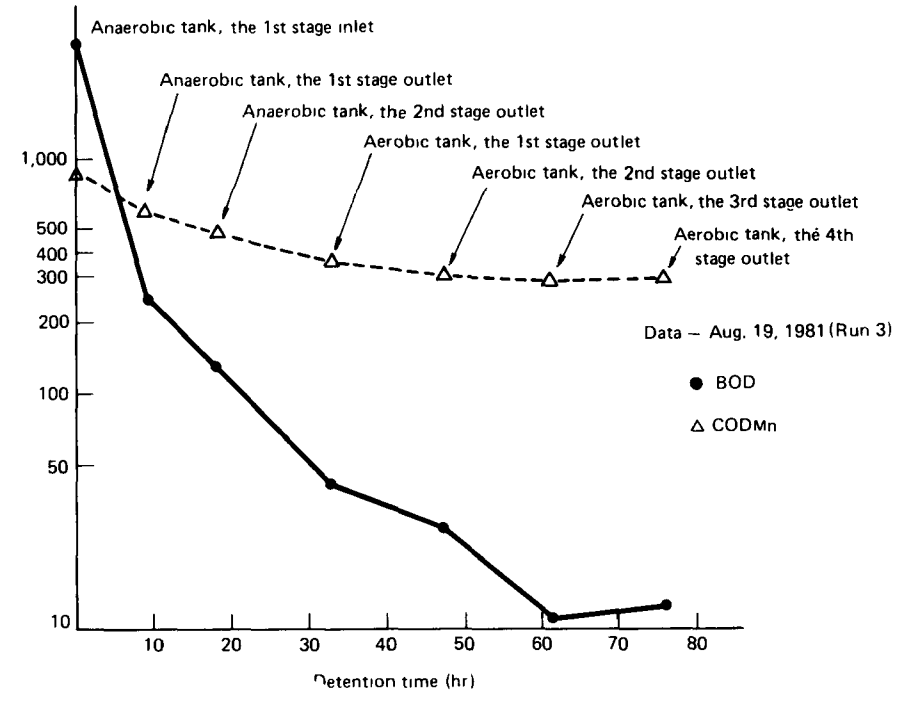


Figure-10 Variation of BOD and CODMn in Mixed-Decomposing Tank

3.2.3 CODMn Removal

The CODMn concentration of raw wastewater was about 2,100mg/L, about 80% of which is accounted for by soluble-CODMn. This is characteristic of supernate from thermal sludge treatment.

Operation was undertaken with CODMn-SS loading of 0.03–0.17kgCODMn/kgSS·day and CODMn hydraulic loading of 0.2–1.2kgCODMn/m^3·day. The COD decreased from 1,700–2,500mg/L for inflow wastewater to 240–470mg/L with the removal percentage of 75–88%.

As described above, BOD is removed in a mixed-decomposing tank to such a high extent taht BOD of treated wastewater is 19–62mg/L, but, because CODMn contains hard-biodegradable substances, CODMn remains at about 300–400mg/L, even when BOD is most removed.

Between CODMn and BOD of treated wastewater, the following relation was obtained.

$$COD_{Mn} = 1.551BOD + 309$$

CODMn removal efficiency is closely related to color degree removal. The ave. color degree is 1,687 for inflow wastewater, and 1,173 for treated wastewater, while the percentage of color degree removal was 30.5%. (Color degree measuring method: Platinum-Cobalt method).

3.2.4 Nitrogen Removal

In terms of ave. values for each Run, the ave. water temperature of tank interior was 15°C–33°C (20°C–33°C, when excluding Run 5), Kj-N loading 0.012–0.048 (0.012–0.041, when excluding Run 5) kg·Kj-N/kgMLSS·day, and Kj-N hydraulic loading 0.07–0.29 (0.07–0.29, when excluding Run 5)kg·Kj-N/m^3·day. As a result of this treatment, T-N removal percentage was 55–81% (72–81%, when excluding Run 5), and NH_4-N removal percentage 0–99% (86–99%, when excluding Run 5), when the detention time was 2.0–8.9 days (2.4–8.9 days, when excluding Run 5) in relation to the inflow wastewater volume, and the recycling rate 3.0 except for that of Run 1. Among the above, Run 5 was extremely low in nitrogen removal percentage because of reduced water temperature combined with increased nitrogen loading. Fig.-11 shows the relationship between T-N-SS loading and T-N removal level per unit SS.

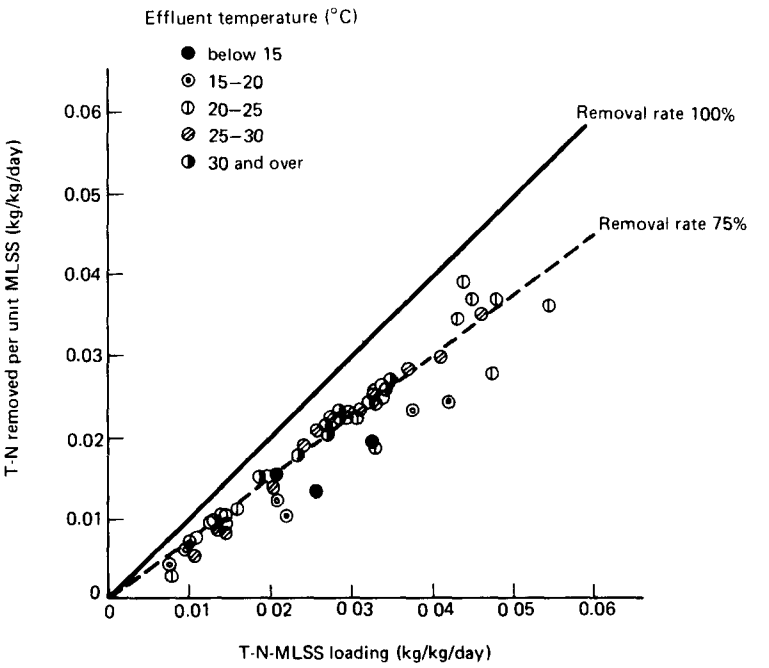


Figure-11 Relation between T-N-MLSS Loading and T-N Removed per Unit MLSS

The Kj-N-SS loading (kg·Kj-N/kgMLSS·day) in the aerobic tank was increased in sequence with 0.016, 0.030, 0.036, 0.046, and 0.063, which, respectively, corresponded in hydraulic loading to 0.09, 0.17, 0.23, 0.31, and 0.43kg·Kj-N/m^3·day, for which the Kj-N removal percentage were 88%, 89%, 91%, 93%, and 67%, respectively. This displays that the removal percentage sharply lowered between the Kj-N·SS loading of 0.046kg·Kj-N/kg·MLSS·day and 0.063kg·Kj-N/kg·MLSS·day. For the above-mentioned Kj-N-SS loading conditions, the T-N removal percentage is, respectively, 72%, 77%, 78%, 76%, 62%. In Run 4, the air-tank volume ratio in the aerobic tank, in which considerable bubbling occurred was 13 (Nm^3 –air/m^3 tank volume·H) in comparison with below 10 for the other Runs.

3.2.5 Nitrification Rate

Nitrification reaction by nitrifying bacteria is influenced by temperature. With nitrification reaction assumed as zero order reaction, the nitrification rate was determined according to the following equation.

$$K = \frac{\text{N-out} - \text{N-in}}{\text{Sa} \cdot t}$$

where K : nitrification rate mgN/mgMLVSS·day
N-in : initial concentration $(NO_2\text{-N}) + (NO_3\text{-N})$ mg/L
N-out : treated wastewater $(NO_2\text{-N}) + (NO_3\text{-N})$ mg/L
Sa : MLSS·mg/L
t : detention time (day)

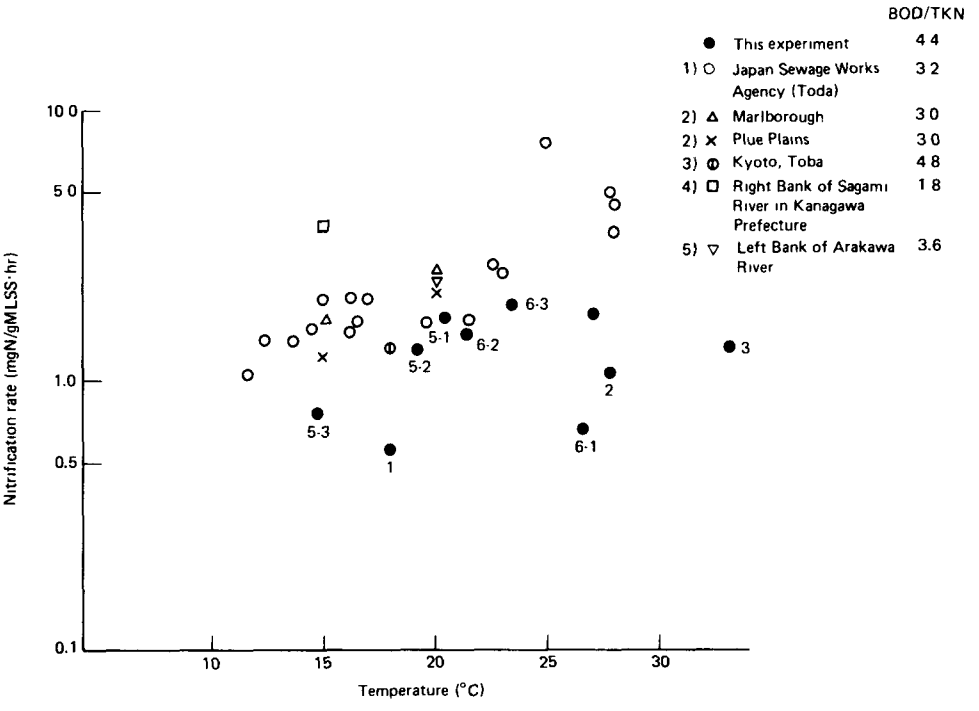
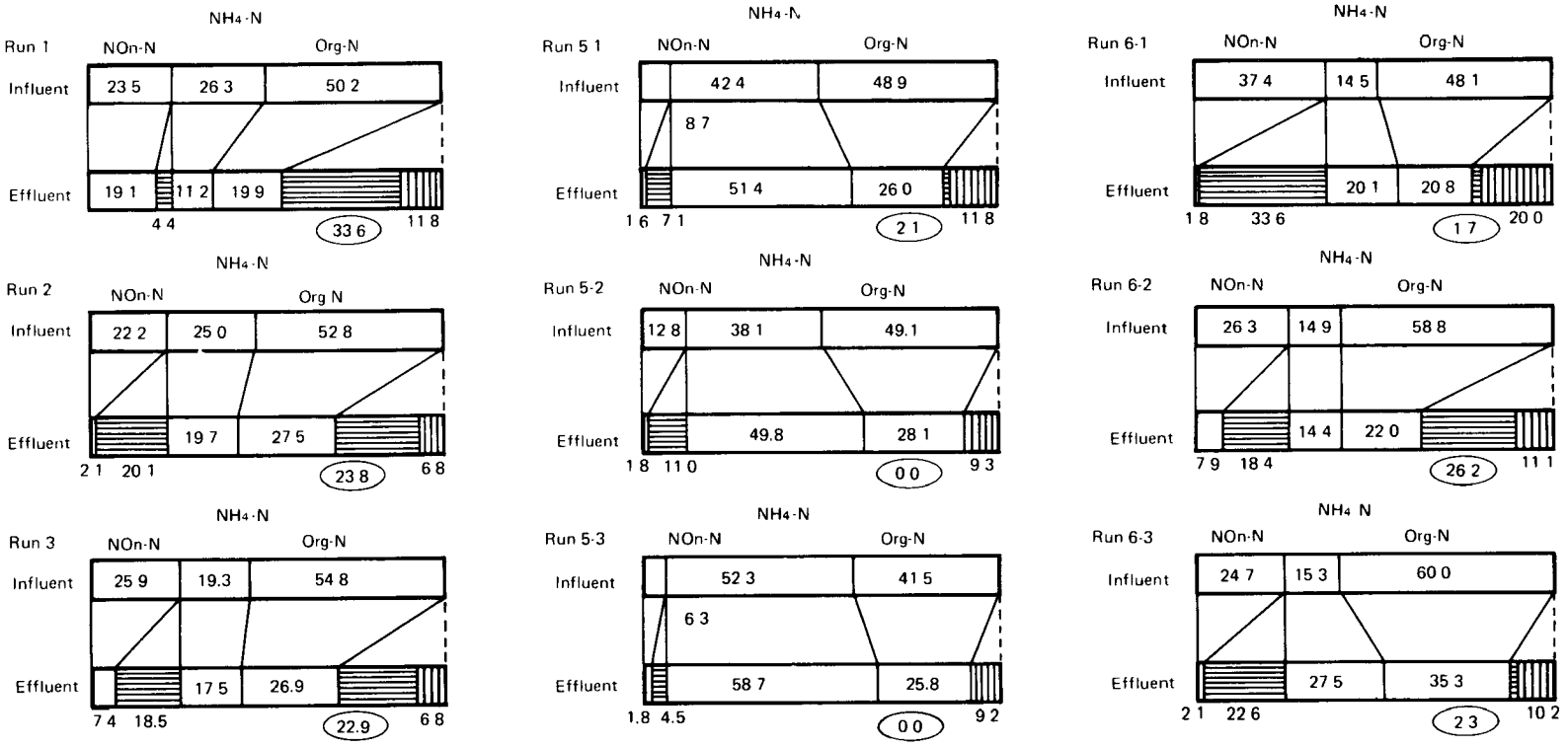


Figure-12 Relation between Temperature and Nitrification Rate

1) Construction Ministry, Shiga Pref., Japan Sewage Works Agency, "Development and Investigation on Advanced Sewage Treatment Technology" (1980)
2) US, EPA, "Process Design Manual for Nitrogen Control" (1975)
3) Sakai, Sumiyama, "Collection of Lectures Presented at the 17th Sanitary Engineering (Research) Meeting" (1981)
4) Kanagawa Pref., "Report on Experiment and Research Related to Advanced Sewage Treatment" (March, 1979)
5) Shiga Pref., Japan Sewage Works Agency, "Engineering Research relating to Management for Operation of Nitrified Liquor Recycled Denitrification Process" (1981)



Note 1 Numbers given in the figure indicate the percentage (%) of each nitrogen concentration in relation to T-N at the inflow point.

2 The concentration at the inflow point was calculated from the quantity of and each nitrogen concentration in inflow wastewater, recycling water, and return sludge.

3 indicates nitrogen removed by sludge withdrawal

4 indicates nitrogen biologically denitrified, the left side part of which shows nitrogen denitrified from NOn-N at the inflow point, while the right part shows denitrified nitrogen nitrified in the anaerobic tank with the values given in

Figure-13 Fractional Variation of Nitrogen of Different Forms at the Influent and Effluent Points of Anaerobic Tank (air blowing system)

Fig.-12 is the relationship between nitrification rate and temperature in a mixed-decomposing tank compared with other investigation results obtained from other municipal sewage. The nitrification rate for Runs 1, 2, 3 and 6-1 was low in comparison with that for the other experimental examples, while Runs 4, 5, and 6-2 provided the values close to those given in the other reports. Regarding the former Runs, in which Kj-N load was low, and detention time was long, calculation as zero order reaction led to apparently low values. On the other hand, with the latter Runs, high Kj-N load and short detention time allowed the nitrification rate not to become low.

3.2.6 Nitrification Reaction in Anaerobic Tank

Fig.-13 shows changes in level of nitrogen by form in the anaerobic tank. The concentration of each nitrogen at the entrance of the anaerobic tank was determined by calculation from each nitrogen concentration in the recycling liquor return portion and raw wastewater. Together with this, the figure also indicates nitrogen removal by sludge withdrawal. The percentage of nitrogen contained in sludge was, when expressed in Kj-N/MLSS, 9.52% on the average, and 11.08% when expressed in Kj-N/MLVSS. The percentage of T-N removal by sludge withdrawal was about 20%.

Fig.-13 shows a marked decrease in Kj-N for Runs 1, 2, 3 and 6-2. In the other Runs, little change was observed in Kj-N, taking into account the intake of nitrogen by sludge.

The decrease in Kj-N for the Runs mentioned above was estimated to be attributable to nitrification reaction in the anaerobic tank.

On the other hand, consideration of nitrification reaction in the anaerobic tank allowed alkalinity balance to approach to the theoretically calculated value.

The nitrification reaction in the anaerobic tank, in which air stirring was performed to prevent sludge putrefaction, thus permitting DO concentration in the tank to be maintained at 0.1—0.5mg/L, is considered to possibly occur.

3.2.7 Denitrification Reaction

Fig.-14 shows the relationship between BOD removal level and denitrification level in the anaerobic tank. The regression formula is expressed by the following equation.

$$\text{BOD Removal (mg/L)} = 4.4 \times \text{denitrified nitrogen quantities (mg/L)} + 270$$

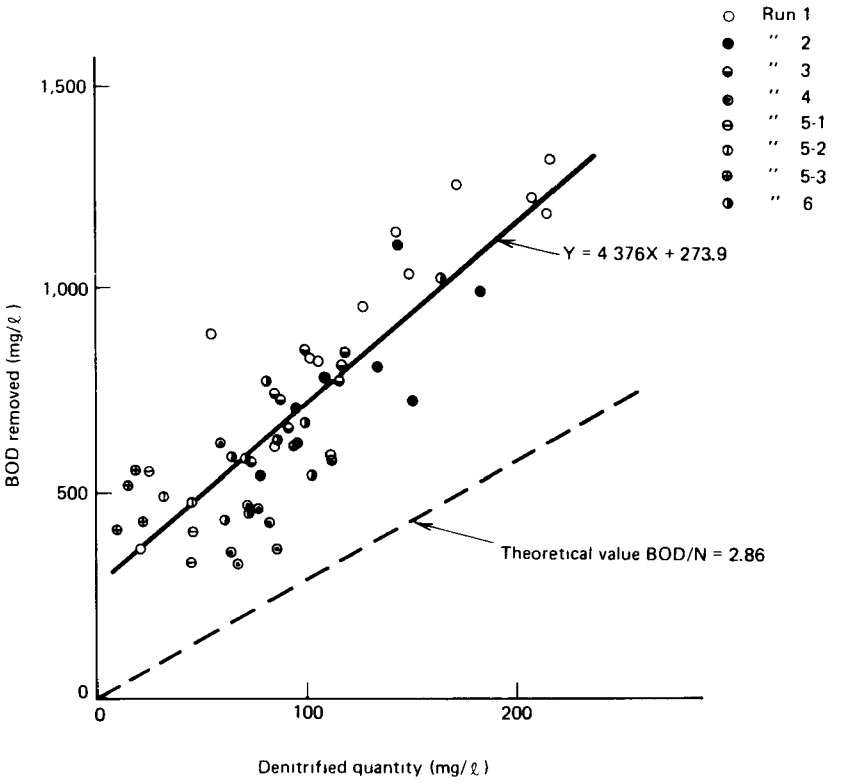


Figure-14 Relation between BOD Removed and Denitrified Quantity

Usually, BOD equivalent to two–three times as much as denitrified nitrogen quantities is used, but, according to the above equation, about 50% more was used. This is considered to be attributed to a small percentage of T-BOD used as hydrogen donor. In addition, as the reason for this, a decrease in BOD concentration, which is removed in other than denitrification reaction with about 300mg/L, with the removal of suspended solid mainly in the anaerobic tank, and a decrease in BOD by biological decomposition in an anaerobic tank are considered. In particular, this can also be inferred from the fact that the detention time for the aerobic tank, in which air stirring was performed, was long with 10–50 hours.

With denitrification reaction in each Run taken as zero order reaction, the denitrification rate in the anaerobic tank was calculated to determine its relation to water temperature, which is given in Fig.-15. The figure shows that the denitrification rate rises with an increase in water temperature.

The denitrification rate, which is 30–90mgN/gMLSS when excluding that for Run 5, is apparently low due to the long detention time for the anaerobic tank.

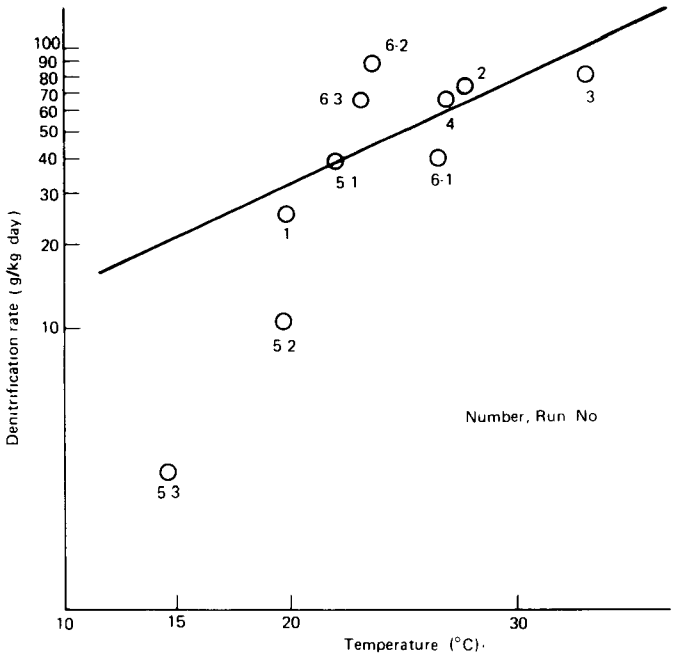


Figure-15 Relation between Temperature and Denitrification Rate in Mixed-Decomposing Tank (air blowing system)

3.2.8 Supplied Air Volume and Dissolved Oxygen Concentration

Table-8 gives supplied air volume, air-wastewater ratio, air-tank volume ratio, and DO concentration in each Run.

Air was blown into the anaerobic tank at the air-wastewater ratio (blowing air volume / inflow volume) of about 90 to prevent the putrefaction of mixed liquor in the tank with the average DO of 0.2mg/L in the second tank of the anaerobic tank. On the other hand, air was blown into the aerobic tank at the air-wastewater ratio of about 440 with the average DO of 4mg/L in the final stage tank. Initially, in Run 1, supplied air volume was varied with $10m^3/hr$, $8m^3/hr$, $7m^3/hr$, and $5m^3/hr$ for each tank of the aerobic tank so that DO of about 2mg/L might be attained for each tank of the aerobic tank.

In a final settling tank, however, DO concentration dropped with the occurrence of dentrification, which caused SS to be floated with the result that SS of supernate from the final settling tank was 200mg/L.

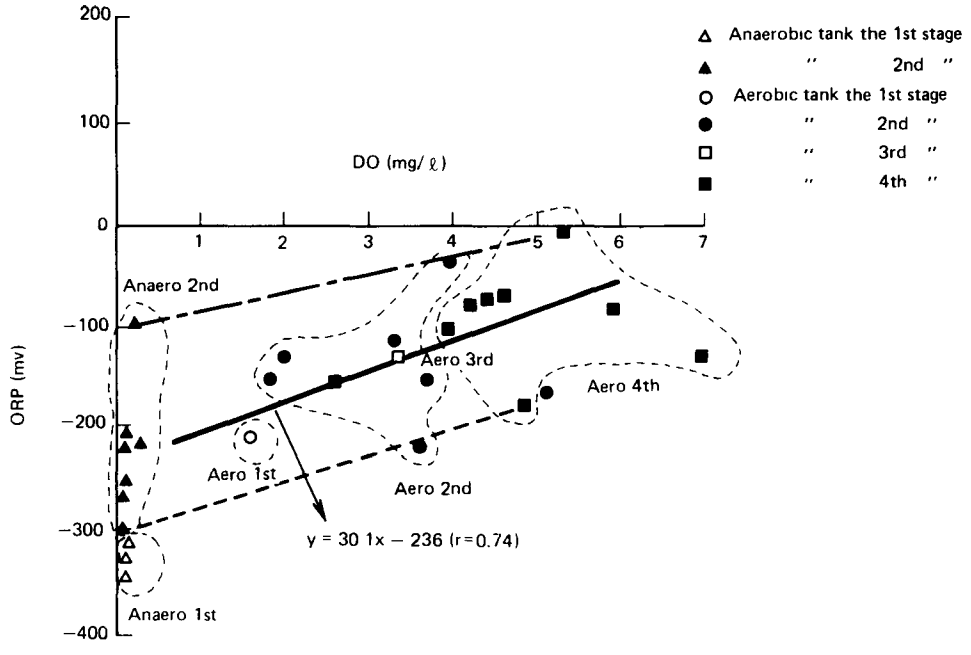
Accordingly, in Run 2 and the following Runs, DO was decided to be increased in the final stage tank of the aerobic tank. Air was supplied to each tank of the aerobic tank at almost the same level. As a result, DO was 3mg/L–6mg/L in the final stage tank of the aerobic tank, which presumably caused denitrification to difficultly occur in the final settling tank, allowing the outflow SS to be reduced to 50–100mg/L.

The air-wastewater ratio of the total mixed-decomposing tank is about 530, which is very high as compared with that of the conventional activated sludge process. In addition, the air-tank volume ratio [blowing air volume (m^3/hr) / tank volume (m^3)], G/V value, is considerably high with 3–5 for the anaerobic tank and 5–13 for the aerobic tank, in comparison with 0.4–0.7 for the conventional activated sludge process.

The reasons why air was supplied in such a large volume may be high BOD and N concentration, which result in a large amount of oxygen used for BOD removal and denitrification reaction, MLSS concentration as high as 6,000mg/L, and tank water depth as shallow as 2.5m compared with the water depth for the conventional aeration tank.

Table-8 Air Volume, DO and ORP in Mixed-Decomposing Tank

Items / Run No	Raw wastewater		Anaerobic tank					Aerobic tank					Total, mixed-decomposing tank	
	m^3/day	ORP (mV)	Air m^3/hr	Air-waste-water ratio m^3/m^3	Air-tank volume ratio (G/V) m^3/m^3/hr	ORP (mV)	DO (mg/ℓ)	Air m^3/hr	Air-waste-water ratio m^3/m^3	Air-tank volume ratio (G/V) m^3/m^3/hr	ORP (mV)	DO (mg/ℓ)	Air-waste water ratio m^3/m^3	Air-tank volume ratio (G/V) m^3/m^3/hr
1	1 28	–	10 9	204	3 89	–	0 23	29 8	559	3 39	–	1 99	763	7 28
2	2 43	-244	8 1	80	2 89	-311	0 09	66 1	653	7 51	– 3	3 24	733	10 40
3	3 55	-228	10 0	68	3 57	-252	0 07	85 2	576	9 68	-132	2 86	644	13 25
4	2 40	-262	6 0	60	4 29	-216	0 29	56 9	569	12 93	– 86	3 89	629	17 22
5 – 1	2 97	-228	8 0	65	5 71	-300	0 11	39 5	319	8 97	-175	4 85	384	14 68
5 – 2	2 38	– 98	7 26	73	3 05	-220	0 06	36 0	363	8 18	– 80	5 03	436	11 23
5 – 3	1 78	-135	6 40	86	4 57	-320	0 16	26 6	359	6 05	-125	5 93	445	10 62
6 – 1	1 39	-230	5 48	95	3 91	-265	0 09	23 1	399	5 25	-100	4 53	494	9 16
6 – 2	1 77	-178	6 28	85	4 49	– 98	0 10	23 3	316	5 30	– 20	4 34	401	9 79
6 – 3	2 09	-201	6 38	73	4 56	-208	0 05	24 8	285	5 64	– 73	2 87	358	11 4
Average	–	-200	–	89	4 09	-213	0 13	–	440	7 29	– 88	3 95	529	11 38



(Average values have been used)

Figure-16 Relation between ORP and DO in Mixed Decomposing Tank (air blowing system)

Fig.-16 shows the relationship between oxidation ruduction potential and dissolved oxigen in the anaerobic tank and aerobic tank.

As shown in the figure, correlation exists between both of them. The figure, which has given using average values, shows that the interior of the mixed-decomposing tank, including the aerobic tank, in which a good amount of dissolved oxygen is present, is in the anaerobic atmosphere.

The broken line in the figure indicates the time, when the loading became high and, at the same time, no denitrification proceeded with reduced ORP of the whole tank. The alternate long and short dash line indicates the case, in which the application of a heater allowed nitrification to satisfactorily progress with increased ORP.

3.2.9 Alkalinity Balance

Alkalinity balance in the anaerobic tank and aerobic tank was determined. As a calculating expression, the following equation was used, assuming that each formwise nitrogen varies in concentration in each tank.

$$(Alk)out - (Alk)in = \Delta(Org\text{-}N) \times 3.57 - \Delta(Kj\text{-}N) \times 7.14 + \Delta(T\text{-}N) \times 3.57 - (Alk)b$$

where (Alk)in : inflow wastewater alkalinity
(Alk)out: effluent alkalinity
(Alk)b : added alkalinity
(Org-N) : inflow wastewater (Org-N) – effluent (Org-N)
(Kj-N) : inflow wastewater (Kj-N) – effluent (Kj-N)
(T-N) : inflow wastewater (T-N) – effluent (T-N)

Fig.-17 and Fig.-18 show the relationship between theoretical alkalinity values and measured alkalinity values, using this equation.

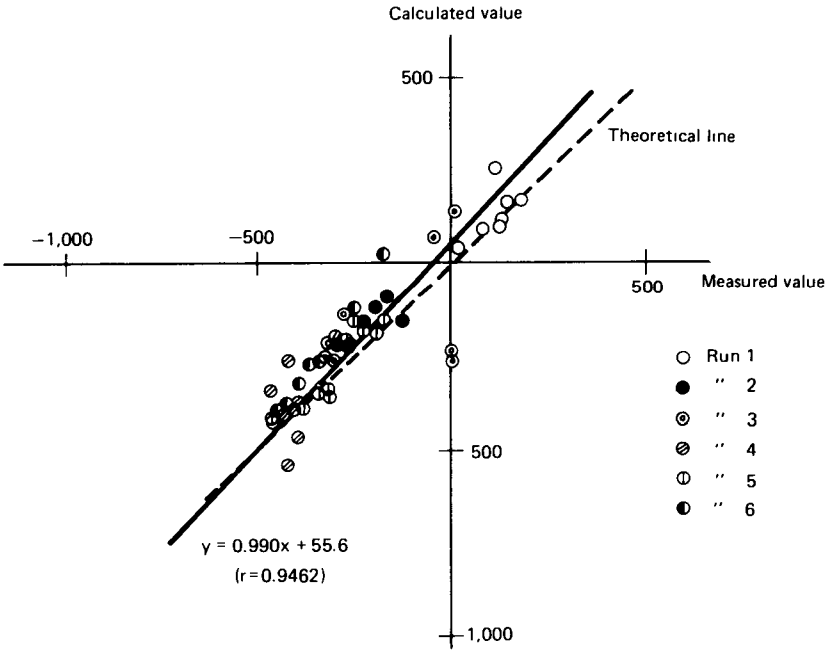


Figure-17 Alkalinity Balance in Anaerobic Tank (air blowing system)

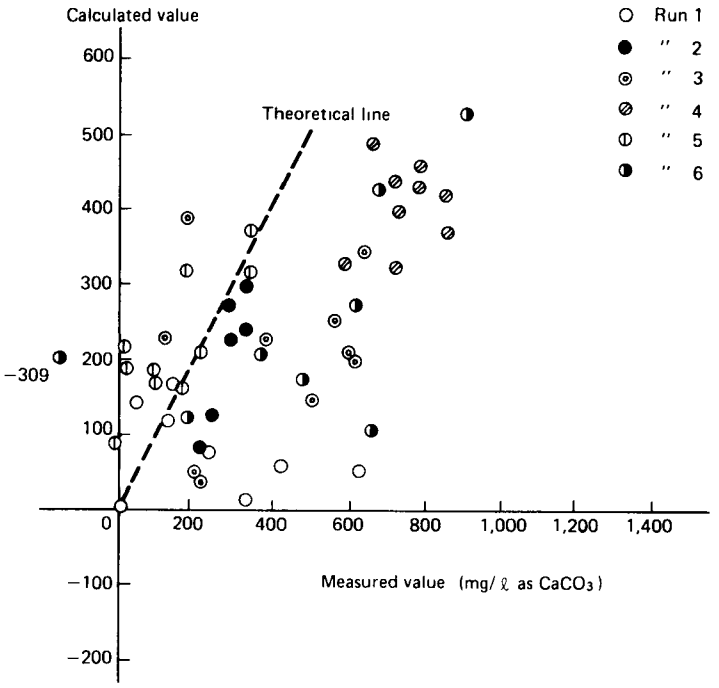


Figure-18 Alkalinity Balance in Aerobic Tank (air blowing system)

With regard to the anaerobic tank, the theoretical alkalinity values and measured alkalinity values are in considerable agreement with each other, while the aerobic tank provided great variations in measured alkalinity values, showing a tendency of alkalinity being larger in measured values, that is to say, a tendency of alkalinity having been supplied to the aerobic tank. This is estimated to be attributed to 500–1,200mg/L of volatile organic acid contained in inflow raw wastewater, which volatilized mainly in the aerobic tank, thus resulting in relatively increased alkalinity.

Stoichiometrically volatilization of 1mg of acetic acid equals an increase of 0.833mg in $CaCO_3$. Accordingly, volatilization of the total amount of organic acid mentioned above as acetic acid leads to an increase of 400–1,000mg/L in alkalinity. pH in the aerobic tank lower than in the anaerobic tank, which makes it difficult to catch hold of organic acid to the aerobic tank, the detention time for the aerobic tank about three times that for the anaerobic tank, and the blowing air volume for the former about five times that for the latter suggest volatilization of organic acid in the aerobic tank.

3.2.10 Phosphorus Removal

The T-P removal percentage was about 14%–50%.

In the present experiment, no biological dephosphorization was particularly observed in Runs 1 – 4. The relationship between T-P removal percentage and nitrogen removal percentage in each Run, however, shows that Run 5, in which little nitrification proceeded, provided a higher T-P removal percentage than the other Runs. Progressed nitrification reaction is said to result in increased concentrations of NO_2-N and NO_3-N contained in recycling liquor, which causes the anaerobic tank not to attain complete anaerobic condition with restricted release of phosphorus, thus leading to difficultly proceeding biological dephosphorization.

4. MODIFIED MIXED-LIQUOR RECIRCULATION ACTIVATED SLUDGE PROCESS BY PURE OXYGEN BLOWING

With a covered pure oxygen activated sludge system adopted in the aerobic tank, experiment on a modified recycling method was conducted. A pilot plant was operated with the nitrified liquor recycling rate of 100% and return sludge rate of 200%. In the anaerobic tank, aeration was performed for stirring with MLDO of 0.1–0.3mg/L. In the aerobic tank, biological oxidation was carried out at supplied oxygen volume of 2–4m^3 O_2 /m^3 of raw wastewater with MLDO of 7–13mg/L.

With respect to removal of BOD and COD_{Mn} this process gave good results as in the case of the modified mixed liquor recirculation process by air blowing. Regarding removal of nitrogen, however, good results were not obtained.

Therefore, in the latter part of the experiment, the final stage tank of the aerobic tank was changed from a closed-type to an open-type with air blowing performed in the final stage tank. This resulted in progress of nitrification, which allowed the percentage of nitrogen removal to be improved.

5. FIXED MEDIUM TYPE NITRIFICATION-DENITRIFICATION PROCESS

Wastewater treated by the modified mixed liquor recirculation process by air blowing was fed to a submerged biooxidation tank for fixed medium bed nitrification. The filling-up media had the specific surface of 100m^2 /m^3, supplied air volume was 40–80 times as much as filling-up media volume, and DO concentration in the tank was maintained at 2–6mg/L. However, good removal of BOD and T-N in a mixed-decomposing tank resulted in poor propagation of adhering microbe, causing little nitrification reaction to proceed.

The nitrification percentage for Run 6 was relatively high with 27.2% on the average (water temperature; 10°C). This is 54% in terms of values at 20°C. The coefficient of temperature used here, whose value is θ = 1.11, was determined from the nitrification rate in a mixed-decompsing tank. In Run 6, when Kj-N loading was 0.13kg/m^3/day, the nitrification percentage of 80%, which can be expected with a large adhering microbe level, was obtained as one experimental value.

In the modified mixed liquor recirculation process, experiment on Run 2, in which pH control was not performed, was carried out by dosing alkali agents.

The alkalinity balance in the tank was almost in agreement with the theoretical alkalinity value.

In the mixed-decomposing tank and submerged biooxidation tank, BOD, which can be used as microbial mutrient by denitrifying bacteria was most removed. Thus to the denitrification tank, methanol was added.

When a value [ratio of methanol to (NO_2-N)+ (NO_3-N)] is 3 or more, no effect of carbon source on denitrification percentage was observed, while, with a value of 2 or less, a decrease in denitrification percentage was observed. With NO_2-N + NO_3-N loading in the range of 0.05–0.18kg/m^3·day, the denitrification percentage is about 70–95%. Also in this case, in which no information was provided on adhering microbe level, no quantitative analysis was conducted. Table-9 and Table-10 give the results on fixed medium type nitrification-denitrification process for air-blowing mixed-decomposing tank effluent.

Table-9 Treatment Efficiency of Submerged Biooxidation Tank
(treated; effluents from mixed-decomposing tank by air blowing)
(Table indicates average value of experimental data of each Run)

Items \ Run No.	1	2	3	4	5−1	5−2	5−3	6−1	6−2	6−3
Inflow volume m^3/day	1.07	2.26	3.29	2 20	2.50	2.02	1.61	1.27	1.57	1.77
Temperature °C	17.9	25.2	30.7	24.5	17.8	17.0	12 9	13.9	10.0	14.6
Detention time hr	61	28	19	28	25	31	39	49	39	35
pH in	5.48	5.51	6.67	6.69	7.57	7.43	7.80	6.60	6.86	6.99
out	5.49	6.64	7.09	6.65	7.50	7.25	7.61	5.26	6.91	7.16
Alkalinity in	18	14	60	74	489	330	716	67	118	311
[NaOH dosing]	0	28	0	0	0	0	0	0	0	0
mg/ℓ out	12	76	98	60	415	214	443	5	84	251
BOD in	58 (10)	42	36	24	51	62	37	19 (3)	49	46
mg/ℓ out	45	28	9	3	25	14	24	33	12	22
Ave. removal rate (%)	19	33 3	75.0	87.5	50 9	77 4	351	△	74.4	52.1
COD_{Mn} in	382	467	388	321	403	391	421	235	313	387
mg/ℓ out	359	406	328	270	364	347	394	249	254	363
Ave. removal rate (%)	6.0	13	15.4	15.8	9 6	11.2	6.4	△	18.8	6.2
COD_{cr} in	944	844	670	581	813	728	750	388	516	645
mg/ℓ out	697	776	536	478	675	607	656	407	406	591
Ave. removal rate (%)	26.2	8.1	20	17.7	17 0	16 6	12.5	△	21.3	8.4
SS in	207	142	148	85	52	86	60	30	67	43
mg/ℓ out	200	64	38	16	10	20	15	13	23	50
Ave. removal rate (%)	3.3	54.9	74.3	81.8	80.7	76.7	75.0	56.6	65.6	△
T-N in	181	141	128	139	250	246	304	124	120	165
mg/ℓ out	177	134	114	126	221	224	281	168	115	157
Ave. removal rate (%)	2.2	4 9	10.9	9.3	11.6	8.9	7.5	△	4.1	4.8
Kj-N in	78	69	51	40	215	193	280	27	37	78
mg/ℓ out	76	67	45	29	187	162	223	48	30	65
Ave. removal rate (%)	△	1.4	11.6	26.2	13.1	15.1	20.5	△	27.2	28.7
NH_4-N in	28	16	10	5.3	155	133	221	2	2	25
mg/ℓ out	33	18	12	1.6	134	110	174	27	1	15
Ave. removal rate (%)	△	△	△	69.8	13.5	17.3	21.3	△	50	40
Org-N in	50	53	40	34.7	60	60	59	25	36	53
mg/ℓ out	43	49	33	27 4	53	52	49	21	29	50
Ave. removal rate (%)	14	7.5	17 5	21.0	11.7	13.3	11.9	16	19.4	5.7
NOn-N in	103	72	77	99	35	53	24	97	83	87
mg/ℓ out	101	67	69	77	34	62	58	120	85	92
Effluent;NOn-N/T-N(%)	57.1	50.0	60.1	61 1	15.4	27.7	20.6	71.4	73.9	58.6
T-P in	64	73	35	44	61	61	58	50	57	106
mg/ℓ out	60	69	34	44	60	53	60	60	50	112
Ave. removal rate (%)	6.2	5.4	2.8	0	1.6	13.1	△	△	12.2	△

BOD, ( ) soluble BOD, △ ; minus value

* The removal percentage given in the table indicates the average value of removal percentage in relation to the individual experimental results.

Table-10 Treatment Efficiency of Denitrification Tank
(treated; submerged biooxidation tank effluent)

(Table indicates average value of experimental data of each Run)

Items \ Run No.	1	2	3	4	5−1	5−2	5−3	6−1	6−2	6−3
Inflow volume m³/day	1.07	2.26	3.29	2.20	2.50	2.02	1.61	1.27	1.59	1.77
Temperature °C	18.0	25.1	31.2	24.5	18.0	16.8	13.0	13.5	10.5	11.3
Detention time hr	49	22	15	23	20	25	31	40	31	28
DO mg/ℓ	0.24	0.12	0.08	0.27	0.20	0.09	0.18	0.10	0.14	0.12
Methanol added mg/ℓ	465	319	282	468	153	260	224	365	360	303
Methanol/NOn-N ratio(%)	4.5	4.9	4.1	4.8	4.5	4.1	3.7	3.1	4.4	3.4
pH in	5.49	6.64	7.09	6.65	7.60	7.25	7.61	5.26	6.91	7.16
out	7.49	7.22	7.48	7.26	7.49	7.50	7.60	7.28	7.08	7.35
M-Alkalinity in	12	76	98	60	415	214	443	5	84	251
mg/ℓ out	284	272	307	323	505	382	528	313	191	390
BOD in	45	28	9	3	25	14	24	33	12	22
mg/ℓ out	93	52	14	5	25	15	22	134	87	94
Ave. removal rate (%)	△	△	△	△	0	△	8.3	△	△	△
CODMn in	359	406	328	270	364	347	394	249	254	363
mg/ℓ out	357	409	290	250	347	288	366	324	285	381
Ave. removal rate (%)	0.56	△	11.6	7.4	4.7	17.0	7.1	△	△	△
SS in	200	64	38	16	10	20	15	13	23	50
mg/ℓ out	212	91	27	8	16	16	19	11	13	36
Ave. removal rate (%)	△	△	28.9	50	△	20	△	15.4	43.5	28
T-N in	177	134	114	126	221	224	281	168	115	157
mg/ℓ out	92	76	62	59	179	191	236	123	83	92
Ave. removal rate (%)	48.0	43.3	45.6	53.2	19.0	14.7	16.0	26.8	27.8	41.4
Kj-N in	76	67	45	29	187	162	223	48	30	65
mg/ℓ out	79	67	51	36	172	168	213	88	32	56
Ave. removal rate (%)	△	0	△	△	8.0	△	4.5	△	△	13.8
NH_4-N in	33	18	12	1.6	134	110	174	27	1	15
mg/ℓ out	26	15	14	1.5	125	120	165	52	3	6
Ave. removal rate (%)	21.2	16.7	△	6.3	6.7	△	5.2	△	△	60
NOn-N in	101	67	69	97	34	62	58	120	85	92
mg/ℓ out	13	9	12	23	7.3	23	23	35	51	36
Ave. removal rate (%)	87.1	86.6	82.3	76.3	78.5	62.9	60.3	70.8	40	60.9
T-P in	60	69	34	44	60	53	60	60	50	112
mg/ℓ out	53	65	31	43	62	53	58	55	49	117
Ave. removal rate (%)	11.7	5.8	8.8	2.3	△	0	3.3	8.3	2.0	△

△ ; minus value

* The removal percentage given in the table indicates the average value of removal percentage in relation to the individual experimental results.

A re-aeration tank is located behind a submerged biooxidation tank. In the re-aeration tank, the percentage of BOD removal was 35% on the average. Also in Kj-N, NH_4-N, NOn-N, and T-P other than this, little change was observed between before and after treatment.

6. COAGULATIVE PRECIPITATION

Table-11 shows the efficiency of treatment by coagulative precipitation. In the physico-chemical treatment experiment following coagulative precipitation, alterations were not made in inflow wastewater volume, but in conditions, such as chemicals addition percentage. For Run 5, no experiment was performed on coagulative precipitation. For Run 3, wastewater treated by a pure oxygen blowing process was used.

As coagulant, combination of 200mg/L of aluminium sulphate and 0.5mg/L of anionic polymer, and combination of 500mg/L of ferric chloride and 0.5mg/L of anionic polymer were used with good results obtained.

That is, these cases both provided good removal percentage in relation to CODMn, T-P, and color degree. The average CODMn removal percentage was 38%, average T-P removal percentage 93%, and average color degree removal percentage 55%. A proportional relation was shown between color degree removal percentage and CODcr removal percentage.

Table-11 Results of Coagulative Precipitation Experiment

(Table indicates average value of experimental data of each Run)

Run	Items	pH	M-alkalinity mg/ℓ	BOD mg/ℓ	CODMn mg/ℓ	CODcr mg/ℓ	SS mg/ℓ	Kj-N mg/ℓ	NH_4-N mg/ℓ	NO_2-N NO_3-N mg/ℓ	T-P mg/ℓ	Color degree	Coagulant dosing
1	Raw waste	8 02	326	32	337	802	72	76	25	4	58	–	
	Effluent	8 01	195	11	237	546	142	57	22	4	13	–	Al_2O_3 200mg/ℓ
	Removed quantity	–	131	21	100	256	−70	19	3	0	45	–	Polymer
	Removal rate (%)	–	40 2	65 6	29 7	31 9	–	25 0	120	0	77 6	–	0 5mg/ℓ
2	Raw waste	7 36	276	24	386	659	56	66	18	6	61	1,061	Al_2O_3 200mg/ℓ
	Effluent	6 36	48	6	224	416	24	45	17	4	2	420	
	Removed quantity	–	228	18	162	243	32	21	1	2	59	641	Polymer
	Removal rate (%)	–	82 6	75 0	42 0	36 9	57 1	31 8	5 6	33 3	96 7	60 4	0 5mg/ℓ
3	Raw waste	7 87	1,118	22	392	676	17	275	240	4	18	1,083	$FeCl_3$ 500mg/ℓ
	Effluent	7 16	530	12	262	480	16	246	218	6	0 9	747	
	Removed quantity	–	588	10	130	196	1	29	22	−2	17	336	Polymer
	Removal rate (%)	–	52 6	45 5	33 2	29 0	5 9	10 5	9 2	–	94 4	31 0	0 5mg/ℓ
4	Raw waste	7 59	327	13	246	427	6	35	2	24	41	979	$FeCl_3$ 500mg/ℓ
	Effluent	7 18	93	0 4	130	201	9	16	1	18	1	294	
	Removed quantity	–	234	0 9	116	226	−3	19	1	6	40	685	Polymer
	Removal rate (%)	–	71 6	69 2	47 2	52 9	–	54 3	50 0	25 0	97 6	70 0	0 5mg/ℓ
6	Raw waste	7 43	319	42	308	514	20	41	8	31	73	1126	$FeCl_3$ 300mg/ℓ
	Effluent	6 79	99	24	211	302	13	35	7	27	3	481	
	Removed quantity	–	220	18	97	212	7	6	1	4	70	645	Polymer
	Removal rate (%)	–	69 0	42 9	31 5	41 2	35 0	14 6	12 5	12 9	95 9	57 3	0 5mg/ℓ

7. HIGH-RATE SAND FILTRATION

Using downflow and pressure type high-rate filtration, operation was performed with the condition of LV = 188m/day. As the treatment efficiency, the average SS removal percentage was 30%. With regard to the other items, little treatment efficiency was observed.

Table-12 gives the experimental results on high-rate sand filtration.

Table-12 Results of High-Rate Sand-Filter Experiment

(Table indicates average value of experimental data of each Run)

Run	Items	Alkalinity (mg/ℓ)	BOD (mg/ℓ)	CODMn (mg/ℓ)	CODcr (mg/ℓ)	SS (mg/ℓ)	Kj-N (mg/ℓ)	NH_4-N (mg/ℓ)	NO_2 NO_3 >N (mg/ℓ)	T-P (mg/ℓ)	Color (degree)
1	Raw waste	195	11 4	237	546	142	57	22	42	127	–
	Effluent	203	10 8	274	546	100	60	23	41	92	–
	Removal rate(%)	(−4)	5	(−16)	0	30	(−5)	0	0	28	–
2	Raw waste	48	6 2	224	416	237	45	17	37	20	420
	Effluent	47	6 4	218	401	125	44	17	33	17	416
	Removal rate(%)	0	0	3	4	47	0	0	10	15	0
3	Raw waste	530	11 7	263	480	16	246	218	56	09	747
	Effluent	492	10 9	258	459	11	243	213	44	08	763
	Removal rate(%)	7	7	2	4	31	1	2	21	0	0
4	Raw waste	93	0 4	130	201	93	16	14	178	12	294
	Effluent	92	0 6	129	204	68	16	14	187	06	290
	Removal rate(%)	0	0	0	0	27	0	0	(−5)	50	0
5	Raw waste	452	15 6	341	620	18	175	126	14	56	1,161
	Effluent	454	15 4	339	599	11	177	126	11	57	1,161
	Removal rate(%)	0	0	0	3	39	0	0	21	0	0
6	Raw waste	99	23 5	211	302	13	35	7	27	3	481
	Effluent	98	20.3	206	291	22	36	7	26	3	482
	Removal rate(%)	0	14	2	4	(−70)	0	0	0	0	0

8. ACTIVATED CARBON ADSORPTION

Experiment was conducted using a downflow and pressure type granular activated carbon adsorption tower pilot plant. The adsorption tower, composed of 3 towers with effective length of 1.68m, was operated in a merry-go-round style with a contact time of 100 minutes and LV of 72m/day. The average CODMn removal percentage for the treated wastewater was 81% (range: 72–92%). The adsorption in the first tower, when the CODMn of wastewater in the third tower was below 30mg/L, was 160–200g/kgAC with the water-AC ratio of about 550, which made the life of activated carbon before replacement about 30 days. An activated carbon adsorption process with coagulative precipitation omitted, which allows only three days as the life of activated carbon before replacement, was confirmed to permit no practical application.

Table 13 and Fig.-19 give the experimental results on activated carbon adsorption treatment.

Table-13 Efficiency of Activated Carbon Treatment

(Table indicates average value of experimental data of each Run)

Run \ Items		Alkalinity (mg/ℓ)	BOD (mg/ℓ)	CODMn (mg/ℓ)	CODcr (mg/ℓ)	SS (mg/ℓ)	Kj-N (mg/ℓ)	NH_4-N (mg/ℓ)	NO_2 NO_3 >N (mg/ℓ)	T-P (mg/ℓ)	Color (degree)	Alb-N (mg/ℓ)
1	Raw waste	203	10 8	274	546	100	60	23	4 1	9 2	–	21 8
	Effluent	219	4 1	49	100	12	29	21	2 3	3 1	–	4 4
	Removal rate(%)	(-8)	62	82	82	88	52	9	44	66	–	79 8
2	Raw waste	47	6 4	218	401	12 5	44	1 7	3 3	1 7	416	15 1
	Effluent	98	4 1	40	69	5 5	20	16	1 4	1 2	11	2 7
	Removal rate(%)	(-109)	36	82	83	56	55	0	58	29	97	82 1
3	Raw waste	492	10 9	258	459	1 1	243	213	4 4	0 8	763	16 0
	Effluent	484	6 8	42	67	10	201	195	1 6	0 6	112	2 6
	Removal rate(%)	2	38	84	85	9	17	8	64	25	85	83 8
4	Raw waste	92	0 6	129	204	6 8	16	1 4	18 7	0 6	290	8 4
	Effluent	129	0 5	10	23	2 1	2	1 1	16 2	0 2	6	0 6
	Removal rate(%)	(-40)	17	92	89	69	88	21	13	67	98	92 9
5	Raw waste	454	15 4	339	599	1 1	177	126	1 1	57	1,161	23 1
	Effluent	475	16 2	95	152	10	135	122	8 6	32	123	7 0
	Removal rate(%)	(-5)	(-5)	72	75	9	24	3	22	44	89	69 7
6	Raw waste	98	20 3	206	291	22	36	7	26	3	482	14 4
	Effluent	129	8 6	53	80	6	14	7	26	2	34	2 1
	Removal rate(%)	(-32)	58	74	73	73	61	0	0	34	93	86 1
Designed value	Raw waste	–	15	200	–	10	78	3	1	5	–	75
	Effluent	–	5	30	–	5	9	3	1	5	–	6
	Removal rate(%)	–	67	85	–	50	88	–	–	–	–	92

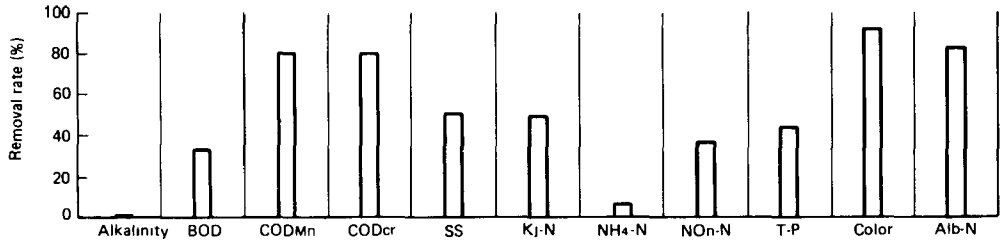


Figure-19 Removal Rate of Respective Water Quality Items in Activated Carbon Adsorption Process

9. COST FOR TREATMENT

9.1 Flow Sheet of Plant for Treatment of Supernate from Thermal Sludge Treatment

In calculating the treatment cost, the same process as used in the experimental plant was intended to be adopted as a treatment method. In the experiment, however, the removal efficiency in a fixed medium type nitrification-denitrification process fell short of the planned value due to shortage of adhering microbe level, etc. Therefore, studies were undertaken on the following three cases.

1st Case Adoption of the same process as used in the experimental plant
- Assumption that the Kj-N removal percentage in the submerged biooxidation tank (fixed medium bed) is 54%
- Assumption that the T-N removal percentage in the denitrification tank (fixed medium bed) is 78%

2nd Case Adoption of the same process as used in the experimental plant
- Assumption that the Kj-N removal percentage in the submerged biooxidation tank (fixed medium bed) is 80%
- Assumption that the T-N removal percentage in the denitrification tank (fixed medium bed) is 89%

3rd Case No fixed medium type nitrification-denitrification to be performed.

Fig.-20 gives a flow sheet used for design calculation, while Table-14 provides estimated treatment efficiency for each case.

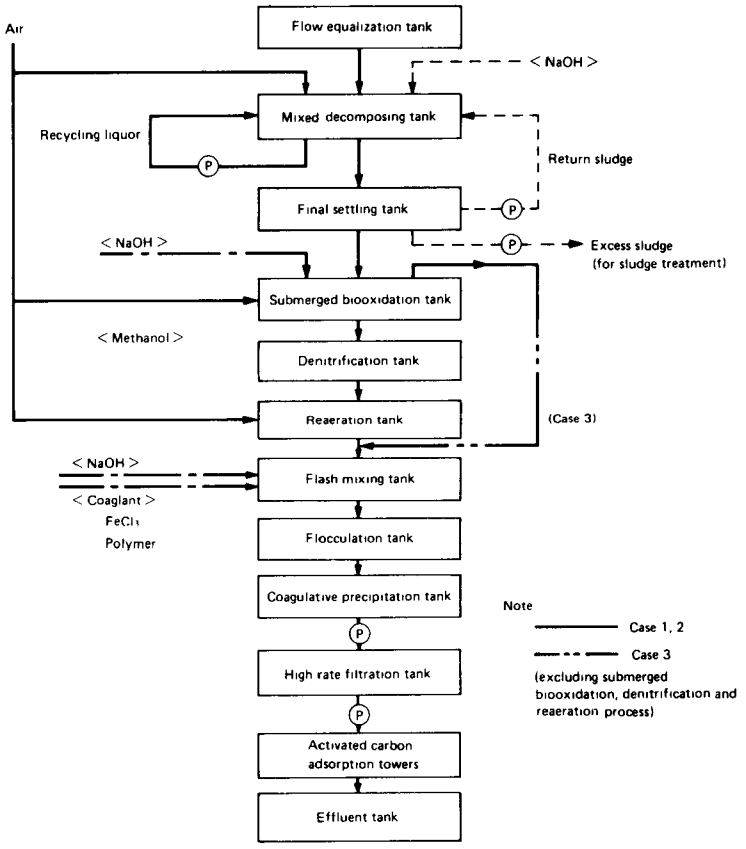


Figure-20 Flow Sheet of Treatment Plant of Supernate from Thermal Sludge Treatment Process

Table-14 Expected Treatment Efficiency

Items	Raw wastewater	Case 1		Case 2		Case 3	
		Effluent	Removal rate (%)	Effluent	Removal rate (%)	Effluent	Removal rate (%)
BOD mg/ℓ	3,200	5	99.8	5	99.8	10	99.7
COD_{Mn} mg/ℓ	2,200	30	98.6	30	98.6	30	98.6
SS mg/ℓ	640	5	99.2	5	99.2	5	99.2
T-N mg/ℓ	640	17	97.3	10	98.3	120	81.3
Kj-N mg/ℓ	628	10	98.4	4	99.4	22	96.5
NH_4-N mg/ℓ	182	7	96.2	3	98.4	16	91.3
NOn-N mg/ℓ	12	7	–	6	–	98	–
T-P mg/ℓ	110	5	95.5	5	95.5	5	95.5

9.2 Construction Cost and Operation Cost

Table-15 shows calculations on construction cost and operation cost in relation to the above three cases. The construction cost for this pilot plant, when roughly calculated with its scale enlarged to 1,000^3 m/day, is 1,500–1,800 million yen, or 1.5–1.8 million yen per m^3 of treated wastewater. Omission of fixed medium type nitrification-denitrification equipment, which provided poor treatment efficiency, makes the construction cost 1,200 million yen.

Of the operation management cost, the maintenance management cost (mainly power charges and chemicals expenses) is 400–500 yen/m^3. The summation of depreciation expense and maintenance management cost is 600–650 yen/m^3. A little less than 50% of this was accounted for by facilities for the modified mixed liquor recirculation activated sludge process, 20% by fixed medium type nitrification-denitrification equipment, and the remaining 30% by facilities for physico-chemical treatment.

Table-15 Construction Costs, Depreciation Expenses, Chemicals Expenses and Power Charges of the Treatment Facility for Supernate from Thermal Sludge Treatment Process

Process	Cost		Case 1	Case 2	Case 3
Suspended sludge type recyclation process	Construction costs	(million yen)	704	673	703
	Depreciation expenses	(yen/m^3)	892	864	899
	Chemicals expense + Power charge	(yen/m^3)	2100	210.0	201.2
	Sub-total	(yen/m^3)	2992	2964	3001
Fixed medium bed type nitrification and denitrification process	Construction costs	(million yen)	707	449	–
	Depreciation expenses	(yen/m^3)	87 0	55.6	–
	Chemicals expense + Power charge	(yen/m^3)	453	495	–
	Sub-total	(yen/m^3)	132.3	105.1	–
Coagulative precipitation	Construction costs	(million yen)	67	67	67
	Depreciation expenses	(yen/m^3)	105	105	10.5
	Chemicals expense + Power charge	(yen/m^3)	380	380	380
	Sub-total	(yen/m^3)	48.5	48.5	485
High-rate sand filtration	Construction costs	(million yen)	58	58	58
	Depreciation expenses	(yen/m^3)	97	97	97
	Chemicals expense + Power charge	(yen/m^3)	17	17	17
	Sub-total	(yen/m^3)	11.4	11.4	114
Activated carbon adsorption	Construction costs	(million yen)	201	201	261
	Depreciation expenses	(yen/m^3)	349	34.9	45.9
	Chemicals expense + Power charge	(yen/m^3)	1162	116.2	2495
	Sub-total	(yen/m^3)	1511	151.1	2954
Common cost	Construction costs	(million yen)	80	80	80
	Depreciation expenses	(yen/m^3)	7.3	73	73
	Chemicals expense + Power charge	(yen/m^3)	0	0	0
	Sub-total	(yen/m^3)	7.3	7.3	7.3
Total	Construction costs	(million yen)	1,817	1,528	1,169
	Depreciation expenses	(yen/m^3)	238.6	204.4	163.3
	Chemicals expense + Power charge	(yen/m^3)	411.2	415.4	499.4
	Sum Total	(yen/m^3)	649.8	619.8	662.7

Fig.-21 shows the percentage, at which each unit process holds the construction cost, depreciation expense and operation cost (power charges and chemicals expenses), power charges, and chemicals expenses in relation to Case 1.

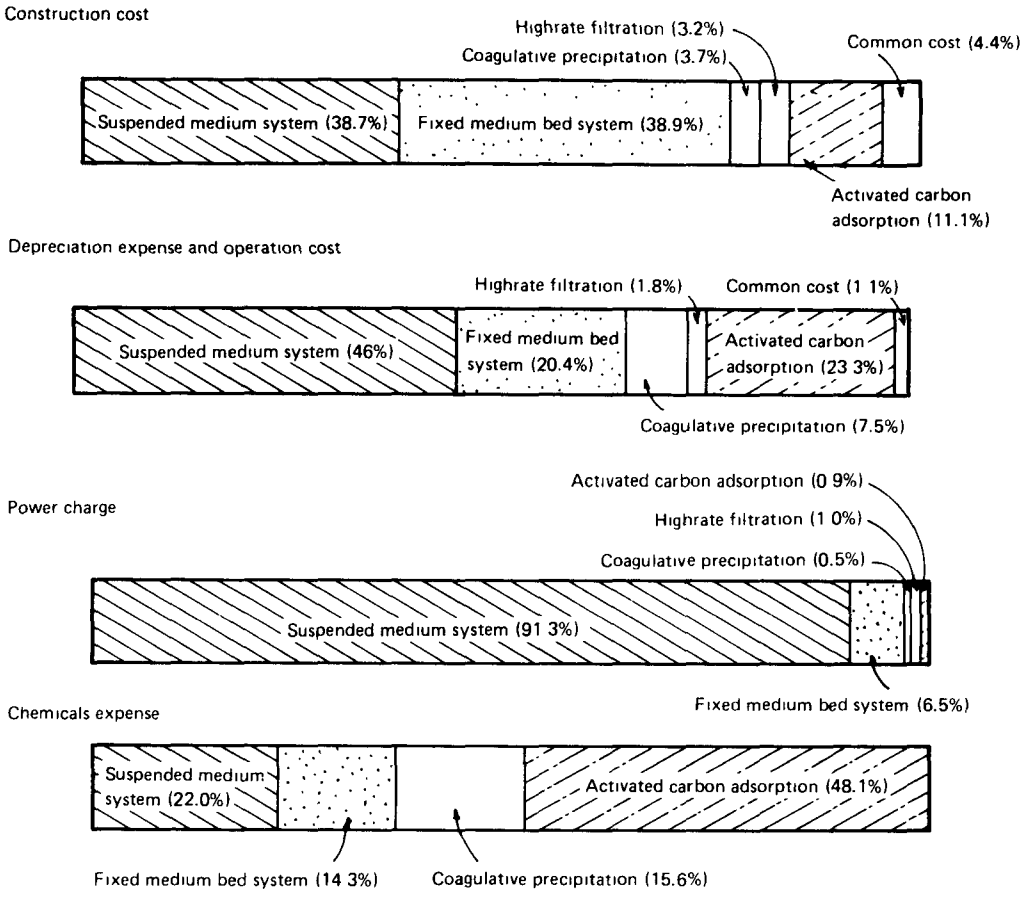


Figure-21 Percentage of Cost Accounted for by Each Unit Process

Table-16 gives estimates on operation cost relating to thermal treatment and melting equipment for reference.

The total waste liquid volume, when sludge with 100 ton-DS/day is subjected to low-temperature thermal treatment, is 3,880m^3/day.

Estimation was performed with a treatment scale taken as 20 ton-DS/day.

Table-16 Operating Costs of Thermal Treatment and Melting Process

(yen/ton)

Process \ Cost	Power charge	Heavy oil	Water for industrial use	Chemicals expense	Total
Pretreatment (liquid cyclon)	108	–	–	–	108
Centrifugal concentration	4,675	–	–	–	4,675
Low temperature thermal treatment	4,447	–	–	–	4,447
Dewatering (press filter)	2,199	–	–	–	2,199
Drying	1,048	–	–	–	1,048
Melting	2,873	13,289	608	49,952	66,722
Deodoration	1,584	–	224	644	2,452
Common	118	–	–	–	118
Total	17,052	13,289	832	50,596	81,769

Note) Assuming that treated water is used for filter media washing water and seal water, only electric charges have been added up.

10. CONCLUSION

It was confirmed that supernate from thermal sludge treatment, when doubly diluted and individually treated by a series of unit procesees, modified mixed liquor recirculation activated sludge process, fixed medium type nitrification-denitrification process, coagulative precipitation process, high-rate sand filtration process, and activated carbon adsorption, can be discharged directly into the sea. The actual design, however, is considered to require further studies on recycling rate for mixed liquor recirculation process, volume of nitrification tank, and other problems.

The present report was compiled from "Technological Assessment on Experimental Plant for Treatment of Supernatant from Thermal Sludge Treatment in Kobe City, March 1982, J.S.W.A." I wish to thank Dr. Kazuhiro Tanaka, Chief of Research and Technology Development Section of Japan Sewage Works Agency for his kind instruction in this compilation.

I also wish to express our appreciation to Mr. Kazuo Omiya*1 and Mr. Isao Kinoshita,*2 who took direct charge of this assessment.

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REFERENCE

OPERATION PERFORMANCE OF SLUDGE TREATMENT PILOT PLANT WITH THERMAL TREATMENT AND MELTING SYSTEM

(ABSTRACT)

(1) Centrifugal Thickening Equipment

a) The centrifugal thickening machine is retentive of sludge for 5 to 10 minutes. Concentrated sludge of given concentration can be obtained by properly combining centrifugal effect, and sludge supply time and sludge amount supplied per one cycle as shown in Fig.-1 and Table-1.

b) Electric energy required for operating inspected machine was 1.3Kwh to 2.5Kwh per cubic meter for raw sludge (Concentration of SS was 2%) and 130Kwh per ton for concentrated sludge solid.

c) It is economical to increase SS amount wasted during the unit time by raising concentration of SS contained in supplied sludge as high as possible with gravitational thickening.

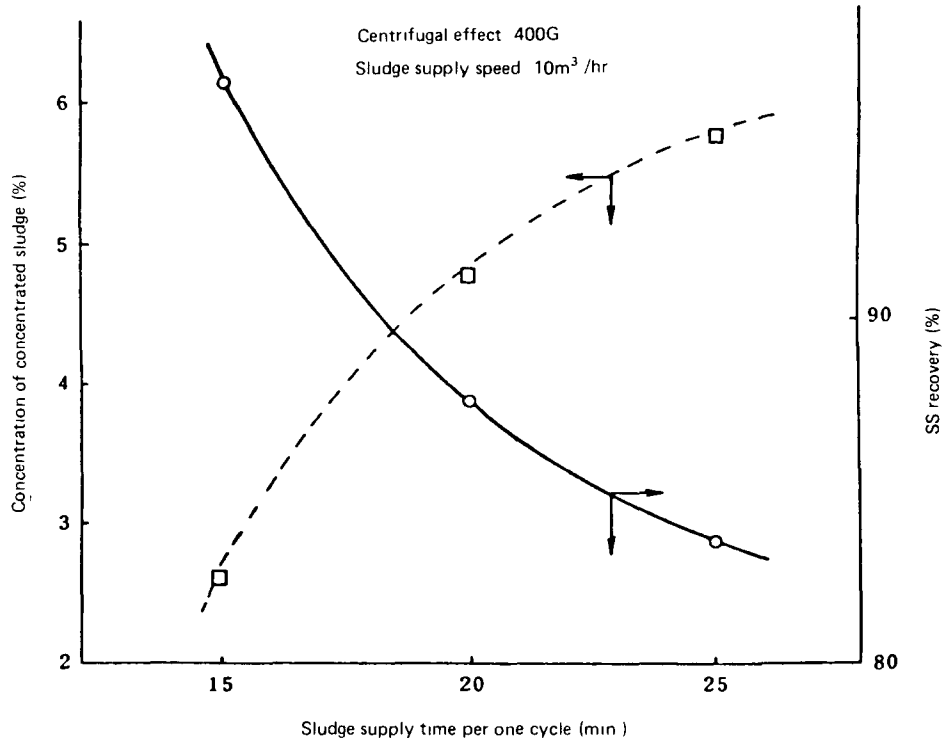


Figure-1 Sludge Supply Time vs Concentration of Concentrated Sludge

Table-1 Result of Concentration Test

Operational conditions				Supplied sludge (in)			Concentrated sludge (out)				Supernatant (out)		
Centrifugal effect	Sludge supply speed		Sludge supply time	Supplied sludge amount	Concentration of SS	Solid amount	Concentrated sludge amount	Concentration of SS	Solid amount	SS recovery	Supernatant amount	Concentration of SS	Solid amount
	Objectives	Results											
(G)	m^3/hr	m^3/hr	min/cycle	m^3/cycle	%	kg/cycle	m^3/cycle	%	kg/cycle	%	m^3/cycle	%	kg/cycle
	5	4.6	60	4.62	1.70	78.9	1.156	56600	65.5	88.2	3.460	0.275	9.6
200	10	8.4	40	5.62	1.70	95.2	1.176	67600	79.7	76.4	4.440	0.502	22.2
	15	11.8	30	5.90	2.00	118.0	1.049	81000	85.0	25.5	4.850	1.59	77.1
300	10	7.9	25	3.30	1.67	54.9	1.113	51900	57.6	81.9	2.190	0.418	9.2
	5	4.4	50	3.69	2.08	76.8	1.005	73100	73.5	86.0	2.690	0.399	10.6
400	10	8.1	25	3.38	2.11	76.0	0.996	57500	57.3	81.7	2.610	0.549	14.4
	15	12.7	20	4.23	2.07	87.7	1.005	64900	65.3	59.5	3.230	1.01	32.6
	5	4.7	40	3.15	2.12	66.8	1.081	52300	56.5	92.3	2.070	0.262	5.5
500	10	8.2	25	3.43	1.69	57.8	1.113	55400	61.7	79.8	2.310	0.441	10.1
	15	13.1	20	4.35	1.69	73.1	1.107	66700	73.9	69.3	3.210	0.635	20.3
	10	8.2	15	2.05	1.86	38.3	1.083	26100	28.3	93.3	0.970	0.35	3.4
400	10	8.2	20	2.73	2.20	60.1	1.086	48800	53.0	88.8	1.640	0.411	6.7
	10	8.6	25	3.57	2.26	80.6	1.092	58000	63.6	86.8	2.470	0.472	11.6

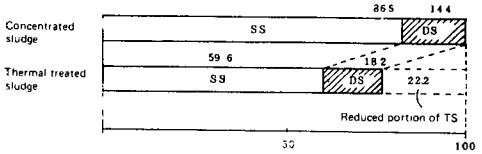
(2) Equipment for Thermal Treatment

a) Steam increased volumes of thermal-treated sludge by 11%. TS contained in sludge decreased by 15 to 20% by thermal treatment. It was estimated that about 30% of SS, about 40% of nitrogen and almost none of phosphorus contained in raw sludge dissolved. (Refer to Table-2 of original) (Table-2,Fig.-2)

b) The decomposition rate of SS was in inverse proportion to the concentration of SS. This means that treatment at high sludge concentration reduces BOD and COD load contained in supernate. The concentration of supplied for thermal treatment must be 5 to 6%. (Fig.-3, Fig.-4)

c) When sludge was treated at 165°C for 60 minutes, Kerosene of 13 to 15 liters per inputted sludge of one cubic meter was consumed to generate steam supplied for the reactor.

d) It was hard to dehydrate sludge when it was treated at 155°C, so reaction temperature must be kept 165°C for satisfactory dehydration.



Remarks
1 Figures shown above are the ones when TS contained in concentrated sludge is regarded as 100
2 DS = TS − SS

Figure-2 SS Balance in Thermal Treatment Equipment

Table-2 Dissolution rates of Phosphorus and Nitrogen

Items \ Run No	1	2	3	4
Phosphorus (%)	0	7.3	0	0
Nitrogen (%)	38.9	45.9	36.8	37.4

Remark
1) $\text{(Dissolution rates of phosphorus)} = \frac{(\text{Sol. phosphorus in thermal treated sludge}) - (\text{Sol. phosphorus in raw sludge})}{(\text{All phosphorus in supplied sludge}) - (\text{Sol. phosphorus in raw sludge})}$
2) Dissolution rates of nitrogen was evaluated by the similar calculation

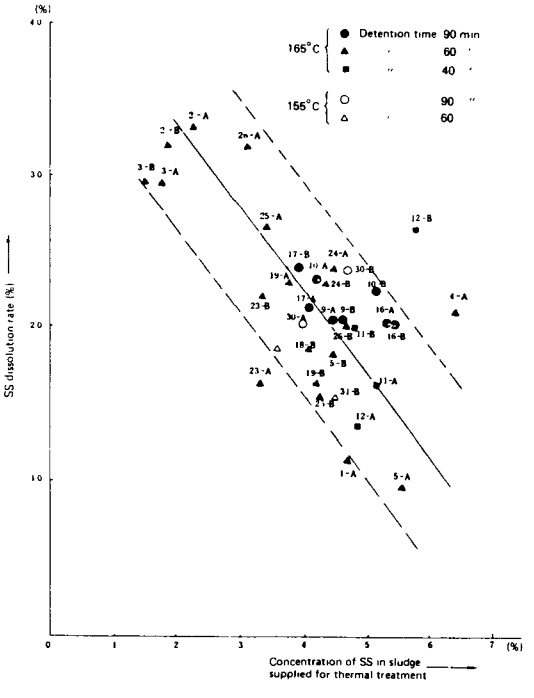


Figure-3 Concentration of SS in Supplied Sludge – SS Dissolution Rate

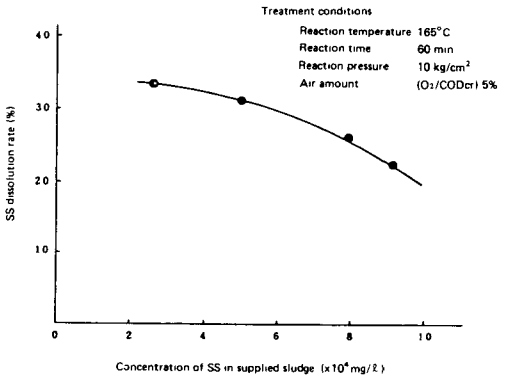


Figure-4 Autoclave Test Results

(3) Dewatering Equipment

Supplied sludge was easy to be dewatered, and the proportion of gross SS to all the SS contained in the sludge was very large. Therefore, dewatered cake with low moisture (30 to 45%) could be obtained by the use of Filter press., when filtration rate was 6 to 10kg/m^2·hr. (Table-3)

(4) Equipment for Melting Furnace

It was favorable for melting treatment that a small quantity of moisture and a large quantity of organic components was contained in supplied cake and melting point of ash content of the cake was low.

a) The amount of kerosene consumed in melting furnace was about 170 liters per inputted solid of one ton under the condition that DS of 20% moisture and 4,000Kcal/kg low calorific power was inputted with rated load.

b) The amount of kerosene consumed in melting furnace depends on inputted amount of dried cake, moisture contained in sludge, and combustion calory. (Fig.-5 to Fig.-8)

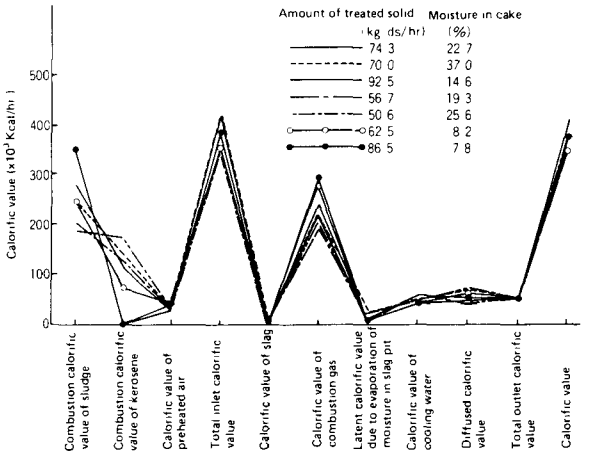


Figure-5 Heat Balance in Melting Furnace

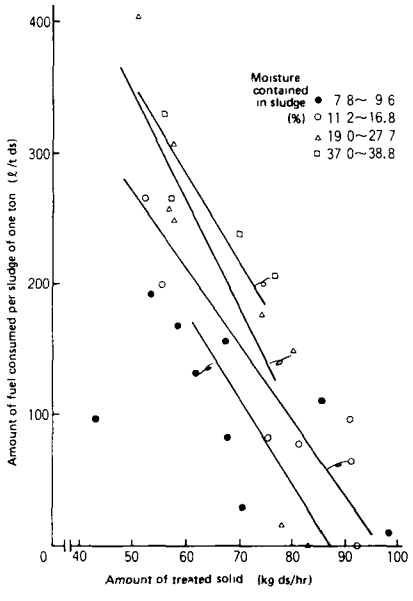


Figure-6 Amount of Treated Sludge vs Amount of Consumed Fuel

Table-3 Thermal-treated Sludge Dewatering Test by the Use of Filter Press

Dewatering conditions		Thermal-treated concentrated sludge						Dehydrated cake			Filtrate from pressure filter			Filtration rate	SS recovery
Sludge supply time (min)	Compression time (min)	Sludge amount (m^3/cycle)	SS (mg/ℓ)	SS amount (kg/cycle)	Gross SS (mg/ℓ)	Vol. Solid Gross SS (%)	Gross SS/SS (%)	Produced amount (kg/cycle)	Moisture rate (%)	Solid amount (kg/cycle)	Drained amount (m^3/cycle)	SS (mg/ℓ)	SS amount (kg/cycle)	(kg/m^2 hr)	(%)
		①	②	③=①×②×10^{-3}				④	⑤	⑥=④×(1−⑤/100)	⑦	⑧	⑨=⑦×⑧×10^{-3}		⑩=⑥/(②+⑨)×100
5	15	0938	47,400	44461	32,400	866	684	81	398	48762	–	–	–	468	–
10	10	2167	54,800	112752	44,100	889	805	172	424	99072	–	–	–	951	–
15	5	2297	51,600	118525	–	–	–	195	443	108615	–	–	–	1043	–
5	15	0938	80,000	75040	53,900	871	674	91	408	50687	1197	982	1156	487	978
10	10	2070	73,700	152559	47,400	870	643	158	419	91798	1694	764	1294	881	986
15	5	2200	69,700	153340	27,200	908	390	196	458	106232	2210	878	1940	1020	982
5	15	1008	78,000	83304	43,500	910	558	103	430	58710	1254	1,120	1404	564	977
10	10	1650	63,200	104280	36,500	912	578	156	442	87048	1650	899	1483	836	983
15	5	2006	65,200	130791	36,100	900	554	189	461	101871	1801	1,040	1873	978	982
Remark		Sludge supply pressure 8kg/cm², Squeezing pressure 15kg/cm² (Both fixed)													

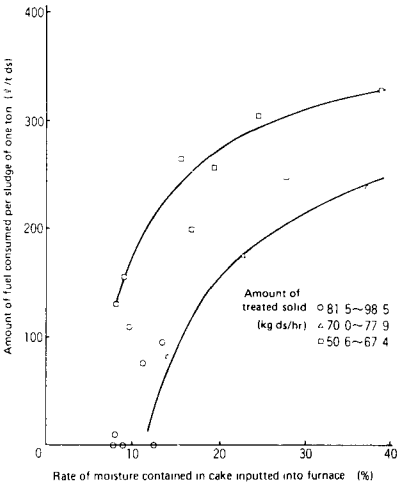


Figure-7 Moisture in Sludge vs Consumed Fuel

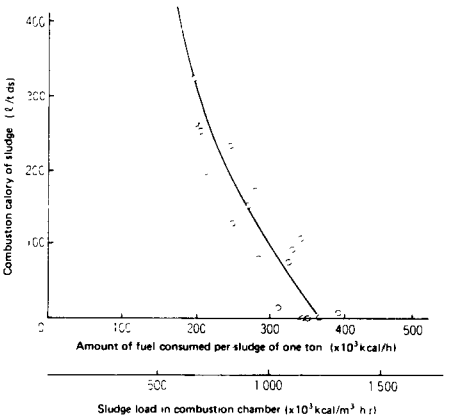


Figure-8 Combustion Calory of Sludge vs Amount of Consumed Fuel

c) Auxiliary fuel for drying was not required because heat recovery was performed from gas exhausted by melting furnace by the use of waste heat boiler. Steam recovered by waste heat boiler was used for drying and heating for heat treatment, which could reduce amount of kerosene consumed during thermal treatment by 30%. It is expected that further efficient heat recovery will lead to more economical running. (Fig.-9)

d) Slag, whose bulk specific gravity and true specific gravity are 1.2 to 1.3 and 2.6 to 2.7, respectively, can be used for reclamation. There are few possibility that the slag scatters. (Table-4, Table-5)

Table-4 Characteristics of Melting Slag

Treatment conditions / Items					
Treatment temperature (°C)		1,400			
Moisture in cake (%)		20	40	20	20
Amount of treated solid (kg-DS/hr)		70	70	90	50
Unburned portion	% [Dry]	0 86	0 66	0 05	0 05
Bulk specific gravity	—	1 29	1 33	1 33	1 17
Apparent specific gravity	—	2 61	2 59	2 63	2 56
True specific gravity	—	2 63	2 66	2 70	2 71

Table-5 Result of Dissolution Test

Items	Dried cake	Melting slag	Supernatant from slag
Unit	Cn, Zn, Fe····· % Others ·· · ····· · mg/kg		mg/ℓ
Cd	3.3	<0.4	<0 02
CN	<0.1	0.3	<0.01
Organic P	<1	<1	<0 01
Pb	110	4.1	<0 2
Cr⁺⁶	<2	<2	<0 02
As	2.4	1.7	<0.005
T-Hg	1.7	0 013	<0 000 5
Alkyl Hg	<0 01	<0 01	<0 000 5
PCB	2 4	<0 1	<0 000 5
Phenol	11	6 4	<0 1
Cu	0 04	0.15	<0.02
Zn	0 22	0 11	0 04
Fe	1 39	6 83	0 27
Mn	180	820	0 11
Cr	82	630	<0 02
Ni	74	260	0 37

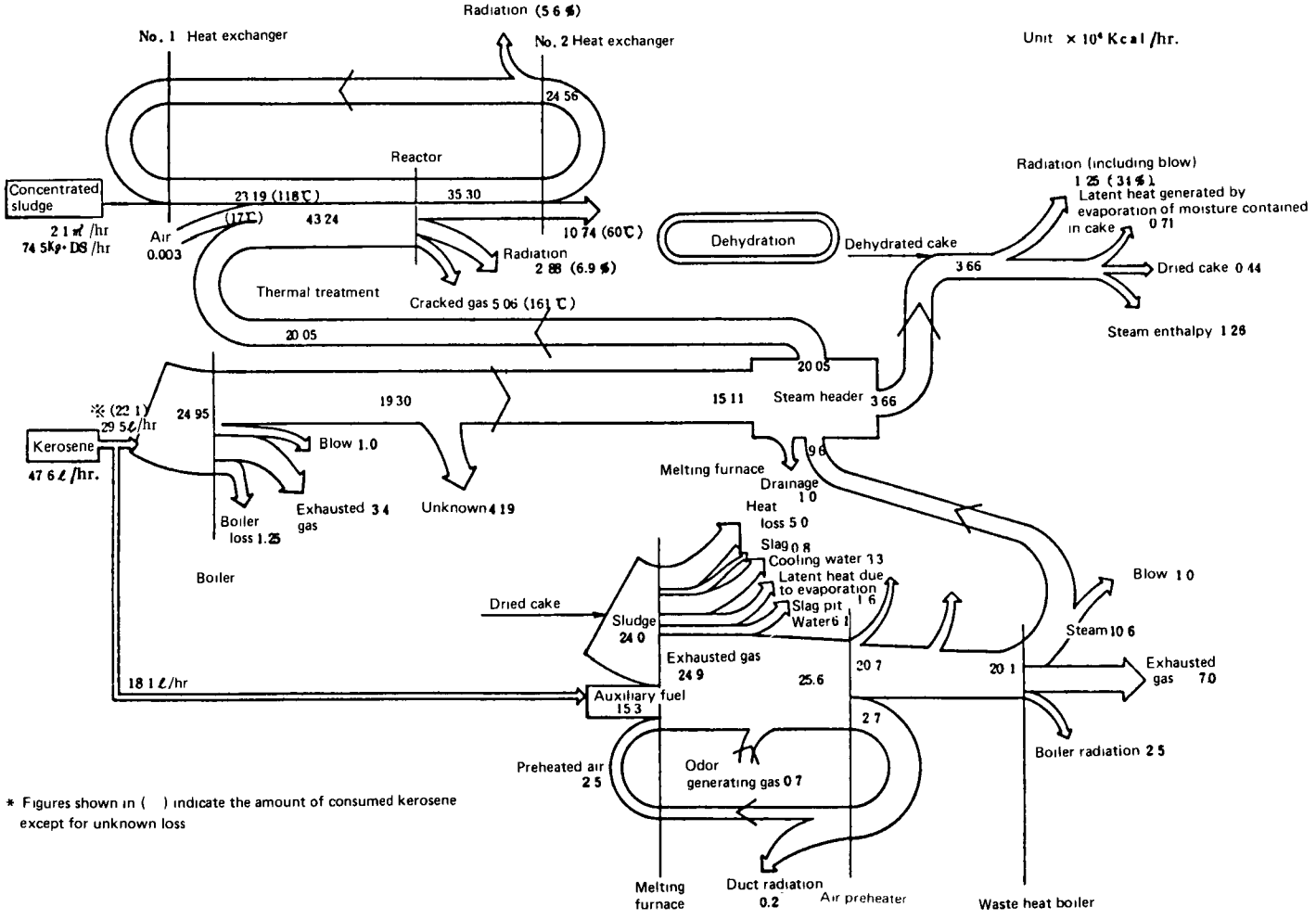


Figure-9 Heat Balance in Total Test through All the Process

Ninth US-Japan Conference
on
Sewage Treatment Technology

Study on Fluidized Bed Incineration with a Drying System

September 19-21, 1983
Tokyo

The work described in this paper was not funded by the U.S. Environmental Protection Agency. The contents do not necessarily reflect the views of the Agency and no official endorsement should be inferred.

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Foreword

This report presents the results of experiments with a pilot incineration plant comprising a fluidized-bed incinerator and a dryer, and describes a working facility built in accordance with those experimental results. Processing the dewatered cake in a dryer before feeding it into the incinerator reduces the amount of exhaust gas, thus lowering fuel consumption and increasing incineration capacity.

By way of introduction, though, a few words are in order on the evolving incineration needs of the City of Yokohama. As of March 1983, about half of the population of the city was served by sewer lines. Aiming to extend sewer service to everyone in Yokohama by the year 2000, the Sewage Works Bureau is currently laying sewer at the rate of 1,000 m per day, as well as building new sewage treatment plants and expanding existing plants. All of the city's plants are set up for secondary treatment, using the activated sludge process, and broadening sewer service is rapidly increasing the amounts of sludge turned out by these plants (*Fig. 1*).

Yokohama's high population density of 67 persons/hectare and the ongoing residentialization of surrounding areas make it virtually impossible to secure new fill sites for disposing of sludge in the city. Sludge should ideally be returned to the natural cycle of decay and renewal, and Yokohama did in fact begin supplying a certain amount of dehydrated cake as fertilizer for green zones and farms in 1977. However, given the impossibility of substantially boosting demand for fertilizer in very-urban Yokohama, the city has decided to incinerate all sludge for the time being, to reduce its volume and thereby extend the useful life of fill sites.

The limited availability of land does not allow for building incinerators at all of the treatment plants, so Yokohama is having to construct central incinerators, each of which is to take dewatered cake from a number of treatment plants. Moreover, these treatment plants, which have been constructed over a span of some twenty years, turn out differing sorts of dewatered cake by a variety of processes, not all of which were originally intended to facilitate incineration.

Incinerators for Yokohama must therefore be both large and capable of operating reliably on a diet of various kinds of dewatered cake. Air-quality safeguards are also absolutely essential, as is deodorization of exhaust gas. Incinerators are furthermore expected to minimize fuel consumption.

From the standpoint of fuel consumption, self-sustaining combustion would of course be the ideal mode of sludge incineration. Unfortunately, thoroughly reworking the entire treatment process at each of Yokohama's historically diverse sewage plants so as to permit this would require huge investment, and would not be economical. The city therefore opted to build new incineration facilities to handle the sludge output of existing sewage treatment plants, and to study means for minimizing fuel consumption by these facilities.

Incinerators can be broadly divided into multiple-hearth furnaces and fluidized bed furnaces, with two characteristic patterns of heat consumption (*Fig. 2*). Multiple-hearth furnaces feature reliable operation and have been used extensively in large-scale facilities, but they are heavy consumers of fuel. This is because the huge amount of air required for combustion translates into a proportionally large volume of exhaust gas, and also because fuel must be used in deodorization.

On the other hand, fluidized bed furnaces require neither a large amount of air for combustion nor deodorization of their exhaust gas, and therefore consume relatively little fuel. Fluidized bed furnaces have yet to see service in many large-scale facilities, but recent progress in micro-computerization and other ancillary technology promises to ensure reliable operation.

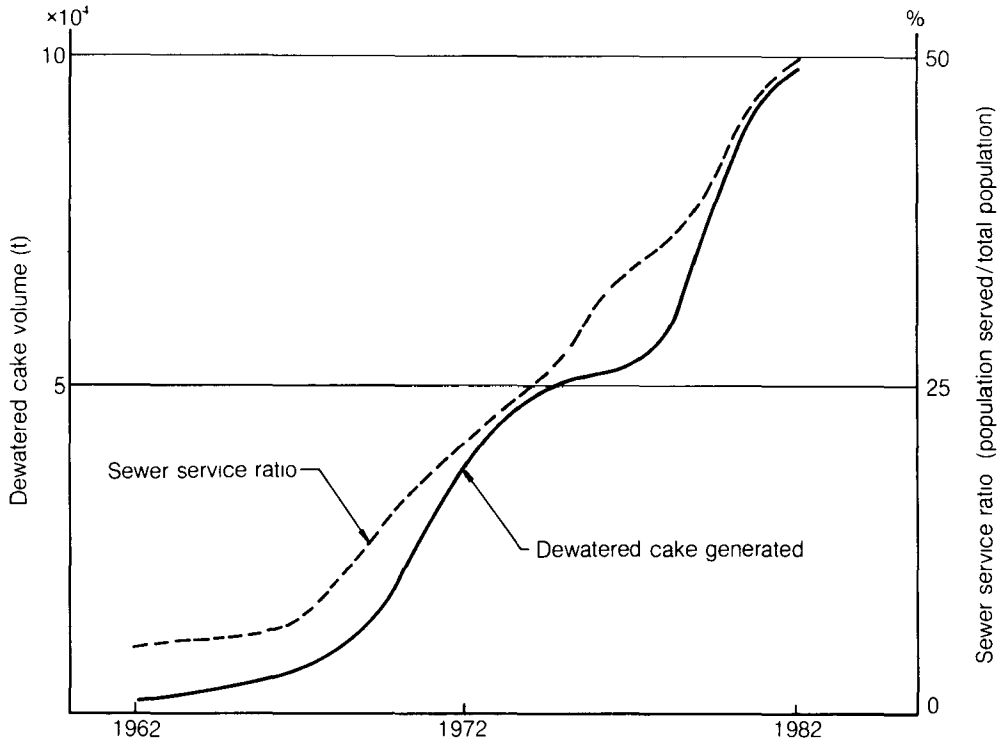
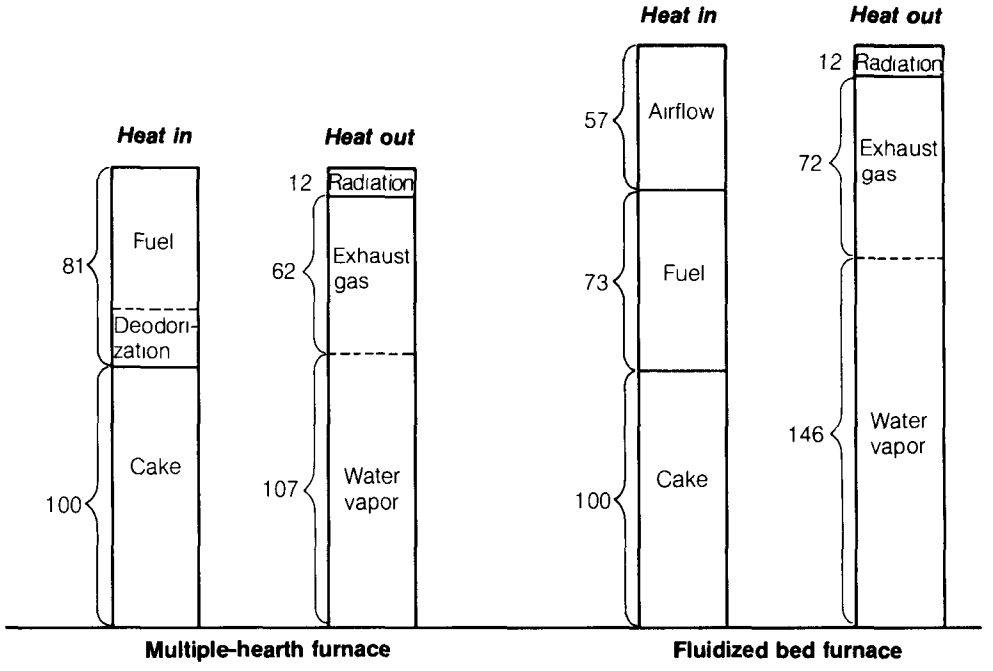


Fig. 1 Service ratio and dewatered cake generation



Note Figures represent percentages of the calorific value of the dewatered cake

Fig. 2 Heat input/output

Drying and incineration are performed as a single process in conventional multiple-hearth furnaces and fluidized bed furnaces, and so the water content vaporized out of the dewatered cake is all heated to the temperature of the incinerator exhaust gas.

In order to avoid having to heat the full water content of the cake in the fluidized bed furnace, systems have been devised in which the dewatered cake is dried in a separate process prior to incineration. Drying the cake beforehand at low temperature (about 100° C) to bring the water content down to the point where the cake will burn by itself reduces the amount of supplementary fuel needed when the cake is then incinerated. And less vapor from the cake and less exhaust from fuel combustion translate into greater incinerating capacity.

Since no such separate-drying system had previously been employed in a full-scale plant, the Sewage Works Bureau of the City of Yokohama installed a pilot system at its Kanazawa Sewage Treatment Plant and conducted testing from June 1980 to May 1982 to confirm system performance and operability.

1. PRELIMINARY EXPERIMENTAL RESULTS

Before the pilot system study got underway, preliminary experiments were done to determine the optimum type of dryer. Calculations indicated that drying by indirect heat would offer better fuel economy and less exhaust gas than drying by direct-heat, so dryers of the former kind were used for the preliminary experiments.

Since different sewage treatment plants turn out cake of differing water content, calorific value, and physical composition, the primary object of the preliminary experiments was to select the dryer featuring the most consistently reliable operation with all kinds of cake. Experiments to this end were carried out with steam-tube dryers, high-speed paddle-mixing dryers, and a low-speed disk-mixing dryer (*Table 1*). The low-speed disk-mixing dryer performed best in terms of the above-mentioned selection criterion, and was therefore chosen for the pilot plant testing. (Details of the preliminary experiments are described in Chapter 3, "Experiments on Separately Dried Sludge Incineration," of the report, *New Aspects of Sludge Incineration in Yokohama,* submitted at the Eighth US-Japan Conference on Sewage Treatment Technology.)

Table 1. Summary of preliminary experiments

Dryer	Feed process		Sticking	Stability	Operation	Controllability of dried cake water content	Evaporation rate kg $H_2O/m^2 h$
Steam tube heating type	Direct feed		Unable to operate	—	—	—	—
	Feed-back process	Mixer	No	Poor Dried cake grains gradually expand	Poor Many accessories are required	Although controllable by changing the dried material feed back ratio, it is hard to operate practically	—
		Grinder mixer	No	Good	Poor Many accessories are required	Although controllable by changing the dried material feed back ratio it is hard to operate practically	4-6
High speed paddle mixing type	Direct feed	Paddle angle 0°C	No	Poor Cake sticking at inlet causes over-drying at outlet (25% or less)	Poor Motor load sometimes fluctuates to point of overload	—	—
		Paddle angle 30°C	No	Poor Operation is stabilized with a narrow range at 300 rpm rotating speed and 150 kg/h feed rate	Good Poor when feed rate is varied	—	14-17
Low-speed disk-mixing type	Direct feed		No	Good	Good	Correlation cannot be obtained (at 20-40rpm rotating speed and 2-7 kg/cm²G steam pressure)	20-30

2. PILOT PLANT STUDY

2.1. OUTLINE

The pilot plant consisted of the low-speed disk-mixing dryer selected in the preliminary experiments and a fluidized bed furnace. Its sludge capacity was 10 t/day.

2.1.1. Equipment

A flowchart and specifications for the pilot plant equipment appear in *Fig. 3*.

Dewatered cake from the hopper is metered into the dryer by a weight sensitive conveyor. After drying, the cake is then fed into the furnace. Exhaust gas from the dryer is purged of moisture in a cooling condenser and then recycled for use as carrier gas for steam generated in the dryer and as fluidizing airflow for the furnace.

Ash is separated out of exhaust gas from the furnace in the cyclone, after which the gas is fed into the waste-heat boiler. There it is dehumidified and desulfurized in an absorptive cooling tower and then released into the atmosphere.

Steam from the waste-heat boiler and supplementary boiler is stored in a steam header for use as the heat source for drying. The waste-heat boiler was initially equipped with an auxiliary burner, but when it proved too difficult to adjust the pressure inside the furnace the auxiliary burner was abandoned in favor of a separate, supplementary boiler.

2.1.2 Dryer testing

The following experiments were carried out to investigate the performance of different methods of dryer operation:

a. Water content reduction

It was assumed that dryer disk r.p.m., steam pressure, drying time, and dryer exhaust treatment would affect the water content of the dewatered cake. Optimum values for these variables were therefore sought and dryer controllability investigated.

b. Energy efficiency

In order to quantify the energy efficiency of respective methods, the rate of water content vaporization per unit of dryer heating surface was investigated in conjunction with electrical consumption.

c. Dried cake

The effects of different percentages of dried cake water content on the handling and other properties of the cake were studied.

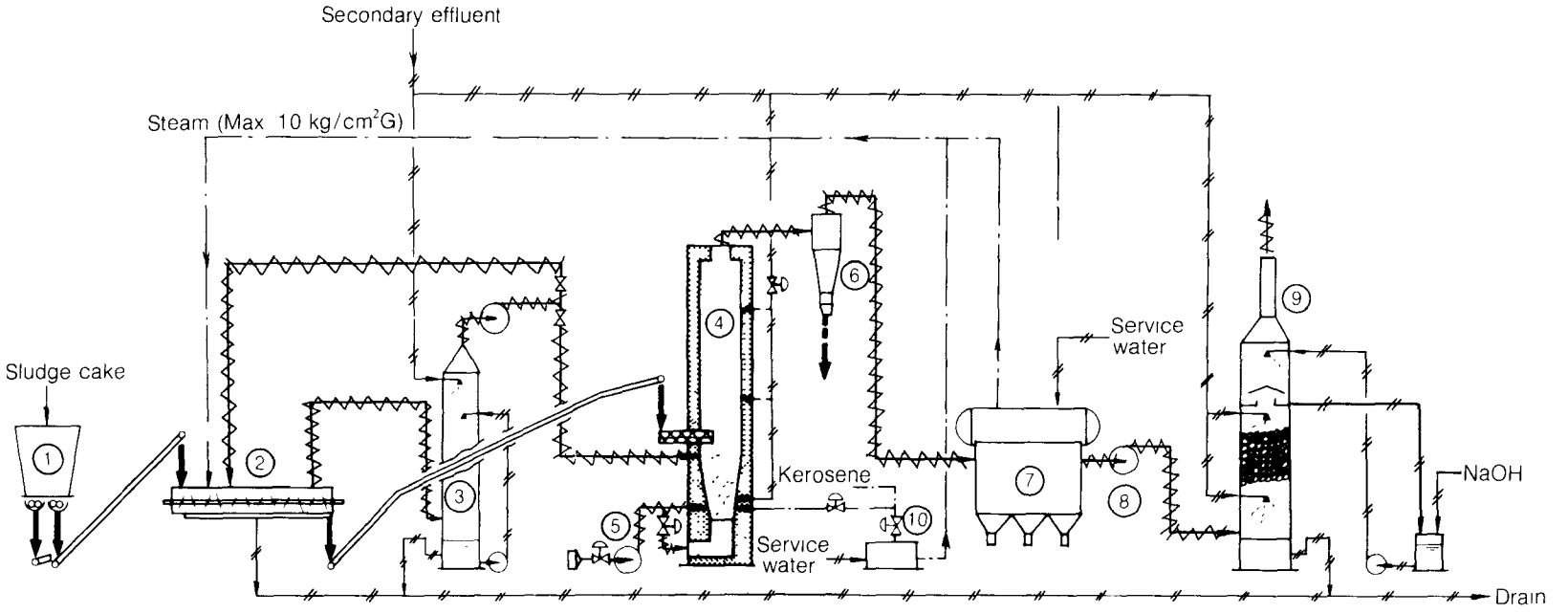
d. Dryer material

Test pieces of SUS304 and SUS316 were mounted at the dryer casing inlet and outlet. Corrosion of these pieces was checked by penetrative flaw detection and by microscope.

2.1.3. Incinerator testing

a. The maximum water content for self-sustaining combustion by dried cake in the furnace was determined.

b. Differences in combustion characteristics and heat distribution inside the furnace between when dried cake is incinerated alone and when dewatered sludge (not yet dried in the dryer) and dewatered cake are incinerated together were investigated.



No	1	2	3	4	5	6	7	8	9	10
Unit	Cake hopper	Dryer	Condenser	Incinerator	Fluidizing air blower	Cyclone	Waste-heat boiler	Induced draft fan	Scrubber	Boiler
Model	Cubical type	Low-speed disk-mixing type	3-stage spraying direct cooling type	Fluidized bed furnace	Multiple stage turbo-blower	Refractory castable lining	Water tube type	Plate type	Upper part Spray type Lower part Packed tower	Single pass type
Main dimensions	2,350mm(W)x3,000 mm(L) x 2,150(H)	Heating surface area 10m²	760mm(dia) x 6,000mm(H)	1,100/1,600mm (dia) x 9,200mm(H)	125mm(dia)	980mm (dia) x 4,300mm(H)	Heating surface area 29m²	250/200mm(dia)	750mm(dia) x 15,000mm(H)	Heating surface area 5.8m²
Volume	mm(H) 12m³	10 t/day	12m³/min at 100°C	Dewatered cake 10t/day	10m³/min x 3,500mmAq	40m³/min at 800°C	194 kg/h x 10 kg/cm²G	30m³/min x 450mmAq	20m³/min at 250°C	350kg/h x 10 kg/cm²G

Fig. 3 Flowchart and equipment specifications

2.1.4. Waste-heat boiler testing

a. Waste-heat boiler performance was investigated in terms of heat efficiency, coefficient of overall heat transfer, and amount of steam generation.

b. Test pieces of SUS304, SUS310S, SUS316, and STPG38 (calorized) were mounted in the boiler water pipe and furnace outlet duct to gauge the effects of sulfur oxides, HCl, and other corrosive gases thought to occur in the furnace exhaust gas. Test piece corrosion was checked, as before, by penetrative flaw detection and by microscope.

2.1.5. Integrated operation testing

The integrated operation of the dryer, furnace, and waste-heat boiler was tested in relation to the following items:

a. Total heat input and output

b. Fuel consumption

Fuel consumption was compared between the conventional and separate-drying systems for each type of sludge.

c. Composition of exhaust gas, drain and furnace ash

2.2 RESULTS

2.2.1 Characteristics of dewatered cake

Six types of dewatered cake, from four sewage treatment plants, were used in the pilot plant testing (*Table 2*). It can be seen from *Table 2* that water content, VTS, net calorific value, and other characteristics of the dewatered cake vary greatly with the type of dewatering machine, the coagulant used, and whether or not an anaerobic digestion process is used.

2.2.2. Dryer

a. Water content reduction

Experiments revealed the following as factors affecting the water content of dried cake:

i. Dryer disk r.p.m. (*Fig. 4*)

As is clear from *Fig. 4*, varying the speed of disk rotation has no effect on the water content of the dried cake up to 15 r.p.m., but begins to increase the water content at higher speeds. At 10 r.p.m., however, the cake turns together with the disk shaft, and since it is not churned it does not heat evenly. Accordingly, 15 r.p.m. would seem to be the optimum speed for disk rotation.

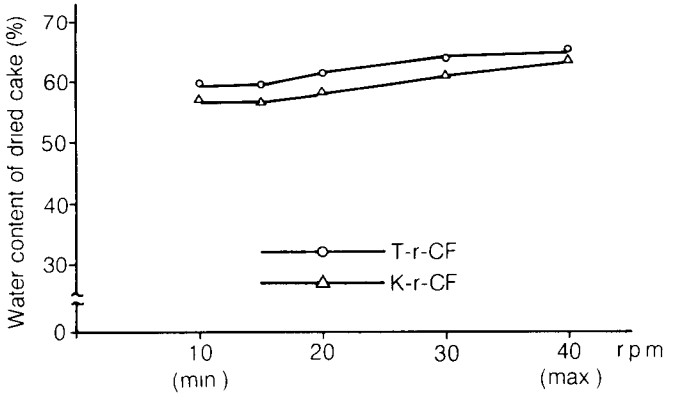


Fig. 4 Dryer disk r.p.m. and water content of dried cake

Table 2. Dewatered cake characteristics

Sewage treatment plant	Chubu S T P (C)	Totsuka II S T P (T)	
Treatment	Thickening and anaerobic digestion	Thickening (r)	Thickening (r)
Dewatering machine	Belt filter (BF)	Pressure filter (PF)	Centrifuge (CF)
Coagulent	$Ca(OH)_2$ and Fe_2Cl_3	$Ca(OH)_2$ and Fe_2Cl_3	Polymer
Sludge type abbreviation	C-d-BF	T-r-PF	T-r-CF
Water content (%)	78 (77~79)	67 (61~71)	80 (77~82)
VTS (%)	41 (39~43)	44 (40~45)	73 (68~78)
Net calorific value (kcal/kgDS)	2,000 (1,900~2,200)	2,200 (1,800~2,500)	3,700 (3,300~4,100)

Sewage treatment plant	Kanagawa S T P (K)	Nambu S T P (N)	
Treatment	Thickening (r)	Thickening (r)	Thickening and anaerobic digestion(d)
Dewatering machine	Centrifuge (CF)	Centrifuge (CF)	Centrifuge (CF)
Coagulent	Polymer	Polymer	Polymer
Sludge type abbreviation	K-r-CF	N-r-CF	N-d-CF
Water content (%)	77 (74~80)	78 (75~80)	74 (70~77)
VTS (%)	57 (53~64)	66 (60~73)	47 (43~53)
Net calorific value (kcal/kgDS)	3,200 (3,000~3,500)	3,800 (3,500~4,000)	2,600 (2,400~2,900)

Note Figures indicate average values, the overall ranges being given in parentheses

ii. Steam pressure

The temperature of the steam rises in proportion to its pressure, thus increasing the amount of heat transferred to the sludge and lowering the water content of the dried cake. But cake will bake onto the heating surfaces if they are too hot, and this will diminish the amount of heat transferred and raise the water content of the dried cake.

Varying the steam pressure between 5 and 9 kg/cm^2G produced no change in the water content of the dried cake (*Fig. 5*). However, a thin film of cake sometimes baked onto the dryer shaft and disk when cake having a raw protein content of more than 25 percent was dried at a steam pressure of 8 kg/cm^2G. It was therefore concluded that a steam pressure of 5 kg/cm^2G would be best, since this will ensure reliable operation even with dewatered cake that is high in raw protein content.

iii. Drying time

Increasing the drying time increases the amount of heat transferred to the sludge and thereby lowers the water content of the dried cake. Drying time is varied by adjusting the angle of the dryer and the height of the outlet weirs.

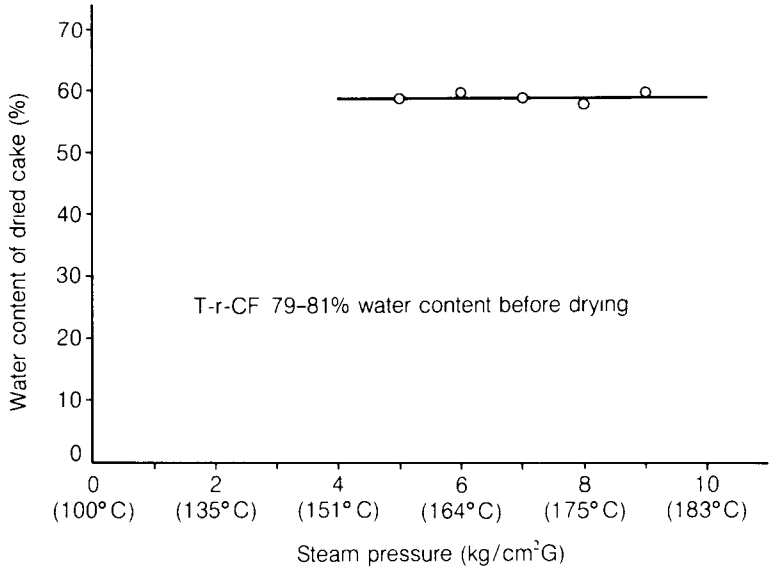


Fig. 5 Steam pressure and water content of dried cake

Dryer angle

The dryer itself must be set at an angle to make the cake flow. Too slight an angle will not produce any flow at all, while an excessive angle will move the cake through the dryer too fast for proper drying. The experiments conducted in regard to dryer angle are summarized below:

0/100 angle:	Dewatered cake having a water content of less than 75 percent did not flow at all, collecting instead at the dryer inlet.
0.75/100:	All types of dewatered cake flowed smoothly, regardless of water content.
1/100 or greater:	Dewatered cake containing more than 80 percent water liquefied when heated and flowed out along the bottom of the casting too fast for proper drying.

Height of outlet weirs

Increasing the height of the outlet weirs (*Fig. 6*) increases the holding capacity of the dryer, thus lengthening the drying time. Accordingly, the water content of the dried cake decreases as weir height increases (*Fig. 7*).

iv. Amount and treatment of exhaust gas from dryer

A proper amount of carrier gas must be used in dryer operation to carry away steam generated from the cake. Insufficient carrier gas will unnecessarily limit vaporization, while too much will increase the volume of dryer exhaust gas to the point of complicating treatment work. Nonetheless, experiments revealed that larger volumes of exhaust gas were accompanied by lower water content in the dried cake (*Fig. 8*).

Experiments also indicated that the water content of the dried cake was 3-5 percent higher when all of the exhaust gas was recycled than when it was all released into the atmosphere (*Table 3*). This is because carrier gas that has cooled somewhat in the course of recycling allows some of the steam from the cake to condense back out onto the cake.

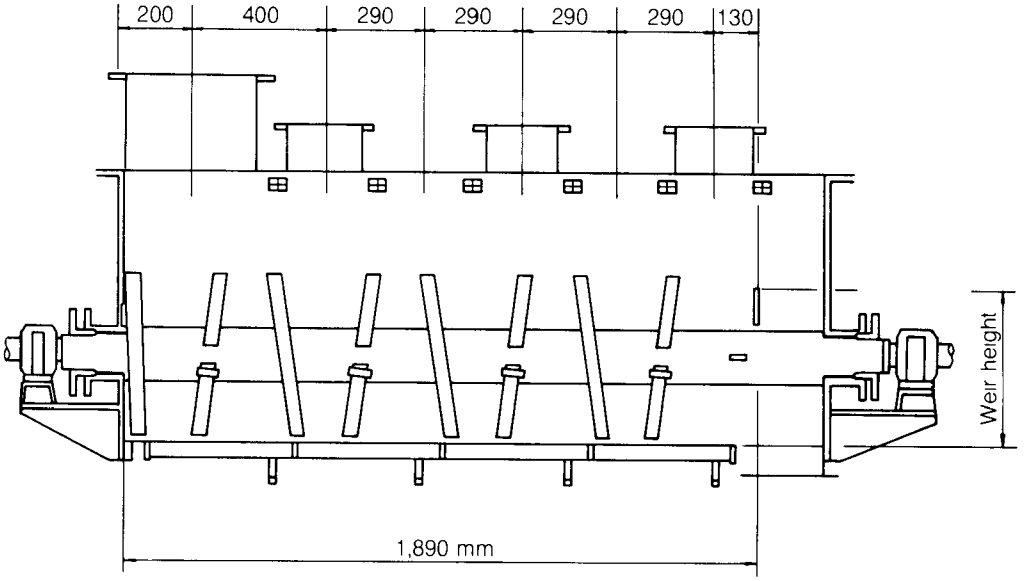


Fig. 6 Dryer weirs

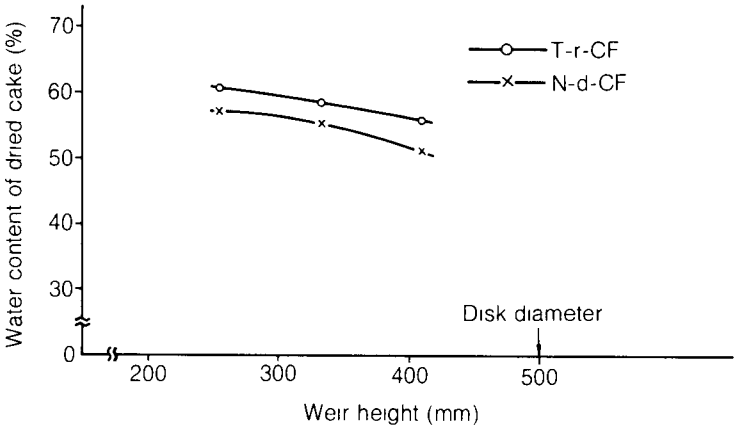


Fig. 7 Weir height and water content of dried cake

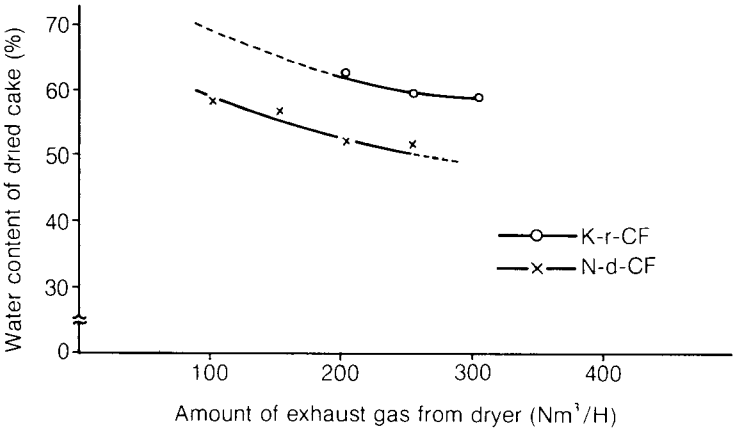


Fig. 8 Amount of exhaust gas and water content of dried cake

Table 3. Exhaust gas treatment and water content of dried cake

	All released into atmosphere	All recycled	
Amount of exhaust gas (Nm^3/H)	170~180	170~180	170~180
Temperature of recycled carrier gas at inlet (°C)	—	30	40
Water content of dried cake (%WB)	55	58	62

Note Experiments were conducted using N-r-CF raw cake fed at the rate of 420 kg/h

b. Performance

i. Dewatered cake type and water vaporization rate

The formula below was used to obtain the rate of water vaporization per unit of heating surface:

$$R = \frac{F \times \{[W/(100-W)] - [W'/(100-W')]\}}{A} \times \frac{100-W}{100}$$

where R is the rate of vaporization (kg/m^2h),
F is the sludge feed rate (kg/h),
W is the water content of dewatered cake (%),
W' is the water content of dried cake (%), and
A is the heating surface area.

The rate of vaporization thus varies according to the type of dewatered cake and the water content to which the cake is dried. *Table 4* shows the relationship between dried cake water content and vaporization rate for different sludge feed rates. The vaporization rate averaged 16 kg/m^2h when cake was dried to a water content of 45-53 percent, and 20 kg/m^2h when the water content of the cake was brought down to 51-62%.

Table 4. Dewatered cake type and water vaporization speed

Cake volume	280-330 kg/h		380-440 kg/h	
Cake type	Water content of dried cake (%)	Vaporization speed (kg/m^2h)	Water content of dried cake (%)	Vaporization speed (kg/m^2h)
C-d-BF	45~52	17 (15~18)	55~61	19 (18~20)
T-r-PF	—	—	55~63	17 (15~18
T-r-CF	46~53	16 (15~19)	55~62	21 (20~22)
K-r-CF	45~53	16 (15~16)	55~62	9 (17~22)
N-r-CF	48~53	17 (16~20)	56~63	20 (17~24)
N-d-CF	45~52	16 (15~17)	55~59	20 (18~23)

Note Figures represent average values, the overall ranges being given in parentheses

ii. Dewatered sludge type and electrical consumption

Different types of dewatered sludge require different levels of power to churn and therefore impose differing loads on the dryer's disk shaft drive motor (*Table 5*). PF cake, being comparatively dry and stiff, requires a particularly large amount of electric power consumption.

Table 5. Electric power consumption

Dewatered cake type	Power consumption
C-d-BF	4 ~ 5
T-r-PF	5 ~ 6
T-r-CF	4 ~ 5
K-r-CF	4 ~ 5
N-r-CF	3 ~ 4
N-d-CF	4 ~ 5

c. Dried cake

Comparison of dried cake and dewatered sludge composition revealed little difference in physical make-up other than water content (*Table 6*). As for the form of the dried cake at different levels of water content, it was found that dried cake consisting of at least 60 percent water becomes a very sticky, amorphous mass. At a water content of around 55 percent, the cake forms into lumps averaging 80 mm in diameter and ranging as large as 200 mm diameter. At a water content of no more than 50 percent, CF cake forms into lumps as large as 100 mm in diameter, while BF cake granulizes into pieces no larger than 50 mm in diameter.

d. Dryer material

Test pieces of SUS304 that were mounted on the dryer casing evidenced corrosive pitting over their entire surface area. Similarly mounted test pieces of SUS316 suffered corrosion only at welded points, where minor corroded depressions occurred.

2.2.3. Incinerator

a. Maximal water content for self-sustaining combustion

Fig. 9 represents the correlation between the percentage of water content of dried cake fed into the fluidized bed furnace and the amount of kerosene that had to be burned as supplementary fuel. It can be seen that cake with a water content of 52–60 percent required no more than 7 liters of kerosene per ton of cake to sustain combustion. This may therefore be regarded as essentially the upper range of water content for nearly self-sustaining combustion.

b. Combustion

When dewatered cake was incinerated alone, vigorous combustion occured immediately above the layer of sand (*Fig. 10*). The temperature inside the furnace was highest at mid-freeboard, with heat discharge from the furnace walls keeping the temperature down at points higher. When dried cake having a water content of 54 percent was incinerated alone, combustion began inside the sand layer, showing mounting intensity up to just above the sand. The temperature inside the furnace was highest at lower freeboard.

Table 6. Dried cake characteristics

Dewatered cake type		C-d-BF		T-r-CF		T-r-PF		K-r-CF		N-r-CF		N-d-CF	
		Before drying	After drying	Before drying	After drying	Before drying	After drying	Before drying	After drying	Before drying	After drying	Before drying	After drying
Water content (%)		77 5	59 4	79 0	55 0	71 0	48 1	77 1	57 8	80 0	49 4	76 5	51 1
VTS (%)		39 4	38 7	77 0	78 0	45 0	39 8	54 7	55 1	66 6	70 1	50 5	51 4
Net calorific value (kcal/kgDS)		1,940	1,950	4,100	4,000	2,500	2,310	3,120	3,260	3,760	3,840	2,780	2,810
Elemental analysis	C (%)	22 4	23 0	40 0	39 0	26 2	26 6	32 1	32 7	32 7	35 0	23 5	23 9
	H (%)	4 0	3 6	7 0	7 0	4 3	5 2	5 1	5 2	5 5	5 8	4 5	4 8
	N (%)	2 6	2 6	4 0	4 0	2 6	2 7	4 1	4 4	4 0	4 5	3 5	3 8
	O (%)	10 0	9 3	25 4	27 4	11 9	5 3	12 1	11 5	23 5	23 9	17 9	17 7
	Combustible S (%)	0 4	0 2	0 6	0 6	0	0	1 3	1 3	0 9	0 9	1 1	1 2
	Noncombustible S (%)	1 0	1 1	0 1	0 1	0 5	0 5	0 1	0 1	0 2	0 1	0 3	0 3
	Cl (%)	0	0	0 26	0 14	0 3	0 4	2 0	1 0	0 7	0 7	0 1	0 1
Raw fiber (%)		—	—	28 0	26 0	—	—	7 4	5 6	—	—	9 2	8 5
Raw protein (%)		—	—	25 0	27 0	—	—	25 7	27 5	—	—	22 0	23 6
Raw fat (%)		—	—	12 0	12 0	—	—	10 4	10 5	—	—	4 9	5 1

Incinerating dried cake having a water content of 46-53 percent together with dewatered sludge having a water content of 77-81 percent resulted in stable combustion extending from the sand up through the lower freeboard. The two types of cake were fed into the furnace by separate conveyors and no special effort was made to mix them together.

When dried cake having a water content of 5 percent was incinerated alone, intense combustion occurred inside the sand layer. The temperature exceeded 1,000° C in the upper part of the sand layer and it was impossible to maintain stable furnace operation.

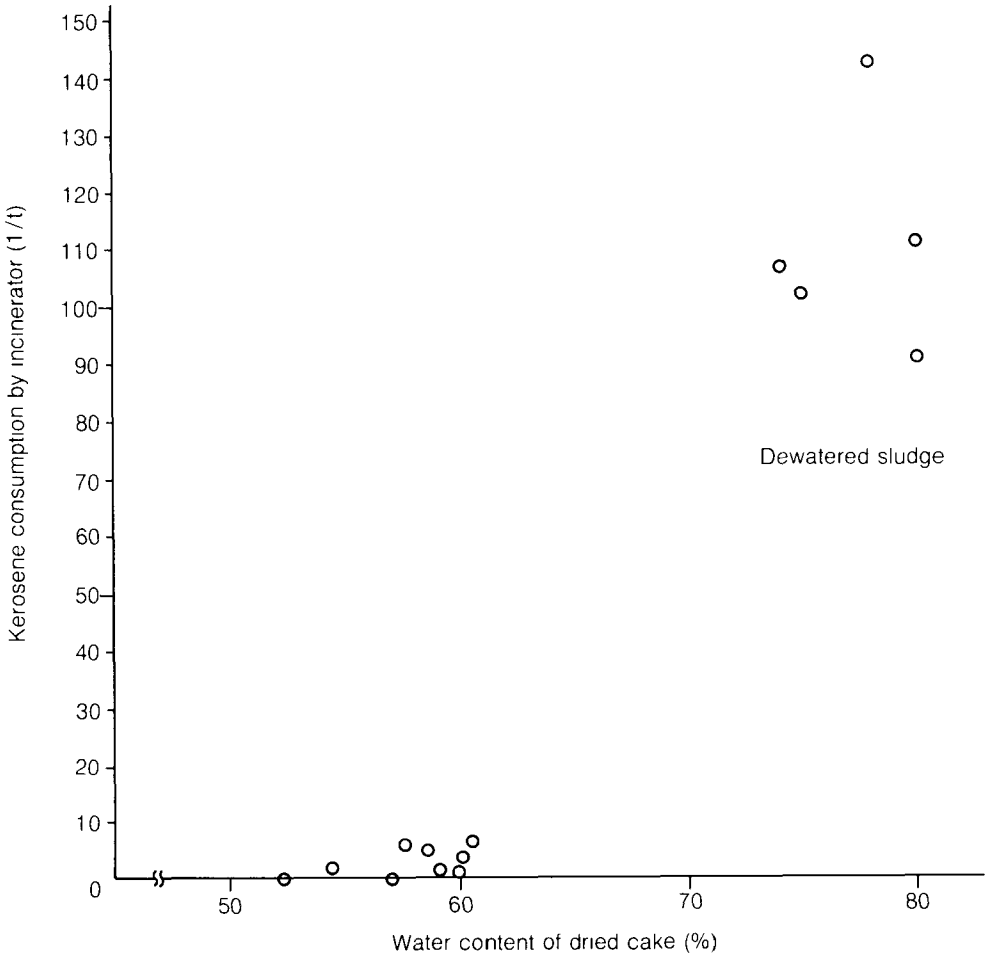


Fig. 9 Water content of dried cake and kerosene consumption

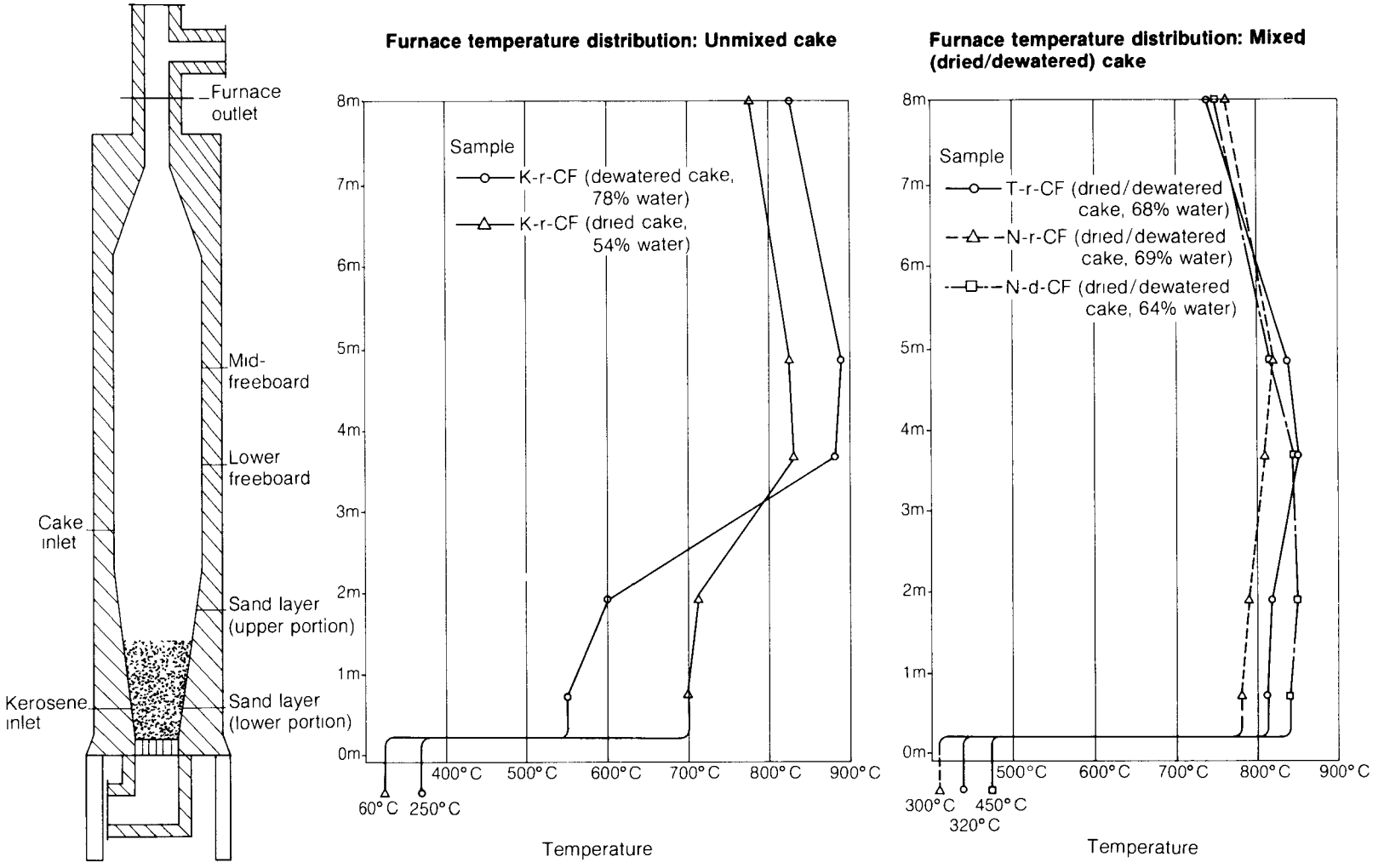


Fig. 10 Furnace combustion

2.2.4. Waste-heat boiler

a. Performance

The temperature differential measured between the exhaust gas inlet and outlet in actual operation of the waste-heat boiler was quite a bit smaller than anticipated at the design stage, as was the amount of steam generated (*Table 7*). This is because, for one thing, heat loss from the ducts was greater than expected and the temperature of the exhaust gas at the inlet to the waste-heat boiler therefore that much lower. For another thing, blowing the water pipes in the boiler clear of accumulated soot only once every 5–6 hours diminished the efficiency of heat transfer.

Soot accumulation in the pilot-system boiler allowed the outlet temperature of the exhaust gas to rise at the rate of about 10° C/h. This temperature dropped some 50° C each time the water pipes were blown clear of soot (*Fig. 11*).

Meanwhile, incorporation of an auxiliary burner lowered boiler efficiency by increasing its overall size and thereby permitting more heat loss through radiation.

Table 7. Waste-heat boiler performance

	Design conditions	Actual performance
Exhaust gas volume		
Dry gas (Nm^3/h)	400	700
Water content (Nm^3/h)	230	200
Temperature		
Inlet temperature (° C)	800	635
Outlet temperature (° C)	250	290
Efficiency (%)	95	86
Coefficient of overall heat transfer ($kcal/m^2h°C$)	24	21
Steam generation (kg/h)	190 at 10 kg/cm^2G max	150 at 9 kg/cm^2G normal

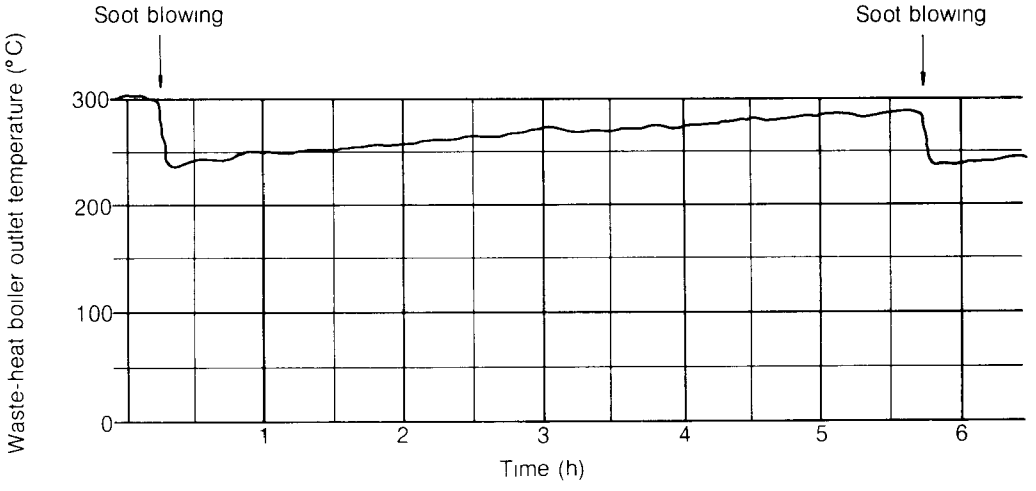


Fig. 11 Soot blowing and waste-heat boiler outlet temperature

b. Corrosion

Test pieces of SUS mounted in the waste-heat boiler showed no abnormality, but those of STPG38 corroded over their entire surface. Calorization proved ineffective in preventing corrosion. Water pipes (STPG38) in the test system also suffered corrosion, this being particularly severe at the boiler outlet.

2.2.5. Integrated operation

For integrated operation, an auxiliary boiler was operated in addition to the waste-heat boiler in order to supply enough steam to the dryer to ensure self-sustaining combustion of the cake in the furnace. For the sake of comparison, experimenters also carried out incineration by the conventional method (i.e., burning dewatered cake alone in the fluidized bed furnace without running it first through the dryer). While the system using separate-drying and incineration was able to process 10 tons of dewatered sludge per day, the system in which dewatered sludge was fed directly into the furnace could handle only 4.1 tons per day.

a. Heat input and output

Table 8 presents a detailed accounting of heat input and output. *Fig. 12* groups component inputs/outputs by category to graphically illustrate the heat balance, while *Fig. 13* is a heat flowchart. It will be noted in *Table 8* that radiation from the furnace accounted for only 17 percent of total heat output in the system employing separate-drying and incineration, but represented 32 percent of the total in the conventional system. Also significant is the relatively large, 9.5 percent share for steam drain residual heat in the overall heat output of the separate-drying system.

b. Fuel consumption

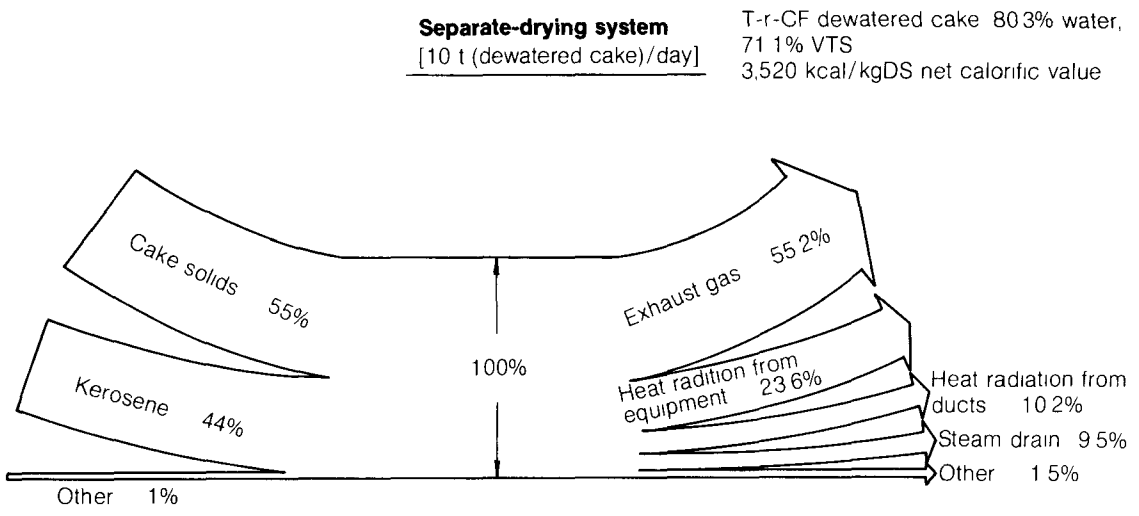
Table 9 gives the fuel consumption rates for the two systems with different types of cake. These rates relate to fuel burned by the auxiliary boiler in the case of the separate-drying system and, in the case of the conventional system, to supplementary fuel burned in the furnace. The separate-drying system consumed around 60 percent as much kerosene per ton of dewatered cake as the conventional system, regardless of cake type.

c. Exhaust gas, drain and furnace ash

The findings of analysis of exhaust gas and wastewater can be found in *Fig. 14*. Exhaust gas ultimately released into the atmosphere from the furnace after reduction processing showed an odor index of 920 (as determined through triangle testing). There were substantial amounts of sulfur oxides and HCl in the exhaust gas from the dryer, and even still in that from the condenser, so water-scrubbing will not always be sufficient in itself to absorb all the acidic gases out of the dryer exhaust.

Table 8. Heat input and output

	Separate-drying			Conventional			
Dewatered cake	10 0 t/day			4 1 t/day			
	Heat in			***Heat in***			
	Cake calorification	290,000	kcal/h (55 0%)	Cake calorification	117,000	kcal/h	(42 5%)
	Kerosene calorification	232,000	(44 0)	Kerosene calorification	158,000		(57 5)
	Heating effect of fluidizing air blower	5,000	(1 0)				
	Total	527,000	(100 0)	Total	275,000		(100 0)
	Heat out			***Heat out***			
	Exhaust gas from dryer	155,000	kcal/h (29 4%)	Exhaust gas from preheater	136,000	kcal/h	(49 4%)
	Exhaust gas from waste-heat boiler	109,000	(20 7)	Exhaust gas from furnace	88,000		(32 0)
	Exhaust gas from auxiliary boiler	27,000	(5 1)	Heat radiation from preheater	12,000		(4 4)
	Heat radiation from dryer	12,000	(2 3)	Heat radiation from ducts	18,000		(6 5)
	Heat radiation from waste-heat boiler	10,000	(1 9)	Heat radiation from preheated air lines	20,000		(7 3)
	Heat radiation from auxiliary boiler	14,000	(2 7)	Residual heat in furnace ash	1,000		(0 4)
	Heat radiation from ducts	39,000	(7 4)				
	Heat radition from steam lines	15 000	(2 8)				
	Residual heat in steam drain	50,000	(9 5)				
	Cooling of dried cake	5,000	(0 9)				
	Residual heat in furnace ash	3,000	(0 6)				
	Total	527,000	(100 0)	Total	275,000		(100 0)
Heat output per ton of dewatered cake	61,265,000 kcal/t			1,610,000 kcal/t			
Notes	T-r-CF dewatered cake 80 3% water, 71 1% VTS, 3,520 kcal/kgDS net calorific value, 52 6% max water content (dried) for self-sustaining combustion						



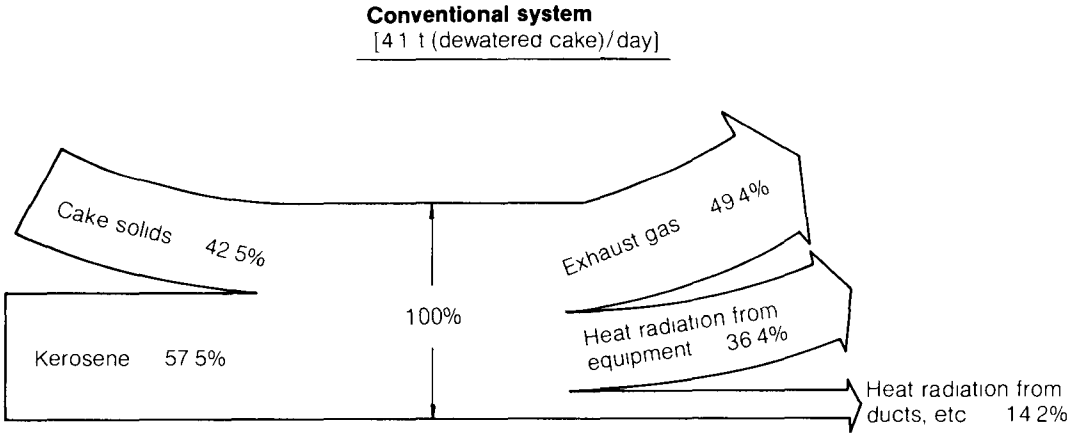


Fig. 12 Heat input and output

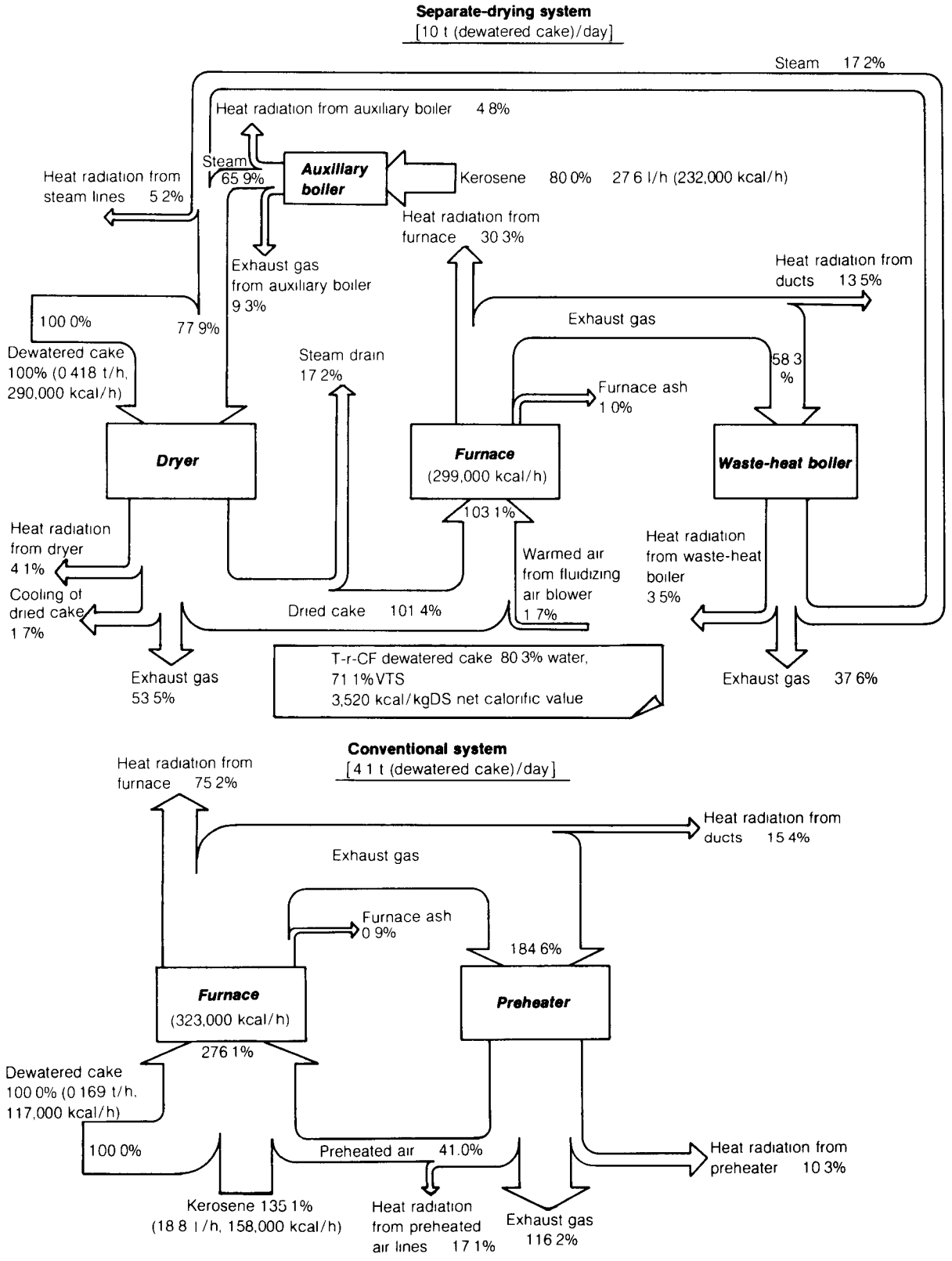


Fig. 13 Heat flow

Table 9. Incineration method and fuel consumption

Dewatered cake		Separate-drying			Conventional			$\frac{A}{B} \times 100$ [%]
		Dewatered cake (t/day)	Kerosene consumption		Dewatered cake (t/day)	Kerosene consumption		
			l/h	l/t A		l/h	l/t B	
N-r-CF								
Water	80%							
VTS	68%	10	23	56	4 5	17	91	62
Net calorific value	3,800 kcal/kg-DS							
T-r-CH								
Water	80%							
VTS	71%	10	28	66	4	19	111	59
Net calorific value	3,500 kcal/kg-DS							
K-r-CF								
Water	77%							
VTS	55%	9 5	24	62	4	18	103	60
Net calorific value	3,200 kcal/kg-DS							
N-d-CF								
Water	74%							
VTS	45%	11 5	30	63 5	4 5	19	107	59
Net calorific value	2,400 kcal/kg-DS							
C-d-BF								
Water	78%							
VTS	38%	15	51	83	4	24	143	58
Net calorific value	2,000 kcal/kg-DS							
T-r-PF								
Water	68%							
VTS	40%	10	19	46	5	15	73 5	63
Net calorific value	2,400 kcal/kg-DS							

1. Dryer exhaust gas	
Dust (g/Nm^3)	0 04
SOx (ppm)	11
Hcl (ppm)	130
Cl_2 (ppm)	0 1
Ammonia (ppm)	44
Trimethyl amine (ppm)	1
Hydrogen sulfide (ppm)	9
Methyl sulfide (ppm)	1
Methyl disulfide (ppm)	0 7
Methyl mercaptan (ppm)	5
Odor index	1 100 x 10^3

2. Condenser exhaust gas	
Dust (g/Nm^3	0 02
SOx (ppm)	5
HCl (ppm)	61
Cl_2 (ppm)	0 1
Ammonia (ppm)	0 8
Trimethyl amine (ppm)	0 9
Hydrogen sulfide (ppm)	0 5
Methyl sulfide (ppm)	0 9
Methyl disulfide (ppm)	0 8
Methyl mercaptan (ppm)	4
Odor index	230 x 10^3

3. Condenser wastewater	
pH	7 2
SS (ppm)	21
BOD (ppm)	160
COD (ppm)	28

4. Furnace exhaust gas	
Specific gravity (air)	1 4–1 7
Dust (g/Nm^3	39
CO (ppm)	2,000
SOx (ppm)	520
NOx (ppm)	20
HCl (ppm)	530

5. Cyclone exhaust gas	
Dust (g/Nm^3)	6

6. Furnace ash	
Water (%)	0 5
Ignition loss (%)	1 0
Elemental analysis	
C (%)	0 6
H (%)	ND
N (%)	ND
Cl (%)	0 012
Combustible sulfur (%)	ND
Noncombustible sulfur (%)	1 2
Cr^{6+}	
Content (ppm)	ND
Effluent (ppm)	ND

7. Waste-heat boiler exhaust gas	
Dust (g/Nm^3)	3

8. Scrubber exhaust gas	
Dust (g/Nm^3)	0 07
CO (ppm)	2,000
SOx (ppm)	0 5
NOx (ppm)	14
HCl (ppm)	10
Cl_2 (ppm)	0 1
Odor index	920

9. Scrubber	
pH	6 1
SS (ppm)	260
BOD (ppm)	8
COD (ppm)	20
Phenol (ppm)	ND
PCB (ppm)	ND

Sample T-r-CF

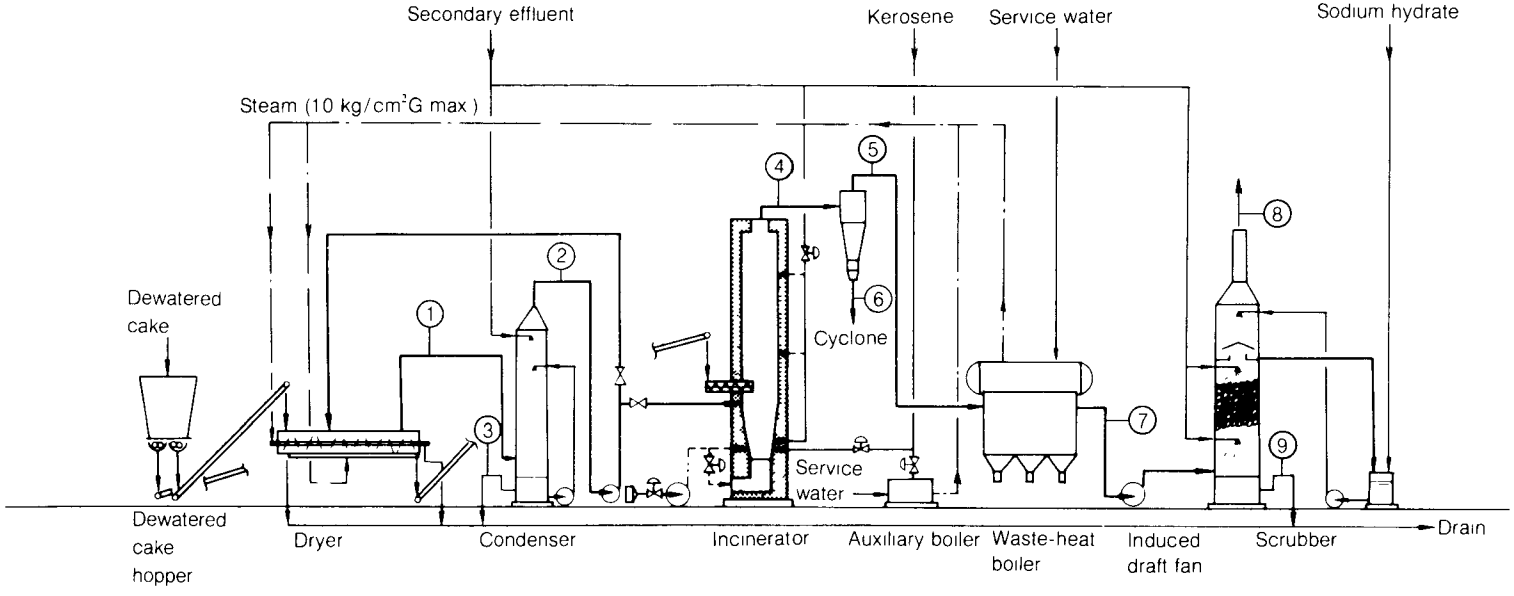


Fig. 14 Analysis of gas, drain and furnace ash

2.3. DISCUSSION

2.3.1. Energy efficiency

The separate-drying system consumed less fuel in experiments than the conventional system. However, steam from the waste-heat boiler proved to be insufficient in itself to heat the dryer enough to bring the water content of the sludge cake down to the point where self-sustaining combustion could be obtained. A kerosene-burning auxiliary boiler therefore had to be installed to provide additional steam, and this limited the energy savings over the conventional system to 40 percent. Factors presumed to be responsible for this are as follows:

— The surface area of the furnace, boilers, pipe and ducts used in the pilot system was large in proportion to the capacity of the system, resulting in an unnecessarily large amount of heat loss through radiation (2.2.5).
— The waste-heat boiler was inefficient (2.2.4).
— A larger than anticipated share of total heat output was lost as steam drain residual heat (2.2.5)

The *Table 9* values for sludge cake net calorification and fuel consumption are represented graphically in *Fig. 15*. As is clear from the graph, the pilot system required no fuel to process dewatered cake offering net calorification of 870 kcal/kg. It is because the sewage treatment facilities now in operation in Yokohama cannot turn out sludge of this quality that the auxiliary boiler had to be installed. However, a larger-scale system will lose a smaller percentage of its total heat output through radiation, thus greatly lowering the level of net calorification required of sludge cake for processing without supplementary fuel (3.1.3). Moreover, eliminating the auxiliary burner chamber and implementing continuously-operating automatic mechanisms for soot blowing and ash removal will improve the efficiency of the waste-heat boiler. Recycling steam drain as feedwater for the boiler will also help, this not having been done in pilot system testing (3.1.2).

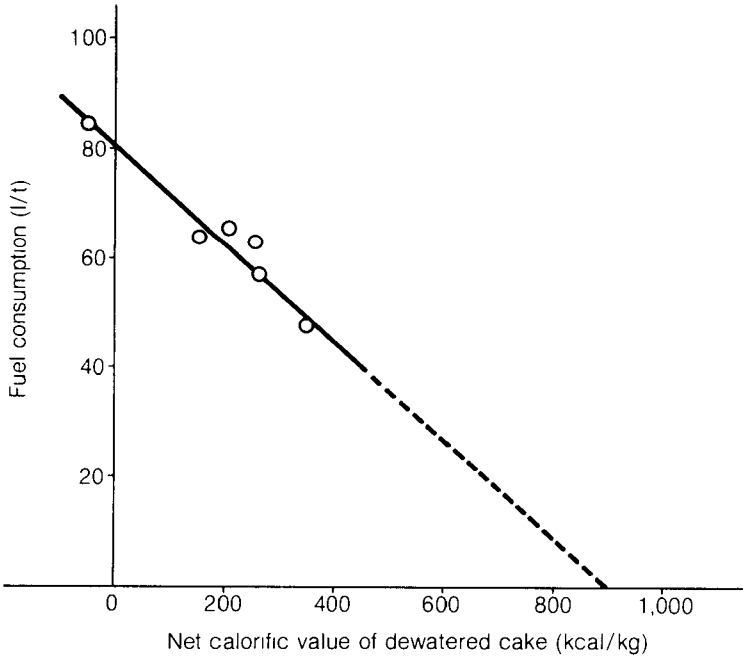


Fig. 15 Net calorific value of dewatered cake and fuel consumption

2.3.2. Controlling the water content of dried cake

Controlling the water content of cake fed into the furnace is a key to maintaining stable, self-sustaining combustion. Disk speed, steam pressure and drying time were regarded as potential means to this end.

a. Disk speed

The disk speed must be neither too fast nor too slow for good mixing (2.2.2). Given the possibility of different disk diameters, optimum disk speed should perhaps be considered in terms of circumference speed rather than r.p.m. Thus the 20–30 r.p.m. at which good mixing was obtained in preliminary testing with 250 mm diameter disks translates into a circumference speed of 0.26–0.39 m/s, while the 15 r.p.m. that yielded good mixing in pilot-system operation with 500 mm-diameter disks works out to a circumference speed of 0.39 m/s. A circumference speed of 0.3–0.4 m/s would therefore seem to produce the best mixing, regardless of r.p.m.

b. Steam pressure level

Varying the steam pressure brought about virtually no change in the water content of the dried cake (2.2.2).

c. Drying time

Experiments confirmed that adjusting the drying time by changing the height of the outlet weirs allows a certain degree of control over the water content of the dried cake (2.2.2). It was found in batch-operation testing, for instance, that cake can be dried to a water content of less than 20 percent if a long enough drying time is used. In any case, handling requirements dictate that the water content of dried cake be brought down at least as low as 50 percent or so, even if this happens to be lower than the maximum water content for self-sustaining combustion. Stable combustion was achieved by burning dried cake of this 50 percent water content together with dewatered cake, the feed-rate of the latter being varied to control the overall water content (2.2.3).

2.3.3. Material

a. Dryer

HCl in the dryer exhaust gas caused pitting on SUS304. Corrosive pitting also occurred on welded portions of SUS316 test pieces, but the parent metal was unaffected. For the dryer, then, it would appear necessary to use good-welding, low-carbon steel of at least the quality of SUS316L.

b. Waste-heat boiler

Corrosion was found all over STPG38 test pieces and the water pipes of the waste-heat boiler used in the pilot system. This was due to the action of sulfur oxides and HCl in water that condensed out of the furnace exhaust gas on account of both frequent starting and stopping and failure to keep the apparatus sufficiently clear of ash and soot. It is felt that STPG38 will be an adequate material for a working boiler, since a full-scale plant will feature uninterrupted operation and automatic and continuous removal of soot and ash.

3. INTRODUCTION OF A FULL-SCALE PLANT

3.1. SYSTEM PLANNING

3.1.1. Incinerating capacity

Yokohama's sewage works plan calls for a total of eleven sewage treatment plants, of which nine were operating as of March 1983. Central incinerators for burning sludge generated by the different treatment plants are being constructed at the Kanazawa and Hokubu No. 2 coastal plant sites. Treatment plants will ultimately be connected to these two incinerators by sludge lines, with sludge cake to be transported by truck until the pipelines are completed.

The Kanazawa facility is to process sludge cake from the six treatment plants in southern Yokohama, meaning that the separate-drying incineration system used will need to have an incinerating capacity of 150 tons per day. On the basis of findings of the previous investigation, water content of 80 percent, VTS of 65 percent, and net calorification of 3,300 kcal/kgDS were assumed for purposes of basic calculations. These values would result in a maximum water content of about 65 percent for self-sustaining combustion by cake fed into the furnace. However, pilot system testing showed that dried cake containing as much as 65 percent water would cause problems by sticking to the conveying apparatus. It was therefore decided to dry some of the dewatered cake from treatment plants to 50 percent and then mix this with the rest of the (undried) cake to bring the overall water content of cake fed into the furnace to 65 percent. Thus 107.2 tons of the 150 tons of dewatered cake processed daily would be run through the dryer before incineration, and 42.8 tons fed directly into the furnace (*Fig. 16*).

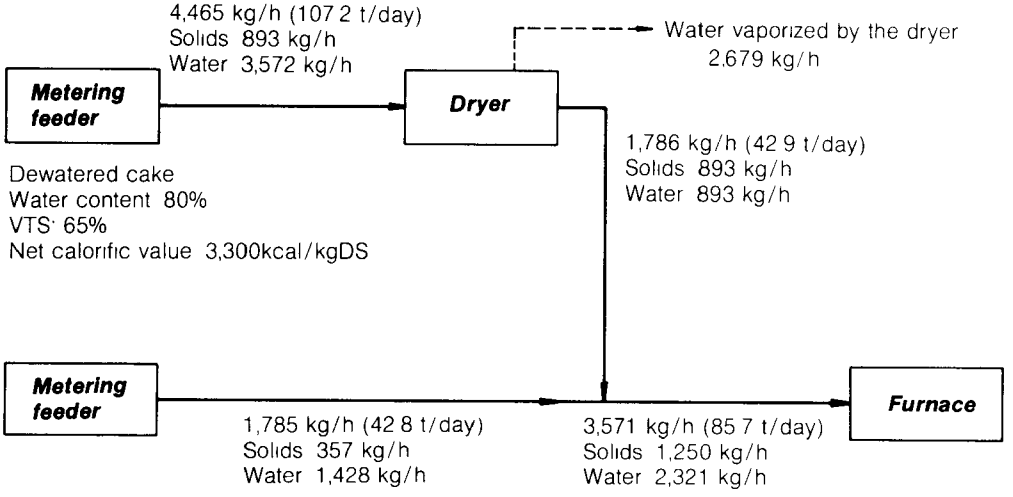


Fig. 16 Feed rates to dryer and furnace

3.1.2. System outline

Table 10 presents basic design criteria for the separate-drying system, *Table 11* specifications for main units, *Fig. 17* a system flowchart, and *Fig. 18* a layout diagram. The flowchart is explained below:

Dewatered cake from the various treatment plants is dumped into the cake pit (1), from which a crane (2) loads it into the incinerator feeder (3) and cake hopper (4). As already described, about two-thirds of the daily volume of 150 tons of dewatered cake is loaded into the cake hopper for metering into the dryer (5). The dryer heats the cake indirectly at a steam pressure of 5 kg/cm^{2}G to bring the water content down to 50 percent. The remaining dewatered cake is metered out of the feeder (3) at the rate of about 50 t/day.

Dried and non-dried cake are metered onto the same conveyor for transport into the furnace (6), where they are incinerated together. Since the composite water content of the cake is low enough to permit self-sustaining combustion, no supplementary fuel (LNG in the case of non-self-sustaining combustion) need be used in the furnace. Exhaust gas at a temperature of around 800° C is directed from the furnace to the cyclone (7), which separates dust out of the gas. When the system is operated conventionally, exhaust gas that has been purged of dust in the cyclone (7) passes through an air preheater (8) en route to the cooler (9). In the case of operation as a separate-drying system, gas from the cyclone passes next through the waste-heat boiler (14) and then goes to the cooler (9). After passing through the cooler the exhaust gas is directed through an electrostatic dust precipitator (10) and absorptive cooling tower (11). An induced draft fan (21) drives the gas to the stack (12), where it is mixed with air to prevent dense smoking and then released into the atmosphere.

Table 10. Design criteria

Incinerating capacity		150 t/day	
Dewatered cake	Coagulent	Inorganic	Polymer
	Water content (%)	55 ~ 80	70 ~ 80
	VTS (%)	25 ~ 55	35 ~ 80
	Net calorific value (kcal/kgDS)	1,100 ~ 2,600	1,600 ~ 3,700
Furnace	Exhaust gas temperature at furnace outlet (°C)	750 ~ 850	
	Excess air ratio	Approx 1 3	
	Furnace ash VTS (%)	Less than 3	
	Fuel	LNG (9,900 kcal/Nm^3)	
Drying	Water vaporization (kg/h)	Approx 3,200	
	Water content of dried cake (%)	Approx 50	

Steam generated by the waste-heat boiler (14) is supplied as a heat source to the dryer (5). Steam drain from the dryer is recovered at 4 kg/cm^{2}G (151° C) into a drain tank (13) to be reused as feedwater for the waste-heat boiler and auxiliary boiler (15).

A condenser (17) removes moisture from dryer exhaust gas. A blower (22) then drives some of the gas into the furnace (6) to oxidize combustion, but circulates most back into the dryer for reuse as carrier gas for carrying off, as steam, water vaporized out of the sludge cake.

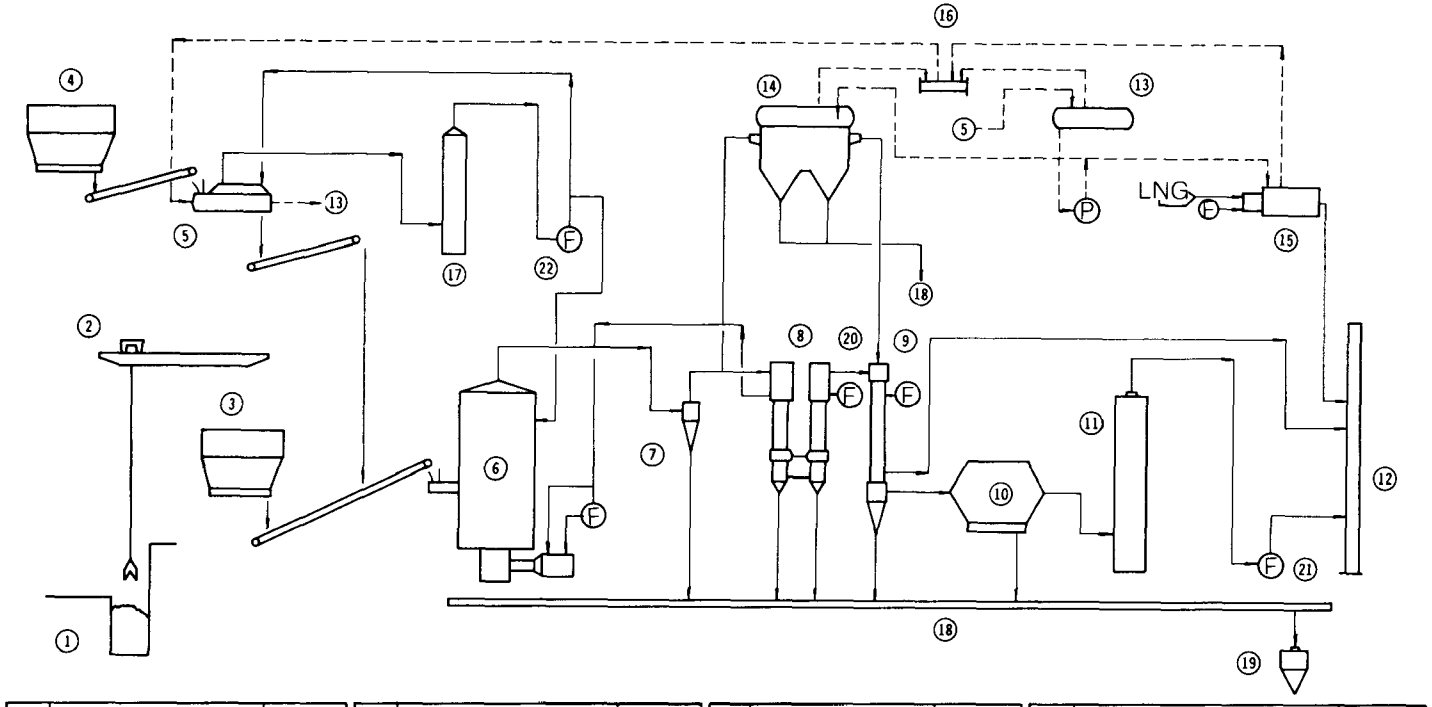
Furnace ash discharged from the cyclone (7), waste-heat boiler (14), air preheater (8), cooler (9), and electrostatic dust precipitator (10) is carried by a conveyor (18) to an ash hopper (19). Ash collected there is moistened with water and then carried away in trucks.

Table 11. Specifications of min units

Unit	Specifications	Quantity
Incineration equipment		
Furnace	Fluidized bed furnace, 5,500 mm (dia) x 12,270 mm (h), 100 t/day capacity (150 t/day as separate-drying system)	1
Hot air generator for system start-up	220 x 10^4 kcal/h, 700° C max	1
Cyclone	420 Nm^3/min, 85% dust-separating efficiency, 1,430 mm (dia) x 5,190 mm (h)	2
Air preheater	Countercurrent, vertical multi-cylinder type, 354 6 m^2 heating surface	2
Fluidizing air blower	Multi-stage turbo blower, 150 Nm^3/min x 3,600 mmH$_2$O x 150 kW	1
Electrostatic precipitator	Dry horizontal flow type, 14,000 Nm^3/h, 190-340° C, dust concentration less than 0 02 g/Nm^{3}DG at outlet	1
Absorptive cooling tower	3-stage spray type (sodium hydrate and water) 2,100 mm (dia) x 14,500 mm (h), 14,000 Nm^3/h, 40° C at outlet	1
Induced draft fan	Radial type, 250 Nm^3/min x 900 mmH$_2$O x 75 kW	1
Drying equipment		
Dryer	Indirect heating, low-speed mixing type, 100 m^2 heating surface, 10 kg/cm^2 max steam pressure, 37 kW	2
Condenser	3-stage spry type (sodium hydrate and water), 1,250 mm (dia) x 10,000 mm (h), 50 Nm^3/min, 30° C at outlet	2
Waste-heat boiler	Natural circulation, pipe type, 3 5 t/h vaporization at 10 kg/cm^{2}G, 294 m^2 heating surface	1
Auxiliary boiler	Furnace flue type, 1 t/h vaporization at 10 kg/cm^{2}G, 24 7 m^2 heating surface	1

3.1.3. Heat flow and supplementary fuel consumption

Fig. 19 shows the flow of heat input and output as calculated for the system on the basis of the pilot system study and basic calculations for the full-scale plant. Since heat loss via furnace radiation is directly proportional to the square root of the incinerating capacity, this loss as a percentage of heat input to the furnace will be much smaller with the full-scale plant (7 percent) than with the pilot system (30 percent). And relatively less radiative heat loss means that self-sustaining combustion can be obtained with a given type and condition of sludge cake at a higher water content (*Fig. 20*). Thus the maximum water content for self-sustaining combustion rises to 65 percent with a system capacity of 150 t/day.



No	Unit	Quantity	No	Unit	Quantity	No	Unit	Quantity	No	Unit	Quantity
1	Cake pit	1 (5)	7	Cyclones	2	13	Steam drain tank	1	19	Ash hoppers	2
2	Crane	1 (dual)	8	Air preheaters	2	14	Waste-heat boiler	1	20	Fluidizing air blower	1
3	Incinerator feeder	1	9	Cooler	1	15	Auxiliary boiler	1	21	Induced draft blower	1
4	Cake hopper	1	10	Electrostatic precipitator	1	16	Steam heater	1	22	Condenser exhaust blowers	2
5	Dryers	2	11	Absorptive cooling tower	1	17	Condensers	2	·		
6	Furnace	1	12	Stack	1	18	Ash conveyors	3			

Fig. 17 Flowchart

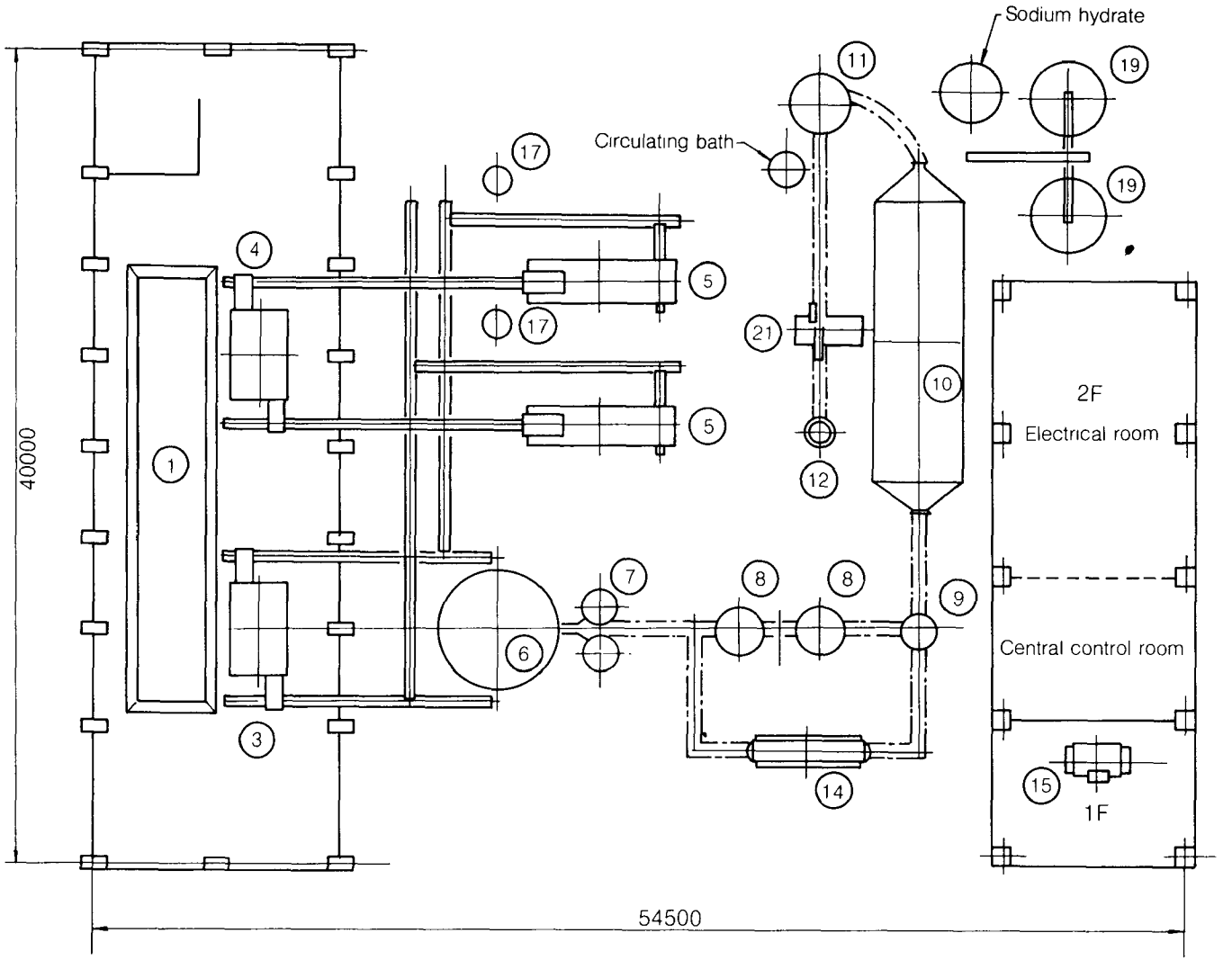


Fig. 18 System layout (w/reference numbers from Fig. 17)

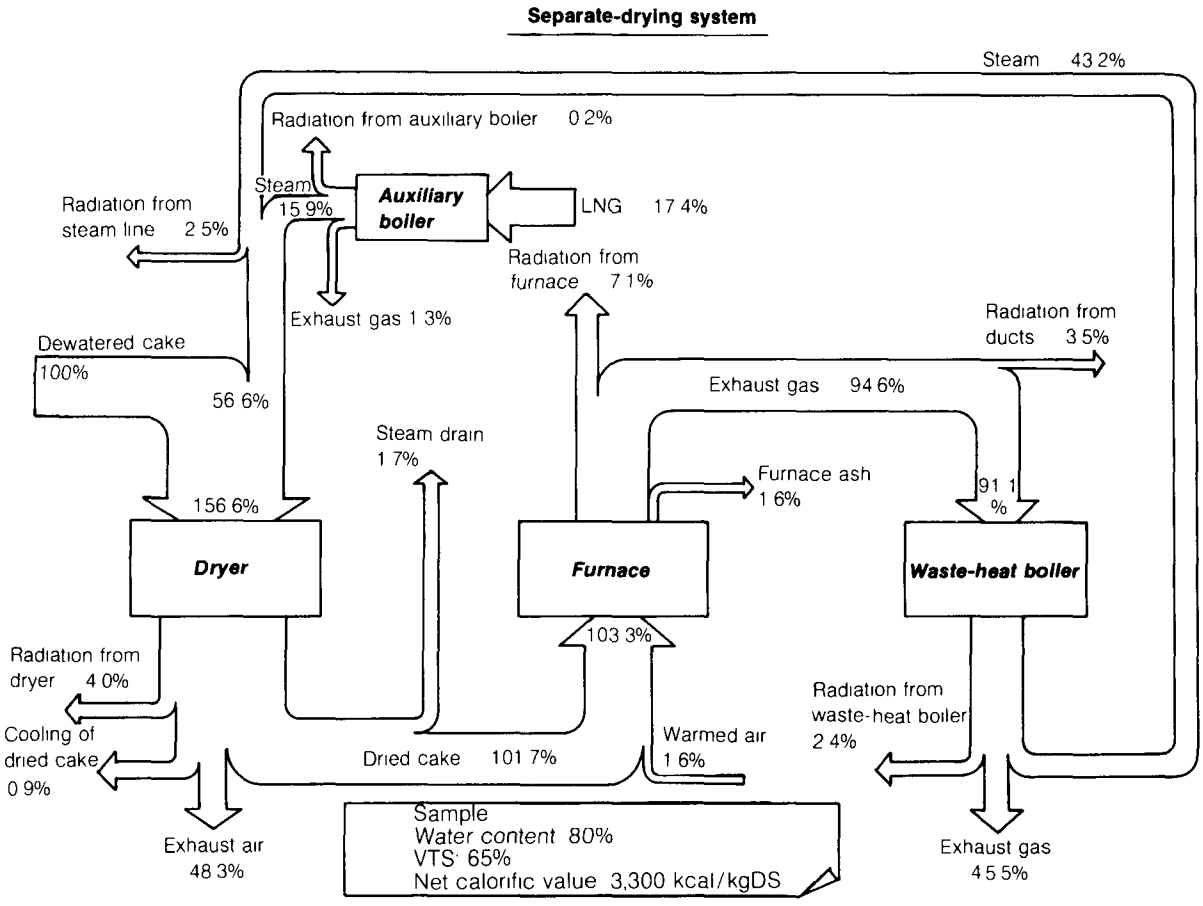


Fig. 19 Heat flow

Heat flow comparison between the pilot system (*Fig. 13*) and the full-scale plant (*Fig. 19*) reveals that specific heat consumption with the latter will be only about 1/5 that seen with the former. The primary reasons for this are to be found not only in the relative reduction of radiative heat loss from the furnace and ducts achieved through larger scale, but also in more efficient operation by the waste-heat boiler and in recovery of steam drain from the dryer at 4 kg/cm^2G for recycling as boiler feedwater.

Moreover, just as with the pilot system, it should be possible to reduce specific fuel consumption by about a third with the working facility by employing the more heat-efficient, separate-drying system.

Fuel consumption with the full-scale plant, however, will of course vary according to the net calorific value of the dewatered cake (*Fig. 21*). As can be seen from *Fig. 21*, incineration with the separate-drying system requires 30–40 Nm^3 less fuel (LNG) per ton of sludge cake than conventional incineration, in which all of the dewatered cake is fed directly into the furnace.

Points A and A' in the graph indicate fuel consumption with dewatered cake having a net calorific value of 300 kcal/kg. Whereas a conventional system consumes some 30 Nm^3 of LNG to incinerate a ton of sludge in this case, a separate-drying system requires no fuel at all. The relationship between water content and the net calorific value of solids is shown in *Fig. 22* for dewatered cake of this calorific value.

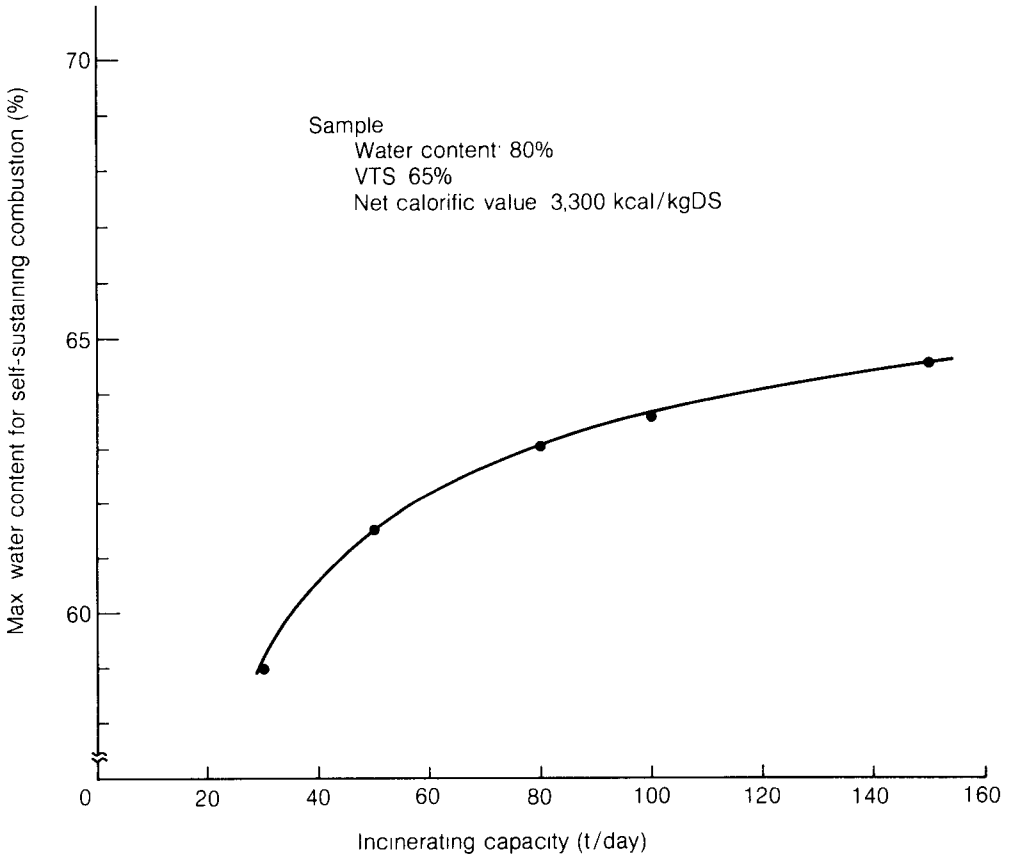


Fig. 20 Incinerating capacity and maximum water content for self-sustaining combustion

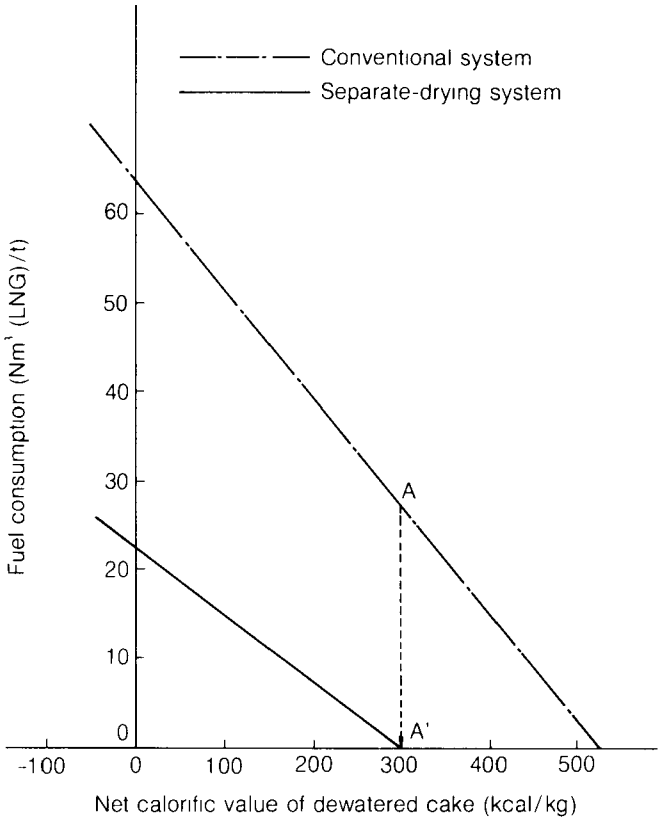


Fig. 21 Net calorific value of dewatered cake and fuel consumption

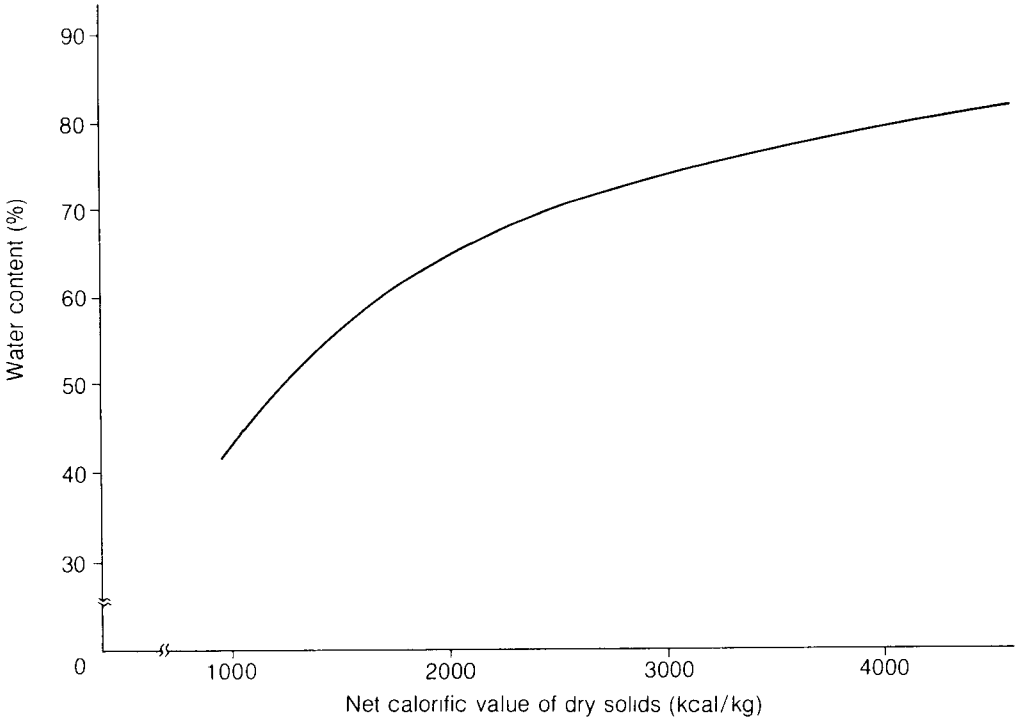


Fig. 22 Net calorific value of dry solids and water content (dewatered sludge having a net calorific value of 300 kcal/kg)

3.2. ESTIMATED COSTS

3.2.1. Construction costs

Table 12 gives current construction costs by capacity for separate-drying incineration systems. These totals include equipment, piping, electrical instrumentation, engineering, and installation costs, but exclude the cost of buildings and land. Manufacturer estimates were used to compute equipment costs, with the other costs then being calculated as percentages of these. It is plain from the table that specific construction cost declines as incinerating capacity increases, the inference being that a 150 t/day facility could be built for only three-times the cost of a plant having a fifth (30 t/day) that capacity.

Table 12. Incinerating capacity and construction cost

(unit ¥ million)

	30 t/day	50 t/day	80 t/day	100 t/day	150 t/day
Construction cost	973 (43)	1,391 (62)	1,932 (86)	2,259 (100)	2,999 (133)
Specific construction cost (¥ million/t capacity)	32 (139)	27 (117)	24 (104)	23 (100)	20 (87)

Note Figures in parentheses are percentages of the corresponding costs with a 100 t/day facility

3.2.2. Operating costs

Table 13 lists specific LNG consumption and specific electrical consumption for separate-drying incineration systems of different capacities. Fuel consumption was calculated on the basis of test results, electrical consumption in terms of the rated power requirements of principal equipment in continuous operation. The table suggests that, while specific electrical consumption will not change very dramatically with incinerating capacity, merits of scale will indeed manifest significantly in the case of specific LNG consumption.

Table 13. Incinerating capacity and specific consumption of LNG and electricity

Incinerating capacity	30 t/day	50 t/ day	80 t/day	100 t/day	150 t/day
LNG (Nm^3/t)	34	23	17	14	11
Electrical capacity (kWh/t)	68	59	57	54	48

Notes
1 Dewatered cake
Water content 80%, VTS 65%, net calorific value 3,300 kcal/kgDS
2 LNG
Net calorific value 9,900 kcal/Nm^3

SUMMARY AND ACKNOWLEDGEMENTS

As described in this paper, the City of Yokohama conducted a pilot system study to determine how effectively an incineration facility comprising a fluidized bed furnace and separate-dryer would be able to incinerate the various types of dewatered cake generated by the city's eleven sewage treatment plants. This study confirmed the energy efficiency and reliability of such a system. In fact, study findings suggest that a full-scale facility of 150 t/day capacity could incinerate sludge without using any supplementary fuel for combustion.

The author wishes to acknowledge the untiring effort by all the staff who worked so hard in carrying out the testing, and to express his appreciation for valuable assistance provided in preparing this report by Mr. Eiichi Oiwa, of the Electrical and Mechanical Designing Division, and Messrs. Yoshiyuki Nakamura and Hitoshi Kobayashi, of the Planning Division.

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MANUFACTURING ARTIFICIAL LIGHT WEIGHT AGGREGATES FROM SEWAGE SLUDGE

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1. SLUDGE DISPOSAL AND ITS COST IN TOKYO

In the past, sewage sludge could easily be disposed of by simple incineration. Sewage ash was used for coastal reclamation and was beneficial for soil conditioning. This is now nothing more than a legend. Problem of hexa valence chromium contamination began. Because it was feared that the leachate of the hexa valence chromium-loaded sludge ash might possibly contaminate the environment, it was decided that the disposal of sludge ash from all Tokyo Metropolitan urban area should be done at the dump site located beyond the central breakwater of Tokyo Bay. This landfill was planned to fill up to 32 meters above sea level and, sludge ash cannot be disposed of directly, because landfill must possess a required level of bearing strength.

For this reason, sludge ash, sludge cake and cement are mixed and formed into tough blocks before disposal at the site. Figure 1 shows a flowchart of present sludge handling and disposal.

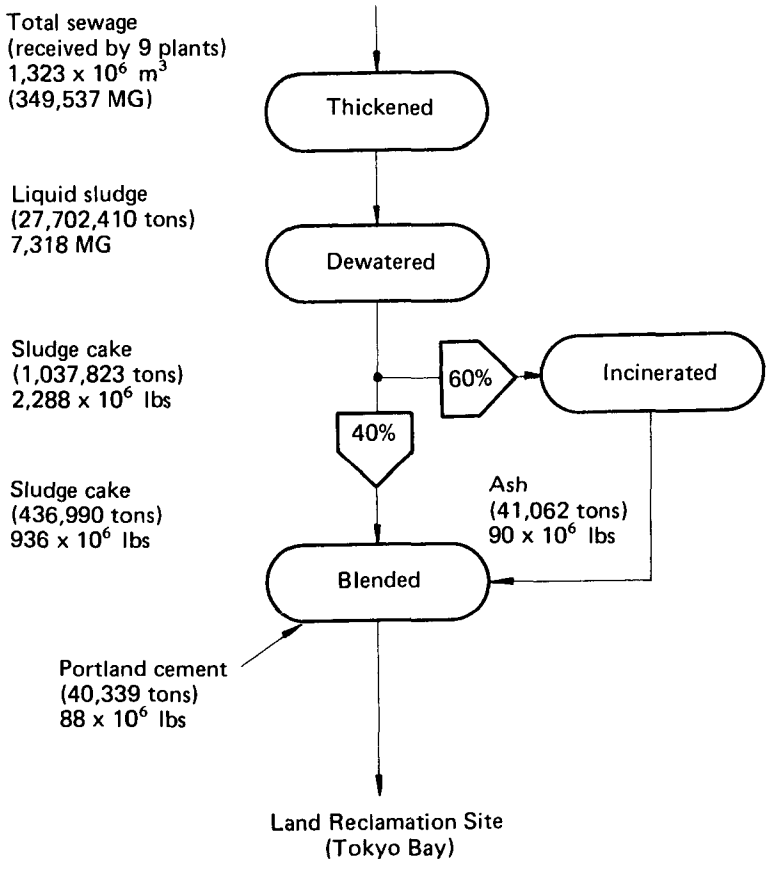

Fig. 1 Flowchart of sludge handling and disposal

Table 1 Statistics of sludge handling in Tokyo's sewage plant

Name of Sewage Plant	Influent rate (m^3)		Liquid Sludge (m^3)		Sludge cake (t)		Incinerated cake (t)	
	annual	daily	annual	daily	annual	daily	annual	daily
Shibaura	272,509,390	746,600	2,494,390	6,830	150,691	413	-	-
Sunamachi	176,572,350	483,760	3,991,070	10,930	316,571	867	316,520	867
Mikawashima	231,054,270	633,030	Pumped to Sunamachi Plant		-	-	-	-
Odai	110,806,870	303,580	9,255,940	25,360	207,427	568	201,693	553
Ochiai	181,776,030	498,020	Pumped to Odai Plant		-	-	-	-
Morigasaki	370,607,760	1,015,360	6,995,800	19,170	281,221	770	72,465	199
Shinkashi	133,337,700	365,310	4,463,660	12,230	67,680	185	10,274	28
Kosuge	45,879,510	125,700	378,480	1,040	11,120	30	-	-
Kasai	12,169,330	63,050	123,070	1,130	3,113	29	-	-
Total	annual 1,534,713,210	daily 4,204,690	annual 27,702,410	daily 75,900	annual 1,037,823	daily 2,843	annual 600,952	daily 1,646

For these reasons, the net operating cost of ash disposal after incineration has risen to approximately ¥6,500 per ton of solid (1.2¢/lb). Naturally, this motivated us to find uses for the sludge and produce somethings from the sludge ash.

The first trial to this purpose was to produce lightweight coarse aggregate (LWCA).

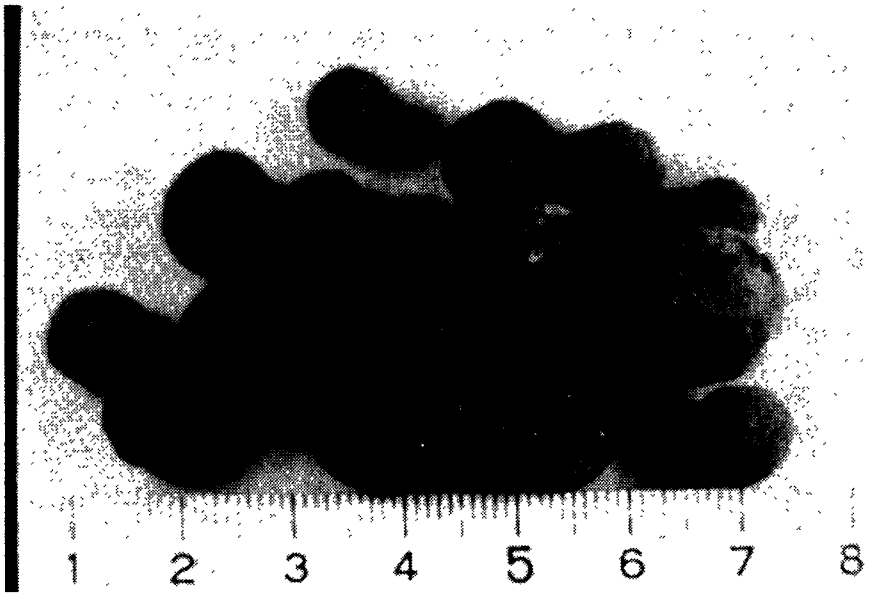

Photo 1 Light weight coarse aggregates made of sludge ash

LWCA made of sludge ash is shown in Photo 1. It has a grain size of 1 to 1.5 cm and a specific gravity of 1.3, and its quality compares well with that of other natural coarse aggregates.

2. FROM COARSE TO FINE AGGREGATE

The lightweight coarse aggregates available on the market are usually made of shale. The main ingredients of shale are silica and aluminum. Shale also contains a small amount of volatile materials. When fragments or pellets of shale are heated to 1000 to 1200^{o}C, the silica becomes fused, and at the same time, the volatile materials become gas. Thus, the grains of shale are puffed out. When they are cooled, they become a hard, yet lightweight aggregate because they are porous cores covered with finetextured adamant shells.

As reported at the 8th conference, we developed a process of converting a mixture of 70% shale and 30% sludge ash into a lightweight coarse aggregate with quality equal to what is available on the market.

Although it is good news to find the way of turning sludge ash into being a profitable operation, it leaves much to be desired as is enumerated below.

(1) Sludge ash accounts for only 30% of the lightweight coarse aggregate, and this percentage must be improved for total disposal of sludge ash.

(2) Sludge-converted lightweight coarse aggregate has no advantages to the aggregate now available on the market, and thus stands no comparative advantage to compete with it.

(3) Lightweight coarse aggregate is used only for construction purposes, and supply exceeds demand.

To overcome these problems, we modified the goals for sludge ash technology development project as follows:

(1) To develop products consisting mainly of sludge ash.

(2) To develop products which can compete with similar existing products.

(3) To develop products which have a wide market.

3. DEVELOPMENT OF ARTIFICIAL LIGHTWEIGHT FINE AGGREGATE PRODUCTION PROCESS

3.1 Characteristics of Sludge Ash

Table 2 shows a chemical analysis of shale and sludge ash.

Table 2 Composition of sludge ash and shale

	Ash		Shale
	Ave.	Var.	Ave.
SiO_2	39.55	4.90	64.30
Al_2O_3	16.41	1.35	16.34
Fe_2O_3	11.75	0.73	1.34
FeO	2.09	0.48	3.23
P_2O_5	10.10	2.43	-
CaO	8.36	1.76	1.20
MgO	2.46	0.11	1.23
Na_2O	1.84	0.51	1.94
K_2O	1.36	0.27	2.55
SO_3	0.84	0.22	0.65
C	0.67	0.12	0.66
CO_2	0.13	0.07	1.75
Cl	0.05	0.05	-
Total	95.60	1.05	95.19
Ign. Loss	2.67	0.96	7.34

Shale is the principal ingredient of the lightweight aggregate now available on the market. We sampled and analyzed the sludge ash discharged from the Shinkashi Sewage Plant operated by the Tokyo Metropolitan Government. The results are as summarized in Table 2. It was found that the ratios of the main ingredients, such as SiO_2, Al_2O_3 and Fe_2O_3, varied greatly among samples. This forced us to develop an artificial lightweight fine aggregate production process that allows a large variance in the principal ingredients.

Figure 2 is a triangular chart showing the chemical composition of shale and sludge ash.

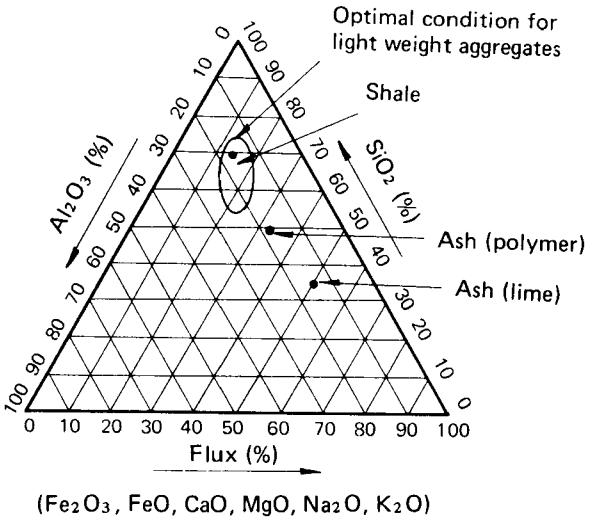

Fig. 2 Composition of ash and shale

When compared with shale, sludge ash contains less SiO_2 but more Fe_2O_3 and CaO. Generally, the higher the content of CaO, the higher the melting point of the feedstock. The higher the melting point, the lower the viscosity of the fused feedstock; namely, the substance cannot contain gases within it, reducing the structural strength of the resultant fine aggregate. It would be assumed, therefore, that sludge ash would produce aggregate of poor quality. To clarify this point, we conducted a thermal analysis, with the findings summarized in Table 3.

Table 3 Thirmal characteristics of sludge ash

Sample No.	Previous incineration temp. (°C)	Softening temp. at $10^{6.7}$ poise (°C)	Melting temp. at $10^{3.8}$ poise (°C)	Gas generation		Calcination	
				range (°C)	volume (%)	range (±°C)	Optimum temp. (°C)
1	873 - 930	966	1110	873 - 966	0.29	8	1056
2	898 - 950	982	1155	898 - 982	0.23	11	1086
3	840 - 953	1000	1134	840 - 1000	0.26	9	1080
4	836 - 914	993	1146	836 - 993	0.32	10	1081
5	779 - 856	980	1090	779 - 980	0.50	8	1048
6	863 - 922	982	1142	863 - 982	0.32	8	1078
7	872 - 948	958	1112	872 - 958	0.33	9	1051
8	887 - 948	960	1113	887 - 960	0.23	8	1046
9	860 - 937	964	1105	860 - 960	0.34	9	1051
10	810 - 912	984	1100	810 - 984	0.61	8	1055
Ave.	-	976.9	1120.7	-	0.34	8.8	1063.2
Var.	-	13.5	20.8	-	0.12	0.98	15.1

As Table 3 clearly shows, the difference between the softening point and melting point of the sludge ash is no more than 50 degrees centigrade, and the temperature difference where the softening of sludge ash surface and the foaming inside occurs is as only 10 degrees centigrade. Obviously, this demands that we develop a kiln that can control the calcining temperature quite accurately. It seems probable that the ash produced by incinerating sludge cake at high temperatures releases little gas during calcination causing the lightweight fine aggregate quality to be poor. Thus it may be important to maintain the sludge cake incineration temperature as low as possible.

3.2 Binder for Pelletizing

The sludge ash is pelletized, dried, then calcined into fine aggregate.

First, the particles of sludge ash are mixed with glue (binder) into paste, and then made into pellets by a pelletizer. The binder is required to hold the pellets in shape and at the same time to vesicate the pellets with heat. The quality of the product is determined by the quality of binder used. We tested waste alcohol, sludge slurry and sludge cake as binders.

Table 4 Effect of glue on dried pellet strength

Glue (Binding material)	% of glue added	Moisture content in raw pellet (%)	Compressive strength of dried pellet (10mmø) (kg)
Waste alcohol	0	25.6	1.3
	10	24.7	2.8
	20	22.9	8.7
	30	20.7	16.3
Liquid sludge	30	30.0	1.4
Sludge cake	45	28.2	9.1

Table 4 shows the effect of binders on pellet strength. When water or sludge slurry was used as a binder, the compressive strength of the pellets remained as low as 1.3 to 1.4 kg. On the other hand, sludge cake improved the pellet strength to as high a value as 9.1 kg. However, blending sludge cake and sludge ash uniformly is not easy. The final results showed that waste alcohol worked best. Figure 3 shows the relationship between the proportion of alcohol and pellet strength.

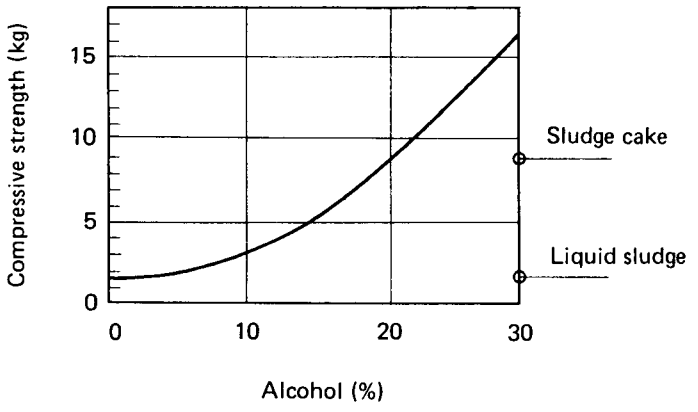

Fig. 3 Strength of dried pellet vs % of waste-alcohol added

When the proportion of waste alcohol added was 20% or more, the compressive strength of pellets could be increased above the practical value of 2.5 kg. Figure 4 shows the specific gravities of fine aggregates produced by calcining the pellets with various binders.

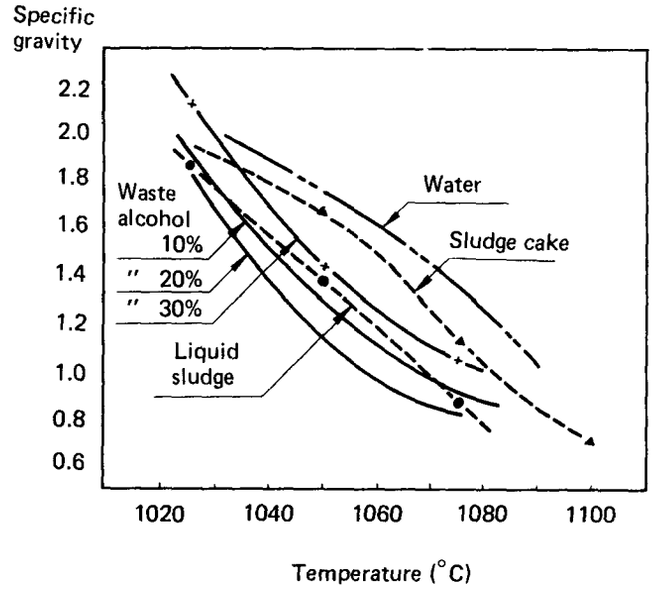

Fig. 4 Specific gravity of calcined pellet vs calcining temperature

As shown, the pellets made with 20% waste alcohol as a binder have the lowest specific gravity. It was later than a 20% mix proportion of alcohol was much liable to decrease the undersize during calcination thus improving the yield. However, a 10% mix proportion of alcohol can be sufficient enough to decrease the undersize.

3.3 Pelletizing and Drying

Based on the findings of the basic studies discussed above, a mini-plant was constructed to continuously produce lightweight fine aggregate from sludge ash. Its flow sheet is shown in Figure 5.

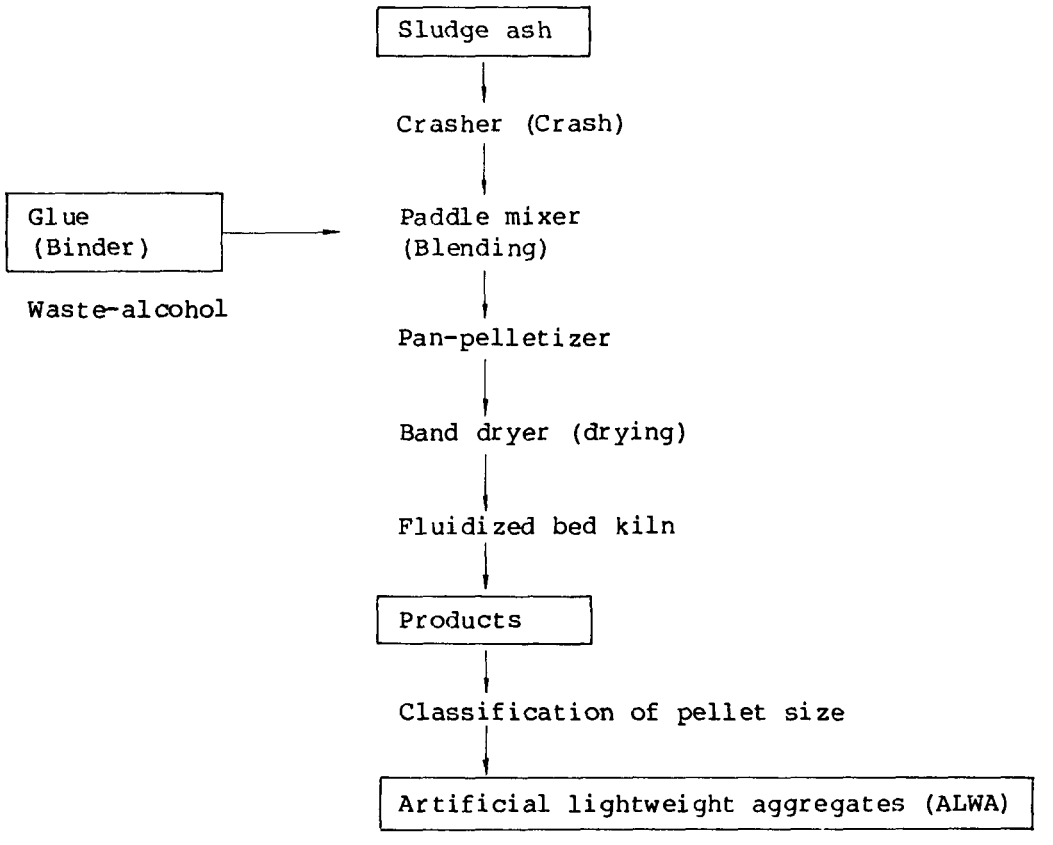

Fig. 5 Flow sheet of pilot plants

First, a vibration mill pulverizes sludge ash. The pulverized sludge ash is then passed through a 200-mesh screen, and the undersize is then is mixed with waste alcohol and water in a paddle mixer (Photo 2) into paste. The paste is then converted into pellets of 0.6 to 3.0 mm in diameter through a pan type pelletizer (Photo 3). The pellets are dried by a band dryer and calcined in a kiln (Photo 4).

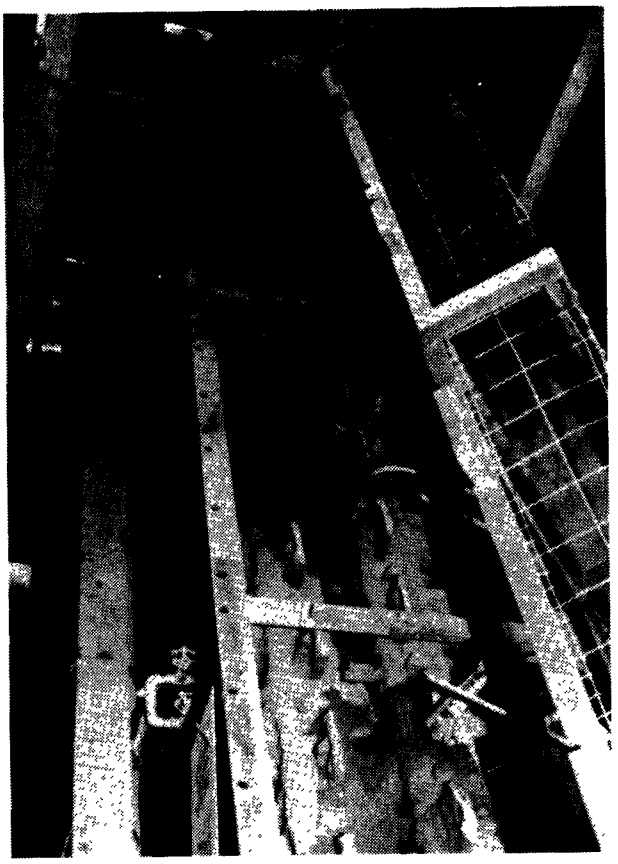

Photo 2 Paddle mixer

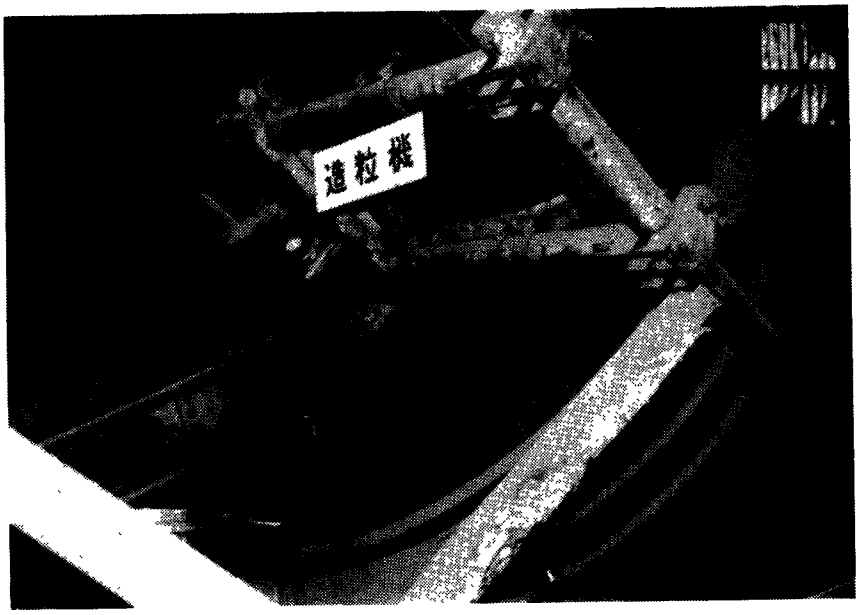

Photo 3 Pan-type pelletizer

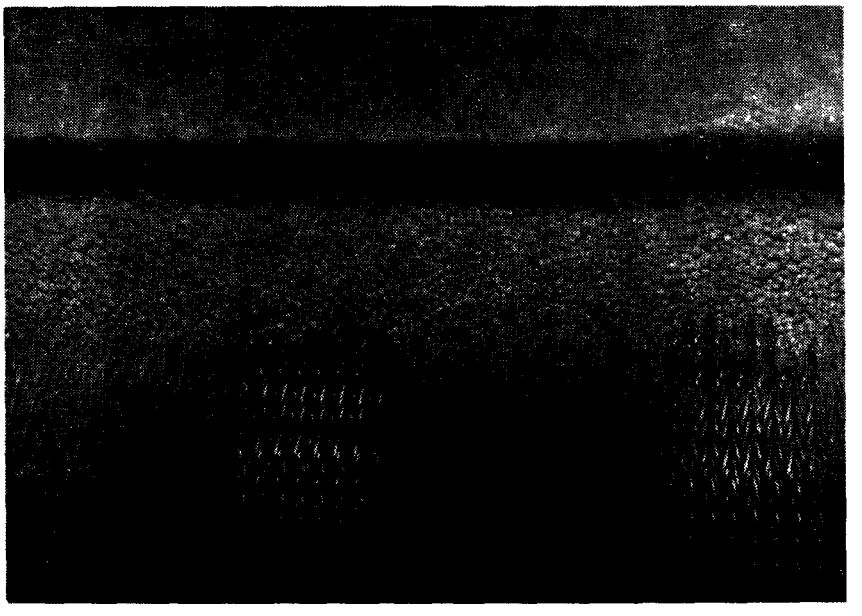

Photo 4 Band dryer

3.4 Kiln

The kiln which calcines the dry pellets into lightweight fine aggregate is required to fulfill the following requirements:

(1) To heat the pellets quickly to promote foaming.

(2) To fluidize the pellets to prevent cohesion of pellets.

(3) To keep high thermal efficiency to control temperature accurately.

To achieve these requirements, we developed a kiln as illustrated in Figure 6, and called it a 3-stage fluidized kiln. Photo 5 shows the cross section of this kiln in operation.

Traditionally, a rotary kiln was used for this purpose. One of the disadvantages of the rotary kiln is cohesion between raw materials during calcination. The performance is entirely dependant on whether ALWA can be efficiently produced from sole sewage ash in the kiln. A 3-stage fluidized kiln proved to be the most effective. The developed kiln consisted of three cylindrical chambers. Each chamber had a different diameter and was interconnected to the other via a tapered neck. From the top to the bottom of the chambers, the diameters were 22, 18, and 15 centimeters (the cross-sectional area ratio of the three chambers was 2.5/1.5/1).

Raw materials, pellets, were fed into the top chamber and air, mixed with natural gas, was put into the bottom chamber. The bottom chamber was for calcination, the middle and top chambers were for drying.

With the given air flow rate supplied into the bottom chamber, a certain amount of pellets held in each chamber was set automatically because of the balance between gravity and floating force affect the pellets. As air enters each chamber through the necked port, the pellets are moved vigorously. When raw materials are fed into the top chamber of the kiln, the same amount of calcinated pellets come out from the bottom outlet. Complete mixing of the pellets is performed within each chamber, and a plug flow is seen in each chamber. Thus, a continuous operation is performed.

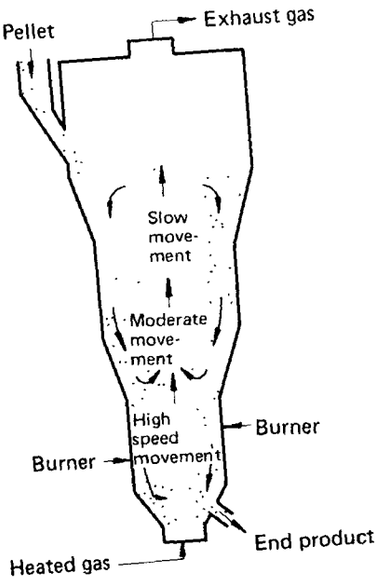

Fig. 6 Fluidized kiln

Photo 5 Cross section of fluidized kiln

4. PHYSICAL PROPERTIES OF ARTIFICIAL LIGHTWEIGHT AGGREGATE (ALWA)

Photo 6 shows the appearance of various grain sizes of ALWA produced by the process discussed above. Compared with the products now available on the market, our ALWA exhibits a higher degree of sphericalness.

Table 5 shows the physical properties of the ALWA produced compared to conventional products.

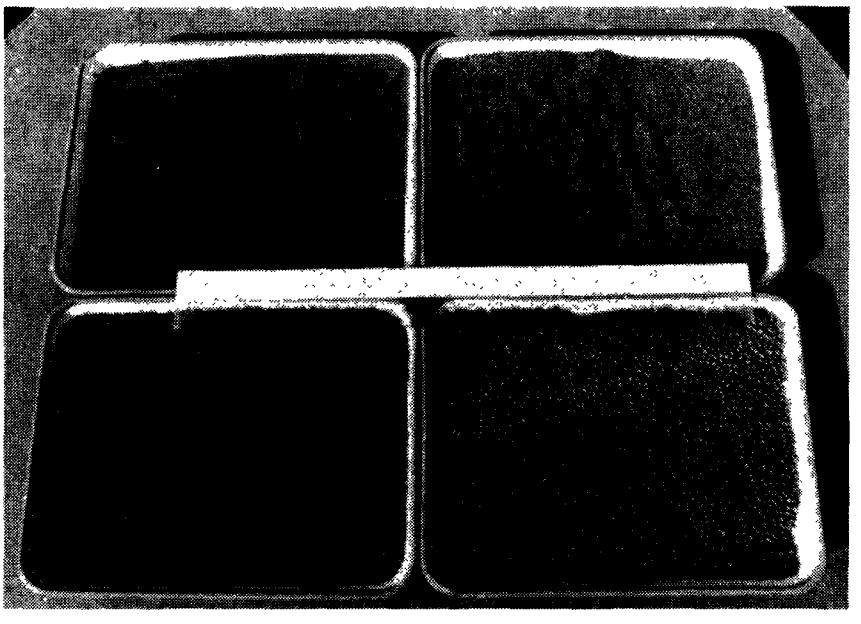

Photo 6 Outlooking of different size of ALWA

Table 5 Physical characteristics of lightweight aggregates

Size	Moisture absorbance (24HR, %)	Specific gravity	Weight/volume ratio	Compressive strength (kg)
0.6 - 1.2 mm	8.4	1.57	0.944	1.7
	(15.4)	(1.64)	(0.954)	(1.8)
1.2 - 1.7 mm	9.0	1.47	0.875	4.1
	(11.9)	(1.66)	(0.938)	(4.7)
1.7 - 2.5 mm	8.5	1.35	0.812	4.8
	(13.2)	(1.61)	(0.921)	(7.9)
2.5 - 3.5 mm	9.2	1.21	0.716	5.3
	(11.1)	(1.60)	(0.902)	(9.7)
Mixture	8.8	1.40	0.868	
	(12.9)	(1.63)	(0.977)	

( ): existing conventional LWA

It was found that our ALWA has a lower specific gravity and slightly less compressive strength. Table 6 shows the compressive strength of cement mortar mixed with ALWA.

Table 6 Compressive strength of cement mortar mixed with ALWA

Test piece including:	Compressive strength (kg/cm^2)	
	7 days	28 days
New-ALWA	244	418
Conventional LWA	282	457
Sand	307	478

Its test showed a compressive strength of 244 kg/cm^2 after eight days and 418 kg/cm^2 after 28 days.

Unlike other conventional products, the ALWA is producible in any grain size from 0.3 to 5 mm in diameter. The ALWA can be used as a filler for plastic board, the base material for tennis courts or as artificial soil for flower pots. In addition, it may be used as material for concrete blocks, heat-insulating panels and other secondary cement products.

Photos 7 through 11 demonstrate some applications of ALWA.

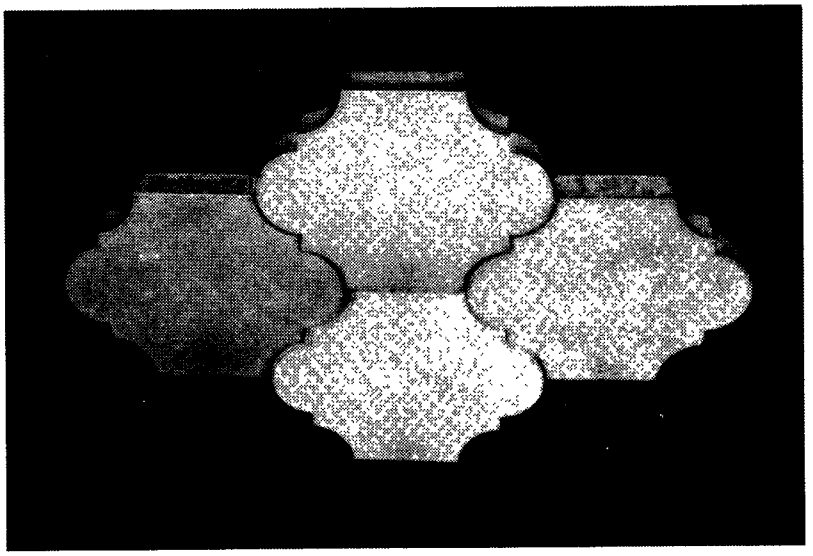

Photo 7 Tile

Photo 8 Flower pot

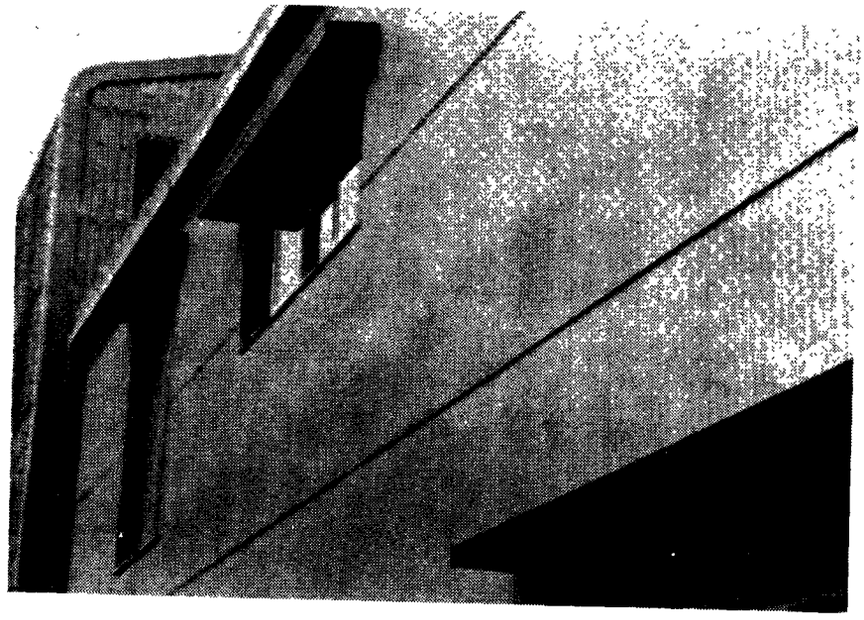

Photo 9 Thermal insulating siding

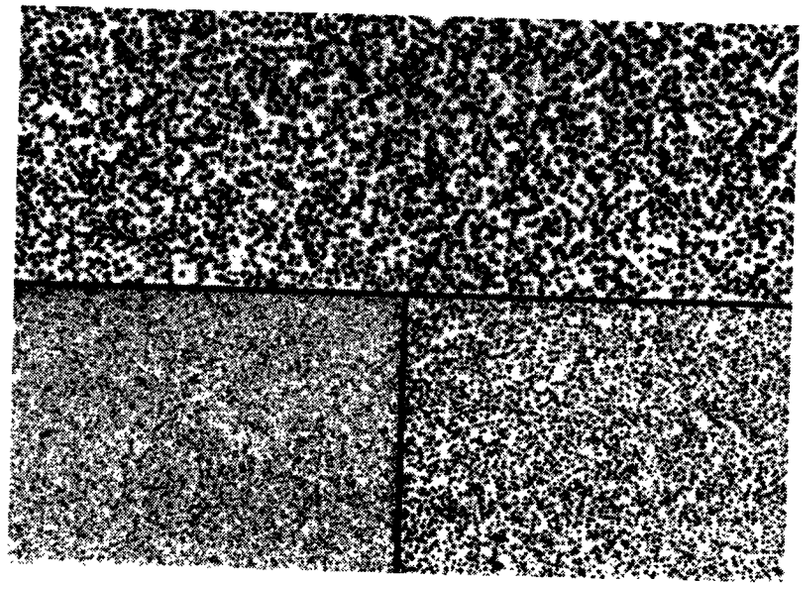

Photo 10 Wall panel

Photo 11 Artificial soil

5. DEMONSTRATION PLANT

A demonstration plant is designed using the findings of the basic studies and it is installed in the Odai sewage plant. Figure 7 shows flow sheet of the demonstration plant. It has an hourly production capacity of 500 kg---probably the smallest practical output. Since July 1983, the plant has been in operation. Both technical and economical feasibility of manufacturing ALWA will be fully evaluated through operational data observed at the demonstration plant. Photos 12 to 22 show the unit processes of the demonstration plant.

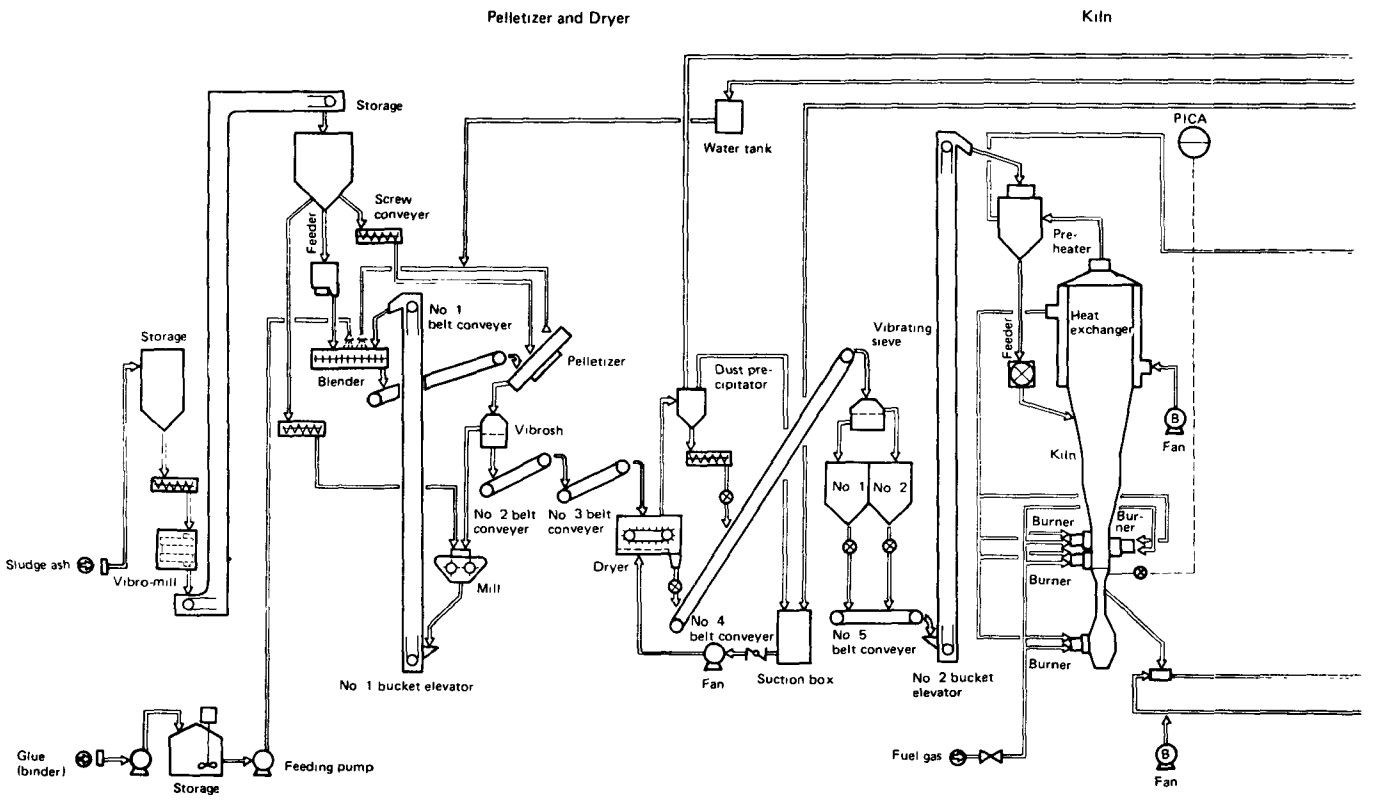

Fig. 7 Flow sheet of LWA manufacturing plant (capacity: 1)

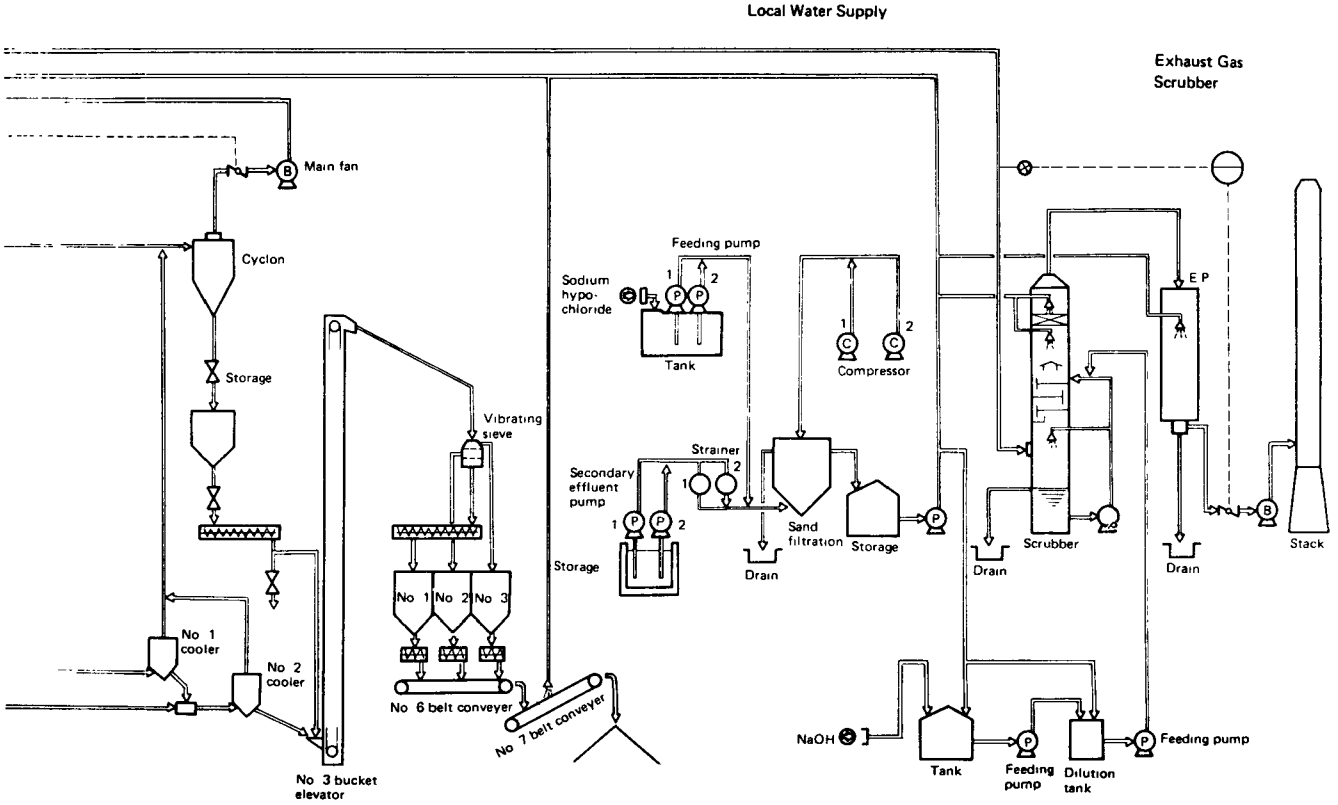

Fig. 7 (Continued) Flow sheet of LWA manufacturing plant (capacity:1)

6. COST

Now the running data of the demonstration plant are accumulated. Later the precise costs for the operation of large scale commercial plants can be demonstrated. At present, only rough cost estimation is available. This is shown on Table 7. In the table, the operational costs include depreciation, interest expense, repair cost, labor cost, fuel cost, and electric charges. In a small plant with a daily output capacity is 12 tons (26,460 lbs.), the production cost is ¥38,590/ton (7.3¢/lb.). The existing similar materials retail at ¥17,000/ton (3.2 ¢/lb.). As it clear from Table 7, ALWA process becomes viable with the existing similar material only when it is produced by a large-scale plant with a daily capacity to 120 tons (264,000 lbs). On the other hand, the cost for disposal of sludge ash averages ¥6,500/ton (1.2 ¢/lb.). The ALWA plant will pay for itself, if the ALWA can be sold at a price which is less 1.2 cents than a net production cost. The ALWA produced at the demonstration plant is more spherical and more uniform in grain size than that of the existing ones in market. Our ALWA must be more attrative. We have already received many inquiries from manufacturers and traders. They wish to be a sole agency.

Another thing to consider is the construction cost of the demonstration plant. As shown in Table 7, the construction cost for the ALWA plant is much higher than that for a similar manufacturing plant. This is because of the tight specifications for sewage works which have been applied. This resulted in over-engineering. The construction cost can be recouped if the engineering level is lowered. The number of operators can be reduced. In any case, the cost for the ALWA will be lowered. We are optimistic of the future of sewage sludge harnessing technology.

Table 7 Initial and running cost estimation of ALA manufacturing plant

Daily capacity (tons) (lbs)	12 26,460	24 52,920	72 158,760	120 264,600 lb
Const. cost (¥) ($)	598,200,000 2,492,500	598,200,000 2,492,500	1,156,470,000 4,819,000	1,570,960,000 6,546,000
Running cost (¥/ton) (¢/lb)	38,590 7.3	25,200 4.8	18,450 3.5	16,760 3.2

Notes

1. 1 US$ = 240 Japanese Yen
2. Running cost includes: labour, repair, amortization, interest, fuel, and electricity
3. Cost of disposing ash: ¥6,500/ton, 1.2¢/lb
4. Retail price of existing light weight aggregater: 3.2¢/lb
5. Number of operator: 10
6. Period of amortization: 7 years
7. Interest: 7.5 % per year
8. Working day: 300 day/year
9. Net production rate: 80%

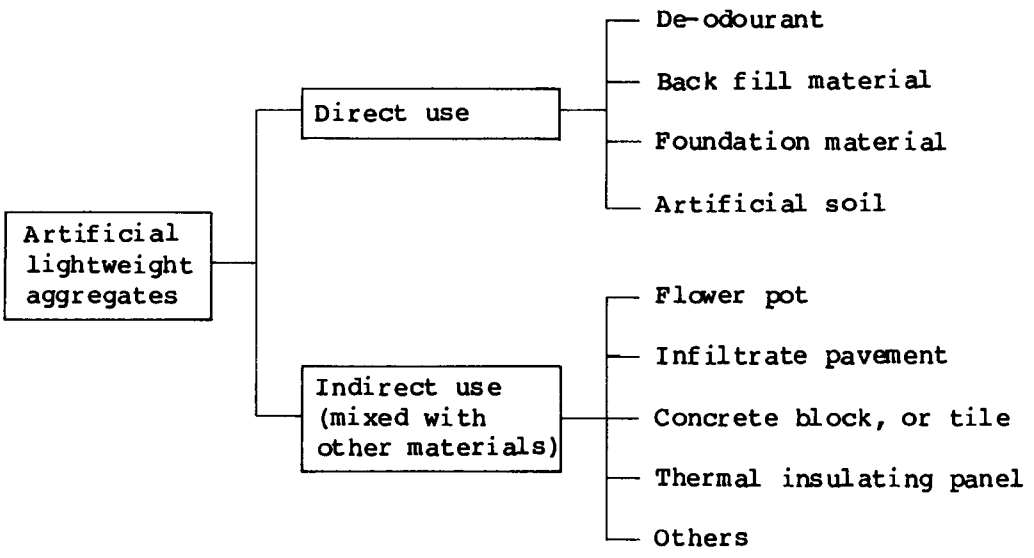

Fig. 8 Potential use of ALWA

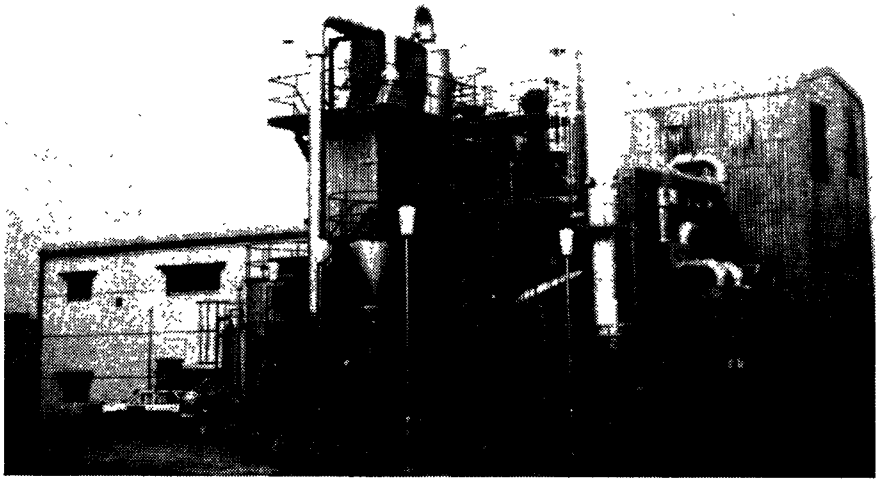

Photo 12 Outlooking of plants

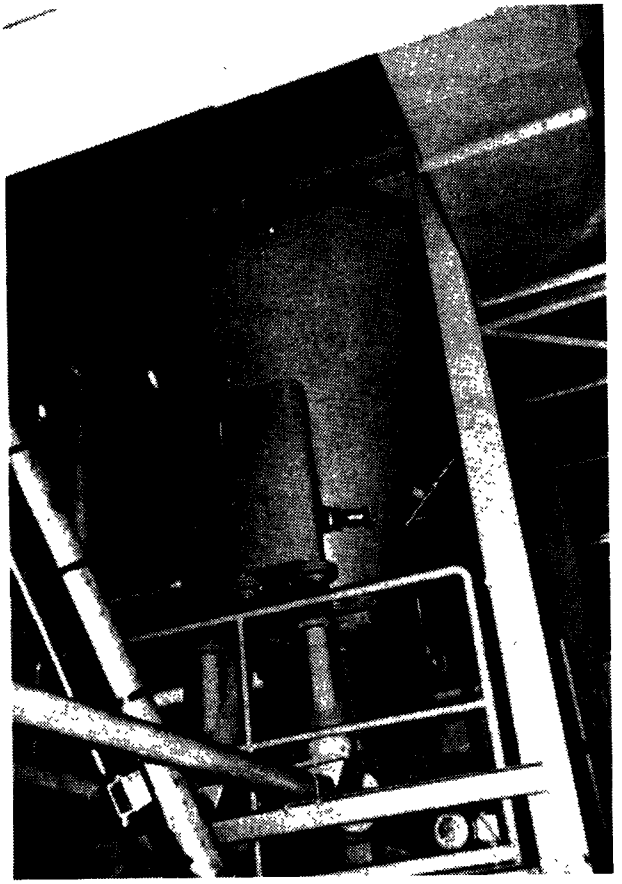

Photo 13 Storage of ash

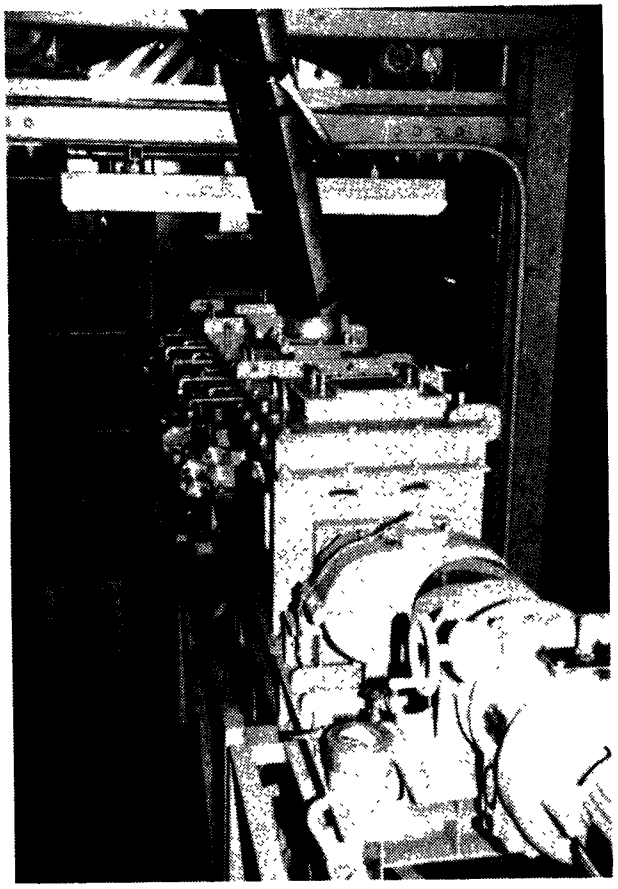

Photo 14 Paddle mixer

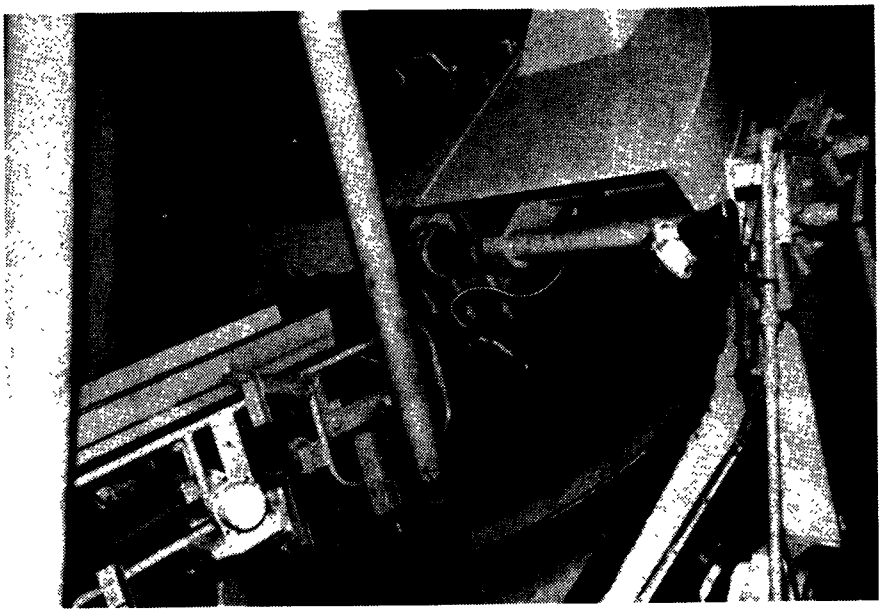

Photo 15 Pelletizer

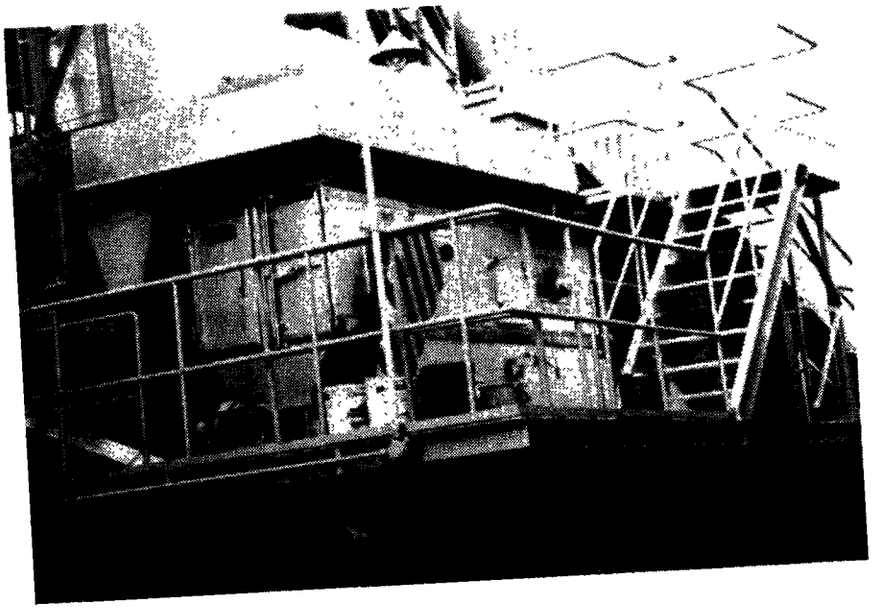

Photo 16 Band dryer

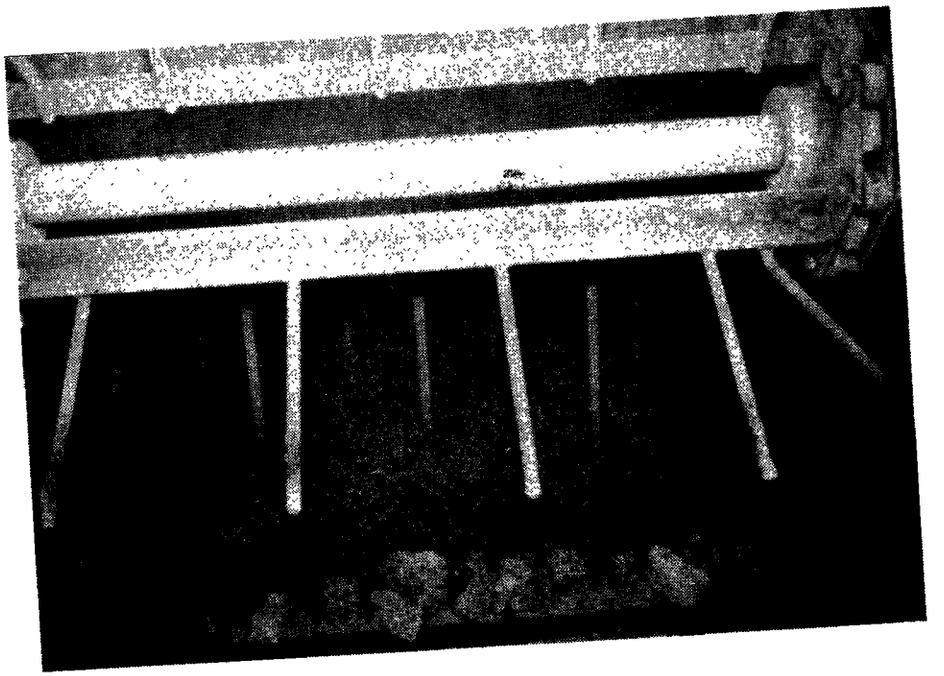

Photo 17 Inside of dryer

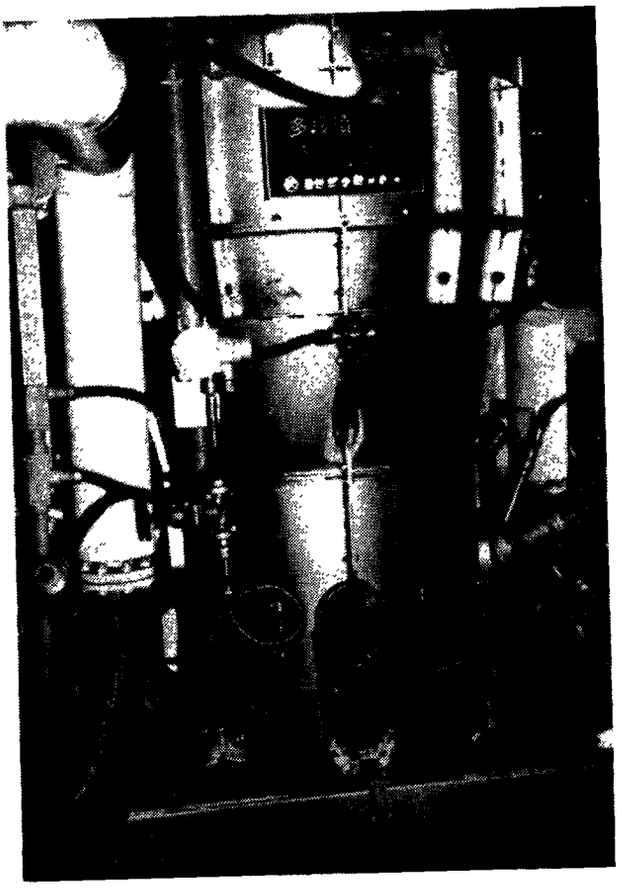

Photo 18 Kiln

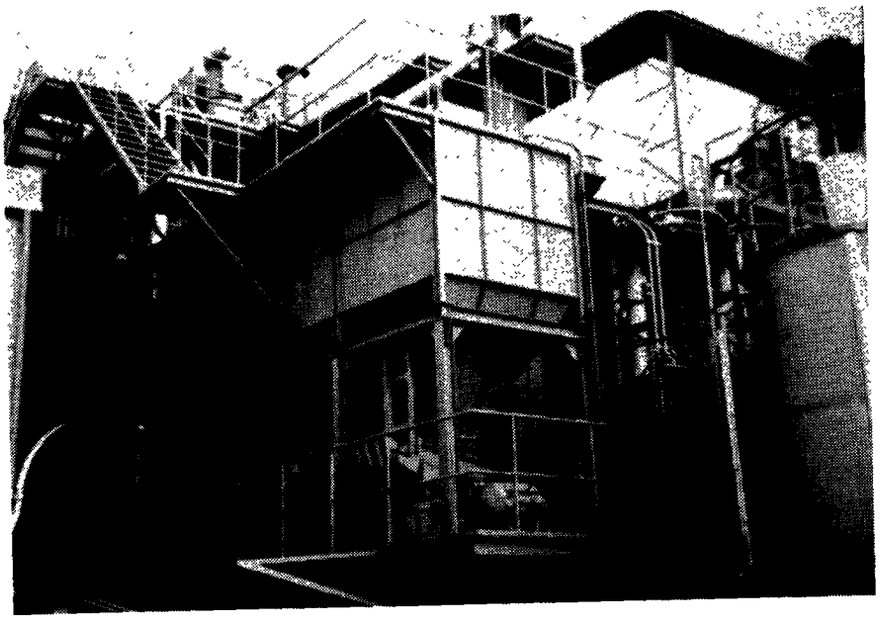

Photo 19 Storage of ALWA

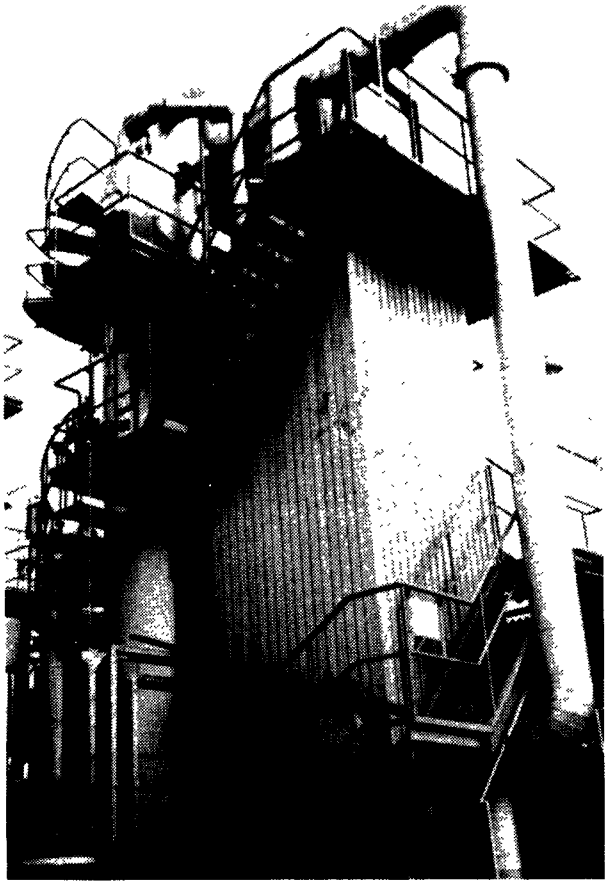

Photo 20 Scrubber and EP

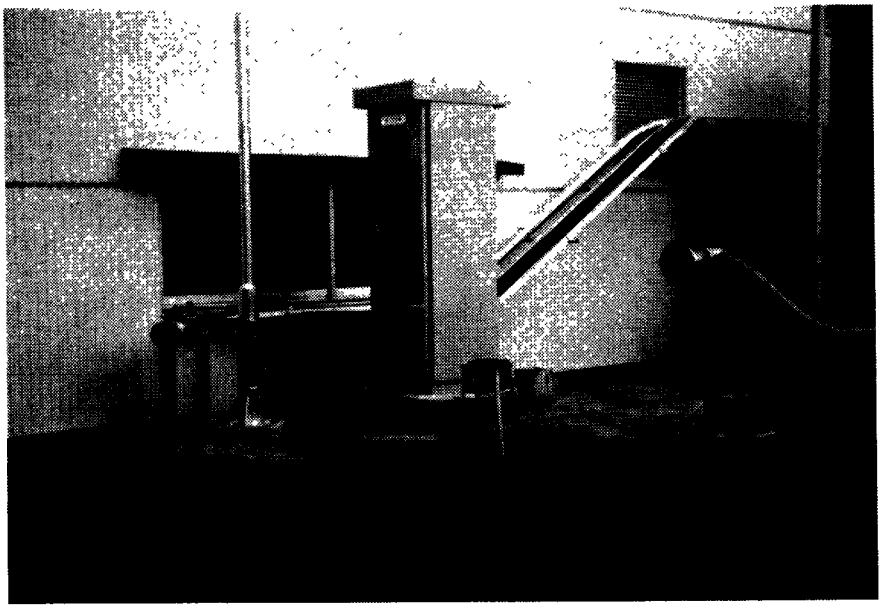

Photo 21 Feeding inlets for ash and binder material

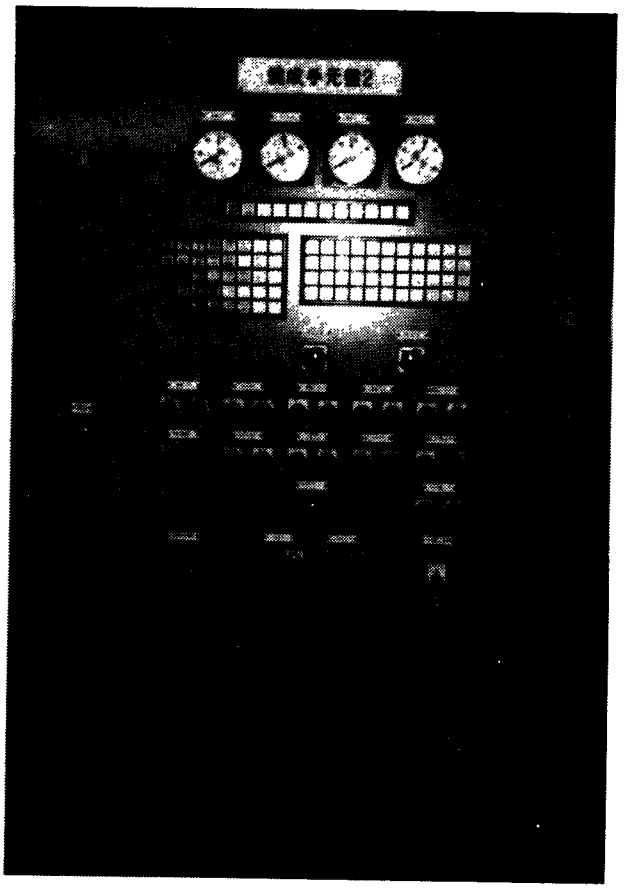

Photo 22 Control panel

Ninth US/Japan Conference
on
Sewage Treatment Technology

SECONDARY SEWAGE TREATMENT USING CONTACT AERATION PROCESS

The work described in this paper was not funded by the U.S. Environmental Protection Agency. The contents do not necessarily reflect the views of the Agency and no official endorsement should be inferred.

September 20, 1983

Tokyo, Japan

Kazunari Matsunaga

Director of Construction Division

Sewage Works Bureau

Osaka City Office

EXPERIMENT OF SECONDARY SEWAGE TREATMENT USING CONTACT AERATION PROCESS

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1. INTRODUCTION

The activated sludge process is highly appreciated at present as a superior method for sewage treatment. However, it has been pointed out that this process has problems in that it requires special techniques for control and maintenance, gives rise to a large quantity of sludge, requires much energy for aeration, and so on. Recently, various wastewater treatment technologies using biofilm have been studied and, based on such studies, a number of treatment processes have been proposed. The major processes are the rotating biological contactor (RBC) process and the contact aeration process in the municipal wastewater treatment. The RBC process is said to take less time to treat sewage, as compared with the activated sludge process, but space saving cannot be expected since the reaction tank is shallow, offering virtually no difference from the activated sludge process.

As a process worthy of study, especially for large cities under space constraint due to high population density, such as in Osaka, the contact aeration process was chosen out of many biological filters, and conducted experiments to evaluate it.

The contact aeration process can be roughly classified into two types: one is to install packing media in the downstream portion of the spiral or circulating flow; the other is to effect multi-line aeration (in which diffusers are spaced uniformly across the entire tank bottom) and to install packing media where bubbles come up. The honeycomb tube contact aeration process — belonging to the former type — was once investigated for effects on tertiary treatment.

This study revealed that tertiary treatment, although performed at a far lower BOD concentration than secondary treatment, often caused clogging of packing media. This clogging necessitated stoppage of operation several times a month to clean the media with air and water.

In the latter aeration type, equipment for conducting multi-line aeration from below the packing media, consisting of synthetic resin nets, has already been developed for treatment of industrial wastewater. This equipment was considered worthy of further study as viable secondary sewage treatment equipment, on the basis of the results of the tertiary treatment experiment conducted by the municipality. Therefore, after the small-scale pilot experiment, another experiment was carried out on this equipment using an actual sewage treatment facility.

The results of this experiment showed that the process could be applied to secondary treatment as an alternative to the activated sludge process. This paper presents the results of these experiments.

This paper also includes some considerations with regard to the possibility of using a fine screen for solid-liquid separation so as to minimize plant site area requirements.

2. CONTACT AERATION EQUIPMENT

2.1 FEATURES AND OPERATING PRINCIPLE OF EQUIPMENT

Fig. 1 shows a conceptual drawing of the contact aeration equipment used in these experiments.

In the aeration tank, plastic net media are arranged in parallel with the flow. The perforated diffuser pipes installed across the flow below the packing media are used for multi-line aeration with the objectives of supplying dissolved oxygen to the sewage and agitating it. Organic substances contained in the sewage are removed by the activity of the microorganisms adhering to the packing media. The microorganisms accumulated on the media surface are detached while the sewage is agitated by means of aeration, and undergo solid-liquid separation in the final settling tank.

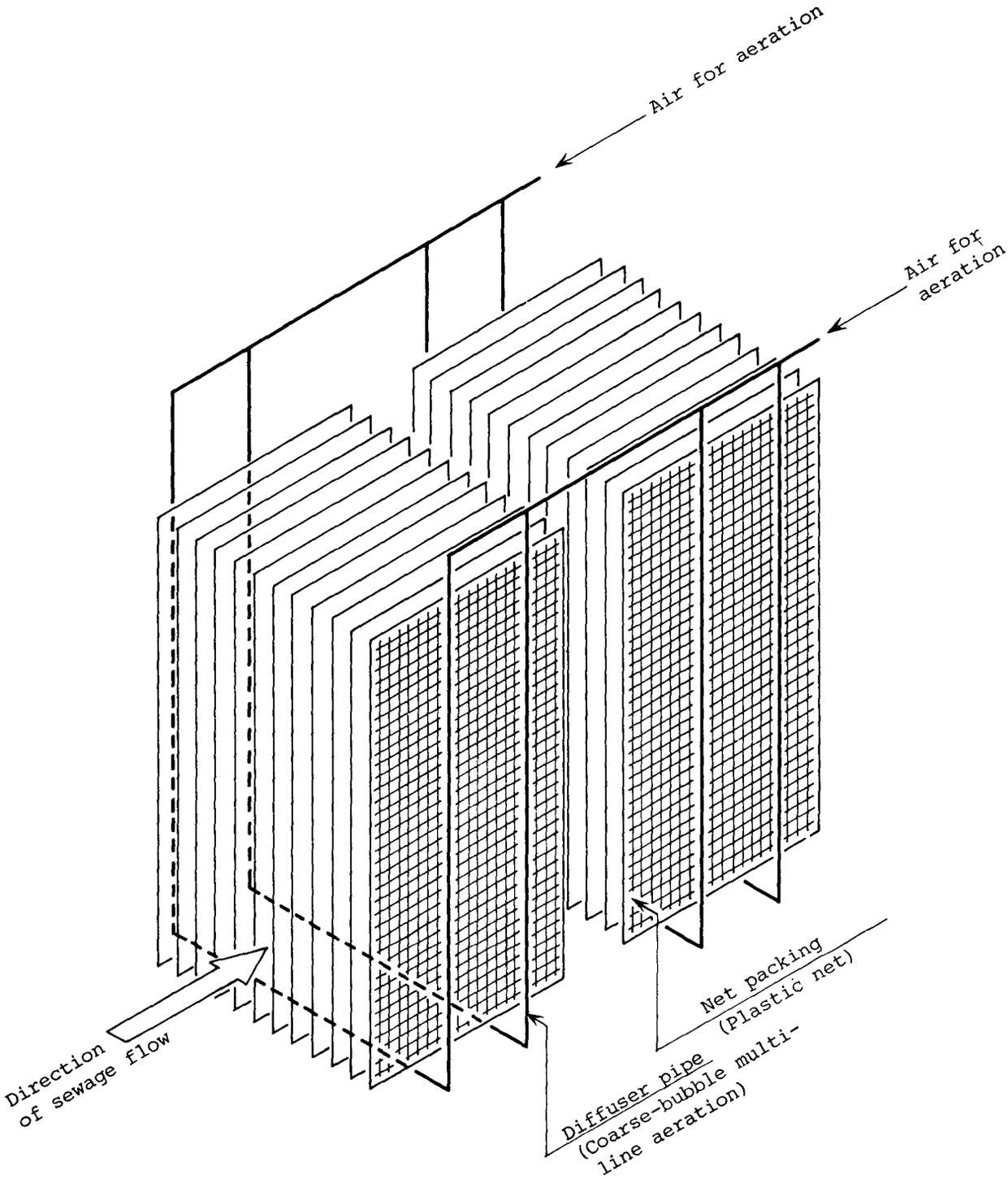


Fig. 1 Conceptual Drawing of Contact Aeration Equipment

The major trouble with the contact aeration process is the clogging of the packing media. In this equipment, the clogging trouble is eliminated by using net media and renewing accumulated microorganisms through increased agitation intensity by means of aeration from below the packing media.

The increase in agitation intensity is also expected to promote substrate removal by microorganisms accumulated on the packing media.

2.2 FULL-SCALE EXPERIMENTAL EQUIPMENT

The full-scale experiment was carried out, using a facility consisting of aeration tanks and a final settling tank, at the Suminoe Sewage Treatment Plant in Osaka City. The aeration tank had been designed for spiral-flow step aeration using diffuser plates. The Suminoe Sewage Treatment Plant has a treatment capacity of 220,000m^3/day. Since only about 5% of the influent at this treatment plant is industrial wastewater, this plant has the higher percentage of domestic wastewater among the 12 treatment plants in the city.

The layout of the experimental equipment is shown in Fig. 2. Out of the four aeration tanks constituting the experimental system, the latter two tanks were used for installation of packing media. The contact aeration tank process is preceded by the primary sedimentation and followed by the final settling where solid-liquid separation was carried out.

Photo 1 shows the experimental equipment under operation.

Table 1 shows the specifications of the equipment.

The contact aeration tank was composed of net media, their supports and the aeration device. The packing media consisting of polyethylene nets of 13mm meshes were reinforced with stainless steel binding frames and fixed to the supports at predetermined intervals. The nets were arranged 40 $\sim$ 100mm apart in parallel with the flow. (Photo 2)

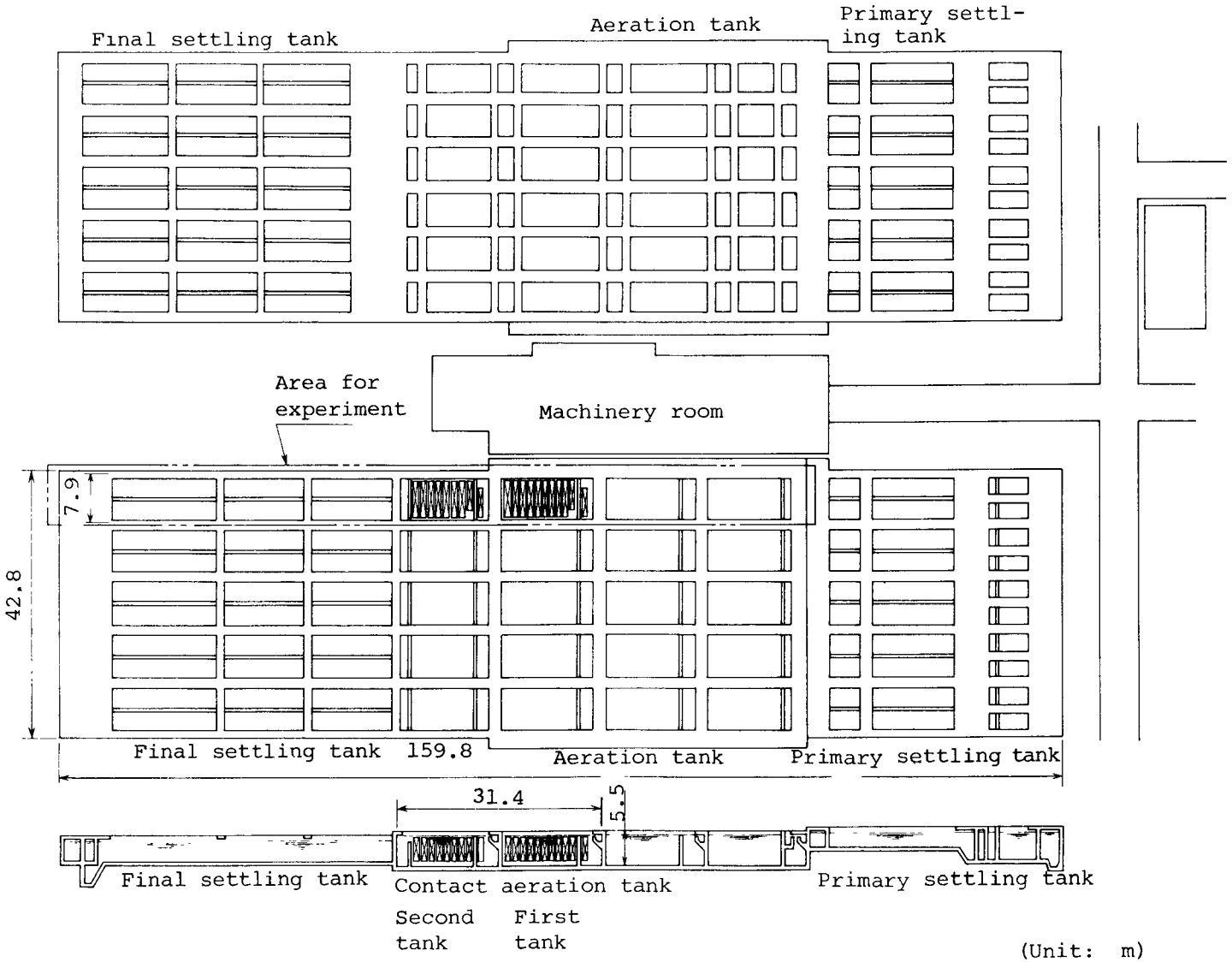


Fig. 2 Layout of Experimental Equipment

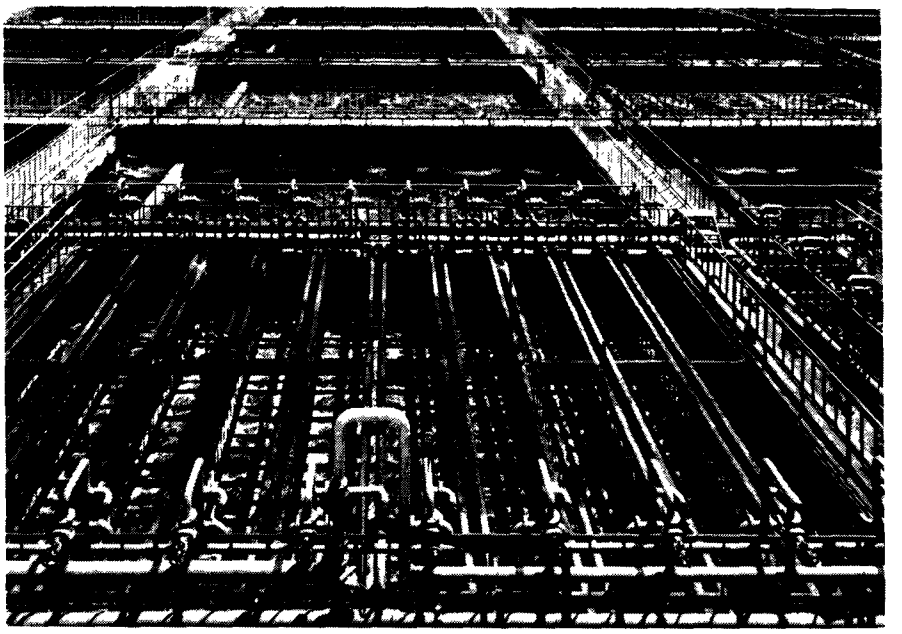


Photo 1 General Views of Contact Aeration Tanks

Table 1 Specifications of Experimental Contact Aeration Equipment

Effective capacity of aeration tank		:	$860m^3$
Packing media capacity		:	$500m^3$
Packing media	Material	:	Polyethylene
	Shape	:	Net
	Mesh	:	13mm
Installation intervals			
	Inlet	:	100mm
	Middle	:	60mm
	Rear	:	40mm
Diffuser device	Pipes with perforation	:	(ø2mm; 40 ∿ 60mm pitch)
Water surface area of final settling tank		:	$370m^2$

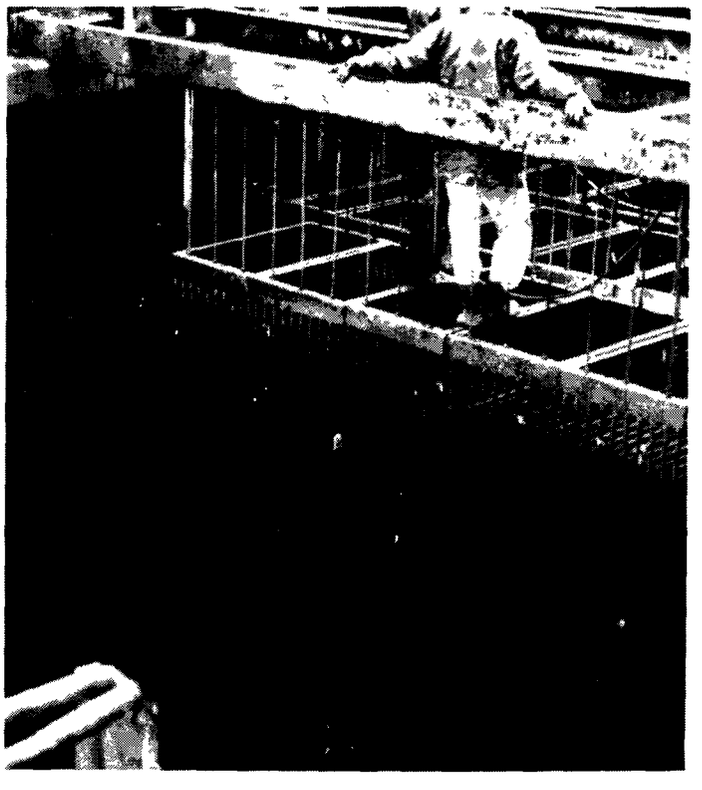


Photo 2 Packing Media under Construction

The volume occupied by the installed packing media was approximately 60% of the total effective volume ($860m^3$) of the tank.

The aeration device consisted of steel risers and stainless steel diffuser pipes. The diffuser was of the coarse bubble aeration type, with 2mm perforations.

For control of air supply, four flow meters were installed. Air supply was designed to be large in the inlet region under high BOD load and taper off toward the outlet. The flow rate of the effluent was measured with a weir-type flow meter installed at the effluent channel in the final settling tank.

3. RESULTS OF OPERATION

3.1 RESULTS OF NORMAL OPERATION

In order to study whether this process is applicable to secondary sewage treatment as an alternative to the activated sludge process, experiments were continued under similar operating conditions throughout a year, with a 4 ~ 5-hour aeration time and an air-water ratio of 5 ~ 6. Fig. 3 shows the average operating conditions for individual months. During the experimental period, the overflow rate in the final settling tank was small 10 ~ $15m^3/m^2 \cdot day$. As for hourly fluctuation of influent sewage, the ratio of maximum to minimum flow was 1.5 ~ 2, or within 50% of the daily average flow rate.

Water quality was measured for a composite sample taken 3 times during the daytime twice a week. The monthly average water quality is shown in Table 2. The values concerning water temperature, BOD and SS are converted into graphic forms in Fig. 4. During the period of measurement, the concentration of dissolved oxygen at the outlet of the aeration tank was 2 ~ 4mg/ℓ.

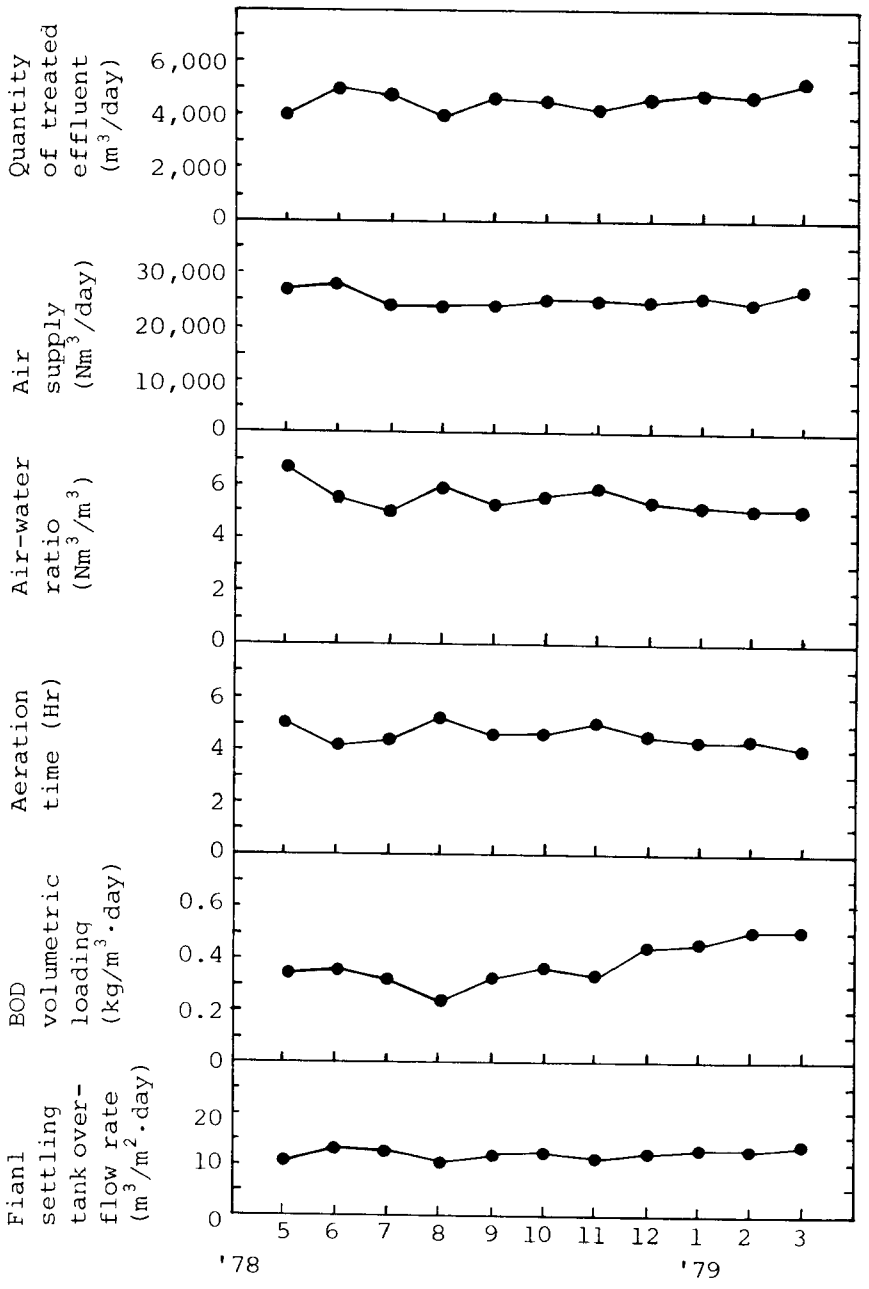


Fig. 3 Operating Conditions (Monthly Average)

Table 2 Monthly Results of Normal Operation

Values in parentheses indicate standard deviation.

		Unit	May 1978	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.	Jan. 1979	Feb.	Mar.
Water temperature	Influent	°C	20.8 (1.1)	23.4 (1.3)	27.3 (1.1)	28.6 (0.3)	26.3 (1.1)	21.0 (1.4)	17.9 (1.4)	15.6 (0.6)	12.7 (0.9)	12.9 (1.0)	13.0 (0.4)
pH	Influent	—	7.31 (0.19)	7.26 (0.18)	7.22 (0.12)	7.08 (0.17)	7.03 (0.21)	7.12 (0.20)	7.18 (0.30)	7.24 (0.25)	7.44 (0.10)	7.44 (0.10)	7.36 (0.13)
	Effluent	—	7.43 (0.11)	7.29 (0.11)	7.29 (0.04)	7.24 (0.09)	7.05 (0.22)	7.12 (0.23)	7.25 (0.21)	7.22 (0.13)	7.40 (0.08)	7.33 (0.08)	7.33 (0.07)
Transparency	Influent	cm	6.0 (1.4)	6.5 (1.1)	7.0 (1.3)	7.6 (1.6)	6.3 (0.8)	6.5 (1.3)	7.8 (4.3)	7.3 (1.9)	9.9 (1.8)	5.5 (1.6)	6.5 (2.1)
	Effluent	cm	34 (11)	35 (8)	44 (7)	> 50 (9)	45 (9)	31 (12)	40 (13)	42 (10)	46 (9)	38 (6.6)	40 (6.9)
SS	Influent	mg/ℓ	86.0 (14.0)	53.8 (6.4)	53.7 (13.5)	60.2 (13.6)	70.9 (16.9)	71.9 (18.8)	93.7 (43.2)	78.5 (23.6)	49.8 (17.4)	114 (36)	78.8 (34.4)
	Effluent	mg/ℓ	12.0 (1.0)	8.7 (0.4)	7.6 (1.9)	8.4 (2.0)	10.0 (3.4)	13.1 (4.9)	11.8 (5.9)	9.4 (3.6)	7.8 (2.9)	12.8 (5.6)	9.4 (2.4)
$T\text{-}BOD_5$	Influent	mg/ℓ	72.0 (9.9)	61.2 (11.7)	56.7 (6.8)	50.8 (7.6)	58.8 (10.6)	67.1 (6.5)	74.2 (19.6)	80.0 (1.8)	73.0 (9.9)	84.1 (10.0)	82.4 (8.2)
	Effluent	mg/ℓ	8.8 (2.1)	9.8 (0.8)	8.1 (2.2)	9.8 (1.4)	9.2 (1.6)	10.4 (1.5)	8.7 (1.4)	9.5 (3.9)	9.7 (0.5)	11.4 (1.0)	11.2 (0.7)
	Removal	%	87.8	84.0	84.3	80.7	84.4	84.5	88.3	88.1	86.7	86.4	86.4
$S\text{-}BOD_5$	Influent	mg/ℓ	37.6 (9.0)	37.1 (8.5)	35.7 (7.1)	33.8 (7.3)	37.0 (3.4)	46.4 (5.2)	53.5 (18.7)	55.6 (2.4)	57.5 (9.7)	52.5 (7.2)	50.0 (2.8)
	Effluent	mg/ℓ	5.4 (2.1)	7.0 (0.6)	5.4 (0.7)	6.4 (2.0)	6.7 (0.8)	7.5 (0.7)	5.8 (1.5)	6.3 (2.9)	6.6 (0.6)	7.0 (2.0)	8.0 (1.5)
	Removal	%	85.6	81.1	84.9	81.1	81.9	83.8	89.2	88.7	88.5	86.7	84.0
$T\text{-}COD_{Mn}$	Influent	mg/ℓ	63.9 (11.5)	43.9 (4.0)	38.5 (3.6)	44.8 (5.1)	48.5 (2.2)	50.7 (3.4)	53.2 (7.7)	57.5 (2.5)	51.5 (7.3)	55.9 (8.8)	60.3 (5.1)
	Effluent	mg/ℓ	19.4 (1.6)	17.6 (1.9)	16.4 (2.4)	18.9 (3.9)	19.8 (1.1)	18.8 (1.6)	17.9 (1.8)	19.1 (2.3)	17.5 (2.1)	17.0 (3.2)	19.5 (0.9)
	Removal	%	69.6	59.9	57.4	57.8	59.2	62.9	66.4	66.8	66.0	69.6	67.7
$S\text{-}COD_{Mn}$	Influent	mg/ℓ	36.0 (7.9)	33.1 (4.0)	30.2 (3.1)	33.7 (3.3)	33.8 (2.0)	40.5 (2.6)	44.1 (11.1)	47.3 (2.7)	41.8 (5.7)	42.5 (5.2)	47.2 (3.9)
	Effluent	mg/ℓ	16.2 (2.4)	15.9 (1.6)	15.4 (2.1)	17.0 (2.4)	17.8 (1.1)	17.3 (1.2)	17.0 (1.9)	17.8 (4.5)	15.9 (2.6)	14.7 (2.5)	16.7 (1.4)
	Removal	%	55.0	52.7	49.0	49.6	47.3	57.3	61.5	62.4	62.0	65.4	64.6
S-TOD	Influent	mg/ℓ	226 (28)	161 (51)	148 (21)	158 (23)	166 (25)	160 (34)	178 (36)	178 (24)	185 (31)	183 (22)	193 (19)
	Effluent	mg/ℓ	106 (20)	84 (27)	75 (11)	72 (10)	71 (15)	68 (14)	69 (14)	81 (13)	80 (11)	86 (18)	87 (17)
	Removal	%	53.1	47.8	49.3	54.4	57.2	57.5	61.2	45.5	56.8	47.0	54.9
S-TC	Influent	mg/ℓ	85 (8)	67 (17)	68 (9)	68 (10)	71 (8)	74 (7)	75 (10)	75 (8)	79 (10)	73 (8)	75 (5)
	Effluent	mg/ℓ	54 (8)	47 (11)	45 (5)	43 (6)	41 (7)	44 (7)	42 (7)	47 (7)	52 (6)	51 (8)	49 (5)
	Removal	%	36.5	30.0	33.8	36.8	42.3	40.5	44.0	37.3	34.2	30.1	34.7
$NH_3\text{-}N$	Influent	mg/ℓ	18.8 (2.2)	19.4 (4.1)	15.5 (6.3)	15.1 (6.3)	17.0 (2.2)	16.9 (3.0)	18.7 (4.0)	15.1 (0.9)	19.9 (4.4)	19.6 (4.3)	19.7 (2.6)
	Effluent	mg/ℓ	19.6 (1.8)	17.7 (4.3)	14.4 (2.2)	12.4 (4.4)	15.1 (3.1)	15.6 (3.2)	17.1 (5.9)	15.9 (4.8)	20.8 (3.6)	17.4 (5.3)	17.6 (1.5)

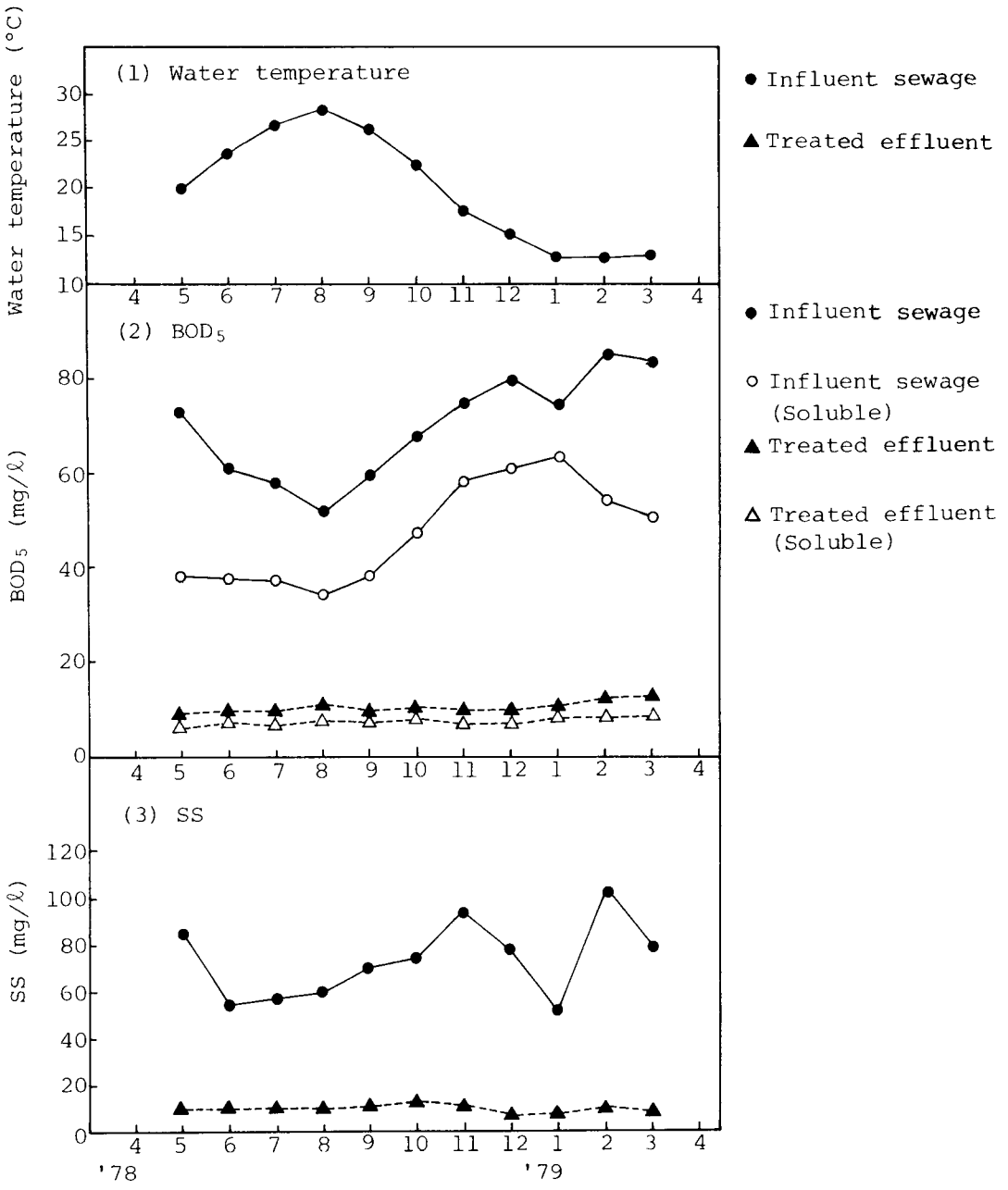


Fig. 4 Monthly Change in Quality of Influent and Effluent

The quality of effluent was stable, with both BOD and SS being at approximately 10mg/ℓ. These values were not significantly different from the values for effluent treated by the activated sludge process during the same period under lighter BOD volumetric loading. However, according to visual observation, the effluent treated with this process was less transparent than that treated with the activated sludge process, appearing slightly opaque.

Despite a water temperature difference of approximately 15°C between summer and winter, no significant difference was observed in the quality of the effluent. It is thought that the equipment had surplus capacity, since operation was carried out with relatively low BOD volumetric loading.

Virtually no nitrification occurred throughout the experimental period. This is probably because the quantity of air supplied for aeration was not enough to initiate a nitrification reaction in the latter half of the tank where nitrification may advance, since a large quantity of air was supplied at the inlet of the aeration tank to remove organic substrates such as BOD, while the air quantity was limited on the outlet side.

An understanding of the quantity of microorganisms adhering to the packing media is essential for this biological treatment process. Nevertheless, no easy method of quantification was found.

In this experiment, test pieces were placed at 6 locations inside the tanks and the quantity of microorganisms in one entire tank was estimated based on the amount of microorganisms attached to the 6 test pieces.

The test pieces used were the same packing media as installed in the tank, measuring 220cm × 45cm. The test pieces were taken out of the tank in summer and winter to determine the quantity of microorganisms in the tank as a whole, based on the dry weight

of the biofilm separated from the test pieces. The quantity of microorganisms per tank capacity was approximately 2,500mg/ℓ in summer and approximately 3,200mg/ℓ in winter. These values are similar to the MLSS concentration for the activated sludge process.

3.2 EFFECT OF LOAD FLUCTUATION

A sewage treatment plant must achieve stable and high effluent quality. For that, BOD loading should be kept constant at all times. In the actual plants, however, hourly load fluctuation is unavoidable.

Generally speaking, sewage treatment using biological filters is superior to the activated sludge process in coping with load fluctuation. Hence, an attempt was made to compare the effect of load fluctuation between the two processes.

The experiment on the activated sludge process was carried out in an existing facility next to the one used in the experimentation on the contact aeration process. The facility was made operable independently, with the addition of a sludge return device.

The experiment on load fluctuation was carried out by artificially causing the flow to fluctuate according to a pattern with a peak during the daytime, as shown in Fig. 5. In this experiment, the daily average flow was kept constant and operation targets were set at three stages, with the ratios of hourly peak flow to daily average flow at 1, 2 and 2.5.

At each ratio, operation was continued for 1 $\sim$ 2 weeks, and in the latter half of the experiment, fluctuation was measured every two hours for a 24-hour period. Runs were made twice a year — in autumn and winter. The results of the runs are shown in Tables 3 and 4, for the contact aeration process and the activated sludge process, respectively.

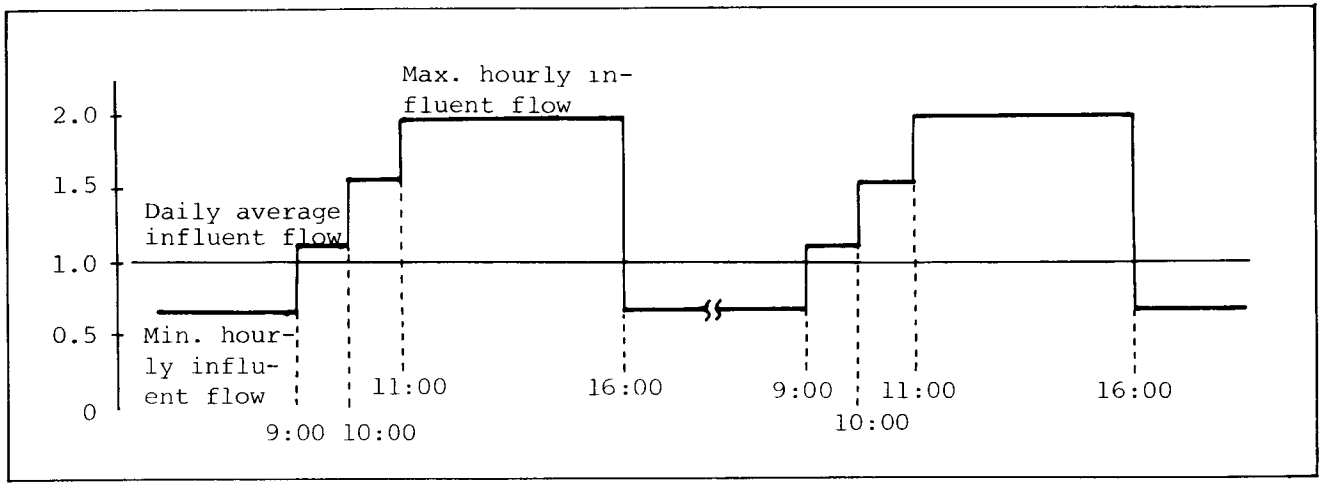


Fig. 5 Load Fluctuation Pattern (When $\frac{\text{Max. hourly influent flow}}{\text{Daily average influent flow}} = 2.0$)

The typical examples of hourly change in BOD volumetric loading and BOD concentration of the effluent under different operating conditions are given in Figs. 6 through 9.

As an actual facility was employed for this measurement, it was impossible to apply exactly the same BOD loading and aeration time to every experiment. Therefore, most data were gathered for an aeration time of about 6.5 hours. The target aeration time of 6.5 hours is the same as the design value for the activated sludge process now employed in Osaka City.

Tables 3 and 4 show that the daily average BOD can be kept below 20mg/ℓ for both contact aeration process and activated sludge process, if the fluctuation is within 2.4 ~ 2.5 times the daily average influent flow rate.

As long as the flow fluctuation is small, both processes produce effluent with little fluctuation over time, as shown in Figs. 6 and 8. However, if the hourly fluctuation becomes large, the BOD of the effluent temporarily exceeds 20mg/ℓ, even if the daily average BOD is below 20mg/ℓ. The reason: in the contact aeration process, soluble BOD remains, and in the activated sludge process, SS flows out of the final settling tank despite the fact that soluble BOD is stabilized at a low value.

Judging from the fact that soluble BOD shows a stabler, lower value in the activated sludge process than in the contact aeration process, the former process is superior to the latter in its capacity to remove substrate in the aeration tank when BOD load fluctuatin occurs due to flow fluctuation.

With regard to solid-liquid separation, comparison between the activated sludge process and the contact aeration process is practically impossible, since overflow rates in the final settling tanks are different.

Table 3 Treatment Performance of Contact Aeration Process (Fluctuation with Time)

Period	Qmax/Qave	Average Water Temperature (°C)	Daily BOD Loading (kg/m³·day)	Aeration Time (hr)	Settling Tank Over-flow Rate (m³/m²·day)	Treated Effluent (mg/ℓ) T-BOD	S-BOD	SS	Remarks
Autumn (Oct.∿ Nov.)	1.20	21.8	0.19∿0.43 (0.32)	5.73∿8.60 (6.8)	6.6∿9.7 (8.1)	7.5∿10.2 (8.7)	6.4∿9.2 (7.7)	2.8∿8.0 (5.9)	Constant air supply
	2.04	23.0	0.27∿0.79 (0.40)	3.03∿8.87 (6.1)	4.9∿18.8 (9.2)	10.5∿17.1 (13.1)	7.8∿12.1 (10.1)	6.0∿15.0 (8.8)	"
	2.50	21.2	0.12∿0.99 (0.34)	2.55∿12.30 (6.4)	3.5∿22.3 (8.9)	11.2∿25.9 (16.4)	6.8∿22.3 (12.0)	6.0∿17.3 (10.8)	"
	3.00	20.5	0.08∿0.95 (0.31)	2.53∿28.70 (7.8)	1.3∿22.3 (7.4)	9.6∿15.5 (12.6)	5.4∿9.8 (7.6)	7.2∿13.2 (9.2)	Proportional air supply
Winter (Jan.∿ Mar.)	1.30	11.8	0.35∿0.45 (0.39)	5.21∿6.88 (6.1)	7.4∿11.6 (8.9)	8.8∿13.6 (12.2)	7.3∿9.9 (8.4)	11.3∿19.0 (15.7)	Constant air supply
	2.01	11.2	0.26∿0.75 (0.36)	3.20∿9.25 (6.6)	5.0∿18.1 (9.0)	8.5∿27.2 (16.2)	6.5∿23.5 (14.1)	3.4∿19.6 (10.2)	"
	2.80	12.3	0.19∿1.00 (0.39)	2.41∿12.60 (6.2)	4.1∿25.2 (9.0)	14.5∿35.2 (21.4)	11.0∿24.7 (13.9)	13.0∿20.0 (15.1)	"
	2.20	12.7	0.24∿0.99 (0.44)	2.44∿10.10 (5.4)	5.5∿22.8 (10.3)	9.3∿27.7 (15.5)	6.2∿18.9 (9.3)	10.0∿32.7 (17.8)	Proportional air supply

Notes: 1. Values in parentheses indicate the average of all data.

2. Air volume was 5 times the daily average flow of influent sewage under each experimental condition.

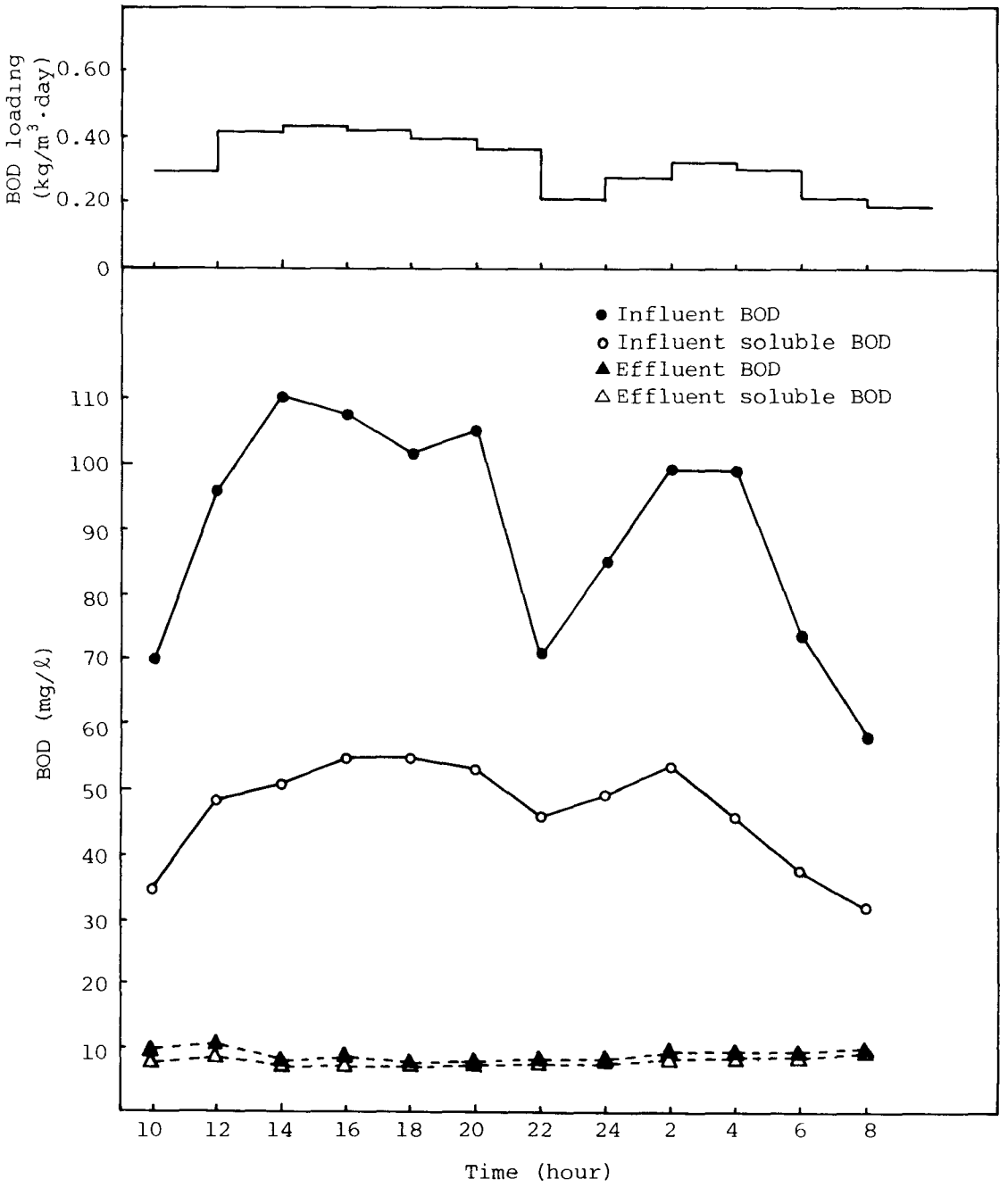
Table 4 Treament Performance of Standard Activated Sludge Process

(Fluctuation with Time)

Period	Qmax/Qave	Average Water Temperature (°C)	Daily BOD Loading (kg/m³·day)	Aeration Time (hr)	MLSS (mg/ℓ)	Settling Tank Over-flow Rate (m³/m²·day)	Treated Effluent (mg/ℓ)			Remarks
							T-BOD	S-BOD	SS	
Autumn (Oct.∿ Nov.)	1.12	25.1	0.19∿0.29 (0.24)	6.08∿7.00 (6.84)	1,600∿2,130 (1,905)	18.7∿21.6 (19.2)	3.6∿6.2 (4.9)	1.1∿2.6 (1.9)	1.0∿9.4 (5.3)	Constant air supply
	2.41	22.3	0.14∿0.98 (0.34)	2.60∿11.73 (6.28)	1,270∿1,610 (1,505)	11.2∿50.5 (20.9)	5.1∿41.6 (18.8)	1.4∿7.0 (3.4)	7.3∿25.5 (14.8)	"
	2.54	21.5	0.10∿0.75 (0.25)	2.60∿13.53 (6.61)	1,060∿2,190 (1,500)	9.7∿50.5 (19.9)	17.0∿37.7 (27.7)	1.9∿6.0 (3.5)	15.8∿57.1 (31.5)	Proportional air supply
Winter (Jan.∿ Feb.)	1.06	12.7	0.28∿0.42 (0.32)	6.15∿7.25 (6.55)	1,700∿1,910 (1,835)	18.1∿21.3 (20.1)	7.6∿11.5 (9.3)	1.2∿5.2 (3.5)	6.4∿17.2 (8.8)	Constant air supply
	2.10	12.8	0.21∿0.64 (0.33)	3.25∿9.95 (6.55)	1,150∿1,890 (1,418)	13.2∿40.4 (20.1)	5.3∿14.3 (9.2)	1.4∿6.5 (3.7)	4.8∿11.0 (7.0)	"
	2.43	12.7	0.18∿0.92 (0.37)	2.60∿11.15 (6.32)	1,830∿1,950 (1,890)	11.8∿50.5 (20.8)	4.0∿32.9 (11.6)	1.1∿6.7 (3.3)	3.8∿58.0 (12.6)	"

Notes: 1. Values in parentheses indicate the average of all data.

2. Air volume was 5 times the daily average flow of influent sewage under each experimental condition.



Max. hourly flow/daily average flow = 1.20
(Water temperature: 21.8°C)

Fig. 6 Change with Little Load Fluctuation
(Contact Aeration Process)

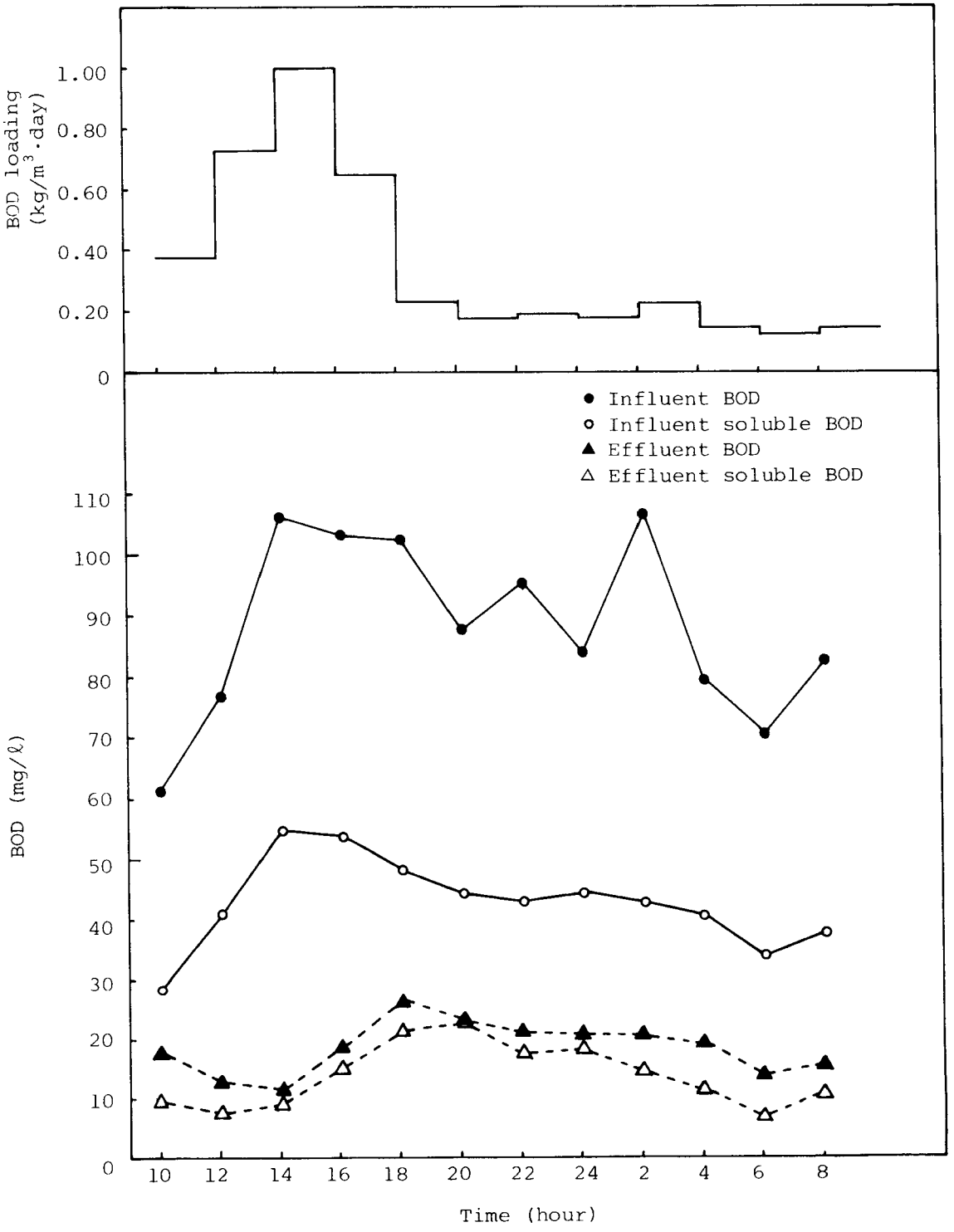
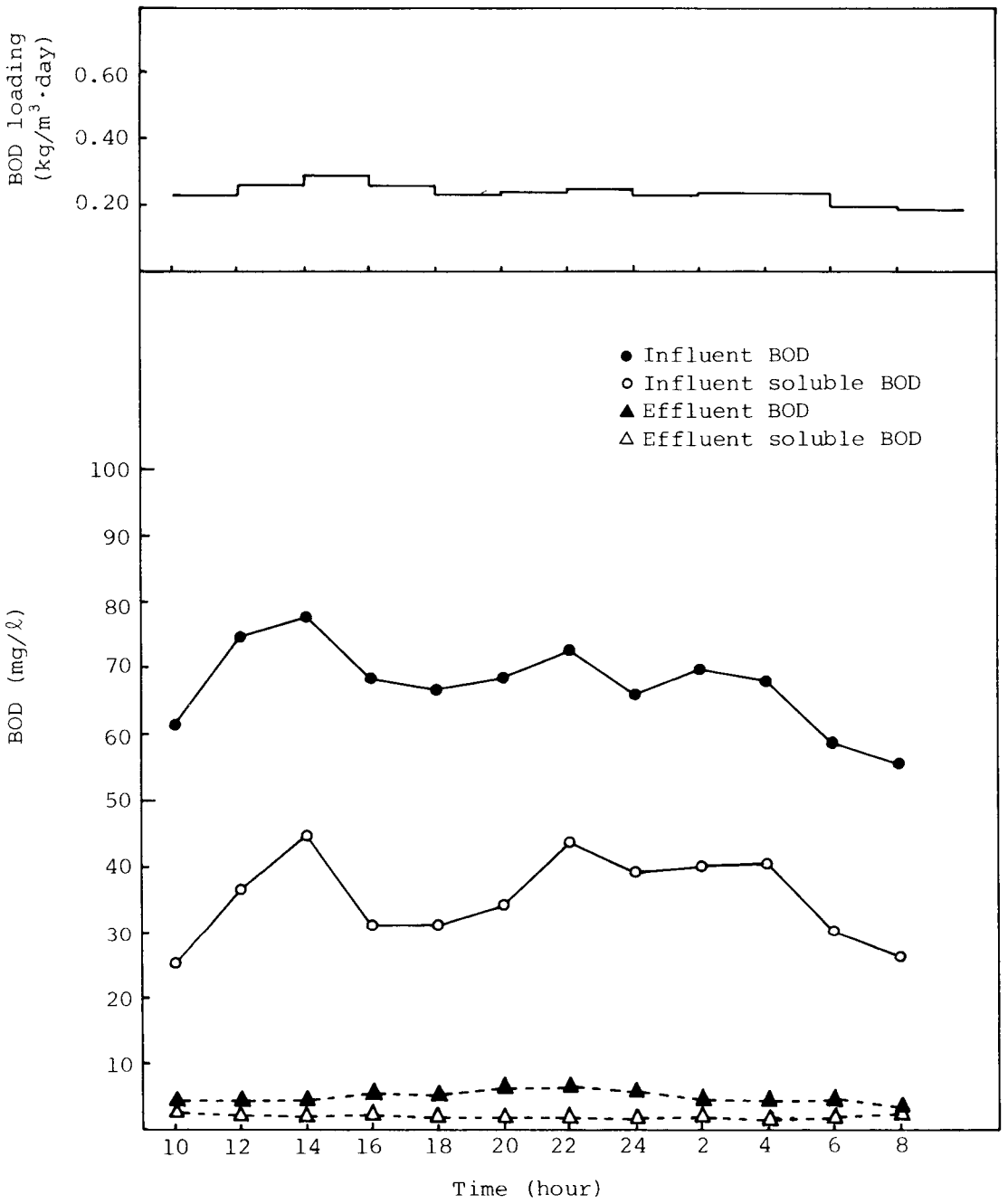
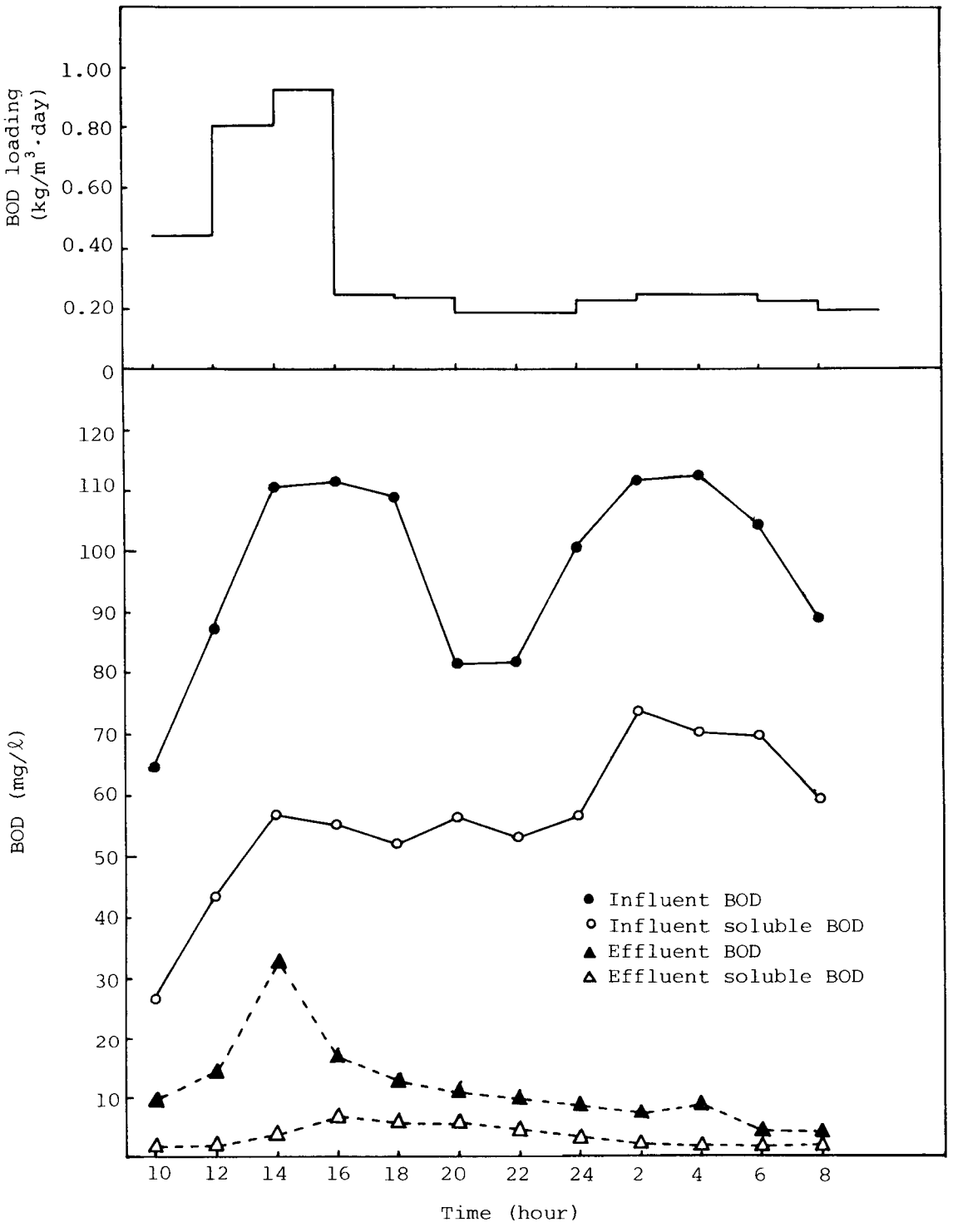


Fig. 7 Change with Load Fluctuation (Contact Aeration Process)



Max. hourly flow/daily average flow = 1.12
(Water temperature: 24.8°C)

Fig. 8 Change with Little Load Fluctuation
(Activated Sludge Process)



Max. hourly flow/daily average flow = 2.43
(Water temperature: 12.6°C)

Fig. 9 Change with Load Fluctuation
(Activated Sludge Process)

Table 3 shows the results of an experiment in which the quantity of air supply for aeration was changed in proportion to influent flow fluctuation. When air supply was made proportional to flow rate, the effluent quality tended to improve, as compared with effluent from equipment operated with a constant air supply.

This is probably because the rate of substrate removal by microorganisms is increased by the reduction in thickness of the liquid film on the surface of the biofilm, when the quantity of air supplied is increased during the daytime as the equipment receives a large quantity of influent with high BOD load, thereby increasing the agitation intensity on the surface of the packing media.

3.3 MAINTENANCE AND CONTROL

Since the present process requires no sludge return, the quantity of air supply is the sole operating factor. In addition, no particular care need be taken as to sludge blanket depth in the final settling tank, due to low SS concentration of influent flowing into the tank. Therefore, this process is believed to be easier to maintain and control than the activated sludge process. Another advantage with regard to maintenance and control is that this process is free of the bulking which is typical of the activated sludge process.

The reduction of operating factors means, on the other hand, that once the equipment is built, it is not capable of responding to changes in circumstances through operations. This is why more than enough research must be carried out and appropriate considerations introduced at the design stage.

Particularly, the air flow rate, the sole operating factor in this process, must be determined so as to permit uniform formation of air bubbles over the entire bottom area.

4. QUANTITY AND QUALITY OF SLUDGE PRODUCED

In the actual treatment plant, it is generally difficult to measure the quantity of sludge from the activated sludge process, since the quantity of sludge existing in the treatment system is by far larger than that of the sludge produced.

In the contact aeration process, on the other hand, the quantity of withdrawn sludge in the final settling tank equals the quantity of produced sludge, since no sludge is returned. Therefore, it is relatively easy to determine it.

From the process flow rate and the SS concentrations of the contact aeration tank influent and effluent and the secondary settled effluent, the SS balance was calculated to determine the quantity of sludge produced. Meanwhile, for reference, the concentration of the sludge withdrawn from the final settling tank was measured on a composite sample taken 7 times during the daytime and, based on this value and the quantity of withdrawn sludge, the sludge production was determined.

The ratio of produced sludge to the quantity of SS removed in the secondary treatment by the contact aeration process was approximately 60 ~ 80% when the equipment was operated normally under similar conditions throughout a year. The percentage was higher in winter and lower in summer. In the activated sludge process over the same period at the Suminoe Treatment Plant as a whole, the annual average excess sludge was nearly equal to the quantity of removed SS. The annual average BOD volumetric loading of the activated sludge process was about $0.3kg/m^3 \cdot day$, slightly lower than that of the contact aeration process. Thus, it can be concluded that the contact aeration process produced less sludge than the activated sludge process.

In Osaka City, sludge treatment generally consists of gravity thickening and subsequent anaerobic digestion. A gravity thickening experiment using a 1-liter cylinder and an anaerobic digestion

experiment using 2-liter batch equipment were conducted on the sludge from the contact aeration process and the activated sludge process. The results are shown in Tables 5 and 6.

Table 5 shows that there is no significant difference in gravity thickening between the sludges from the contact aeration process and the activated sludge process.

Based on Table 6, which describes the results of the anaerobic digestion experiment on both sludges, it can be concluded that the contact aeration sludge was better treated than activated sludge in terms of organic substance reduction ratio, volume of gas generated per unit weight of volatile solids decomposed and volume of gas generated per unit of volatile solids added, etc.

It is therefore assumed that sludge from the contact aeration process can be treated in the same way as activated sludge, in so far as gravity thickening and anaerobic digestion are concerned.

Table 5 Thickening Characteristics

Activated Sludge			Contact Aeration Sludge		
Initial Concentration (mg/ℓ)	Concentration after 24 Hours (mg/ℓ)	Ratio of Concentration	Initial Concentration (mg/ℓ)	Concentration after 24 Hours (mg/ℓ)	Ratio of Concentration
3,570	17,000	4.8	3,654	31,774	8.7
8,446	24,841	2.9	3,658	22,863	6.3
16,000	25,806	1.6	12,500	26,600	2.1

5. SOLID-LIQUID SEPARATION WITH FINE SCREEN

From the viewpoint of effective utilization of land, Osaka City has conducted solid-liquid separation in the activated sludge process, using 3-story settling tanks at three of twelve treatment plants.

We researched the solid-liquid separation method for the contact aeration process likely to reduce area of the treatment plant.

The experiment was based on the assumption that solid-liquid separation of effluent from the contact aeration tank can be achieved using a fine screen, since the effluent contains slightly lower SS than effluent from the primary settling tank and most of the suspended solids are large and strong flocs detached from the packing media.

Table 6 Results of Experimental Anaerobic Sludge Digestion

Sludge Mixing Ratio	Measuring Item	Unit	Activated Sludge	Contact Aeration Sludge
Sample : Seed sludge 1 : 0.3	Initial VSS concentration	mg/ℓ	11,890	11,720
	VSS reduction	%	27.3	32.0
	Gas generation per unit weight of volatile solids decomposed	ℓ/g	0.97	1.18
	Gas generation per unit weight of volatile solids added	ℓ/g	0.265	0.378
Sample : Seed sludge 1 : 0.3	Initial VSS concentration	mg/ℓ	8,370	8,690
	VSS reduction	%	44.5	46.2
	Gas generation per unit weight of volatile solids decomposed	ℓ/g	0.73	0.85
	Gas generation per unit weight of volatile solids added	ℓ/g	0.325	0.393
Sample : Primary sedimentation : Seed sludge 1 : 1 : 1	Initial VSS concentration	mg/ℓ	13,630	13,720
	VSS reduction	%	29.9	35.4
	Gas generation per unit weight of volatile solids decomposed	ℓ/g	1.33	1.34
	Gas generation per unit weight of volatile solids added	ℓ/g	0.400	0.474

Notes: 1. In the reaction tank with a capacity of 2ℓ, sludge was digested by the batch method at a constant temperature of 35°C.

2. The organic substance content of sample sludge was 58∿63% for activated sludge and 56∿62% for contact aeration sludge.

5.1 PRINCIPLE AND DESCRIPTION OF FINE SCREEN

The fine screen used in the experiment is illustrated in Fig. 10. It is a solid-liquid separation device in which a number of wedge-shaped wires, triangular in cross-section, are arranged in parallel at fixed intervals. This screen was erected at an inclination of approximately 60° from the horizontal plane. When sewage was made to flow down onto the screen, water passed the screen through the slits between the wedge-shaped wires, during which process the sewage was treated. Meanwhile, separated sludge slid down the screen with natural gravity, to produce separated sludge. (Photo 3)

In order to prevent clogging, a rotating brush was installed on the back surface of the screen so that it could be cleaned from the rear. The brush, made of synthetic resin, moved reciprocally beneath the screen, while rotating, to clean the entire back surface of the screen.

The operating ratio of the brush was set at 25% (7-second operation and 21-second stoppage) for the 33cm-wide screen used in the experiment.

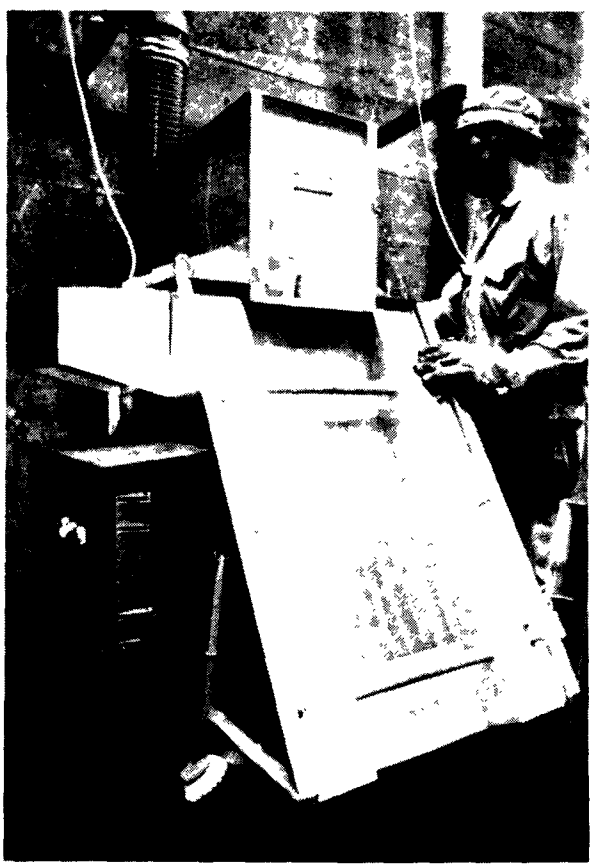


Photo 3 The Fine Screen Used in the Experiment

In the experiment, influent was fed from the outlet of the contact aeration tank down to the screen by means of a siphon so as not to break the floc, as illustrated in Fig. 11. For the experiment, 33cm-wide 60cm-long screens were mainly used.

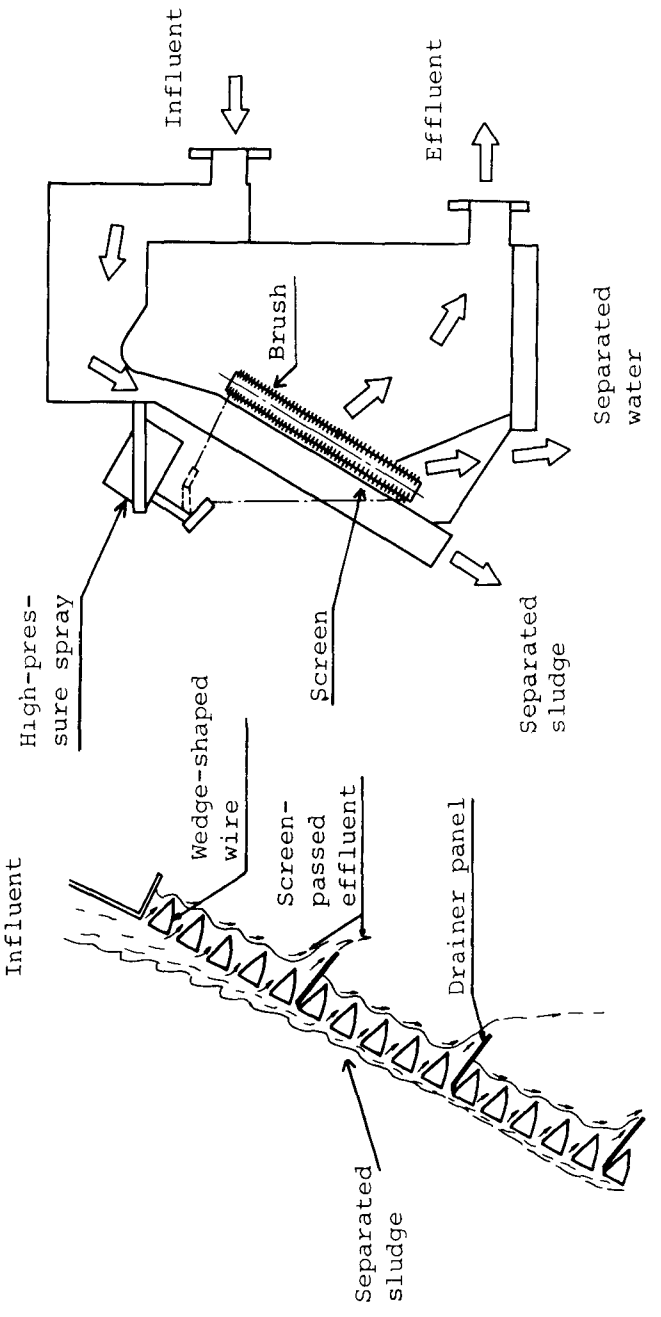


Fig. 10 Fine Screening Device

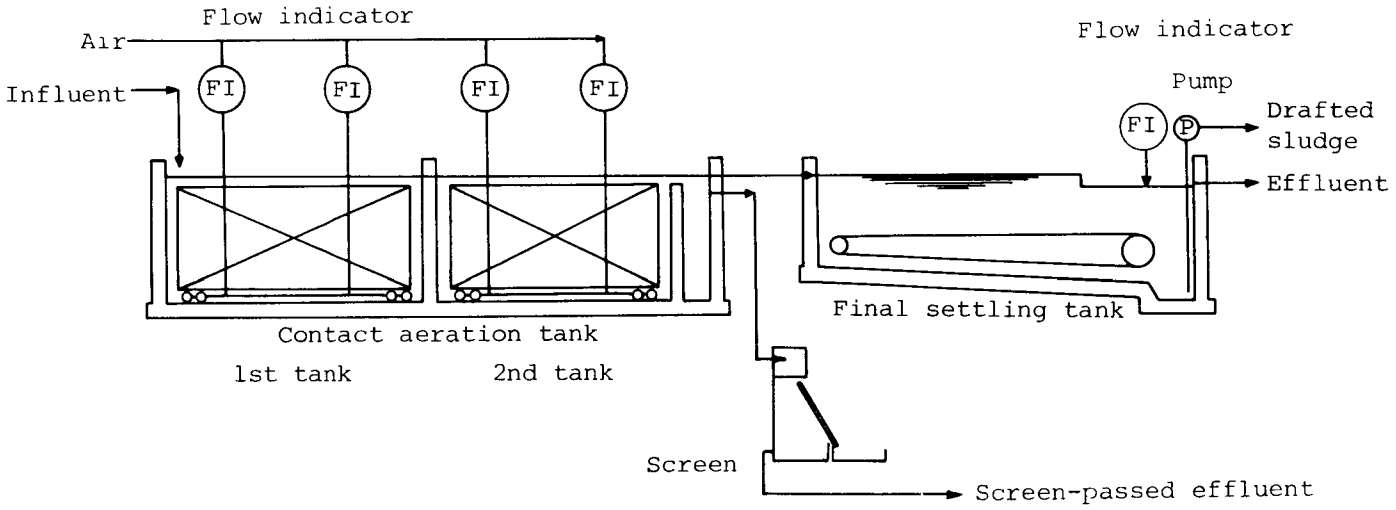
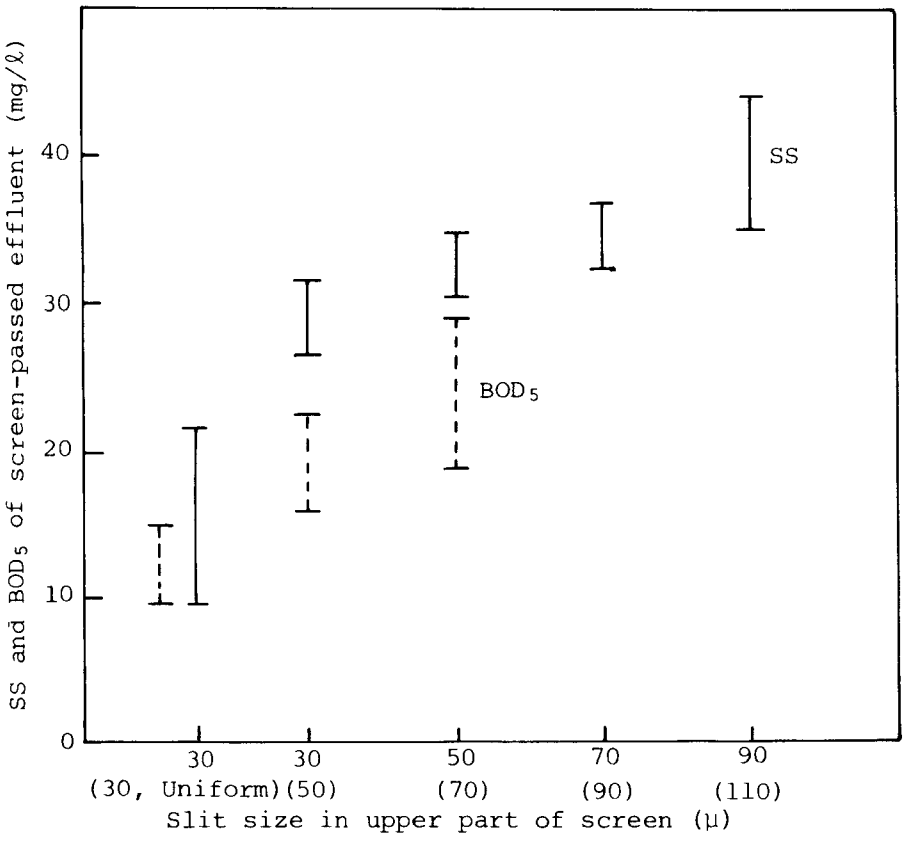


Fig. 11 Flow Sheet of Fine Screen Experiment

5.2 SELECTION OF SCREEN

This screen was originally developed for the purpose of thickening sludge. In order to maximize concentration of separated sludge, the slits between wedge-shaped wires in the lower part of the screen were made wider than those in the upper part.

From Fig. 12, which shows the relationship between slit size and quality of screen-passed effluent, it was concluded that the sludge thickening screen with a wider slit in the lower half was not a suitable alternative to the final settling tank and that it was necessary to use screen with a 30μ slit size — the minimum size available at present — over its entire surface.



Notes: Values in parentheses indicate slit size in lower part.

Fig. 12 Screen Slit Size vs. Screen-passed Effluent Quality

As a means of improving the quality of screen-passed effluent, isolation of effluent passing the lowest portion of the screen was effective, since such effluent was by far lower in quality than that passing the upper portion of the screen.

The most effective ratio of the quantity of separated effluent to the quantity of sewage made to flow down the screen was 5%, as shown in Fig. 13. Effluent thus separated should be taken out together with the separated sludge, or returned to water treatment facilities.

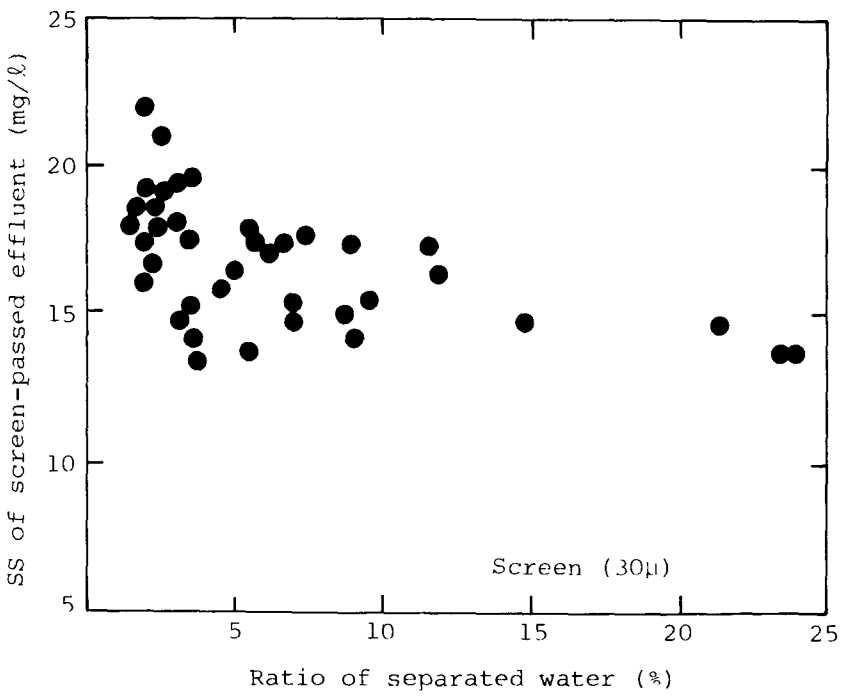


Fig. 13 Ratio of Separated Water vs. SS of Screen-passed Effluent

5.3 HYDRAULIC LOADING ON THE SCREEN

Unless the hydraulic loading on the screen is increased, plant site area cannot be reduced. Therefore, an increase in the hydraulic loading on the screen was emphasized even at the expense of reduced concentratoin on separated sludge.

According to the estimate by the municipality, solid-liquid separation using the screen can be achieved in an area about 20% of the existing single-story final settling tank, if sewage is treated at a rate of $30m^3/hr$ per meter of screen width. This means that the plant site area requirement is even smaller than that for a 3-story settling tank.

Fig. 14 shows the relationship between the hydraulic loading on the screen and effluent SS. As the loading increased, the effluent quality tended to deteriorate. Yet, the degree of deterioration was very low: Effluent SS was approximately 20mg/ℓ when the hydraulic loading on the screen was $36m^3/m \cdot hr$.

The relationship between the concentration of separated sludge and the loading is shown in Fig. 15. As the flow increased, the quantity of separated sludge increased, while its concentration declined. The ratio of the quantity of sludge to the flow was approximately 1.3%, when the rate of treatment was $36m^3/m \cdot hr$. If it is necessary to raise concentration of sludge, sludge once separated with the screen can be further concentrated with the use of another screen.

5.4 PROTECTION AGAINST CLOGGING OF SCREEN

The screen used in the experiment was prevented from clogging by cleaning it with a rotating brush fitted on the back surface of the screen. However, when the screen was operated continuously, the quantity of separated sludge flowing down on the screen surface tended to increase. This was probably due to the generation and growth of biological slime on the surface of the screen.

Since such slime could not be removed perfectly by cleaning from the back using the rotating brush alone, the surface of the screen was also cleaned by means of high-pressure spray. The specifications of the spray are shown in Table 7. Fig. 16 shows the relationship between the percentage of separated sludge quantity and the number of days elasped using operation frequency of the spray as parameter.

The term "percentage of separated sludge quantity" means in this context the ratio of separated sludge quantity to the quantity of influent to the screen. Fig. 16 indicates the remarkable effectiveness of the high-pressure spray.

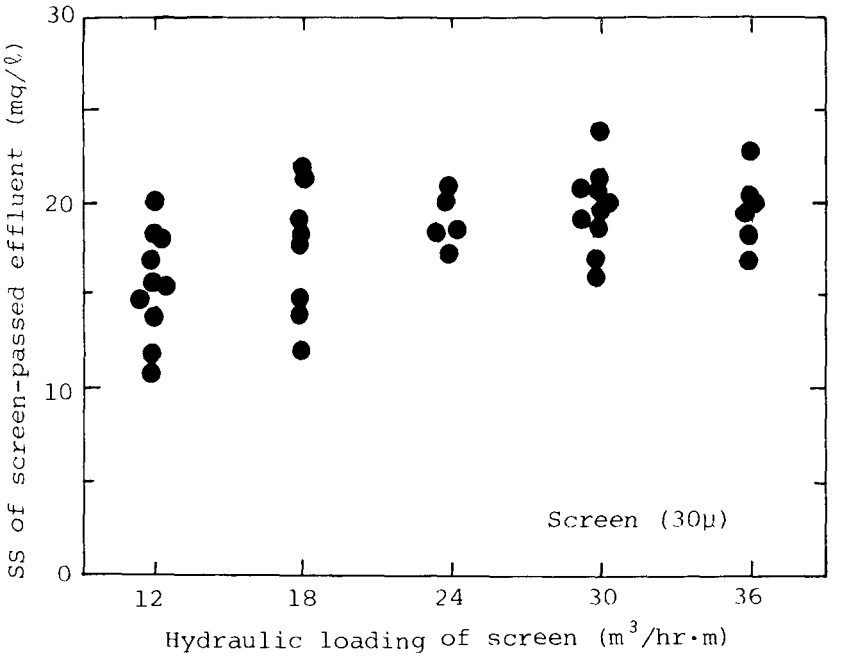


Fig. 14 Quantity vs. SS of Screen-passed Effluent

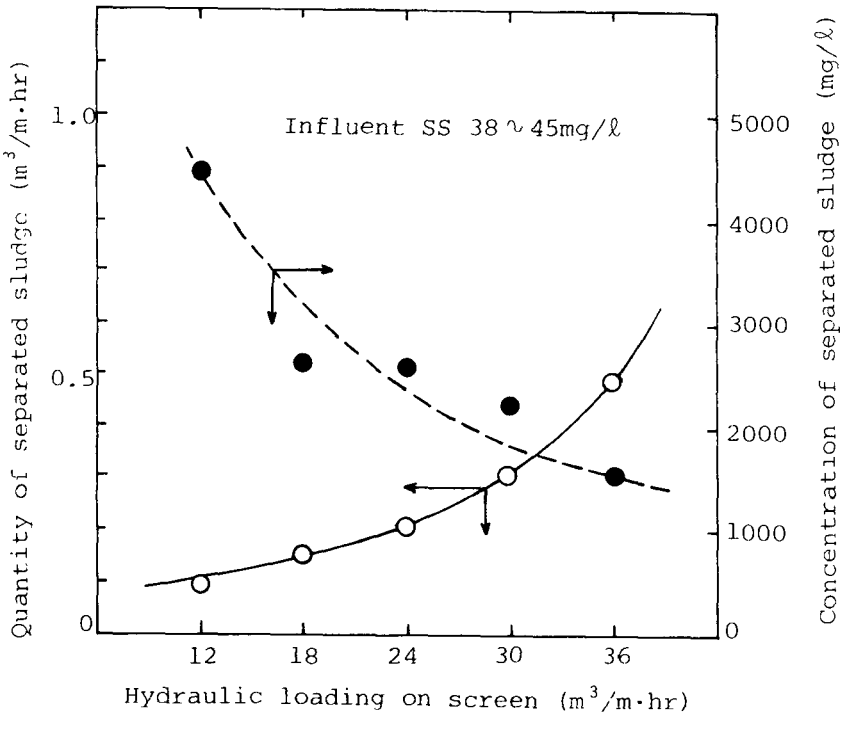


Fig. 15 Hydraulic Loading on Screen vs. Ratio of Separated Sludge and Separated Sludge Concentration

Table 7 High-pressure Spray Cleaning Device

Quantity of water used	10ℓ/min.
Spraying pressure	20kg/cm^2
Unit cleaning time	Average 3 min. (Max. 5 min.)

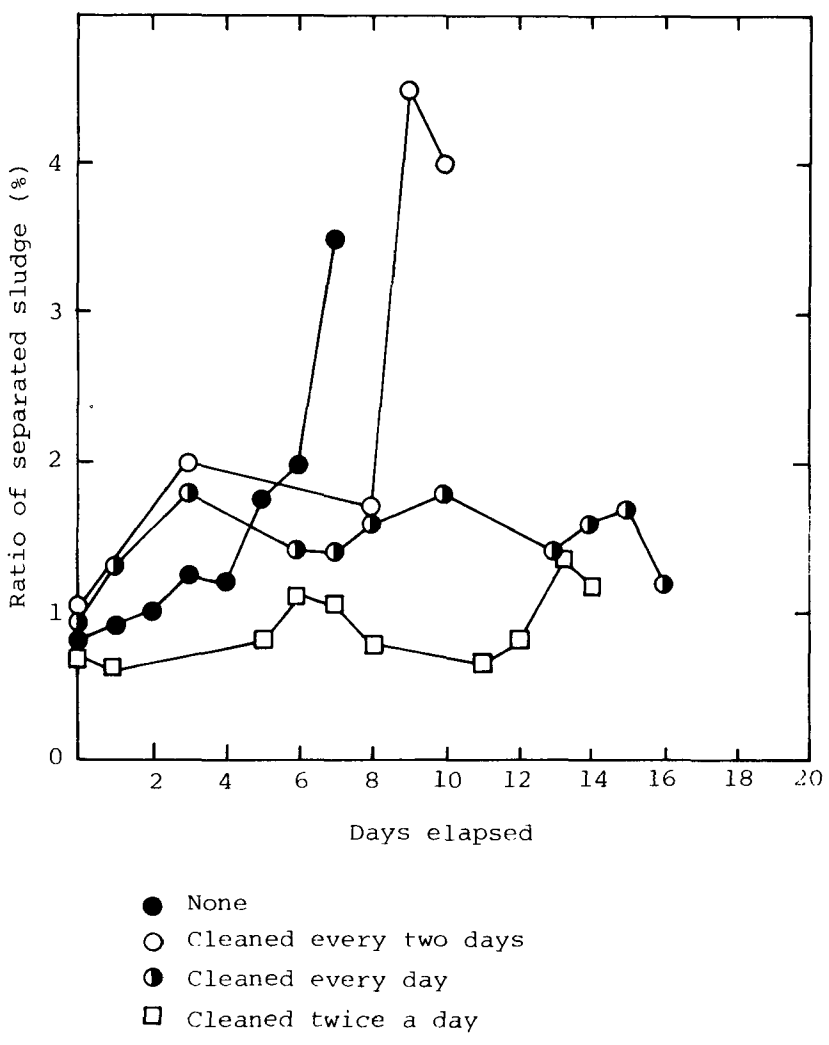


Fig. 16 Effect of High-pressure Cleaning of Fine Screen

5.5 COMPARISON OF QUALITY BETWEEN SCREEN-PASSED EFFLUENT ANd FINAL SETTLED EFFLUENT

Figs. 17 and 18 show a comparison between effluent passing the screen having uniform 30μ slits, with a percentage of separated sludge quantity of 5% and hydraulic loading of 30 ~ 36m^3/m•hr, and effluent from the final settling tank. At the time of this experiment, the final settling tank was operated with an overflow rate of 5 ~ 26m^3/m^2•day, under relatively light load.

The values concerning the screen-passed effluent were obtained by analyzing a composite sample taken 3 times during the daytime, while the values concerning effluent from the final settling tank were obtained by analyzing a sample subjected to mixing for 24 hrs.

On the average, the screen-passed effluent showed BOD higher by approximately 2mg/ℓ and SS higher by approximately 4mg/ℓ than effluent from the final settling tank.

The screen treatment experiment was conducted in the last year of the 4-year period of operation of the contact aeration equipment. In the latter half of the year especially, loss of aeration uniformity in the contact aeration tank was observed visually, and as a consequence the quality of effluent from the final settling tank deteriorated.

The investigation conducted at the time of removal of the equipment upon completion of the experiment revealed that some of the steel risers used for aeration were corroded, with some holes formed on their surfaces. This seems to have caused an insufficient supply of air to the diffuser pipes, promoting clogging and, as a result, lessening the quality of effluent immediately before the end of the experiment.

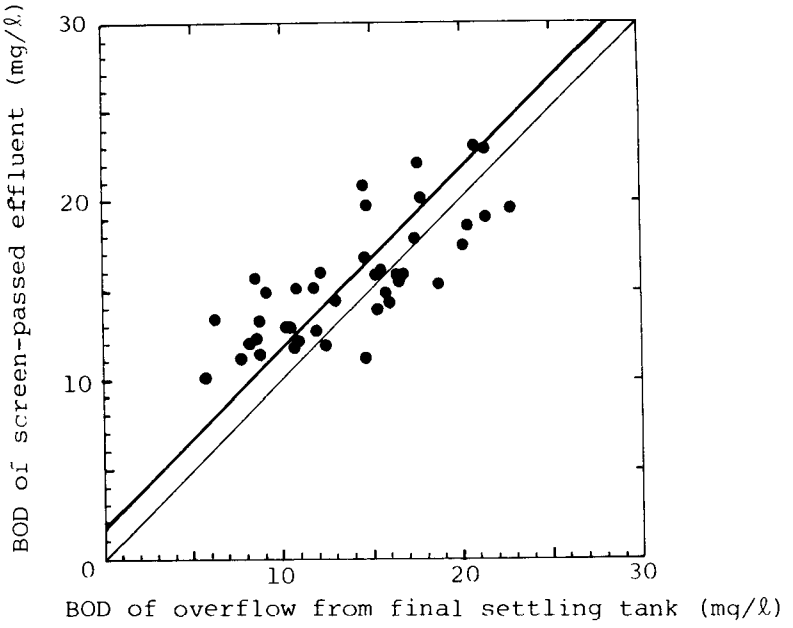


Fig. 17 Comparison of Effluent BOD (Daily Average)

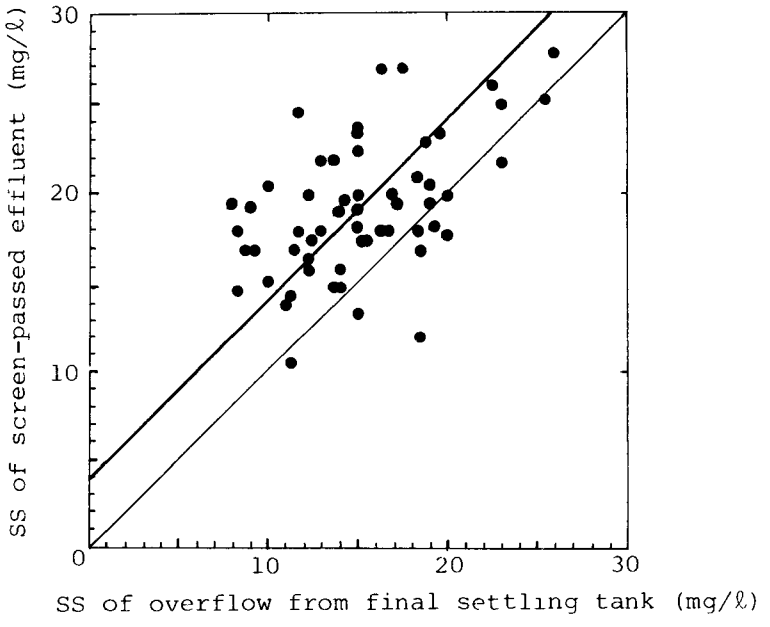


Fig. 18 Comparison of Effluent SS

6. SUMMARY

To sum up the findings of these experiments on the contact aeration process using net packing media, employing the full-scale equipment.

① The contact aeration process in which multi-line aeration is carried out from below the net packing media can be used for secondary sewage treatment as an alternative to the activated sludge process.

② As advantages of the contact aeration process, easy maintenance and control and reduced production of sludge are noted. A disadvantage is the poorer appearance of the screen-passed effluent, as compared with the effluent treated by the activated

③ The capacity of the contact aeration process to remove soluble BOD is more vulnerable to load fluctuation than that of the activated sludge process. The treatment efficiency of the contact aeration process tends to increase if the quantity of air supply is made proportional to the flow rate of influent and if the air supply is increased when BOD load is high.

④ The sludge from contact aeration process is not significantly different from the activated sludge, with regard to gravity thickening and anaerobic digestion.

⑤ If solid-liquid separation of the contact aeration process is achieved with the use of a fine screen instead of the final settling tank, the plant site area can be reduced, though the quality of effluent is diminished, compared with effluent obtained when a final settling tank is employed.

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Sewage Treatment Technology

CONSTRUCTION TECHNOLOGY ASSESSMENT SYSTEM AND DEVELOPMENT OF MECHANICAL AERATORS FOR THE OXIDATION DITCH PROCESS

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1. PREFACE

Ministry of Construction is currently carrying out research and development of various kinds of basic construction techniques in order to improve social capital and promote smoothly and effectively various policies required to meet the needs in construction administration which have become quite diversified and sophisticated in recent years.

The studies concerning sewerage are being performed under the budgets available, mainly sewerage survey costs and administrative department costs. Themes of study are varied; some themes are related to basic study but others are immediately applicable to actual work, and the rest are related to the study of future directions in the administration of sewerage works.

The Water Quality Control Department of Public Works Research Institute, an affiliated organization to Ministry of Construction, and Research and Technology Development Division, Japan Sewage Works Agency is doing most of the research and development, but the Ministry of Construction is also carrying out research and development generally on themes required for carrying out sewerage projects.

On the other hand, themes of research and development by the private sector as well as by the government on construction techniques are extremely progressive and highly sophisticated, and it has become vitally important to carry out the research and development with close collaboration between the government and private sector in order to improve construction technology as a whole.

Ministry of Construction established in 1977 the basic policy for research and development "5-year Plan for Research and Development of Construction Technology (FY 1977 to 1981)" as follows.

① In carrying out comprehensive research and development projects as well as overall research and development, it is required to establish clear research goals and programs and to organize the research and development organizations in such a manner that the purposes can be achieved within a predetermined period of time

with close cooperation among government, private sector and academic circles.

② New research and development methods such as joint study between the government and the private sector are to be developed in order to aggressively promote research and development on construction technology and enable the private sector to utilize the results.

As one of systems for more aggressively promoting the research and development of construction technology by the private sector in accordance with the policy stated above, the construction technology assessment system was established in FY 1978.

This system will properly assess the function and performance of the research and development results of construction technology conducted by the private sector and will publicize them in order to promote aggressive utilization of new techniques and to further accelerate the research and development in the private sector.

This system has been adopted because it become necessary to properly assess and to organize the diffusion of the new techniques in order to quickly introduce them to construction sites since people tend to be conservative in introducing new techniques because public costs are used as financial resources in the area of public construction projects, estimation of economic effects due to introduction of new techniques is not easy, and failures associated with introduction of new techniques are not easily permitted.

2. CONSTRUCTION TECHNOLOGY ASSESSMENT SYSTEM

Actual policies to be carried out under the construction technology assessment system are prescribed in "Construction Technology Assessment Rules (Issue No.976 by Ministry of Construction on May 24, 1978)" which is outlined in Fig. 1.

That is, the Minister of Construction will announce the technical subjects to be assessed in Official Gazette thus inviting private firms to submit their proposals. Then, the achievements of research and development submitted by applicants (enterprises and corporations) will be thoroughly assessed, and assessment papers will be issued based upon the results of the assessment and publicly announced.

In this assessment, preliminary technical assessment will be performed by Technology Center for National Land Development and then, based upon the results, the new techniques will be carefully assessed by the assessment committee organized by the Ministry of Construction.

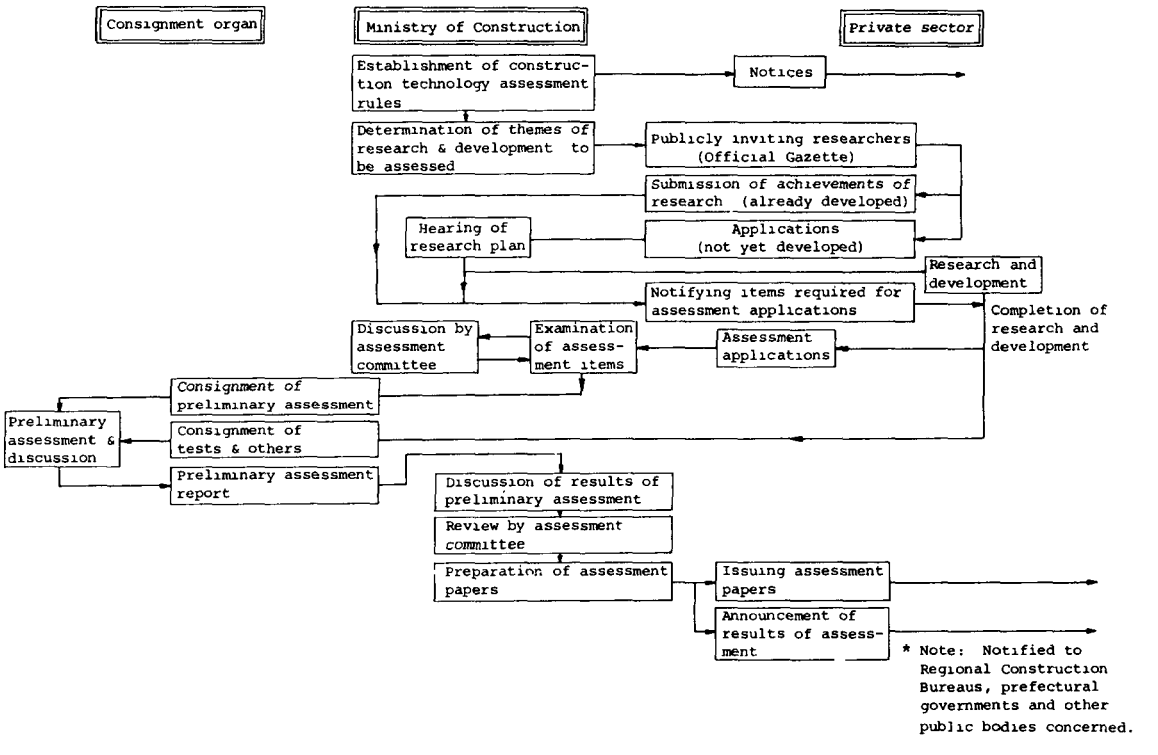


Fig. 1 Flow of Construction Technology Assessment System

3. THEMES OF THE SYSTEM RELATED TO SEWERAGE

Since the start of the construction technology assessment system in 1978, approximately 3 to 5 themes related to construction are assessed every year.

Themes of research related to sewerage for which researchers were already invited are "development of microwave furnace for sewer sludge to melt" and "development of large-diameter egg-shaped hard vinyl chloride pipe with high rigidity" for FY 1978, "development of ultra-deep aeration system for sewer treatment" for FY 1979, "development of energy-saving type aerator by diffuesed air" for FY 1980, "development of mechanical aerator for oxidation ditch process" for FY 1981 and "development of sewage solid-liquid separation method by screen process" for FY 1982 (refer to Table 1).

Themes being publicly announced now consist of four themes related to water treatment, one theme related to sludge treatment, and one theme related to sewer pipe. In recent two years, themes tend to be related to the small-scale-sewerage system because new public sewerage projects tend to be small in Japan. That is, as shown by Fig. 2, most municipalities with populations greater than 50,000 have already started sewerage projects but the municipalities with populations of less than 50,000 have delayed in starting the projects and in particular most municipalities with populations of less than 10,000 have not yet started sewerage construction projects. Therefore, in order to achieve a target value of 90% for the population coverage rate of sewerage by the beginning of 21st century, it is necessary to make more efforts to improve sewerage even for these small municipalities. Since small municipalities have different financial abilities and organizing powers from those of large municipalities, approaches differing from conventional sewerage processes will be required. That is, it is not easy to find skilled workers for maintaining and controlling the sewage treatment plant in a small city and, thus, the maintenance and control must be simple, and since population density of a small city is low, a pipe and pumping

station system for small-scale-sewerage must be newly developed. Thus, development of techniques related to small-scale-sewerage will increasing important.

Table 1 Themes of Construction Technology Assessment by Ministry of Construction

Fiscal Year of Invitation	1978	1978	1979
Subjects	Development of microwave furnace for sewer sludge to melt	Development of large-diameter egg-shaped hard vinyl chloride pipe with high rigidity	Development of ultra-deep aeration system for sewer treatment
Objects of Development	To develop microwave melting furnace capable using microwaves to melt and solidify sewer sludge, turning it into chemically stable glassy material without however generating harmful exhaust gas but permitting, the melted and solidified substances to be used as construction materials.	To develop hard vinyl chloride pipe having an egg-shaped section with excellent hydraulic properties, large diameter, sufficient strength and endurance.	To develop activated sludge process using ultra-deep aeration tank requiring only a small area.
Goals of Development	1) Maximum treating capacity of furnace of about 5 tons/day. 2) Exhaust gas generated from the process, being capable of meeting the requirements set forth in Air Pollution Prevention Law (Law No. 97 in 1968). 3) Melted and solidified glassy materials being capable of meeting the Order of Prime Minister's Office (Order No. 5 in 1973) determining the criteria for industrial wastes. 4) Melted and solidified glassy substances being capable of providing the strength required for aggregates. 5) Comparatively low costs of manufacturing, operating and maintaining the melting furnace.	1) Strength determined by the flat test (in accordance with test method established by Standard K-1 of Japan Sewage Works Association) being capable of meeting the strength of class 1 centrifugal reinforced concrete pipe (external pressure strength against cracking load of JIS A 5303). 2) Nominal diameter of about 500 mm. 3) Endurance almost equal to or higher than that of class 1 centrifugal reinforced concrete pipe.	1) Required processing capacity greater than 1,000 m^3/day. 2) Process providing values of biochemical oxygen demand (BOD) and suspended solids (SS) of final effluent capable of meeting the technical requirements set forth in Para. 1 of Article 6 of the Enforcement Ordinance of Sewerage Law (Cabinet Order No. 147 in 1959). 3) Simple operation. 4) Low maintenance and management costs. 5) Simple environmental measures.
Applicants	1 firm	2 firms	2 firms

Table 1 Themes of Construction Technology Assessment by Ministry of Construction (Cont'd)

Fiscal Year of Invitation	1980	1981	1982
Subjects	Development of energy-saving type aerator by diffused air.	Development of mechanical aerator for oxidation ditch process	Development of sewage solid-liquid separation method by screen process
Objects of Development	To develop energy-saving type aerator to be used for diffused air type activated sludge process at the sewage treatment plant.	To develop efficient mechanical aerator using oxidation ditch process as one of sewage treatment methods.	To develop solid-liquid separation process using screens for economicaly separation of sewage solids from liquids, conventionally performed with settling basin.
Goals of Development	1) Power consumption is to be reduced by more than 20% compared to conventional air diffusing equipment. 2) For ordinary sewage, the aerator is to be able to provide the values of biochemical oxygen demand (BOD) and suspended solids (SS) of final effluent meeting the technical requirements set forth in Para. 1 of Article 6 of the Enforcement Ordinance of Sewerage Law. 3) Connection to existing aeration tank is to be easily made. 4) Simple operation.	1) Required current speed is to be obtained. 2) Oxygen is to be efficiently supplied. 3) Sufficient mixing and agitating ability. 4) Easy equipment maintenance and operation. 5) Simple environmental measures.	1) Quality of treated water almost equal to that of settling basin is to be obtained. 2) Easy maintenance and operation. 3) Sufficient endurance. 4) Economical.
Applicants	5 firms	8 firms	

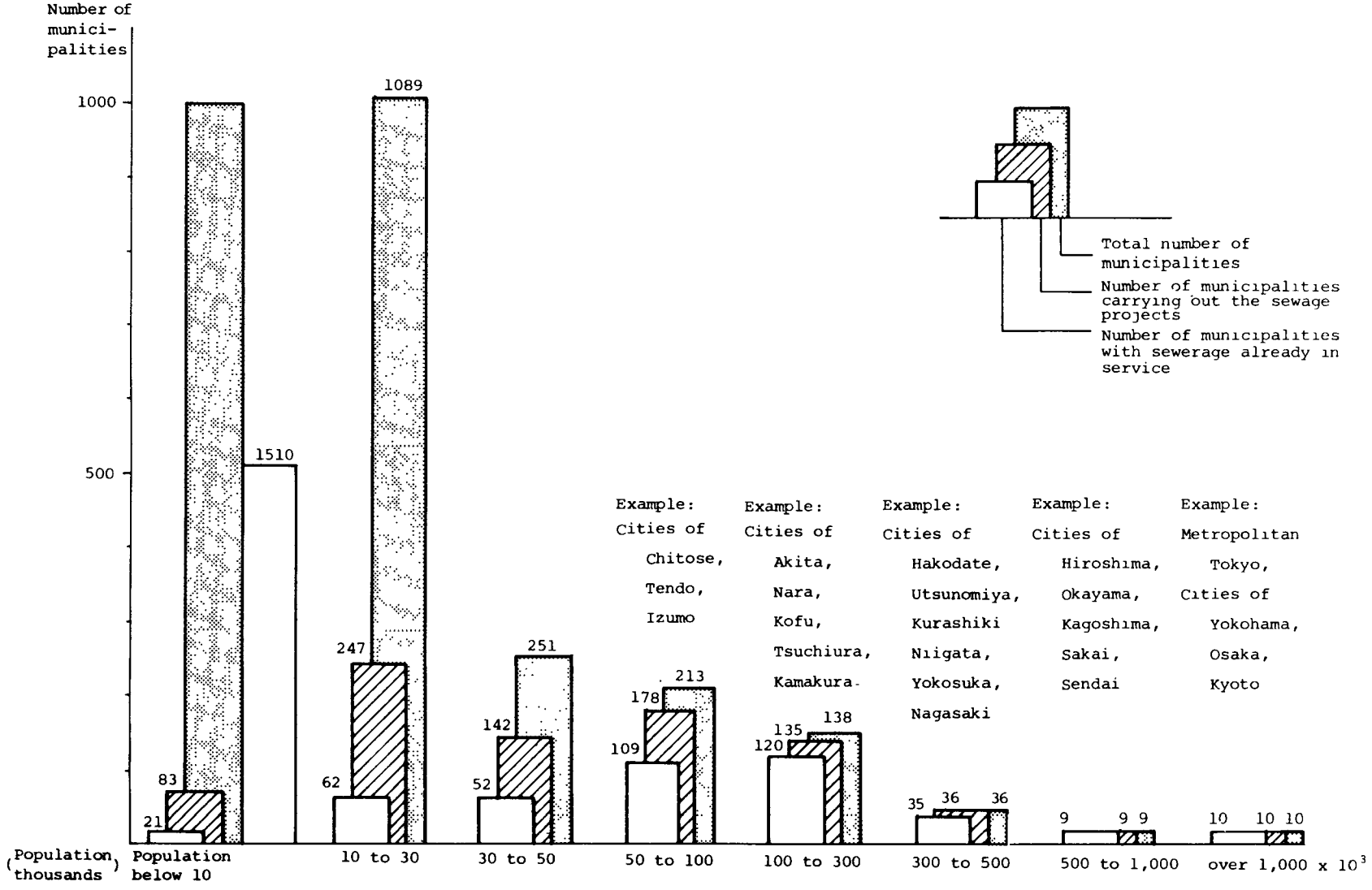


Fig. 2 Number of Municipalities Carrying Out the Sewerage Projects by Population Rank (end of FY 1982)

4. DEVELOPMENT OF MECHANICAL AERATOR FOR OXIDATION DITCH PROCESS

Ministry of Construction invited firms interested in developing a "mechanical aerator for oxidation ditch process" as a theme for construction technology assessment on July 1, 1981. In response to this invitation, eight firms submitted applications, preliminary assessment was performed by the Technology Center for National Land Developments in 1982, subsequent assessment was performed by the Technology Assessment Committee in Ministry of Construction in March 1983, and it was recognized that all eight firms satisfied the development goals.

(1) Reasons for the Selection of the Theme

When constructing sewage treatment plants in small cities, unlike conventional plants built for large and medium cities, the following items must be taken into account:

① Because of the small scale, no benefits due to a large scale can be expected for construction, maintenance and administration costs, and these costs tend to become high.

② Since it is difficult to obtain professional maintenance engineers, complicated and sophisticated equipment are not appropriate.

③ It is required to respond to considerable fluctuations of influent water which is peculiar to small-scale sewage treatment plants.

It becomes difficult to cope with the situations described above by using the conventional activated sludge process which is widely being used for large and medium municipalities. (Refer to Table 2.) On the other hand, compared to highly dense large cities, large sites for sewage treatment plants relative to unit amount of water can be more easily obtained in small cities. The oxidation ditch process is worthy of note since it can use this advantage in order to cope with the difficult situations stated above.

Table 2 Numbers of Sewage Treatment Plants by Treatment Process

(At the end of FY 1980)

Process of treatment / Design capacity ($10^3 m^3$/day)	Low class	Medium class		High class							Total
	Sedimentation process	High rate trickling filter process	Aero-accelerator	Conventional activated sludge process	Stepped aeration process	Extended aeration process	Contact stabilization process	Pure oxygen aeration process	Oxidation ditch	Rotary biological contacter process	
Less than 5	2	4	2	31	5	16	3	3	4	3	73
5 ∿ 10	2	3		35	5						45
10 ∿ 50		7	14	97	14	1				2	135
50 ∿ 100		1	3	55	24					1	84
100 ∿ 500	1	1	2	72	31						107
More than 500				6	7						13
Total	5	16	21	296	86	17	3	3	4	6	457

Notes: 1. If more than two treatment methods are used in one plant, the method with the larger capacity is indicated in this table.

2. Detail of 457 treatment plants in total:

Public sewerage;	423	treatment	plants
River basin sewerage;	27	"	"
Specified public sewerage;	2	"	"
Public sewerage to preserve the national environment;	5	"	"

According to the oxidation ditch method, a relatively shallow elliptic water channel is installed, high-concentration activated sludge is mixed with sanitary sewage by a mechanical aerator, and then the mixture is circulated and treated by aeration for a long time.

Features of this method are:

① Since ditch capacity is large and retention period is long, amount of MLSS in ditch is large and thus stability is high against a large load fluctuation. Also, even though the amount of return sludge is made constant, the amount of MLSS fluctuates little in response to the change in sludge concentration, so that maintenance can be performed easily.

② Since primary sedimentation basin can be omitted, the portions requiring maintenance work can be reduced and the total labor required can be reduced.

③ Since a mechanical aerator is used for ditch aeration, the equipment is simple and can be easily repaired.

④ Since sludge age is long, 15 to 30 days, sludge is subjected to a sort of aerobic digestion and, thus, the sludge can be treated easily.

Because of the features listed above, the theme of "development of efficient mechanical aerator for oxidation ditch process" was selected in order to promote efficient sewerage construction in small municipalities as key point of future sewerage construction.

(2) Conventional Oxidation Ditch and Aerator

Conventional oxidation ditch has a track-shaped endless water channel about 1 m deep and treats sanitary sewage by circulating and aerating the sanitary sewage by rotors. As shown in Fig. 3, the treating process has no primary sedimentation basin and consists of ditch and final sedimentation basin, but the final sedimentation basin is not installed when performing batch treatment.

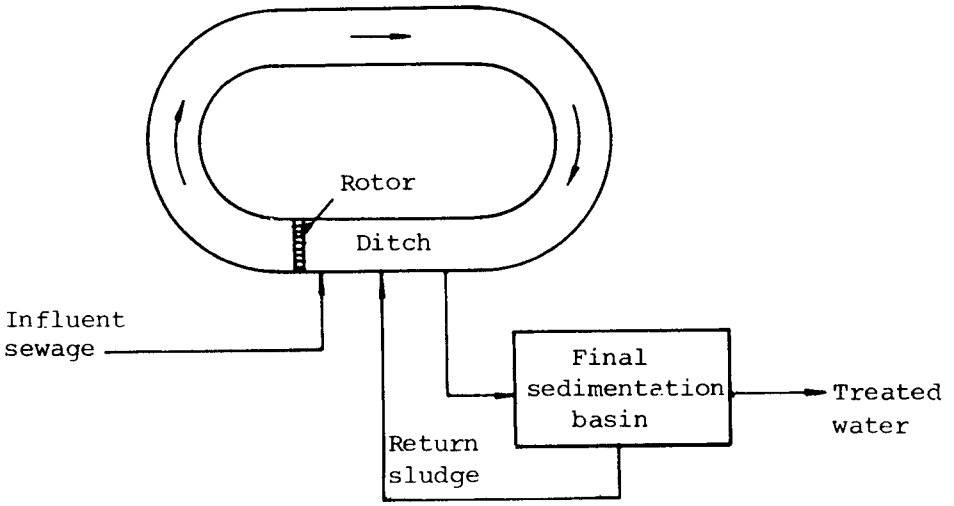


Fig. 3 Flowchart of Oxidation Ditch

This conventional method is designed for MLSS of 3,000 - 5,000 mg/ℓ, hydraulic retention time of 24 - 48 hours, BOD-SS load of 0.03 - 0.05 kg/SS kg.day, and a sludge return ratio of 50 to 150%.

Kessner brush rotors were used many years ago but recently flat steel bar rotors, steel angle rotors and cage type rotors are widely being used. Generally flat steel bar rotors have been used for the public sewerage in this country. The flat steel bar rotor is made of steel vanes, each about 15 to 30 cm long attached in parallel to the periphery of a rotating shaft with a diameter greater than 0.5 m, which is rotated at a speed of 60 to 120 revolutions per minute. One or more rotors are normally installed facing the direction of flow of each ditch.

At present, there are seven sewage treatment plants using oxidation ditches in Japan and their specifications are as listed below.

Table 3 Sewage Treatment Plants Based on Oxidation Ditch Process

Name of municipalities	Plant name	Design population (persons)	Design capacity (m^3/day)	Quality of influent water (mg/ℓ)	Quality of treated water (mg/ℓ)	Sludge treatment (disposal)	Remarks
Nikko	Yumoto Plant	6,840	3,250	BOD: 94 SS: 113	BOD: 20 SS: 30	Thickening → mechanical dewatering	Residential and sightseeing waste water
Toyohashi	Takane Plant	1,650	650	BOD: 200 SS: 200	BOD: 20 SS: 60	Solar drying → land application	Residential and livestock waste water
Tomakomai	Yufutsu Plant	7,200	4,300	BOD: 200 SS: 250	BOD: 20 SS: 50	Thickening → vacuum carryer → to Nishi-machi treatment plant	Residential waste water
Ohgata, Akita Pref.	Ohgata Plant	4,000	1,536	BOD: 150 SS: 200	BOD: 20 SS: 40	Thickening → mechanical dewatering → disposal	Residential waste water
Ohmihachiman	Okinoshima Purification Center	1,010	420 for overall, 210 for 1st step	BOD: 235 SS: 210 COD: 120 T-N: 40 T-P: 4	BOD: 20 SS: 20 COD: 20 T-N: 20 T-P: 0.5	Solar drying → land application	Residential waste water started in July 1982, secondary treatment + sand filtering
Terai, Ishikawa Pref.	Public Sewerage Eastern Purification Center	3,600 for overall, 1,800 for 1st step	1,800 for overall, 900 for 1st step	BOD: 220 SS: 200	BOD: 18 SS: 50	Thickening → storage → (vacuum carryer) → scheduled to dispose outside of plant	Underground structures, aboveground park facilities, residential waste water
Hikami, Hyogo Pref.	Nishinaka Plant	5,100	3,860	BOD: 210 SS: 160	BOD: 20 SS: 40	Thickening → solar drying → land application	Residential waste water, started in April 1983

(3) Determination of Goals of Development

Oxidation ditch has a long retention time (3 to 4 times) in aeration tank and a high concentration of MLSS (2 to 3 times) compared to the conventional activated sludge process, so that a larger amount of oxygen is needed for the treatment.

Amount of oxygen consumed is given by the following formula:

$$N = a \frac{S_o \eta}{1000} Q + b \frac{X_s}{1000} V \quad \cdots\cdots\cdots\cdots\cdots \quad ①$$

where, N : Amount of oxygen consumed (kg-O_2/day)

S_o : Concentration of influent BOD (mg/ℓ) (200 mg/ℓ)

η : BOD removal factor (0.9)

Q : Amount of influent water (m^3/day)

X_s : Concentration of MLSS (mg/ℓ)

V : Capacity of aeration tank (m^3)

a : Amount of oxygen required per 1 kg of BOD removal (0.5 kg-O_2/kg-BOD)

b : Amount of oxygen required per 1 kg of MLSS daily (0.1 kg-O_2/kg-BOD)

Design parameters for conventional activated sludge process and oxidation ditch process are given below.

ⓐ Design parameters for conventional activated sludge process:

BOD-SS load: 0.4 kg/SS kg·day

MLSS concentration: 1500 mg/ℓ

ⓑ Design parameters for oxidation ditch process:

BOD-SS load: 0.05 kg/SS kg·day

MLSS concentration: 4000 mg/ℓ

For ⓐ : $V = \frac{1}{3} Q$

For ⓑ : $V = Q$

Therefore, from equation ① , the amount of oxygen consumed is given by

Standard activated sludge process: $\frac{N}{Q} = 0.14$ kg-O_2/day · m^3

Oxidation ditch process: $\frac{N}{Q} = 0.49$ kg-O_2/day · m^3

Therefore, in the oxidation ditch process, the amount of oxygen supplied is 3.5 times greater than that needed in the conventional activated sludge process. Thus more energy is required.

Therefore, the greatest problem in the oxidation ditch process is to develop an aerator having a high oxygen supply efficiency.

The following five items were thus established as goals of development:

① To secure the current speed required.
② To supply oxygen efficiently.
③ To provide a sufficient mixing and agitating capacity.
④ To assure easy maintenance and control of equipment.
⑤ To assure measures that can be easily implemented for the surrounding environment.

Among the five goals of development listed above, efficiency of oxygen supply of item ② is the most important. For the current speed of item ①, a minimum current speed of 10 cm/sec was established from the results of survey conducted by Public Works Research Institute, Ministry of Construction. For the oxygen supply efficiency of item ②, 1.8 kg-O_2/kwh minimum was established since the efficiency must be greater than that of conventional aerator. For the mixing and agitating capacity of item ③, the distribution of both MLSS and DO was checked and it was determined that the standard deviation for MLSS should be less than 10% from the mean value within the same section. DO distribution was used only for reference.

In performing the assessment, the following items were considered as prerequisites for the sanitary sewage and sewage treatment facilities being considered:

① Sanitary sewage is to be ordinary sewage, such as residential waste water and business waste water (from offices, hospitals and places of business), that can be biologically decomposed.

② Sewage treatment facilities other than those being considered in the assessment are required to have the predetermined functions provided, as a rule, in accordance with "Design manual for Sewerage Facilities (JSWA)".

③ Equipment used for the aerator are required to have been manufactured under proper quality control.

(4) Results of Assessment

Equipment considered for assessment were 8 products manufactured by 8 companies: IHI Superrotor by Ishikawajima Harima Heavy Industries Co., Aqubarotor by Kawasaki Heavy Industries Co., Kubota KR-2 type rotor by Kubota Steel Co., Mammothrotor by Kurita Industries Co., Orrotor by Organo Co., Sigmarotor by Shinko Pfaudler Co., Carousel Aerator by Sumitomo Heavy Industries Co., and Draft Tube Aerator by Ataka Industries Co.

Four of them, IHI Superrotor, Aqubarotor, Kubota KR-2 Rotor, and Mammothrotor, are of horizontal shaft type, while three of them, Orrotor, Sigmarotor and Carousel Aerator are of vertical shaft type. The remaining rotor of Draft Tube Aerator will mix and agitate activated sludge and create a current speed by means of an rotor placed at the upstream side of draft tube and also supply oxygen from blower.

All of these aerators were recognized to satisfy the above mentioned goals of development.

Detailed results of assessment for these aerators are shown in separate reports, however, the outline of the assessment for the eight aerators is shown in Table 4 and Table 5.

Table 4 Comparison Table for Results of Assessment of Mechanical Aerator for Oxidation Ditch

Assessed item \ Aerator name	IHI Superrotor	Aqubarotor	Kubota KR-2 Rotor	Mammothrotor
1 Rotor type	Horizontal rotor	Same as the left	Same as the left.	Same as the left.
2 Principle	Mechanical aeration equipment developed for the oxidation ditch has a horizontal rotor which is rotated in the straight section of ditch in order to supply oxygen, mix and agitate activated sludge and cause it to flow.	Same as the left.	Same as the left.	Same as the left.
3 Summary of aeration performance (refer to drawings)	When MLSS concentration is about 2,500 to 3,500 mg/ℓ and speed of aerator is 60 rpm, the power density is 8.7 W/m^3, mixing and agitating conditions in oxidation ditch are good, and a current speed required for circulation can be obtained. In the range where the efficiency is the highest, the oxygen supply efficiency is about 2.3 kg-O_2/kwh, and thus this equipment is recognized to be efficient for the oxidation method.	When MLSS concentration is in the range of about [a] 3,300 to 4,400 mg/ℓ, power density is [b] 4.1 W/m^3, mixing and agitating conditions within the oxidation ditch are good and current speed required for circulation can be obtained. In the range where the efficiency is highest, the oxygen supply efficiency is about [c] 2.2 kg-O_2/kwh and, thus, this equipment is considered to be efficient for mechanical aeration to be used with the oxidation ditch method.	a; 1,800 ∿ 3,000 mg/ℓ b; 10.7 W/m^3 c; 2.3 Kg-O_2/kwh	a; 3,500 ∿ 5,200 mg/ℓ b; 8.1 W/m^3 c; 2.4 kg-O_2/kwh
4 Influence upon environment (1) Noise	Noise sources are driving section such as motor reduction gear and aerator which generates splashing sounds. Noise can be considerably reduced with soundproof cover for driving section and with splash-proof cover. By taking these measures depending upon the environmental conditions, most problems can be solved.	Same as the left.	Same as the left.	Same as the left.
(2) Splash of water-drops	This problem can be solved by covering the apron of aerator with splash-proof cover.	Same as the left.	Same as the left.	Same as the left.
(3) Vibration, odor	There will be no special problem.	Same as the left.	Same as the left.	Same as the left.
5 Endurance	It is reported that there has been no endurance problem since this equipment should operate for ten years.	According to the results of operation of experimental facilities for the period of [d] 9 months in the past, there has been no endurance problem. However, long-term endurance has not yet been confirmed.	d; 17 months	d; 9 months

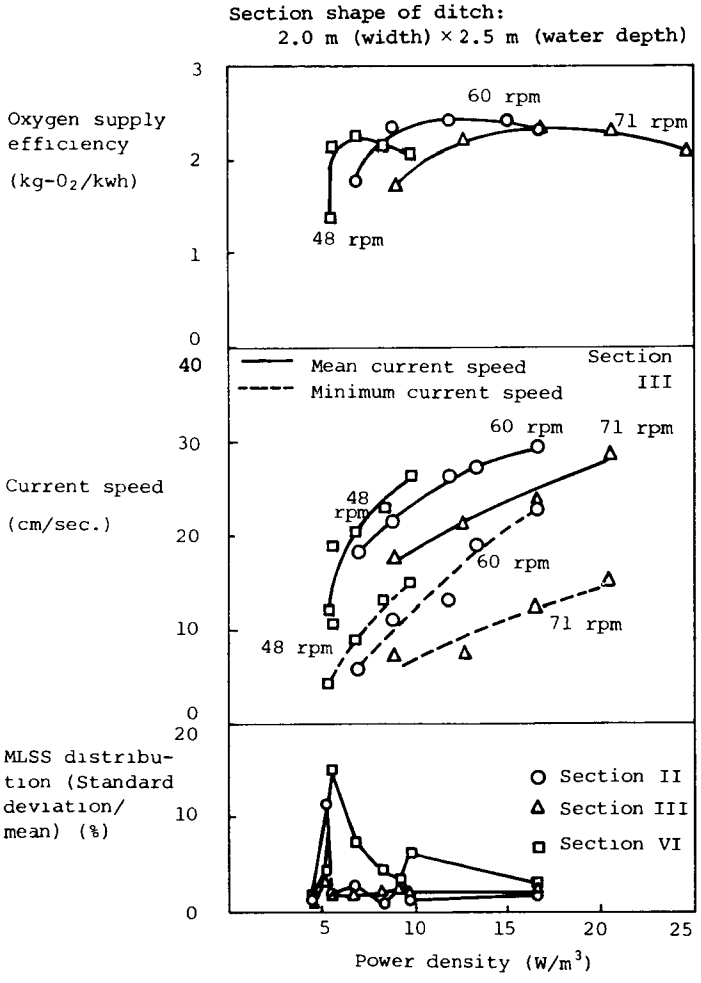
Continued

Table 4 (Continued) Comparison Table for Results of Assessment of Mechanical Aerator for Oxidation Ditch

Aerator name / Assessed item	Orrotor	Sigmarotor	Carousel Aerator	Draft Tube Aerator
1 Rotor type	Vertical rotor	Same as the left.	Same as the left.	Draft tube
2 Principle	Mechanical aeration equipment developed for oxidation ditch has a vertical shaft type rotor which is rotated at a corner in the ditch in order to supply oxygen, to mix and agitate activated sludge and to causes it to flow.	Same as the left.	Same as the left.	Mechanical aeration equipment developed for oxidation ditch method mixes and agitates the activated sludge and creates a current by the rotor at the upstream side of draft tube connected from the upstream side to downstream side of a partition while oxygen is being supplied by a blower.
3 Summary of aeration performance (refer to drawings)	a; 3,922∿4,082 mg/ℓ b; 7.6 W/m^3 c; 2.2 kg-O_2/kwh	a; 3,020∿3,120 mg/ℓ b; 5.1 W/m^3 c; 2.4 Kg-O_2/kwh	a; 4,190∿4,690 mg/ℓ b; 4.9 W/m^3 c; 2.8 Kg-O_2/kwh	a; 3,590∿4,450 mg/ℓ b; 5.0 W/m^3 c; 2.2 kg-O_2/kwh
4 Influence upon environment (1) Noise	Same as the left.	Same as the left.	Same as the left.	Noise sources are driving section such as motor reduction gear and the blower and tube outlet which generate air noise. However, noise from the driving section and blower can be considerably reduced by covering them with sound-proof covers. Noise problems can be solved by taking these measures depending upon environmental conditions. There will be no special problems.
(2) Splash of water-drops	Same as the left.	Same as the left.	Same as the left.	
(3) Vibration, odor	Same as the left.	Same as the left.	Same as the left.	Same as the left.
5 Endurance	d ; 11 months	d; 12 months	d; 1 year & 4 months	d ; 1 year & 6 months

Table 5 Overview of Aeration Performance (relations among oxygen supply efficiency, mean and minimum current speed, MLSS distribution and power density) (1/4)

IHI Superrotor



Aqubarotor

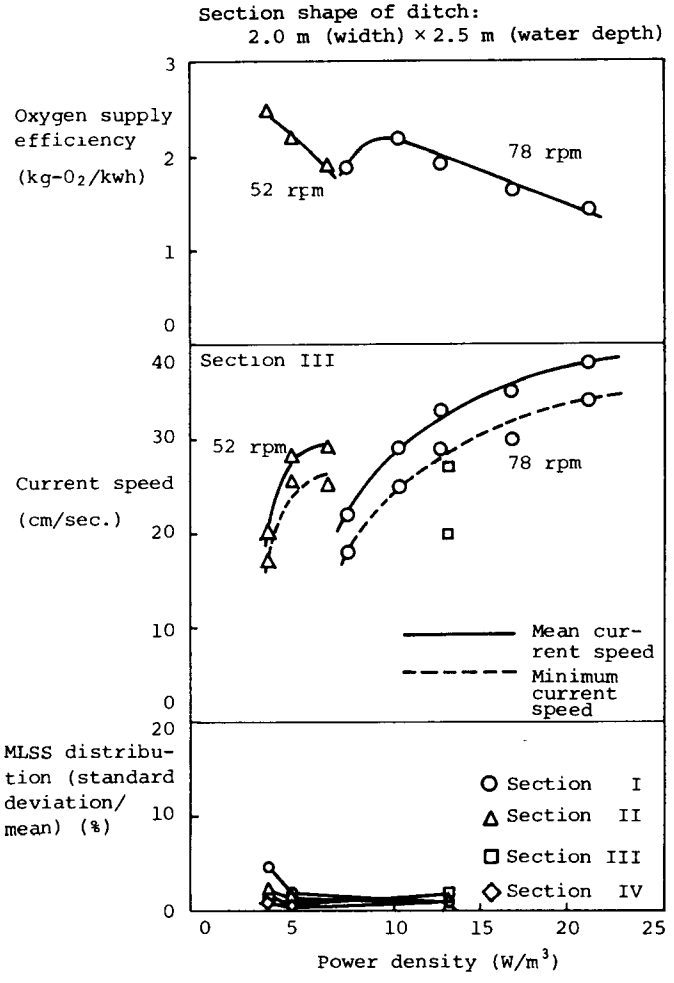
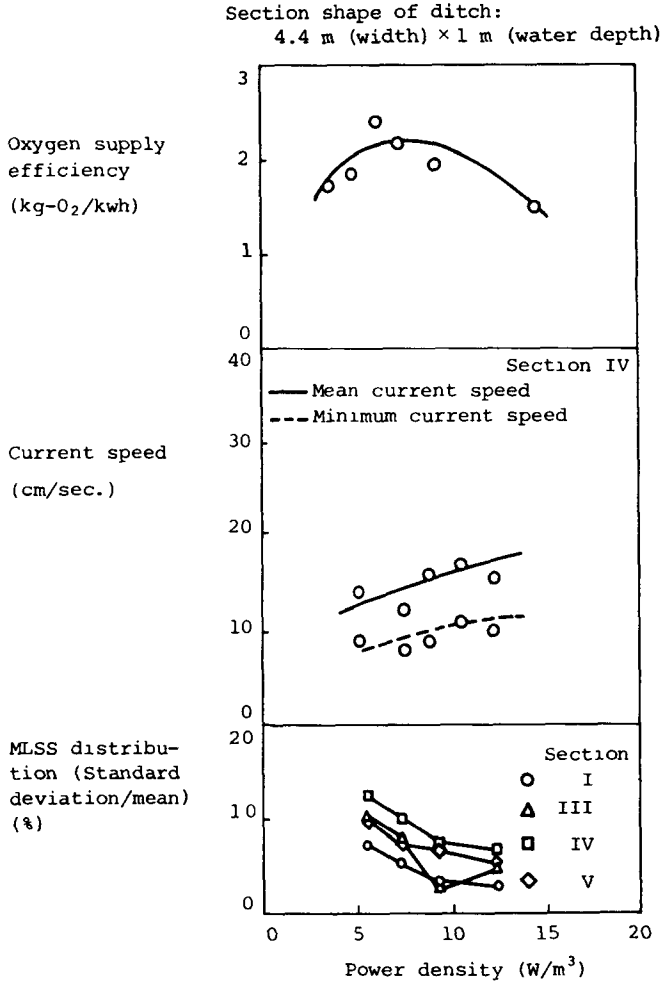


Table 5 Overview of Aeration Performance

Kubota KR-2 Rotor



Mammothrotor

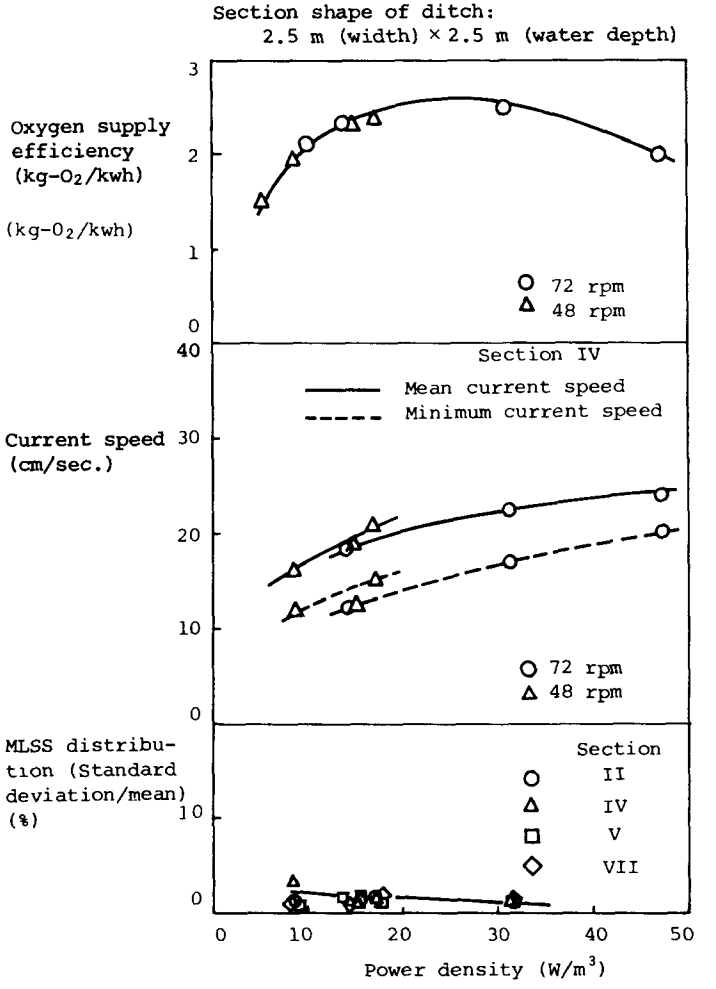
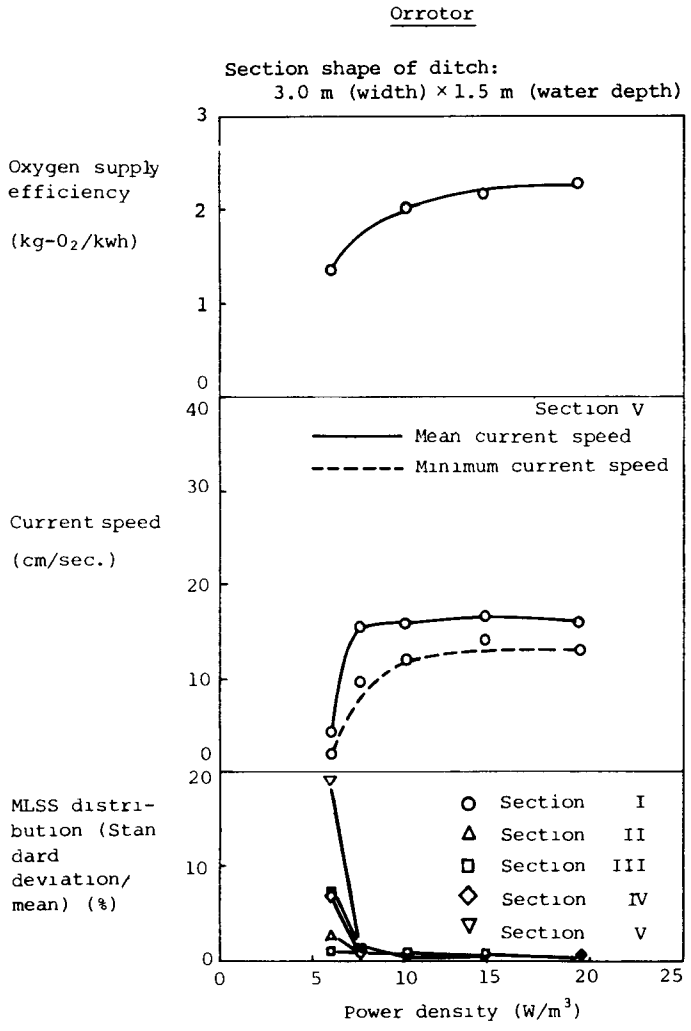


Table 5 Overview of Aeration Performance (3/4)



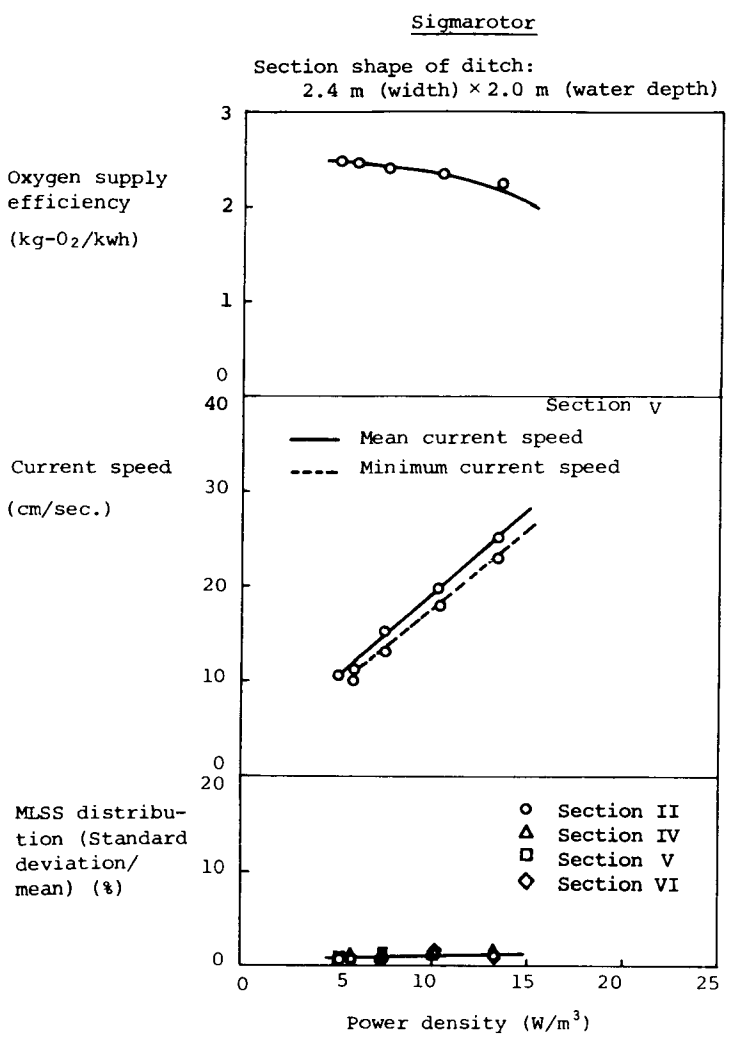
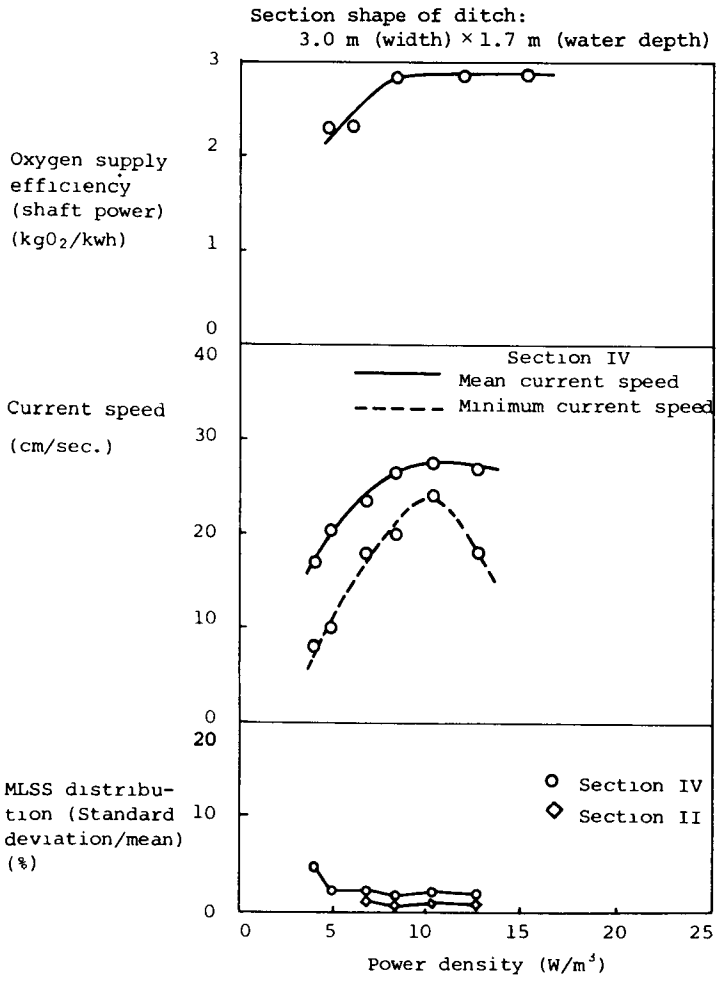
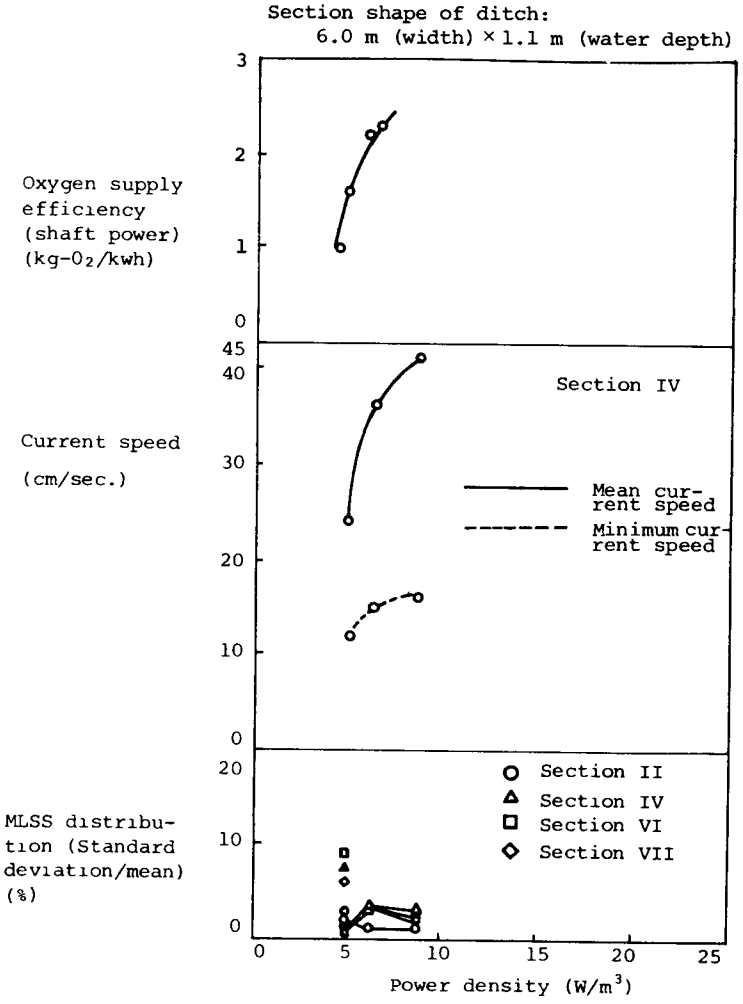


Table 5 Overview of Aeration Performance (4/4)

Carousel Aerator



Draft Tube Aerator



(5) Discussions on Oxidation Ditch Process

The technology of the aerators was assessed, lead to many discussions were of the oxidation ditch itself.

At first, the problem of load fluctuation was discussed. Load fluctuation is great in small sewage treatment plant and, even though the ability of the ditch method to withstand the fluctuation is high, it is still limited. If the amount of influent water increases, the surface overflow rates at final sedimentation basin increases, the size of floc becomes small after aeration for a long time, and settling of sludge in the ditch becomes increasingly difficult.

Because of this, a method of changing the water level in the ditch can be considered, with which the direct influence of fluctuation in water volume upon the final sedimentation basin can be prevented. This can be easily performed by connecting the ditch to the final sedimentation basin with a pipeline instead of the open channel conventionally used. Another advantage of this method is that, if water volume increases, the submerged depth of aerator blade increases, resulting in an increased amount of oxygen supply. If water volume decreases, the opposite phenomenon will occur. The idea of this method was proposed by a manufacturer who used a vertical shaft type aerator, and this idea is considered to be very interesting since it is also able to provide a stable quality of treated water.

Secondly, the problem of treatment flow was discussed. Oxidation ditch method was originally invented by Dr. Pasveer of Holland. Pasveer's ditch consists of a simple batch-operated ditch. However, treatment of continuously influent sewage by the batch method alone was difficult to perform by the control techniques at that time and, thus, the flow has changed to the present type of flow having an independent final sedimentation basin as the size of sewerage has increased.

However, sludge in the ditch does not easily settle, MLSS concentration is high, and a small surface overflow rates of 10 to 15 $m^3/m^2 \cdot day$ is requested for the final sedimentation basin,

thereby increasing the construction cost. On the other hand, in the case of batch method, a large area of ditch itself can be utilized as sedimentation ditch, which is advantageous in view of both the construction cost and treatment performance. At present, control devices such as automatic valves and level switches are widely being used and, thus, the usefulness of batch type Pasveer's ditch should be revalued.

Thirdly, the problem of ditch water depth was discussed. Conventionally, water depth of about 1 m was prescribed in view of current speed and low construction cost. Therefore, a large site was needed and installation was possible only when the land was inexpensive. However, unlike the past when work was mainly performed by manual labor, most work is now performed mainly by machinery so that deep excavation is now possible by the same machine and at the same price. Also, it is now possible to efficiently impart a current speed by improving the aeration equipment. During technology assessment, it was found that a required current speed can be obtained from an economical ditch with a depth of 2.5 m, so that now it seems to be possible to build and use inexpensive ditches.

Ninth US/Japan Conference
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Sewage Treatment Technology

CURRENT STATUS OF DEVELOPMENTS IN AUTOMATIC WATER QUALITY MONITORING EQUIPMENT FOR WASTEWATER TREATMENT

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1. INTRODUCTION

Automation of the measurement of water quality is important for the sewerage because it concerns the control of treatment processes and the monitoring of raw sewage and treated effluent. In view of this, both users and manufacturers are endeavoring to develop instrumentation technologies. Fully aware of the needs for the improvement of water quality measurement, the Ministry of Construction is conducting a contract research program, covering evaluation of the performance of automatic water quality analyzers for treatment process control, development of automatic sampler for water quality monitoring, development of automatic instruments for measuring total phosphorus and total nitrogen, and automation of analyzers for laboratory use. This paper deals with the performance evaluation of the automatic water quality analyzers and the development of automatic sampler for which there has been some progress.

2. PERFORMANCE EVALUATION OF WATER QUALITY ANALYZERS FOR TREATMENT PROCESS CONTROL

In recent years, more and more sewage treatment plants have come to use automatic water quality analyzers for water quality monitoring and automatic control of treatment processes. Almost all new plants use semi or fully automatic water quality analyzers. The effectiveness of these instruments in the operation of sewage treatment plant is largely dependent on the measuring accuracy, mechanical reliability, and maintainability. Particularly in the case of automatic control of treatment processes, the reliability of the automatic control is dependent on the reliability of the sensors used.

In general, the performance analysis of automatic water quality analyzers is conducted in two stages: (1) evaluation of basic performance using clean water, and (2) evaluation of practical performance under actual service conditions. In many cases, the performance stated on the manufacturer's catalog refers to the basic performance (1). The inspection standard of the manufactures association and the Japanese Industrial Standards guarantee the basic performance. But, regarding the practical performance (2), it is rarely the case that users can obtain substantial information from manufacturers because manufacturers usually do not have suitable sites for carrying out field tests. Some information is available from users. Such information, however, is based on an individual instrument, and is not made public in many cases.

Concerned about this situation, the Department of Sewerage and Sewage Purification, Ministry of Construction, started a field survey for evaluation of performance and reliability of automatic water quality analyzers which were commonly used in sewage treatment plants. The survey has been carried out as contract research with the Association of Electrical Engineering. The Association organized a committee consisting of members representing both manufacturers and users to conduct the survey. The committee began with the pH meter, and conducted field tests over a period of 8 months in cooperation with four manufacturers. Then, the field tests on DO meters were begun in cooperation with eight manufacturers, and will be completed shortly.

2.1 PERFORMANCE EVALUATION OF pH METERS

1) Test methodology

The evaluation of an instrument may change depending on the test methodology. After a series of heated discussions concerning methodology, the committee decided upon the following method for the purpose of examining the maintenance frequency to ensure correct measurement and the mechanical reliability of the instruments.

(1) Each participant manufacturer to offer two identical pH meters, one for operation without maintenance (unattended electrode) and one for operation with maintenance including calibration at specified interval (attended electrode).

(2) Test at two sites, one representing bad water condition (such as grit chamber) and one representing fair water condition (such as aeration tank).

(3) Design and installation method of the test instrument to be left to the discretion of each participant manufacturer; provision of an automatic cleaning device also to be decided by manufacturer.

(4) The installation of each instrument to be by its manufacturer, but maintenance to be the responsibility of the Association of Electrical Engineer.

(5) All the electrodes furnished by the manufacturers to be operated in so far as possible under the same water quality conditions.

(6) One series of tests to last 2 months with regard for the limit of unattended operation; two series of identical tests to be performed at both grit chamber and aeration tank.

(7) Weekly or fortnightly measurement of pH by a meter for laboratory use on the occasion of instrument maintenance, which is considered a good approximation of the true value.

(8) Checkup of both attended and unattended electrodes with a known pH solution for responsiveness after every series of tests.

(9) Every manufacturer to inspect and adjust the electrodes after each series of tests; the same electrodes to be used over the entire test period unless damaged.

2) pH meters tested and their installation

The pH meters furnished by the four participant manufacturers were all fitted with an automatic cleaning device. Their installation schemes are as shown in Figs. 1 to 4. Electrode holders at the grit chamber, were quite lengthy, because the depth from ground level to the water surface would reach to about 7 m.

The pH meters furnished by manufacturers A and B used disposable cartridges for the sensing elements, while those furnished by manufacturers C and D employed a combination electrode of a glass electrode, reference electrode and a thermistor. The automatic cleaning device was of the water jet type for A's test instrument, of the ultrasonic type for B's, of the continuous air bubble type for C's and of the ultrasonic/intermittent air jet combination type for D's.

3) Results

The tests started on September 10, 1981. In the first two series of tests conducted at the grit chamber, the recorders and other parts were frequently out of order. The period during which measurement was obtained compared to total test period was 91%, 58%, 88% and 89% for A, B, C and D, respectively. The failures were mostly attributed to the recorders, though some of the pH meters stopped intermittently. The failures of the pH meters themselves included electrode damage due to floating materials, deterioration of electric insulation between the glass electrode and reference electrode, the slackening of internal solution makeup pipe, disconnection of cleaning water pipe, etc.

As the return flow with high pH from the sludge treatment process is drained just upstream of the grit chamber, the measurements were affected by this return flow. Therefore, pH readings differed depending on the location of the meters, making it difficult to compare the readings directly. The water level in the grit chamber changed for more than expected, hampering normal measurement when the water level dropped so much as to expose the electrodes above water.

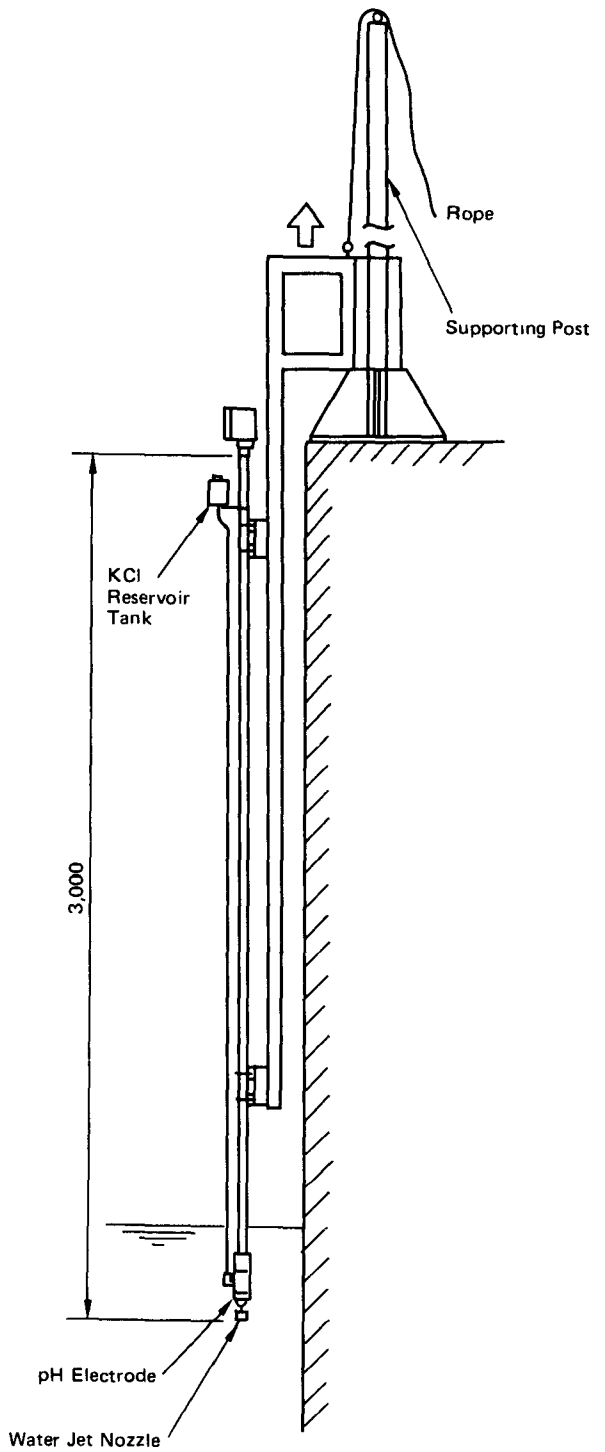

Fig. 1 Conceptional diagram of installation, model A pH meter

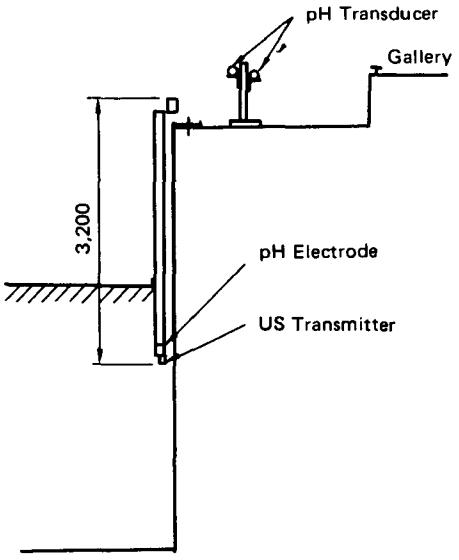

Fig. 2 Conceptional diagram of installation, model B pH meter

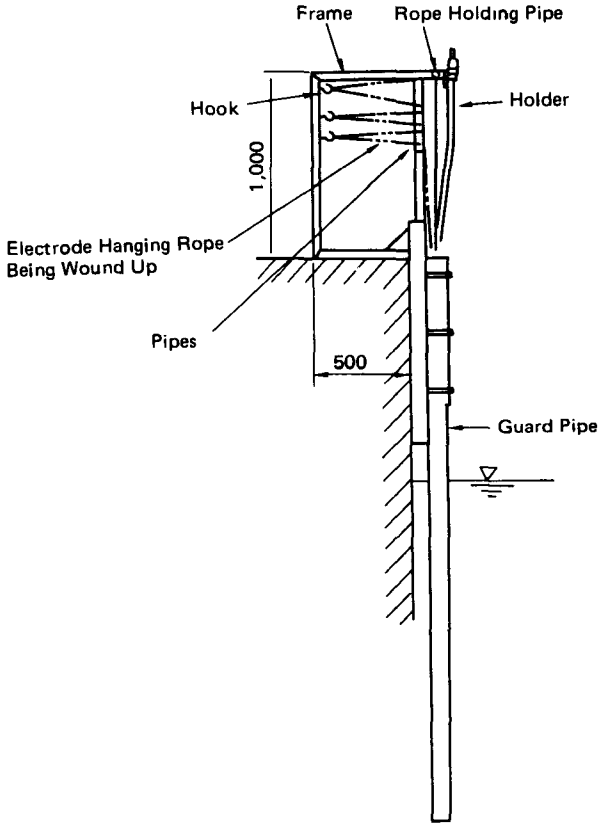

Fig. 3 Conceptional diagram of installation, model C pH meter

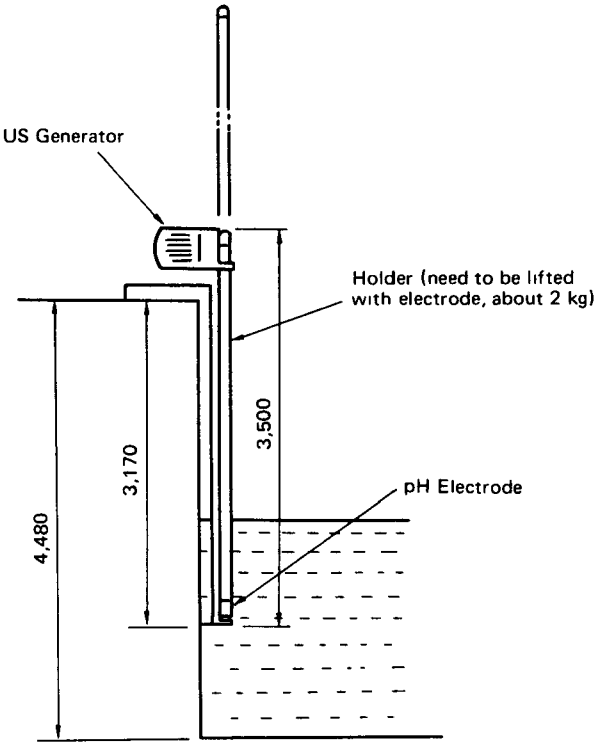

Fig. 4 Conceptional diagram of installation, model D pH meter

During the test period, there was noticed a considerable period during which the response of the unattended electrodes seemed to be lost as compared with the attended ones. One of the assumed causes was that foreign objects encrusted the entire surface of the unattended electrode and in consequence checked the water exchange between main stream and electrode surface. The degree to which the electrodes were covered with foreign objects varied widely from model to model, and the difference in the structural design of electrodes, holders and cleaning device, etc. was considered a major factor, though the location was also a factor because the electrodes in the upper reaches would catch foreign objects more easily than those in the lower reaches.

On the other hand, all the tested models experienced the same problems considered mainly attributable to poor response of attended electrodes. It was found that these troubles were developed by the clogging of the reference electrode junction with greasy matter.

The electrodes were wiped with soft paper at the time of maintenance which resulted in accelerating the cloggings. This is a matter of maintenance procedure, and every manufacturer should improve the maintenance manual with due consideration given to the quality of water to be measured.

At the grit chamber, the maintenance cycle was governed by the need for removal of bulky waste from the electrode surface. Some electrodes were covered entirely within a week. If this problem could be improved, then the measurement accuracy would be in a range of pH ±0.2 to 0.3 (about 0.5 at the worst) without maintenance for about a month, though the maintenance cycle may vary from model to model.

The tests at the aeration tank started on January 20, 1982. For this test, manufacturers removed or turned off the automatic cleaning devices. Just as in the case of grit chamber tests, two, series of 2 months tests were conducted. As the attended and unattended pH meters showed no significant difference in measurements even after the first two months, the calibration of the unattended electrodes was done only with those which the manufacturers considered necessary. The maintenance cycle for the attended electrodes for the second series was set at once every two weeks.

The period during which record of measurement was obtained compared to the total test period was 98%, 74%, 84% and 100% for A, B, C and D, respectively. Most of the failures were attributed to the failure of the recorders. Only a single failure was noticed of a pH meter when its reference electrode internal solution makeup pipe came off. Fig. 5 shows part of the recording taken after about the first two months of tests. The first series ended on March 25 and the unattended electrodes were serviced. After cleaning, the unattended electrodes were used to measure a standard solution at pH 6.92; the readings with A, B, C and D electrodes were 6.96, 7.15, 7.16 and 6.92, respectively. For a standard solution of pH 4.00, they showed respectively 4.00, 3.90, 4.05 and 3.93. Thus, A's unattended electrode was remitted from calibration, and the unattended electrodes furnished by B, C and D were calibrated.

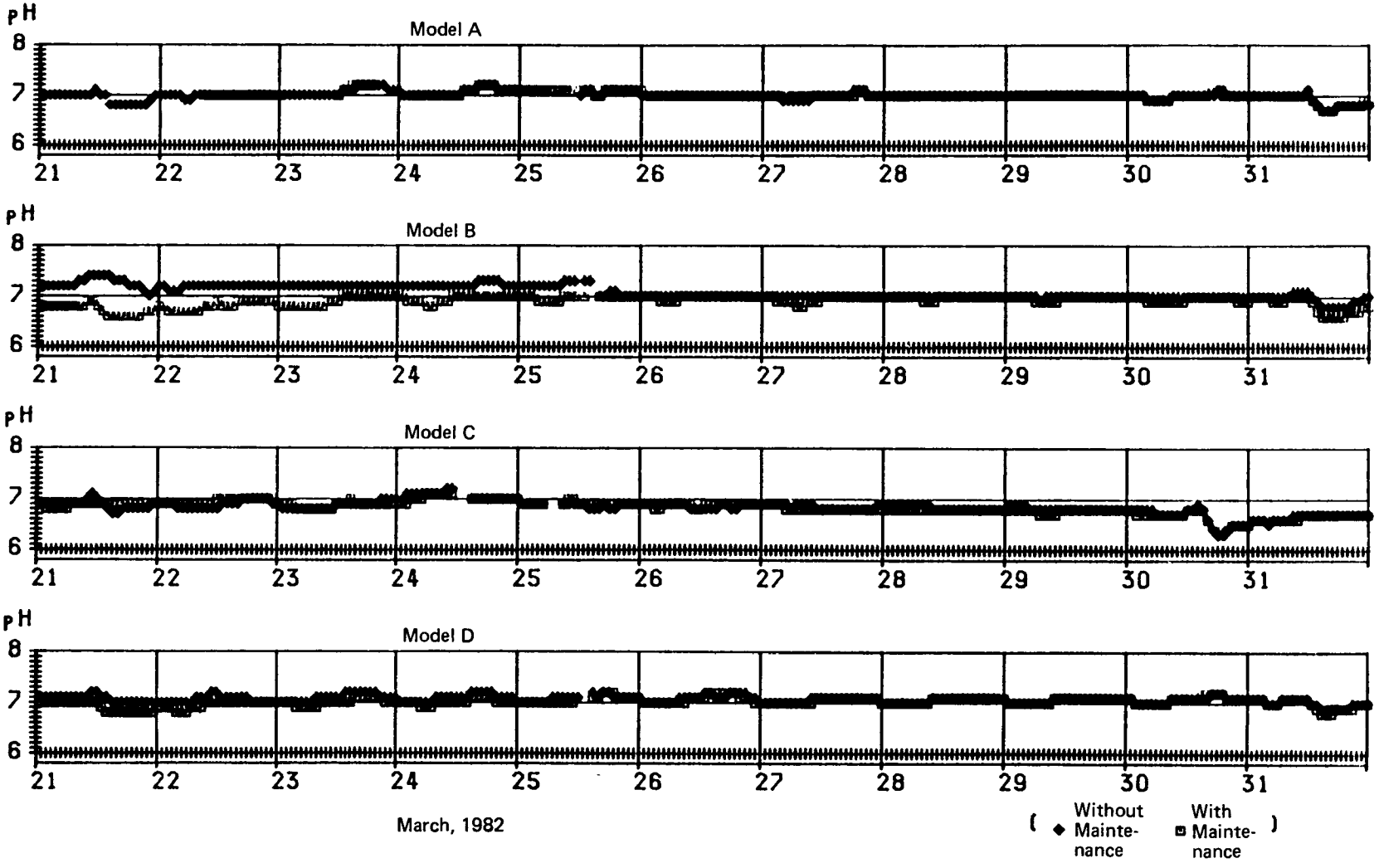

Fig. 5 Comparison of pH readings (Last portion of the lst series and first portion of the 2nd series tests in an aeration tank)

In Fig. 5, it is shown that B's unattended electrodes produced a considerable error at the end of the first series. The solids lodged between the electrode surface and ultrasonic cleaning device were considered responsible for the trouble. Though not shown in the figure, C's attended and unattended electrodes sometimes created a differential of about 0.4 pH in measurements. This may have been caused by the accumulation and subsequent dropping-off of slime on the electrode surface. The measurement errors were not always limited to the unattended electrodes, but were found with the attended electrodes as well. As in the case of grit chamber tests, the fouling of the reference electrode junction may have been the cause of the measurement errors by the attended electrodes.

A record of measurements taken about 1 week after the start of the second series (about 80 days after the start of the first series) is shown in Fig. 6, and a record of measurements taken about 2 months after the start of the second series (about 4 months after the start of the first series) is given in Fig. 7. According to Fig. 6, it is found that C's unattended electrode began to record a measurement error about 10 days after the start of the second series. About 2 months after the start of the second series, C's recorder got out of order and, therefore, record was not obtained. Errors went up and down cyclically just as in the case of the first series. As regards B's unattended electrode, similar fluctuations though with much less magnitude were noticed.

As is evident from Fig. 7, the measuring was of fairly good standard. Judging from the May 15 data showing pH changes, the unattended electrodes showed good response characteristics.

After tests, a standard pH solution was measured with the unattended electrodes for the purpose of checking their response time. A's and B's electrodes showed a response time of 2 to 3 min., while C's and D's were about 10 min. The difference between the pH value of the standard solution and the measured pH value was within 0.2 by A's and D's and in the range of 0.2 - 0.6 by B's and C's.

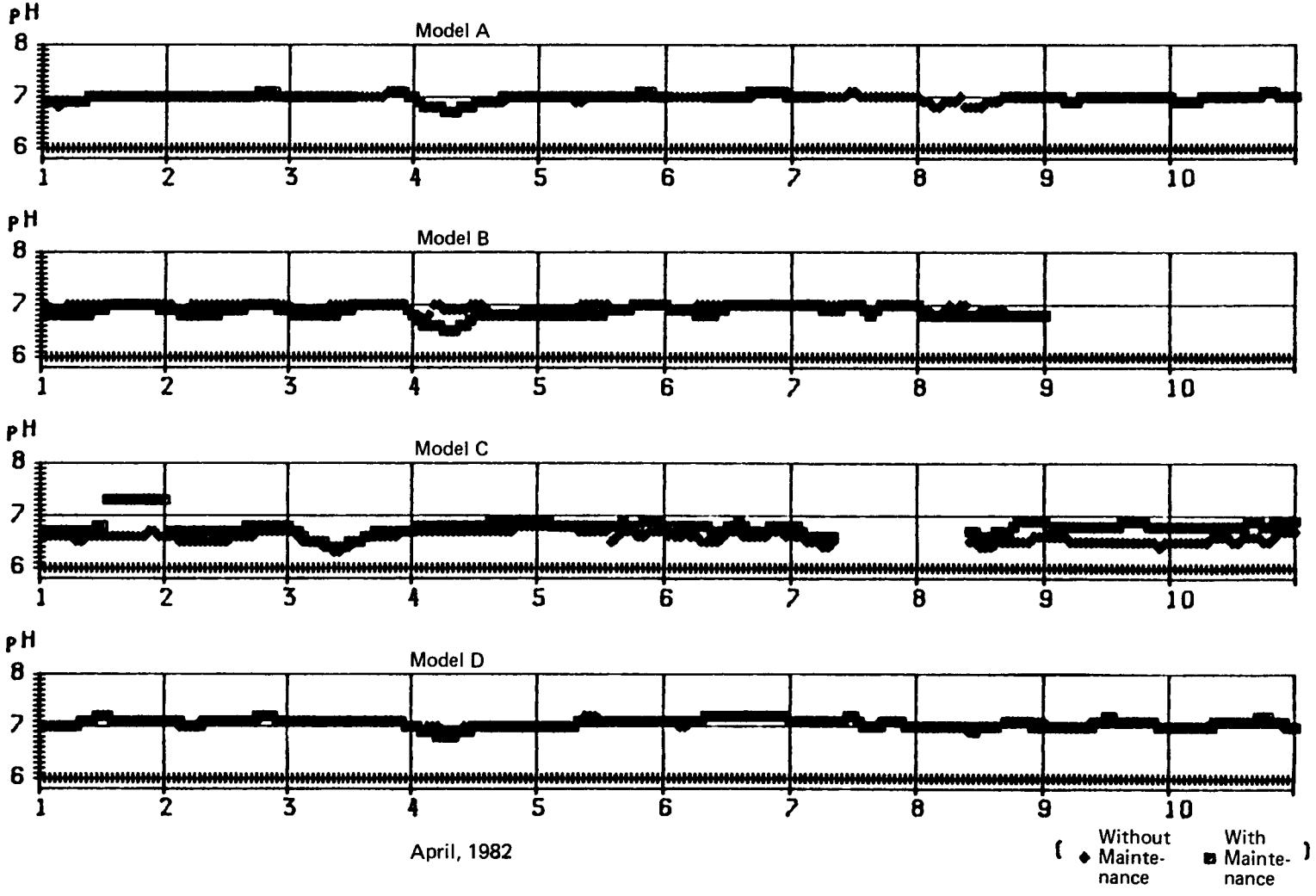

Fig. 6 Comparison of pH readings (First portion of the 2nd series test in an aeration tank)

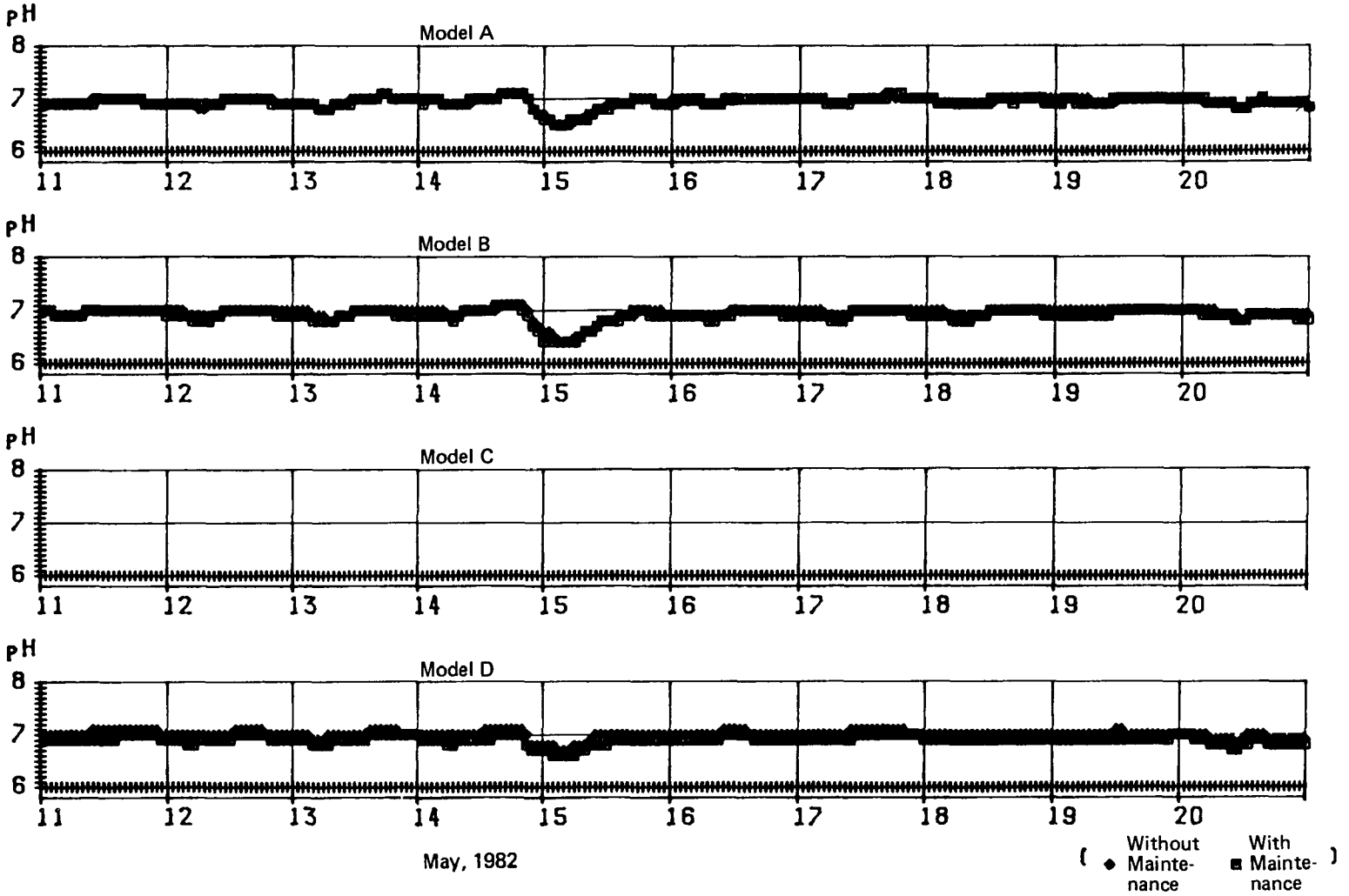

Fig. 7 Comparison of pH readings (Last portion of the 2nd series test in an aeration tank)

4) Summary

So long as the electrode surface can be prevented from becoming encrusted with foreign matter, the pH measurement of influent at the grit chamber will be accomplished with an accuracy of ±0.2 to ±0.3 (±0.5 max.) if the pH meter is maintained about once a month. All the models need much improvement from the viewpoint of structural design. The electrode holder and automatic cleaning device should be designed not to become easily tangled with large floating substances. At the grit chamber, the flow velocity is comparatively high, and the distance is great between the ground level and the water level. Accordingly, the electrode holder becomes awkwardly large, making the maintenance work very difficult. Cleaning the electrode surfaces with a soft material like tissue paper or gauze is liable to clog up the reference electrode junction. Cleaning with a detergent or acid solution is a better method.

With regard to the aeration tank measurement, maintaining an accuracy level of ±0.2 in pH value required a maintenance cycle of about once a month for some models and once every two to three months for other models. This difference would have been caused by the electrode installation method rather than by the performance of the electrode itself. Most of the electrodes could be cleaned satisfactorily with soft materials like tissue paper, but for some models particularly when their reference electrode junctions were made of ceramices this method was inadequate as in the case of measurement at the grit chamber.

2.2 PERFORMANCE EVALUATION OF DO METERS

1) Test methodology

Basically the same methodology as with the pH meter was used with the exception of the following points with regard for DO meter characteristics.

(1) At the sewage treatment plant, the DO meter is used mostly in the aeration tank. Therefore, the test site was limited to the aeration tank only.

(2) One series of tests was set at 4 months as it was considered that the instruments would serve for a considerably long period without maintenance at the aeration tank. The total test period was set at 8 months to repeat the same tests by two series.

In the pH meter tests, there was a considerable period during which the recordings were unavailable because of recorder failures. For this reason, each manufacturer was asked to use a recorder which was unlikely to fail.

2) DO meters tested, and their installation

Eight manufacturers participated in the DO meter test program. Of these eight, two manufacturers used electrodes of the same make, and another two manufacturers used electrodes of another make. The remaining four manufacturers used electrodes of their own. Therefore, six types of electrodes were used in all. E's and F's electrodes were of the galvanic type without membrane, and the other manufacturers used the galvanic electrodes with membrane. The DO meters were installed at the tail end of the aeration tank at an interval of about 3 m, so that the DO meters could be tested at a place with large DO fluctuation. Figs. 8 through 13 show the installation schemes. Model E and F DO meters employed the electrodes of the same make, and G and H DO meters also used identical electrodes. For this reason, the installation schemes for E and H are not shown.

A's electrode was installed in a protection tube, permitting removal for maintenance by pulling up on the electrode cable. Other manufacturers' used an electrode integrated rigidly with its holder. The size and weight of the electrode holders vary considerably for different manufacturers. This factor affected the ease of maintenance also greatly.

The electrodes of A, C and D were not fitted with an automatic cleaning device. B's electrode used a continuous air bubble cleaning device, and E's and F's used a continuous filing for cleaning. G's and H's employed an automatic intermittent water-jet cleaning device.

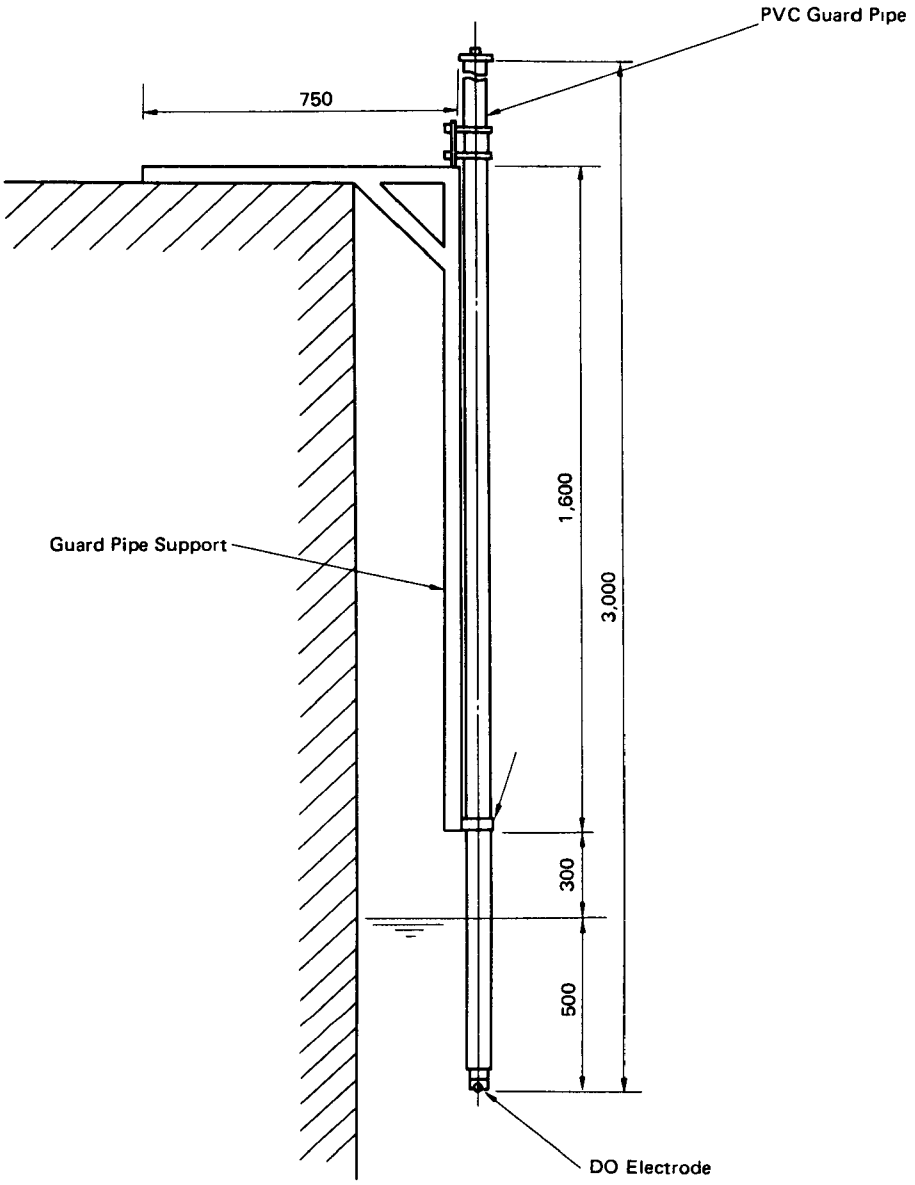

Fig. 8 Conceptional diagram of installation, model A DO meter

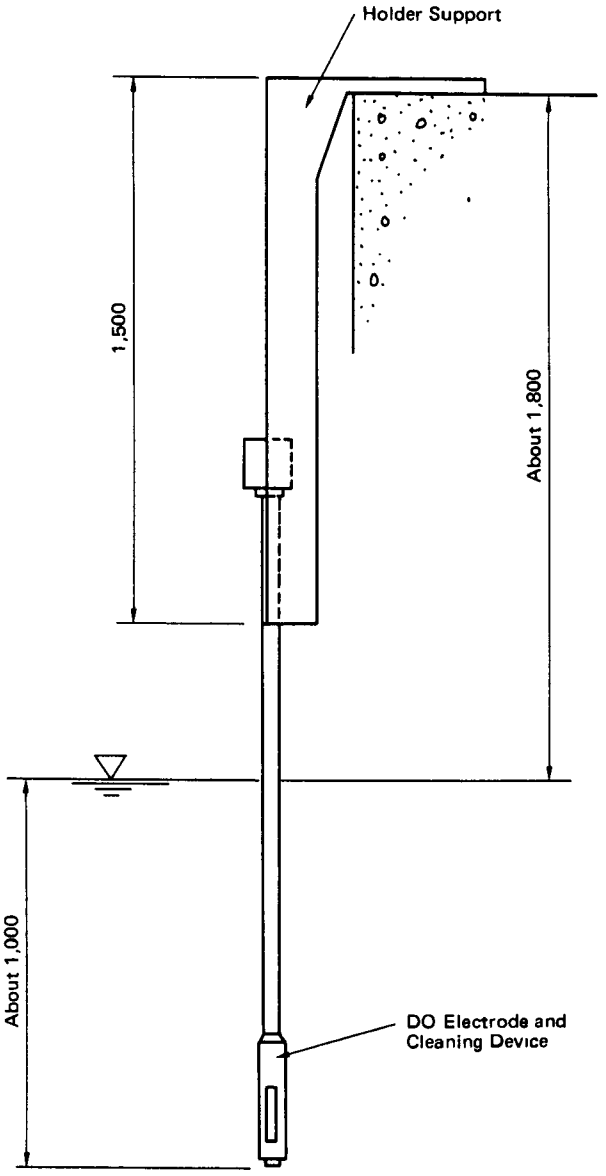

Fig. 9 Conceptional diagram of installation, model B DO meter

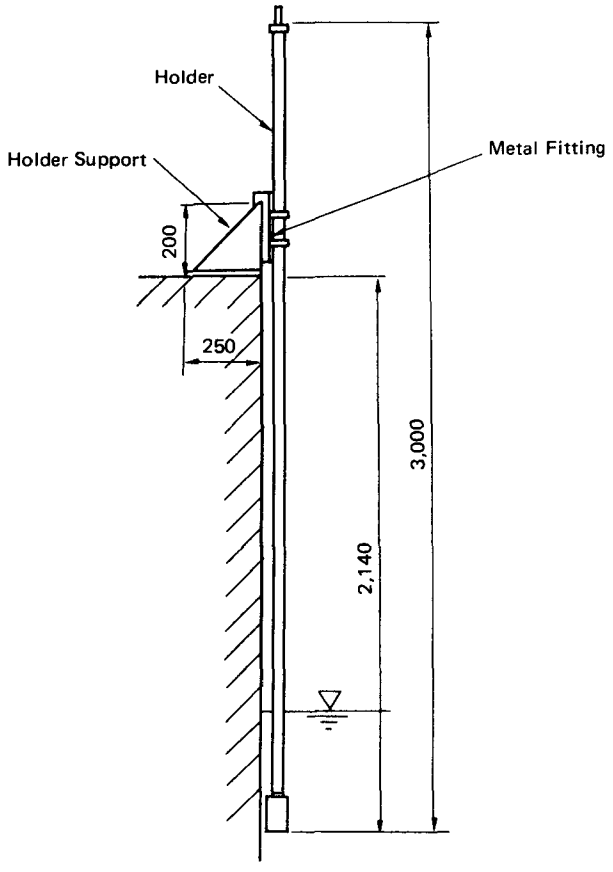

Fig. 10 Conceptional diagram of installation, model C DO meter

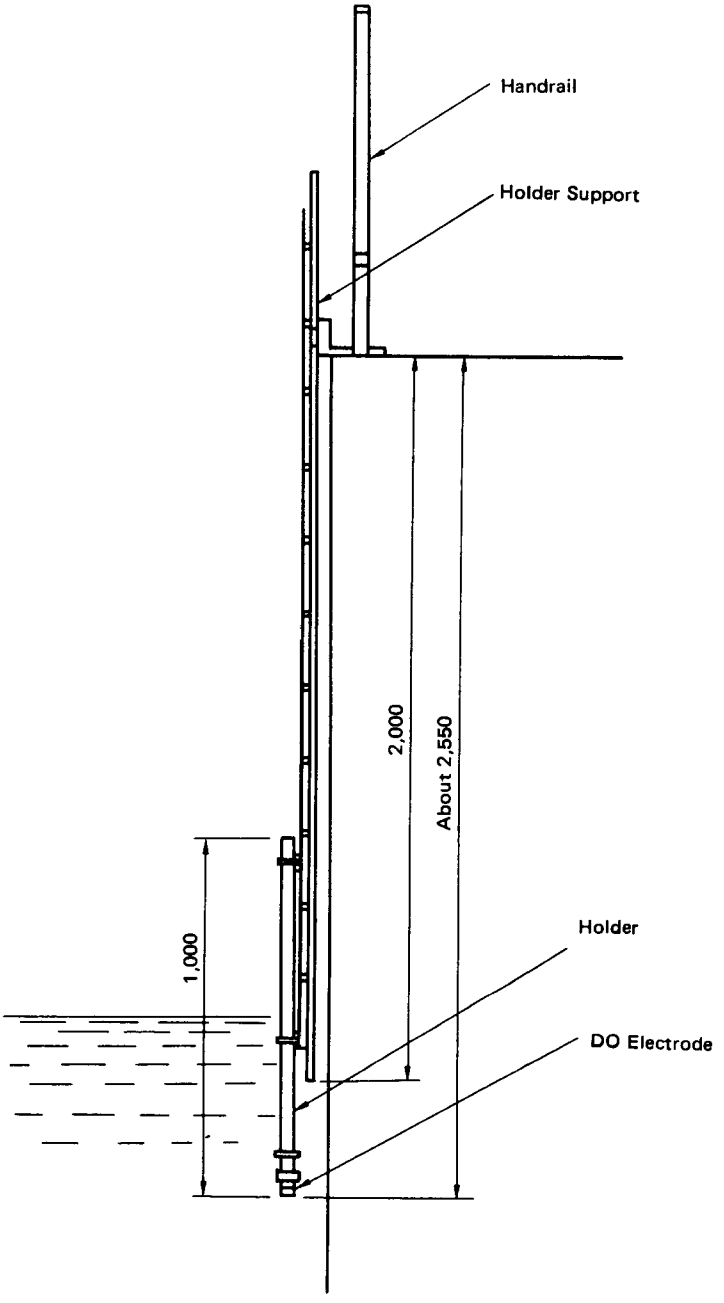

Fig. 11 Conceptional diagram of installation, model D DO meter

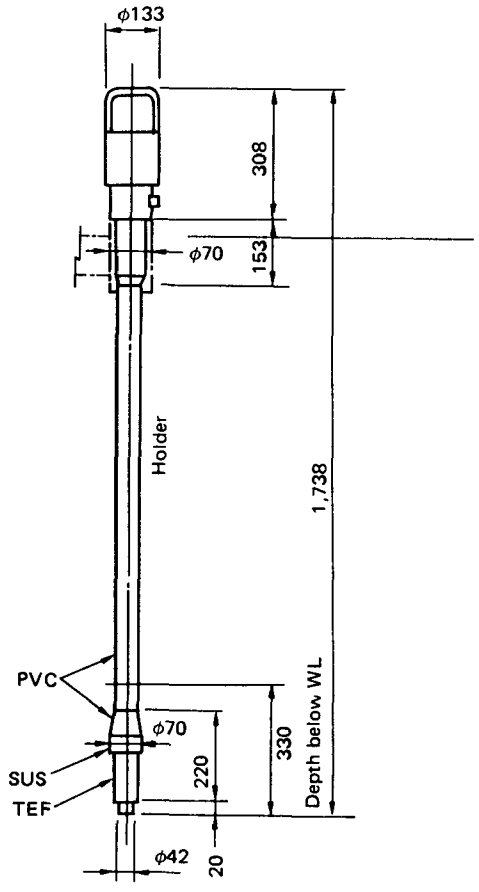

Fig. 12 Conceptional diagram of model F DO meter

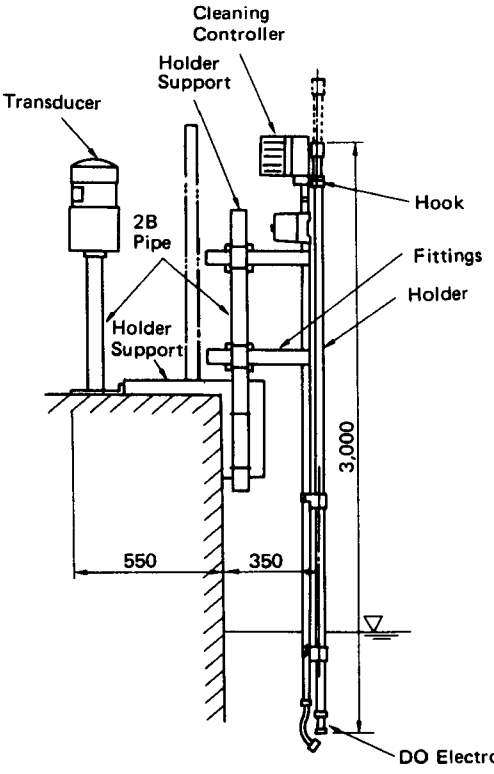

Fig. 13 Conceptional diagram of installation, model H DO meter

3) Results

The first series of tests started on October 25, 1982, and ended on March 2, 1983. The second series is now in progress. In the first series, almost every DO meter could furnish data for the entire period as the recorder failure was minimal.

Figs. 14 through 18 show examples of DO recordings taken 4 weeks, 8 weeks, 10 weeks and 17 weeks after the first series began. In these figures, the solid lines show the measurements by the attended electrodes, and the dotted lines by the unattended electrodes. In the 4th week, a slight difference in measurement between the attended and unattended electrodes of A, D, E and F were seen. Of them, the measurement errors of A, E and F were found ascribable to the attended electrodes. The errors with A's attended electrode was due to the fact that the calibration was made before the temperature of the electrode internal solution was balanced with that of the standard solution.

As the ambient temperature fell, the temperature difference between the aeration tank water and the standard solution reached 15 degrees centigrade or more, and the trouble occurred probably because the volume of internal solution in A's electrode was relatively large. The troubles with E's and F's electrodes were found to be invited because the ionic strength of standard solution used at the time of calibration was below a specified value. F's attended electrode disclosed erratic measurements on and off in the 4th week, and thereafter. Later the trouble was identified as attributable to the exposure of the electrode due to a drop in water level. D's measurement error was caused by the degradation in sensitivity of the unattended electrode as its membrane surface began to be contaminated.

Entering the 8th week, the measurement by D's unattended electrode fell further as shown in Fig. 15. Also, H's unattended electrode showed a slight decline in measurement. The measurements by the unattended electrodes of A, B and C tended to fall slightly in a higher range of DO.

According to the recordings for the 10th week appearing in Fig. 16, it is found that the measurements by the unattended electrodes of D, H, etc. fell markedly. As there was little point in continuing tests with these electrodes, it was decided that the manufacturers could maintain the unattended electrodes if they wanted to, and could restart the test. On January 11, D and H conducted the servicing of unattended electrodes, and on January 18, A and B conducted servicing. After washing of the membrane surfaces, these electrodes showed little or no drift in both zero and span; the drift was about 1% for most electrodes and 5% at the worst. It was therefore concluded that the degradation in measurements must have been due to the fouling of membrane surfaces. C, E, F and G chose to continue the four-month continuous tests as originally planned without servicing the unattended electrodes.

According to Fig. 17 showing the measurements in the 14th week, the electrodes showed satisfactory values on the whole, though there was some difference in measurement between the attended and the unattended. The differences of E's and F's electrodes were fairly significant.

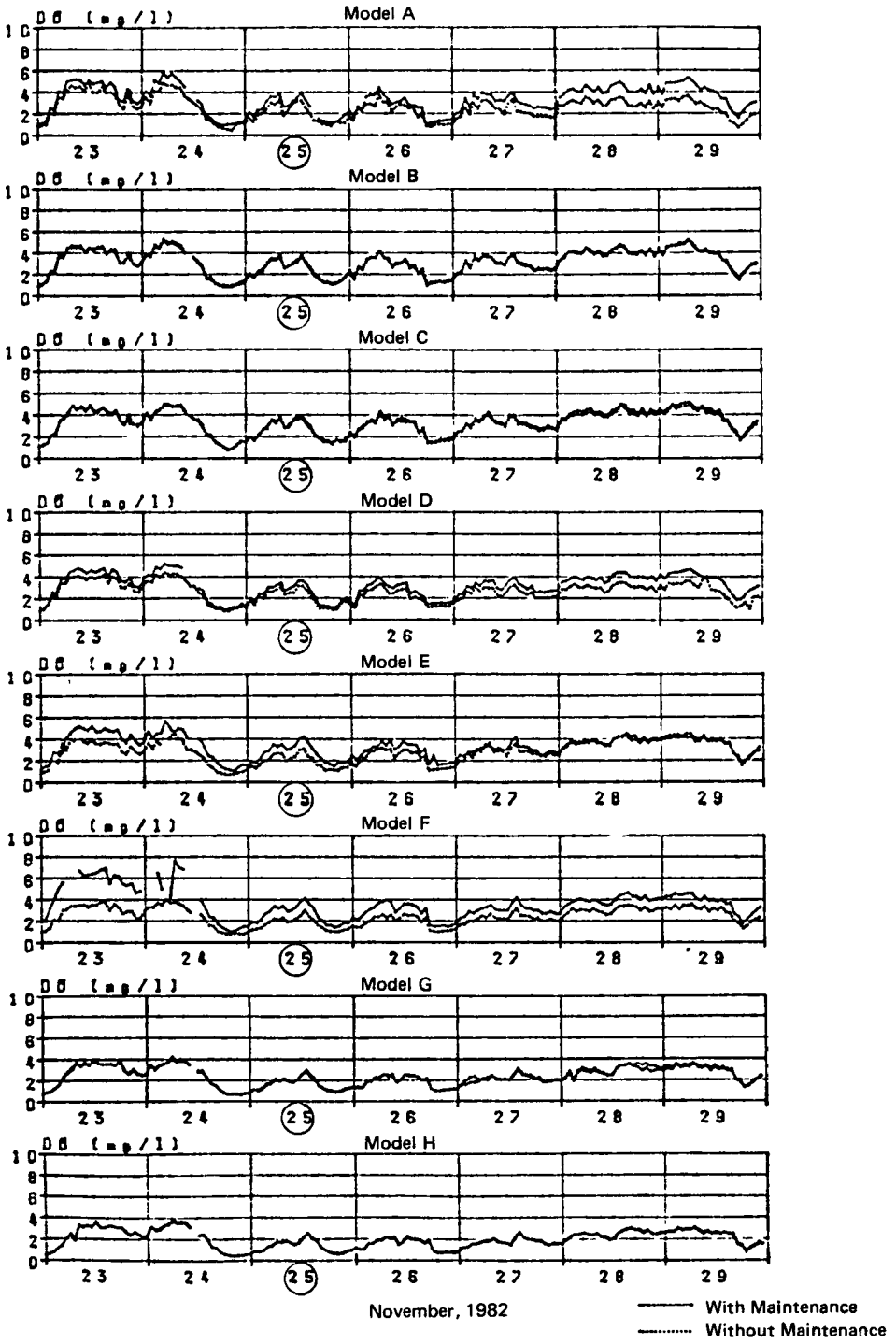

Fig. 14 Comparison of DO readings (4th week)

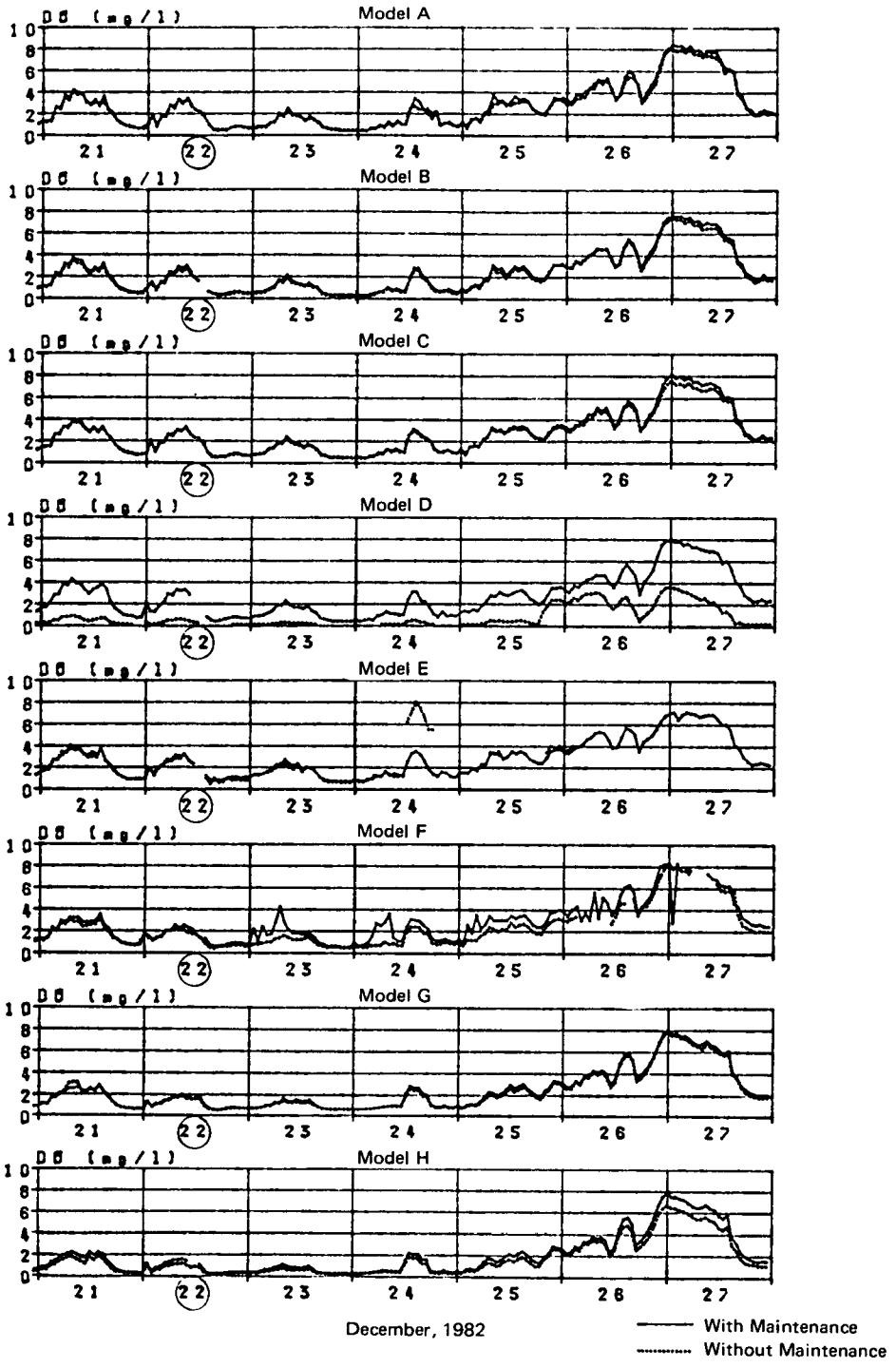

Fig. 15 Comparison of DO readings (8th week)

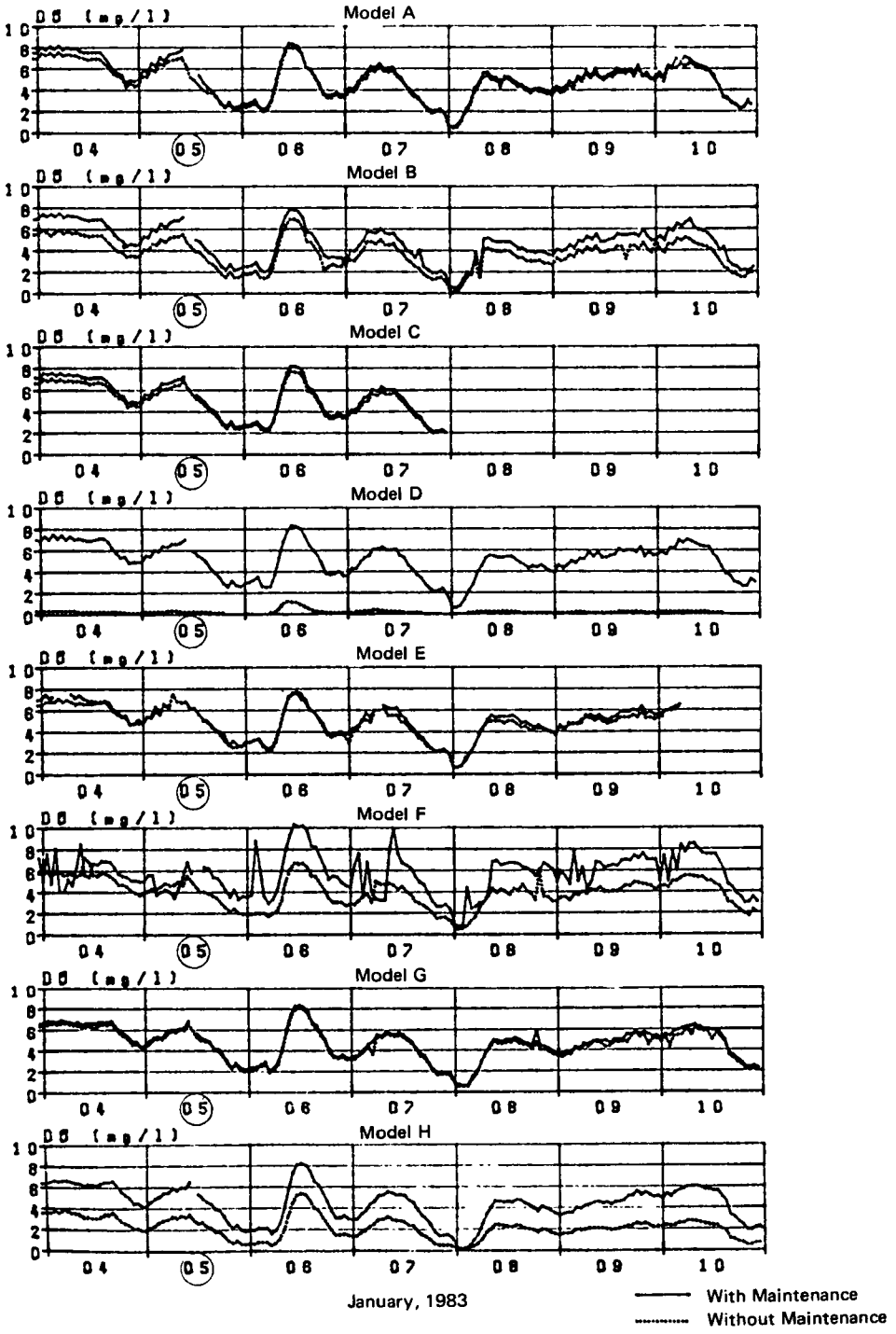

Fig. 16 Comparison of DO readings (10th week)

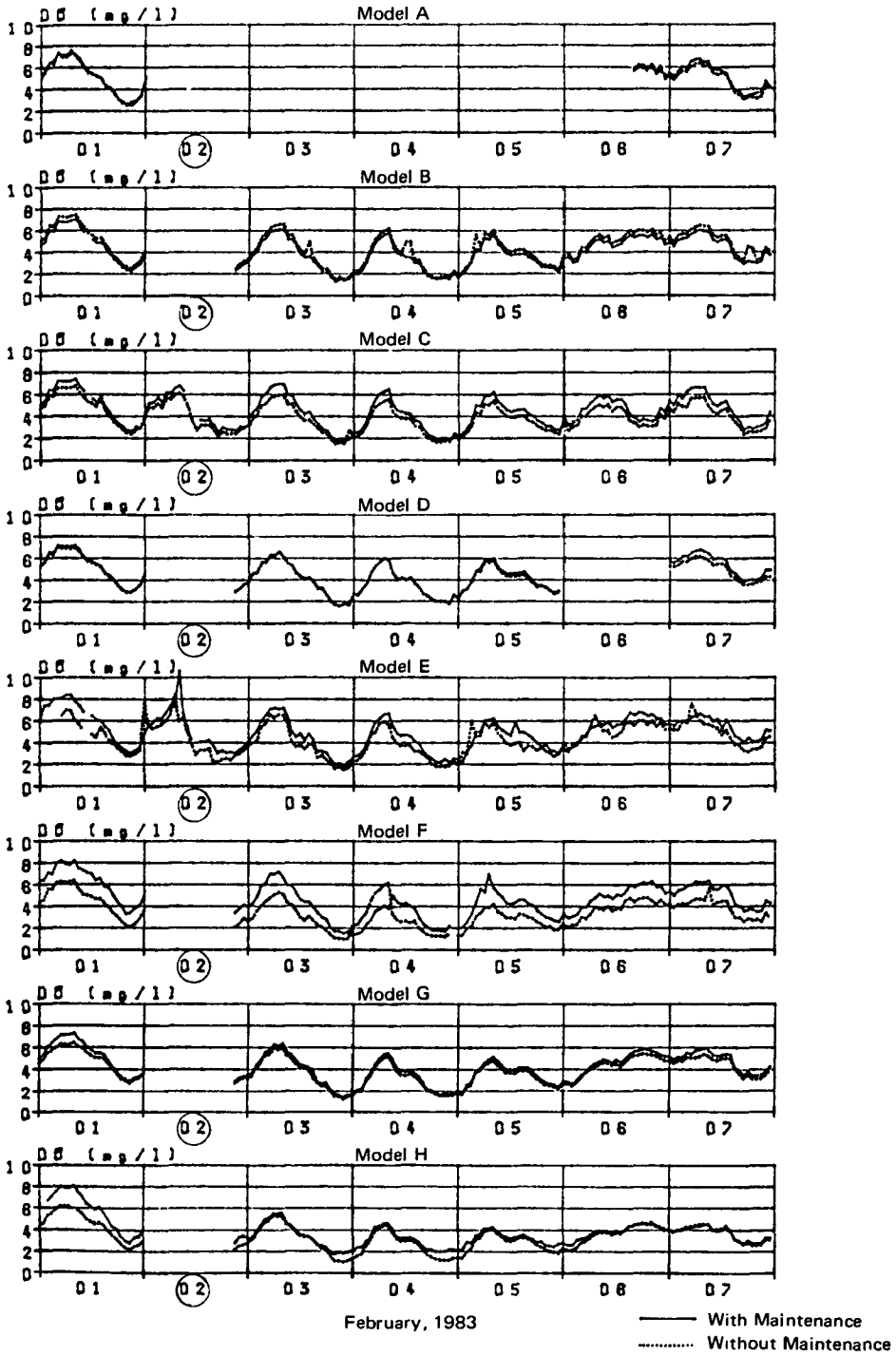

Fig. 17 Comparison of DO readings (14th week)

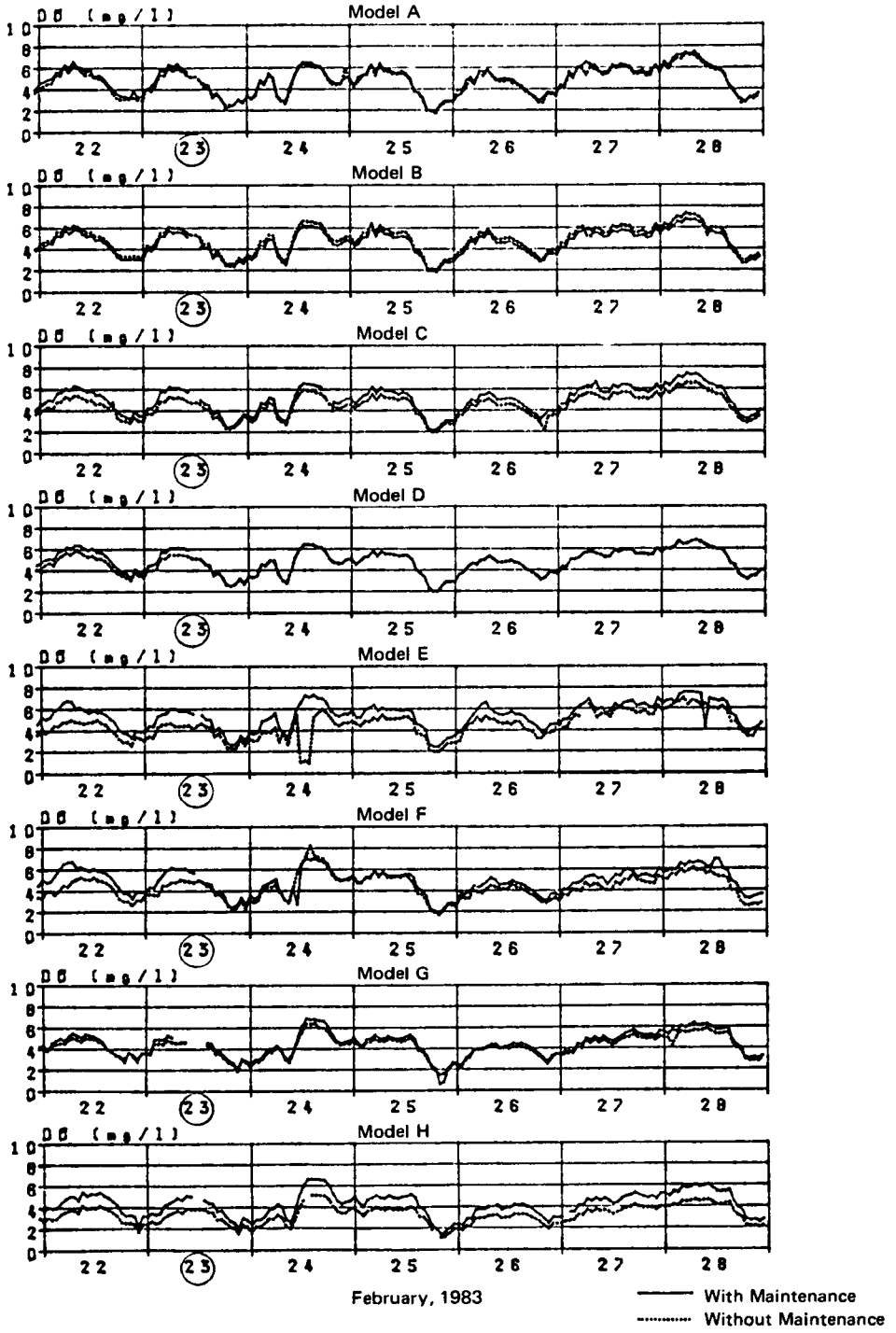

Fig. 18 Comparison of DO readings (17th week)

There were two probable causes: a reduction of flow through the measuring cell of the unattended electrode due to the clogging of flow passage, and a failure to calibrate the attended electrode accurately in winter when the saturation DO concentration exceeded the full scale of the instruments (12 mg/ℓ).

Entering the 17th week, some models began to show a large measurement error as shown in Fig. 18. But C's and D's unattended electrodes which ran on for four months without any care proved their excellent performance.

The repairs and adjustments made during the test period included the following. The membraneless galvanic electrodes furnished by E and F showed wide variances in measurement in the early stages of the test period. To solve this problem, a condenser with larger capacity was put into the electrical circuit to increase the integral time constant. But this detracted from their response, taking much time for calibration work. One time each A's and H's attended electrodes experienced a bubble trapped to inner side of the membrane. The membrane or electrode were replaced.

Automatic cleaning devices were found to be effective. G and H used identical electrodes, and their automatic cleaning devices were also the same type, using a water jet system. But, H's unattended electrode deteriorated faster than G's. Probably this was due to delicate differences in design such as cleaning intervals, water jet pressure, and clearance between the nozzle and membrane. In other words, the automatic cleaning is effective so far as it is carried out properly. C tested electrodes without automatic cleaning device, but they also installed an electrode with an automatic intermittent air jet cleaning device at the same place as the test electrode for the purpose of investigating the effects of automatic cleaning. The electrode with the cleaning device was operated for four months without servicing, but its measurements showed quite a good agreement with those obtained by the attended electrodes even after four months of tests.

4) Summary

The frequency of maintenance required for continuous measurement of DO in the aeration tank at an accuracy of ±0.2 to ±0.3 mg/ℓ will be about once a month at the shortest and once every two to three months at the longest. Most of measurement error was attributed to the fouling of electrode membrane. This can be eliminated by washing the membrane surfaces.

The absolute value of error is high when DO is high, and low when DO is low. It is found that the contamination of electrode membrane surfaces little affects the response of the electrode. The ease of maintenance differs considerably with models. Those models which are difficult to maintain are likely to be operated without good maintenance when actually used. The manufacturers are required to expend more effort for improving the maintenability.

3. AUTOMATIC SAMPLER FOR WATER QUALITY MONITORING

It is of great importance not only for proper treatment of sewage but also from the viewpoint of sludge disposal to monitor whether the concentrations of hazardous substances in industrial effluents discharged into public sewer system are below the limits specified in the pretreatment standards. At present, the harmful substances contained in the industrial effluents are usually sampled and analyzed manually. An automatic sampler that detects abnormalities in water quality and takes samples automatically would be an useful tool to establish a more effective monitoring system. In view of this, efforts have so far been made to develop an automatic sampler which can be installed at a sewage treatment plant, pump station or within the manhole of a public sewer and which is equipped with a pH meter that responds to abnormal pH values and signals the sampler to take samples automatically. The antecedents of the development of this sampler have been reported at the preceding conference.

This kind of automatic sampler, however, cannot identify the sources of pollutants. It is therefore strongly urged to develop an automatic sampler that is small enough to be installed within a connection manhole between public and private sewers.

At present, the development of this sampler has been promoted by the Association of Electrical Engineering at the request of the Ministry of Construction. Three different prototypes are under field tests for improvement. The following is an overview of the current development status.

1) Specifications

The major performance characteristics required of the automatic sampler are as listed below.

(1) To be small enough to be installed within the connection manhole (standard size: 300 mm in diameter and 600 to 650 mm in depth).
(2) To be made of corrosion-resistant materials; and to accommodate electrical components into a totally sealed enclosure.
(3) To be able to sample more than 100 mℓ at a time, and to be able to sample more than 4 times in succession.
(4) To sample water when the pH value of the water has run in excess of a preset range, when the water level in the manhole has exceeded a preset value, or when the above two conditions have concurred, or to sample water at predetermined intervals.
(5) To be capable of recording or memorizing the times sampled and pH values.
(6) To be powered with a battery with a capacity to run the sampler for at least a week within the connection manhole.
(7) To permit free adjustment of waiting time from the installation in the connection manhole to the start of operation and of the standby time between two successive sampling operations.
(8) To weigh less than 30 kg in gross weight or preferably less than 25 kg.

2) Prototypes and field tests

According to the specifications above, three different prototypes were produced. They are being improved through field tests. Shown on Photo 1 is A's prototype which uses a bellows-shaped sampling bottle as illustrated in Fig. 19 instead of a pump. The sampling bottle is set contracted in advance. When a sampling signal is given when an abnormal pH value is detected, the sampling bottle expands to take a sample by suction. There are four sampling bottles in all, each with a sampling capacity of about 120 mℓ. For the purpose of protecting the pH electrode, a trough with a semicircular cross section contoured to fit the invert is provided at the bottom. The weirs on both sides of the trough hold water in the trough even when there is no flow in the manhole. This automatic sampler is equipped with an IC memory capable of memorizing about 1,000 pH data, and can be used as an automatic pH monitor. What can be memorized is pH values measured every hour on the hour and pH values measured at an interval of 1 min. after sampling.

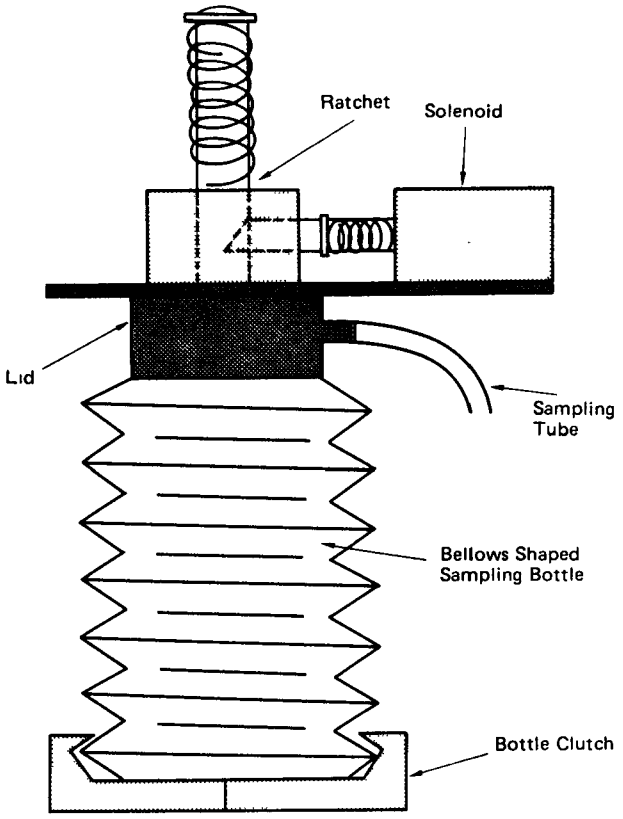

Fig. 19 Sampling mechanism

Prototype B is as shown on Photo 2. It uses a vacuum pump to take samples into a glass sampling bottle. The prototype has six sampling bottles in all, each having a capacity of 600 mℓ. The bottom is provided with a trough having the same shape and function as A's.

Prototype C is shown on Photos 3 and 4. The sampling is carried out by making use of a vacuum pump just as with Prototype B. It has six sampling bottles with a capacity of about 200 mℓ each. This prototype is not equipped with a trough, instead a slitted cylinder is used to accommodate the pH electrode and sampling tube as shown on Photo 4. At present, field tests are under way using these automatic samplers, and efforts are being made to improve their performance with emphasis on the mounting methods of pH electrode and sampling tube head. It is expected that they will be refined enough to withstand practical usage.

Photo 1

Photo 2

Photo 3

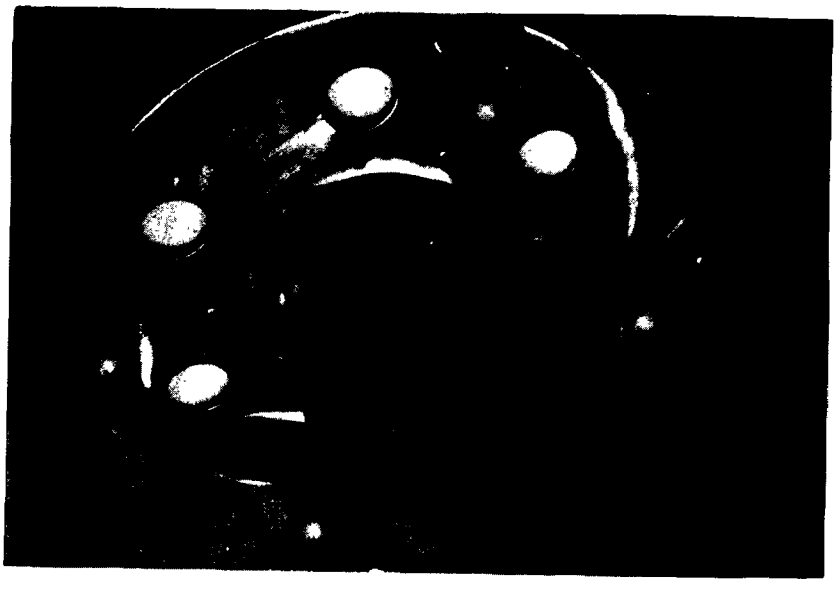

Photo 4

4. AFTERWORD

As regards the automatic water quality analyzers for treatment process control, the performance evaluation of pH meters and DO meters has been nearly completed, and the maintenance frequencies necessary for required accuracy and the modes in which the errors appear have been clarified. The series of field tests conducted would have been usefull and rewarding to all the participating manufacturers in various ways as they could learn what the actual installation environment is. The automatic sampler for water quality monitoring would go a long way toward improving the monitoring systems for industrial effluents. Some sewerage authorities have already started a study to apply this kind of automatic sampler for reinforcing their water quality monitoring systems.

To close this paper, the author would like to express profound gratitude to the engineers of the sewage treatment plants and manufacturers for their unlimited assistance and cooperation extended during the survey.

Ninth US/Japan Conference
on
Sewage Treatment Technology

NITROGEN AND PHOSPHORUS BIOLOGICAL REMOVAL SYSTEMS RESEARCH

September 19 ~ 21, 1983
Tokyo, Japan

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Research and Technology Development Division
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1. INTRODUCTION

The phenomenon of eutrophication in Japanese closed waters has been in progress for a long time, but it is only since about 1979 that definite countermeasures have begun to be taken. Specifically, it was in 1979 that the Environmental Agency first indicated its guidance policy concerning phosphorus reduction in the Seto Inland Sea. Furthermore, prefectural ordinance effluent standards for nitrogen and phosphorus were set for Lake Biwa and Kasumigaura, as shown in Table 1, and definite control of nitrogen and phosphorus was commenced accordingly.

Table 1 Nitrogen and Phosphorus Effluent Standards at Lakes Biwa and Kasumigaura

(mg/ℓ)

	Volume of discharge (m^3/day)	Total nitrogen		Total phosphorus	
		Existing	Newly Constructed	Existing	Newly Constructed
Lake Biwa	30 or more	20	20	1	0.5
Kasumigaura	20 ∿ 100,000	20	20	1	1
	100,000 or more	15	15	0.5	0.5

In December, 1982, the Environmental Agency announced environmental standards for nitrogen and phosphorus in lakes and marshes, and the work of setting effluent standards for nitrogen and phosphorus is now being handled by the Central Council for the Control of Environmental Pollution.

Eutrophication control has thus become nationwide and technological steps to remove nitrogen and phosphorus are now required for sewage systems, too.

The research for advanced wastewater treatment in Japan commenced around 1971 when the Public Works Research Institute of the Ministry of Construction established a 250 m^3/day pilot plant at the Shitamachi Sewage Treatment Plant in Yokosuka City and commenced research into the removal by chemical coagulation of phosphorus from secondary effluent. Later, surveys and research were also conducted by the Japan Sewage Works Agency, Sapporo City, Tokyo Metropolis, Yokohama City, Yokosuka City, Osaka City, Kobe City and Fukuoka City etc. as developments towards advanced wastewater treatment technology.

Surveys and research concerned mainly with the physicochemical treatment and their results were brought to the Tertiary Treatment Cooperative Conference (later renamed the Advanced Treatment Conference) established by the Ministry of Construction in 1971, and the collection and exchange of information were actively carried out at this conference.

With the continuing oil crisis of the first four years of the 1970s as a juncture, the reduction of necessary materials and energy consumption began to be required of sewage treatment. Under these circumstances, the emphasis in advanced wastewater treatment technology development shifted from the physicochemical process to the biological process for nitrogen and phosphorus removal.

Since 1978 the Public Works Research Institute has been studying the removal of nitrogen by single-sludge nitrification/denitrification process and also the simultaneous removal of nitrogen and phosphorus by the addition of alum at the pilot plant established at the Toba Sewage Treatment Plant in Kyoto City. Furthermore, it

has been conducting demonstrative studies into biological nitrification and denitrification at a 1,500 m^3/day sewage treatment plant since 1981, in cooperation with Hamamatsu city.

In 1980, the Japan Sewage Works Agency began demonstrative studies into nitrogen removal by the single-sludge nitrification/denitrification process at the Arakawa Wastewater Treatment Plant in Saitama Prefecture and research concerning the removal of nitrogen and phosphorus by the anaerobic and aerobic systems is continuing.

Local public entities are also deeply interested in the removal of nutrient salts by biological systems, and related surveys and reserach are being conducted by Ibaraki Prefecture, Nagano Prefecture, Tokyo Metropolis, Kawasaki City, Yokosuka City and Fukuoka City, etc. Research and development relative to the biological removal of nitrogen and phosphorus is also being actively conducted by private research organizations.

In the 20th annual meeting of the Japan Sewage Works Association held in Nagoya City in May 1983[1], as many as 30 research reports were made concerning the biological removal of nitrogen and/or phosphorus and there was heated debate on the subject. A wide range of studies was reported on, including not only the removal of nitrogen and/or phosphorus by anaerobic and aerobic activated sludge processes but also the combined use of these with the physicochemical process and the application of the anaerobic and aerobic methods to the oxidation ditch and rotating biological contactor processes.

As seen above, the development of advanced treatment technology concerning nutrient salts is being advanced with the view to expand the function of the conventional biological process and to improve the secondary treatment facilities for the future setting of effluent standards for lakes and marshes.

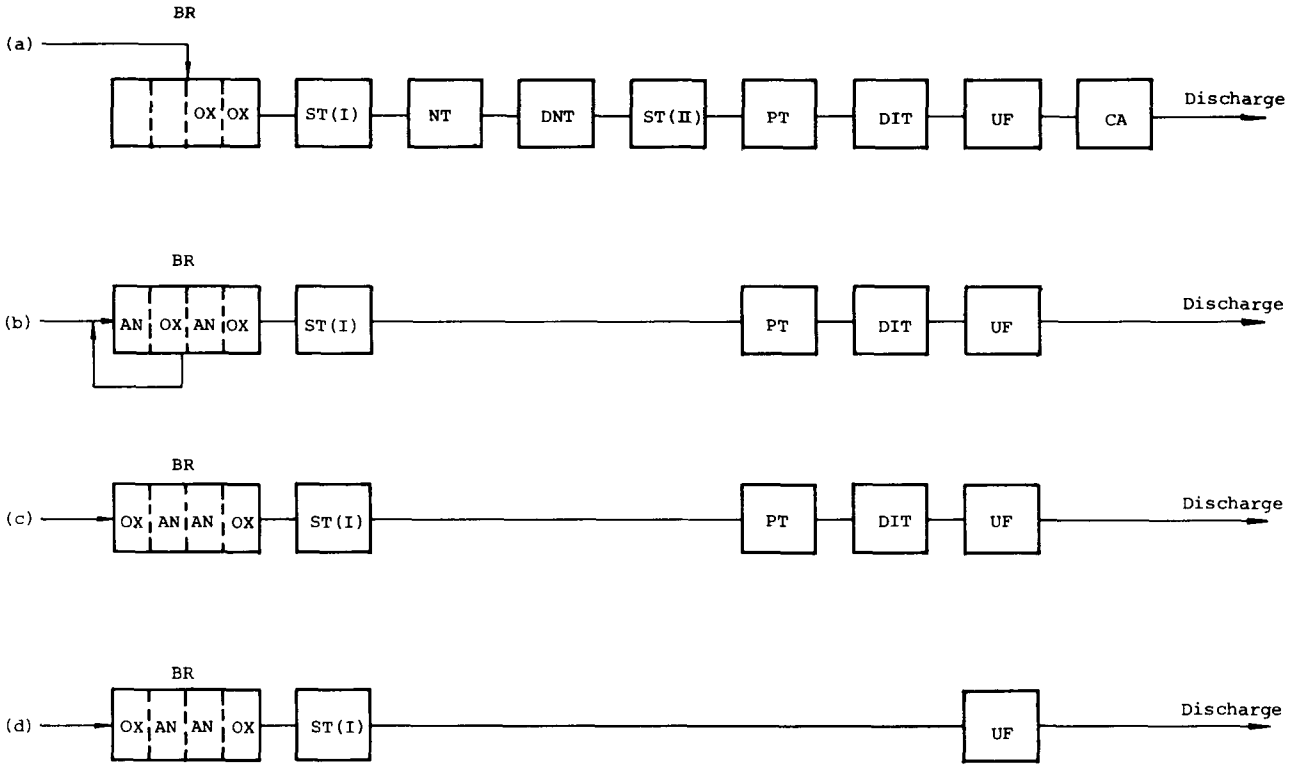
This article deals with the surveys and research conducted by the Public Works Research Institute and the Japan Sewage Works Agency concerning the biological removal of nitrogen and phosphorus.

2. OUTLINE OF SURVEY BY PUBLIC WORKS RESEARCH INSTITUTE

2.1 Improvement of Nitrification and Denitrification Process[2]

In cooperation with Hamamatsu City, the Public Works Research Institute has been improving existing biological nitrification/denitrification facilities in order to bring about system simplification and treatment cost reduction.

The Hitomigaoka Purification Center in Hamamatsu City is a small sewage treatment plant with a designed daily average flow of about 1,500 m^3/day. Since its discharge goes into Shonai Bay, the targeted quality of effluent discharge is extremely severe, as indicated in Table 2. Originally, this sewage treatment plant was designed as an extended aeration process but advanced treatment facilities comprising a nitrification tank, a denitrification tank, coagulation and flocculation tanks, a rapid sand filter and an activated carbon column were put into operation in June 1981 as an addition to the existing secondary treatment facility in order to meet these stringent targets. The flow diagram of this process is given in Fig. 1 (a). In this flow, nitrogen is removed by nitrification/denitrification (addition of methanol) after secondary treatment; there is no clarifier between the nitrification tank and the denitrification tank and a two-sludge system is at work there. The final effluent fully satisfied the target values shown in Table 2 thanks to the addition of the advanced treatment facilities. However, there were such problems as a complicated process configuration and the high operational cost of power and chemicals. Meanwhile, the secondary treatment facility is, as already state, designed as an extended aeration process, and, moreover, the volume of sewage that actually flowed in represented only 50∿60% of the designed value and thus the secondary treatment facility could be diverted to other uses. So, the operation of the two-sludge nitrification/denitrification system was stopped and operation was shifted to a single-sludge recycling nitrification/denitrification system



BR : Biological Reaction Tank, AN: Anoxic, OX: Oxic, DNT: Denitrification Tank
DIT: Chlorination Tank, ST(I): Sedimentation Tank (I), ST(II): Sedimentation Tank (II)
UF : Upflow Filter, NT: Nitrification Tank, PT: Coagulation/Filocculation Tank, CA: Carbon Column

Fig. 1 Flow Diagram of Hitomigaoka Purification Center

(recycling system) and single-sludge non-recycling nitrification/denitrification system (non-recycling system), using the existing aeration tank. It was found from the operation results in flow (a) that an activated carbon column was not especially necessary to achieve the targeted effluent quality.

Operation was thus carried out according to the flows shown in (b), (c) and (d) of Fig. 1. In Flow (b), the first and the third of the four cells into which the aeration tank was divided were anoxic cells while the second and the fourth were aerobic cells. Nitrified liquid was recycled from the second cell to the first cell.

In Flow (c), the first and fourth cells were aerobic cells while the second and third cells were anoxic cells and denitrification was effected by endogenous respiration.

In Flow (d), the coagulation/flocculation tanks and the chlorination tank were eliminated from Flow (c).

The survey took place from September 1981 to June 1982. Table 3 shows the conditions of operation and Table 4 shows the summary of operational data.

Table 2 Target Effluent Quality for Hitomigaoka Purification Center

BOD	5 mg/ℓ
COD	10 mg/ℓ
SS	3 mg/ℓ
T-N	5 mg·N/ℓ
T-P	1 mg·P/ℓ

Table 3 Operational Conditions

Flow				(a)	(b)-I	(b)-II	(b)-III	(c)	(d)
Operating period			From	Sep 5, 1981	Oct 5, 1981	Feb 3, 1982	Feb 23, 1982	Apr 10, 1982	May 11, 1982
			To	Oct 4, 1981	Feb. 2, 1982	Feb. 22, 1982	Apr. 9, 1982	May 10, 1982	Jun. 16, 1982
Volume of influent ($Q \cdot m^3$/day)				907	735	644	707	808	829
Biological reaction tank	Return sludge ratio (r.%)			66	100	110	85	74	72
	Recycling ratio (R.%)			0	200	220	200	0	0
	Detention time (hr)	Q base	Per cell	8.6	10.6	12.1	11.1	9.7	9.4
			Total	17.2*	42.6	48.6	44.2	38.7	37.7
		(Q+r+R) base	Cell 1, 2	0*	2.7	2.8	2.9	5.6	5.5
			Cell 3, 4	5.2	5.3	5.8	6.0	5.6	5.5
			Total	10.4*	16.0	17.2	17.8	22.4	22.0
	Ratio of Air Flow to Influent Flow			16.7	11.1	10.3	9.4	10.8	11.8
	MLSS (mg/ℓ)			2,800	3,480	1,740	3,500	3,390	3,440
	MLVSS (mg/ℓ)			1,670	2,410	1,310	2,620	2,550	2,510
	MLVSS/MLSS			0.60	0.69	0.75	0.75	0.75	0.73
	SV (%)			21	56	23	83	90	93
	SVI			75	162	131	228	265	270
	SRT (day)			14.7	58.8	57.2	78.2	41.3	31.0
	SAT (day)			14.7	29.4	28.6	39.1	20.7	15.5
	Return sludge concentration (mg/ℓ)			8,880	6,550	3,350	6,320	7,830	9,840
	Volume of excess sludge (m^3/day)			10.9	9.2	7.6	7.1	11.4	13.6
Amount of Al addition (mg·Al/ℓ)				2.7	3.0	3.4	3.1	2.9	0
Amount of chlorine addition (mg·Cl/ℓ)				2.5	1.5	1.7	1.6	2.1	0
Amount of methanol addition (mg/ℓ)				36	0	0	0	0	0

* Cells 1 and 2 were not used.

Table 4 Summary of Operational Data

Flow		Treatment stage	Water temperature	pH	SS	BOD	DBOD	COD	DCOD	Alkalinity	T-N	NH_4-N	NO_X-N	T-P	PO_4-P
(a)		Influent	25.2	7.1	105.0	84.3		40.8			16.7	12.3	0.7	4 6	2.1
		Secondary effluent	25.5	6.7	30.0	13.2	5.1	15.7	4.8	51	13 9	2.8	9 4	2.5	1.4
		Denitrified effluent	26.0	7.0	11.0	3.4	1.3	8 4	4.5		2.9	ND	2.0	3 1	1 0
		Effluent of coagulation/ flocculation tank	25.7	6.9	19.0	2.2	1.3	5.9	3.7		2.8	ND	1.9	1.5	0.3
		Filtrate	25.7	7.2	3.0	1.1	1.1	3.8	3.5		2.2	ND	1.3	0.6	0.3
		Effluent of carbon column	25.3	7.4	2.0	0.6	0.4	1.8	1.7		2.9	ND	2.3	0 3	0.3
(b)	I	Influent	16.9	7.3	143.0	150.0	55.0	62.2	31.0	143	29.5	20.0	0.5	4.6	1.9
		Biologically treated effluent	16.8	6.9	23.0	13.0	1.9	11.0	5.3	52	8.3	0.7	5.4	2.5	1.5
		Effluent of coagulation/ flocculation tank	16.9	6.7	10.5	4.0	1.4	6.2	4.0		7.9	0.6	5.3	0.9	0.3
		Filtrate	16.9	7.0	5.0	1.8	0.7	4.5	3.9		8.2	0.7	6.2	0.6	0.2
	II	Influent	10.5	7.6	84.0	150.0	34.0	68.0	44.0	190	41.0	25.0	1.8	8.7	3.0
		Biologically treated effluent	11.0	6.5	22.0	4.6	1.5	13.0	6.8	43	16.0	0.6	13.0	3.4	2.0
		Effluent of coagulation/ flocculation tank	11.0	6.1	4.5	2.1	1.2	6.6	4.6		15.0	0.6	13.0	0.3	0.1
		Filtrate	11.0	6.5	1.5	1.0	0.4	4.9	4.2		14.0	0.6	13.0	0.3	0.2
	III	Influent	13.5	7.3	130.0	200.0	89.0	67.0	40.0	140	32.0	23.0	0.5	4.1	2.1
		Biologically treated effluent	14.0	6.6	19.0	15.0	2.3	11.0	6.2	57	5.4	0.6	2.8	3.0	1.7
		Effluent of coagulation/ flocculation tank	14.0	6.6	5.4	4.2	0.8	5.6	4 8		4.6	0.7	3.3	0.7	0.1
		Filtrate	13.5	6.9	1.4	1.5	0.4	4.4	3.8		4.5	0.6	3.4	0.3	0.2
(c)		Influent	17.7	7.1	200.0	250.0	51.0	62 0	30.0	130	30.0	21.0	0.2	3.7	2.3
		Biologically treated effluent	18.2	6.8	22.0	11.0	3.7	7.8	6.4	52	3.5	0.1	2.5	1.8	1.3
		Effluent of coagulation/ flocculation tank	18.0	6.8	9.0	1.6		5.4		44	3.1	0.2	2.2	0.8	0.4
		Filtrate	18.2	7.0	4.0	1.8		4.1		43	2.9	0.2	1.8	0.5	0.4
(d)		Influent	22.0	7.2	109.0	180.0	100.0	51.0	31.0	140	33.0	20.0	0.2	5.2	3.2
		Biologically treated effluent	21 9	6.7	13.0	6.2	1.7	6.8	3.2	52	4.8	0.1	2.9	1.7	1.3
		Filtrated	22.1	7.0	1 0	1.9	1.5	4.1	3 7	52	4 0	0.3	2.5	1.2	1.2

The following conclusions were derived from these survey results:

(1) Biological nitrification/denitrification process was improved from a two-sludge system (Flow (a)) to a single-sludge recycling system (Flow (b)) to a single-sludge non-recycling system (Flows (c) and (d)) and the T-N target quality could be achieved by any of these systems. It is therefore desirable that the single-sludge non-recycling system which costs the least to operate, be adopted.

(2) Biological dephosphorization occurred in the biological reaction tank by the single-sludge non-recycling system and T-P in filtrate of biologically treated effluent decreased to a level only slightly above the target value of final effluent quality. It is therefore considered that the T-P target quality can even be achieved by omitting the coagulation/flocculation tanks, as in Flow (d), by the addition of a small quantity of alum to the end of the biological reaction tank or to the sand filter.

(3) If chlorine for disinfection is injected into the sand filter influent, SS leaks into the filtrate. To achieve the target quality value of SS = 3 mg/ℓ, it is advisable to omit chlorine disinfection. Even in this case, the number of coliform groups in the filtrate decreases to less than 3,000 per milliliter.

(4) The target quality values of BOD = 5 mg/ℓ and COD = 10 mg/ℓ could be achieved by any of the treatment flows.

(5) From the above, the present optimum treatment flow at the Hitomigaoka Purification Center is the formula adding a small quantity of alum to Flow (d). The treatment cost (power cost + chemical cost) by this flow is 49% and 75%, respectively, compared with Flow (a) and Flow (b).

2.2 Survey Concerning the Simultaneous Removal of Nitrogen and Phosphorus through Addition of Coagulant to Single-Sludge Recycling System[3]

A survey designed to simultaneously remove nitrogen and phosphorus by adding a coagulant to the single-sludge recycling system is being conducted at the Toba Sewage Treatment Plant in Kyoto City, using a 300 m^3/day pilot plant. The flow diagram of the pilot plant is shown in Fig. 2. This survey concerns the phosphorus removal effect by the addition of alum to the recycling system, its impact on the removal of nitrogen and the properties of sludge. Table 5 and Table 6 shows the conditions of operation and the summary of operational data. The following has so far been revealed by this survey:

(1) SS in effluent is high in inverse proportion to the concentration of the added alum and the T-P and COD also increase accordingly.

(2) SS in effluent from this system is more filterable than that from conventional activated sludge process.

(3) With the addition of alum, an alkalinity equivalent to 5.5 times the concentration of aluminum is required.

(4) If the concentration of aluminum added is less than 4 mg/ℓ, it does not affect the nitrification and denitrification reactions.

(5) If the concentration of aluminum added is within the range of 2 ∿ 6 mg/ℓ, the biota do not seem to be adversely affected.

(6) With the addition of alum, there is a sludge corresponding to about 5.6 times the concentration of the aluminum added.

(7) The concentration of gravitationally thickened sludge is 2 ∿ 3%.

(8) Aluminum does not appear to inhibit anaerobic digestion.

(9) Sludge from this system is somewhat inferior in dehydration capacity to that from the conventional activated sludge process.

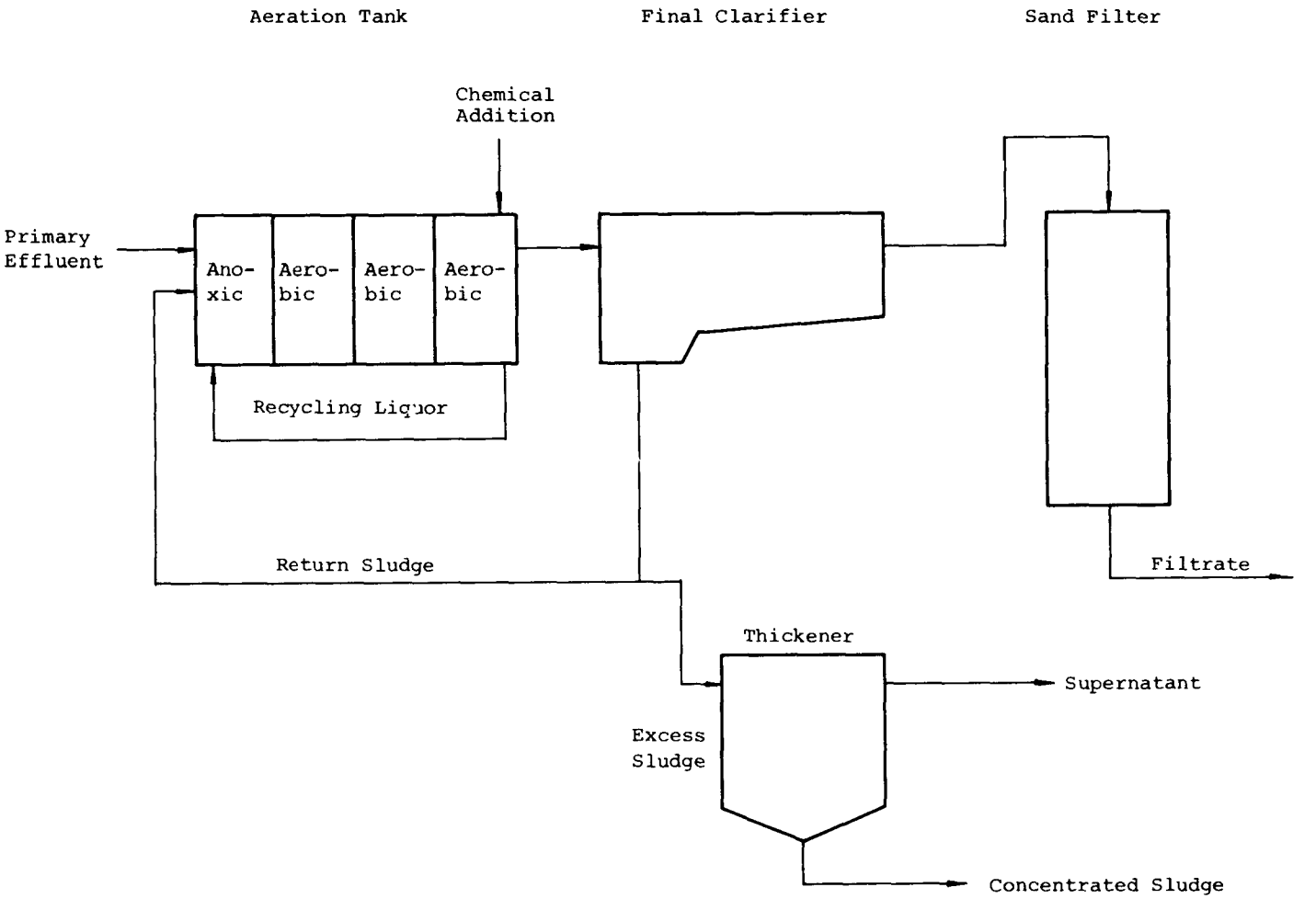


Fig. 2 Flow Diagram of Kyoto Pilot Plant

Table 5 Operational Conditions of Pilot Plant

Case	I	II	III	IV	V
Aeration Tank					
Influent Flow (m^3/d)	300(S)	300(S)	300(S)	300(S)	300(V)
Detention Time (hr)	8	8	8	8	8
Anoxic	2	2	2	2	2
Aerobic	6	6	6	6	6
Sludge Return Ratio (%)	50	50	50	50	50
Recycling Ratio (%)	100	100	100	100	100
Ratio of Air Flow to Influent Flow	8.6	6.2	6.8	8.4	9.6
Excess Sludge Ratio (%)	0.50	0.40	0.45	0.86	1.01
MLSS (mg/ℓ)	2218	1960	2914	3354	2434
BOD-SS Loading (kg/kg·day)	0.073	0.084	0.064	0.071	0.109
SRT (day)	10.3	9.1	13.2	11.7	8.8
Temperature (°C)	25.1	25.9	25.7	21.0	17.3
Alum Dosage (mgAl/ℓ)	0	2	4	6	4
Final Clarifier					
Detention Time (hr)	4	4	4	4	4
Overflow Rate ($m^3/m^2/day$)	15	15	15	15	15
Sand Filter					
Filteration Rate (m/day)	180	180	180	180	140

(S): Steady Flow

(V): Varied Flow

Table 6 Summary of Operational Data

Case / Items	I		II		III		IV		V	
	Inf.	Eff.	Inf.	Eff.	Inf.	Eff.	Inf.	Eff.	Inf.	Eff.
BOD	52.5	9.3	54.6	10.4	57.5	8.2	81.0	7.5	87.7	13.7
ATU-BOD		5.5		6.3		4.6		4.7		5.1
COD_{Mn}	52.3	19.4	41.2	22.1	45.2	17.6	54.9	12.2	62.9	17.0
COD_{Mn}-D	29.7	9.4	23.3	5.8	27.6	6.9	29.9	8.3	39.3	9.6
SS	60.1	39.6	57.4	55.5	55.8	33.3	55.7	15.0	62.4	19.2
K-N	17.34	3.05	14.9	2.82	17.68	2.47	24.20	2.30	24.81	3.46
K-N-D		1.09		0.62		1.09		1.47		2.10
NH_4-N	12.7	0.0	11.2	0.2	14.5	0.6	13.7	1.7	16.8	1.0
NO_T-N		5.6		6.4		4.4		5.1		5.2
T-P	2.57	2.49	2.07	1.43	2.67	1.45	3.11	0.52	3.00	0.67
T-P-D		1.95		0.20		0.16		0.03		0.16
Alkalinity	118.2	47.7	104.2	28.8	122.8	42.2	133.3	24.4	132.4	34.3

Inf: Primary Effluent, Eff.: Bio-Treated Effluent

3. SURVEY BY JAPAN SEWAGE WORKS AGENCY

The Japan Sewage Works Agency, jointly with Shiga Prefecture and Saitama Prefecture, conducted a demonstrative study concerning the removal of nitrogen by the single-sludge recycling system in 1980 ∿ 1982[4], using one train of the Arakawa Wastewater Treatment Plant, and is still studying the single-sludge recycling system for simultaneous removal of nitrogen and phosphorus.

3.1 Experimental Facility and Operating Method

One train of the Arakawa Treatment Plant (conventional activated sludge process) was used for this survey. This train can be operated completely independently of the others including the return sludge system. The dimensions of the train used for the study are shown in Table 7. This facility was remodeled according to the flow indicated in Fig. 3. Main items for remodeling are as follows:

(1) The air diffuser was dismantled from the first cell of the aeration tank and two agitators (11 KW each) were installed instead.

(2) The air diffuser was dismantled from the second cell of the aeration tank and two aeration agitators (7.5 KW each) were installed instead. These agitate, but can also be used as an aerating unit by connecting a blastpipe.

(3) An submersible pump was installed at the fourth cell of the aeration tank so that nitrified liquor could be recycled to the second cell.

Table 7 Dimensions of Demonstration Train

	Width (m)	Length (m)	Effective water depth (m)	Number of Tanks	Capacity (m^3)	Remarks
Primary Clarifier	4.3	50.0	4.0	2	1,720	
Aeration Tank	9.0	85.0	5.0	1	3,825	Divided into four equal cells
Final Clarifier	4.3	56.0	4.45	2	2,143	

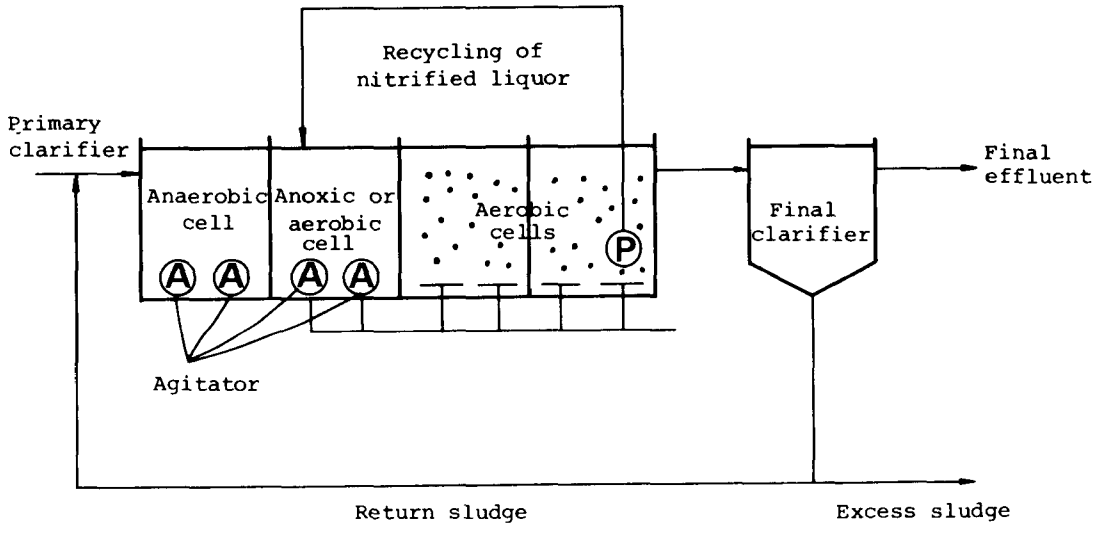
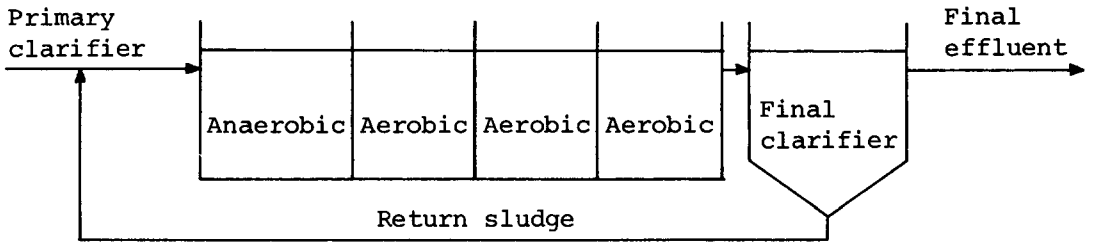


Fig. 3 Flow Diagram of Experimental Train

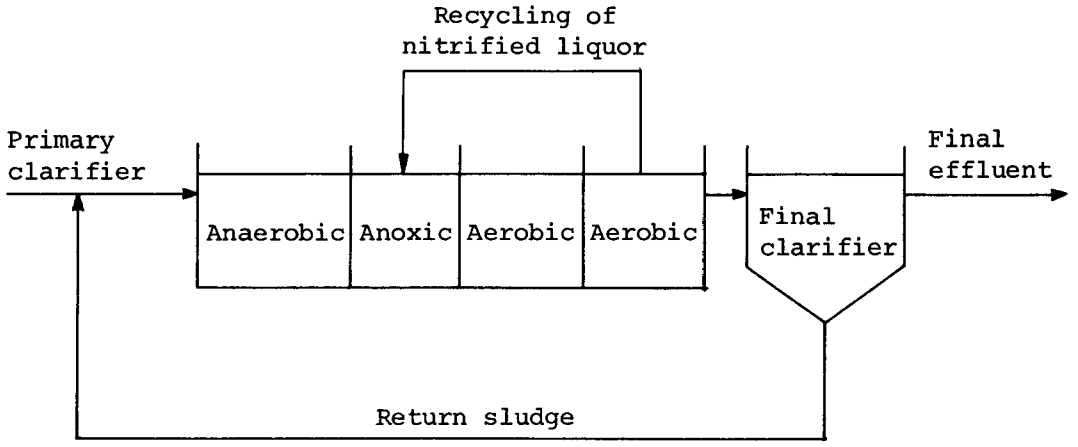
The demonstration train was operated under two different conditions: Operation Modes I and II. The flow of each mode is shown in Fig. 4. In Mode I, operation designed solely for the biological removal of phosphorus was carried out, using the first cell as an anaerobic tank, the second and subsequent cells as aerobic tanks, while in Mode II, the first cell was used as an anaerobic tank, the second cell as an anoxic tank (denitrification cell), and the third and subsequent cells were operated as aerobic tanks, with nitrified liquor recycled from the bottom end of the fourth cell to the second cell, and with the biological removal of nitrogen being carried out in addition to the removal of phosphorus.

The major operating conditions in Modes I and II are shown in Table 8.

The influent was constantly supplied and the return sludge ratio was also constant.



(a) Operation Mode I



(b) Operation Mode II

Fig. 4 Operation Mode

Table 8 Operation Conditions

	Operation Mode I				Operation Mode II		
	Run-1	Run-2	Run-3	Run-7	Run-4	Run-5	Run-6
Volume of influent (m^3/day)	14,340	16,580	9,620	10,850	7,110	8,590	5,980
Volume of return sludge (m^3/day)	3,480	4,230	2,450	2,740	2,430	3,210	2,810
Volume of recycled liquor (m^3/day)	-	-	-	-	5,700	5,700	5,700
Aeration tank detention time (hr)	6.4	5.5	9.5	8.5	12.9	10.7	15.4
MLSS (mg/ℓ)	2,040	2,330	1,940	2,620	1,950	3,740	4,050
MLRSS (mg/ℓ)	14,090	16,370	14,800	19,330	12,720	19,680	20,480
SRT (day)	12.1	11.7	12.3	13.7	33.1	63.9	30.8
SAT (day)	9.1	8.8	9.2	10.3	16.5	31.9	15.4
BOD-SS load (kg/kg·day)	0.16	0.20	0.10	0.13	0.06	0.04	0.04
Period	Jun.7 ∿ Jun.28,'82	Jun.29 ∿ Jul.14,'82	Jul.15 ∿ Jul.30,'82	Jan.24 ∿ Feb.9,'83	Aug.27 ∿ Oct.25,'82	Nov.8 ∿ Nov.24,'82	Nov.25 ∿ Jan.19,'83

(Notes) 1. Each figure is a mean.
2. The aeration tank detention time is based on the volume of influent.
3. SAT is SRT for sludge in the nitrification cells.

3.2 Operational Results

Summary of operational data for each run (1, 2, 3, 7) in Operation Mode I is shown in Table 9 and for each run (4, 5, 6) in Operation Mode II is shown in Table 10. Of these runs, Run 4 saw heavy typhoon rain in early August and mid-September and the concentration of influent was lower than ordinary due to the influx of stormwater and this condition continued for a considerable period after the rain.

Table 9 Summary of Operation (Mode I)

	Run-1		Run-2		Run-3		Run-7	
Water temperature (°C)	21.6 (20.5 ∿ 22.1)		22.1 (21.2 ∿ 23.3)		22.8 (21.8 ∿ 23.2)		14.4 (14.0 ∿ 14.9)	
PH	7.1 (7.0 ∿ 7.3)	7.0 (6.5 ∿ 7.5)	7.2 (7.1 ∿ 7.3)	7.3 (7.2 ∿ 7.6)	7.2 (7.1 ∿ 7.3)	7.1 (6.8 ∿ 7.4)	7.3 (7.1 ∿ 7.4)	7.2 (7.2 ∿ 7.3)
T-P	3.43 (1.75 ∿ 4.8)	0.44 (0.14 ∿ 0.74)	4.78 (2.28 ∿ 10.26)	0.41 (0.14 ∿ 0.85)	3.30 (1.58 ∿ 4.13)	0.80 (0.17 ∿ 1.24)	4.16 (3.32 ∿ 5.07)	0.80 (0.61 ∿ 1.18)
Soluble T-P	1.84 (1.00 ∿ 2.83)	0.36 (0.08 ∿ 0.84)	2.33 (1.18 ∿ 4.35)	0.33 (0.08 ∿ 0.80)	2.04 (1.00 ∿ 2.73)	0.72 (0.11 ∿ 1.12)	2.42 (2.05 ∿ 2.83)	0.39 (0.39 ∿ 1.02)
Soluble O-P	1.57 (0.83 ∿ 2.58)	0.30 (0.05 ∿ 0.70)	2.05 (0.72 ∿ 4.25)	0.23 (0.02 ∿ 0.76)	1.71 (0.74 ∿ 2.57)	0.63 (0.05 ∿ 1.06)	2.13 (1.78 ∿ 2.43)	0.46 (0.27 ∿ 1.0)
T-N	21.6 (14.5 ∿ 26.8)	13.2 (9.9 ∿16.1)	29.1 (17.9 ∿ 43.4)	14.6 (12.8 ∿ 16.6)	19.5 (17.9 ∿ 20.7)	11.3 (10.2 ∿ 12.2)	31.8 (30.5 ∿ 34.6)	20.7 (19.1 ∿ 22.6)
Soluble T-N	-	-	-	-	-	-	24.8 (23.2 ∿ 26.5)	19.6 (18.3 ∿ 21.0)
NH_4^+-N	15.0 (10.0 ∿ 17.2)	5.5 (3.7 ∿ 8.0)	13.9 (10.7 ∿ 16.7)	10.9 (5.4 ∿ 13.0)	9.6 (4.1 ∿ 12.8)	1.4 (1.0 ∿ 2.2)	20.8 (19.8 ∿ 22.3)	17.4 (14.2 ∿ 20.3)
BOD	87.2 (52.9 ∿112.7)	6.0 (4.1 ∿ 7.5)	106.4 (72.2 ∿161.9)	5.3 (4.3 ∿ 7.3)	73.9 (67.0 ∿ 79.1)	4.2 (2.1 ∿ 5.7)	115.9 (100.5 ∿131.4)	6.7 (5.8 ∿ 7.5)
COD	37.9 (22.8 ∿ 54.3)	8.3 (5.8 ∿11.5)	52.8 (33.1 ∿113.6)	8.8 (5.2 ∿ 10.6)	40.7 (25.6 ∿ 53.5)	8.0 (6.9 ∿ 9.2)	56.6 (54.2 ∿ 59.5)	14.2 (13.2 ∿ 15.2)
SS	71.4 (40 ∿ 120)	4.5 (1.8 ∿11.4)	109.3 (30 ∿ 260)	3.1 (1.5 ∿ 4.6)	62 (24 ∿ 130)	2.0 (0.7 ∿ 3.7)	83.0 (48 ∿ 114)	9.2 (4.6 ∿ 13.8)
M-alkalinity	127.2 (101.9 ∿144.0)	72.2 (58.4 ∿89.7)	138.7 (115.2 ∿154.2)	124.7 (102.5 ∿147.5)	131.0 (91.0 ∿146.0)	65.9 (48.7 ∿ 95.3)	150.5 (145.5 ∿156.1)	145.3 (133.5 ∿152.5)
P/MLSS (%)	4.09 (3.69 ∿ 4.49)		3.91 (3.60 ∿ 4.25)		3.98 (2.75 ∿ 5.08)		3.82 (3.65 ∿ 4.10)	

(mg/ℓ)

(Notes) 1. 24-hour composite samples were used for analysis.
2. Left: Primary effluent. Right: Final effluent. Top: Means value. Bottom: Range.
3. P/MLSS is by a sample at the bottom end of the aerobic cell.

Table 10 Summary of Operation (Mode II)

	Run-4		Run-5		Run-6	
Water temperature (°C)	22.4 (18.6 ∿ 25.1)		19.2 (19.0 ∿ 19.5)		16.2 (14.3 ∿ 17.8)	
PH	7.1 (6.8 ∿ 7.3)	7.1 (7.0 ∿ 7.5)	7.3 (7.2 ∿ 7.5)	7.3 (7.1 ∿ 7.5)	7.2 (6.9 ∿ 7.4)	7.1 (7.0 ∿ 7.4)
T-P	2.16 (1.11 ∿ 4.93)	0.52 (0.09 ∿ 1.13)	2.57 (1.85 ∿ 3.67)	0.19 (0.14 ∿ 0.23)	4.35 (2.50 ∿ 6.71)	0.28 (0.21 ∿ 0.53)
Soluble T-P	0.99 (0.44 ∿ 1.79)	0.44 (0.05 ∿ 1.02)	1.12 (0.63 ∿ 1.71)	0.11 (0.09 ∿ 0.14)	2.41 (1.21 ∿ 3.84)	0.18 (0.13 ∿ 0.34)
Soluble O-P	0.79 (0.33 ∿ 1.59)	0.37 (0.01 ∿ 0.95)	0.94 (0.45 ∿ 1.41)	0.05 (0.03 ∿ 0.07)	2.13 (0.82 ∿ 3.66)	0.11 (0.05 ∿ 0.24)
T-N	16.5 (8.0 ∿ 22.9)	8.3 (4.8 ∿ 10.7)	22.1 (16.1 ∿ 29.7)	10.6 (7.7 ∿ 13.6)	28.1 (17.2 ∿ 34.8)	11.8 (7.8 ∿ 17.1)
Soluble T-N	12.4 (6.3 ∿ 16.7)	8.0 (4.7 ∿ 10.4)	17.6 (12.7 ∿ 23.2)	10.3 (7.5 ∿ 13.5)	21.1 (12.1 ∿ 24.7)	11.4 (7.4 ∿ 15.2)
NH_4^+-N	9.9 (3.0 ∿ 14.6)	0.2 (0.0 ∿ 2.0)	14.3 (9.6 ∿ 19.6)	5.6 (0.0 ∿ 10.1)	17.7 (9.5 ∿ 20.9)	5.0 (0.0 ∿ 8.2)
BOD	60.1 (27.6 ∿112.4)	2.7 (1.4 ∿ 10.3)	70.7 (53.0 ∿ 95.4)	4.5 (1.1 ∿ 6.6)	99.5 (45.4 ∿130.0)	4.0 (1.6 ∿ 6.4)
COD	30.2 (18.8 ∿ 46.2)	7.0 (5.4 ∿ 12.1)	35.8 (29.1 ∿ 42.9)	8.4 (5.4 ∿ 16.2)	45.3 (36.4 ∿ 58.6)	7.7 (5.9 ∿ 8.7)
SS	55.2 (20 ∿ 144)	3.5 (0.5 ∿ 19.1)	59.7 (36 ∿ 72)	3.0 (0.3 ∿ 4.4)	74.1 (42 ∿ 150)	3.7 (0.1 ∿ 13.0)
M-alkalinity	123.2 (58.1 ∿146.8)	66.4 (57.2 ∿ 75.8)	145.7 (128.7∿157.3)	100.9 (62.0 ∿121.0)	146.6 (104.8∿158.1)	92.7 (71.5 ∿110.0)
P/MLSS (%)	3.52 (2.88 ∿ 4.75)		3.62 (3.42 ∿ 3.77)		4.04 (3.60 ∿ 4.70)	

(mg/ℓ)

(Notes) 1. 24-hour composite samples were used for analysis.
2. Left: Primary effluent. Right: Final effluent. Top: Means value. Bottom: Range.
3. P/MLSS is by a sample at the bottom end of the aerobic cell.

3.2.1 Phosphorus Removal

Figs. 5 and 6 are cumulative frequency curves prepared for the data of the entire period on the respective T-P concentration of the primary and final clarifier effluents. From the figure, the T-P concentration of primary effluent is 3.38 mg/ℓ on the average. The mean value of the final effluent is 0.47 mg/ℓ and the T-P concentration for the accumulative frequency of 90% is 0.94 mg/ℓ.

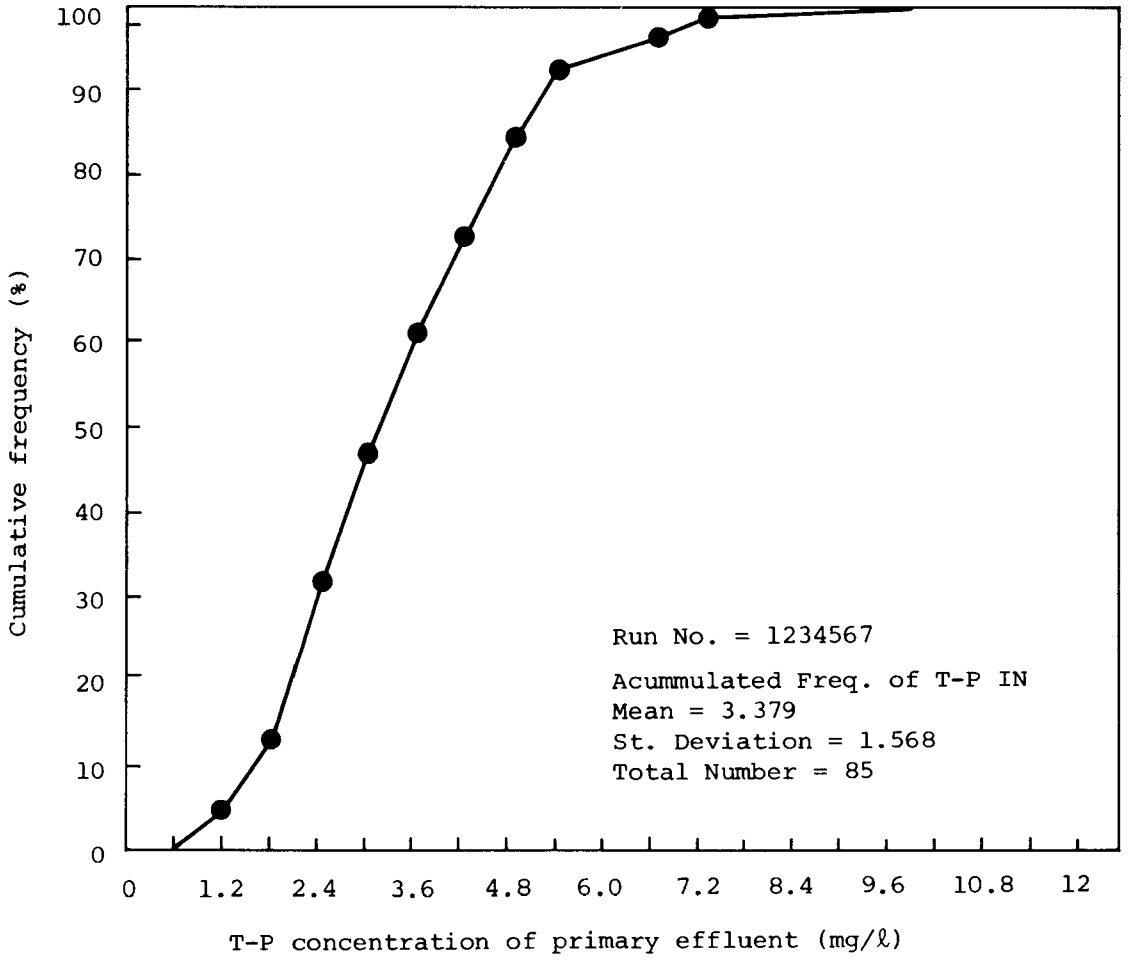


Fig. 5 Cumulative Frequency of Primary Effluent T-P Concentration

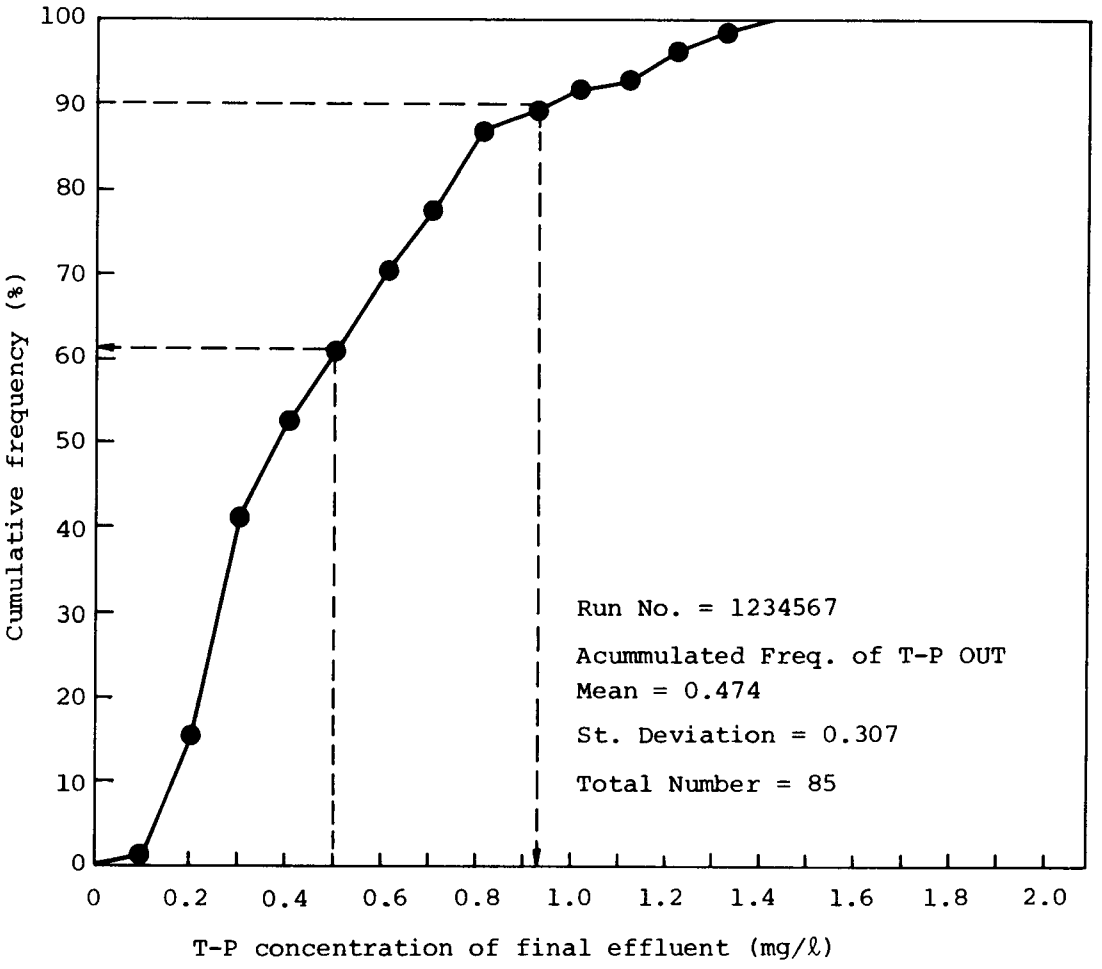


Fig. 6 Cumulative Frequency of Final Effluent T-P Concentration

This indicates that during the period of this survey, the final effluent T-P concentration was less than 1 mg/ℓ in 90% of all the measured data.

The cumulative frequency at the effluent T-P concentration of 0.5 mg/ℓ is 61%, and it is expected that this value will be exceeded in a considerable number of cases if the target value of effluent T-P concentration is set at 0.5 mg/ℓ.

However, it must be noted that the operation was carried out under varied conditions with the object of studying how different operation control factors affect the removal of phosphorus and, after slight remodeling, an existing secondary treatment facility was used in the experiment. Therefore, operation by this system was not necessarily made under the optimum conditions.

(1) Phosphorus Release in Anaerobic Tank and T-P Removal Efficiency

Fig. 7 shows the relation between the soluble ortho-phosphate concentration (to be abbreviated hereafter as S-O-P) in a anaerobic cell and the T-P removal efficiency of the system. On the whole, T-P removal efficiencies are in a higher range if the concentration of phosphate in the anaerobic cell is higher. It can also be seen that when it rains, phosphate concentration in the anaerobic cell is low and the T-P removal efficiency is also often low. The T-P removal efficiency is sometimes more than 90%, even in times of rain, but this is due to the fact that the concentration of the influent itself is low as the result of influx of rainwater.

Fig. 8 shows an example of measurement of S-O-P concentrations at different points of the system in both fine and rainy weather. There was heavy typhoon rain on September 12 and the concentration of influent continued to be very low for the measurement of September 27. As indicated by this figure, no release of phosphate occurred in the anaerobic cell after heavy rainfall, and therefore, no phenomenon of biological luxury uptake was observed either. However, the effluent T-P concentration itself in rainy weather is not greatly different to that for fine weather because the T-P concentration of the influent is low.

The fact that no phosphate release takes place during heavy rain is considered to be attributable not only to the increase of NO_T-N (NO_2^--N + NO_3^--N) and the rise of DO in influent but also to the decrease of BOD in the influent with the influx of rainwater.

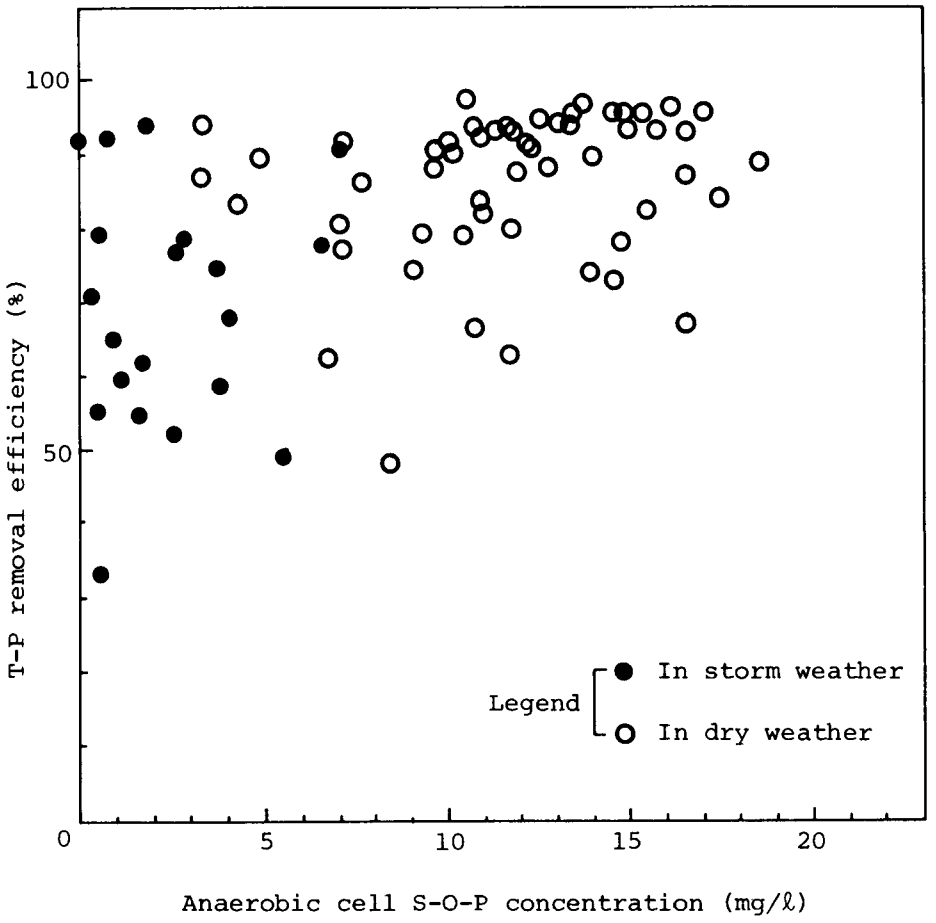


Fig. 7 Relation between Anaerobic Cell S-O-P Concentration and T-P Removal Efficiency

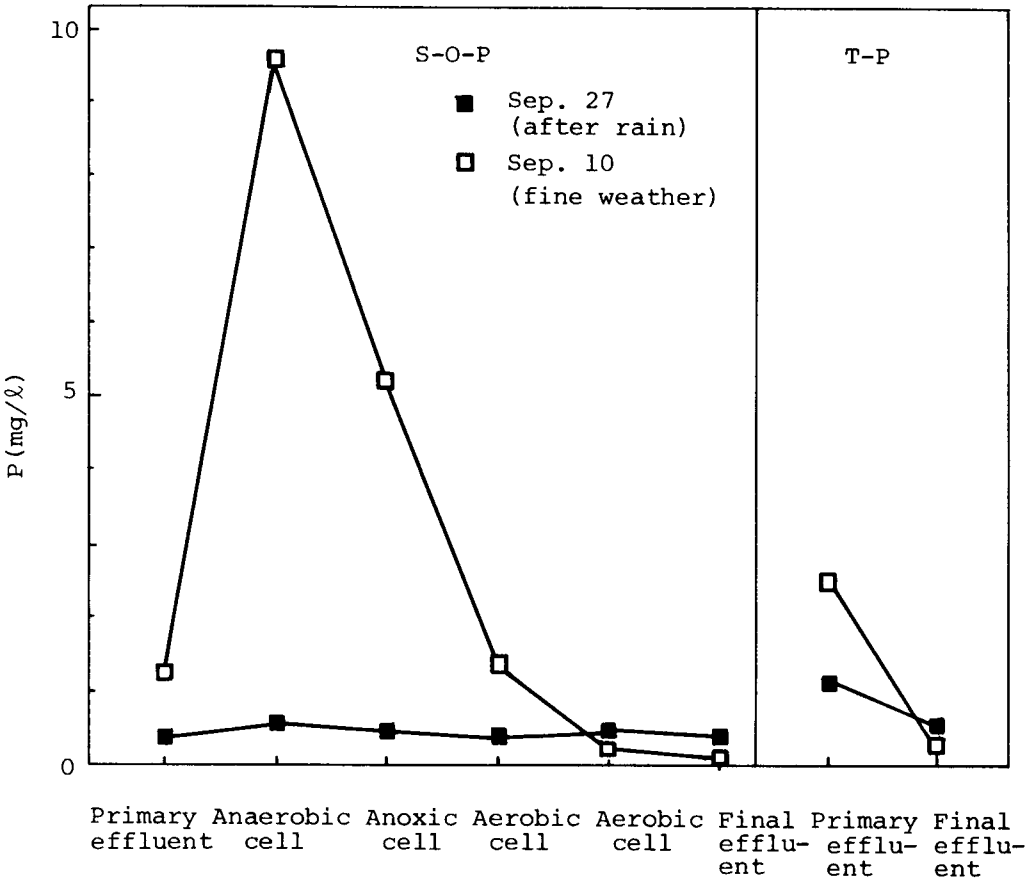


Fig. 8 Example of Measurement of S-O-P Concentrations at Different Points of the System

Fig. 9 shows the relation between the amount of denitrification and the S-O-P concentration in anaerobic cells particularly with respect to Run 4 where the effect of rain was great. It can be seen from this figure that if the quantity of NO_T-N coming into an anaerobic cell increases, so increasing the amount of denitrification in the cell, then phosphate release is suppressed. Fig. 10 shows the relation between ORP and the S-O-P concentration in an anaerobic cell. If the ORP is positive, there is no phosphate release. If the ORP value is negative, then the S-O-P concentration in the tank is large in inverse proportion to the ORP value. It is therefore important to keep the ORP in the anaerobic cell negative and small to ensure the satisfactory removal of phosphate.

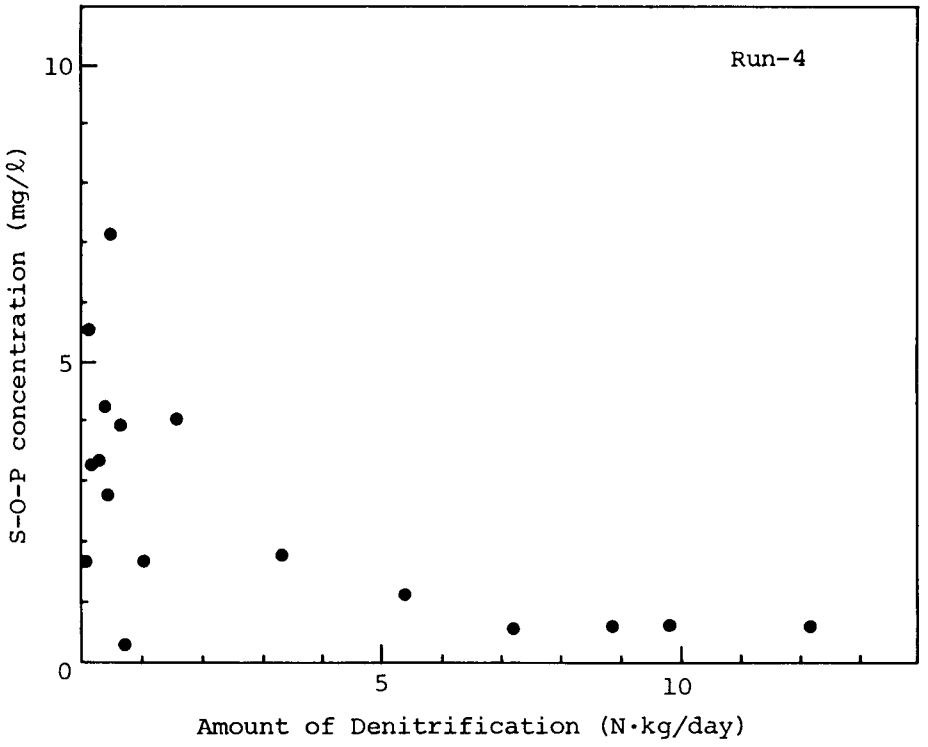


Fig. 9 Relation between Quantity of Denitrification and S-O-P Concentration in Anaerobic Cell

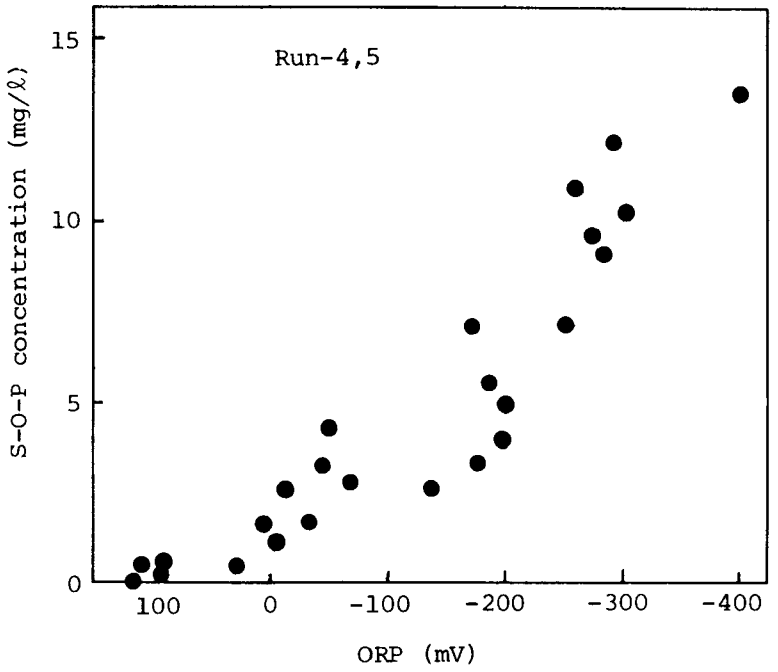


Fig. 10 Relation between ORP and S-O-P Concentration in Anaerobic Cell

(2) BOD Concentration in Influent and the Removal of Phosphorus

Fig. 11 shows the relation between the concentration of BOD at the head of an anaerobic cell and the S-O-P concentration in this cell.

Phosphorus release, which is a prerequisite to the biological removal of phosphorus, is satisfactory in direct proportion to the concentration of BOD at the head of the anaerobic cell.

The tendency for the T-P removal efficiency of the system to increase with the increase of BOD in the influent could be seen but this was not clear. Furthermore, nothing was found concerning the minimum BOD concentration necessary to stabilize the T-P removal efficiency.

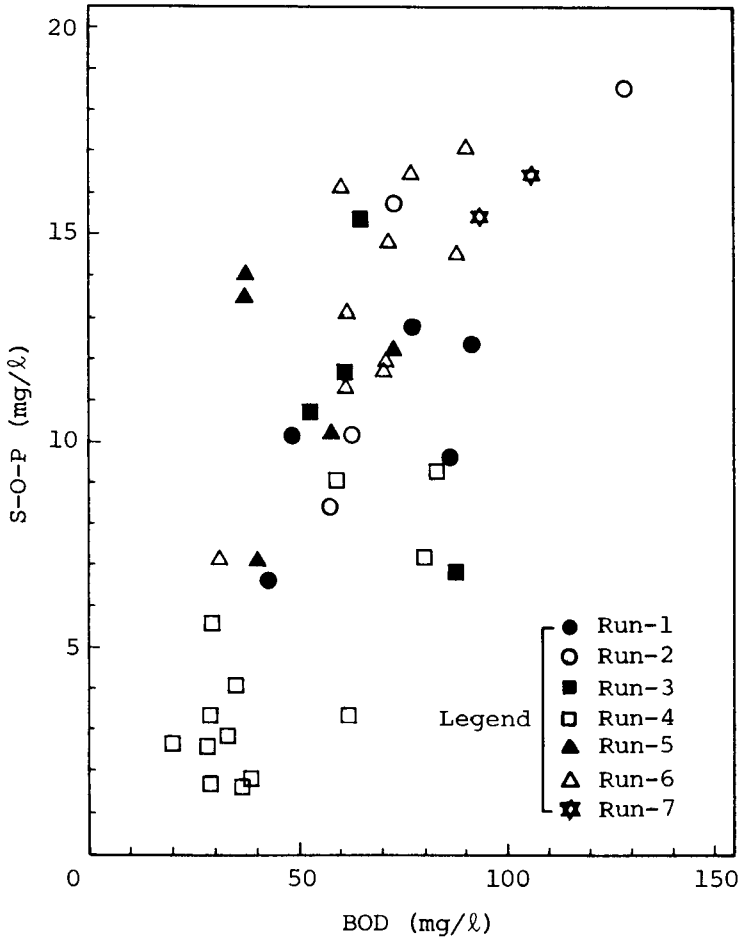


Fig. 11 Relation between Influent BOD and S-O-P in Anaerobic Cell

(3) Phosphorus Release and Uptake Rate

Phosphorus Release Rate

In calculating the phosphorus release rate, it was presumed from the results of the laboratory experiment in 1981[5] that the release rate of ortho-phosphate could be expressed by the following equation:

$$\frac{dc}{dt} = Kr \cdot X \quad \cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots \quad (1)$$

Where,

C : S-O-P concentration (mg/ℓ)

Kr: S-O-P release rate constant (mg/gMLSS·hr)

X : MLSS concentration (g/ℓ)

Fig. 12 shows the relation between Kr and water temperature. In the laboratory experiment[5], using sludge which was not acclimatized to anaerobic/aerobic operation, Kr was dependent on the water temperature but this was not clear in this full-scale survey.

The above-mentioned 1981 laboratory survey also showed that Kr increased in proportion to the BOD concentration. The relation between the BOD concentration at the head of the anaerobic cell and Kr is shown in Fig. 13 for full-scale data.

Fig. 13 indicates that Kr generally tends to increase if BOD is high. Kr can be expressed as the following equation:

$$Kr\ (mg/gMLSS \cdot hr) = 0.0236 \times BOD\ (mg/\ell) - 0.0036 \quad \cdots\cdots \quad (2)$$

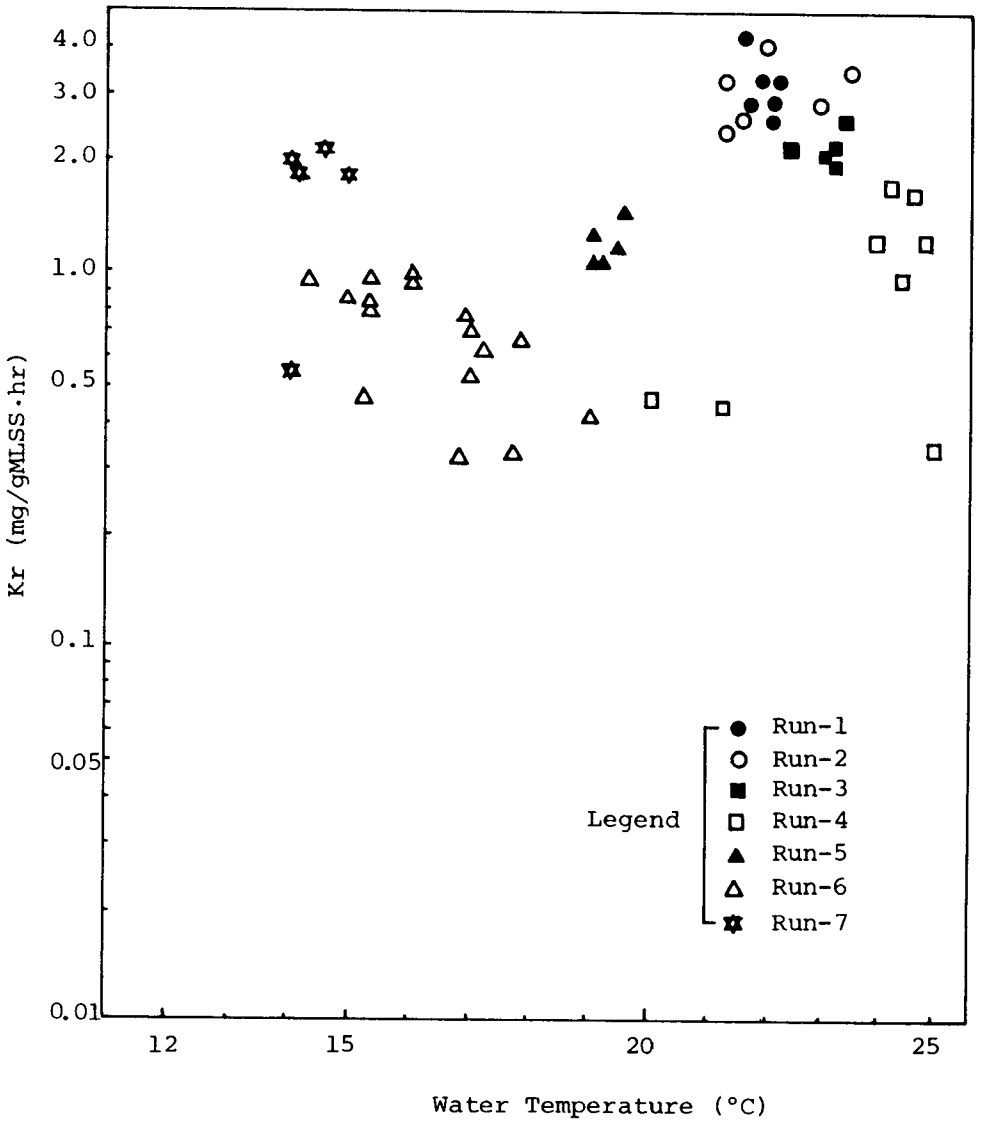


Fig. 12 Relation between Kr and Water Temperature

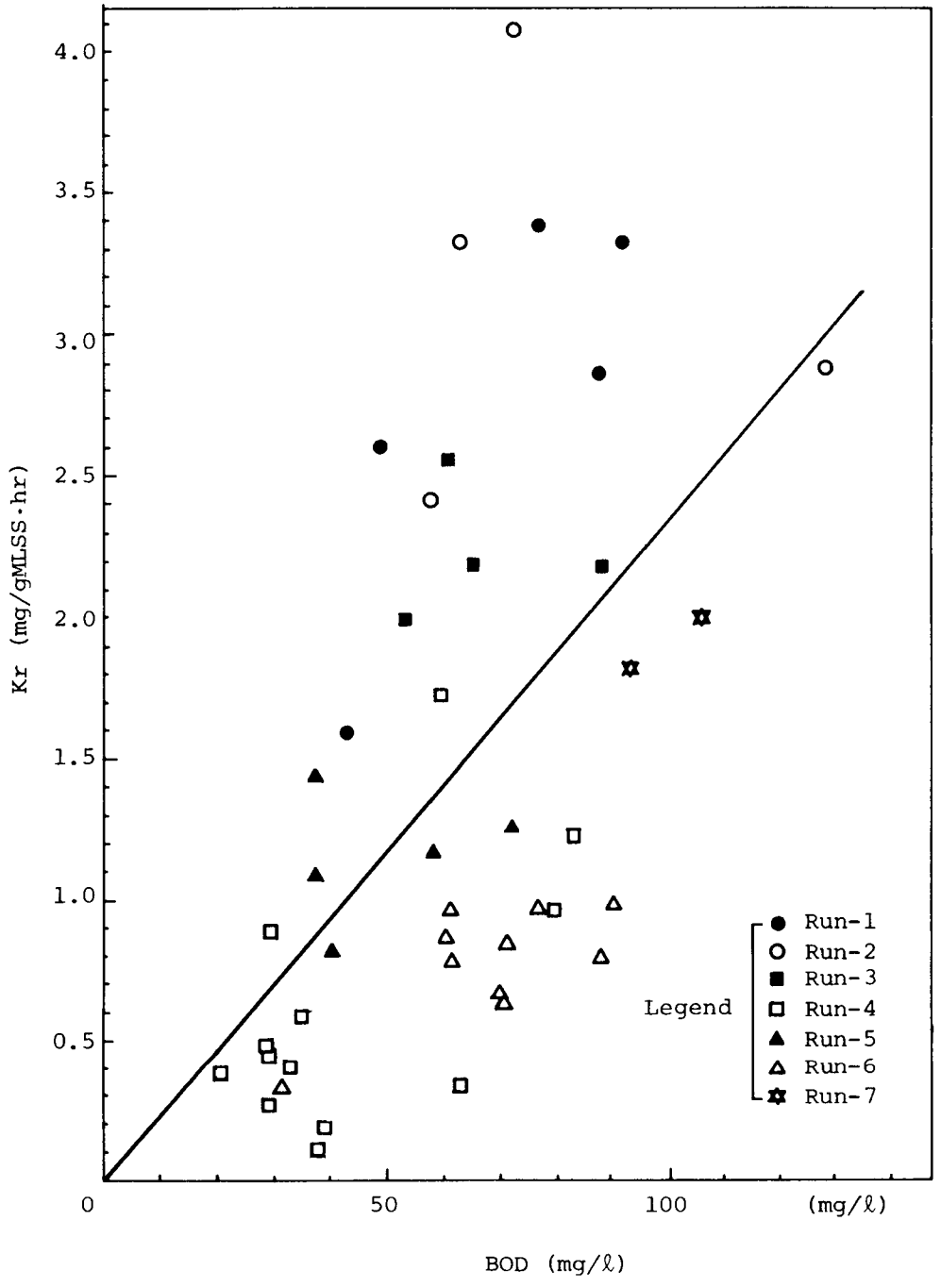


Fig. 13 Relation between Kr and BOD

Phosphorus Uptake Rate

It was presumed from the results of the 1981 survey[5] that the phosphate uptake rate could be expressed as:

$$\frac{dc}{dt} = -Ku \cdot X \cdot C \quad \cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots \quad (3)$$

Where,

Ku: S-O-P uptake rate constant (1/gMILSS·hr)

Fig. 14 shows the semi-logarithm plot of Ku to water temperature. Ku is distributed mainly within the range of 0.2 ∿ 0.6 with approximately 0.35 as the center.

(4) Re-release of Phosphate in the Final Clarifier

When sedimented and deposited in the clarifier, sludge finally becomes completely anaerobic after endogeneous denitrification and may re-release phosphate that it absorbed for once.

The concentration of S-O-P in the supernatant of the returned sludge was far greater than that at the bottom end of the aerobic cell - except for Run 4. Fig. 15 shows the relation between S-COD and S-O-P in the supernatant of the return sludge and the amount of phosphate re-release in the final clarifier. As can be seen from this figure, S-O-P in the supernatant of the return sludge is high in proportion to its S-COD. In the case of Run 4, the quantity of residual organic matter (BOD) in the return sludge was small because of the dilution of the influent with rainfall and the longer detention in the aeration tank. Therefore, the quantity of re-released phosphate was smallest.

By contrast, in Run 7 the quantity of residual organic matter in the return sludge was large because of the increased inflow during the period of low water temperature. Thus, the quantity of re-released phosphorus was large.

Fig. 16 shows the relation between the quantity of re-released phosphate and the sludge surface level in the final clarifier. The mean sludge level did not differ much between Rund 5 ∿ 7 but phosphorus re-release was largest in Run 7.

It is considered from the above that in order to prevent the

re-release of phosphate in the final clarifier, it is important to not only appropriately control the sludge surface level in the clarifier but also to remove organic matter thoroughly in advance.

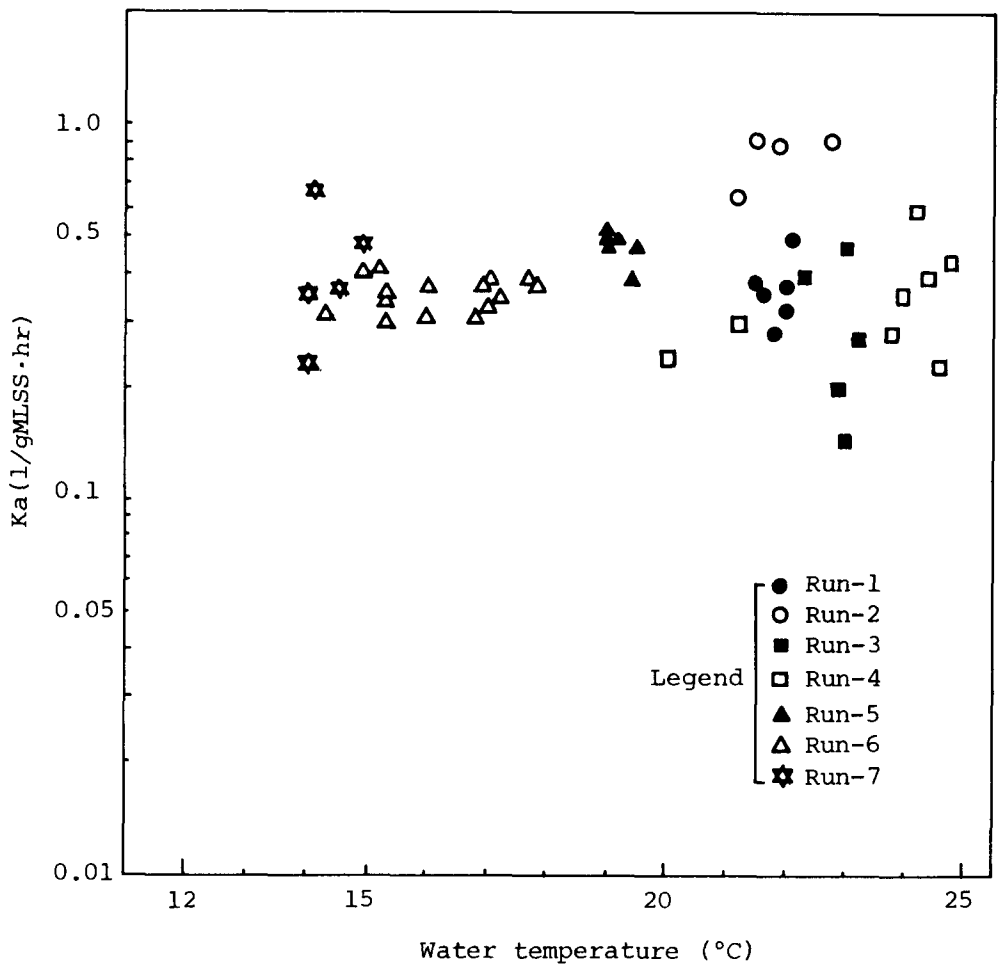
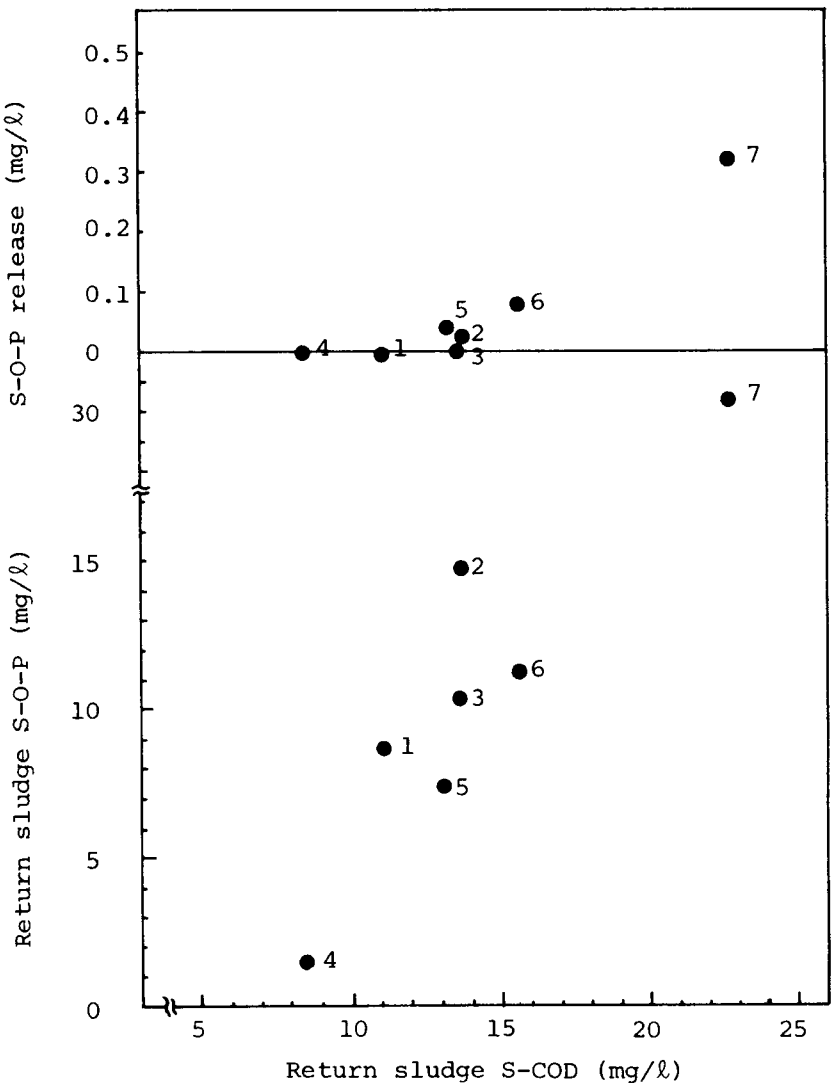
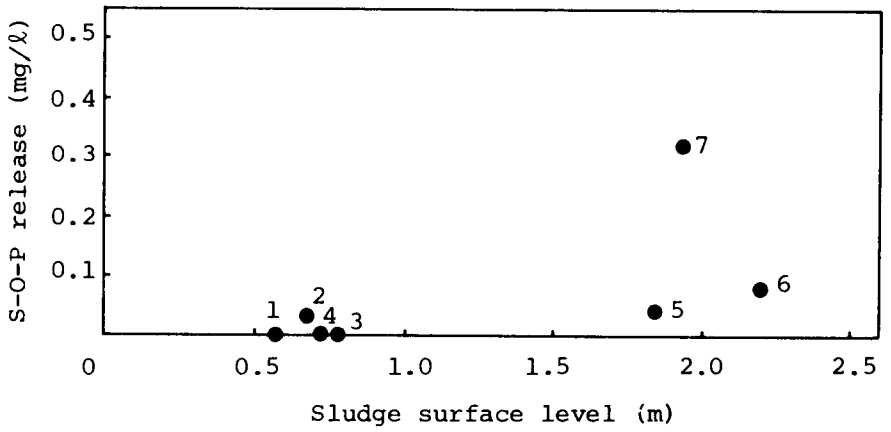


Fig. 14 Relation between Ku and Water Temperature



(Note) The mean value for each run.
The figure shows Run number.

Fig. 15 Relation between Return Sludge S-COD and S-O-P Concentration and S-O-P Release in Final Clarifier



(Note) The mean value for each run.

Fig. 16 Relation between Sludge surface Level in Final Clarifier and S-O-P Release

3.2.2 Nitrogen Removal

During the period of Operation Mode II (Runs 4 ∿ 6), not only phosphorus removal but also the removal of nitrogen by the recycling formula were carried out. According to the mean values in Table 10, the respective T-N removal efficiency was 49.7%, 52.0% and 58.0% for Run 4, Run 5 and Run 6. Since this survey stressed the removal of phosphorus, the operation for the removal of nitrogen was not necessarily under optinum conditions. To achieve high nitrogen removal, influent Kj-N should as a principle, be nitrified as much as possible but during the period of Operation Mode II, Kj-N remained in the final effluent.

(1) Nitrification Rate

The nitrification rate in the aerobic cells was calculated by the following equation:

$$K_N = \frac{CN_2 - CN_1}{DTN \cdot X} \quad \cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots \quad (4)$$

Where,

K_N : Nitrification rate constant (mg/gMLSS·hr)

CN_1: NO_T-N concentration at bottom end of denitrification cell (mg/ℓ)

CN_2: NO_T-N concentration at outlet of Nitrification cell (mg/ℓ)

DNT: Real detention time

X : MLSS concentration (g/ℓ)

In this calculation, it was assumed that nitrification follows the zero-order reaction and that the nitrification cell was a plug flow reactor. Only data with at least 0.5 mg/ℓ of HN_4^+-N remaining at the nitrification cell outlet were used.

The relationship between K_N and water temperature is expressed as follows:

$$K_N = 0.0985 \times e^{0.1156T} \text{ (mg/gMLSS·hr)} \quad \cdots\cdots\cdots\cdots \quad (5)$$

Taking Run 6 as an example, the nitrification rate from equation (5) is 2.60 mg/hr for the mean water temperature of 16.2°C, and the mean MLSS concentration of 4,050 mg/ℓ. Therefore, the aerobic tank detention time necessary to completely nitrify the influent Kj-N concentration of 27.6 mg/ℓ was 10.6 hr, whereas the actual aerobic tank detention time was 7.7 hr, indicating that the nitrification time was insufficient.

(2) NO_T-N Balance

An NO_T-N balance in the system was prepared for Runs 4 ∿ 6. A typical example is shown in Fig. 17. In this figure, "+" means the increase of NO_T-N through nitrification and Δ means its decrease through denitrification. The figure shows that denitrification occurs in both the denitrification cell and the final clarifier. Thus, the NO_T-N in the return sludge was mostly denitrified in the final clarifier and very little returned to the anaerobic cell. The results of the survey of single-sludge recycling nitrification/denitrification system conducted in 1980 ∿

1981[4] revealed that hardly any NO_T-N remained in the return sludge if the concentration of the sludge was kept more than 9,000 mg/ℓ. Therefore, great care must be exercised about the concentration of return sludge in order to minimize the influx of NO_T-N into the anaerobic cell.

After heavy rain, the denitrification capacity of the final clarifier declined and the concentration of NO_T-N in the return sludge increased. This, along with the increase of NO_T-N in the influent itself, suppressed the release of phosphorus in the anaerobic cell. It will thus be necessary to consider the increase of NO_T-N in the anaerobic cell not only by influent, but also that by return sludge in order to cope with the effect of rain.

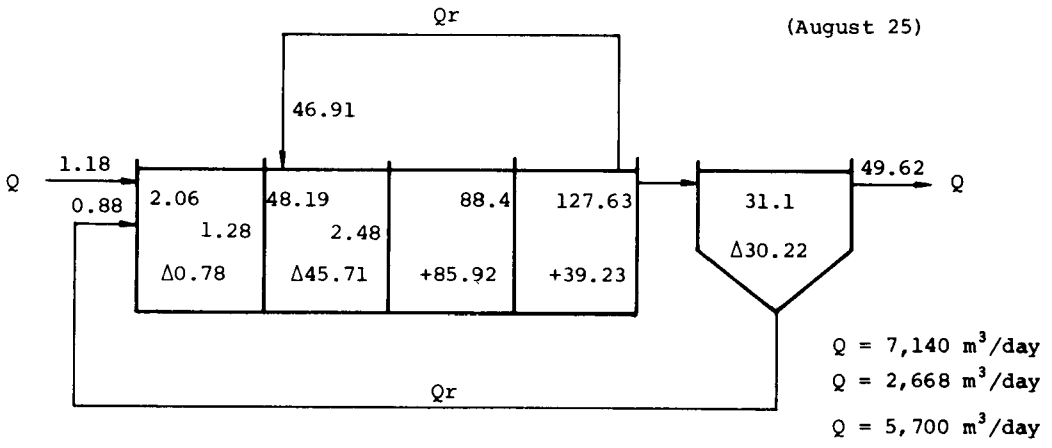


Fig. 17 NO_T-N Balance During Run 4

3.3 Results of Flow Rate Fluctuation Survey

In the last part of Run 4, the effect of flow fluctuation upon phosphorus removal was surveyed. Fig. 18 shows an inflow fluctuation pattern and the S-O-P change with time. The removal of S-O-P was always satisfactory regardless of flow fluctuation, even though the S-O-P concentration in the anaerobic cell changed considerably with fluctuations in flow.

The change of S-O-P in the denitrification cell was almost similar to that of the anaerobic cell.

The S-O-P concentration in the first aerobic cell was slightly affected by flow fluctuation but the S-O-P concentration in the second aerobic cell showed no change at all, presumably because the capacity of the aerobic cells was sufficient and the fluctuation of phosphorus concentration in the anoxic cell was absorbed in the aerobic cells.

3.4 Sludge Production

Table 11 shows the excess sludge production. Excess sludge production per BOD removed was much the same regardless of operation mode, water temperature, etc. and its mean value throughout all data was 0.569 kg/kg. In the survey conducted in 1980 ∿ 1981 on single-sludge recycling nitrification/denitrification system, using the same facility[4], excess sludge production was 0.297 kg/kg in the period of low water temperature and 0.335 kg/kg in that of high water temperature. Thus, the value obtained in this survey was considerably larger.

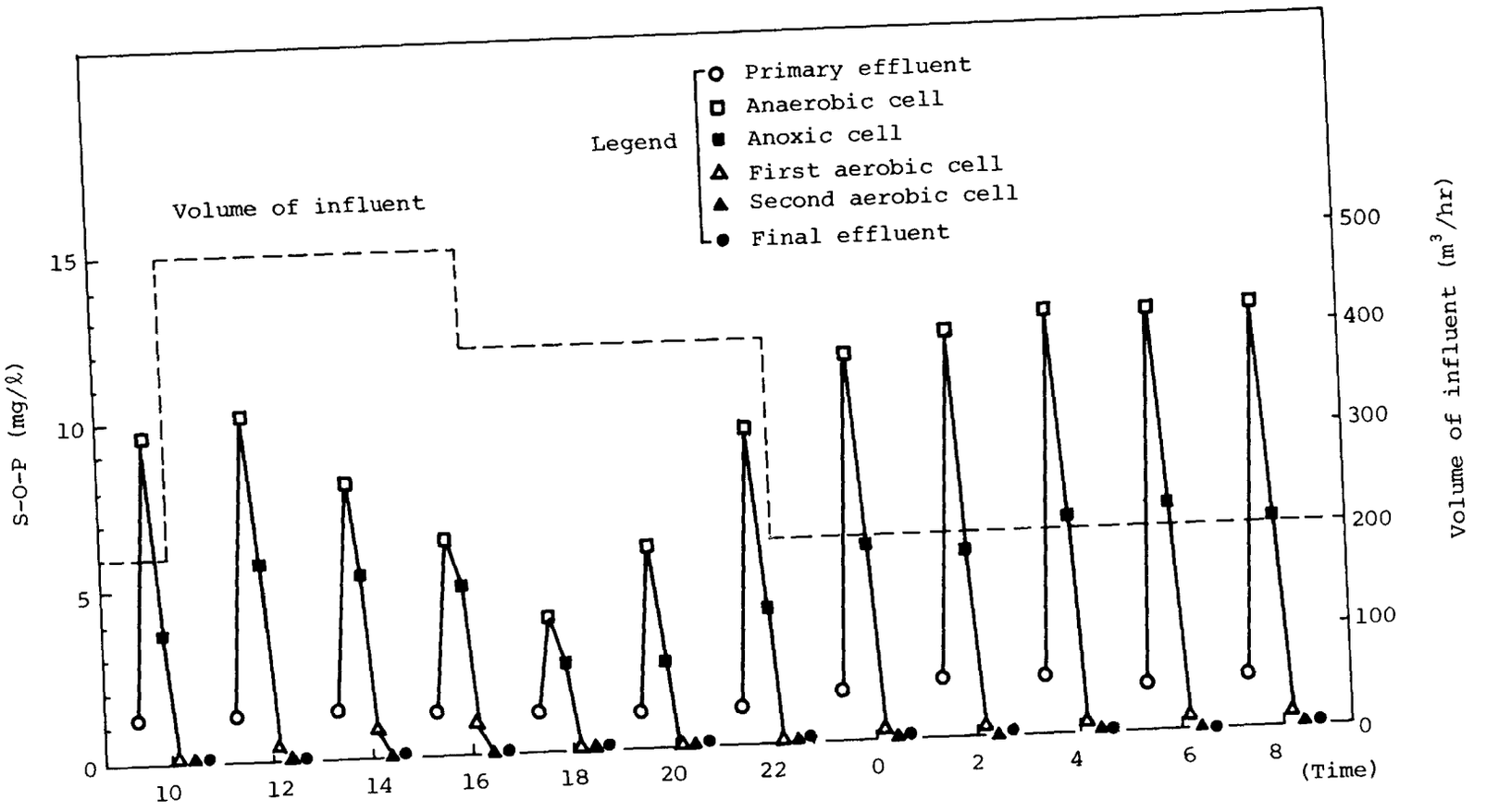


Fig. 18 Effect of Flow Fluctuation

Table 11 Excess Sludge Production

Period	Run	Water temperature (°C)	MLSS (mg/ℓ)	(A) BOD removed (kg/day)	(B) SS removed (kg/day)	(C) Excess sludge produced (kg/day)	(C)/(A)	(C)/(B)	$\frac{(C)}{(A)+(B)}$
June 11 ∿ June 30	1,2	21.5	2,090	1,065.2	856.5	608.5	0.571	0.710	0.317
August 25 ∿ September 6	4	24.6	2,330	474.3	464.5	283.5	0.598	0.610	0.302
November 22 ∿ December 22	5,6	17.4	4,160	531.9	460.6	293.5	0.552	0.637	0.296
Average	-	-	-	684.1	582.9	389.4	0.569	0.668	0.307

3.5 Others

3.5.1 BOD, COD and SS Removal

As indicated in Tables 9 and 10, the removal of BOD, COD and SS was satisfactory and stable throughout the entire period of the survey.

The transparency of the final effluent was 70 ∿ 100 cm on the average, with the exception of Run 7. In this run, the SS concentration in the final effluent was somewhat higher than that in other runs and its transparency remained at an average of 33 cm.

3.5.2 Sludge Characteristics

During the period of this survey, SV30 was at 15 ∿ 40 and SVI was within the range of 70 ∿ 100. As far as can be determined by these indices, the settling characteristics of sludge did not differ greatly when compared to the conventional activated sludge at the other trains of the plant.

3.6 Summary

The following has determined so far from the demonstration operation of biological phosphorus and nitrogen removal system:

(1) A final effluent T-P concentration of less than 1 mg/ℓ can be obtained by an anaerobic/aerobic system.

(2) Nitrate nitrogen suppresses phosphorus release in the anaerobic cell and the phosphorus removal efficiency drops.

(3) ORP is useful as a control index of anaerobic cell.

(4) Phosphorus release and uptake rates were not so affected by the water temperature. Phosphorus release is affected by the influent BOD.

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Ninth US/Japan Conference
on
Sewage Treatment Technology

SURVEY OF INFILTRATION / INFLOW IN SEWER SYSTEMS

September 19 ~ 21, 1983
Tokyo, Japan

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Dupty Director
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1. PREFACE

As a result of the spread of sewage works and the increase of drainage areas, the amount of influent to sewage treatment plants has increased in recent years. In the sewer treatment plants where the influent is likely to exceed the design flow, the re-examination of the design value becomes necessary. The presence of influent sewage exceeding the design flow has become obvious in recent years.

Influent sewage other than sanitary sewage is called "Infiltration/Inflow (I/I)" and is considered to be generally related to rainfall and groundwater. It is only very recently that Infiltration/Inflow has begun to be studied quantitatively.

Between fiscal years 1978 to 1981 and after reference from the Ministry of Construction, JSWA surveyed the Infiltration/Inflow to sewerage and possible measures for its prevention. This report has been prepared after examination of the inpouring mechanism of I/I based on the above survey and also proposes some improved design and maintenance procedures.

2. PROCEDURES FOR INFILTRATION/INFLOW SURVEY

Since it is usually possible for I/I to inpour through all portions of sewer in a drainage area, it is not possible to directly take out the I/I in the influent sewage. Ideally, the amount of infiltration may be determined by stopping all sanitary sewage within the drainage area, but sanitary sewage through a sewer line in service can in general be stopped only partially. If, however, it can be stopped like this, then the amount of infiltration can be estimated by measuring directly the influent in that portion. The estimation accuracy of the sanitary flows will be directly reflected upon the accuracy of the amount of infiltration, so that the accurate grasping of the sanitary flows become extremely important.

Described in this chapter, are the outline of the I/I survey performed for the test areas as well as the statistical estimation of sanitary sewage.

2.1 Outline of the Surveyed Areas

The I/I to sewer system was surveyed for test areas preselected. Most analyses stated up to Chapter 4 were based on the data obtained from this survey. Outline of these areas as well as observations will be described in this Section.

Drainage subsections A and B of Y-city satisfying the following conditions were selected as the survey areas:

① The drainage area is to be small so as to make the detailed survey possible.

② The majority of the survey area is to be composed of residential districts.

③ The anticipated amount of groundwater infiltration is to differ between the two subsections.

④ The survey area is to be served with separate sewers.

Y-city is located on diluvial plateau and alluvium formed by a river running through the central part of the plateau. The surface of the diluvial plateau is covered with a loam layer 3 to 5 m thick, beneath which are alternating fine sand and clay layers. The alluvium is covered with a humus soil layer about 3 m thick, with soft clay layer underneath.

Subsections A and B in the survey area are closely located as shown in Fig. 2.1. Most of subsection B is located on the diluvial plateau, but one-third of subsection A is on the alluvium and the remaining two-thirds is on a land gently sloping from the alluvium to the diluvial plateau and having a difference of about 7 m in elevation.

Outline of the sewer lines embedded in both the drainage subsections is shown in Table 2.1 and Fig. 2.1. Also shown in this figure are the sewer lines located below the average groundwater level.

Table 2.1 Outline of Sewer Facilities

Name of drainage subsection		A		B	
Area of district	(ha)	8.8		14.8	
Pipe diameter	(mm)	250		250	
Total length of sewers	(m)	2,041	(1,008)	2,927	(98)
Number of spans		65	(32)	80	(3)
Earth covering	(m)	1.0∿2.9	(1.85)	0.9∿3.5	(3.22)
Mean level of sewer bottom	(T.P.m)		(7.75)		(10.46)
Number of house sewers		220	(127)	266	(1)
Number of manholes		66	(33)	81	(4)

Values shown in ( ) concern with sewers below groundwater level.

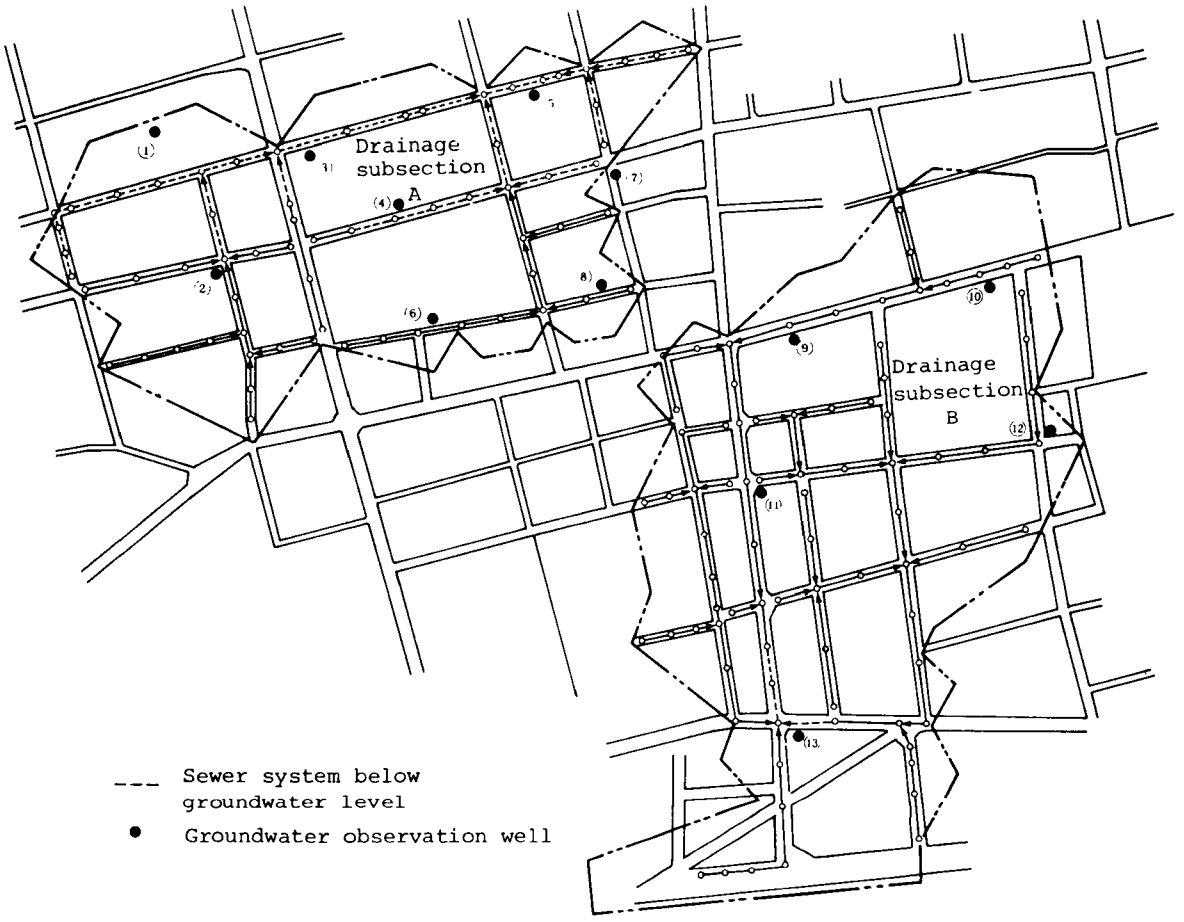


Fig. 2.1 Plan of district

In order to grasp the real quantity of the I/I, it becomes necessary to observe the sewage flow at the lowest point of drainage subsection, mean groundwater level and rainfall depth within the area. The groundwater level was regularly observed once a day by using the existing 12 wells in the area, and additionally at one bore hole where its level was recorded automatically. The amount of rainfall was automatically recorded at three places in that and adjacent areas, and then it was hourly averaged and commonly used for both the drainage subsections.

The purpose of these observations was to obtain mean values of I/I instead of the detailed distribution of I/I throughout the whole survey area. Therefore, the mean daily groundwater level obtained from the observation was used as analysis data for each subsection.

Fig. 2.2 shows the daily changes in sewage flow, groundwater level and amount of rainfall for a survey period of about 3 months in subsection A. For subsection B, various misconnections to sanitary inlets were investigated, as these are one of causes of stormwater inflow.

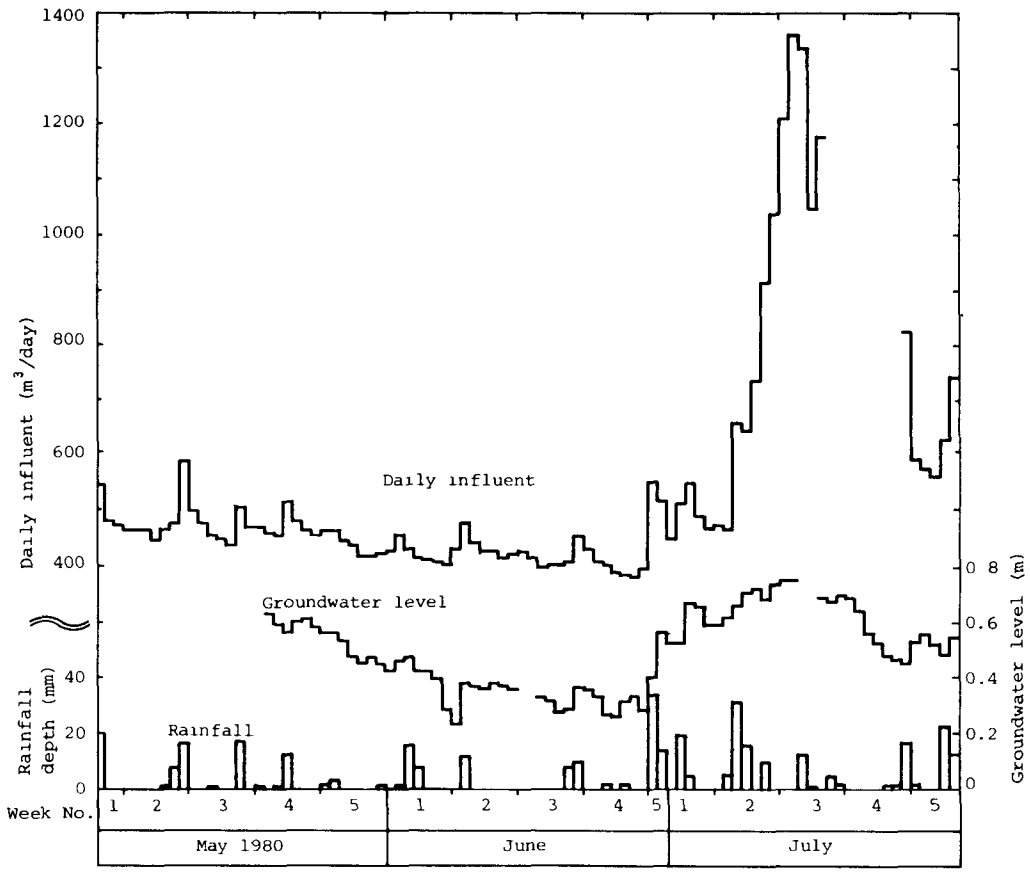


Fig. 2.2 Fluctuations in daily influent and explanatory factor (drainage subsection A)

2.2 Fluctuation Analysis of Daily Sewage Flow

Sewage flow to a sewage treatment plant or a pumping station is normally measured in hour units. For both of the drainage subsections in the survey area, the hourly sewage flow was measured at the end of each subsection. In this Section, the factors affecting daily sewage flow fluctuation obtained from the hourly sewage flow will be determined. If these factors are quantitatively determined, then the daily sewage flow can be estimated.

Where there seems to be no change such as an additions to the sewer line in the drainage subsection, the factors of fluctuation in daily sewage flow can be abstracted by the statistical handling. Domestic sewage flow will indicate certain tendencies. That is, it is known that the amount of domestic water used varies by season and a day of the week (weekly fluctuation), and the amount of laundry water varies with rainy and fine days.

Generally, the factors of fluctuation affecting the daily sewage flow are as follows:

① Season;
② Day of the week;
③ Amount of rainfall; and
④ Groundwater level.

In addition to the above, other factors such as geological conditions and kinds and states of sewers may exist but these cannot be easily handled statistically if a particular area is preselected as the survey area. Therefore, only the four factors listed above will be considered hereinafter.

Of these four factors of fluctuation, the seasonal fluctuation factor and weekly fluctuation factor are qualitative variates while the amount of rainfall and groundwater level are quantitative variates. The statistical analysis method for explaining the daily sewage flow as a dependent variate by such two kinds of fluctuation factors has not been generally established. However, since the quantitative variates can

be easily transformed to qualitative variates, quantification analysis was used by handling all of four factors as qualitative variates. Quantification regression analysis is one of multivariate analyses and its concept is shown in the references at the last part of this report.

Four qualitative items were classified into categories in accordance with the following criterions:

① Seasonal fluctuation:

Two weeks were considered as an unit, and the period of observation was divided into 6 categories.

② Weekly fluctuation:

Days of the week from Sunday to Saturday were used as variates. Therefore, the variates were divided into 7 categories.

③ Amount of rainfall:

Daily rainfall is an quantitative statistic but was divided into 6 categories within the range obtained during the observation period. Also, fine days (days without any rainfall) became one category.

④ Groundwater level:

As for rainfall, the ranges of groundwater level were classified into a total of 6 categories, thereby each groundwater level being divided into one of categories. The ranges of these variates differ between drainage subsections A and B but the number of categories was the same.

Results of application of the quantification analysis are indicated in Fig. 2.3.

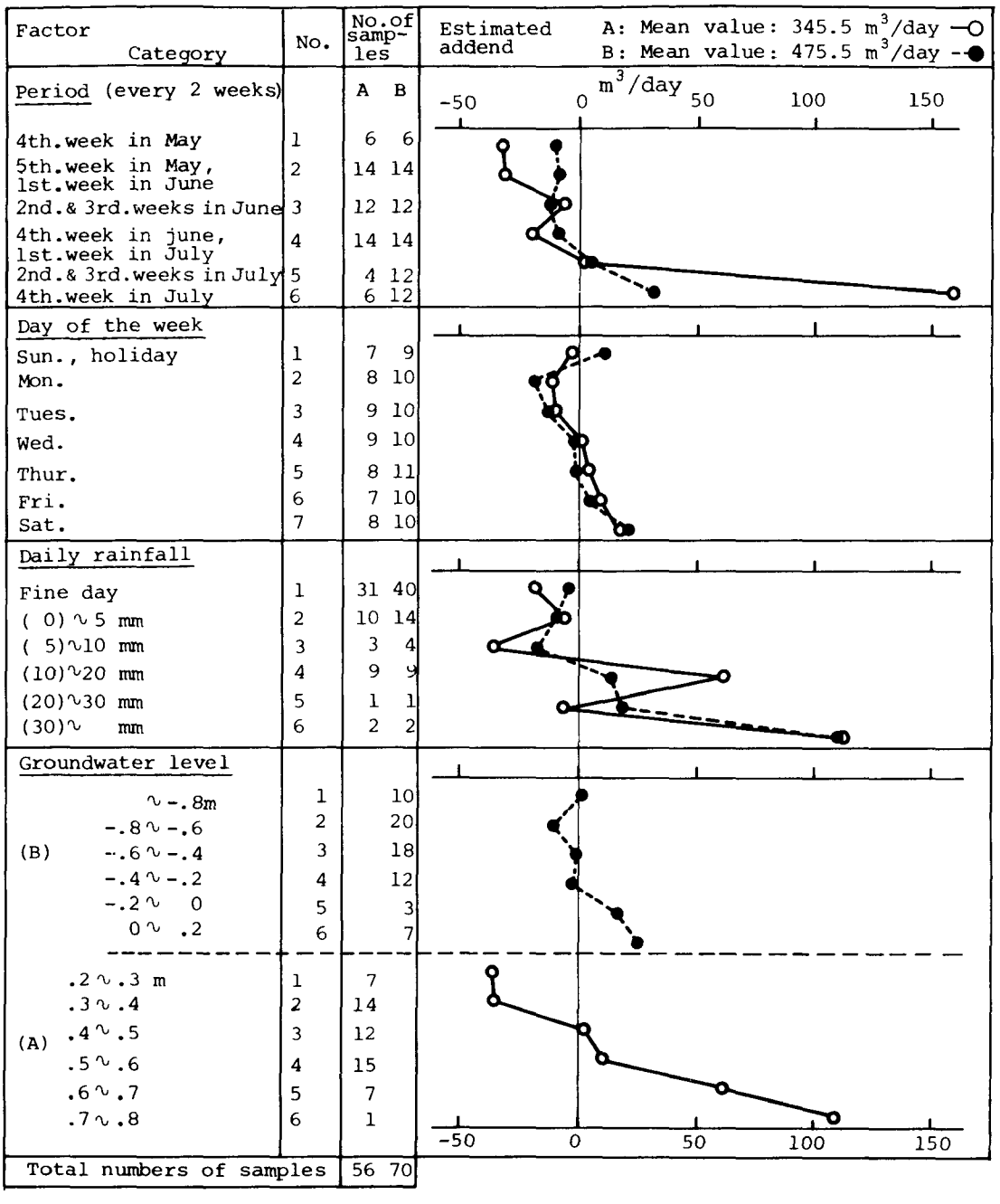


Fig. 2.3 Estimation of daily influent by quantification analysis

The estimated value of the influent for a particular day is obtained by adding the mean value to the sum of regression coefficients corresponding to categories for four factors. The correlation coefficient between estimated values and observed values is 0.95 for subsection A and 0.87 for subsection B, so that the application of the quantification analysis is considered to be appropriate.

According to Fig. 2.3, an obvious correlation is recognized between the groundwater level and the regression coefficient (estimated addend) in subsection A. Since this groundwater level is based on the average level of the sewer base in the subsection, the presence of positive correlation can be considered when the groundwater level is over the bottom level. Also, the same correlation will be obtained in subsection B when the groundwater level rises.

With respect to the daily rainfall, a correlation the same as that of the groundwater level can be expected but a fluctuation appears to some extent. This seems to be caused by the fact that the number of days of heavy rainfall is small and that there is no clear difference in the amount of water supply when the daily rainfall is less than 10 mm.

The regression coefficients due to the factor of day of the week are almost the same between both subsections, which means that this factor must be considered as one of fluctuation factors. Seasonal fluctuation has been generally known but it is not possible to grasp the tendency of seasonal fluctuation within such a short observation period of as 3 months.

2.3 Fluctuation Analysis of Hourly Influent

It is generally known that the fluctuation in hourly sewage flows in 24 hours a day has a particular pattern for each day of the week. In addition, calculating the amount of I/I, accurate estimated values of hourly flow becomes necessary. Therefore, the estimated hourly flow in the drainage subsection was calculated by the method described below.

That is, it was first confirmed by factor analysis that each day of a week has the particular pattern of daily fluctuation. Then, mean water flow in every hour of each day of the week was determined as a proportion to the daily sewage flow and it was confirmed that the daily fluctuation pattern actually varies. Finally, the estimated hourly flow was obtained by multiplying the daily sewage flow estimated in the previous section by the estimated proportion in every hour.

In the factor analysis, the 24 values for the hourly influent per day were expressed as proportions to the daily water flow, and a set of these values was used as a sample. Samples corresponding to 52 days were used for each of drainage subsections A and B in the analysis. Consequently, the contribution of the first factor was 85% and about 92% up to the third factor for both subsections, and the outline of fluctuation pattern of each day of the week can be explained using up to the third factor.

Since scores of the first factor of each sample were in the range between 0.8 and 1.0, the fluctuation pattern cannot be classified. Each day was then classified based on the scores of second and third factors. Scores of two factors are shown in Fig. 2.4 for subsections A, and in Fig. 2.5 for subsection B.

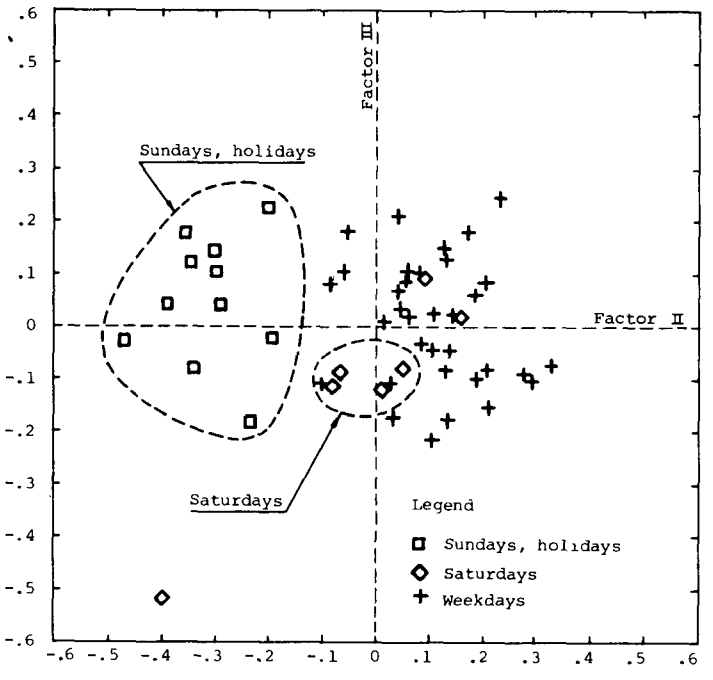


Fig. 2.4 Results of factor analysis of daily fluctuation in influent (drainage subsection A)

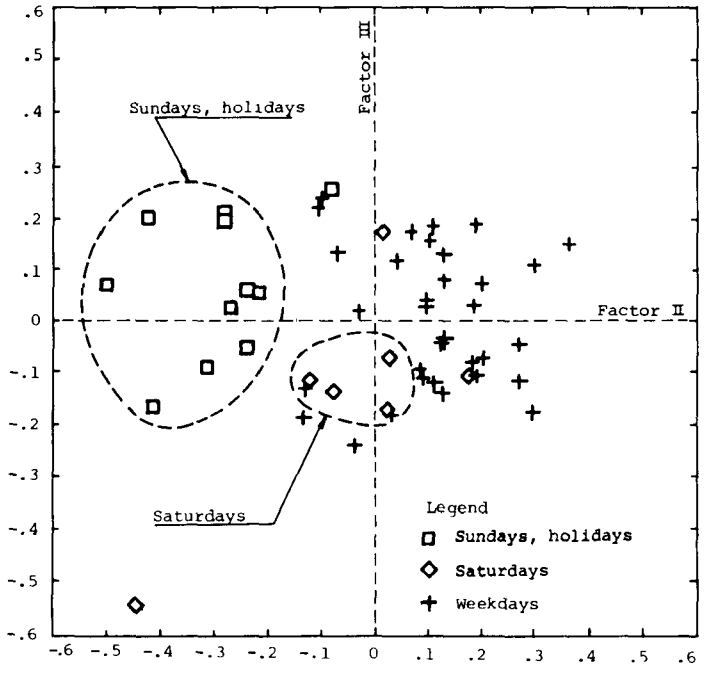


Fig. 2.5 Results of factor analysis of daily fluctuation in influent (drainage subsection B)

According to these figures, the daily fluctuation patterns can be classified into two major groups of the Sunday and holiday group and the weekday group. Within the weekday group, there is a Saturday group slightly separate from the weekdays so that it is possible to distinguish the Saturday group from the remaining weekdays. Since there is no great difference in the daily fluctuation pattern between the remaining weekdays, it is more appropriate to classify the daily fluctuation patterns into three groups; the Sunday and holiday group, the Saturday group and the weekday group.

The average daily fluctuation pattern of each group for both of the drainage subsections is indicated in Figs. 2.6 and 2.7. If the fluctuation pattern of the weekday group is used as a reference, then the fluctuation of the Sunday and holiday group has a delay of 1 to 2 hours from 7:00 AM to 4:00 PM, and the fluctuation of Saturday group has an increase of about 1.5% in daily sewage flow from 12:00 AM to 4:00 PM. Also, the Saturday group sometimes indicates a local maximum at about 2:00 PM. The daily fluctuation patterns also have other features but these will not be explained here.

According to the analysis stated above, hourly sewage flow/daily sewage flow ratio will sometimes vary more than 5% between groups depending upon the time, so that it seems to be necessary to at least take account of the weekly fluctuation when estimating each hourly flow.

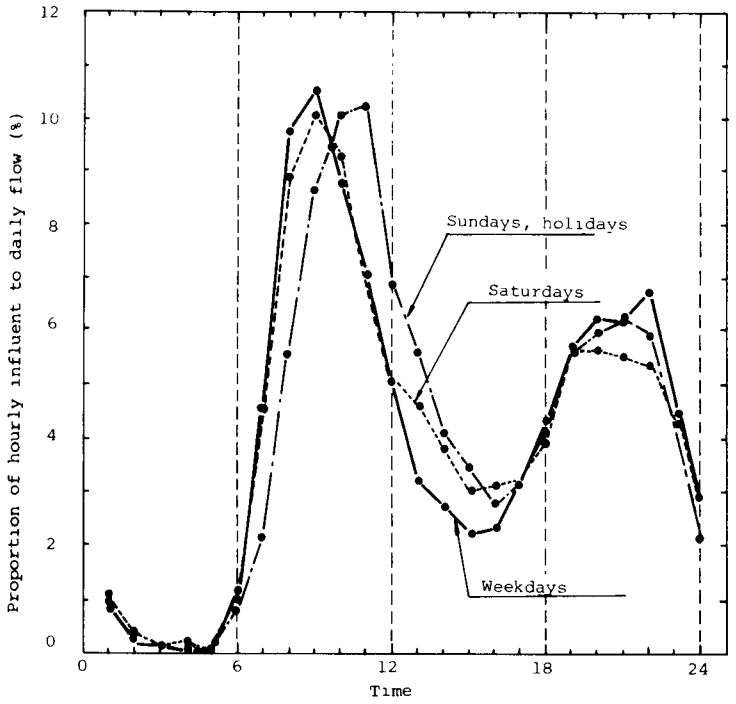


Fig. 2.6 Daily fluctuation patterns of influent (drainage subsection A)

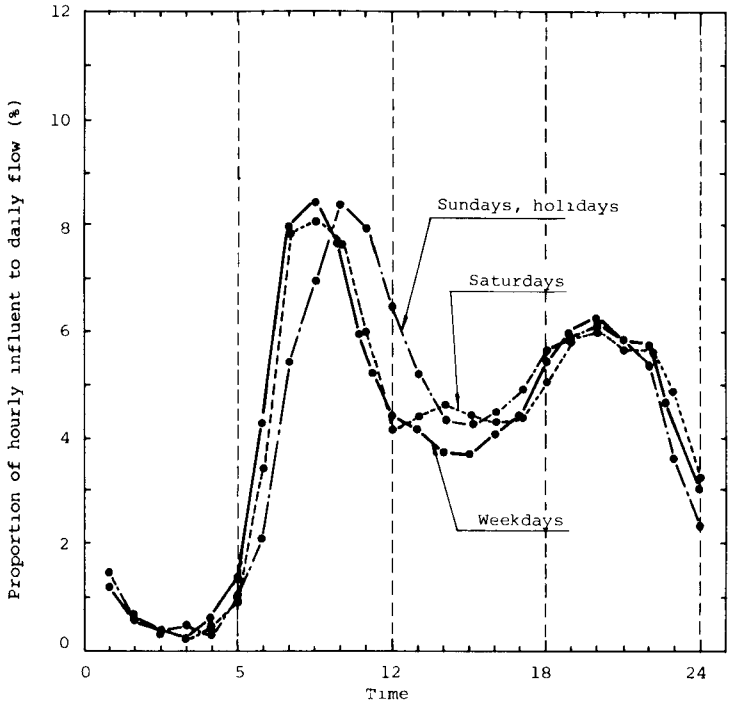


Fig. 2.7 Daily fluctuation patterns of influent (drainage subsection B)

3. ESTIMATION OF INFILTRATION/INFLOW

3.1 Estimation of Stormwater Inflow

The infiltration/inflow can be mainly divided into two kinds; the I/I due to stormwater and the I/I due to groundwater. The amount of stormwater inflow is, as same as ordinary runoff phenomena,considered to appear in the influent after a certain time of the rainfall. On the other hand, the groundwater infiltration generally fluctuates in response to the groundwater level and thus its influctuation is considered to be smaller than that of stormwater inflow. Therefore, the amount of stormwater inflow contained in the influent can be obtained by subtracting the mean sanitary flow during rainy days from the measured sewage flow.

Though mean sanitary flow during rainy days should be estimated for the condition of no rainfall, it usually cannot be calculated. Then, it is normally estimated on the base of the mean sewage flow for the fine days. The amount of stomwater inflow was estimated by the following sequence in this analysis:

① To estimate the daily sewage flow by taking account of the seasonal and weekly fluctuations.

② To assume that there will be no daily fluctuation in the amount of groundwater infiltration and that a constant flow will inpour every hour in the day.

③ To calculate the hourly sanitary flow by the procedures described in the previous chapter.

④ To assume particular hours during which the influent will be influenced of rainfall. Then, for the hourly influent in hours other than these, to change the amount of groundwater infiltration in such a manner that the sum of squares of difference between the hourly flows derived as sum of steps ② and ③ listed above will become a minimum value. Sewage flow for every hour when the sum of squares of difference becomes a minimum is the mean hourly sewage flow.

⑤ Amount of stormwater inflow is determined by the sum of the difference between the measured sewage flows and the estimated sanitary flows within the assumed hours.

The results obtained are indicated in the form of a relation betwen amount of stormwater inflow and amount of single rainfall in Figs. 3.1 and 3.2. In this case, the amount of single rainfall is the amount of rainfall in a continuous period of time when the hourly rainfall depth becomes higher than 0.5 mm. In calculating the amount of stormwater inflow the duration of rainfall and one hour after rainfall were used.

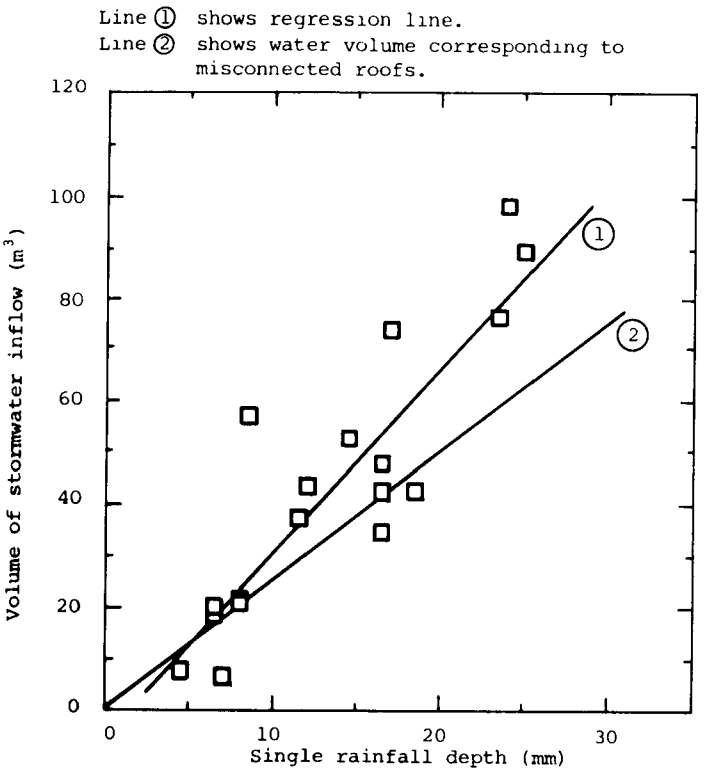


Fig. 3.1 Relation between rainfall and stormwater inflow (drainage subsection A)

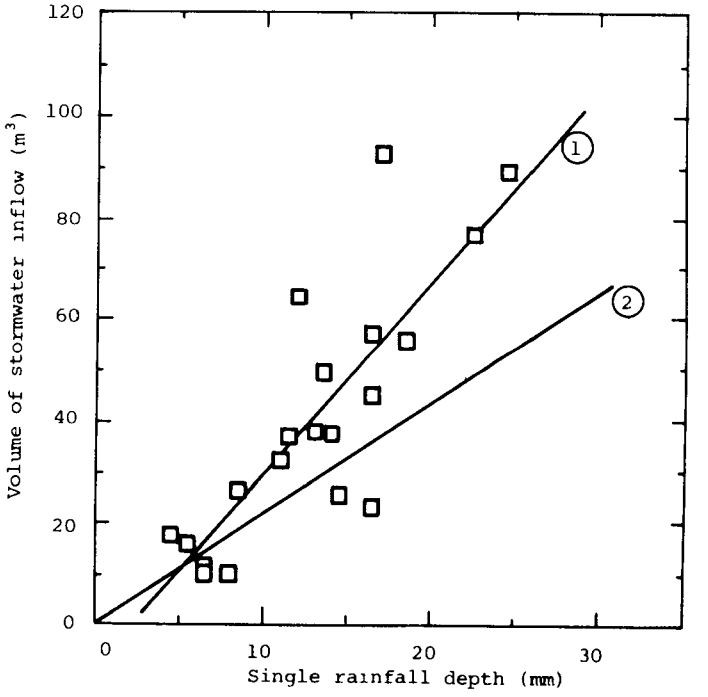


Fig. 3.2 Relation between rainfall and stormwater inflow (drainage subsection B)

For the drainage subsection B, misconnections of stormwater catchments to sanitary sewer inlets were investigated, which are considered to be one of causes of stormwater inflow. The results of this investigation are shown in Table 3.1. According to this table, the total roof area related to the misconnections in subsection B is to 2,173 m^2 which is about 7% of the total roof area in the same subsection. The roof area related to the misconnections in subsection A was estimated to be 2,517 m^2 based on the measured value of the subsection B.

Table 3.1 Survey on misconnected roofs

Name of drainage subsection	B	A
Number of houses surveyed	325	——
Number of houses with sanitary sewage inlets	253	293
Residential area	71,897 m^2	
Roof area	32,720 (45.5%)	——
Open space	39,177 (54.5%)	
Number of connections with roof drains	1,339	——
to stormwater catchments	190	
to sanitary sewage inlets *	110	
to lawns	1,039	
Area of misconnected roofs *	2,173 m^2	(2,517) m^2

Fig. 3.2 shows the amount of stormwater inflow indicated by straight line ① derived as the difference between the influent and the mean sanitary flow, and the flow corresponding to the misconnections indicated by straight line ②. According to this figure, if the amount of rainfall is small, most of the

stormwater inflow is directly caused by the inflow of rainwater from roofs related to the misconnections. However, if the amount of rainfall is large, the amount of inflowing rainwater will exceed the value corresponding to the roof area. Major inflow passage of stormwater inflow will start from roofs to stomwater drainage facilities and to sanitary sewage inlets, but other passages can be also considered when the amount of rainfall increases. The other passages are listed below:

① There are certain inflows to sanitary sewage inlets other than those resulting from the misconnections.

② While rainwater on lands such as gardens other than house lots is permeating underground, it flows into the sewers through their defective portions. If the amount of rainfall is large, the groundwater level may rise within a short period of time.

③ Rainwater on roads flows to manholes.

Among these passages listed above, it was confirmed by measurement that the inflow through manholes is very small, but the inpouring passage ① from the sanitary sewage inlets was not confirmed.

The inflow of rainwater to sewers in the middle of permeation into the ground is considered to be proportional to the total area of the openings in the defective portions of sewers but the amount of this inflow may not be large. The other possible cause is the rise in groundwater level and this will temporarily occur when there is a large amount of rainwater. However, though the hourly fluctuation in groundwater level was measured in this survey, the results could not be fully utilized in the analysis so a detailed survey on this fluctuation will be necessary in future.

The abovementioned is also the case with subsection A indicated in Fig. 3.1.

3.2 Estimation of Groundwater Infiltration

The actual estimation of groundwater infiltration will be explained in the next chapter. This section presents the simplified estimation appliable only to subsections A and B. The residential area accounts for the major part of subsections A and B, and there are only few factories and offices that discharge waste water during the night. Then, the minimum value of influent at night after 24 hours without rain, is considered to be caused by only the groundwater.

Therefore, the minimum hourly influent for a fine day was determined as the amount of groundwater infiltration. In the subsection A, the obtained relation between the amount of groundwater infiltration and the groundwater level is shown in Fig. 3.3. In subsection B, the relation with the groundwater level was not obtained since the groundwater level was often lower than the sewer bottom.

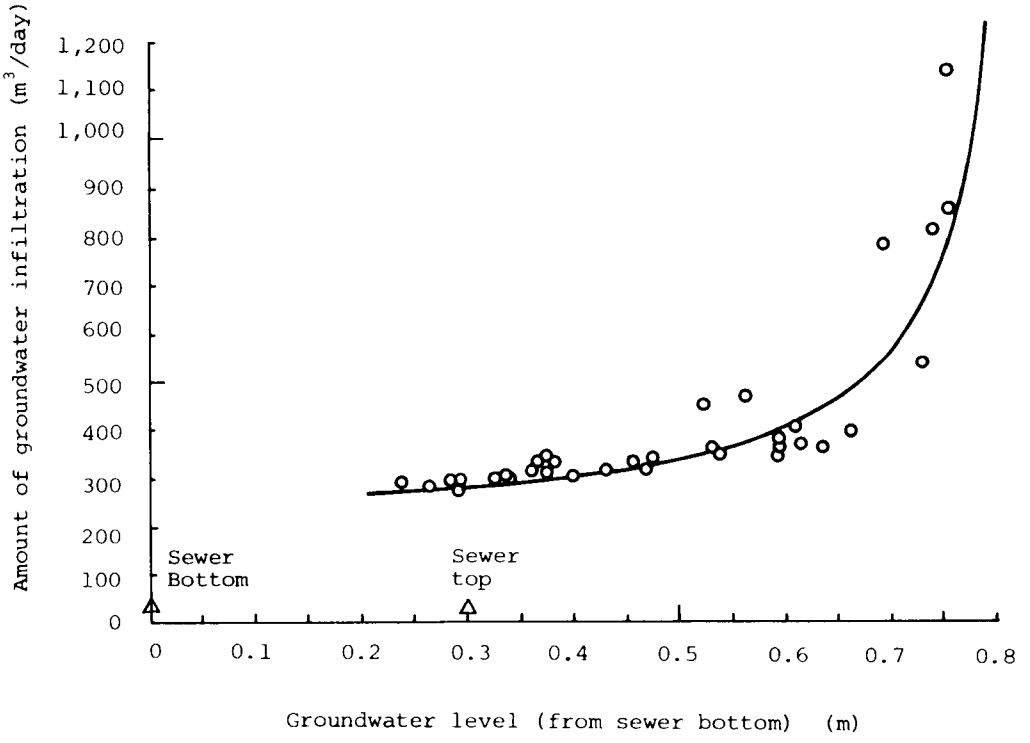


Fig. 3.3 Groundwater level and groundwater infiltration on fine days (drainage subsection A)

According to Fig. 3.3 the rate of increase of infiltration is small for the range where the groundwater level is from the top of main sewer to the central portion of house connections (a height of about 60 cm from the sewer bottom). At the top of house connections or at heights close to the sanitary sewer catchment (a height of more than 70 cm from the sewer bottom), the amount of infiltration is large and the increase rate is also large. The amount of infiltration when the groundwater level was near the main sewer was not measured. Generally, groundwater infiltration is considered to be not present when the groundwater level is under the height of the sewer bottom. Therefore, in this case, it is considered that fluctuation of the distribution of groundwater levels in the area was great and a considerable amount of I/I inpoured through sewers in some areas even though the mean groundwater level was below the mean height of sewer bottom.

4. MEASUREMENT OF INFILTRATION/INFLOW

4.1 Hydraulics of Groundwater Infiltration

In studying the hydraulics of groundwater infiltration into sewers, the location of infiltration (the defective portions of sewers), must be clarified. However, it is generally difficult to survey the location of defective portions (openings) and their area and the geological conditions in surrounding areas. Hence, in this kind of survey, the hydraulics of groundwater infiltration is assumed in advance and then the correctness of this assumption is comprehensively verified. This method was also used in this survey.

If a model as shown in Fig. 4.1 is considered as the relative positional relation between the embedded sewer and groundwater, then the amount of influent (pumped discharge) Q is given by the following formula: [2)]

$$Q = 4kr_o (H - h_o) \quad \cdots\cdots\cdots\cdots\cdots\cdots \quad (1)$$

where, H : original groundwater level

h_o : water level in well

r_o : radius of well

k : coefficient of infiltration

Eq. (1) is applicable only to shallow wells with their bottoms not reaching to an impermeable layer and is related to only the permeation through the bottom while the wall of the well consists of impermeable materials. Also, this equation seems to express the steady state when the amount of influent is considerably large and the surrounding groundwater level becomes equal to the water level in well.

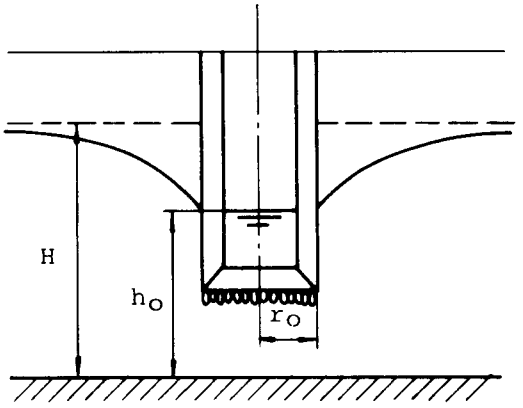


Fig. 4.1 Hydraulics of shallow well

When groundwater infiltration into the sewers is almost neglected in comparison to the groundwater supply, the fluctuation in groundwater level resulting from I/I seems to be very small. In this case, the hydraulics of I/I is considered to be similar to that of the orifice.

When the water level in a tank is H, the quantity of runoff Q is given by Eq. (2) for the orifice.

$$Q = Ca\sqrt{2gH} \quad \cdots\cdots\cdots\cdots\cdots\cdots \quad (2)$$

where, C : coefficient of discharge

a : sectional area of orifice

When the I/I from only one defective portion in a sewer is considered, then Eq. (2) can be applied as it is. Therefore, for calculating the I/I in the whole drainage area by Eq. (2) the following assumptions were prepared and then the phenomenon at points was extended to the phenomenon in the whole area:

① Groundwater level will not fluctuate as a result of the I/I flow.

② The relative height between groundwater level and sewer will be maintained constant within the drainage subsection.

③ Sections of defective portions in a sewer will be continuously distributed above the sewer bottom in average.

Based on the above assumptions, the groundwater infiltration model within the drainage area is considered to have the average section of sewer system as shown in Fig. 4.2.

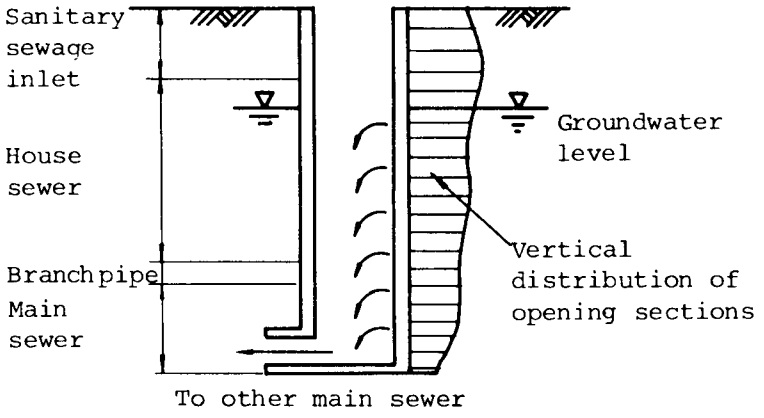
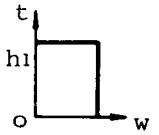
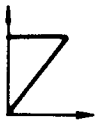
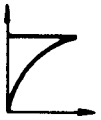
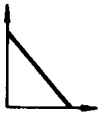
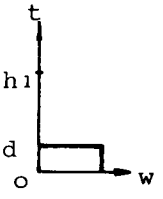


Fig. 4.2 Sewer model

The relation between infiltration flow and groundwater level can be obtained by assuming the vertical distribution of sectional areas of openings at defective sewer (inlets of I/I). From several examples of vertical distribution of sectional areas of openings the relation between groundwater level and infiltration flow was estimated as shown in Table 4.1. This estimation will be explained below.

Table 4.1 Distribution of Opening Widths and Calculating Formulas of Infiltration

No.	Distribution pattern of opening widths	Total sectional area of openings	Opening width distribution formula W(h)	Calculating formula of infiltration Q(h)
1		A	$\frac{A}{h_1}$	$CA\sqrt{2gh_1}\cdot\frac{2}{3}\left(\frac{h}{h_1}\right)^{3/2}$ $(h_1 \geqq h \geqq o)$
2		A	$\frac{2A}{h_1^2}t$	$CA\sqrt{2gh_1}\frac{8}{15}\left(\frac{h}{h_1}\right)^{5/2}$ $(h_1 \geqq h \geqq o)$
3		A	$\frac{3A}{h_1^3}t^2$	$CA\sqrt{2gh_1}\frac{16}{35}\left(\frac{h}{h_1}\right)^{7/2}$ $(h_1 \geqq h \geqq o)$
4		A	$\frac{2A}{h_1^2}(h_1 - t)$	$CA\sqrt{2gh_1}\cdot\frac{4}{15}\left\{5-2\left(\frac{h}{h_1}\right)^{3/2}\right.$ $(h_1 \geqq h \geqq o)$
5		A	$\frac{A}{d}$	$CA\sqrt{2gh_1}\cdot\frac{2}{3}\left(\frac{h}{d}\right)\left(\frac{h}{h_1}\right)^{1/2}\left\{1-\left(1-\frac{d}{h}\right)^{3/2}\right\}$ $(h_1 \geqq h \geqq d)$ $CA\sqrt{2gh_1}\cdot\left(\frac{h}{h_1}\right)^{1/2}$ $(d \to o,\ h_1 \geqq h > o)$

When the groundwater height from the sewer bottom is h, then the flow rate from an opening at a height t will be given by Fq. (3).

$$dQ(t) = C \cdot W(t) \sqrt{2g(h-t)}\, dt \qquad \cdots\cdots\cdots\cdots (3)$$

where, C : coefficient of discharge

W(t) : width of opening at height t

dQ(t) : flow rate from a sectional area W(t)dt at height t.

Therefore, total flow Q(h) is given by Eq. (4).

$$Q(h) = \int_0^h C \cdot W(t) \sqrt{2g(h-t)}\, dt \qquad \cdots\cdots\cdots\cdots (4)$$

When the total area of an opening from height t=o to t=h_1 takes a constant value (A), the vertical distribution of opening widths W(t) will be given and the infiltration flow can be determined as function of groundwater level h from Eq. (4).

Shown in Fig. 4.3 are the relations between the infiltration flow Q and groundwater level h, which corresponds to the distribution of opening sections stated in Table 4.1. In the actual measurement of the amount of infiltration, the relation similar to the distributions ② and ③ of opening sections is often obtained for many cases.

Nos. shown correspond to those shown in Table 4.1.

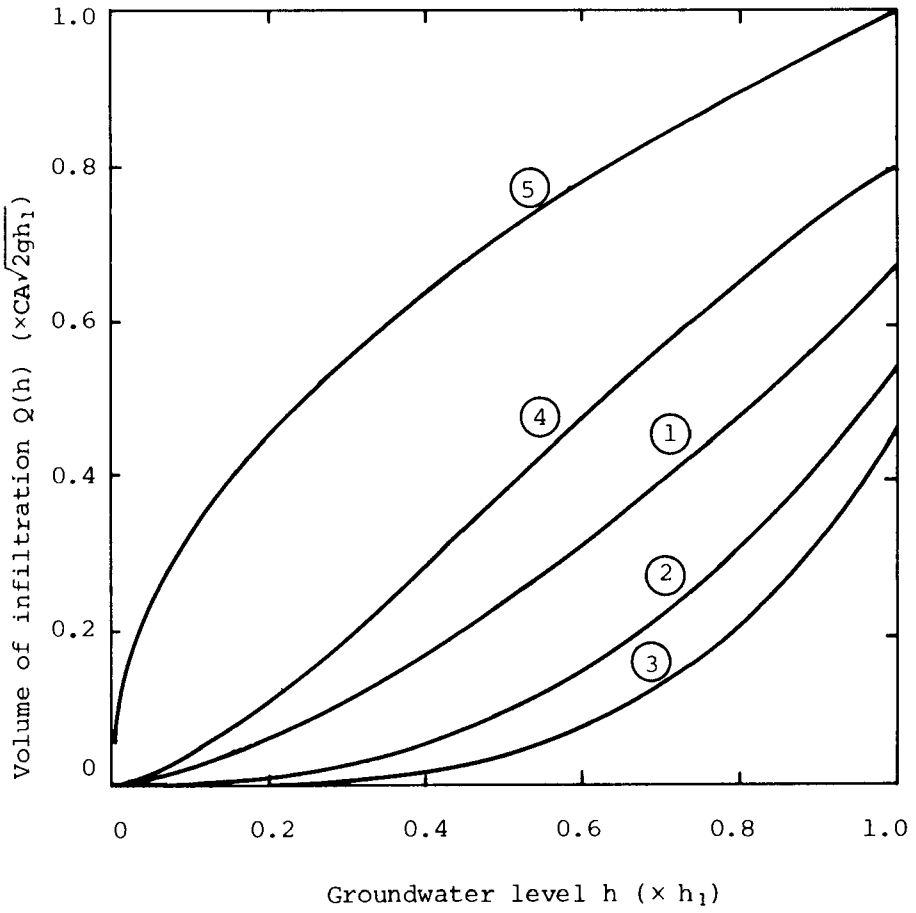


Fig. 4.3 Infiltration volume due to opening width distribution pattern

4.2 Measuring Methods for I/I Flow

Amount of groundwater infiltration may be estimated by comprehensively analyzing the discharge at the end of sewer line or, partially, by measuring the difference in discharge between the downstream end and upstream (inpouring) end of the sewer concerned. The former was already explained. In the latter case, the amount of infiltration is small compared to the influent and thus is within the measuring error of discharge, so that it may not be

obtained without solving this problem. Therefore, sewage flow is temporarily stopped and only discharge corresponding to the I/I is measured.

Three methods have been developed for measuring the I/I flow, pumping method, water-filling method and compressed air supply method. The pumping method will measure the discharge at the downstream end after stopping all I/I sources of sewage. In this measurement, the I/I stored upstream of the weir at an end is pumped up and its volume is measured. In this case, the relation between groundwater level and infiltration flow, is obtained by performing measurement at least two of different groundwater levels.

In the water-filling method, the quantity of leakage from a sewer filled up with fresh water after stopping sewage flows is obtained by measuring the water level in a manhole. Water level in manhole can be adjusted to the desired level above the groundwater level in surrounding areas, so that the relation between infiltration flow and groundwater level may be easily obtained.

In the compressed air supply method, low-pressure air (0.1 to 0.5 kg/m^2) is used instead of water and then the amount of air leakage is measured.

In these measuring methods for infiltration flow, the measured values obtained must be corrected in order to allow the proper estimation of the amount of infiltration. In the case of the compressed air supply method, the conversion of an amount of air to an amount of water and the clarification of air leakage mechanism are necessary, and also the correlation of this method with other methods is not clear. Therefore, the compressed air supply method is not suited to the estimation of the amount of infiltration. It seems to be appropriate to use this method for detecting large defective portions in sewers.

The same sewer was measured with both the pumping method and the water-filling method and then the correlation of the measured values was examined. Fig. 4.4 indicates the comparison between these measured values. According to this figure, the values obtained from the water-filling method are about 10 times those obtained from the pumping method, and the correlation between them appears to be very excellent.

Since the groundwater level is not considered in Fig. 4.4, the difference between the phenomena of I/I and water leakage cannot be clearly determined. Values about 10 times greater are likely to occur as long as the sectional areas of the openings at the defective portions of house sewers and inlets are as large as the case of the vertical distribution patterns ② and ③ shown in Table 4.1.

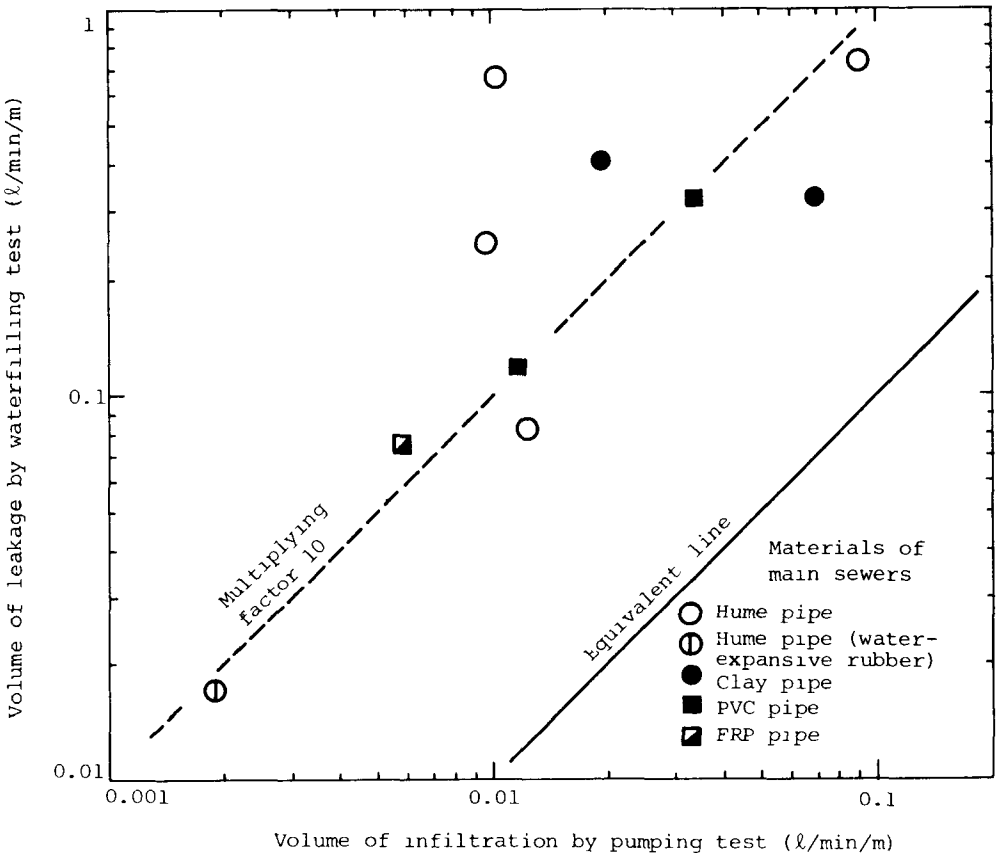


Fig. 4.4 Difference in measured values between pumping and water-filling methods

4.3 Infiltration Flow from Each Sewer Facility

In several cities, the amount of infiltration in existing sewer facilities was measured by the pumping and water-filling methods described in the previous section. Sewer facilities were divided into two groups depending upon the kinds of sewer materials, and the results of measurement of infiltration will be explained hereafter. However, in this section, only the measurement by the water filling method will be explained for more easy determination of the difference in amount of infiltration between different kinds of sewer facilities.

(1) Infiltration from manholes and main sewers

Amounts of infiltration from the main pipe between adjacent manholes and from a manhole at the upstream end of the main sewer were estimated by measuring the quantity of leakage with the water filling method. Two kinds of quantities were measured at each point. That is, the quantity of leakage from manholes and main pipe and also the quantity of leakage from only manholes were measured separately and the difference between these two quantities was calculated as the quantity of leakage from the main pipe. These measuring procedures were applied by moving the water stopping packer.

Results of measurement are shown in Fig. 4.5 in which quantities of leakage from manholes and main pipe are plotted on both axes. According to this figure, the quantity of leakage from the main pipe is 25ℓ/min (0.78ℓ/min/m) as maximum and about 8.1ℓ/min (0.28ℓ/min/m) in average. Also, the quantity of leakage from manholes is 3.23ℓ/min as maximum and 1.01ℓ/min in average.

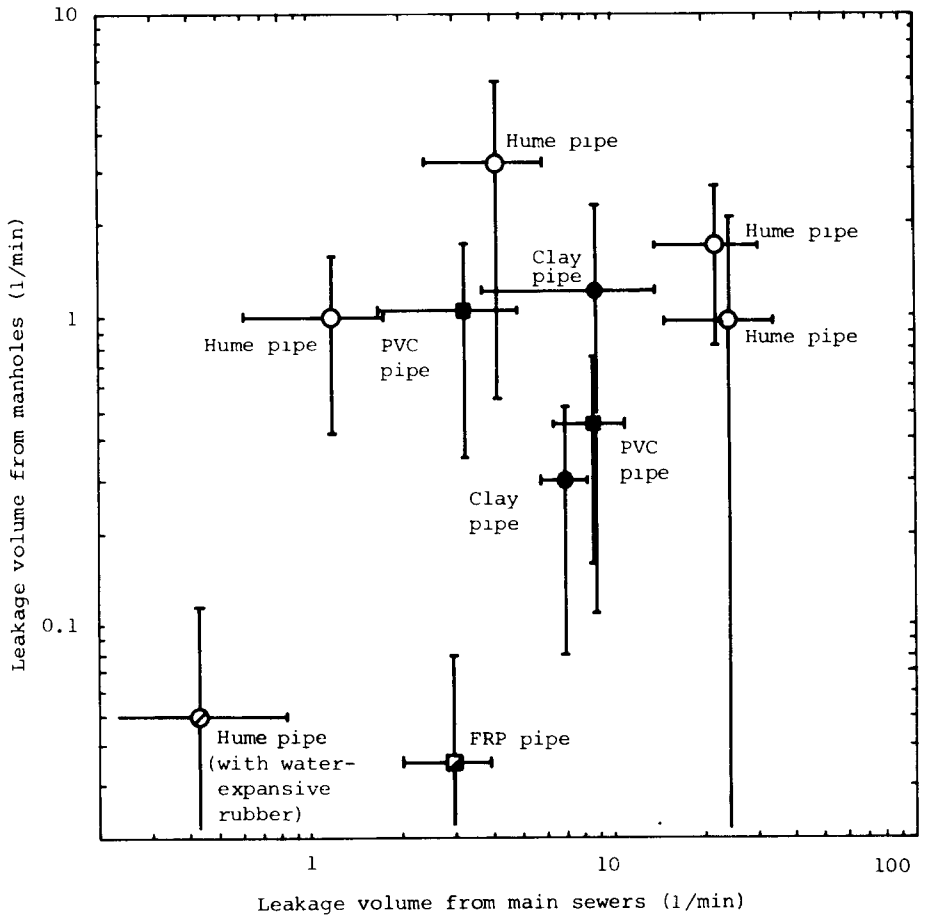


Fig. 4.5 Leakage volume from main sewers and manholes

Measured sewers was chiefly made of three kinds of materials; Hume concrete pipe, vitrified clay pipe and polyvinyl chloride (PVC) pipe. Among these kinds of pipes, Hume concrete pipes indicated a slightly larger quantity of leakage but the difference in the quantity of leakage between kinds of pipe materials was not generally significant. However, the quantity of leakage from main pipes and manholes was small when the main pipe had leak-proof connections such as Hume pipes connected with water-expansive rubber.

(2) Infiltration from house sewers and sanitary sewage inlets

The amount of infiltration from house sewers and sanitary inlets can be estimated by a series of measurement based on the water-filling method. That is, the quantity of leakage from the house sewers alone can be obtained by subtracting the quantity of leakage from the inlets alone from that from a set of house sewers and inlets. Water stopping packers are attached to the lowest point of house sewer for the former and to the highest point for the latter in order to measure the fluctuation in water level in the inlets.

Results of measurements performed for 4 areas are shown in Fig. 4.6. This measurement was performed for clay pipe and PVC pipe used for house sewers and for Hume pipe or clay pipe and PVC pipe used for the main sewer.

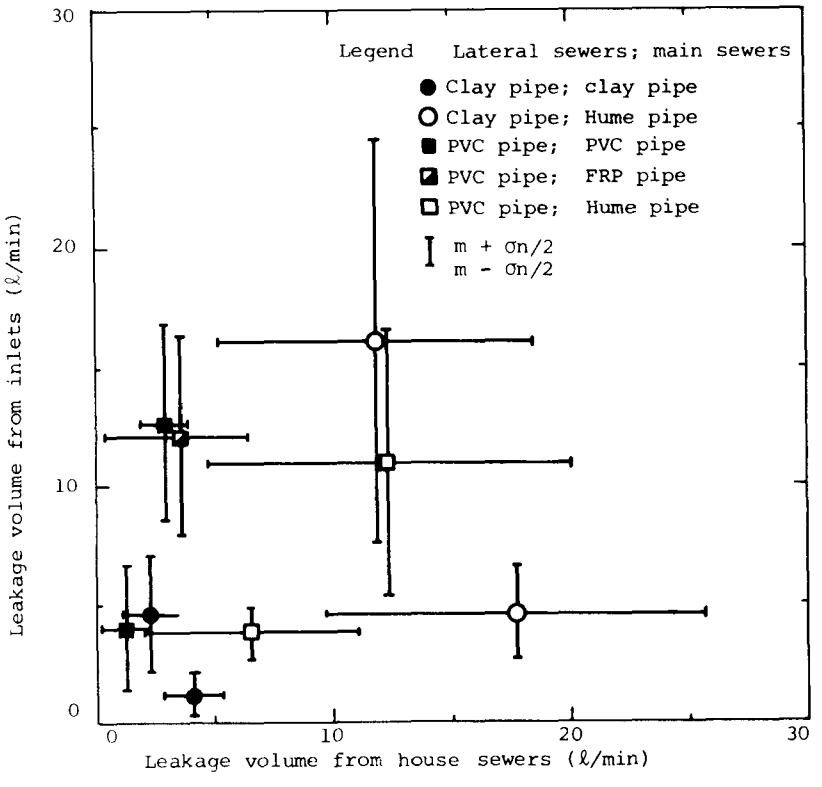


Fig. 4.6 Leakage volume from house sewers and inlets

According to Fig. 4.6, the quantity of leakage from the house sewers is small when the same kind of material is used for both the main sewer pipe and house sewers and is large when the different kinds of materials are used. This means that connections between the main sewer and house sewers are defective at the branches, or the connecting conditions of branch pipes are greatly affecting the connections with house sewers or the joints of house sewers.

On the other hand, the quantity of leakage from the inlets remains almost the same independent of the kind of material of house sewer.

(3) *Mean amount of infiltration at each facility*

Based on the quantities of leakage from manholes, main sewers, house sewers and sanitary inlets obtained by the measurements stated above, the quantities of leakage from standard combinations of these facilities were obtained and indicated in Table 4.2.

Table 4.2 Infiltration volume at each part of sewer system

Name of facility	Sample number	Total leakage volume ℓ/min	Unit quantity of leakage	Standard quantity	Estimated infiltration volume ℓ/min	Percentage (%)
Main sewers	3,573m	1,198.35	0.335 ℓ/min/m	10m	3.35	14.5
Manholes	125 sites	127.12	1.017 ℓ/min/site	0.35 sites	0.36	1.5
House sewers	82 "	470.8	5.74 "	1.46 sites	8.38	36.3
Inlets	82 "	618.4	7.54 "	1.46 sites	11.01	47.7
					23.1	100.0

Note: Standard quantity means the quantity of each facility per 10 m of main sewer and is the mean value in the test district concerned.

Table 4.2 shows the average values based on the water-filling method, and the actual amount of infiltration can be estimated after making corrections with groundwater level. However, the amount of groundwater infiltration usually becomes large when the groundwater level is high. Therefore, the fact that about 80% of the total amount of infiltration is inflow from house sewers and inlets will remain almost the same even after making corrections.

4.4 Presumption of Inpouring Points of Groundwater

Change in the amount of infiltration in response to the different inpouring points of groundwater was determined as the amount of infiltration for each sewer facility in the previous sections. However, in this section, the amount of infiltration will be determined in the form of a relation with the groundwater level, and then the method of assuming the groundwater inpouring points (in the form of height from the bottom of main sewer) will be explained.

If the vertical distributions of opening section in sewer facility are considered to be ①, ② and ③ of Table 4.1 and if the groundwater level (h) is given by the height from the sewer bottom, then the amount of infiltration is given as a function of the groundwater level. This function was generalized from the calculating formula shown in the same table.

$$Q(h) = \alpha \cdot h^n \qquad \cdots\cdots\cdots\cdots\cdots\cdots \quad (5)$$

where, α : constant.

In Eq. (5), the distribution of opening section is expressed as n-th power of h. Therefore, this power value will be considered as an index for expressing the distribution of opening section and will be called "I/I distribution index".

If the amount of infiltration is measured twice when the groundwater levels are different at certain sewer facilities, and the distribution index n can be obtained from Eq. (6).

$$n = \log(q_2/q_1)/\log(h_2/h_1) \quad \ldots\ldots\ldots\ldots\ldots (6)$$

where, $q_i = Q(h_i)$, $i = 1, 2$.

Index, n should here be considered to correspond to the mean height of h_1 and h_2.

The I/I distribution indexes were calculated for several sets of two measurements at the same point with different groundwater levels which were selected of the infiltration measurements by the water-filling method described in the previous section, and then the results were indicated in Fig. 4.7. According to this figure, the distribution index n takes a value close to 2 when the groundwater level is lower than lm, and a value close to the broken line in the same figure when the level is higher than lm. This will explain, as indicated in the previous section, that the inpouring rate of I/I varies with the kind of sewer facility and that the inflow rate becomes large as the location of the sewer facility becomes closer to the ground surface.

Some house sewers had no defective portions and the distribution index n in these cases was small. Also in Fig. 4.7, some indexes n were close to 1.

This kind of I/I distribution index may be also averaged for a particular drainage area and, if this index is built in an I/I inpouring model, then the total amount of infiltration within the drainage area may be given as a function of the groundwater level.

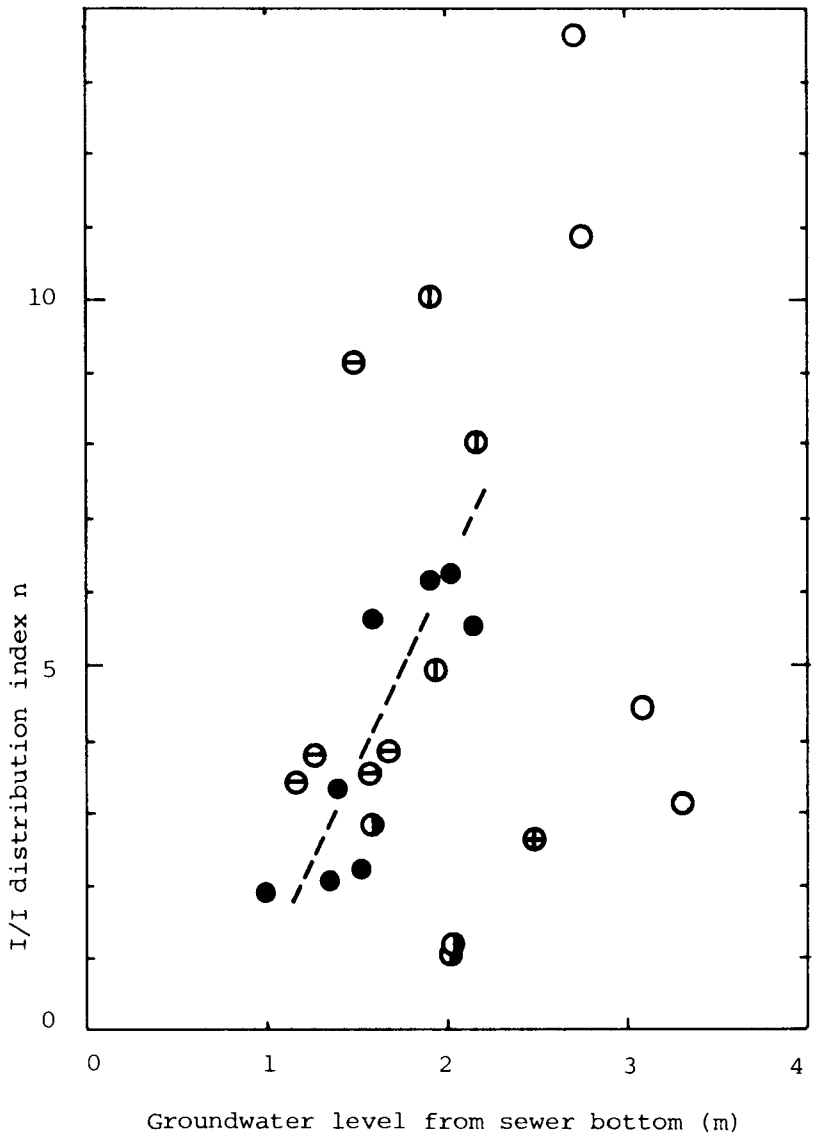


Fig. 4.7 Relation between groundwater level and I/I/distribution index

5. TECHNIQUES FOR I/I REDUCTION IN SEWERS

The I/I flow such as groundwater to the sewers can be prevented either by technical or administrative measures. The administrative measures are performed according to the conditions of I/I and the technical eliminating measures and will therefore not be reviewed in this report.

The technical preventive measures must be taken in each stage of planning, construction, maintenance and management of the sewer facilities. In this chapter, the watertightness of sewer facilities which was examined by the test already performed, and the I/I eliminating techniques which was suggested from the examination of watertightness will be explained.

5.1 Watertightness of Branch Pipe Connections

Watertightness of the connections between main pipes and branch pipes was investigated for the purpose of I/I reduction. The investigation was for the most of connecting methods currently used and developed. In the experiment were used the sewer models, having the main sewer made of Hume concrete pipe and the branch sewer made of two typical kinds of clay pipe and polyvinyl chloride pipe. These pipes were connected together in the same manner as on construction sites, and then the water pressure-resisting strength and tension-resisting strength required for maintaining the watertightness at the connections of branch pipes were measured.

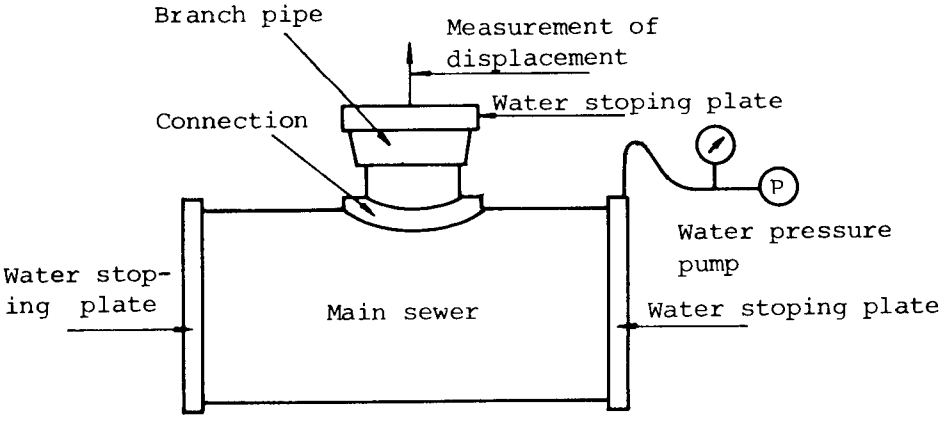


Fig. 5.1 Watertightness testing apparatus

The water pressure-resisting strength was measured in the form of the water pressure at the time when water leaks begin due to pressure at an assembly model of a main pipe and branch pipes filled up with water as shown in Fig. 5.1. Strengths obtained are shown in Fig. 5.2 by comparing the clay pipe with the PVC pipe. Also, the tension-resisting strength was obtained as the load needed for pulling out the branch pipe, and the load at a time of rupture was used as the tensile strength. Strengths obtained are shown in Fig. 5.2 and Fig. 5.3.

In Figs. 5.2 and 5.3, connecting methods used are indicated by symbols. These symbols are listed below.

M : Mortar caulking method

MU : Caulking method using mixture of mortar and urethane resin paste

E : Epoxy resin caulking method (E_1, E_2)

U : Urethane resin caulking method

UG : Urethane resin injecting method

C : Compression joint method (only for clay pipe)

According to Fig. 5.2, the motar caulking method conventionally used provides a low strength for clay pipe and PVC pipe while the caulking method and injecting method using polymer adhesives such as epoxy resin and urethane resin provide a high strength. Caulking method using adhesives made of mortar mixed with urethane resin paste provides a strength higher than that of mortar method for the clay pipes and almost equal to that of mortar method for PVC pipes.

The compression joint method used only for clay pipe is able to provide a water pressure-resisting strength equivalent to that of resin type caulking method when the displacement of branch pipe is forcibly restricted.

Note: Broken line indicates the occurrence of leakage at places other than connecting portions.

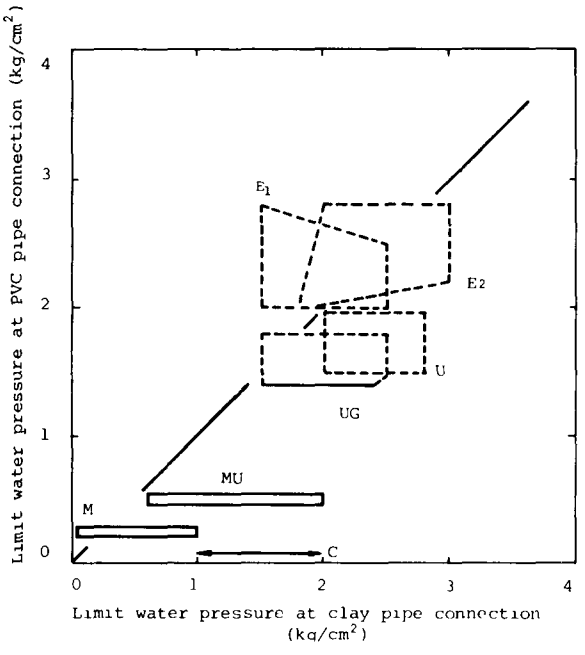


Fig. 5.2 Limit internal water pressure at branch pipe connection

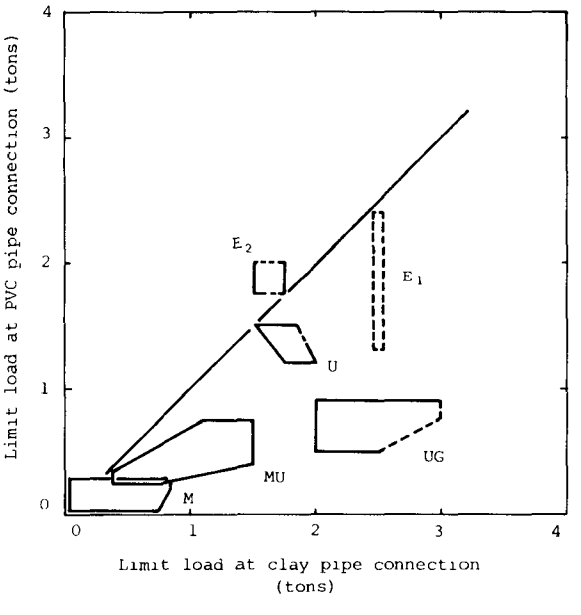


Fig. 5.3 Tensile limit load in branch pipe connection

5.2 Watertightness of Joints in House Sewers

Watertightness of house sewers is concerned with the connecting conditions at joint and connections those of the inlets and the main pipe. At the pipe joints, local failure or connection gap occurs due to differential settlement in surrounding ground, accumulation of repeated load on roads or impact due to earthquake, thereby decreasing the watertightness. Considerable decrease in watertightness occurs also particularly when backfilled earth is insufficiently rolled and compacted. Consequently, the watertightness of house sewer was considered to be mainly related the displacement and then the test was performed as explained below.

When a displacement is given to a main pipe or house sewer, watertightness at the connections will decrease. A testing apparatus shown in Fig. 5.4 applied to simulate the embedding conditions on the site was used, and the displacement required to leak the pressured water from pipe was measured. In this case, the pipe to which a pressure of 1.0 kg/cm^2 was applied prior to the measurement was filled up with water.

Results of the tests conducted for three different kinds of connecting methods are expressed as function of the displacement at loading point at the time of occurrence of water leak and are indicated in Fig. 5.5. On the basis of Fig. 5.5, the watertightness of each joint is considered to have the features as explained below.

When mortar joints were pressured with 1.0 kg/cm^2 applied to the inside of pipe, water leaked out and, even without pressure being applied, joints were broken after a displacement of 1.2 mm. Therefore, its watertightness is considered to be broken even by a slight movement of ground.

Ordinary compression joints are able to maintain their watertightness up to a displacement of 4 to 7 cm. In the case of the improved type of deep socket or socket with spacer, watertightness can be maintained up to a displacements of 15 cm.

Rubber ring socket joints are able to maintain watertightness up to a displacement of 12 to 17 cm. However, this value

of displacement includes the deformation of PVC pipe as house sewer so that these joints themselves are able to withstand displacements almost equal to that of compression joints. Flexible joints or expansion joints as improved types of the rubber ring socket joint will hardly contribute to the improvement of watertightness and are used only for the simplification of piping work.

5.3 On-site Watertightness Tests

In order to confirm the characteristics of connecting methods for branch pipe and house sewer explained in the previous sections, the watertightness of the pipes actually laid on site was tested. On-site watertightness tests were performed by measuring the quantity of leakage based on the water filling method for both the house sewers and branch pipes.

Combinations of branch connections and house sewer joints are shown in Table 5.1. Quantity of leakage at each connection is indicated in Fig. 5.6.

Table 5.1 Combinations of Connections and Joints in On-site Watertightness Test

Main sewer	Branch pipe connection	Branch pipe	Joints for house sewers	House sewers
Hume pipe	Mortar	Hume pipe	Rubber ring socket	PVC pipe
Hume pipe	Mortar & urethane Urethane Epoxy	PVC pipe	Rubber ring socket	PVC pipe
Hume pipe	Mortar Mortar & urethane Urethane Epoxy	Clay pipe	Mortar Mortar & urethane Urethane Epoxy	Clay pipe
Hume pipe	Compression joint	Clay pipe	Compression joint	Clay pipe

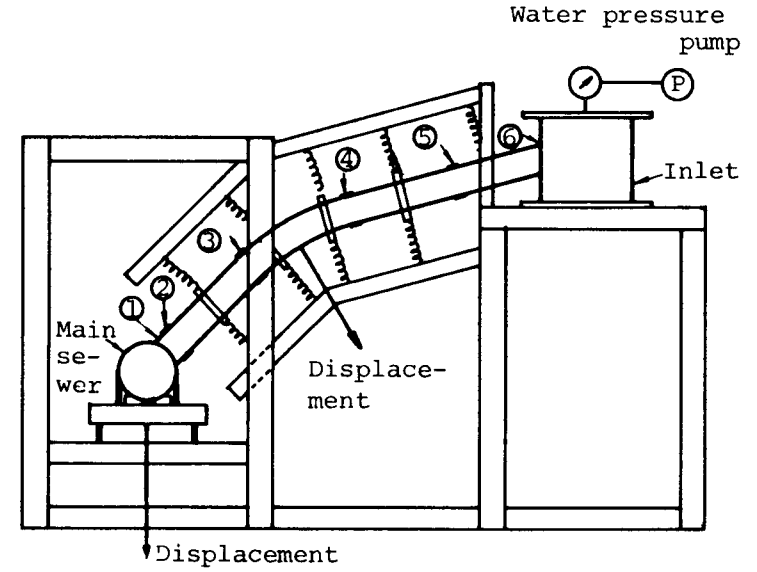


Fig. 5.4 Displacement testing apparatus

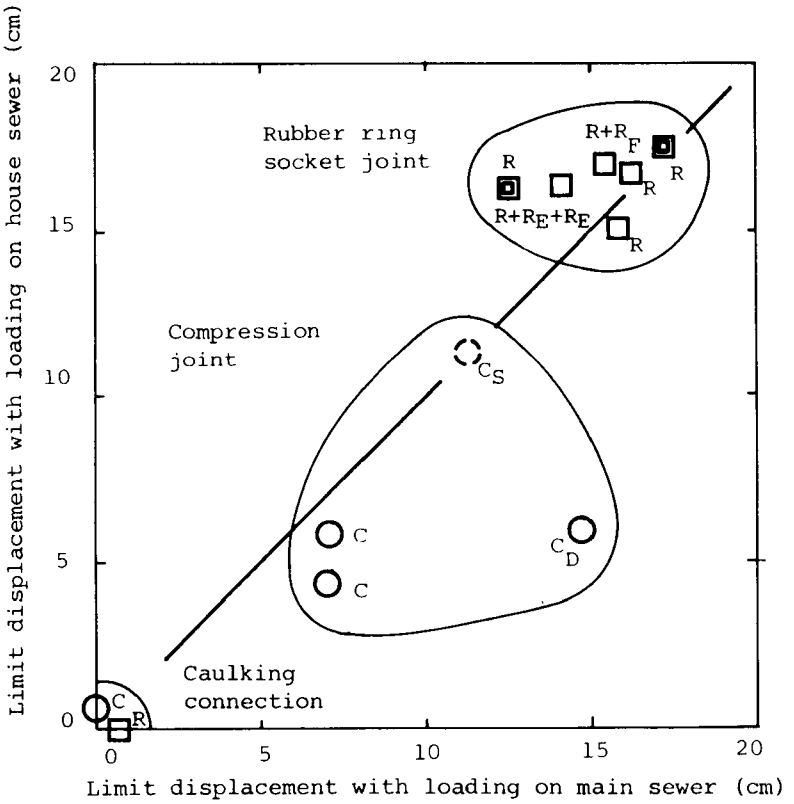


Fig. 5.5 Limit displacement at lateral sewer joint

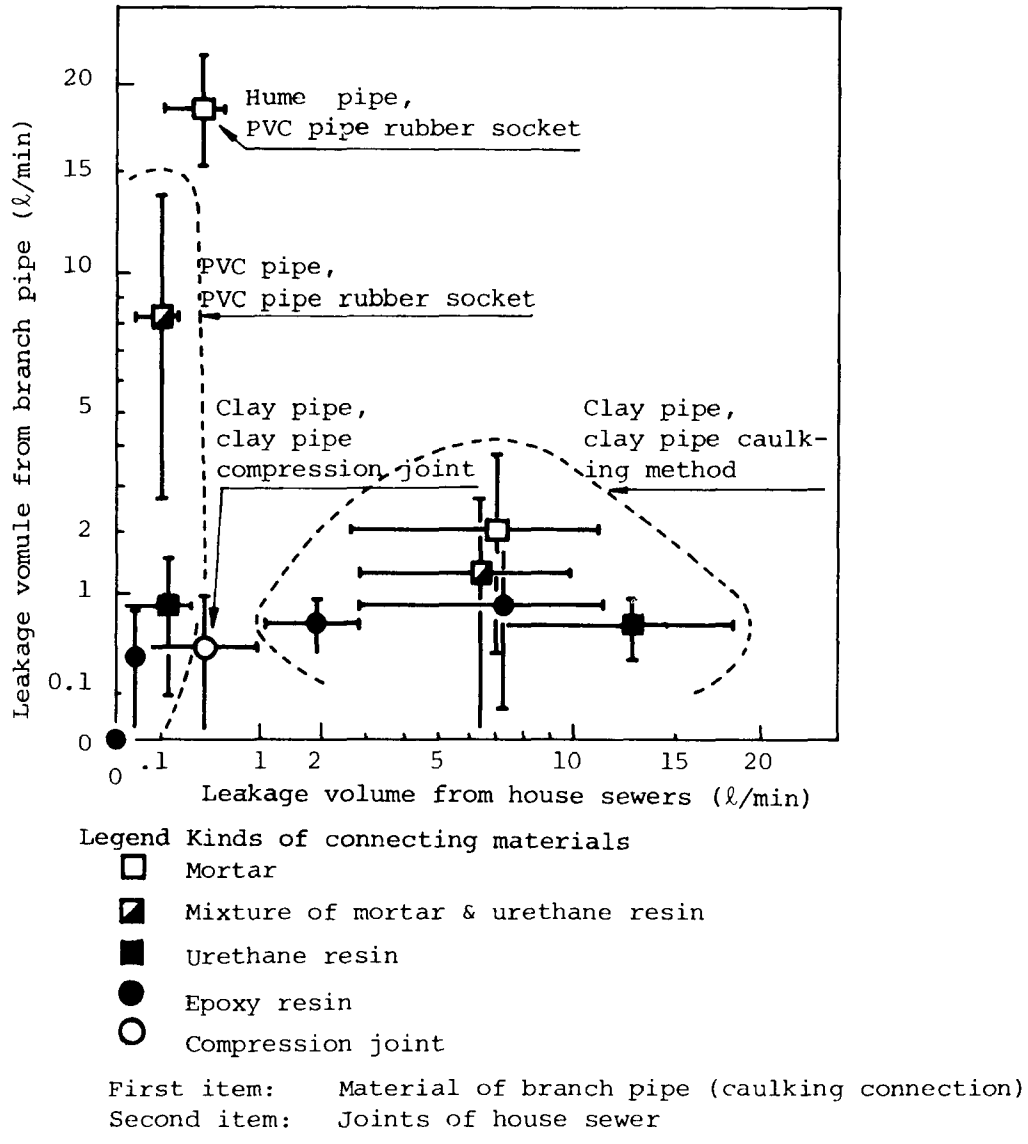


Fig. 5.6 Leakage volume by pipe materials and joints (test by water filling method)

Fig. 5.6 shows the quantities of leakage from house sewer and branch pipe plotted on both axes, from which the combinations of connecting and jointing methods can be classified into four groups relative to the watertightness as listed below.

① A combination of caulking methods used for both the joints of house sewer and connections of branch pipes (clay pipe).

② PVC rubber ring sockets used for house sewers and mortar type adhesive used for connecting branch pipes.

③ Compression joints are used for clay pipes.

④ Branch pipe connections by polymer adhesives and PVC rubber ring socket joints.

Characteristics of watertightness of the above combination of joints and connections will be described below.

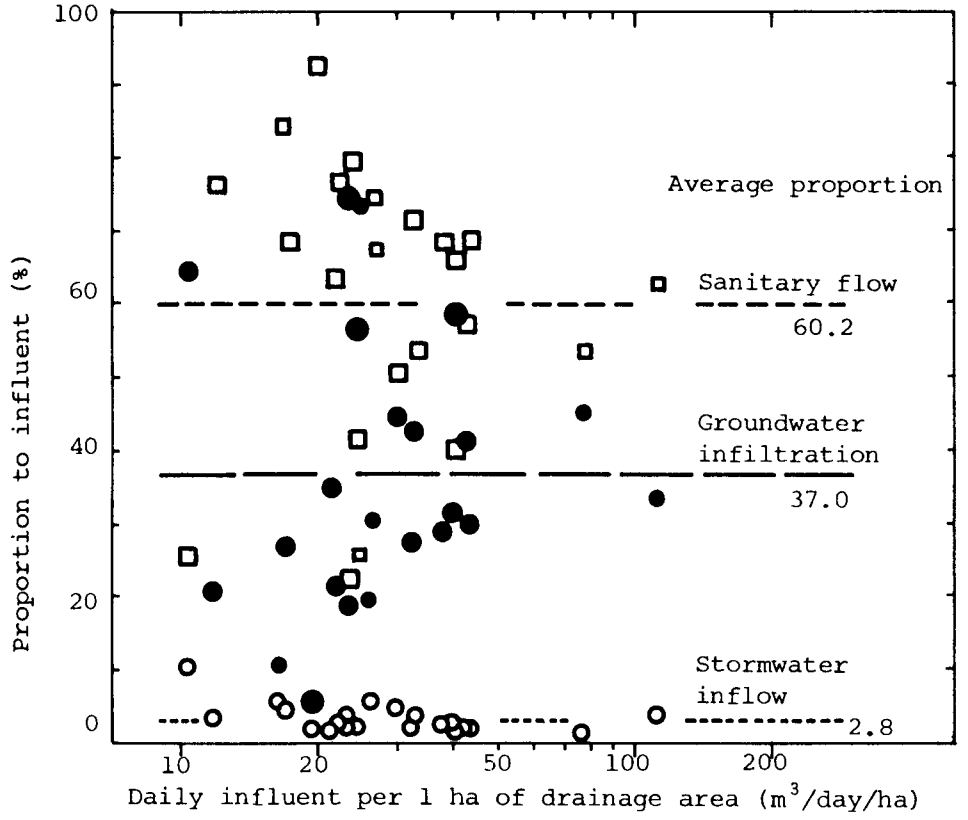
① Connections by compression joints are appropriate when using clay pipes for branch pipes and house sewers. Particularly, when using the caulking method for house sewers accompanied with cumulative displacement, it becomes necessary to install pipes in such a manner that surrounding ground will not move after installation.

② Connection method using rubber ring socket joints and polymer adhesives is suited to the PVC pipes. When the caulking method is applied with bonding agents such as mortar or mixtures of mortar and urethane resin, the watertightness obtained will greatly fluctuate.

③ Among the various kinds of connecting and jointing methods, only a few methods will be able to provide complete watertightness. Realistic standard of watertightness should be about 1 ℓ/min of quantity of leakage (for water-filling method).

5.4 Infiltration/Inflow and its Reduction Techniques

In FY 1979, the proportion of Infiltration/Inflow to the influent was surveyed by a simple method in 26 drainage districts throughout the country. Results of this survey are outlined in Table 5.2. Fig. 5.7 shows the relation between the daily influent per ha. of drainage area and the proportions of sanitary flow, groundwater infiltration, and stormwater inflow.



Small symbols show the proportions based on supplied water volume.

Fig. 5.7 Proportion of infiltration to influent

Table 5.2 Proportion of Infiltration to Influent

Name of drainage district	Area of drainage (ha)	Drainage population ($\times 10^3$)	Length of sewer line (km)	Annual influent ($\times 10^3 m^3$/year)	Proportion to influent			Remarks
					Sewage flow (%)	Groundwater infiltration (%)	Stormwater inflow (%)	
A	1,646	184.1	412.1	24,467	40.1	58.6	1.3	
B	846	97.1	227.4	8,319	(67.5)	(30.6)	(1.9)	
C	702	40.2	172.2	3,046	76.2	20.7	3.1	
D	337	19.8	171.3	3,010	41.7	56.5	1.8	
E	483	62.0	101.9	8,079	(97.9)	(0)	(2.1)	
F	446	44.7	95.0	5,277	71.4	27.2	1.4	
G	511	15.8	70.0	885	(98.2)	(0)	(1.8)	
H	270	9.4	60.8	1,730	22.2	74.7	3.1	
I	302	30.6	59.9	4,660	57.1	41.4	1.5	
J	279	11.3	58.3	2,531	(25.5)	(73.1)	(1.4)	
K	230	28.3	57.3	2,777	53.7	42.9	3.4	
L	131	7.3	47.3	1,126	79.4	18.7	1.9	
M	170	12.5	45.7	1,027	(84.2)	(10.5)	(5.3)	
N	223	9.8	41.1	843	25.5	64.4	10.1	
O	115	17.4	40.7	4,732	(62.6)	(33.7)	(3.7)	Combined system
P	171	7.0	40.3	4,484	(74.8)	(19.7)	(5.5)	
Q	151	13.0	35.6	943	68.5	27.0	4.5	
R	147	21.1	33.4	2,137	66.0	31.6	2.4	
S	129	18.0	29.9	1,780	68.6	29.0	2.4	
T	142	13.9	27.0	1,127	63.3	35.0	1.7	
U	113	15.3	24.4	1,783	68.8	30.0	1.2	
V	110	66.9	22.1	895	76.5	21.5	2.0	
W	59	10.2	12.8	1,164	(96.3)	(0)	(3.7)	
X	21	4.2	4.9	597	(53.6)	(45.2)	(1.2)	
Y	15	0.9	3.6	80	92.7	5.5	1.8	
Z	9	0.8	1.9	72	50.8	44.9	4.3	
Average					60.2	37.0	2.8	

Notes: 1. Values shown in ( ) are based on supplied water volume.

2. Average was obtained by eliminating values based on note 1.

According to Fig. 5.7, the proportion of annual stormwater inflow to the influent is about 3% and the proportion of groundwater infiltration to the influent is about 37%. Though there is no obvious relation between these amounts of infiltration and influent per unit area, it will be known that the range of fluctuation in the proportion of groundwater infiltration decreases as the influent per unit area becomes larger. This probably means that Infiltration/Inflow will flow in sewer systems under a steady state as the sewage flow approaches a certain limit along with population increase within a drainage area.

However, the proportion of groundwater infiltration generally shows no clear correlation with the drainage area, drainage population and length of pipeline. Therefore, it will be more appropriate to consider that the amount of infiltration increases as the number of defective portions in sewer systems becomes larger and that defects in sewer systems occur at random.

The simplified method used in this survey is different from that described in Chapter 2. That is, the amount of stormwater inflow is calculated on the assumption that the hourly fluctuation in daily mean sewage flow is constant through the year. Also, the hourly minimum sewage flow per day is calculated on the assumption that it equal to used as the amount of groundwater infiltration. Accordingly, if a more detailed survey is performed, the estimated proportion of annual groundwater infiltration to the influent is likely to become somewhat smaller.

Infiltration of groundwater into sewer systems is taken into account in the sewer line planning, and about 10 to 20% of daily maximum sanitary flow is used as amount of groundwater. On the other hand, according to the survey of actual conditions, the proportion of groundwater infiltration to the sanitary flow is about 50% as annual average. In this averaged value is included that of the drainage district where the present sanitary flow does not reach the design value. Hence, it becomes necessary to multiply the value obtained by a ratio between the present daily

mean sanitary flow and corresponding design value. At present, since the daily sanitary flow does not reach the design value in most sewage treatment plants, the amount of groundwater infiltration is considered to be less than the expected amount as long as the conversion stated above is performed.

This amount of groundwater infiltration will increase as long as the conventional method of sewer piping work is performed. Also, if the amount of infiltration increases, the flowing capacity of sewer system may become insufficient, quality of treated water may worsen, or the treatment and maintenance costs may increase due to overloading of sewage treatment plants. These undesirable conditions may also create serious technical administrative and financial problems.

From the results of tests obtained up to now, the following techniques can be considered for the prevention or reduction of infiltration/inflow:

① Improvement of ground around sewer system.

② Improvement of durability and strength of materials of sewer pipes.

③ Improvement of watertightness of connections and joints of sewer pipes.

④ Improvement of expansibility of joints of house sewers.

⑤ Prevention of misconnections to sanitary sewage inlets.

Though these prevention techniques are known, previous targets of improvement were not quantitative and the degree of influence upon the reduction of infiltration was not clear. As a result of the present survey, the inflow mechanism of the Infiltration/Inflow and the general relation between the kinds of sewers and the amount of the infiltration have been found, so that proper goals of research and development of the prevention techniques can now be more easily determined. Important improvement techniques will be briefly explained below.

Though the improvement of ground around sewer systems was generally considered for the back-filled earth, low permeability as well as high bearing capacity are needed in areas where the groundwater level is high.

Currently, Hume concrete pipes, clay pipes and polyvinyl chloride pipes are mainly used for sewers with small and medium diameters. Strength of these materials is mainly discussed from a different viewpoint such as workability and economy but the present strength seems to be sufficient for the prevention of. Infiltration/Inflow.

The mortar caulking method used for connecting sewer pipes should not be used when the displacement of sewer system or its surrounding ground is likely to occur. A caulking connection method using polymer adhesives currently developed can provide effective watertightness for the PVC pipes.

Clay pipes themselves cannot respond properly to displacement and thus joints of house sewers should have an expandable structure so as to allow the whole sewer system to properly respond to displacement. The compression joint method currently developed enables both the watertightness and proper response to the displacement to be obtained.

In addition, the amount of infiltration can be reduced also by the administrative guidance but this has not been explained here.

REFERENCES Quantification Regression Analysis (Hayashi's Model)

The purpose of quantification analysis is to explain a certain dependent variate Y (external criterion) by using explanatory factors having r-kinds of qualitative characteristics which are considered to be affecting the fluctuation of dependent variate. The basic formula required for calculation will be described below.

Qualitative explanatory factors will be respectively classified into the categories of P_1, P_2, ... p_r kinds, and the reaction of sample with each category will be expressed by a dummy variable. If a sample ν is considered, the dummy variable can be defined by Eq. (1).

$$X_{j\nu}^{(i)} = \begin{cases} 1 : & \text{when sample } \nu \text{ reacts with } j \text{ category of } i \text{ factor.} \\ 0 : & \text{When sample } \nu \text{ does not react with } j \text{ category of } i \text{ factor.} \end{cases} \quad \ldots(1)$$

Each sample ν must react with one of categories in a certain factor. Therefore, such a relation may be represented by Eq. (2).

$$\sum_{j=1}^{P_i} X_{j\nu}^{(i)} = 1 \qquad i = 1,2,\ldots, \text{ r factor} \quad \ldots\ldots (2)$$

$$\nu = 1,2,\ldots, \text{ n sample}$$

It is assumed that the following linear relation will occur between dependent variate Y and dummy variable:

$$\begin{aligned} Y_\nu = {} & \beta_1^{(1)} X_{1\nu}^{(1)} + \ldots\ldots + \beta_{p_1}^{(1)} X_{p_1\nu}^{(1)} \\ & + \beta_1^{(2)} X_{1\nu}^{(2)} + \ldots\ldots + \beta_{p_2}^{(2)} X_{p_2\nu}^{(2)} \\ & \ldots\ldots \\ & + \beta_1^{(r)} X_{1\nu}^{(r)} + \ldots\ldots + \beta_{p_r}^{(r)} X_{p_r\nu}^{(r)} \\ & + e_\nu \; ; \; \nu = 1, 2, \ldots, n \quad \ldots\ldots\ldots\ldots\ldots (3) \end{aligned}$$

where, e_ν is a disturbance term or a random variable.
Then, Eg.(3) is expressed as follows in matrix notation:

$$\underset{\sim}{Y} = \mathbf{X}\underset{\sim}{\beta} + \underset{\sim}{e} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4)$$

Then, the following basic assumption is made for $\underset{\sim}{e}$:

$$\mathbf{E}(\underset{\sim}{e}) = \underset{\sim}{0} \qquad \text{(expected value of } e_\nu \text{ is 0)} \ \ldots\ldots (5)$$

$$\mathbf{E}(\underset{\sim}{e}\,\underset{\sim}{e}') = \varepsilon \mathbf{I}_n \ \text{(variance of } e_\nu \text{ is constant } \varepsilon, \text{ covariance is 0)} \ \ldots\ldots\ldots\ldots (6)$$

$\mathbf{X}$ is a set of fixed numbers $\ldots\ldots\ldots\ldots\ldots\ldots$ (7)

$$\text{Rank of } \mathbf{X} = p - (r - 1) < n, \quad p = \sum_{i=1}^{r} p_i \ \ldots\ldots\ldots\ldots (8)$$

If the estimate vector of $\underset{\sim}{\beta}$ to be determined is expressed by $\hat{\underset{\sim}{\beta}}$, then the following formula will hold:

$$Y = \mathbf{X}\,\hat{\underset{\sim}{\beta}} + \hat{\underset{\sim}{e}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (9)$$

The "Method of least squares" should be applied to Eq. (9) and $\hat{\underset{\sim}{\beta}}$ should be determined in such a manner that sum of residual squares $\hat{\underset{\sim}{e}}'\hat{\underset{\sim}{e}}$ will become the smallest. Then, this $\hat{\underset{\sim}{\beta}}$ is the coefficient related to each category. If $e_\nu = 0$ is applied in Eq. (3), then the dependent variable Y_ν is obtained as a linear sum of the explanatory factors.

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THE PAST IS PROLOGUE: LOOKING BACK, LOOKING AHEAD AT THE UNITED STATES ENVIRONMENTAL PROTECTION AGENCY'S CONSTRUCTION GRANTS PROGRAM

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THE PAST IS PROLOGUE:
LOOKING BACK: LOOKING AHEAD
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ABSTRACT

The Federal Water Pollution Control Act of 1972 created the multibillion-dollar construction grants program as part of a massive Federal assault on the country's widespread water degradation caused by decades of unchecked municipal and industrial water pollution. The program would attack municipal pollution by helping communities pay for construction of municipal wastewater treatment facilities. Industries were to build treatment facilities with their own resources.

The 1972 legislation's premise was that no one has the right to pollute. It aimed to achieve fishable and swimmable waters by 1983 where possible and a total elimination of all pollutant discharges into navigable waters by 1985. Its stated goal was no less than "... to restore and maintain the chemical, physical, and biological integrity of the Nation's waters."

To achieve its goals, the Act set a minimum national effluent standard for all municipalities and industries that discharged into navigable waters and demanded a technology known as secondary treatment or its equivalent to meet the standard. Where State water quality standards were higher than the minimum national standard, dischargers had to meet the more stringent State standards. To enforce the national and State standards, the Act called for a national permit program that required all who discharged significant amounts of wastewater into navigable waters to get a Federal permit. The Act set strict enforcement procedures that called for heavy fines or possible imprisonment for willful or repeated violators.

To help municipalities meet the new requirements, the 1972 Act authorized $18 billion for the construction grants program to help pay for treatment facilities. The Federal Government paid 75 percent of the project's cost and the community, usually with State help, paid the other 25 percent. Total appropriations for the program through October 1984 amount to over $40 billion.

This paper traces the evolution of Federal water pollution control from the turn of the century to the landmark 1972 legislation and the construction grants program. It describes the growth of the program, its redirection toward the States with the 1977 amendments, and the further acceleration of this direction with the 1981 amendments. The paper recounts the successes and the problems of the construction grants program and looks at its probable future direction as the Federal role phases into one of overview and the States assume management of their own programs.

THE PAST IS PROLOGUE: LOOKING BACK, LOOKING AHEAD AT THE UNITED STATES ENVIRONMENTAL PROTECTION AGENCY'S CONSTRUCTION GRANTS PROGRAM

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INTRODUCTION

Inscribed near the portico of the National Archives building, the repository of my country's historic documents in Washington, D.C., are the words, "The Past Is Prologue." Those words seem almost personally tailored to my subject today--the evolution of the water pollution control programs in the United States.

A little over 10 years ago, the United States Environmental Protection Agency (EPA) submitted its first annual report to Congress on the 1972 Clean Water Act. That Act was the most far-reaching water pollution control legislation in United States history. Its official title was the Federal Water Pollution Control Act Amendments of 1972. It aimed to eliminate all pollutant discharges into navigable waters by 1985 and to make waters fishable and swimmable by 1983. Its stated goal was no less than "... to restore and maintain the chemical, physical, and biological integrity of the Nation's waters." EPA's report to Congress was dated May 31, 1973, just 7 months after the Act went into effect on October 18, 1972.

Today I would like to give you an overview of my country's efforts to clean up its waters, before and since the landmark 1972 legislation. I shall especially focus on the construction grants program, which I administer.

The 1972 legislation created the multibillion-dollar construction grants program to help States and municipalities finance construction of wastewater treatment facilities. The program would be the most visible part of the massive Federal and State assault on the country's widespread

water degradation, the result of centuries of unchecked industrial and municipal pollution.

My review will highlight four areas in the program's evolution: the changing Federal-State role; the technology vs. the instream-water-quality approach emphasized by various legislation; enforcement provisions; and financing.

That we are all here at this conference sharing information on our successes as well as our problems in water pollution control testifies to the continuing dedication to environmental protection, not just in my country and Japan, but in all countries represented here today.

HISTORICAL BACKGROUND

In the United States, this present widespread public support for environmental legislation was slow in coming. All but six Congresses since 1897 considered Federal participation in water pollution abatement. More than 100 bills were introduced in the Congress, but none passed.

BEFORE 1948

The first specific Federal water pollution control legislation was the Rivers and Harbors Appropriations Act of 1899, often called the Refuse Act. This legislation's goal was not basically to control pollution but to prevent obstructions to navigation by prohibiting dumping of solids into navigable waters without a permit from the U.S. Army Corps of Engineers. Later, the 1912 Public Health Service Act authorized investigation of water pollution related to diseases. The 1924 Oil Pollution Act attempted to control oil discharges in coastal waters to protect aquatic life, harbors and docks, and recreation facilities. There were attempts in 1936, 1938, and 1940 to pass comprehensive national legislation to protect water quality but each failed.

FEDERAL WATER POLLUTION CONTROL ACT OF 1948

It was not until 1948 that the first comprehensive national legislation aimed specifically at controlling industrial and municipal water pollution was enacted. This was the Federal Water Pollution Control Act of 1948 (P.L. 80-845).

The Act authorized $22.5 million for each of 5 years from 1948 to 1953 for 2 percent loans to States, municipalities, and interstate agencies to build wastewater treatment works that would control pollutant discharges into interstate waters and tributaries. Loans could be $250,000 or one-third the cost of construction, whichever was less. The Act also authorized Federal grants of $5 million over the same 5 years to be awarded to States and municipalities to help plan and design wastewater treatment facilities. In addition, $5 million, at $1 million each year, was authorized for grants to States to study ways to prevent pollution from industrial wastes.

While the 1948 Act provided the first comprehensive Federal, State, and local cooperative approach to water pollution, it was more important in concept than in effect. It resulted in only a trickle of money to study industrial pollution and to help plan and design treatment works.

The Act recognized and preserved the primary rights and responsibilities of the States to control water pollution. There were no Federally required goals and no effective enforcement. The Federal Government could sue violators only with the consent of the State in which the violation occurred.

In prophetic words the Senate report of July 8, 1947, warned that "... failure to accomplish adequate progress through cooperative efforts of Federal and State agencies will undoubtedly call for much stronger and more direct Federal enforcement measures at some subsequent session of Congress."

WATER POLLUTION CONTROL ACT EXTENSION OF 1952

In 1952 Congress extended the Act for 3 years, from 1954 through 1956, with P.L. 82-579. But, again, the Act had little real national impact.

FEDERAL WATER POLLUTION CONTROL ACT OF 1956

The first major legislation to reflect a significant Federal financial commitment to a national pollution control program came 4 years later, with the Federal Water Pollution Control Act of 1956 (P.L. 84-660). This Act is often referred to as the "Old Law."

The 1956 legislation called for increased cooperation between the Federal Government and the States to develop a broader national effort against water pollution and permitted Federal participation in a wide range of activities. They included Federal-State cooperation in developing programs, increased technical assistance to States, and broadened research.

The law authorized a 10-year grant program at $50 million a year from 1956 to 1966 to help communities finance construction of wastewater treatment facilities. Grants were limited to 30 percent of the total project cost with a maximum of $250,000. This dollar ceiling, in effect, limited grants to small projects.

Federal intervention in enforcement was still limited to cases involving interstate waters, and enforcement still involved joint Federal and State efforts.

As pressures of population and economic growth upon the country's natural resources continued to multiply, it was obvious that authorized Federal funding fell far short of needs.

FEDERAL WATER POLLUTION CONTROL ACT AMENDMENTS OF 1961

These pressures and the public's rising interest in pollution control were reflected in the 1961 amendments to the Federal Water Pollution Control Act (P.L. 87-88). This Act increased authorizations and the Federal grant share and strengthened Federal enforcement for interstate waters.

Federal grants remained at 30 percent of the project's cost but Federal support could now be as high as $600,000. Where communities joined to build one project, Federal grants could go as high as $2.4 million. Authorizations increased from $80 million for fiscal year 1962 to $90 million for fiscal 1963 and $100 million a year each for fiscal years 1964 to 1967.

The law extended Federal enforcement authority to coastal waters and navigable intrastate waters under certain restricted conditions but still required State permission before Federal enforcement. In an important extension of Federal authority, the law permitted Federal suits against polluters of interstate waters without the State's permission.

WATER QUALITY ACT OF 1965

Water quality became a predominant value with the second major amendment to the Water Pollution Control Act, the Water Quality Act of 1965 (P.L. 89-234).

The 1965 Act directed that States develop water quality standards for interstate navigable waters. A water quality standard is a legal expression of the amount of pollutants allowed in a particular water body. The standards are based on water quality criteria that provide a level of water quality needed for a range of designated uses. The Federal government develops the water quality criteria which States include in their standards. The States decide the water body's designated use. The State standards were subject to Federal approval. The Act created the Federal Water Pollution Control Administration to administer the national program.

The Act increased annual authorizations for constructing wastewater treatment works to $150 million for fiscal years 1966 and 1967. It also provided grants for research and development in better ways to control overflow from storm sewers and combined storm-sanitary sewers.

Enforcement continued to be difficult. The Act continued the basic instream water-quality approach to pollution control. Federal enforcement applied only to interstate waters and the burden of proof rested with the enforcing agency. An enforcement action had to prove the waste discharge reduced the quality of receiving waters below the established instream water quality standards or that the discharge endangered health and welfare. The statute also directed that the courts consider the cost of cleanup. These and other limitations led to a widespread perception that more stringent standards and enforcement procedures were needed.

CLEAN WATER RESTORATION ACT OF 1966

A year later Congress authorized a massive expansion in the Federal financial commitment to the construction of wastewater treatment facilities with the Clean Water Restoration Act of 1966 (P.L. 89-753). The legislation authorized a Federal expenditure of $3.5 billion over a 4-year period from fiscal years 1967 to 1971. The Federal share increased from 30 percent to a maximum of 55 percent. While this represented an increased Federal commitment, the actual funds appropriated by Congress for fiscal years 1967 to 1971 were little more than half the authorized amounts.

ENVIRONMENTAL ACTION AND EPA

The slow pace of water cleanup led to mounting public frustration and contrasted sharply with technological breakthroughs in other areas.

In 1969, two seemingly unrelated events captured national attention. On June 22, the Cuyahoga River in the State of Ohio caught fire. Less than a month later, on July 20, Neil Armstrong walked on the moon.

The country's mood then was that man's reach no longer exceeded his grasp. Technology had stretched his arm to the moon. So technology could certainly clean up the environment here on Earth, given the will and the money. And money was no problem in 1969. The national budget showed a $3.3 billion surplus.

Environmental concerns were everywhere in 1969 and they were rapidly translated into action in the following years. Congress passed the National Environmental Policy Act of 1969 (NEPA) requiring assessment of environmental impacts before major Federal actions were initiated. The growing power of the environmental movement came into sharp focus on April 20, 1970, with Earth Day, a nationally organized outpouring of ecological fervor across the country. On July 9 President Nixon created the Environmental Protection Agency by sending to Congress Reorganization Plan Number 3. The Plan transferred 15 government units with their functions and legal authority to the new agency effective December 2, 1970. The Agency saw its first administrator, William D. Ruckelshaus, confirmed by Congress the next day.

WATER QUALITY IMPROVEMENT ACT OF 1970

The 1970 Water Quality Improvement Act (P.L. 91-224), passed in April of that year, repealed the 1924 Oil Pollution Act and added strong oil control provisions to the Federal Water Pollution Control Act. The 1970 Act also added significant innovations to the basic law, such as providing for demonstration projects for Great Lakes cleanup and manpower training programs to increase the number of water pollution control personnel. But Congress and the Administration saw that the public wanted a still stronger and more visible Federal commitment to clean up the Nation's waters.

1970 PERMITS PROGRAM

To get started on immediate cleanup, the newborn EPA and the U.S. Army Corps of Engineers launched a national permit program December 23, 1970, claiming authority under the 1899 Refuse Act. The new program required all industries that discharged into navigable waters to get Federal permits. Unlike the Federal Water Pollution Control Act, which used the water quality standards-enforcement approach to pollution control, the Refuse Act Permit Program used an effluent limitation approach. This meant restricting pollutants at the point of effluent discharge. The Refuse Act also applied to intrastate as well as interstate navigable waters. Though a year later an Ohio Federal judge struck down the permit program, EPA staff continued to check and verify information on the 23,000 permit applications received. They hoped to keep the program going until new legislation could be passed.

Meantime, extensive hearings convinced Congress that the then current legislative approach was not working. The consensus was that the country needed a law with strong teeth and that the teeth should be a permit program and national technology-based standards and effluent controls.

FEDERAL WATER POLLUTION CONTROL ACT AMENDMENTS OF 1972

The result was the Federal Water Pollution Control Act Amendments of 1972 (P.L. 92-500), a sweeping rewrite of all existing water pollution control legislation. The Act's premise was that enforceable national technology based effluent controls were needed in addition to the previous emphasis on achieving instream water quality. It set ambitious new goals and authorized massive Federal funding to achieve them.

Goals

The complex 89-page law, enacted October 18 over Presidential veto, required minimum technology based effluent standards for industrial and municipal wastes. These were described as "best practicable technology" for industrial wastes and secondary treatment as the equivalent for municipal wastes. All dischargers were to achieve these levels of treatment by July 1, 1977, to meet the Act's goals: fishable, swimmable waters by 1983, where achievable, and a total elimination of pollutant discharges into navigable waters by 1985.

NPDES Permit Program

The Act extended Federal enforcement powers to intrastate as well as interstate waters and called for a National Pollutant Discharge Eliminaton System (NPDES) permit program to force dischargers to meet the national goals. The NPDES permit program incorporated many features of the Refuse Act permit program. Concentrating first on the most significant sources of pollution, EPA established a policy that by December 31, 1974, permits would be awarded to each of the approximately 5,000 most significant industrial and municipal facilities that discharged directly into navigable

waters. Each permit set specific limits on the amount and kind of each pollutant discharged and established an enforceable schedule for compliance. Both civil and criminal penalties were provided. Violators could be fined up to $25,000 a day and imprisoned up to a year with increased penalties for repeated violations.

Treatment Levels

In addition to the 1977 treatment requirements, direct industrial dischargers were to have in place "best available technology" (BAT) by July 1983, where economically achievable. EPA was ordered to publish a list of toxic pollutants and effluent standards for these pollutants 6 months after it published the list. Industry was to comply with effluent standards for 65 toxic pollutants within a year after EPA published the standards. Industries that discharged to publicly owned treatment works (POTWs) were to install systems to pretreat waste that could harm the POTW or pass through to pollute receiving waters and sludges.

For municipalities, EPA defined secondary treatment as 85 percent removal of biochemical oxygen demand (BOD) and suspended solids or 30 milligrams per liter for BOD and 30 milligrams per liter for suspended solids. In addition, pH was limited at 6.0 to 9.0 and fecal coliform at 200 per 100 milliliters. This definition meant treatment facilities funded since the 1956 "Old Law" and not designed for secondary treatment would need to be upgraded.

Although the new law shifted emphasis to enforcement through technology based effluent standards, it also retained requirements for achieving instream water quality based on State water quality standards. Where needed to meet water quality standards, industrial and municipal dischargers were required to apply more stringent effluent controls.

The Act demanded a radical turnaround in the way industry, States, and municipalities dealt with water pollution. Congress felt that industries could raise money to accomplish cleanup on their own but that muncipalities could not.

Construction Grants Program

To help communities meet the stringent new treatment requirements, the Act created the construction grants program to help municipalities finance construction of needed wastewater treatment facilities.

The Act made available a staggering $18 billion to fund the program for 3 years from fiscal years 1973 through 1975: $5 billion for 1973; $6 billion for 1974; and $7 billion for 1975.

The money was to be used for 75 percent Federal grants to help municipalities pay for new treatment facilities or upgrade facilities already operating. The municipality, usually with State help, would pay

the other 25 percent. The grants were awarded in three different steps: Step 1 for facility planning; step 2 for design; and step 3 for actual construction.

Almost every component of a wastewater treatment system qualified for funding under the definition of "treatment works." Besides the treatment plant itself, it included major sewer rehabilitation, storm sewers, interceptor and collector sewers, control of infiltration/inflow and combined sewer overflow, and purchase of land for land treatment systems.

The grant money was allocated to States according to an allotment formula specified by Congress. Allocations depended on the State's total needs as compared to national needs. To determine what these needs were, Congress directed that EPA submit a needs survey to Congress every 2 years. Each State had to develop a priority system and draw up a priority list that set the order for project funding. EPA had to approve the priority system and review the list.

The Act also provided the first direct Federal funding for State waste treatment management planning on an areawide basis and authorized major research and development to develop new technology to eliminate pollutant discharges into navigable waters and contiguous zones and oceans. To supply the technical expertise the new treatment facilities would need, the Act provided funds to help States build manpower training centers and to develop training programs and materials through State and academic institutions.

Problems

Funding--

The $18 billion made available over 3 years for the construction grants program in 1972 was thought to be enough to solve the major water pollution problems as they were then defined. But problems surfaced immediately.

President Nixon impounded $9 billion of the available funds as soon as the Act passed because he felt the funds were excessive and could not be effectively spent. Funding in the ensuing years remained unstable and unpredictable. Actual funding for fiscal year 1973 was $2 billion; for 1974, $3 billion; and for 1975, $4 billion. The impounded $9 billion was released in 1976 on a ruling of the United States Supreme Court.

Federal/State Role--

Initially the new program incurred resentment in some State and local governments who felt that their authority decreased as their workload increased. Besides dealing with a complex new law and regulations and a complicated new grants process, States were assigned numerous new tasks.

While the law provided grant funds to assist States financially with their new duties, these and other problems strained the resources of EPA and State and local governments.

Permits--

Though EPA had 23,000 industrial permits almost ready to issue as a result of its 1970 permit program, these were only part of the 60,000 permits that needed to be issued to municipal, industrial, and other dischargers. The law encouraged EPA to delegate permitting and enforcement authority to States that could handle the task. But all States, delegated or not, had to review and certify each permit for consistency with their water quality standards.

Water Quality Standards--

In addition, States had to revise their water quality standards to meet the more stringent requirements of the 1972 Act. The 1965 Act required that States develop water quality standards for interstate waters. The 1972 Act required that these standards be revised and that Federal enforcement authority apply to intrastate as well as interstate waters.

More stringent water quality standards and effluent requirements also often imposed financial hardships on communities who were required to build costly central treatment systems which they determined they could not afford to operate and maintain, much less build.

Secondary Treatment Controversial--

The uniform secondary treatment requirements drew immediate criticism from many dischargers who felt the standard was too rigidly defined. The stringent effluent requirements came under attack by several West Coast communities who argued that secondary treatment was unnecessary for effluent discharged into deep ocean waters where wastes rapidly aerate and disperse and where many fresh-water micro-organisms cannot survive. These communities pressured Congress and EPA to grant waivers from secondary treatment to qualifying ocean-discharging communities.

EPA amended the secondary treatment definition in 1976 and 1977. It deleted the fecal coliform limitation, clarified the pH requirement, and relaxed the suspended solids requirement for ponds that treated less than 2 million gallons of wastewater a day. This left secondary treatment defined as 30 milligrams per liter for biochemical oxygen demand (BOD) and 30 milligrams per liter for suspended solids (SS) or 85 percent removal of BOD and SS on a monthly average.

Compliance--

As the 1977 secondary treatment deadline approached, it became obvious it would not be met. Needs estimated in 1973 at $60.7 billion by 1976 totaled $96 billion. These needs included total system capital costs and allowed for future population growth of approximately 20 years. A

factor contributing to higher costs was that some States set stringent water quality standards which meant higher and more expensive levels of treatment than secondary. Another problem was that EPA failed to publish the required list of toxic pollutants or effluent standards to control them due to a lack of sound scientific information on toxics and limited EPA resources.

Industry outpaced municipalities in meeting the July 1, 1977, best practical technology (BPT) deadline. By that time 81 percent of the most significant industrial dischargers issued permits by the end of 1974 had BPT in place but only 30 percent of municipalities had achieved secondary treatment. Two major factors accounted for this difference in compliance. First, municipalities were not required to be in compliance if they were on schedule to achieve secondary treatment and had received an EPA construction grant or were on the State's priority system to receive a grant. Second, most of the Agency's permitting and compliance efforts were devoted to industrial dischargers.

CLEAN WATER ACT OF 1977

Congress saw the need for midcourse corrections and responded with the 1977 amendments to the Clean Water Act (P.L. 95-217) of December 27, 1977.

The 1977 amendments significantly changed the direction of the construction grants program. They encouraged delegation of the program to the States and authorized delegated States to use construction grants funds to manage their programs. They also provided financial relief for small communities; encouraged innovative and alternative technologies and conservation, recycling, and reuse of wastewater constituents; directed that communities choose the most cost effective system to solve their wastewater problems; and extended the secondary treatment compliance deadline.

State Delegation

The 1977 amendments authorized States to take over major portions of the construction grants program through delegation agreements. Delegated States could use 2 percent or a minimum of $400,000 of their annual construction grant allotment to manage their programs. But not all activities could be delegated. EPA still retained statutory authority to make grant awards, resolve audits and appeals, make judgments on National Environmental Policy Act requirements, and, by order of Congress, review high-cost advanced treatment projects.

Extended Deadlines

The Act extended the July 1, 1977, secondary treatment deadline to July 1, 1983, for most noncomplying communities and extended the best practicable technology (BPT) deadline to April 1, 1979, for industrial dischargers. The 1983 best available technology (BAT) requirements for industry were expanded to include toxic pollutants.

Rural States

To help alleviate adverse economic impacts on small communities, the 1977 amendments directed that rural States set aside 4 percent of their allotments for alternative systems in communities of 3,500 or less and allowed these projects to get 85 percent Federal grants. An alternative technology is one that has proven its worth in actual use. These systems include septic tanks and upgraded variations for individual residences or clusters of residences, pressure and vacuum sewers, small-diameter gravity sewers, and many others.

Innovative and Alternative Technology

To force new technology and reduce costs, all States had to set aside a percentage of their annual allotments for innovative or alternative (I/A) technologies. Set-aside percentages were 2 percent in fiscal years 1979 and 1980 and 3 percent in fiscal year 1981. An innovative technology is defined as one that has not yet proven its worth in the anticipated use. I/A portions of projects received 85 percent Federal grants. Provision was also made to reimburse communities for failed innovative approaches.

Waivers for Ocean-discharging Communities

Congress also allowed EPA to grant waivers from secondary treatment requirements to qualifying communities that discharged to the ocean and that applied for waivers by September 13, 1979.

Funding

Congress authorized a total of $25.5 billion for the construction grants program for fiscal years 1978 through 1982. Actual appropriations for fiscal year 1977 through fiscal year 1979, however, were $10.1 billion.

Progress

The construction grants program made impressive progress despite the massive effort it took to get the giant program into operation with such uneven funding. By 1980 the program had matured to a high degree of operational stability. The Federal and State governments had grown accustomed to working together. States understood the program's complexities and many were on their way to full delegation. A national pool of trained State personnel had been created. EPA enhanced State personnel resources by drawing upon the expertise of the Army Corps of Engineers through an agreement in 1978. Because of the Corps' expert knowledge of construction practices and cost control, it was brought into the program to help States and EPA manage construction projects and to help prevent waste, fraud, and mismanagement. As municipal and industrial treatment facilities began to operate around the country, observable water quality benefits appeared. Rivers and lakes that once spawned masses of algae and dead fish now began to support fishing and recreation activities.

Problems

At the same time, however, recurring problems plagued the grants program. Ten years after the 1972 Act and with over $30 billion actually committed to specific projects, the Act's goals were still unmet. Annual Federal funding levels remained unpredictable. Increased costs consumed construction dollars as projects plodded through the three-step grant process. Some projects that took 2 years to 3 years to complete under the 1956 "Old Law" now dragged on for 7 years to 9 years. Through the years money was stretched too thin as funding eligibilities expanded and project completion dates were extended for lack of funds or to allow construction of other projects. As populations exceeded or fell short of planning projections, some plants were either overloaded or underused.
There was also a growing perception that the high levels of Federal funding and inadequate enforcement fostered pubic indifference to construction costs and to effective plant operations and maintenance.

It became evident, too, that secondary treatment was still too rigidly defined and for certain projects not worth the marginal improvement in water quality it produced.

Advanced treatment--

An added problem was that some States set unattainable uses for certain water body segments that then demanded stringent water quality standards. This meant effluent discharged into these segments needed more advanced and more costly treatment than secondary. This occasionally led to construction of expensive advanced treatment projects whose effluent had little overall impact on receiving water quality but a severe impact on the financial resources of the community supporting the project. Concern over advanced treatment costs led Congress in 1979 to require that the EPA administrator personally review every proposed advanced treatment project that exceeded the costs for secondary treatment by $1 million or more. Congress raised this figure to $3 million for fiscal year 1980 because the reviews strained EPA headquarters personnel resources and caused project delays that increased project cost. Though advanced treatment reviews revealed that many proposed projects were badly needed, some were not.

FEDERAL WATER POLLUTION CONTROL ACT AMENDMENTS OF 1980

Congress had made limited changes in the law with P.L. 96-483 enacted October 21, 1980. This legislation allowed a State governor to reduce new Federal grants to 55 percent or less uniformly across the State. It also extended expiring funding for the I/A program, then just beginning to produce results.

But the new Administration in 1981 felt the program was too complex and too costly in view of national budget demands. It also felt the States needed more flexibility and authority to decide how they would meet the Act's requirements.

MUNICIPAL WASTEWATER CONSTRUCTION GRANT AMENDMENTS OF 1981

The Administration in 1981 submitted legislation that restructured the program. Congress enacted most of the Administration's proposals in the "Municipal Wastewater Treatment Construction Grant Amendments of 1981" (P.L. 97-117).

The 1981 amendments further redirect the program toward the States, reduce Federal funding, and more strongly emphasize water quality through targeting of funds to current treatment needs and to projects that impact the country's highest-priority water bodies.

Funding

The amendments reduce potential Federal funding to $36 billion, a 60 percent reduction over the previous $90 billion projected, through limitations on grant eligibility. They authorized $2.4 billion for fiscal year 1982 and similar amounts through 1985.

Extended Deadlines

The Act extends several important deadlines. The July 1, 1983, deadline for municipal secondary treatment is extended to July 1, 1988, for most municipalities. The deadline for ocean-discharging communities to request waivers from secondary treatment requirements was extended from September 13, 1979, to December 29, 1982.

Immediate Changes

The amendments call for immediate as well as future changes in the program. Several changes took effect December 29, 1981, when the law went into effect. The law:

--Eliminates new grants for facility planning and design. These are now locally financed and the architect/engineer guides the grantee. EPA makes an allowance, rather than reimbursement, for these costs when a project gets a grant for construction. The State can reserve up to 10 percent of its allotment for advance allowances to small communities who cannot finance these costs. This allowance is later added as a percentage of the construction grant.

--Redefines secondary treatment to include trickling filters, oxidation ponds, and lagoons as the equivalent of secondary where water quality is not adversely affected.

--Extends the I/A program. The mandatory 3 percent I/A set-aside is increased to 4 percent of the State allotment and can go as high as 7 1/2 percent at a State's discretion. I/A projects get grants 20 percent higher than those for conventional systems but cannot exceed 85 percent of the project's cost.

--Retains the 1977 requirement that rural States set aside 4 percent of their allotment for alternative projects in communities of 3,500 or less or sparsely populated areas of larger communities.

Changes October 1, 1984:

To allow an orderly transition, most of the other major changes take effect October 1, 1984, when:

--The Federal grant share drops from 75 percent to a maximum of 55 percent for new construction funded after October 1, 1984. However, phased or segmented projects, those where the total project is not constructed or funded as one total package, that got 75 percent grants before Oct. 1, 1984, will get 75 percent grants for later phases or segments of the same facility if Federal funds are available.

--There will be little or no funding of reserve capacity for future growth.

--Eligible categories for Federal funding are restricted to treatment, new interceptors, and infiltration/inflow correction. The State can use up to 20 percent of its allotment to fund otherwise ineligible categories such as new collectors or treatment of high-priority combined sewer overflow discharges.

CONSTRUCTION GRANTS: THE PRESENT

PROGRAM PRIORITIES

The Administration undertook several initiatives to achieve new program priorities. These priorities emphasize regulatory reform, accelerated delegation of the program to the States, increased orientation toward water quality improvement, municipal compliance, and assurance of local financial management capability.

REGULATORY REFORM AND GUIDANCE

Regulatory reform translating the 1981 amendments into action is well underway. Final construction grant regulations and discretionary guidance along with new regulations on State delegation and redefined secondary treatment will be published when the new Administrator approves them. However, we had interim regulations in effect May 12, 1982, 5 months after the legislation passed. Also underway is guidance to ensure effective local financial management capability; wastewater utility management; user charges; project operation, maintenance, and performance; and Federal program management.

STATE DELEGATION

Our first priority is accelerated delegation of the program to the States. States are assuming project management functions as the EPA role phases into program overview and monitoring. Delegated States now provide

over 60 percent of program staffing; in the next 18 months State staffing will represent about 70 percent of the total. By that time, too, more than half the States will manage everything that can be delegated. Forty-seven States have accepted delegation and we expect all States to be delegated by the end of 1985.

Overview

To help States oversee and manage the program, in April of this year we issued a brief but comprehensive guidance booklet that describes the regulatory, policy, and management requirements States must address as they assume major management of the construction grants program. The booklet reviews State and Federal roles and responsibilities under a delegated program and sets forth basic principles for delegation and overview. Our primary objective is to turn over all possible project administration to the States. Certain projects, however, continue to need Federal involvement. These include projects that have interstate impacts; require environmental impact statements; involve Federal court cases, enforcement investigations, or allegations of waste, fraud, or mismanagement of Federal funds; are affected by ocean-discharge waivers; require advanced treatment; or are subject to special eligibility considerations or Congressional inquiry. The guidance discusses the management information EPA needs to assess progress toward national program goals, manage the national program in a fiscally responsible manner, and respond to national information reporting requests for budget and legislative requirements.

WATER QUALITY

States are redirecting their programs toward an increased emphasis on needs in priority water bodies. They are identifying these waters to concentrate resources where pollution has impaired a desired and attainable water use. They must also review and, if necessary, revise their construction grant priority systems and priority lists. The 1981 amendments require that States review and, if necessary, revise all water quality standards by December 29, 1984. Federal dollars will be targeted to priority water bodies and to construction projects that can most improve water quality or alleviate public health threats. An important new analytical tool for States and EPA will be a new computer system EPA is developing that will link State water quality data with information in the EPA Needs Survey data base and several other Federal data bases.

Advanced Treatment

Headquarters has reviewed 80 proposed advanced treatment projects since 1979 and has saved communities $825 million in projects with capital costs of $1.67 billion. We estimate operation and maintenance savings at another $20 million a year. Of projects reviewed since 1979, 52 percent needed changes. In 1982 Headquarters made changes in all projects it reviewed. We plan to review over 25 advanced treatment projects in fiscal year 1983.

PROJECT COMPLETIONS

An urgent concern is to complete projects under construction and to minimize funding of newly phased or segmented projects. About 2,770 treatment facilities funded since 1972 are completed. Another 2,844 are under construction and are 90 percent completed. A related objective is to accelerate completion of all administrative requirements on completed projects to eliminate backlogs that developed over the years.

COMPLIANCE

The almost 64,000 permits issued since 1972 are the driving force for compliance. These were issued to 15,600 municipal and 48,300 industrial dischargers. Because most of these permits are expiring or have already expired, major efforts are now underway to reissue permits that reflect new requirements.

Industrial

Industries have outpaced municipalities in meeting their permits. Eighty-one percent of the major industrial dischargers met their 1977 deadlines for best practicable technology (BPT) and this increased to 85 percent in 1982. Ninety-six percent have treatment in place or are on schedule to install the necessary equipment.

Municipal

We have witnessed a dramatic improvement in municipal compliance since 1977, when only 30 percent of municipal facilities were achieving secondary treatment. Now 77 percent of all publicly owned treatment works (POTWs) nationwide are meeting their effluent limits or are on schedule to do so. Plants built since 1972 perform better than older plants. In 1982, 82 percent of all major POTWs, those that discharge more than 1 million gallons a day, and 87 percent of major plants funded since 1972 were in compliance.

Small plants that discharge less than 1 million gallons a day account for the majority of compliance problems. We expect improved performance over the next 2 years in these plants, largely as a result of State efforts to provide onsite, over-the-shoulder training to small-plant operators and to help small communities assure adequate, updated user charge and financial management systems. States have developed a substantial training capability with Federal help. Through the years we have progressed from training operators to training instructors to developing extensive curricula and training materials. We have helped fund construction of over 25 State training centers. We believe State and local governments must prepare to accept responsibility to maintain their own training programs to help ensure permit compliance.

An added factor that will improve compliance is that the 1981 amendments and our regulations insist that the project comply with its permit within 1 year of startup. To assure this and to focus

responsibility for compliance, the engineering contractor must stay with the project during the first year of operation. At the end of the year the grantee certifies to the EPA regional administrator that the project can and does comply with its permit. Where compliance has not been achieved, a remedial action program will be developed with the State and grantee.

New Factors

Secondary Treatment Redefined--

The 1981 amendments redefined secondary treatment to include trickling filters, oxidation ponds, and lagoons as the equivalent of secondary treatment where water quality is not adversely affected. This redefinition could help 2,000 small trickling filter systems.

Waivers for Ocean-Discharging Communities--

The 1977 amendments created section 301(h) of the Clean Water Act primarily in response to pressure from ocean-discharging West Coast communities who argued that secondary treatment was unnecessary for effluent discharged into deep ocean waters where wastes rapidly aerate and disperse. Section 301(h) allows EPA to modify permit effluent limitations for BOD, suspended solids, and pH where the applicant discharges into certain ocean and estuarine waters. This does not waive requirements for pretreatment of industrial waste or for a monitoring program to determine the discharge's impacts on the marine environment. The applicant must demonstrate that the modification will not increase toxic discharge or impair the integrity of the receiving waters. Each waiver application must be evaluated individually. Communities were to apply for waivers by September 13, 1979.

The 1981 amendments provided additional time for communities to seek waivers. By the new deadline of December 29, 1982, EPA had received applications from 207 municipal dischargers. EPA has made decisions on 55 of these applications. One decision, a denial for a system in the State of Washington, is final. Of the 54 tentative decisions, we have denied 34 and approved 20. Most of those approved were from West Coast dischargers and most proposed at least primary treatment. All will be required to control toxics by pretreatment and nonpoint source control programs and to monitor compliance with section 301(h) ocean-discharge requirements. Applicants who were denied waivers generally discharged to enclosed embayments rather than to deep ocean water or waters with sufficient tidal action to dissipate the effluent. Responsibility for granting or denying waivers has been delegated to the EPA regional administrators.

Municipal Compliance Policy

The 1972 Clean Water Act said municipalities had to have secondary treatment by July 1977. The 1977 amendments extended that deadline to 1983 for those who requested an extension and, for these, the 1981 amendments again extended the deadline to 1988.

We will soon have ready what we think is a reasonable municipal compliance and enforcement policy to meet these deadlines. It is premised on the Clean Water Act's original assumption that no one has the right to pollute.

Our draft policy states that EPA and the States will set effluent limits and enforceable compliance schedules no later than July 1985 for all POTWs not already in compliance. The schedules will require that POTWs be in compliance by July 1988 wherever possible. The policy will ensure that POTWs already completed, especially those that used Federal funds, achieve compliance and remain in compliance. States must develop compliance strategies by March 1984.

FINANCIAL CAPABILITY

Many performance and compliance problems stem directly from poor financial management of projects and the community's myopic look into its future responsibilities at the time it received a grant.

Under the restructured construction grants program, localities assume far greater responsibility to finance and manage their own projects. The community that applies for a grant must prove the proposed project is anchored in a sound financial footing and that the system selected will solve the community's pollution problems in the most cost-effective way.

The program's new emphasis on financial responsibility is so crucial that the 1981 amendments and our new regulations say a community cannot get a grant unless it demonstrates it has the financial capability to build the proposed project and to operate and maintain it after it is built. For instance, a grant applicant must submit a user charge system that produces enough revenue for effective operation and maintenance. The grantee also has to demonstrate that it has the legal, institutional and management capability to build, operate and maintain the plant.

Financial Capability Policy

These requirements are described in a new policy that clearly outlines State and community responsibilities and helps a community demonstrate its financial capability. It encourages the community to draw up a comprehensive profile of its ability to pay for the proposed facility. The applicant has to answer questions on the project's scope, the local government's roles and responsibilities, the project's capital cost and annual costs per household, the community's ability to afford the project, and how it plans to finance the project.

The grant applicant must certify it has analyzed the project's costs and financial impacts on the community and that the community has the financial capability to build and operate the facility. States should work with grant applicants to review the applicant's financial capability and the financial feasibility of the project.

The financial capability policy's purpose is to get local governments to take an early and realistic look at their ability to finance and operate their proposed treatment works. This is a critical step in preparing them to take on a larger share of the project's capital cost as Federal funding phases down.

I/A and Appropriate Technology

Closely linked to good financial planning is selecting the appropriate technology. Grant applicants must show they have chosen the most cost-effective system. For small, unsewered communities this is usually an alternative system.

The 1977 Clean Water Act amendments created the innovative and alternative (I/A) and rural funding programs that help finance these I/A systems. These programs are proving their worth. We have awarded $200 million in Federal grants to help finance over 1,000 projects that have saved communities over $900 million. About 860 of these projects are alternative systems and 200 are innovative. It is too soon to evaluate the success rate for innovative systems because to date only 11 of these projects are operating. But of over 175 alternative projects in operation, those proving most successful are land treatment of wastewater, land spreading of sludge, containment ponds, small-diameter gravity sewers, and pressure sewers. Many of these Federally funded I/A projects have won national awards for engineering excellence.

We have already issued many publications and are preparing several more to inform State and local governments about I/A and other appropriate technologies. One of our publications has been translated into Japanese by the DoJo Joka Senta Company located in Tokyo. The publication is a foldout that describes 21 alternative systems in plain language. The systems illustrated include septic tanks; mound systems; pressure, vacuum, and small-diameter gravity sewers, and others. Our other publications include a 400-page onsite system design manual, a foldout on overland flow, and a foldout that compares costs of various technologies. More detailed cost comparisons will be avilable in a booklet we will publish in September. Also available then will be six new foldouts, each of which will describe an I/A system. The systems are sequencing batch reactors, counter current aeration, biological aerated filter, aquaculture, wetlands, and intrachannel clarification. Also under consideration are brochures on 90 percent methane recovery, land application of sludge, alternative collection systems for small communities, biological phosphorus removal, vacuum-assisted sludge drying beds, rapid infiltration, composting for small wastewater treatment plants, and intermittent discharge systems.

These technologies will increasingly help small communities solve their water pollution problems at reasonable costs.

FINANCING PUBLICLY OWNED TREATMENT WORKS

States: Restructure Programs

To make the best use of the limited Federal funds available, we encourage States to restructure their construction grants programs. They can reduce the Federal grant share uniformly throughout the State to spread Federal funds to the greatest number of projects of all sizes and strive to ensure that projects are completed. To accomplish this, communities who apply for new grants must state that they will complete the projects with or without Federal funds. The 1980 Clean Water Act amendments allow States to reduce Federal grant shares right now and some States are already doing this. It should be done in a way that minimizes the financial impact, especially on small, low-income communities.

CONSTRUCTION GRANTS: THE FUTURE

FUNDING

Congress has authorized $44.3 billion since 1972 to fund the construction grants program but total appropriations through fiscal year 1984 are $40.4 billion. The Administration asked for and received about $2.4 billion for fiscal years 1982, 1983, and 1984. The current law authorizes funding through fiscal year 1985. We do not know at this point precisely what direction future funding will take.

New Federalism

One direction may be the one President Reagan sent to Congress February 24, 1983. Under the Administration's proposal, construction grants is one of several Federal grant programs that would be returned to the States. It would be funded at $2.4 billion a year for 5 years, from fiscal year 1984 through fiscal year 1988. The States could take over the program if they wished during any one of the 5 years. They would have to use the construction grants portion of the funds entirely for construction grants the first year but could divert 20 percent of the funds for other purposes the second year, 40 percent the third year, 60 percent the fourth year, and 80 percent the fifth year. All grant programs returned to the States would be financed by Federal excise taxes on alcohol, cigarettes, and telephones.

Infrastructure

Other legislation has been introduced in the Congress to fund wastewater treatment facilities through State or Federal infrastructure banks. Infrastructure is the current term in the United States for public works, the permanent physical structures that form the underlying foundation of a society's basic services. Infrastructure includes, among other things, roads, bridges, wastewater treatment and drinking water facilities. The legislation would fund wastewater facilities' capital

costs with grants or loans from infrastructure banks. The banks' capital would be replenished by various means, including loan payback or revenues from user charges.

Privatization

In some areas of the United States, the private sector is experimenting with financing wastewater facilities and either leasing them back to the municipality or running the facilities for profit. The investment tax credit provision of the 1981 tax law would allow a firm operating a leased facility to make a profit.

Self-sustaining Public Utilities

However capital costs are financed, we encourage local governments to operate their facilities as self-sustaining public utilities that generate enough revenue for proper operation and maintenance. This approach strengthens the community's incentive to protect its investment. It also educates the user into an awareness of the true costs of wastewater treatment. The out-of-sight, out-of-mind syndrome that has hidden the treatment plant on the edge of town has applied to its costs as well. The public expects sewage treatment to be cheap. In fact, cheaper than monthly charges for pay television. This unrealistic perception puts local government officials who must provide the service in a difficult position. The user must come to realize that wastewater treatment and protection of public health deserves, and may require, a substantial financial commitment in the town's and the citizen's budgets.

NEW DIRECTIONS

Ten years of experience in directing and working with the massive construction grants program have given EPA, the States, and localities some insights into the potential successes or problems that can occur as certain concepts or systems are actually put into practice. I'd like to share some of our findings with you since they may benefit wastewater programs in other countries as well as those in the United States.

Management

It has become clear that many local governments do not plan or manage their projects carefully. This seems to relate to the high level of Federal and State funding. The 75 percent Federal grant share coupled with 10 percent to 15 percent State funding demands little local capital investment. This creates local indifference to the real cost of the facility. Reduced Federal grants provided by the 1980 and 1981 amendments should spur communities on to more careful evaluation of alternatives and of future growth when they plan their project and to sound management practices once the facility is built.

Facility Planning and Design

Local indifference to project planning and cost was also reinforced by the 75 percent Federal grants for facility planning and design. Many communities failed to weigh the real need for their project or its impact on the community until the project was ready for construction. Now facility planning and design are locally financed. This attracts community attention at the planning stage and increases the sense of local responsibility. The 1981 Clean Water Act amendments authorize an allowance that pays a percentage rather than the full cost of planning and design and the allowance is authorized only when a project gets a grant for construction. This prevents Federal funding of planning and design for projects that may never be built.

Secondary Treatment

EPA's rigid definition of secondary treatment proved to be unrealistic and unnecessarily costly for many communities adequately served by technologies other than activated sludge which was used as the base for the definition. The definition before the 1981 Amendments stood at 30 milligrams per liter for biochemical oxygen demand (BOD) and 30 milligrams per liter for suspended solids (SS) or 85 percent removal of BOD and SS on a monthly average. The definition precluded use of such proven technologies as trickling filters, oxidation ditches, and lagoons unless they were upgraded with an additional treatment process to reach the secondary standard. The definition also drove the use of sophisticated technologies that many communities could not afford to operate and maintain, much less build. The 1981 Amendments redefined secondary to include trickling filters, oxidation ditches, and lagoons where water quality is not adversely affected. This redefinition will especially help small communities of less than 10,000 people which are usually well served by these and other less costly systems.

Innovative and Alternative Technology

We are heartened by the success of the innovative and alternative (I/A) and rural funding programs created by the 1977 Clean Water Act amendments. Systems funded under these programs have already saved communities over $900 million in building and operating costs and many have won national awards for engineering excellence. Over 175 of the 860 Federally funded alternative projects are now operating. Those proving most successful are land treatment of wastewater, containment ponds, small-diameter gravity sewers, pressure sewers, and land spreading of sludge. Alternative systems are usually the most cost-effective systems for small communities who often have cost and operational problems with conventional activated sludge technology and central sewering. It is still too early to judge the success rate for innovative systems because to date only 11 of the 200 systems funded are completed and operating.

Small Communities and Appropriate Technologies

Most communities of less than 10,000 people probably should not even consider conventional activated sludge systems but should rely instead on I/A or other appropriate technologies. The redefinition of secondary treatment in the 1981 Amendments will help small communities by allowing lagoons, trickling filters, and oxidation ditches to be considered the equivalent of secondary treatment where water quality is not adversely affected. These systems are among those generally considered more appropriate for small communities than activated sludge.

We've found that communities with less than 1,000 people simply cannot afford activated sludge treatment with central sewers regardless of Federal funding. For communities of 3,500 or less or sparsely populated areas of large communities, alternative systems are usually the most cost-effective choice. Systems such as land treatment, containment ponds, small-diameter gravity sewers, pressure sewers, septic systems and upgraded variations of them cost less to build, operate, and maintain; have lower user costs; use less energy; create less sludge; and are simpler to operate than conventional central systems.

Use of these systems could have prevented some small communities from building activated sludge treatment facilities they could not afford. This overbuilding sprang from various causes; among them, excessive population projections, overly stringent State water quality standards and resultant stringent effluent limitations, and the rigid definition of secondary treatment. It's true that small communities should provide adequate capacity for reasonable future growth. But they must project their needs with care. Under-used facilities are costly to operate and maintain and may not last to the end of their design life. We expect the 1981 Amendments and our program reforms to help curb overbuilding of treatment systems. Among other changes, the restriction on funding of reserve capacity for future growth and our financial capability policy will encourage communities to select the most appropriate technology that will solve their wastewater problems at a reasonable cost.

Advanced Treatment

EPA Headquarters review of 80 advanced treatment (AT) projects since 1979 has justified congressional concern that some AT projects might be built that were not necessary. The reviews revealed that AT elements of some proposed projects could be eliminated. Between 1979 and 1982 the reviews saved an estimated $825 million in capital costs and another $20 million a year in operation and maintenance costs. The Agency had pressured the States in the early 1970s to move rapidly toward "fishable, swimmable" waters wherever attainable. The result was increasingly stringent State water quality standards that drove many communities to advanced treatment. The congressional appropriations committees in 1979 became concerned that AT facilities would be built that did not provide a corresponding improvement in water quality or public health for the additional dollars needed. The comittees required the EPA Administrator to

personally approve each proposed AT project that exceeded the costs for secondary treatment by $1 million or more. Congress raised this figure to $3 million for fiscal year 1980 because of the drain on EPA personnel resources. The reviews have prevented several unnecessary proposed AT projects from going forward to construction.

Ocean-Discharge Waivers: 301(h)

I am concerned that this section of the Clean Water Act granting waivers from secondary treatment to qualifying communities that discharge to the ocean may be a step backward in the water cleanup effort. These waivers are the first departure from the minimum technology based wastewater treatment standard. Industry will still need to pretreat its waste to remove hazardous and toxic pollutants before discharging to the POTW and the POTW will need to monitor the impact of its discharge. Moreover, the prohibition against discharge of sludge still applies. But we know that secondary treatment removes substantial amounts of toxic metals and toxic organics and without secondary treatment we really don't know how the waivers will impact the marine environment. It's clear we'll have to keep close watch on the effects of these waivers.

Pretreatment of Industrial Wastes

The Administration and Congress are working to resolve one of the most complex issues still confronting the goal of clean water. This is pretreatment of industrial waste. This year's congressional hearings on reauthorization of the Clean Water Act have focused on pretreatment as a priority item. Our legislative proposals and those of other interested parties are still before the Congress. Though EPA realizes some fine-tuning of its approach to pretreatment may be needed, it believes pretreatment remains a necessary part of our nation's effort to achieve clean water.

Pretreatment is a largely unfulfilled Clean Water Act requirement that certain industries who discharge wastewater to publicly owned treatment works (POTWs) pretreat their wastewater to remove specified amounts of toxic and other pollutants before discharging the waste to POTWs. Pretreatment protects the environment and POTWs from damage that can occur when hazardous or toxic waste enters a public wastewater system not designed to treat these wastes. Pretreatment specifically aims to prevent damage to the treatment system itself or interference with its operation, contamination of municipal sludge that inhibits its reuse or disposal, or passthrough of pollutants to receiving waters.

Controversy has surrounded pretreatment and EPA's attempts to implement it since the requirement was first stated in the 1972 Clean Water Act. The 1972 provision and the 1977 amendments of it have, over the years, drawn fire from industry, POTWs, and public interest groups alike. Pretreatment exemplifies the Herculean task EPA faces when it tries to develop equitable national pollutant controls based on national standards that, in turn, are based on scientific information and technology that are sophisticated, complex, and often controversial.

Legislative Background--

1972 Clean Water Act

The 1972 Clean Water Act prohibited discharge of toxic pollutants in toxic amounts into navigable waters. The Act directed that EPA first identify toxic pollutants and then issue two types of national standards to control them. For industries that discharged directly to waterways, EPA was to issue a national uniform effluent standard for each pollutant that applied to all classes of industries. For industries that discharged to POTWs rather than directly to waterways, EPA was to issue national uniform pretreatment standards. The pretreatment standards were to prevent discharge of any pollutant that interfered with, passed through, or was otherwise incompatible with the POTW. But lack of the necessary personnel and of sound scientific and technological information prevented EPA from identifying the toxic pollutants or from issuing the standards to control them.

1976 Consent Decree and the 1977 Clean Water Act

Delay in issuing the effluent standards and the pretreatment standards prompted several environmental groups to sue EPA. This resulted in a 1976 Federal court order that come to be known as the "toxics consent decree." The consent decree ordered that EPA develop national effluent standards for 65 "priority" toxic pollutants which EPA later subdivided into 129 toxic substances. The standards were to be incorporated in the permits of direct industrial dischargers in 21 classes of industries. The decree also ordered that EPA issue regulations setting minimum pretreatment standards to control priority and certain other pollutants discharged to POTWs by industries and other sources of nondomestic waste.

The 1977 Clean Water Act incorporated the terms of the consent decree as the basic national approach to control of toxic and certain other pollutants in the industrial wastestream. The Act defined three classes of pollutants and demanded technology based standards to control them. The classes are conventional, toxic, and nonconventional. Conventional pollutants mean those traditionally controlled by wastewater treatment systems. These include BOD, suspended solids, oil and grease, pH, and fecal coliform. Toxics include those identified in the 1976 consent decree and in the Act. Nonconventional pollutants include those not otherwise designated, such as phosphorus, nitrogen, and ammonia.

Implementation of Clean Water Act Pretreatment Requirements--

EPA's Pretreatment Program

The underlying concept of the EPA pretreatment program is that the municipalities must assume the prime responsibility for enforcing the national program. The framework for the national program consists of the general pretreatment regulations and the national pretreatment standards, both of which aim to control priority and certain other pollutants.

General Pretreatment Regulations. The general pretreatment regulations require States and municipalities to develop local pretreatment programs that enforce national pretreatment standards. The regulations detail the components of a pretreatment program and identify the responsibilities of EPA, State and local governments, and industry. The regulations were issued June 26, 1978, were amended January 28, 1981, and are in force as of March 13, 1981.

The regulations state that any POTW with a design flow of more than 5 million gallons a day must have established a pretreatment program by July 1, 1983. POTWs with flows of less than 5 million gallons a day are also required to establish a pretreatment program if nondomestic waste the POTW treats causes upsets, sludge contamination, violations of NPDES permits, or is discharged to the POTW by a significant industrial user subject to national pretreatment standards.

EPA estimates that 1,700 of the nation's POTWs must develop local pretreatment programs. EPA and authorized States have approved 200 local pretreatment programs and expect this number to grow to 400 by the end of the year. They expect to approve 900 more in 1984 and the remainder by the end of 1985. So far, 15 States have pretreatment programs approved by EPA. A recently formed task force of EPA and State officials expects to have a strategy for POTW compliance with pretreatment requirements ready by the end of this year.

Pretreatment Standards.-- The general pretreatment regulations establish two types of pretreatment standards to control pollutant discharges into POTWs: prohibitive discharge standards and national categorical pretreatment standards.

Prohibitive discharge standards apply to all industrial and commercial establishments connected to POTWs. Prohibitive standards protect the POTW's plant and operations by prohibiting discharge of pollutants that create a fire or explosion hazard in the sewers or treatment works; are corrosive, with a pH lower than 5.0; obstruct flow in the sewer system or interfere with operation; upset the treatment processes to cause a violation of the POTW's permit; or increase the temperature of wastewater entering the treatment plant to above 104°F. The prohibitive standards are expressed in general terms in one comprehensive regulation. The POTWs issue the specific prohibitive standards for their users.

National categorical pretreatment standards apply to discharges from 23 classes of industries thought to be the most significant sources of toxic pollutants discharged to POTWs. The 23 classes of industries cover a total of 15,000 individual industrial facilities. The categorical pretreatment standards protect both the environment and the POTW and are issued by EPA. They apply to industries that discharge to POTWs (indirect dischargers) and correspond to the national uniform effluent standards that apply to industries that discharge directly to waterways (direct dischargers). Both standards contain numerical limitations for toxic pollutants commonly discharged by a particular industry and are published

together as one regulation governing that industry. Altogether EPA will issue national categorical pretreatment standards for 23 classes of industries and national effluent standards for 28 classes of industries. The numbers are different because five classes of industries do not discharge to POTWs and so do not require pretreatment standards. To date, EPA has issued national categorical pretreatment standards for 11 classes of industries and has proposed standards for 7 others. The remaining standards are to be issued by July 1, 1984.

A major concern of the Administration's 1983 legislative proposals is that there may be certain cases where full implementation of national categorical pretreatment standards is not necessary. The Administration is grappling with the problem of how to provide relief where it is justified yet maintain the program's momentum and protection.

Removal Credits

The 1977 Act recognized that secondary treatment actually removes substantial amounts of priority pollutants. To reflect this removal, the law modified the 1972 pretreatment requirement by empowering POTWs to grant their industrial users "removal credits." Removal credits are modifications of the national categorical pretreatment standards. They reflect the amount of toxic and certain other pollutants removed by the POTW. To grant removal credits, a POTW must have a pretreatment program in place and must be meeting secondary treatment or be on schedule to do so. Removal credits must not jeopardize the quality of the POTW's sludge or the quality of receiving waters.

POTWs, industries, and public interest groups have, from various perspectives, attacked the concept of removal credits and its implementation by EPA. Much of the criticism centers on the concept's complexity. Other criticism focuses on the lengthy testing and administrative procedures POTWs must perform to demonstrate removal of priority pollutants. Some industries complain that denial of removal credits results in "treatment for treatment's sake."

We are trying to streamline the process for granting removal credits and have proposed revisions to the general pretreatment regulations to accomplish this. We expect to issue the final revisions on removal credits in early 1984.

EPA Studies and Pretreatment--

Two EPA studies need special mention in discussing pretreatment. One began in 1978 and the second in 1981. The results of the first, commonly referred to as the "40-city Study," were used in arriving at recommended national removal credits that POTWs could use as a base for their own programs. The study results, published as a final report in September 1982, indicated that well-operated secondary treatment plants remove substanial amounts of heavy metals and some toxic organics. Half of the

secondary treatment plants reduced priority pollutant metals by 70 percent, volatile priority pollutants by 82 percent, and base neutral priority pollutants by 65 percent.

The second study analyzed pollutant loads contributed by industrial customers at 2,000 POTWs. The study included all POTWs that discharged more than 5 million gallons a day and some smaller plants with a history of operational problems or with significant industrial contributors of toxics. Among the report's findings were that there has been substantial control of metals by industrial pretreatment--a 65 percent reduction from raw discharge levels; but less control of toxic organics--a 37 percent reduction from raw discharge levels. The study also indicated that pretreatment would, on the average, reduce heavy metals and toxic organics in municipal sludges by 52 percent and 67 percent, respectively.

National Effluent Standards for Direct Dischargers--

After initial delays, EPA is making major progress in developing and issuing the national uniform effluent standards for direct industrial dischargers called for by the 1976 toxics consent decree. These national effluent standards correspond to the national categorical pretreatment standards for indirect dischargers. EPA was unable to issue effluent standards on schedule for the original 21 classes of industries identified by the 1976 consent decree that were thought to be the most significant sources of toxic pollutants. By regrouping, the 21 classes of industries were expanded to 34 by a March 9, 1979, modified consent decree and, later, to 28 by an August 25, 1982, court order. To date, EPA has issued 14 final effluent standards and proposed 10 others. By October 1983 EPA will have issued 21 final effluent standards and by October 1984 will have issued all 28 currently required.

The Administration's proposals to amend the Clean Water Act in 1983 would extend the deadlines for industries to comply with national effluent standards for priority pollutants from July 1, 1984, to July 1, 1988. This would allow industries a reasonable time to build the needed treatment facilities after the standards are issued.

Sludge

Those of you who deal with environmental problems on a daily basis will appreciate a sentence by the great English educator and writer Matthew Arnold, who said of a friend, "He saw life steadily and saw it whole." That is how we constantly strive to look at the environment--to see it whole--as we increasingly discover that all environmental problems are interrelated. The solution to a problem in one area can become part of the problem in another.

A classic example is the challenge to POTWs of what to do with the nearly 7 million dry tons of sewage sludge produced each year by 150 million Americans and some 87,000 industries that discharge waste to more than 15,000 municipal sewage treatment works. The volume of this

product of central treatment will grow markedly as more and more treatment facilities begin to operate. One estimate is that by 1990 United States POTWs will produce 16 million tons of dry sewage sludge every year, nearly three times the volume produced each year in the 1970s. From 30 percent to 50 percent of a conventional treatment plant's capital costs go for the sludge management system and we estimate that POTWs spend more than $500 million a year in operating and maintenance costs for sludge disposal and reuse. Clearly, sludge portends a major waste management problem for the years ahead if we do not work to find an integrated solution right now.

We are working toward that solution by drawing on the collective wisdom of an EPA sludge task force organized in July 1982 to produce an integrated sludge management policy. To assure a broad-based policy, the task force solicits views and information from States, localities, universities, other Federal agencies, and public interest groups. The goal is a balanced policy that weighs environmental risks as well as the benefits and costs of various sludge management options. The group is working toward a comprehensive sludge management guideline under section 405 of the Clean Water Act. The task force expects to have the guideline ready this year.

We still have to deal with the fragmented regulatory approach that now regulates sewage sludge under potentially conflicting provisions of at least seven major laws that deal with discrete environmental concerns. Besides the Clean Water Act, these statutes include the Resource Conservation and Recovery Act of 1976, the Toxic Substances Control Act of 1976, the Marine Protection Research and Sanctuaries Act of 1977, the Clean Air Act, the Safe Drinking Water Act, and the National Environmental Policy Act of 1969.

Sludge as a Resource--

The emerging sludge policy reflects the Agency's view that sludge is a valuable resource that should be used and recycled to capture its nutrient and organic components.

The idea that sludge is an asset to be used rather than a problem to be disposed of takes on added weight now that traditional methods of sludge disposal are becoming environmentally, politically, or economically unacceptable. Incineration is expensive and uses high amounts of energy. Landfills draw public protests, the more so now amid the mounting public clamor over toxic waste. Ocean dumping has largely phased out since the mid-1970s, mostly for environmental reasons, though a limited number of coastal cities are still permitted to use this method if authorized by a Federal court order. Lagoons and storage basins still provide a cheap and convenient way to treat sludge but odor problems and possible groundwater contamination usually arouse public opposition from nearby residents. All of these restrictions have kindled new interest in an old concept--land application of sludge.

Land Application of Sludge--

Land application of sludge is one of the most cost-effective ways to reuse or dispose of sludge. Recent estimates are that 40 percent of all sludge produced in the United States is applied to the land. This compares with 25 percent in 1976. When properly applied, the sludge serves as a fertilizer supplement and soil conditioner that can be used in agriculture, forestry, land reclamation, and other land management activities. A U.S. Department of Agriculture study indicated that sludge can supplant 1 percent of all nitrogen and 4 percent of all phosphorus now supplied in the United States by chemical fertilizers.

We are beginning to see land application projects come into operation that were funded under the innovative and technology provisions of the 1977 Clean Water Act and the 1981 Amendments. Twenty-two of the 150 facilities funded under the I/A program are now operating. Many other land application projects that were not Federally funded have been successfully operating throughout the country for years, most on private land.

Spreading sludge on cropland or other farmland is the most common method of land application in the United States. This technique has been used for decades, especially in the midwestern part of the country. Only in recent years, however, have the necessary research and monitoring studies been undertaken to develop sound guidelines for this use. It requires careful management and application practices and is regulated or otherwise controlled by EPA, the State, and the locality. Most municipalities supply sludge to farmers at little or no cost so there is a decided cost advantage in reduced fertilizer costs. Several areas of the country apply sewage to surface-mined or other drastically disturbed land as an aid to reclamation. The cities of Chicago and Philadelphia transport sludge over 200 miles to surface-mined land reclamation projects. Sludge application to forest land is another option for sludge reuse and disposal but not widely used in the United States because data about the impact of heavy metals on the forest ecosystem is limited. An interesting note is that studies are underway to evaluate the use of sewage sludge in stabilizing certain ash-covered forest production areas near the famous Mount St. Helens volcano in the State of Washington.

Sludge Quality and Land Application

Despite its advantage and widespread use, land application is not problem free. Some pathogenic bacteria, viruses, and parasites survive most sludge processes. To minimize the potential from public exposure to pathogens, EPA has established special criteria to use in processing sludge before it is applied to cropland. This sludge must undergo a process that reduces the pathogen levels. Some of these processes are aerobic and anaerobic digestion, air-drying, heat-drying, irradiation, composting, and lime stabilization.

Toxic pollutants and heavy metals remain a constraint to greater use of sludge in agriculture. Scientic knowledge about these substances has evolved rapidly, but there is still disagreement about what levels are acceptable for sludges applied to agricultural land. Recent U.S. Department of Agriculture research suggests that a good sludge that might be used safely by the general public can be defined. We need more research to verify this. Among other things it is an important concept in setting national pretreatment standards for industrial waste. It is clear that EPA's industrial pretreatment policy must be tied directly to sludge quality since pretreatment enhances the sludge as a resource and will simplify control and regulatory oversight procedures. One study indicates that pretreatment would reduce heavy metals in POTW sludge by 52 percent and toxic organics by 67 percent. Projections are that pretreatment coupled with additional removal of heavy metals and toxic organics by secondary treatment at the POTW should result in a cleaner sludge than is produced by secondary treatment alone.

Marketing Sludge Products--

There is a growing interest in the United States in marketing sludge products as an organic fertilizer or soil conditioner for use by commercial nurseries, gardeners, golf courses, and similar users. The process attracting the most attention is composting, which stabilizes the sludge through microbial decomposition. The "aerated pile" method developed by the U.S. Department of Agriculture is one the most successful processes because it can be used to compost both small and large quantities of either digested or undigested sludges in a relatively short time. The process involves mixing the sludge with a bulking agent, such as wood chips, to increase aeration. The mixture is then placed in carefully constructed piles for several weeks during which stabilization and curing take place. Composting reduces the levels of persistent organic compounds and pathogens. It is considered an alternative technology under the Clean Water Act and is eligible for up to 85 percent Federal construction grants for land, equipment, and related construction costs.

The marketing of sludge products is the only disposition of sludge not now regulated by EPA but we expect this to be a prominent part of the comprehensive sludge regulation or guideline that the EPA sludge task force will develop.

Sludge and the Future--

The greatest challenge for the future is how to capture the resource potential of this valuable byproduct. We have learned a great deal about the safe use of sludge in the last 10 years. Through cooperative efforts of EPA, the U.S. Department of Agriculture, the U.S. Food and Drug Administration, land-grant universities and others, we have identified contaminants of regulatory concern and have issued regulations and guidelines for agricultural use of sludge. But we are really just beginning to see the sludge issue "steadily and to see it whole," to use Matthew Arnold's fine words. I am confident, however, that with a

comprehensive sludge policy and continuing research and other cooperative ventures by Federal and State agencies and universities,we will make the same kind of progress with sludge that we have seen in wastewater treatment and other environmental programs.

THE PAST IS PROLOGUE

PROGRESS

We have come a long way since the "Old Law" of 1956 and the "New Law" of 1972, now simply referred to as the Clean Water Act. Besides this Act, we have seen at least 13 other major environmental laws passed and we have seen considerable progress in meeting the goals of these laws. And, of special importance for this discussion, we have seen the construction grants program mature.

Water Cleanup

We still lack detailed monitoring data that would give us a long-term comparative analysis on water cleanup. We especially lack biological data. But States all across the country report dramatic evidences of cleanup at specific sites. Perhaps the most heartening is Lake Erie, one of the United States' five great lakes. In the 1950s some analysts pronounced Lake Erie a dying lake and predicted it could take up to 500 years for the lake to return to life, if, indeed, it was possible at all. Massive algal blooms covered the lake with slimy green mats. The algae deprived the bottom water of 65 percent of its oxygen in the summer months. By 1979, thanks to municipal and industrial pollution controls, only 6 percent of the bottom water lacked oxygen. Clean-water game fish now survive in the lake, and the beaches, closed for more than a decade, have reopened.

In another area of the country, we have the Neches River tidal area in Texas. In the 1960s, oil refinery and chemical plant wastes colored the river water black. A 65 percent reduction of dissolved oxygen was measured at one location. The fish, mainly large-mouth bass, stayed above the city of Beaumont, avoiding the 25-mile stretch between Beaumont and the tidal area at the mouth of Sabine Lake. Between 1970 and 1975, EPA awarded $10.5 million in grants to build secondary treatment plants and ancillary treatment facilities along the Neches River. Industry built facilities to provide clarification and aeration systems to neutralize high or low pH levels and separation devices to remove oil from the water surface. By 1976 EPA had issued NPDES permits to all major industrial and municipal dischargers along the river. Shrimp moved up the Neches in numbers large enough to plug up industrial intakes, and commercial crabbers worked these waters for profit. In 1976, a tarpon was caught in Sabine Lake, the first in over 30 years.

Equally dramatic success stories are the Connecticut River, where the first salmon was caught in over a hundred years; and the St. Johns River in Jacksonville, Florida, where a massive water and sewer construction program

in 7 years transformed what Jacksonville's mayor called an "open sewer" to a river that now supports swimming and water sports. We have more than 70 examples of such cleanups in waters from Hawaii to Maine and from Alaska to Texas. And we expect to see more of these stories as the thousands of plants now under construction begin to operate.

FOUNDATION FOR FUTURE

It seems clear that, regardless of how wastewater treatment facilities are financed, the day of large Federal categorical grants is phasing out. The major responsibility to finance these projects will gradually shift to local governments which will operate and maintain the treatment works as self-sustaining public utilities. States and localities will need to establish monitoring programs to ensure continued water quality improvement.

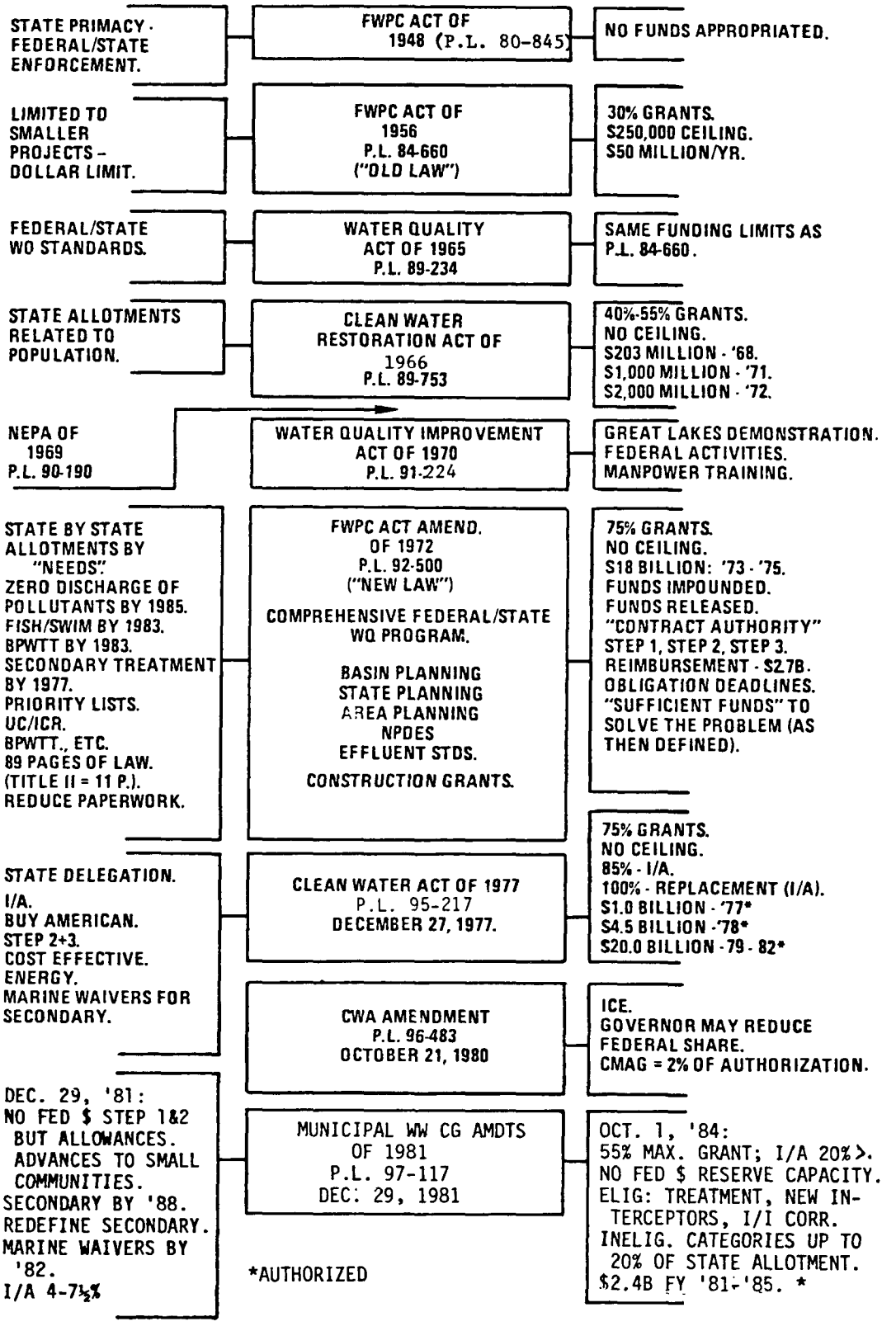
Reforms in Place

As States and localities take over more and more responsibility for meeting water quality goals, they will find they have a good foundation on which to build their own programs. They will find that the 1981 amendments to the Clean Water Act, the simpler and shorter construction grants regulations and guidance, and major management reforms will ease the transition as they assume major program functions. They will find that the municipal waste treatment program has come into better balance as a more cost-effective program.

EPA-State Commitment

EPA and the States face a great challenge in the years ahead but I believe we can look for continued progress as the States exercise their full prerogatives under the 1981 legislation. We have a strong but realistic Clean Water Act with deadlines, permits that demand compliance, and other enforceable requirements. EPA is fully committed to work with the States to carry out its mandate for clean water under this important legislation. The Nation's goal of cleaning up its waters has not changed. The way it will be done HAS.

THE END


STATE PRIMACY · FEDERAL/STATE ENFORCEMENT.
FWPC ACT OF 1948 (P.L. 80-845)
NO FUNDS APPROPRIATED.
LIMITED TO SMALLER PROJECTS - DOLLAR LIMIT.
FWPC ACT OF 1956 P.L. 84-660 ("OLD LAW")
30% GRANTS. $250,000 CEILING. $50 MILLION/YR.
FEDERAL/STATE WQ STANDARDS.
WATER QUALITY ACT OF 1965 P.L. 89-234
SAME FUNDING LIMITS AS P.L. 84-660.
STATE ALLOTMENTS RELATED TO POPULATION.
CLEAN WATER RESTORATION ACT OF 1966 P.L. 89-753
40%-55% GRANTS. NO CEILING. $203 MILLION - '68. $1,000 MILLION - '71. $2,000 MILLION - '72.
NEPA OF 1969 P.L. 90-190
WATER QUALITY IMPROVEMENT ACT OF 1970 P.L. 91-224
GREAT LAKES DEMONSTRATION. FEDERAL ACTIVITIES. MANPOWER TRAINING.
STATE BY STATE ALLOTMENTS BY "NEEDS". ZERO DISCHARGE OF POLLUTANTS BY 1985. FISH/SWIM BY 1983. BPWTT BY 1983. SECONDARY TREATMENT BY 1977. PRIORITY LISTS. UC/ICR. BPWTT., ETC. 89 PAGES OF LAW. (TITLE II = 11 P.). REDUCE PAPERWORK.
FWPC ACT AMEND. OF 1972 P.L. 92-500 ("NEW LAW")
COMPREHENSIVE FEDERAL/STATE WQ PROGRAM.
BASIN PLANNING
STATE PLANNING
AREA PLANNING
NPDES
EFFLUENT STDS.
CONSTRUCTION GRANTS.
75% GRANTS. NO CEILING. $18 BILLION: '73 - '75. FUNDS IMPOUNDED. FUNDS RELEASED. "CONTRACT AUTHORITY" STEP 1, STEP 2, STEP 3. REIMBURSEMENT - $2.7B. OBLIGATION DEADLINES. "SUFFICIENT FUNDS" TO SOLVE THE PROBLEM (AS THEN DEFINED).
STATE DELEGATION. I/A. BUY AMERICAN. STEP 2+3. COST EFFECTIVE. ENERGY. MARINE WAIVERS FOR SECONDARY.
CLEAN WATER ACT OF 1977 P.L. 95-217 DECEMBER 27, 1977.
75% GRANTS. NO CEILING. 85% - I/A. 100% - REPLACEMENT (I/A). $1.0 BILLION - '77* $4.5 BILLION - '78* $20.0 BILLION - 79 - 82*
CWA AMENDMENT P.L. 96-483 OCTOBER 21, 1980
ICE. GOVERNOR MAY REDUCE FEDERAL SHARE. CMAG = 2% OF AUTHORIZATION.
DEC. 29, '81: NO FED $ STEP 1&2 BUT ALLOWANCES. ADVANCES TO SMALL COMMUNITIES. SECONDARY BY '88. REDEFINE SECONDARY. MARINE WAIVERS BY '82. I/A 4-7½%
MUNICIPAL WW CG AMDTS OF 1981 P.L. 97-117 DEC. 29, 1981
OCT. 1, '84: 55% MAX. GRANT; I/A 20%>. NO FED $ RESERVE CAPACITY. ELIG: TREATMENT, NEW INTERCEPTORS, I/I CORR. INELIG. CATEGORIES UP TO 20% OF STATE ALLOTMENT. $2.4B FY '81-'85. *
*AUTHORIZED

Attachment B:

NRDC CONSENT AGREEMENT
and
CLEAN WATER ACT OF 1977
TOXIC POLLUTANTS

Acenaphthene*

Acrolein*

Acrylonitrile*

Benzene*

Benzidine*

Carbon tetrachloride (tetrachloromethane)*

Chlorinated benzenes (other than dichlorobenzenes)*
- chlorobenzene
- 1,2,4-trichlorobenzene
- hexachlorobenzene

Chlorinated ethanes (including 1,2-dichloroethane, 1,1,1-trichloroethane and hexachloroethane)*
- 1,2-dichloroethane
- 1,1,1-trichloroethane
- hexachloroethane
- 1,1-dichloroethane
- 1,1,2-trichloroethane
- 1,1,2,2-tetrachloroethane
- chloroethane

Chloroalkyd ethers (chloromethyl, chloroethyl, and mixed ethers)*
- bis(chloromethyl)ether
- bis(2-chloroethyl)ether
- 2-chloroethyl vinyl ether (mixed)

Chlorinated naphthalene*
- 2-chloronaphthalene

Chlorinated phenols (other than those listed elsewhere; includes trichlorophenols and chlorinated cresols)*
- 2,4,6-trichlorophenol
- parachlorometa cresol

Chloroform (trichloromethane)*

* Specific compounds and chemical classses as listed in the NRDC Consent Agreement and Committee Print 95-30, Data Relating to H.R. 3199 (Clean Water Act of 1977), Committee on Public Works and Transportation, 95th Congress, 1st Session, GPO, 1977

2-chlorophenol*

Dichlorobenzenes*
    1,2-dichlorobenzene
    1,3-dichlorobenzene
    1,4-dichlorobenzene

Dichlorobenzidine*
    3,3'-dichlorobenzidine

Dichloroethylenes*
    1,1-dichloroethylene
    1,2-dichloroethylene

2,4-dichlorophenol*

Dichloropropane and dichloropropene*
    1,2-dichloropropane
    1,3-dichloropropene

2,4-dimethylphenol*

Dinitrotoluene*
    2,4-dinitrotoluene
    2,6-dinitrotoluene

1,2-diphenylhydrazine*

Ethylbenzene*

Fluoranthene*

Haloethers (other than those listed elsewhere)*
    4-chlorophenyl phenyl ether
    4-bromophenyl phenyl ether
    bis(2-chloroisopropyl)ether
    bis(2-chloroethoxy)methane

Halomethanes (other than those listed elsewhere)*
    methylene chloride (dichloromethane)
    methyl chloride (chloromethane)
    methyl bromide (bromomethane)
    bromoform (tribromomethane)
    dichlorobromomethane**
    trichlorofluoromethane**
    dichlorodifluoromethane
    chlorodibromomethane

Hexachlorobutadiene*

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* Ibid

** Deleted from list by EPA. These toxic chemicals have no significant potential for exposure to water.

Hexachlorocyclopentadiene*

Isophorene*

Naphthalene*

Nitrobenzene*

Nitrophenols (including 2,3-dinitrophenol and dinitrocresol)*
- 2-nitrophenol
- 4-nitrophenol
- 2,4-dinitrophenol
- 4,6-dinitro-o-cresol

Nitrosamines*
- N-nitrosodimethylamine
- N-nitrosodiphenylamine
- N-nitrosodi-n-propylamine

Pentachlorophenol*

Phenol*

Phthalate esters*
- bis(2-ethylhexyl)phthalate
- butyl benzyl phthalate
- di-n-butyl phthalate
- di-n-octyl phthalate
- diethyl phthalate
- dimethyl phthalate

Polynuclear aromatic hydrocarbons*
- benzo(a)anthracene (1,2-benzanthracene)
- benzo(a)pyrene (3,4-benzopyrene)
- 3,4-benzofluoranthene
- benzo(a)fluoranthene (11,12-benzofluoranthene)
- chrysene
- acenaphthylene
- anthracene
- benzo(ghi)perylene (1,12-benzoperylene)
- fluorene
- phenanthrene
- dibenzo(a,h)anthracene (1,2,5,6-dibenzanthracene)
- ideno(1,2,3-cd)pyrene (2,3-o-phenylenepyrene)

Tetrachloroethylene*

Toluene*

Trichloroethylene*

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* Specific compounds and chemical classes as listed in the NRDC Consent Agreement and Committee Print 95-30, Data Relating to H.R. 3199 (Clean Water Act of 1977), Committee on Public Works and Transportation, 95th Congress, 1st Session, GPO, 1977

Vinyl chloride (chloroethylene)*

Pesticides and metabolites*
- aldrin
- dieldrin
- chlordane (technical mixture and metabolites)

DDT and metabolites*
- 4,4'-DDT
- 4,4'-DDE(p,p'-DDX)
- 4,4'-DDD(p,p'-TDE)

Endosulfan and metabolites*
- α-endosulfan-Alpha
- β-endosulfan-Beta
- endosulfan sulfate

Endrin and metabolites*
- endrin
- endrin aldehyde

Heptachlor and metabolites*
- heptachlor
- heptachlor epoxide

Hexachlorocyclohexane (all isomers)*
- α-BHC-Alpha
- β-BHC-Beta
- γ-BHC(lindane)-Gamma
- δ-BHC-Delta

Polychlorinated biphenyls (PCB's)*
- PCB-1242 (Arochlor 1242)
- PCB-1254 (Arochlor 1254)
- PCB-1221 (Arochlor 1221)
- PCB-1232 (Arochlor 1232)
- PCB-1248 (Arochlor 1248)
- PCB-1260 (Arochlor 1260)
- PCB-1016 (Arochlor 1016)

Toxaphene*

Antimony (Total)*

Arsenic (Total)*

Asbestos (Fibrous)*

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* Ibid

Beryllium (Total)*

Cadmium (Total)*

Chromium (Total)*

Copper (Total)*

Cyanide (Total)*

Lead (Total)*

Mercury (Total)*

Nickel (Total)*

Selenium (Total)*

Silver (Total)*

Thallium (Total)*

Zinc (Total)*

2,3,7,8-tetrachlorodibenzo-p-dioxin (TCDD)*

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* Ibid

Attachment C: EFFLUENT GUIDELINES DIVISION

7/25/83

FEDERAL REGISTER CITATIONS FOR PROPOSED AND FINAL RULES

Industry	40 CFR PART	TYPE RULE	SIGNATURE*	FEDERAL REGISTER CITATION	
° Aluminum Forming	467	Proposal	11/ 5/82	47 FR 52626	11/22/82
		Promulgation	[09/83]	---	---
° Battery Manufacturing	461	Proposal	10/29/82	47 FR 51052	11/10/82
		Promulgation	[01/84]	---	---
° Coal Mining	434	Proposal	12/30/82	46 FR 3136	01/13/81
		Promulgation	9/30/82	47 FR 45382	10/13/82
° Coil Coating					
Phase I	465	Proposal	12/30/80	46 FR 2934	01/12/81
		Promulgation	11/ 5/82	47 FR 54232	12/ 1/82
Phase II (Canmaking)	465	Proposal	1/31/83	48 FR 6268	02/10/83
		Promulgation	(10/83)	---	---
° Copper Forming	468	Proposal	10/29/82	47 FR 51278	11/12/82
		Promulgation	(07/83)	---	---
° Electrical/Electronic Components					
Phase I	469	Proposal	8/11/82	47 FR 37048	08/24/82
		Promulgation	3/31/83	48 FR 15382	04/08/83
Phase II	469	Proposal	2/28/83	48 FR 10012	03/09/83
		Promulgation	(11/83)	---	---
° Electroplating (Pretreatment)	413	Proposal	1/24/78	43 FR 6560	02/14/78
		Promulgation	8/ 9/79	44 FR 52590	09/07/79
		Proposed Admendment	8/11/82	47 FR 38462	08/31/82
° Foundries (Metal Molding and Casting)	464	Proposal	10/29/82	47 FR 51512	11/15/82
		Promulgation	[06/84]	---	---
° Inorganic Chemicals					
Phase I	415	Proposal	7/10/80	45 FR 49450	07/24/80
		Promulgation	6/16/82	47 FR 28260	06/29/82
Phase II	415	Proposal	(09/83)	---	---
		Promulgation	(06/84)	---	---
° Iron & Steel Manuf.	420	Proposal	12/24/80	46 FR 1858	01/07/81
		Promulgation	5/18/82	47 FR 23258	05/27/82
° Leather Tanning & Finishing	425	Proposal	6/13/79	44 FR 38746	07/02/79
		Promulgation	11/ 7/82	47 FR 52848	11/23/82
° Metal Finishing	433 & 413	Proposal	8/11/82	47 FR 38462	08/31/82
		Promulgation	7/ 5/83	48 FR 32462	07/15/83
° Nonferrous Metals					
Phase I	421	Proposal	1/31/83	48 FR 7032	02/17/83
		Promulgation	(01/84)	---	---
Phase II	421	Proposal	[02/84]	---	---
		Promulgation	[11/84]	---	---

* Administrator's signature; ( ) is the projected schedule approved by the Court on August 25, 1982 and October 26, 1982; [ ] is the revised schedule, pending approval by the Court, that has been requested by EPA (R. Hanmer Affidavit, June 21, 1983).

EFFLUENT GUIDELINES DIVISION

7/25/83

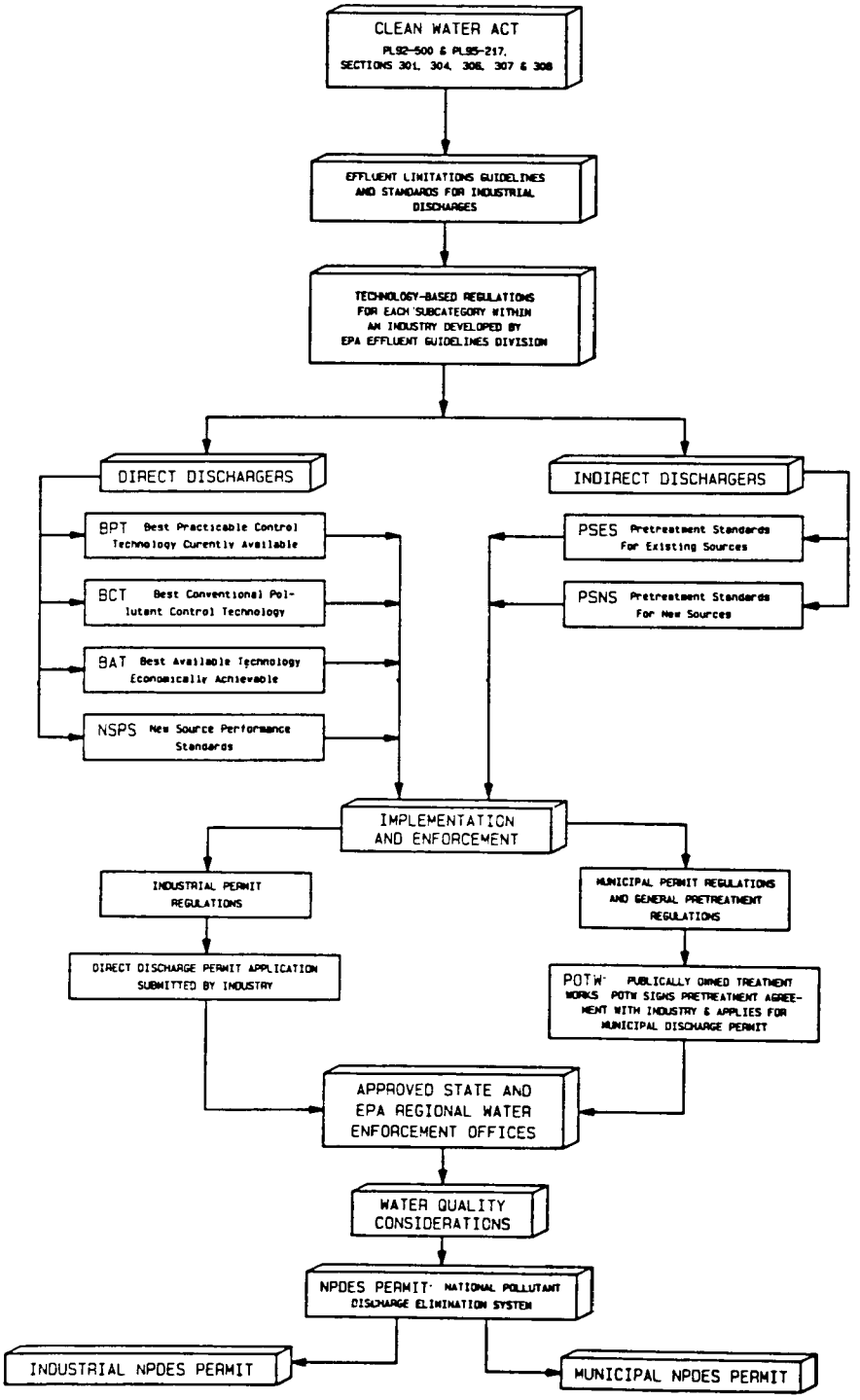
FEDERAL REGISTER CITATIONS FOR PROPOSED AND FINAL RULES

Industry	40 CFR PART	TYPE RULE	SIGNATURE*	FEDERAL REGISTER CITATION	
° Nonferrous Metals Forming	471	Proposal	[01/84]	---	---
		Promulgation	[10/84]	---	---
° Ore Mining	440	Proposal	5/25/82	47 FR 25682	06/14/82
		Promulgation	11/05/82	47 FR 54598	12/ 3/82
° Organic Chemicals and Plastics &	414	Proposal	2/28/82	48 FR 11828	03/21/83
Synthetic Fibers	& 416	Promulgation	(03/84)	---	---
° Pesticides.....	455	Proposal	11/05/82	47 FR 53994	11/30/82
		Proposal - (Analytical Methods)	1/31/83	48 FR 6250	02/10/83
		Promulgation	(12/83)	---	---
° Petroleum Refining.....	419	Proposal	11/27/79	44 FR 75926	12/21/79
		Promulgation	9/30/82	47 FR 46434	10/18/82
° Pharmaceuticals.....	439	Proposal	11/07/82	47 FR 53584	11/26/82
		Promulgation	(09/83)	---	---
° Plastics Molding & Forming	463	Proposal	[01/84]	---	---
		Promulgation	[10/84]	---	---
° Porcelain Enameling.....	466	Proposal	1/19/81	46 FR 8860	01/27/81
		Promulgation	11/ 5/82	47 FR 53172	11/24/82
° Pulp & Paper.....	430	Proposal	12/11/80	46 FR 1430	01/06/81
	& 431	Promulgation	10/29/82	47 FR 52006	11/18/82
		Proposal - (PCB's)	10/29/82	47 FR 52066	11/18/82
° Steam-Electric.....	423	Proposal	10/ 3/80	45 FR 68328	10/14/80
		Promulgation	11/ 7/82	47 FR 52290	11/19/82
° Textile Mills.....	410	Proposal	10/16/79	44 FR 62204	10/29/79
		Promulgation	8/27/82	47 FR 38810	09/02/82
° Timber.....	429	Proposal	10/16/79	44 FR 62810	10/31/79
		Promulgation	1/ 7/81	46 FR 8260	01/26/81

* Administrator's signature; ( ) is the projected schedule approved by the Court on August 25, 1982 and October 26, 1982; [ ] is the revised schedule, pending approval by the Court, that has been requested by EPA (R. Hanmer Affidavit, June 21, 1983).

Attachment D:

CONTROL OF INDUSTRIAL WASTEWATER DISCHARGES
IN THE UNITED STATES*



*James D. Gallup, USEPA

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COARSE BUBBLE TO FINE BUBBLE AERATION RETROFIT

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COARSE BUBBLE TO FINE BUBBLE AERATION RETROFIT

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ABSTRACT

A fine bubble diffuser system retrofit at the Metropolitan District Commission - Hartford Water Pollution Control Plant, shows promise of a two and one-half year payback on an investment of $600,000. The 45 million gallon per day activated sludge process was designed with a coarse bubble diffuser system operating at 7% oxygen transfer efficiency. The system has been replaced with fine bubble ceramic dome diffusers at 27% oxygen transfer efficiency.

The reseach phase of the retrofit program included; investigation of high efficiency oxygen transfer systems, pilot testing of the most promising technology, and examination of the interfacing of existing and new equipment. Design recommendations were concurrently prepared by MDC staff, consultant and equipment manufacturers.

The retrofit construction began in mid 1982 and completed in November. The transitional phase to fine bubble operation and the following five month period presented numerous operating problems which were systematically corrected, resulting in significant savings.

The savings realized from a baseline period of operations were a 22% overall reduction in total plant power consumption and a 54% decrease in the 3,000 horsepower blower power consumption. The projected cost savings for 1983 is $200,000.

INTRODUCTION

From plant start-up in 1972, the coarse bubble diffusser system supplying air to the activated sludge process at the Hartford Water Pollution Control Plant has been an energy intensive operation. This situation has resulted primarily from the fact that one 3,000 horse power blower, rated at 66,000 standard cubic foot per minute, (SCFM) has not provided sufficient air for the 60 MGD activated sludge process. While there has been the capability of placing a second blower on line which would provide the required amount of air, the electrical cost associated with operating two units simultaneously has been prohibitive.

More specifically, the annual electrical consumption at the plant was slowly increasing between 1973 and 1979 (see Figure 1). Between 1979 and 1982 a large decrease in energy use was realized through the upgrading of the sludge handling operation to belt filter presses and a new incinerator operating mode. Despite this reduction in electrical energy consumption, annual electrical costs continued to increase through 1982. With forecasts of continued electrical rate increases and with continued high air consumption in the activated sludge process, the staff of the Hartford Water Pollution Control Plant embarked on a technology investigation to reduce the air consumption by upgrading the efficiency of the system.

Preliminary investigations quickly indicated that the coarse bubble diffuser system was providing a very low transfer efficiency. New air diffusion processes were considered and pilot testing begun. In conjunction with the equipment manufacturers and the consultant, a fine bubble system was chosen which would enhance the oxygen transfer, reduce air consumption, decrease electrical requirements, eliminate the use of the 2nd blower and, finally, maintain control of our operating budget.

After selecting the most efficient fine bubble system, plans and specifications were prepared for bidding purposes. The Contract was awarded to the Norton Company the successful bidder. The new system was installed in mid 1982 and placed in full operation in November.

Since full implementation the overall savings in air consumption, energy and cost have been most impressive, (as indicated in Figure No. 1). The projected electrical savings for 1983 for example, is approximately $200,000 with a payback period of 2-1/2 years.

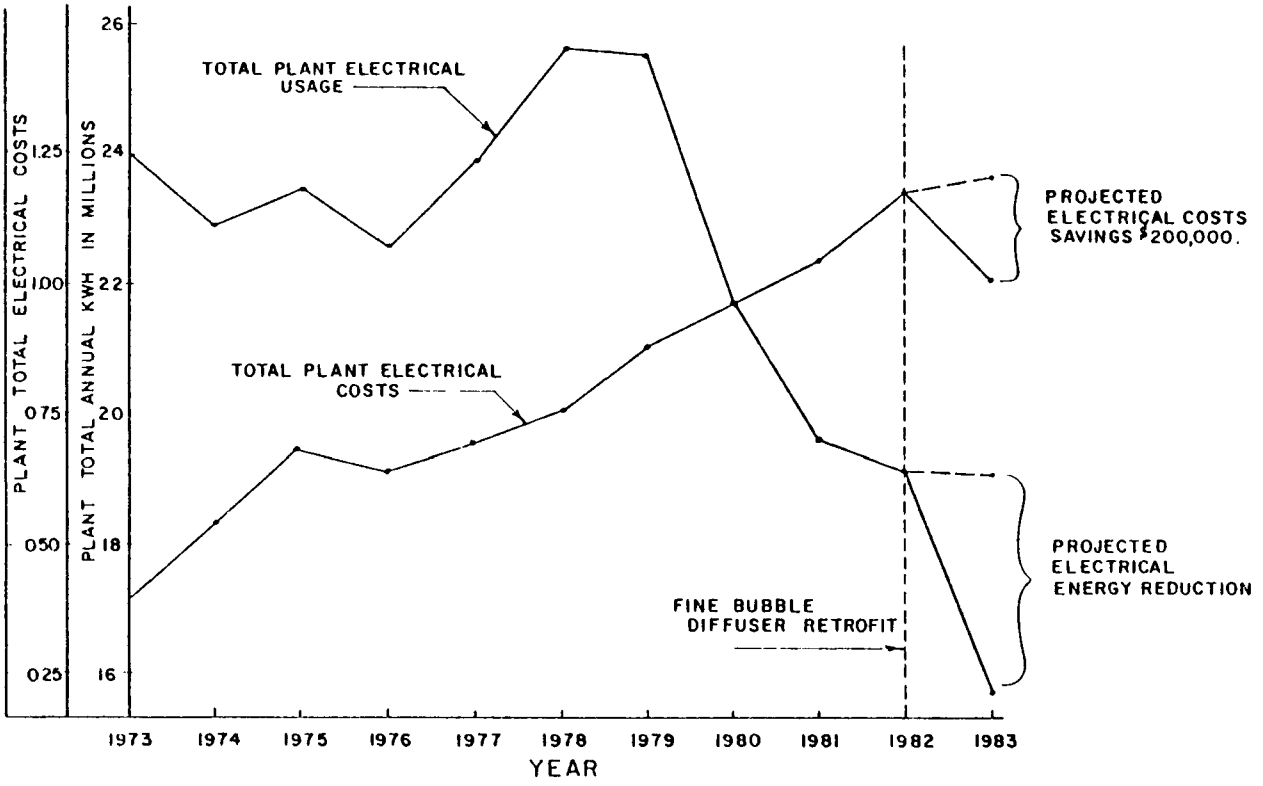

FIGURE 1 ANNUAL PLANT ELECTRICAL CONSUMPTION & COSTS

BACKGROUND

The Hartford Water Pollution Control Plant (HWPCP), designed in 1968 and completed in 1971, treats waste water from six greater Hartford area towns with an average daily flow of 45 million gallons daily (MGD). The plant handles preliminary, primary and secondary activated sludge treatment. The activated sludge and sludges from three additional water pollution control plants are pumped to dissolved air floatation thickeners and subsequently blended with primary sludge. This homogenous mix is then pumped to belt filter presses, after which it is incinerated in two of our three multiple hearth incinerators.

ACTIVATED SLUDGE PROCESS

The activated sludge process is designed for 60 MGD with a current average flow of 45 MGD. The flow from primary treatment is pumped to secondary then split into six aeration tanks. Each tank is 194 ft. long, 80 ft. wide and 15-1/2 ft. deep. The tanks are divided into 4 passes each to permit step-feed. Return sludge is fed into the head end of pass No. 1. Primary sewage is normally split among passes No. 1-2-3- and 4. (see Figure 2).

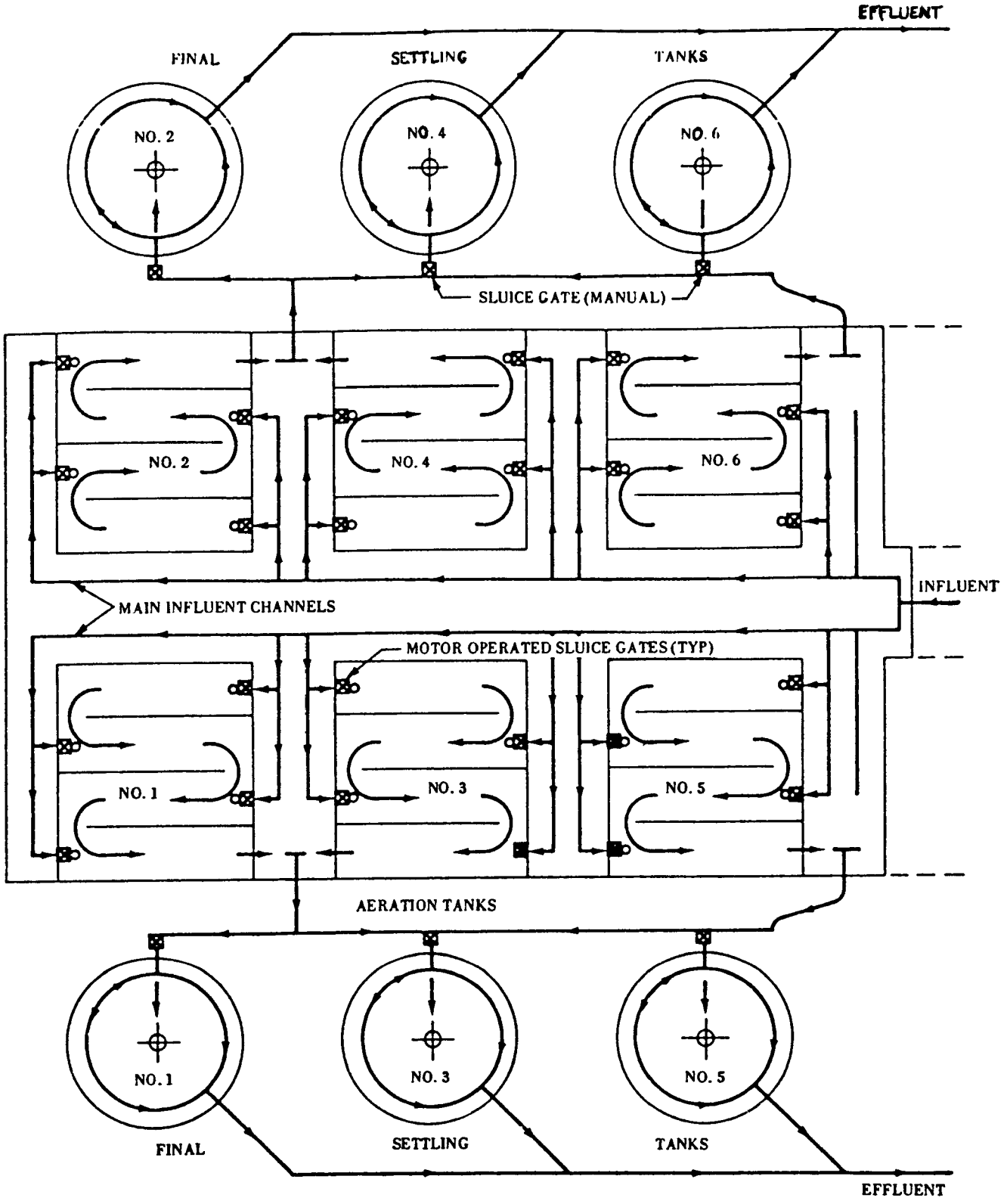

FIGURE 2 SECONDARY TREATMENT SYSTEM

Having the flexibility of a Step Feed configuration is highly beneficial. When the MLSS (mixed liquor suspended solids) is high, the step configuration that is set tends toward a contact mode of operation to relieve the solids loading rate on the final clarifiers. With the MLSS at a desirable range, a more conventional mode is used to shift solids from the aeration tanks to increase clarifer loadings and improve the density of waste sludge to thickening.

Due to Biochemical Oxygen Demand (BOD) and hydraulic loadings, only four out of the six aeration tanks have been operational. The mixed liquor from the aeration basins flows to six circular 125 ft. diameter clarifiers. The overflow from the clarifers is chlorinated between May and September and discharged to the Connecticut River.

The sludge from the final clarifiers is returned to a common wet well, at which point 95% is returned to the aeration system, at a rate equal to 25% of plant flow the balance is wasted to the dissolved air floatation thickening system.

Due to a prior inefficient sludge dewatering operation, which prevented sludge from being wasted in the correct amount, the MLSS and MCRT (mean cell residence time) were far above design. As a result, foam in large amounts was always present on the surface of the aeration tanks, to the point where the foam would infiltrate into the Y-walls. This caused handling problems in the galleries below.

Large quantities of scum were also always present on the clarifiers, creating serious problems in the winter months. The formation of "scumburgs" would trip the collection equipment, necessitating the use of a crane with a clamshell to remove the frozen scum from the clarifiers.

Due to the belt filter press retrofit of the last three years, it has been possible to more efficiently process the sludge, thereby reducing MLSS, MCRT, and sludge inventories. This has benefitted the process tremendously and also helped to solve foam and scum problems.

AIR SUPPLY

The air for the aeration system, as mentioned earlier, is continuously supplied by one of three Brown-Boveri Rotary Vane Blowers. Each unit is rated at 3,000 HP with a maximum output of 66,000 SCFM at 7.5 psi.

The initial air diffusion equipment, installed in 1972, was a Chicago Deflecto Diffuser, a coarse bubble device with large 3/8 in. diameter orifices on the periphery of the diffuser. This diffuser's oxygen transfer rate when tested was found to be 7%; however, with energy being so inexpensive, emphasis was placed on a uncomplicated system that required little maintenance. The spiral roll created by the design required only 252 diffusers per pass and also kept the tank floor free of equipment, facilitating cleaning when necessary. Air filtration was provided by a series of oil cleaned screens.

From the beginning, the one compressor always operated at full capacity; yet it never fulfilled the DO (dissolved oxygen) requirements in the aeration system, particularly in the first pass and during the summer months. Faced with both the expensive possibility of placing the second compressor on line and steady electrical rate increase, the Hartford Water Pollution Control Plant staff began its research and investigation into a more efficient system in early 1978.

RESEARCH AND INVESTIGATION

The objective of the research phase of the project was to determine and select the most efficient, cost effective and compatible air diffusion equipment available. In order to carry out this task, research and investigation were divided into the following categories:

- °Available Technology Investigation
- °Pilot Testing
- °Potential Air Usage with New Technology
- °Main Blower-Turndown Evaluation
- °Electrical Power Monitoring
- °Piping System Investigation
- °Air Filtration Requirements
- °Instrumentation System Requirements
- °Systems Evaluation
- °Final Report and Recommendations

AVAILABLE TECHNOLOGY INVESTIGATION

Manufacturers, suppliers, consultants, and end users were contacted in order to gather as much information on the various types of available diffusion equipment. Realizing at the onset of our research that mechanical aerators and jet aerators would not be compatible with our system, we proceeded to investigate ceramic and plastic tubes, static mixers, ceramic dome diffusers, etc. Preliminary investigations pointed to the ceramic dome as being an efficient system; it was, therefore, chosen for testing.

PILOT TESTING

Our first test was basic in that it consisted of mounting a single ceramic dome on the bottom of a large container and connecting it to an air supply. Observations of the bubble size, formation, patterns, etc. were made and compared to the performance of a coarse bubble diffuser. The comparison looked promising enough to proceed.

The second test consisted of evaluating the performance of two side by side 55 gallon drums outfitted with an air supply, air flow gauges, and a portable dissolved oxygen (DO) meter. This investigation compared "head on" the coarse bubble diffuser against the fine bubble dome. With the drums filled with activated sludge, and air flows held constant, the fine bubble diffuser dome set-up would rapidly develop and maintain at least double the DO level of the coarse bubble diffuser. Conversely, if air flows were adjusted to maintain the same DO in each drum, the dome diffuser required one half as much air as the coarse bubble diffuser.

The original test stand compared one dome against one coarse bubbler. This was not realistic from a design standpoint because one dome required a higher operating pressure than is required by one coarse bubble diffuser.

A third test was conducted with the use of a 300 gallon activated sludge pilot plant, in which could be mounted up to 4 domes and one coarse bubble diffuser, piped independently. The tank was set up as an aeration tank with a continuous flow-through of activated sludge from one of our operating plant aeration tanks.

The tests though essentially a repetition of the barrel studies, were run at the more realistic fine bubble dome to coarse bubble diffuser ratios of 2:1 and 4:1. The two to one ratio turned out to be a lower ratio than necessary; however, sufficiently realistic data was generated by the 4:1 arrangement to estimate the savings of air usage in a full plant scale up. Each diffuser system in the tank was run alternately over short periods of time in order to maintain a steady DO.

The amount of air required by each diffuser system to maintain the steady DO was recorded. Several runs were made and savings in air usage ranged from 50-60%

One unfortunate characteristic of the fine bubble system was the increase in foam experienced in the 300 gallon pilot tank. Since the new dewatering operation with Belt Filter Presses would allow the maintenance of lower MCRT's (by increased wasting) it was felt that this problem could be minimized.

POTENTIAL AIR USAGE WITH NEW TECHNOLOGY

The pilot tests indicated that there would be a 50 to 60% total air reduction. The design data indicated that approximately 8,000 SCFM was required to maintain the solids in suspension in the influent channels, with 14,000 SCFM necessary for aeration. The total anticipated air usage ranged from 18,000 to 22,000 SCFM. With the main compressor normally operating at 66,000 SCFM, the next concern was the turndown capability of the main compressor.

MAIN BLOWER - TURNDOWN EVALUATION

With assistance from an engineer from Brown Boveri (blower manufacturer), the capability of the rotary vane 3,000 HP blower was reviewed. Its surge point, efficiency and power consumption were studied and a procedure was then devised to reduce the output of the blower while maintaining a fixed discharge pressure.

From the initial 66,000 SCFM the blower discharge was slowly reduced to 10,000 SCFM. The unit never reached its surge point and performed well throughout the range, with a minimal loss in operating efficiency.

ELECTRICAL POWER MONITORING

Early in the project, it was realized that it would be necessary to determine the energy usage of the blowers. Therefore, the installation of a watt meter and the plotting of daily kilwatt-hour readings and total air usage was begun. This base line data was necessary to determine what portion of the entire plant's electrical power consumption could be attributable to the blower, in order to accurately calculate potential savings.

PIPING SYSTEM INVESTIGATION

The existing air piping system in the Hartford Water Pollution Control Plant is constructed of spiral welded steel and wrought iron pipe. Some corrosion, rusting, and scaling of these types of pipes usually occur after a few years of service; they do not, however, normally cause operating problems in coarse bubble aeration systems. In order to be certain that the existing air pipes were rust free and capable of being used for fine bubble diffusers, it was necessary to field inspect.

To perform the inspection, the main blower was shut down so that entrance to the piping could be gained through an access manhole on the 5 ft. diameter discharge air main. The piping was examined from the compressor down to the 2 ft. diameter branch lines. Since none of us was slim enough to get through the 12 in. lines, a few fittings were removed to permit inspection. This was also done for the 6 in. drop headers. The suction lines from the air filters to the compressor were also closely inspected.

The results of the inspections revealed the following:

- °The bituminous epoxy coating in the air mains and suction lines were in excellent condition, and would be suitable for the fine bubble system.

- °The coating in the 12 in. lines in tank Nos. 1-2-3-4 was found in good condition. The coating in the similar lines in tank No. 5 & 6 was damaged with resulting rusting and scaling. This situation was caused by the fact that these tanks had not been in use and would remain empty for the near future.

- °The 6 in. drop headers were rusted and would require replacement for use with the fine bubble system.

AIR FILTRATION INVESTIGATION

The fine bubble ceramic domes and tubes would require removal of 95 percent of all particles in the inlet air down to particles 0.3 microns in size. The existing automatic oil bath filters were capable of removing only about 25 to 30 percent of the particles and, therefore, were not useable with the fine bubble system. Several alternative systems were evaluated including American Air Filters' Biocell and Electro-pak filters.

The Biocell filters which would be installed inside the existing inlet plenum, would require a structural frame. There was sufficient space in the inlet plenum to permit the modifications. The Electro-pak, is an electrostatic precipitator, because of its expense and large size, was not deemed practical for the installation.

INSTRUMENTATION SYSTEM REQUIREMENTS

The air control system was designed with flow control and flow indicators to each pass of every aeration tank. A DO probe senses the amount of dissolved oxygen in the pass, the signal is fed to an analyzer, then to an electronic controller where the DO is manually set. The Output signal from the controller modulates a 12 in. motor operated butterfly valve which controls the amount of air to match the set-point DO.

As each pass is modulated, the main venturi senses the change in pressure and flow in the entire system. A pressure controller modulates the vane in the compressor to maintain a constant system pressure.

With low air flows anticipated due to the retrofit to the fine bubble system, the range efficiency of the instrumentation system and the effects of low air flows on the DO probes were potential problems. The equipment manufacturers were contacted and given the projected operating parameters. They recommended that the existing range tubes in each flow transmitter be

replaced with new range tubes so that the transmitters would be sensitive to future operation with the reduced air flows. Recalibration of the transmitters would be necessary. In addition, the air flow totalizer gears and the air flow indicator scales for each flow controller would also require replacement.

The DO probe manufacturer indicated that, despite the change from a spiral roll to vertical uplift diffusion and smaller bubbles, the existing probes would function satisfactorily.

As a follow-up to the study conducted by the Metropolitan District, Metcalf & Eddy Consulting Engineers, the original designers of the plant, (M & E) were retained to evaluate the existing coarse bubble diffuser system as well as the proposed installation of the fine bubble ceramic dome and tubes diffuser systems. The M & E evaluation would include investigating the oxygen transfer efficiences of these systems, present and projected air requirements and power usage of the three systems, present and future wastewater flows and load projections and the system's relative cost effectiveness over a planning period of 20 years (1982 - 2001).

Based on the cost-effectiveness study, some of the major conclusions were:

1. For the future operating conditions expected at the Hartford Water Pollution Control Plant, clean water oxygen transfer efficiences for the existing coarse bubble would be 7%; the proposed fine bubble tube efficiencies would be 15% and the efficiencies for the fine bubble ceramic domes would be 29%.

2. Minimum air requirements for the fine bubble domes would be dictated by the air needed for mixing to keep the MLSS in suspension.

3. Peak air demand for proposed fine bubble domes could be supplied by one existing blower throughout the planning period.

4. Peak air demands for the existing coarse bubble system, as well as for the proposed fine bubble tubes, would require simultaneous operation of two blowers during warm summer months.

5. Even with increased submergence and increased headlosses with fine bubble domes and tubes, the total system head on the compressors would be well within the capacity of the existing blowers.

6. Blower surging would not be expected to occur for the expected air demand with the proposed fine bubble systems.

7. The capital as well as operating costs of the fine bubble domes would be smaller than those for the fine bubble tubes.

8. Due to higher initial capital expenditures and higher operational costs of the fine bubble tubes (resulting from low oxygen transfer efficiencies), the fine bubble domes would have a distinct cost advantage over the tubes.

FINAL RECOMMENDATIONS

On the basis of testing, inspections, research, analysis and evaluations of the Hartford Water Pollution Control Plant operations, and analyses and evaluations of proposed fine bubble diffuser systems and their related cost effectiveness, it was recommended that the MDC:

1. Initiate design to prepare plans and specifications to retrofit four of the six aeration tanks to fine bubble ceramic domes.

2. Modify the existing valves and rate controllers on air lines to each individual pass of each tank and replace the associated range tubes in the transmitters. Change the air flow rate indicator scales.

3. Install bio-cell air filters in all three inlet plenums of the three compressors.

DESIGN CONSIDERATIONS

The layout and design of the fine bubble dome diffuser system was such that a maximum portion of the existing air piping was utilized. Likewise, the submerged air distribution piping feeding air to the ceramic diffusers was designed to facilitate an easy and economical installation. When fully operational and treating a similar strength and volume of wastewater as the previous coarse bubble diffuser system, the system would require only approximately 50% of the required coarse bubble horsepower.

Contract specifications required that, as a minimum average, 7,550 lbs. O_2/day had to be transferred to the wastewater, while 14,000 lbs. day was needed to accommodate the maximum average loading. Other wastewater parameters, such as alpha .75, beta = .95, D O = 2.0 mg/1 and Tw=15° C, were also specified as well as the airflow distrubtion per pass for each of the four passes per tank. The total air supply to each aeration tank was specified to be distributed uniformly in each pass, according to the following proportions in conjunction with the minimum diffusers:

PASS	PERCENT DISTRIBUTION	MINIMUM DIFFUSERS/TANK
#1	35%	1,085
#2	25%	775
#3	25%	775
#4	15%	465
Total Per Tank	100%	3,100

The required minimum number of domes were 12,400 for the total system of four tanks.

The selected fine bubble diffuser system had to have guaranteed oxygen transfer efficiencies of 28%, 26% and 23% at diffuser flow rates of .5, 1.0 and 2.0 scfm/dome, respectively. The minimum air flow criteria of 0.5 scfm/dome was established to prevent water side fouling. Furthermore, a mixing requirement of 0.12 scfm/ft^2 of tank bottom was established along with a minimum dome spacing of 2 feet on center. The mixing requirement had to maintain solids in suspension and prevent solids accumulation on the tank floor.

The most efficient dome layout would require uniform air flows through each dome, while satisfying the natural variation in air demand along the length of a step feed aeration tank. Locations with high demand would need proportionately more domes than locations with low demand. Therefore, the dome distribution pattern for tanks one and two was based on oxygen uptake studies.

The studies were conducted by District staff at sixteen locations along the four passes of one tank. These studies indicated the need for a tapered distribution pattern. The first design pattern was based on this tapered distribution and other previously indicated design constraints.

Possible alterations of the step feed configuration was another design consideration. Additional base plates were available in most passes for future modifications of dome patterns and each of the seven grids in a pass was to have a throttling valve for fine tuning.

The first two operational tanks were to be field tested and additional modifications would be designed into tanks three and four as needed.

DO measurements were taken in tanks one and two as they came on line with the fine bubble system. Operational changes since the initial design stage had caused a shift in the demand pattern. The DO profiles indicated that more domes were necessary at the influent end of the passes.

Redistribution of the domes was easily accomplished in tanks three and four by use of the spare base plates available in the original design.

SYSTEM HARDWARE

The air distribution piping as installed has the built-in capability for expansion to accommodate additional diffuser domes since extra (50%) dome baseplates were initially fabricated to the plastic piping. Diffuser locations not in use were made inoperative by simply plugging off the air release holes. The plastic air distribution piping network was easily assembled using 4 in. UPVC (unplasticized polyvinyl chloride) pipe and 4 in. (UPVC) expansion tees. The system is installed one foot off the tank floor thus providing a 14.5 ft. vertical clearance to the water surface. Pipes and fittings are supported and anchored to the concrete floor using a Norton designed adjustable stainless steel pipe support assembly.

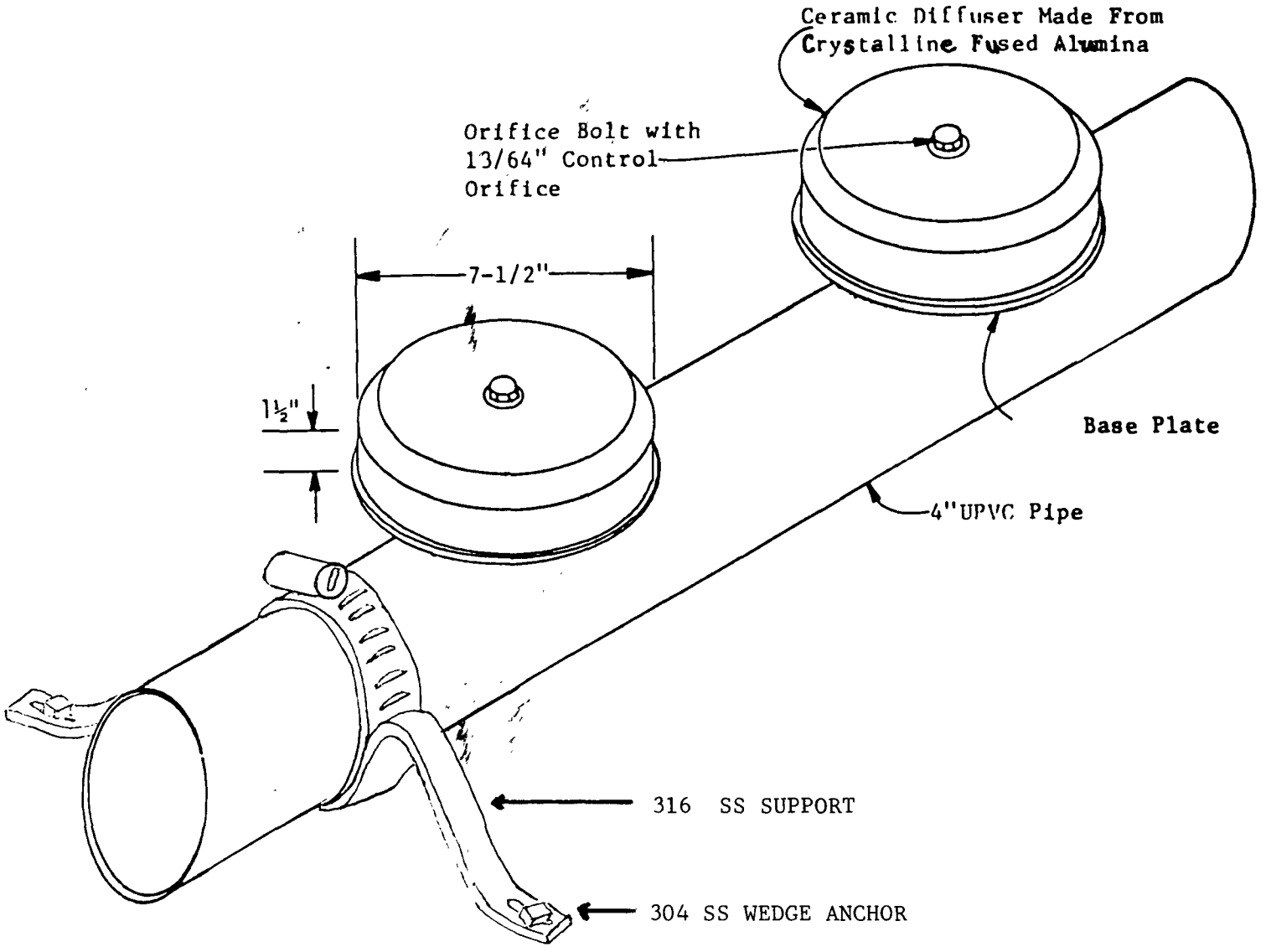

FIGURE 3 TYPICAL DOME DIFFUSERS & PIPING

Two 1/4 in. diameter stainless steel anchor bolts secure the pipe to the floor while a stainless steel band holds the 4" UPVC to the pipe support. Air is delivered to each of the seven air distribution grids per pass by a single vertical 4 in. stainless steel pipe which drops directly into a 4 in. UPVC tee. Air emerges from the piping network up through a plastic bolt having a specified orifice size (13/64 in. dia.). This orifice bolt also serves as a means of attaching the porous ceramic dome diffuser to its plastic baseplate. Neoprene sealing gaskets are placed under the head of the orifice bolt and between the dome edge and the UPVC baseplate (see Figure 3). Ideally, only fine bubbles on the order of 2mm will be emitted from the pores of ceramic diffusers. In fact, the many small oxygen bearing bubbles that are released into the wastewater comprise the main reason for a very high system oxygen transfer efficiency. The Hartford aeration system, with its numerous diffuser domes, provides highly efficient oxygen transfer and mixing.

Each of the independent UPVC air distribution grids contains a simple moisture blowout assembly which removes any residual moisture after the start of airflow to the system.

INSPECTION & TESTING

During all phases of construction, MDC personnel continously inspected the performance of the independent contractor. It was most important that the system be installed exactly as designed, to assure an efficient operating system.

As each piping network was installed, alignment and elevations were verified. The ceramic domes were thoroughly inspected prior to installation and further checked in place for proper gasketing and bolt torque.

Upon completion of each tank, clean water was metered to 3-in the piping for a system leakage test. With corrections made, the water level was slowly increased to full depth while observing the air distribution patterns. The tank was then stabilized and placed into operation.

OPERATIONAL RESULTS

As shown in Fig. 4, baseline data prior to the fine bubble retrofit spans the period of mid-May to August 1982. Air usage averaged 64 MCFD (Mil, Cu. ft./day) and Power usage average 31,000 KWH/day during this period.

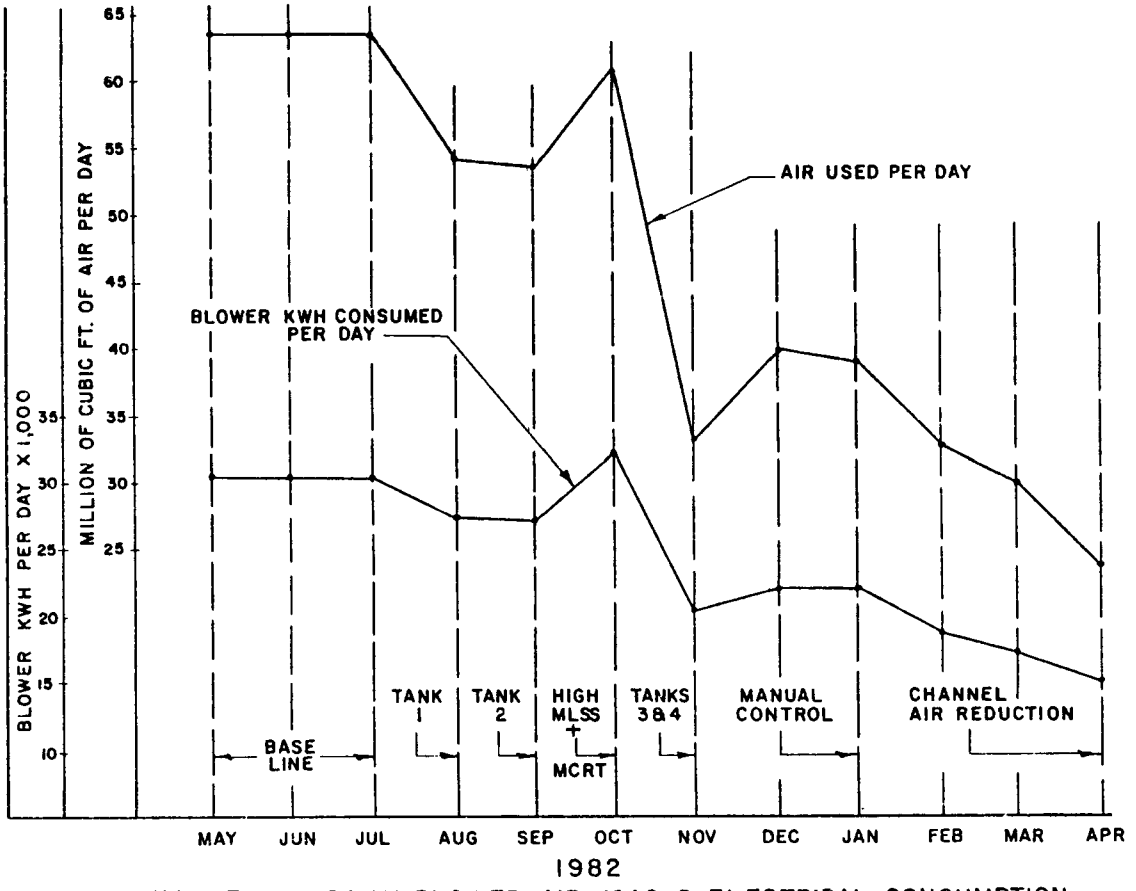

FIGURE 4 DAILY BLOWER AIR USAG & ELECTRICAL CONSUMPTION

During this same period MLSS averaged between 2500-4000 mg/L; the blower discharge pressure ranged between 7.5-7.7 psi. The air control was on automatic, with air valves controlled by signals from DO probes in each pass which maintained an averaged of 1 ppm DO.

When the first fine bubble aeration tank was placed on line on August 9, the air usage immediately dropped to 54 MCFD; power usage dropped into the 28,000 KWH/day range.

The second fine bubble aeration tank went on line September 2; during this time, MLSS concertrations had been rising, and a small initial drop in air usage was followed by an intentional increase over the previous base line value. As the MLSS rose into the 5000 MG/L range, increased air requirements began to exaggerate the different pressure requirements of the coarse and fine bubble systems which were on line concurrently with the same automatic control system. The resulting anaerobic conditions in much of the fine bubble system caused a bulking condition with subsequent loss of solids in the

effluent. The air supply to the entire system was increased until a solution to the distribution problems could be found. During this time sludge was vigorously processed to return MLSS to a 2500 MG/L design range and to ensure that excess DO was established throughout the system. After this operational control problem, the air usage was decreased to seek an economoical operating condition.

On October 28, the installation of the entire project was completed and the fine bubble operation established in all aeration tanks. When fully on-line, air usage dropped to 36 MCFD and the power fell to the 22,000 KWH/day range. This range was maintained until year's end while working on the automatic air controls.

During the first week of 1983, a brief increase in MLSS caused air distribution problems with a consequent large increase in KWH/day. It became apparent that the automatic air control in its present state would not allow further reductions in KWH. The very act of controlling the system caused the blower to work under a greater load than was necessary to delivery an adequate air supply. Some passes were also being locked out by the automatic equipment which increased the risk of water side fouling on the fine bubble diffusers.

In mid-January the system was placed in the manual mode of operation, with DO probes being used for monitoring purposes only. In the automatic mode the discharge pressures on the compressor varied between 7.1 and 7.4 psi. After the switch to the manual air control operation, the blower's discharge pressure was adjusted to 6.8 and all air control valves were locked fully open. Air usage dropped slightly but power consumption fell by over 10%; further experiments with the blower vane position control resulted in another 10% reduction in power.

In mid-April, influent channel air consumption was carefully reduced by one-half. The resulting surplus in air to the aeration tanks allowed further air reductions to 22 MIL cu. ft./day and power reduction to 15,000 KWH/day.

Foam problems have always existed and were experienced during the entire baseline period of operation; then became worse as the fine bubble tanks came on line.

In mid-January, after a short period of time at an MCRT of 3-5 days, foam problems disappeared completely. The only hint of a reoccurrence of the foam resulted from an increase in MCRT above the 3-5 day range.

Placing the fine bubble system on line on an incremental basis caused numerous operational problems, as just described. However, the activated sludge process has been stabilized and is performing exceptionally well.

SAVINGS

The following figures serve to demonstrate the electrical savings and annual cost savings due to the retrofit program.

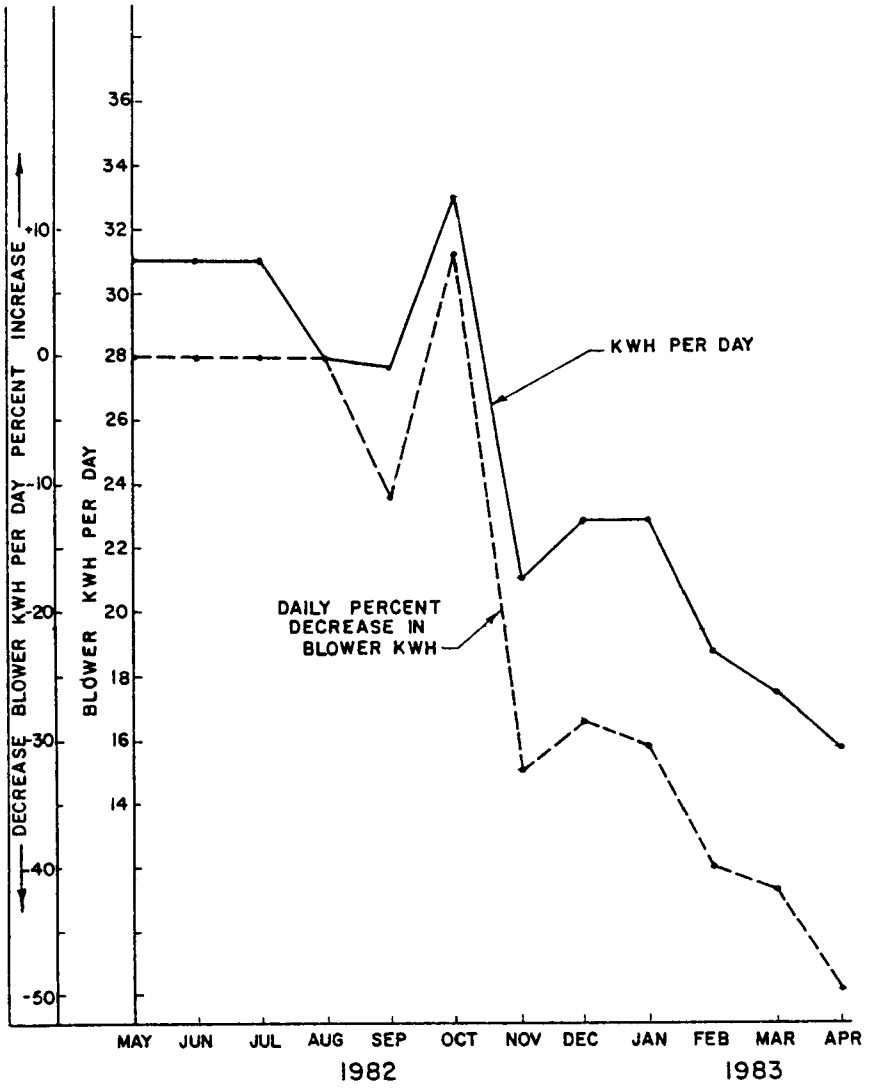

FIGURE 5 DAILY BLOWER ELECTRICAL CONSUMPTION & PERCENT REDUCTION

Figure 5. Daily Blower Electrical Consumption & Percent Reduction

The base line period energy consumption for the 3,000 HP blower was 31,000 KWH/day. As tanks were retrofitted, the consumption dropped to 15,000 KWH.

Figure 6. Daily Plant Electrical Consumption & Percent Reduction

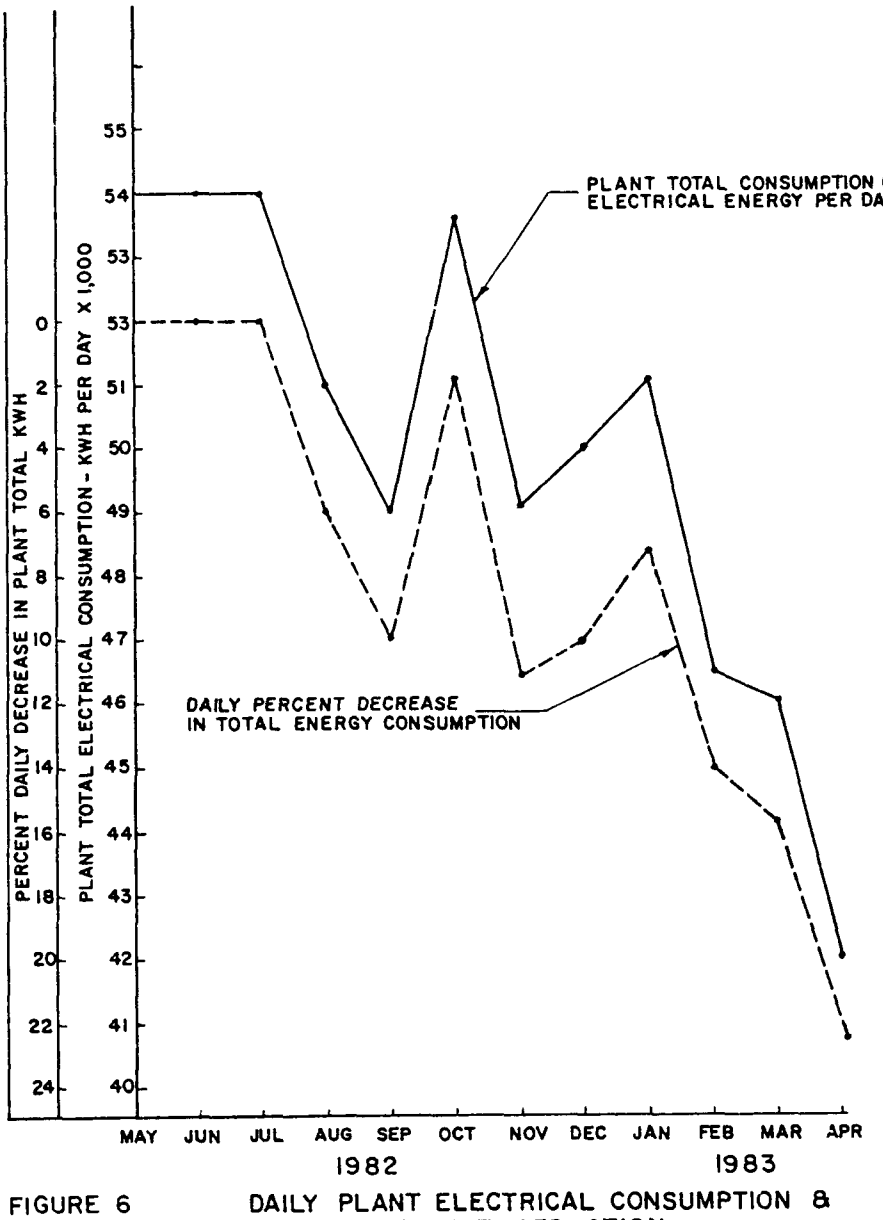

FIGURE 6 DAILY PLANT ELECTRICAL CONSUMPTION & PERCENT REDUCTION

During the baseline period (May-July) the total plant electrical consumption was 54,000 KWH/day. This total includes power for Preliminary treatment, Primary Treatment, Effluent Pumping, Secondary Treatment and Sludge Processing. The retrofit program has reduced the total plant electrical consumption to 42,000 KWH/day, a 22% overall decreases.

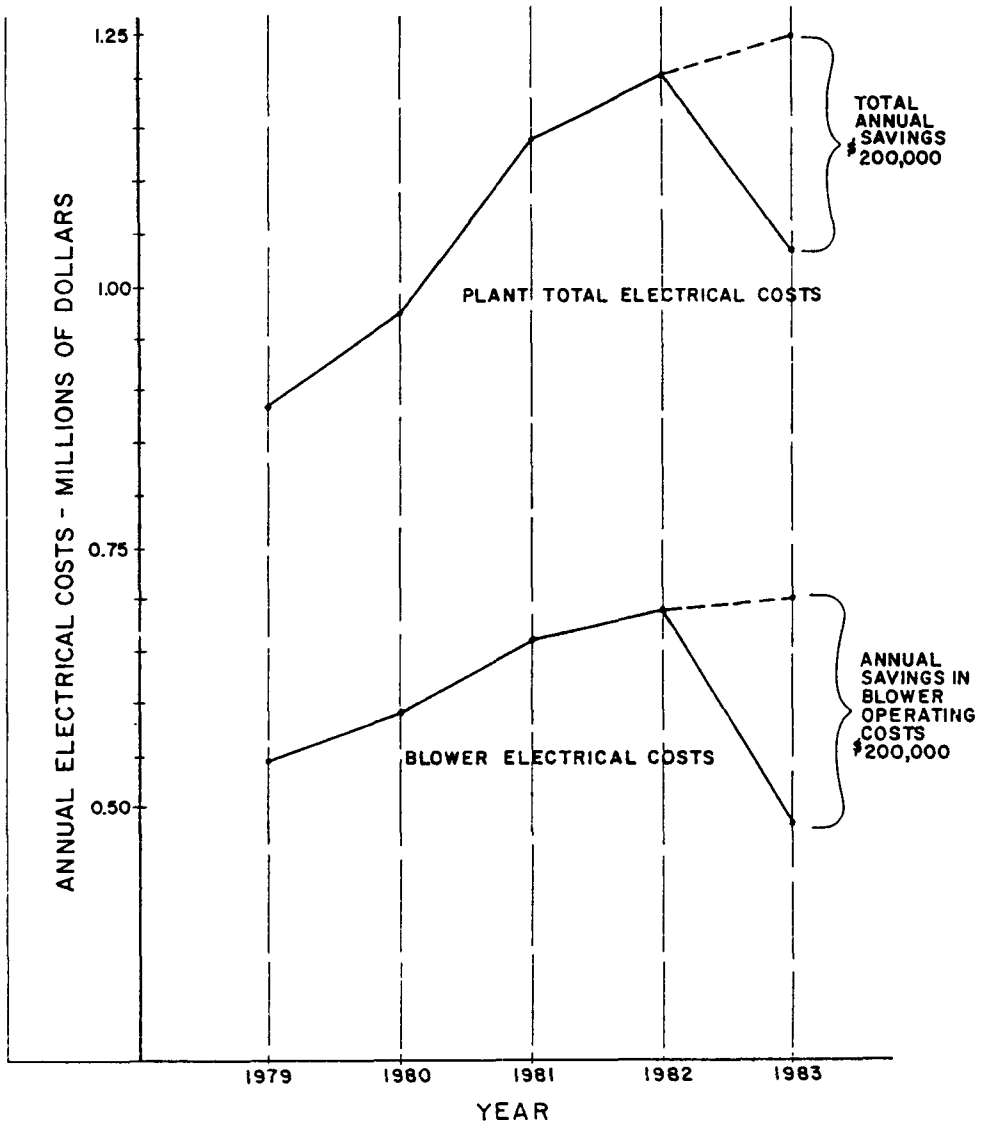

FIGURE 7 ANNUAL PLANT& BLOWER ELECTRICAL COST

Figure 7. Annual Plant & Blower Electrical Costs

This figure demonstrates the continuous rising trend in electrical costs over the past few years, up to 1983. The broken line indicates the continued increase in costs for the plant and blower coarse bubble had continued. The solid line represents the projected electrical costs with the fine bubble aeration system. Based on the above, the project electric costs for the blower will be reduced by $200,000 - and result in a like savings in the anticipated plant costs. At this continued rate, the payback period will be 2-1/2 years.

CONCLUSIONS

The Metropolitan District's fine bubble retrofit program represents the coordinated efforts of the MDC's management and staff, and our consulting engineers, manufacturers and contractor. As you have seen, the investment has been very beneficial, on a cost benefit basis, and the upgrading of our system has improved the efficiency of our operation.

The entire undertaking has been a learning process. Placing the system on line and making the transition to fine bubble aeration reminded us of the initial phases of start-up when the Hartford plant was initially placed in operation in the early 1970's.

As we continue to fine tune the new system, we are planning the following associated projects:

°Computer Feasibility - the Norton Co., has been retained to investigate the application of a computer to monitor the status of all 16 DO probes, acquire operating data, and eventually apply the computer to system control.

°Grid Adjustment - Plans are being made to empty tanks 1 and 2 to readjust the grid pattern to match tanks 3 and 4. This will also give us the opportunity to inspect the system.

°Manometers Installation - Manometers will be installed on all 16 flow venturies to permit the operators to personally check the accuracy of the instrumentation. Operation at air flows close to water side fouling limits requires an accurate mode of operation.

°Retrofit Investigation at Satellite Plants - With two satellite plants operating with coarse bubble aeration systems, the acquired knowledge will be utilized to investigate the potential retrofits to fine bubble at these plants.

The work described in this paper was not funded by the U.S. Environmental Protection Agency, and therefore, the contents do not necessarily reflect the views of the Agency and no official endorsement should be inferred.

7/20/83

CLEAN WATER IN WISCONSIN

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CLEAN WATER IN WISCONSIN

Paul N. Guthrie, Jr.*

Wisconsin Department of Natural Resources

The adoption of the United States Constitution, coming after the failing years under the Articles of Confederation created an opportunity for states to act in unison and for the United States Congress to act nationally without inhibition caused by a single state's political desire.

Over the nearly 200 years since the creation of the Constitution, the national government has evolved into a strong, centralized power. Yet, ironically, the decentralized administration of many national laws allows individual states, along with their local subdivisions and citizens (rightfully clothed with a democratic cynicism of central authority) to engender a vast diversity in implementation of national programs.

Thus, in each section of the United States, local background and history coupled with state law and political nativity act together to fulfill the national interest. Regionalism is a reality in the administration of national environmental objectives, and in the pursuit of clean water in Wisconsin.

Water Resources and Wisconsin

In order to understand the evolution and significance of water resource protection in Wisconsin, it is helpful to understand the resource itself. Water is plentiful in the state. It occurs in several distinctly different conditions, each with a characteristic vulnerability to use and to pollution. The three major types of surface water bodies in Wisconsin are:

		Acreage
1.	Major rivers and tributary streams	176,358
2.	Inland lakes	970,869
3.	The Wisconsin Great Lakes	6,439,700
		7,586,927

Major River Basins

The largest river in the state is the Mississippi which forms part of Wisconsin's western border. The Wisconsin River is the major Mississippi

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*Author's Note: The writer of this paper wishes to acknowledge the many persons within the Wisconsin Department of Natural Resources whose work has been summarized in this paper and to give a special note of thanks to Mary Ann Heidemann and Daniel Moran for their personal assistance and counsel.

tributary within the state, flowing as a spine through central Wisconsin. Two-thirds of the state's surface run-off drains into the Mississippi system. The remaining one-third drains into the Great Lakes, principally through the Fox River. (See maps 1 and 2)

Historically, the Fox and Wisconsin rivers have been the state's most important waterways. Near Portage, Wisconsin, these rivers lay less than a mile apart, making a water linkage possible between the Mississippi and the Great Lakes. Early exploration and settlement of the state was keyed to the exploration of the Fox-Wisconsin transportation routes. These two rivers continue to host large portions of the water and industrial development within Wisconsin, most notably the pulp and paper industry.

Small feeder streams in the upper watersheds of the state are far more likely to be affected by agricultural use and rural development than by industry. Rich soils in the southern portion of Wisconsin and proximity to major mid-western markets have combined to make Wisconsin the nation's number one dairy products supplier. Control of agricultural run-off is a concomitant water quality problem in the small and intermittent streams of rural Wisconsin.

Inland Lakes

Wisconsin boasts one of the greatest concentrations of lakes in the world--more than 14,927 at last count. These lakes are largely the result of glaciation. They formed in shallow depressions of the ground or as "kettle Lakes" in glacial outwash, marking the melting spot of large, buried blocks of ice.

While northern Wisconsin's soils are too thin, rocky and wet for most agricultural crops, the northern Wisconsin's "lake country" did support a dense coniferous forest. Early in the state's history, this forest was exploited for the region's burgeoning lumber industry. By the beginning of the 20th century, only a cut-over wasteland remained. Conservation and reforestation efforts in the beginning in the early 1900's are responsible for the resurgence of nortnern Wisconsin as a prime recreation and tourism area.

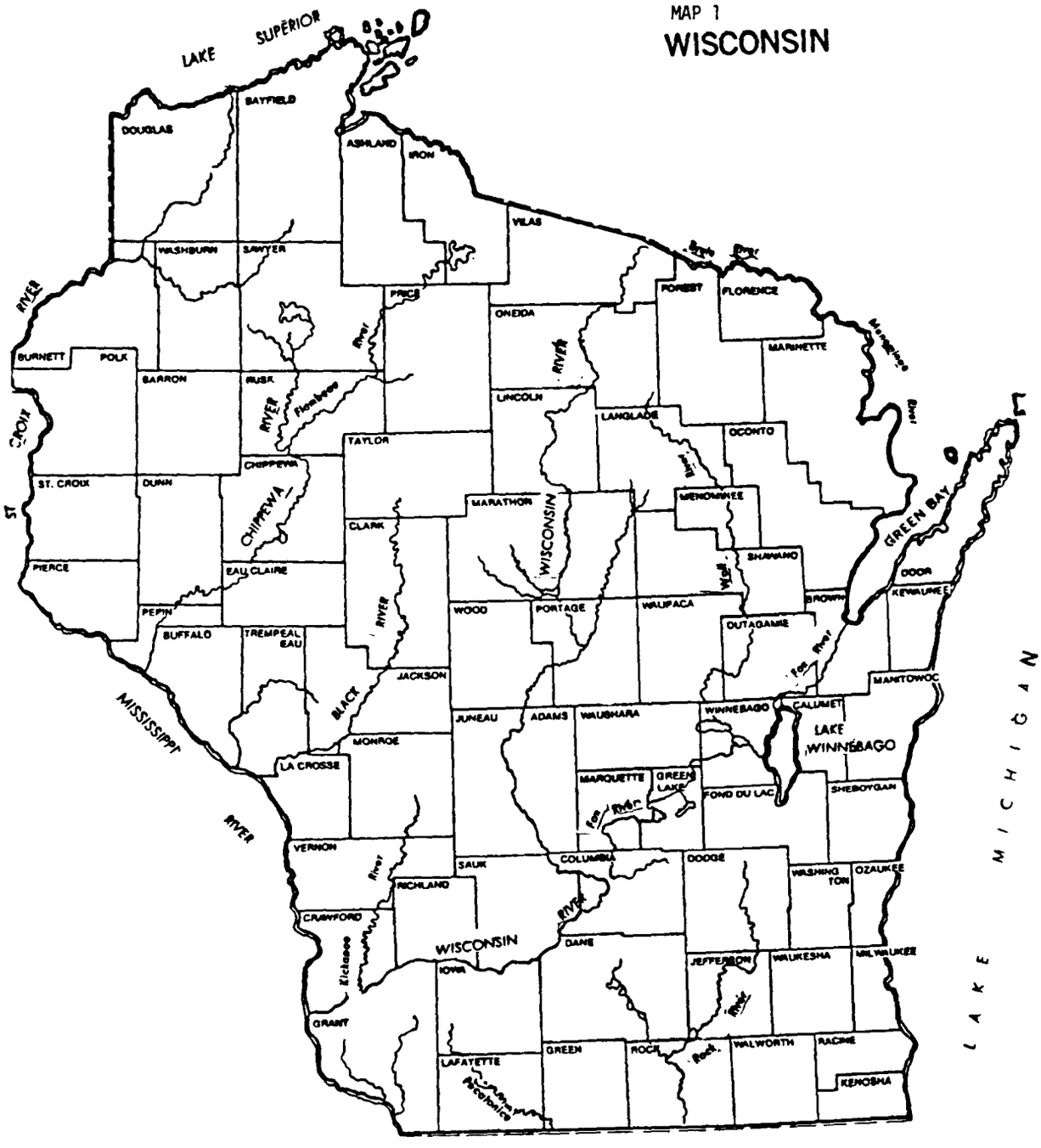
The Wisconsin Great Lakes

The Great Lakes system comprises the largest supply of fresh water in the world. With Lake Superior on its northern border, and Lake Michigan to the east, the State of Wisconsin has a special interest in the quality of the waters, and the effect that quality has on limiting resource use.

The Great Lakes were first exploited as transportation linkages. Green Bay, the largest bay of Lake Michigan, was locus for the earliest trading posts and settlements in the state. Explorers landed near the present City of Green Bay, in 1634, less than twenty years after the initial arrival of the pilgrims on the eastern seaboard. This early interest was based on the soon fulfilled hopes of finding a water route from the Lakes to the Mississippi River.

MAP 1

WISCONSIN



MAP 2

LAKE SUPERIOR
ST. CROIX
UPPER CHIPPEWA
UPPER GREEN BAY
UPPER WISCONSIN
LOWER CHIPPEWA
WOLF
TWIN-DOOR-KEWAUNEE
BLACK
BUFFALO-TREMPEALEAU
MANITOWOC
LOWER FOX
UPPER FOX
SHEBOYGAN
BAD AXE-LA CROSSE
LOWER WISCONSIN
ROCK
SOUTHEASTERN
GRANT-PLATTE
SUGAR-PECATONICA
LEGEND
State Planning Areas

Later, the Great Lakes provided cheap freight routes to ship grains and ores from the Upper Midwest to markets and manufacturing centers in Michigan, Illinois, Ohio and Pennsylvania and New York. Along with industrial development came significant water quality deterioration.

Quality Control for a Diverse Resource

The diverse pollution problems endemic to Wisconsin's inland lakes, stream systems and to the Great Lakes require a diverse set of pollution control measures. The administrative challenge, then, is to structure state law and agency program development to fit the unique needs of Wisconsin's resource base. A fortunate blend of national and state legal tradition has allowed Wisconsin to tailor a uniquely successful approach to managing the state's water resource problems.

The Public Trust

For many centuries there has existed in some western cultures, a theory and practice for the protection and control of navigable waters and of the seashores. This "public trust" doctrine has identifiable origins in early Roman law, and was perpetuated into English practice via the Magna Charta (1215).[1] The trust doctrine established a principle of the sovereign's limited and protective ownership[2] of the tidelands and the beds of navigable waters.[3]

Similarly, the public rights in water and navigation were recognized in the provisions of the Ordinance of the United States Congress in 1787, which said in part:

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[1]VED P. Nanda and William K. Ris, Jr. "The Public Trust Doctrine: A Viable Approach to International Environmental Protection". Ecology Law Quarterly 5(1976):291; University of California, Berkley.

[2]"The beds of all navigable rivers where the tide flows and reflows, and of all estuaries or arms of the sea, is by all vested in the Crown. But this ownership of the Crown is for the benefit of the subject, and cannot be used in any matter so as to derogate from, or interfere with the right of navigation, which belongs by law to the subjects of the realm". Gann v. Free Fishers of Whitstable, 11 Eng. Rep. 1305 (H.L. 1865) as quoted in Nanda and Ris, "The Public Trust Doctrine ..." page 298.

[3]See also: Victor J. Yanneconne, et al., Environmental Rights and Remedies, Vol. 1, (Rochester, New York: the Lawyers Cooperative Publishing Company, 1972) chapter 2.3, 16.

"...the navigable waters leading into the Mississippi and St. Lawrence, and the carrying places between the same, shall be common highways and forever free, as well as to the inhabitants of the said territory as to the citizens of the United States and those of any other states that may be admitted to the confederacy without any tax imposed or duty thereof."[4]

Since the earliest of times, Wisconsin's well-being has been linked to it's waters. The earliest European explorers followed the rivers, the lakes, the bays, and the portages which had been used for centuries by native inhabitants. Along these travel corridors people settled and economic activity developed. River junctions and portages were natural meeting points and modern settlements grew where these early places of commerce were situated. These realities of settlement, coupled with the provisions of the Northwest Ordinance, were passed along into the organic law of the Territory of Wisconsin.[5] Public rights in navigable waters were finally locked into the fabric of modern Wisconsin with their adoption into the State's constitution, upon entry into the union in 1848.[6]

To understand the pursuit of clean water in 1983, one must understand these historical linkages. For Wisconsin, water is the state's most important legacy and in it's pursuit of pollution abatement, not only is the state fulfilling the need for the protection of the public health of it's citizens, but is also carrying out it's responsibilities under what has become known as the public trust doctrine. Through a long history of judicial decisions unique to Wisconsin, state courts have widened the scope of the public trust, moving from a more narrow interpretation of the doctrine as a restriction on physical obstruction, taxation or impost to a

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[4]1 U.S. Stats., 51, article 4. An Ordinance for the Government of the Territory of the United States North West of the River Ohio, July 13, 1787.

[5]U.S. Stats., Chapter LIV, Sec. 12, April 20, 1836.

[6]Constitution of the State of Wisconsin, Article IX, Section 1 (1848).

view encompassing the public's right for recreation, fishing and hunting, and to the prevention of pollution.[7]

The public trust legacy in Wisconsin has not made the road to effectively managed pollution abatement an easy progression. But it is one factor, that coupled with 20th century concerns for the public health and the recent increase in public environmental awareness, that has helped to solidify public support for aggressive pollution abatement efforts.

Public Health

Second in importance to the broadened judicial backing provided by the public trust doctrine has been the slow, often eradicate but steady increase in the public perception of public health.

For example, Earl Murphy in his study cites the fact that while the use of chemicals for breaking up and disinfecting sewage was known since 1802, it was not until 1844 in Manchester, England where primary treatment using chemical precipitants, filtration, settling and agitation was employed, and in the United States in the year 1939, only 50% of the population had sewer connections, and of this 50% only one-half (e.g., 25%) had sewage treatment of any kind before discharge to water bodies.[8]

In Wisconsin, understanding was equally slow, consider the following:

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[7]"The wisdom of the policy which in the organic laws of our state steadfastly and carefully preserved to the people, the full and free use of public waters, cannot be questioned. Nor should it be limited or curtailed by narrow constructions. It should be interpreted in the broad and beneficial spirit that could rise to it in order that the people may fully enjoy the intended benefits. Navigable waters are public waters and as such should endure to the benefit of the public. They should be free to all commerce, for travel, for recreation and also for hunting and fishing..." Dana Shooting Club v. Hustings, 156 Wis. 261, 271. "We are the trustee of navigable waters within our borders, for the benefit not only of the people of our state, but the benefit of the people of the whole United States." In Re Crawford, L&D District, 182 Wisconsin 409. "This case causes us to reexamine the concept of public benefit in contrast to public harm and the scope of an owner's right to use of his property....We start with the premise that lakes and rivers in their natural state are unpolluted and the pollution which now exists is man-made. The State of Wisconsin under the Trust Doctrine has the duty to eradicate the present pollution and to prevent future pollution in it's navigable waters." Just v. Marinette County, 56 Wis. 2nd 7.

[8]Earl Finbar Murphy, Water Purity, A Study in Legal Control Natural Resources (Madison: University of Wisconsin Press, 1961) PP 38-39.

[1893]: "The result is that in every city in this country are found the most superficial and temporary expedients in nearly all sanitary work, and every year the problems become more intricate and difficult a solution, for the evils multiply as the population in each city increases."[9]

[1950]: "I lived here 70 years and a little sewage never harmed anyone."[10]

Generally in Wisconsin, early efforts at public health protection and pollution abatement were ones of forcing municipalities and private entities to spend monies for the protection of their citizens and their customers. For the most part early statutes succeeded that had an economic rather than a public health or safety basis. Throughout history, cost and private economics have retarded competent pollution abatement and appropriate infrastructure development.[11] However, by the end of the 19th century changes were beginning.

The 19th century in the United States was a century of local responsibility with limited communications, and while legislation might be theoretically sound, it often left issues totally to local administration, and thus, vulnerable to local manipulation. In this century and especially in Wisconsin, with the advent of progressive state government, an overall "statewide" constituency evolved. And with this broaden constituency came a less parochial view of public health and public protection.

Public Opinion

The third and critical ingredient in the increased statutory and financial support of pollution abatement has come as a result of a major shift in public opinion. In the last 15 years public interest and support for environmental protection has grown substantially. In Wisconsin, this degree of public acceptance has been large. For example, in a 1978 poll[12] produced by the Wisconsin Center for Public Policy, a

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[9]Murphy, Water Purity p. 42; quoting W.O.B. Lingade, Second Secretary of the Wisconsin State Board of Health in 1893.

[10]Murphy, Water Purity, p. 43; Quoting the Milwaukee Journal of 10/29/50.

[11]IBID; Especially chapter 2. See also: The Natural Resources of Wisconsin, An Inventory of the Natural Resources Committee of State Agencies (Madison, 1956).

[12]The Wisconsin Center for Public Policy, "The People on Wisconsin" (Madison, copyrighted 1978).

carefully selected cross section of adult Wisconsin residents indicated that environmental problems were second[13] only to taxes as the state's "biggest problem".

In a 1981 survey, it was reported that "the Wisconsin adult population is very interested in nature and the environment. Eighty-one percent of all persons interviewed were very interested in at least one nature topic. Eighty-seven percent were very interested in at least one environmental quality/management topics".[14]

Recently, these Wisconsin findings are being paralleled in national surveys. In a 1982 survey conducted by the Lou Harris organization, 74% of those polled felt curbing water pollution was a very important factor in improving the quality of American life. Sixty percent wanted the Federal Clean Water Act made stricter and 70% indicated a willingness to pay $100 or more in new taxes and costs to achieve clean water.[15]

In an April 1983 survey for the New York Times/CBS News, 58% of those surveyed agreed with the following:

> "Protecting the environment is so important that requirements and standards cannot be too high and continuing environmental improvement must be made regardless of cost".[16]

The Development of Environmental Management Structures and Institutions

Environmental management in Wisconsin has emerged from several historical roots. Notable, because of both their diversity and their commonality.

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[13]Also, a similar finding is found in: "Public Survey, July 1978, WDNR Office of Policy and Analysis" conducted by the University of Wisconsin Survey Research Lab. See also, a 1976 survey, Wisconsin Center for Public Policy Studies, "A Survey of the Attitudes of Wisconsin Citizens Towards Public Policy and Their Media, August 1976, Hart Research Associates, Inc."

[14]Environmental Resources Unit, University of Wisconsin, Environmental Education Needs and Interest of Wisconsin Adults (Madison, 1961) p.61.

[15]Lou Harris and Associates, Inc., "A Survey of American Attitude Towards Water Pollution", prepared for the Natural Resources Council of America, December 1982.

[16]The New York Times, April 17, 1983.

Conservation

Many current state environmental activities are derived from what can be described as traditional resource conservation functions. In 1867, Wisconsin's first forestry commission was created and a Board of Fish Commissioners was established in 1874. Fish and game wardens were appointed in 1885 and 1887, and a state Department of Forestry was created in 1897. A State Park Board began in 1907.

In 1915, a full time, three person Conservation Commission was established which pulled together into one administrative organization, fisheries, forestry, parks, and fish and game management and enforcement. From 1915 until 1967 through various legislative structurings, the "Conservation Department" carried out its activities.[17]

Public Health[18]

In 1876, out of the growing health problems of the era, there was formed the State Board of Health. As apart of its activities, it began to test the waters of the state for pollution and to recommend methods of water purification and sewage treatment. After 29 years, in 1905, the Board was given official powers to require water and sewer systems in cities and villages. In 1911, legislation was passed, assigning power to regulate sewerage facilities and to enforce the laws with regard to the protection of water supplies. Sanitary engineering as an activity of the Board was funded in 1919 as part of the state's first water pollution act.[19]

Water Regulation

In 1874, Wisconsin created the Board of Railroad Commissioners, later titled the Public Service Commission (the chief state public utility regulation organization). In 1915, this organization was assigned responsibility for the administration of Wisconsin's laws regulating lakes and streams.[20] Prior to this time, the Legislature, by statute, directly managed the waters of the state.

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[17]John Bubolz, "A History of Wisconsin Water Pollution Central Agencies and Factors to Consider in Their Reorganization"; River Basin Planning Seminar Unpublished Paper, May 1966.

Wisconsin Legislative Reference Bureau, 1981-1982 Blue Book, (Madison, Dept. of Administration) 1981, p. 468.

[18]Murphy Water Purity, Ch. 4.

[19]Wis. Stats. 1919, ch. 447.

[20]Regulatory jurisdiction over lake levels, rates of stream flow, dams, dock lines and alterations of navigable waters.

Over the years, (the Conservation Commission because of its role as protector of the resource, and the State Board of Health because of it's concern for disease prevention), both agencies became involved in water pollution prevention. And each agency came to its perceived role from a different perspective.[21]

Coordination

In 1927, the Committee on Water Pollution[22] was established to try and coordinate the activities of the several agencies vis a vis water pollution. The Committee was composed of representatives of the State Board of Health, the Conservation Commission and the Railroad Commission. Members of the Committee were presumed to represent their agency's point of view in the establishment of a unified state water pollution policy. The Committee was also assigned certain powers[23] in it's own right, even though it used the State Board of Health, Bureau of Sanitary Engineering as its administrative staff.[24]

In addition to these principal historical actors in water management, others, notably the Office of the Attorney General, the Natural Resources Committee of State Agencies, the State Soil and Water Conservation Committee, the State Laboratory of Hygiene and the State Geologist have also played roles.[25]

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[21]Murphy, Water Purity p. 85; The health agency wanted to study causes, and make inquiries into the ways of controlling pollution and economic conditions. The Conservation Commission believed that pollution was illegal and that the dischargers should be made to stop.

[22]Wis. Stats., 1927 ch. 264.

[23]These included administration and enforcement of all laws relating to pollution of surface waters; the authority to study and investigate surface water pollution; and the authority to discover economical and practical means of pollution control. In later years, the Committee would also be assigned the ability direct experimental facilities, to treat aquatic nuisances and to enter into interstate agreements.

[24]Sources on the Committee include: Donald M. Carmichael, "Forty Years of Water Pollution Control in Wisconsin: A Case Study"; 67 Wisconsin Law Review 350 pp. 353-357.

Bubolz, "A History of Wisconsin Water Pollution Control".

Murphy, Water Purity, p. 85.

[25]Harold H. Ellis, et al. Water Use Law and Administration in Wisconsin, (Madison: Univ. of Wisconsin Extension, 1970), page 195.

The Department of Resource Development

In August, 1966[26], a new water resources act abolished the Committee on Water Pollution and transferred it's functions into a newly created Division of Water Resources, the Department of Resource Development. This act also transferred most of the water functions of the State Board of Health to the same division. One year later, water regulatory powers of the old Railroad Commission, (in 1967 called the Public Service Commission) were consolidated within the Department of Resource Development.

The Department of Natural Resources (WDNR)

In 1967, as a part of a total functional reorganization of the executive branch of Wisconsin state government, the consolidation of water management was completed with the creation of the Department of Natural Resources. This new Department finally merged the water activities of the Department of Resource Development (formerly State Board of Health, Public Service Commission) and the Conservation Department into one administrative agency[27] thus, tieing together closely related traditional conservation activities, historical state water regulations functions with newly emerging, environmental regulation and public health protection. The Department of Natural Resources is currently the agency charged with management of Wisconsin's environment. The Department is responsible for implementing the laws that protect and improve Wisconsin's air, land, water, wildlife, fish and forests. The WDNR is also assigned responsibility under numerous federal laws that delegate management and enforcement tasks to the state.

WDNR: Organization and Structure

In 1983 the Department of Natural Resources is governed by a seven member board of private citizens appointed by the Governor, with the advise and consent of the State Senate. This policy board appoints it's Secretary who serves as the Chief Executive Officer of the Department.

Major staff organizational units of the Department include four divisions: Environmental Standards, Resource Management, Enforcement and Management Services. Six field districts provide line operational capacity for the numerous program functions. In addition to the specified staff functions noted above, the Office of the Secretary has specialized units in Legal Services, Intergovernmental Programs and Management and Budget.[28]

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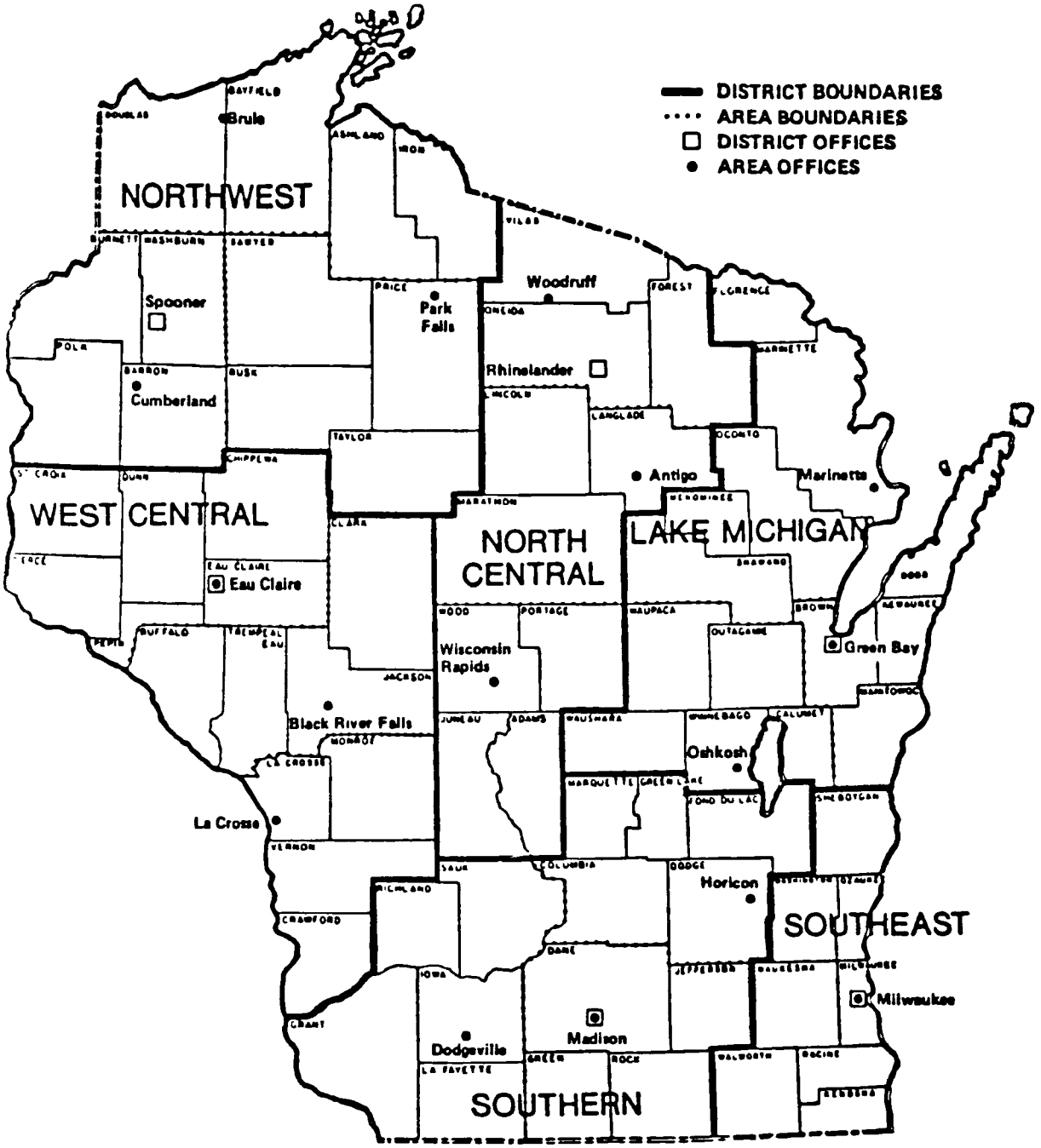
[26]Wis. Stats. 1965, ch 614.

[27]Wis. Stats. 1967, ch. 75.

[28]See Attachment A. and MAP 3

DNR FIELD DISTRICTS and AREAS

MAP 3



WDNR: Water Functions:

Within the current Department of Natural Resources several units are responsible for water-related activities. Functions, historically associated with water quantity and navigation are housed within the Division of Enforcement's Bureau of Water Regulation and Zoning. This Bureau also has responsibility under current law relative to floodplain and shoreland zoning.

Traditional conservation functions (including fisheries) are operated within the Resource Management Division.

Grants administration activities for wastewater facilities construction are housed in the Bureau of Water Grants, Office of Intergovernmental Programs. Technical engineering support for construction projects is supplied by the Bureau of Waste Water Management, Division of Environmental Standards.

The remaining elements of water activities are housed within the Division of Environmental Standards, with its Bureau of Wastewater Management, Bureau of Water Resources Management and Office of Operations and Maintenance, Bureau of Water Supply (drinking water) and Bureau of Solid Waste Management (groundwater, etc.).[29]

In a more program oriented approach, water activities to the Department can be defined in terms of point source, nonpoint, stream water quality, lake water quality, and groundwater management. Attached are tables and figures that more thoroughly demonstrate WDNR cross organization water activities.[30]

Wisconsin Water Quality Standards[31]

Wisconsin's water quality management program supervised by the Department of Natural Resources is designed to improve and protect the state's waters. This task involves many activities from monitoring wastewater treatment plants to seeking to curb farm runoff. While Wisconsin has had water quality standards for many decades, earlier requirements were based on protection of public health or navigability. Today, water characteristics or indicators which are more directly related to pollution are monitored, analyzed and governed.

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[29]See Attachment B.

[30]See Tables 1, 2, 3, 4, 5 and Figures 1 and 2. Source: Wisconsin Dept. of Natural Resources "Development Document for a Post - 1983 Water Quality Management Strategy" - (Madison: May 1982).

[31]Wisconsin Department of Natural Resources, Water Quality Management: Putting the Pieces Together (Madison, 1980)

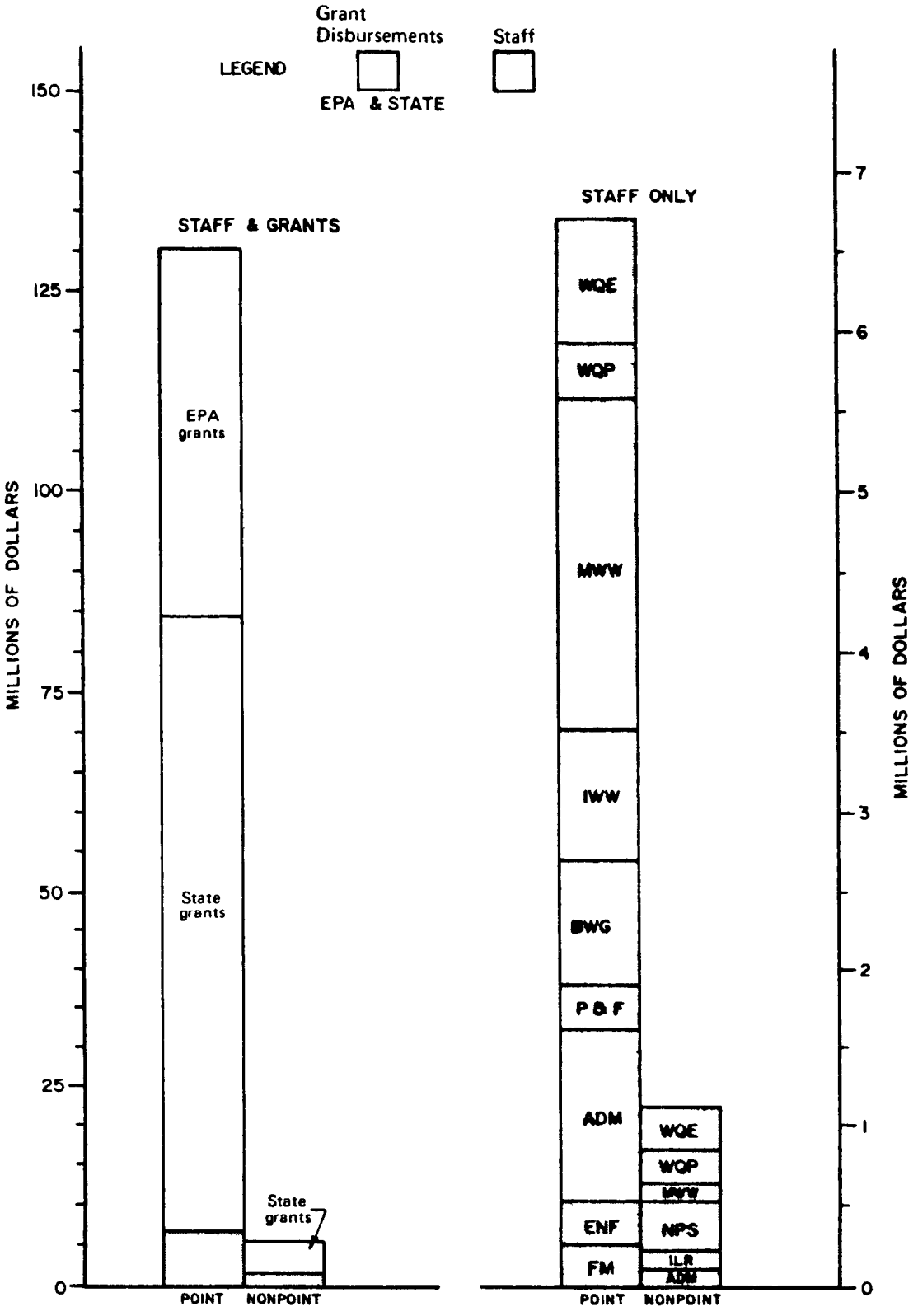


Figure 1. Water Quality Management expenditures by pollution source 1982.

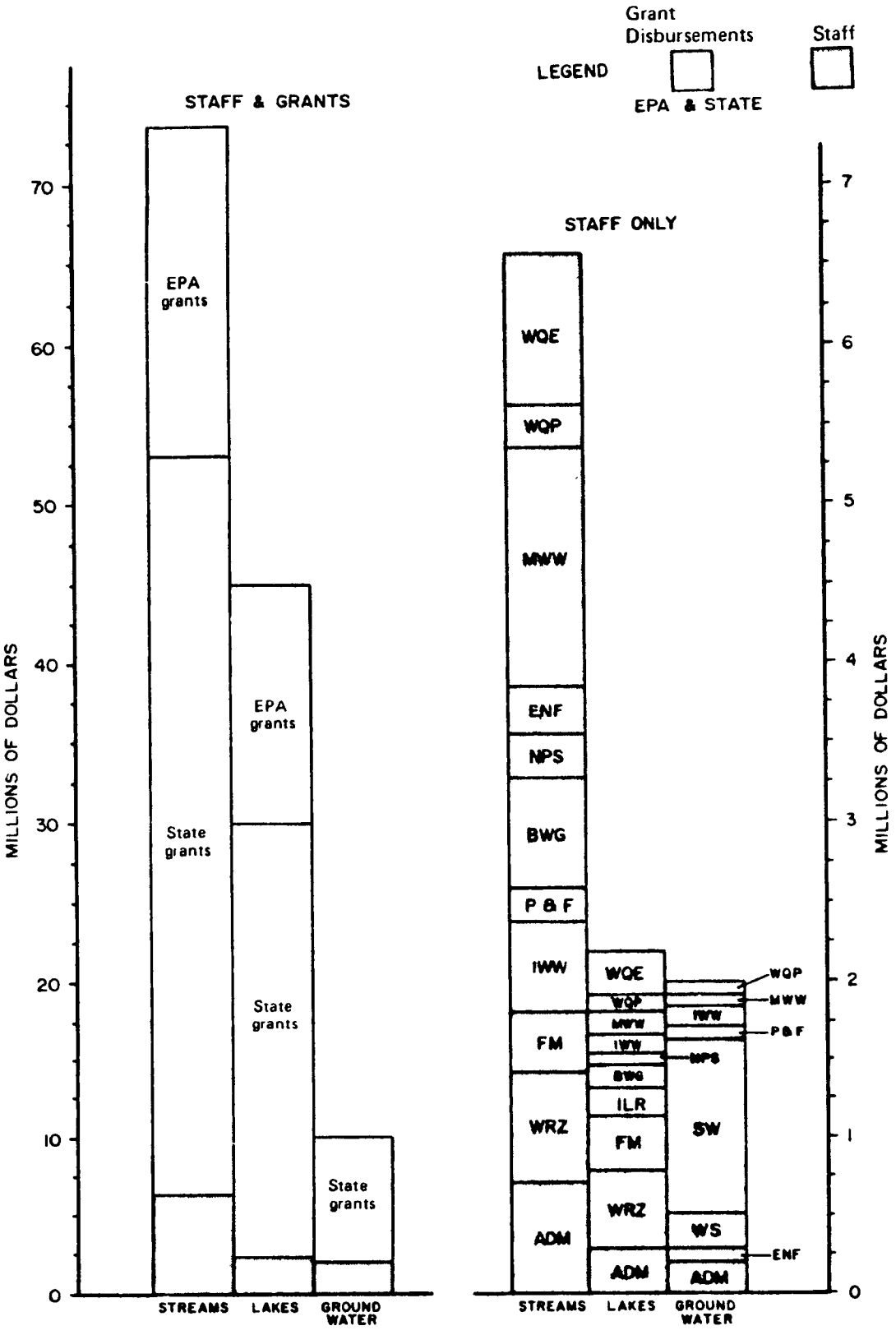


Figure 2. Water Quality Management expenditures by resource base 1982.

Abbreviations Used in Figures 1 and 2

ADM - Administration, training, professional development, etc.
BWG - Bureau of Water Grants
ENF - Environmental Enforcement
FM - Bureau of Fish Management
ILR - Inland Lake Renewal
IWW - Industrial Wastewater Section
MWW - Municipal Wastewater Section
NPS - Nonpoint Source Section
P&F - Pretreatment and Fees Section
SW - Bureau of Solid Waste Management
WQE - Water Quality Evaluation Section
WQP - Water Quality Planning Section
WRZ - Bureau of Water Regulation and Zoning
WS - Bureau of Water Supply

TABLE 1. CURRENT LEVEL OF EFFORT FOR POINT SOURCE MANAGEMENT (FY 1982)

Unit or Program	Staffing		Grant $ Disbursements
	Staff Years	$ Expenditure	
Water Quality Evaluation	22.9	618,300	
Water Quality Planning	22.4	605,610	
Municipal Wastewater	57.8	1,560,870	
Industrial Wastewater	21.5	580,500	
Water Grants	30.1	813,240	
Pretreatment and Fees	11.4	307,800	
Administration	31.4	848,880	
Enforcement	5.8	155,790	
Fish Management	5.9	100,300	
EPA Municipal Grants			46,000,000
State Municipal Grants			75,500,000
State Septic System Grants			2,000,000
Subtotals	209.2	$5,591,290	$123,500,000
TOTAL EXPENDITURES (staff and grant disbursements)		$129,091,290	

TABLE 2. CURRENT LEVEL OF EFFORT FOR NONPOINT SOURCE MANAGEMENT (FY 1982)

Unit or Program	Staffing		Grant $ Disbursements
	Staff Years	$ Expenditure	
Water Quality Evaluation	13.6	366,930	
Water Quality Planning	9.6	260,280	
Nonpoint Source	11.1	299,430	2,811,900
Inland Lake Renewal	6.5	174,150	1,150,000
Administration	9.7	261,900	
Subtotals	50.5	$1,362,690	$3,961,000
TOTAL EXPENDITURES (Staff and Grant disbursements)	$5,324,590		

TABLE 3. CURRENT LEVEL OF EFFORT FOR STREAM WATER QUALITY MANAGEMENT (FY 1982)

Unit or Program	Staffing		Grant $ Disbursements
	Staff Years	$ Expenditure	
Water Quality Evaluation	25.2	$ 679,590	
Water Quality Planning	25.7	693,900	
Municipal Wastewater	48.8	1,316,250	
Industrial Wastewater	16.5	445,770	
Nonpoint Source	9.2	249,210	2,342,500
Water Grants	24.1	651,240	
Pretreatment and Fees	8.1	219,240	
Enforcement	3.4	90,990	
Fish Management	15.8	268,600	
Water Reg and Zoning	27.0	729,000	
Administration	31.3	845,100	
EPA Municipal Grants			19,380,000
State Municipal Grants			46,035,468
Subtotals	235.1	$6,118,890	$67,757,967
TOTAL EXPENDITURES (staff and grant disbursements)		$73,946,857	

TABLE 4. CURRENT LEVEL OF EFFORT FOR LAKE WATER QUALITY MANAGEMENT (FY 1982)

Unit or Program	Staffing				Grant $ Disbursements	
	Staff Years		$ Expenditure			
Water Quality Evaluation	11.0	(3.5)	296,190	(94,500)		
Water Quality Planning	5.7		153,090			
Municipal Wastewater	4.2	(4.0)	112,320	(108,000)		
Industrial Wastewater	1.0	(1.0)	27,000	(27,000)		
Nonpoint Source	2.0		53,190		468,500	
Water Grants	6.0	(5.0)	162,000	(135,000)		
Pretreatment and Fees	1.0	(1.0)	27,000	(27,000)		
Inland Lake Renewal	6.5		174,150		2,300,000	
Fish Management	21.4	(4.7)	363,800	(79,900)		
Water Reg and Zoning	23.0	(5.0)	621,000	(135,000)		
Administration	7.4	(2.0)	199,800	(54,000)		
EPA Municipal Grants					25,620,000	(25,620,000)
State Municipal Grants					24,915,000	(24,915,000)
Subtotals	89.2	(26.2)	$2,189,540	(660,400)	$53,303,500	(50,535,000)
TOTAL EXPENDITURES (staff and grant disbursements)			$55,493,040	(51,195,400)		

* Numbers in paranthesis are staff years, staff expenditures, and grants for Great Lakes management activities only.

TABLE 5. CURRENT LEVEL OF EFFORT FOR GROUNDWATER QUALITY MANAGEMENT (FY 1982)

Unit or Program	Staffing		Grant $ Disbursements
	Staff Years	$ Expenditure	
Water Quality Planning	1.1	30,780	
Municipal Wastewater	4.4	118,260	
Industrial Wastewater	4.0	107,730	
Pretreatment and Fees	2.3	61,560	
Solid Waste	47.5	1,282,500	
Water Supply	7.0	189,000	
Enforcement	2.0	54,000	
Administration	2.5	67,500	
State Municipal Grants*			4,549,532
State Septic System Grants			2,000,000
Solid Waste Planning Grants			500,000
Subtotals	70.8	$1,911,330	$7,049,532
TOTAL EXPENDITURES (staff and grant disbursements)		$8,960,862	

* This entry is for land disposal system grants.

Following the lead of the Clean Water Act of 1972, the state's policy has been a goal of fishable, swimmable waters, wherever possible. Deciding where it is possible to achieve this goal is a challenge involving the examination of natural conditions of a particular waterway, absent man-induced pollutants.

For those waterways deemed to be able to achieve fishable and swimmable and for other uses, standards are established, which place limits on water quality indicators for each class and use of water. Table 6 shows current basic standards.

These standards and classifications give us the definition of clean for each waterway and thus we have assigned for each stream a minimum quality definition. Once these are assigned then actions to identify and to appropriately correct sources of pollution can be begun.

Limiting what and how municipalities and industries discharge to the state's waters, is the soul of Wisconsin's water quality management program. Determining to what degree a discharger must treat its wastewater is a function of two things: (1) Minimum standards of the Federal Clean Water Act and State Statute and (2) The water quality standards appropriate for each stream. The Federal Clean Water Act mandates basic treatment requirements for industries and municipalities. Wisconsin has also established for municipalities effluent limits based upon the classification of the receiving waters (see Table 7).

For waters where basic treatment levels are not sufficient to meet water quality standards, special attention is given to each with a process wasteload allocation. In such an analysis the total assimilative pollutant level for stream is determined and dischargers are then allocated a prorated discharge amount. The most complicated allocation of this nature is found on the Fox and the Wisconsin rivers.

Operating under federal delegation and state law, these standards of discharge are managed through the Wisconsin Pollution Discharge Elimination System (WPDES). Under this permit system, all discharges must have a permit with a stated requirements prior to discharge. All must comply with these levels. Under Wisconsin and federal law, failure to comply is subject to severe penalty including civil and criminal actions. The impact of this management system can best be seen by a profile of water quality in Wisconsin, historic and current.

Water Quality in Wisconsin: Historical

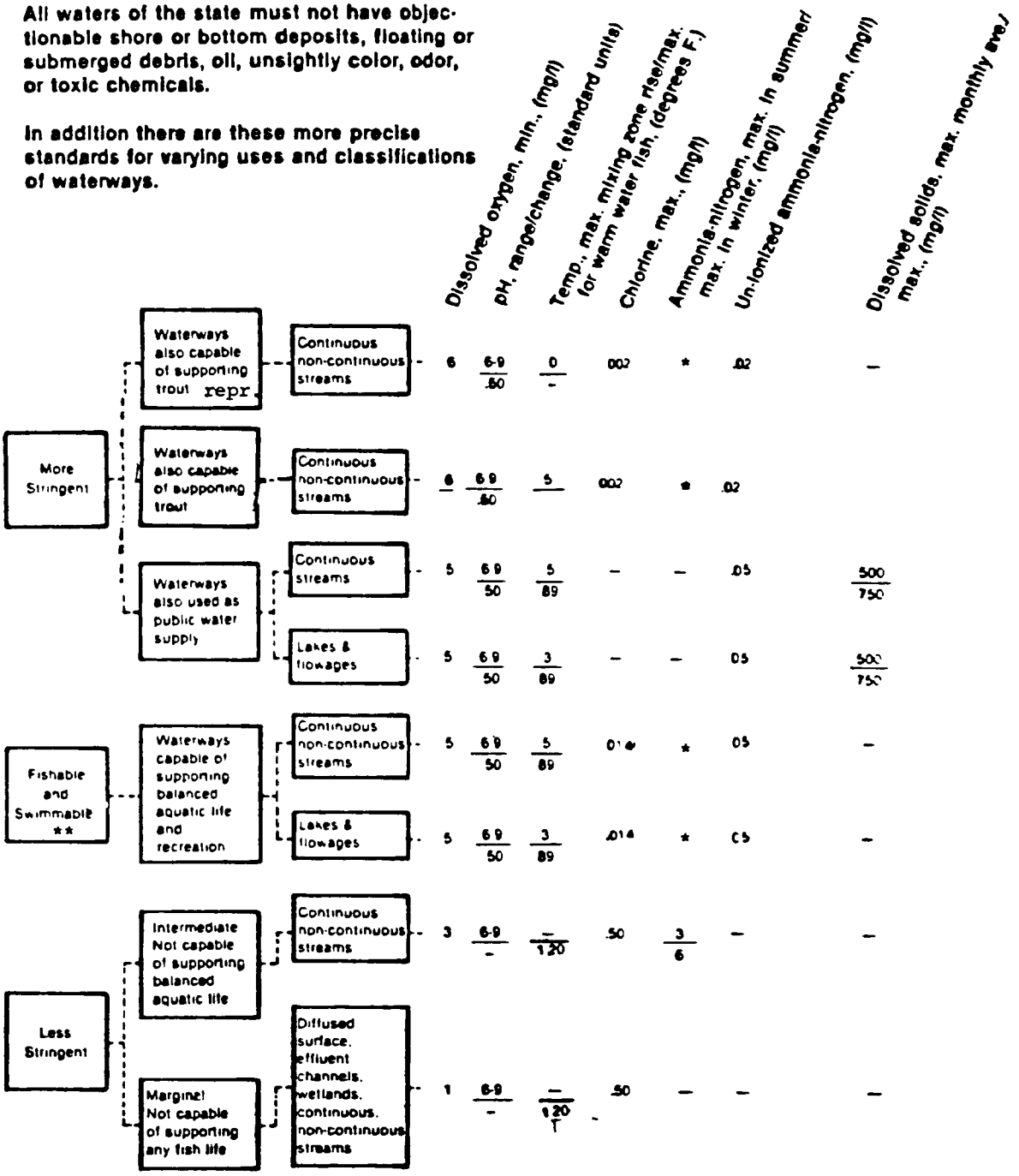
To someone arriving on the scene today it is difficult to fully describe the degree of water degradation that Wisconsin suffered as a result of its long struggle to organize and manage its resources. In its first 135 years of statehood, Wisconsin has destroyed its waters and forests and then reclaimed them. And as the institutional organization of management agencies finally became coordinated, about at the same point in time, water quality management began to have notable impact on the waters of the state.

WISCONSIN'S WATER QUALITY STANDARDS

TABLE 6

All waters of the state must not have objectionable shore or bottom deposits, floating or submerged debris, oil, unsightly color, odor, or toxic chemicals.

In addition there are these more precise standards for varying uses and classifications of waterways.

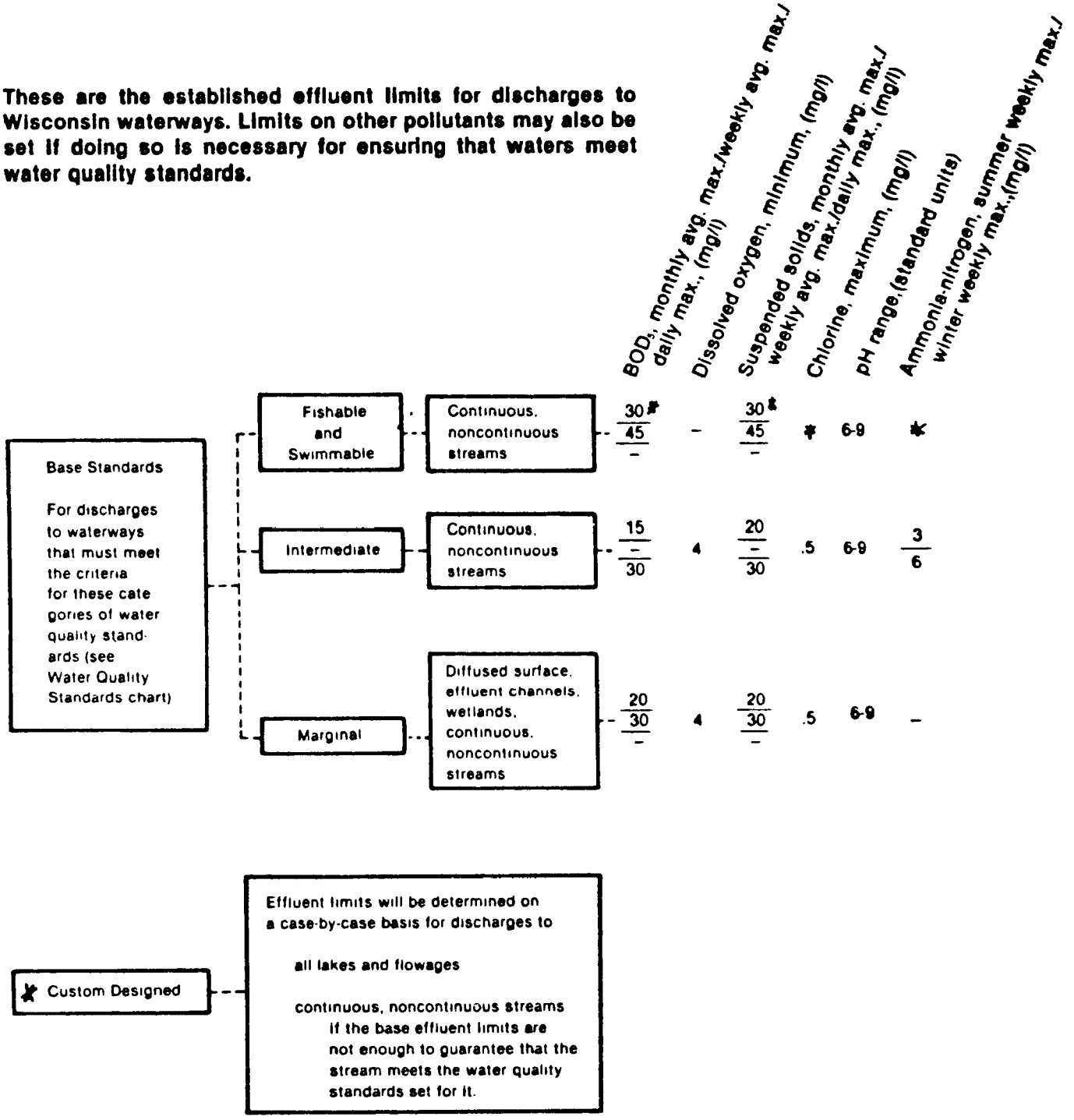


* Based on site specific analysis

** Criteria providing for the protection and propagation of fish, shellfish, and wildlife and for recreation in and on the waters.

Table 7
MUNICIPAL WASTEWATER TREATMENT EFFLUENT STANDARDS

These are the established effluent limits for discharges to Wisconsin waterways. Limits on other pollutants may also be set if doing so is necessary for ensuring that waters meet water quality standards.



While water can truly be described as Wisconsin's most precious resource, it has been regularly exploited with utter disregard for the impact on water quality. This degradation has been as a result of four primary sources: point source discharges from industries and municipalities; the cutting over and transportation of timber; poor soil protection and animal waste practices in agricultural communities; and the impacts of growing urbanization on natural water courses and drainage.

The early history of Wisconsin is marked with events of public outrage of water conditions results leading to new laws, agencies and procedures to eradicate damaging practice, but as likely as not, the public outcries were focused as easily manifested symbols of problems rather than fundamental causes. Before the 1920's data compatible with the current diagnostic measurements were rare. Early public concerns on water pollution evolved around odor, unsitelyness, disease transmission, navigation barriers, and the survival of species of popular sport fishes.

As a method of introduction to current water quality in Wisconsin and as a frame of reference to the changes that have occurred in the last few years, early data is most striking. For simplicity sake, the state's two most utilized rivers (the Fox and Wisconsin River) provide ample documentation of change.

Fox River 1926[32]

> "The Lower Fox River in past years has been used rather extensively as a source of recreation. Boating, swimming, and fishing constitute the main pleasures the river affords, but largely as a result of increased stream pollution of an objectionable character, the recreational value has been lost. At times ... the river is often unsightly and ill-smelling....the sewage pollution constitutes a continual menace of water born disease....[33]

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[32]The Fox River (Wisconsin) rises near Portage, Wisconsin, passes through several lakes into Lake Winnebago, then flows northeasterly to the Green Bay of Lake Michigan. The Lower Fox (from Neenah-Menasha to Green Bay) constitutes one of the principal industrial centers in the state with 14 dams in the 31 miles from Lake Winnebago to Green Bay. In 1926, 34 pulp and paper mills producing 1,034 tons daily were utilizing the stream for discharge. In addition, six small cities with population of approximately 90,000 persons were also discharging to the river. In 1982, in this same reach of stream approximately 5,700 tons of daily production was recorded. Figure 4 shows major discharges to the Lower Fox.

[33]Wisconsin State Board of Health Stream Pollution in Wisconsin, a Joint Report of the Conservation Commission and State Board of Health of Wisconsin concerning activities in the control of stream pollution from July 1, 1925 to December 31, 1926, Madison 1927.

Figure 3

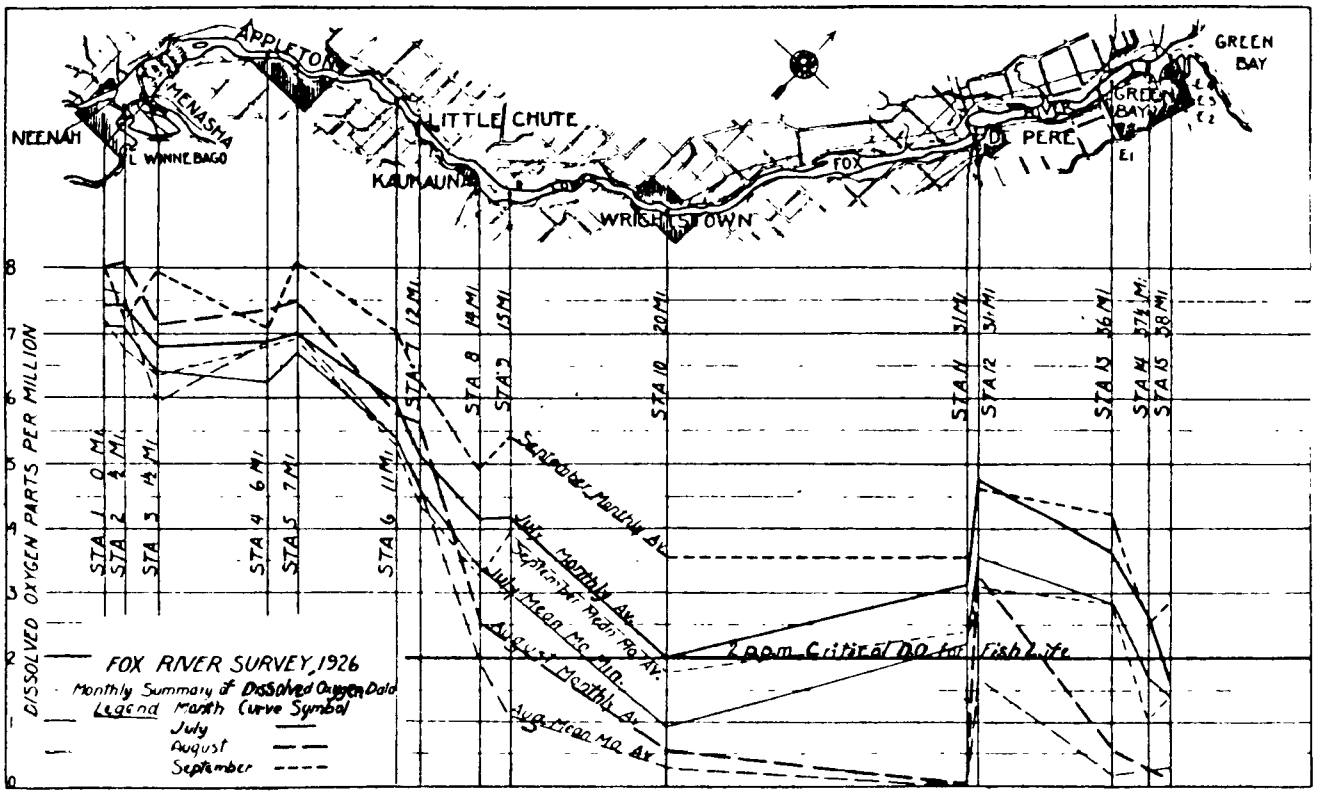
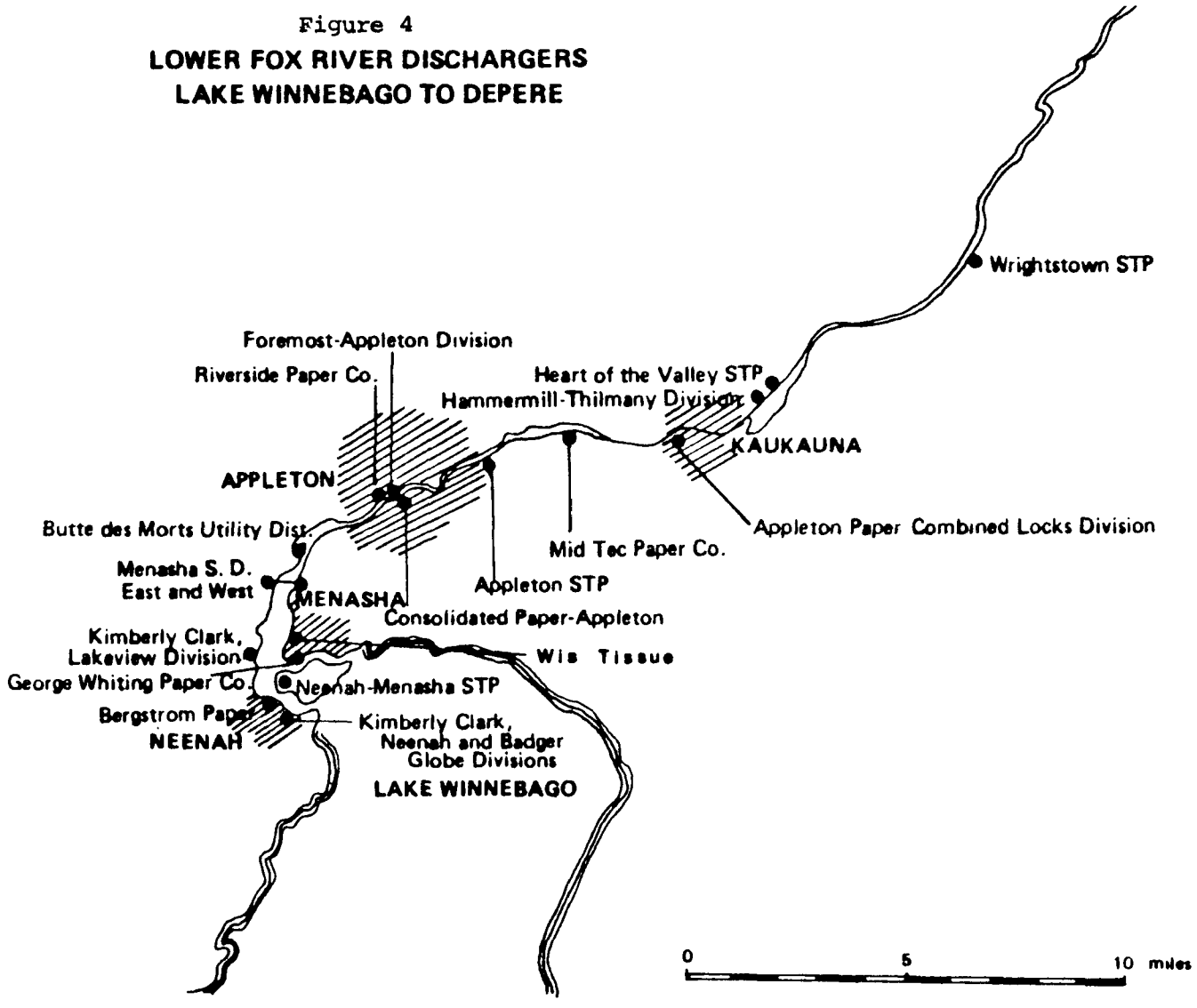

APPLETON
MENASHA
NEENAH
L WINNEBAGO
LITTLE CHUTE
KAUKAUNA
WRIGHTSTOWN
DE PERE
FOX
GREEN BAY
GREEN BAY
DISSOLVED OXYGEN PARTS PER MILLION
STA 1 0 MI
STA 2 2 MI
STA 3 4 MI
STA 4 6 MI
STA 5 7 MI
STA 6 11 MI
STA 7 12 MI
STA 8 18 MI
STA 9 15 MI
STA 10 20 MI
STA 11 31 MI
STA 12 34 MI
STA 13 36 MI
STA 14 37½ MI
STA 15 38 MI
September Monthly Av.
July Monthly Av.
September Mean Monthly Av.
July Mean Monthly Av.
August Monthly Av.
Aug. Mean Monthly Av.
2 ppm Critical DO for Fish Life
FOX RIVER SURVEY, 1926
Monthly Summary of Dissolved Oxygen Data
Legend Month Curve Symbol
July
August
September

Figure 4

LOWER FOX RIVER DISCHARGERS
LAKE WINNEBAGO TO DEPERE



Lower Fox River, July 8-9, 1926[34]

Location	July 8 Temp. (C°)	July 8 DO (PPM)	July 9 Temp. (C°)	July 9 DO (PPM)
Lake Winnebago Outlet			25	6.4
Appleton (above dam)	26	5.7	27	5.4
Rapids Croche (above dam)	27	3.7	25	3.6
Little Rapids (above dam)	29	2.1	24	2.3
DePere (above dam)	29	2.4	25	2.0
Green Bay (Main Street)	28	2.0	25	1.9

As a result of these preliminary investigations more detailed studies were carried out. Figure 3 show data for the summer of 1926 in graphic form.

Twenty-nine years later in 1955 comparable location data shows worse conditions below Lake Winnebago than previously recorded.

LOWER FOX RIVER, 1955[35]

Location	June Temp. (C°)	June DO (PPM)	July Temp. (C°)	July DO (PPM)
Lake Winnebago Outlet (Neenah channel)	19.2	8.68	26.5	10.81
Menasha Channel (Neenah channel)	20.5	10.74	27.1	11.41
Appleton (above dam)	21.9	6.9	30.0	6.98
DePere (above dam)	22.0	1.21	28.2	1.47
Green Bay	20.5	.64	26.3	.52

Ten years later (1965) data for Green Bay showed little change, but by 1967-68 change was beginning.

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[34]State Board of Health, Stream Pollution, p. 144. Location shown are edited to relate closely with later data.

[35]Committee on Water Pollution, the Sulphite Pulp Manufacturers' Research League; Evaluation of Stream Loading and Purification Capacity; Cooperative State-Industries Studies in the Lower Fox River in 1956, (Madison, 1957).

LOWER FOX RIVER AT GREEN BAY 1965-68[36]

Year	Survey Dates	Early Temp. C°	Early DO (PPM)	Mid Temp. C°	Mid DO (PPM)	Later Temp. C°	Later DO (PPM)
1965	(6/7, 6/29 8/24)	21	6.0	24	3.6	23	.9
1966	(5/31, 6/29, 8/31)	17.5	4.3	26.5	1.7	23	.1
1967	(6/1, 6/23, 7/26)	18.5	4.6	21	5.2	21	7.2
1968	(6/25, 7/16, 8/20)	22	2.7	27	5.2	25	4.7

Data for 1974-1981 indicates still further improvement on the Lower Fox. These changing conditions are especially notable because during the 50 year period profiled a five fold increase in paper production occurred along this stretch of the river. In addition, as environmental management continued, between 1973-81 BOD_5 loadings have dropped from 208,296 pounds a day to 39,947 pounds a day along the stream.

THE LOWER FOX RIVER SUMMER 1974[37]

Location	June Temp. (C°)	June DO (PPM)	July Temp. (C°)	July DO (PPM)	August Temp. (C°)	August DO (PPM)
Lake Winnebago Outlet	18	8.5	22	6.6	24	9.5
Appleton	-	7.6	-	7.2	-	6.4
DePere	-	7.1	-	7.5	-	7.7
Green Bay	20	7.9	24	6.7	-	8.8

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[36]Wisconsin Department of Natural Resources, Surface Water Quality Monitoring Data, 1965-68, (Madison).

[37]Data is for June 24, July 25, August 21, 1974. Appleton and DePere data is estimated from continuous data plots: Department of Natural Resources, State of Wisconsin Surface Water Quality Monitoring Data, 1973-1976 (Madison); Automatic Water Quality Monitoring of the Fox and Wisconsin Rivers in 1972-1981 (Madison 1982).

LOWER FOX RIVER SUMMER 1981[38]

Year	June Temp. (C°)	June DO (PPM)	July Temp. (C°)	July DO (PPM)	August Temp. (C°)	August DO (PPM)
Lake Winnebago Outlet	20.9	6.9	23.1	7.9	22.2	7.9
Appleton	21	6.0	22.9	4.5	22	6.7
Rapid Croche (Plot Est)	-	6.8	-	7.6	-	7.8
DePere	22	6.8	23	10.2	24	8.5
Green Bay (Plot Est)	-	7.0	-	5.2	-	6.2

The Upper Wisconsin 1926 to 1981[39]

The Upper Wisconsin River from Rhinelander to Petenwell flowage is one of the hardest working rivers in the world with 29 major industrial and municipal dischargers (Figure 5). In a July 1926 stream survey results showed conditions ranging from good D.O. levels at Rhinelander (above the first discharger) to 0 D.O. conditions below Wausau and a repeated condition of .5 DO below Nekoosa.

In 1956, studies indicated little change in the conditions of 30 years earlier with 0 D.O. situations immediately below Rothschild and again in Nekoosa. A summary of Wisconsin River sampling data is shown in Table 8 for the years 1926 to 1981 which like Fox River data reflects a major shift in river conditions, beginning in the late 1960's.

In addition to the regular monitoring data available for the Upper Wisconsin, beginning in 1973 and continuing each year until 1980, a series of synoptic

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[38](a) IBID; WDNR Automatic Water Quality Monitoring, (b) Wisconsin Department of Natural Resources; State Surface Water Quality Monitoring Data, 1981 (Madison, 1983)

[39]The Wisconsin River rises at Lac Vieux Desert, and flows south and westward to the Mississippi at Prairie du Chien, a distance of almost 400 miles. The part of the Wisconsin that is most impacted by pollution occurs between Rhinelander and Nekoosa, a distance of 150 miles. During its passage of this reach, the river crosses 20 dams and receives the discharges of many industrial concerns and municipalities. In terms of the pulp and paper industry, 18 discharged to the river in 1926.

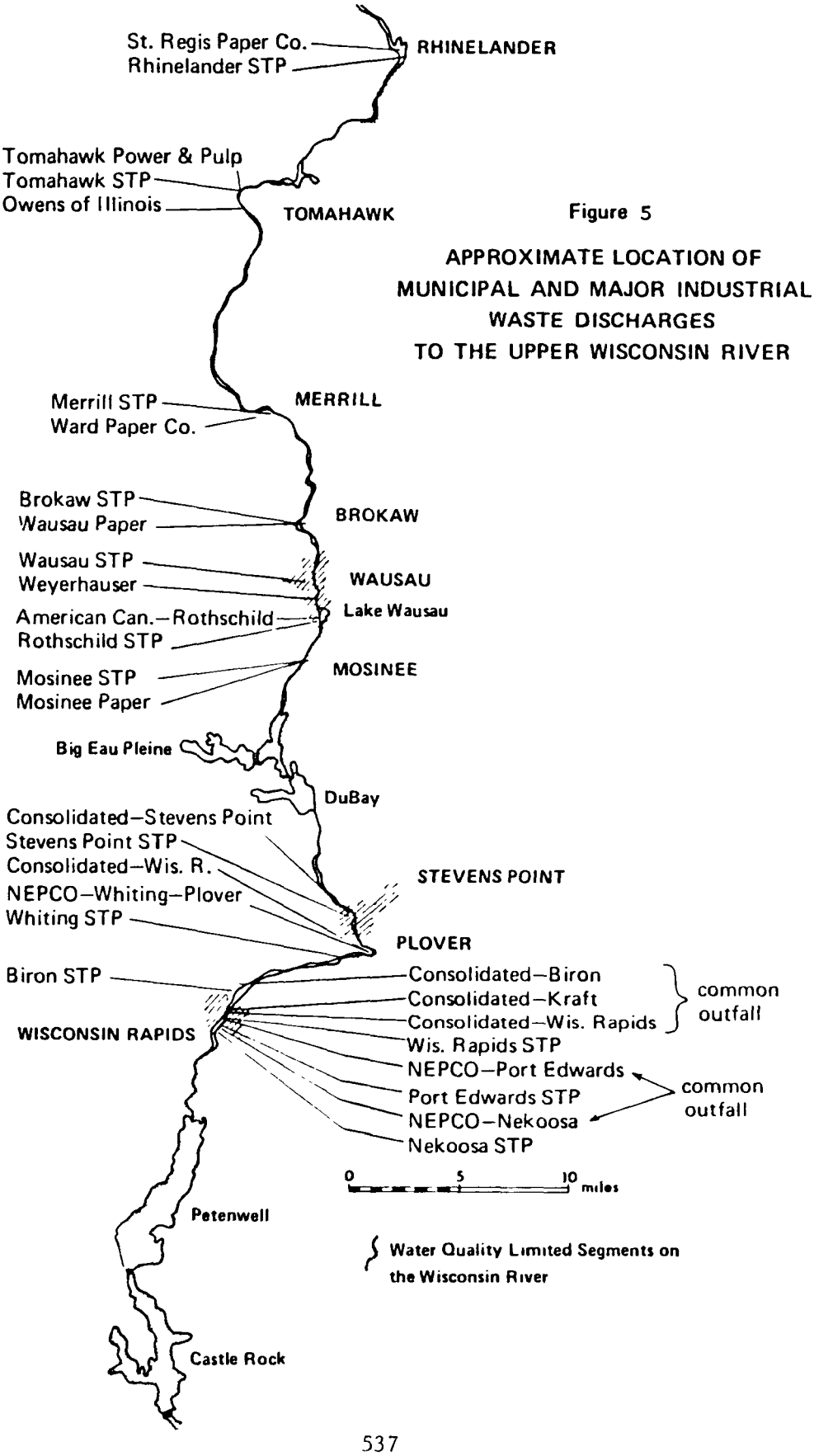

Figure 5

APPROXIMATE LOCATION OF MUNICIPAL AND MAJOR INDUSTRIAL WASTE DISCHARGES TO THE UPPER WISCONSIN RIVER

TABLE 8
Wisconsin River - Dissolved Oxygen (ppm)
1926-1981

Year/Month/Day	Locations															
	Rhinelander		Hat Rapids		Merrill		Wausau		Green Bay		Biron		Nekoosa		Petenwell	
	C°	DO	C°	DO	C°	DO	C°	DO	C°	DO	C°	DO	C°	DO	C°	DO
1926-July (15-22)	20	7.8			22	7.2	23	1.0			26	1.6	26	.1	26	2.5
1955-August (10-18)	23.3	6.6	23.8	5.3			26.3	.5					282	.57		
1965-June (17, 8)							21.5	4.3							18	7.8
July (13, 27)							24	2.4							21	5.9
August (8, 24)							23	2.1							23	8.4
1966-June (8, 13)							24	2.6							18	6.6
July (12)							27	1.1						2	24	6.4
August (28, 22)							24	1.7							22	5.4
1967-June (8, 13)							19	5.7							19	6.8
July (12)							22	3.7							21	5.4
August (18, 22)							21	5.8							21	6.0
1968-June (18, 25)							19.5	6.2							21	7.6
July (29, 23)							22.5	5.4							24	6.7
August (20, 27)							24.5	4.6							22	6.6
1974-June (6, 24)	19	8.2	19	4.3			19	5.5		3.8		6.2				6.4
July (25)	23	6.7	23	3.1			23	5.3	(Plot	2.8	(Plot	4.8			(Plot	3.0
August (23)	22	7.0	23	4.5			22 (8/23)	8.2	Est.)	3.4	Est.)	5.6			Est.)	5.2
1981-June (17,23, 24,25)	18	7.8	17	6.3	16	8.8	16	8.5	18	8.0	17	8.5	20	8.4	20	7.2
July (16,22 12,13)	20.5	7.8	21	5.3	19.5	6.7	20	7.4	22	8.1	22	8.4	23	8.9	25	12.5
August (3,13,2)	20.5	8.0	22	6.5	21	8.3	21	8.3	23.5	8.8					22	5.5

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surveys[40] were carried out on the entire upper river from Rhinelander (mile post 340.4)to Petenwell Dam(mile post 171.9).

These surveys provide some of the most detailed information ever collected on the river. And while it is not practical in this paper to detail all the dissolved oxygen data, a few unsophisticated highlights are symptomatic of the changes occurring in the River.

WISCONSIN RIVER SURVEYS 1973-80
DISSOLVED OXYGEN VALUES

Survey Dates	Total Samples	Values Below 3(ppm)	%	Values Above 7(ppm)	%	Lowest Value(ppm)	Highest Value(ppm)
July 9-18, 1973	625	128	20.5%	59	9.4%	.2	8.6
Aug 6-17, 1975	338	91	26.9%	19	3.3%	.0	9.6
June 27-29, 1978 July 15-17	578	7	1.2	115	20%	1.7	9.0
Aug 18-19, 1980	332	3	1.1%	199	59.9%	1.8	11.0

As was the case with the Fox River during this period, paper and pulp production increased to 6051.4 average tons daily and BOD_5 discharged dropped. (From 462,817 daily average lbs in 1973 to 28,314 daily average lbs in 1981.) Thus with water management has come increased and improved water quality and economic expansion.

Water Quality in Wisconsin: Current

A good profile of current water quality is gained by reviewing ambient water quality data, contained in the National Water Quality Information System

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[40]The general procedure for synoptic surveys was to have one or more river crews begin at Rhinelander and over the course of several days progress to Petenwell Dam taking field measurements and obtaining water quality samples. Field measurements were made every .1 to one mile depending in river characteristics. For example, dissolved oxygen measurements were typically made above dams and below dams (1/10 mile or less) while stretches of river with consistent hydraulic characteristics were measured only every mile. Readings were also taken at a variety of depths and at several points across the River. Source: Wisconsin Department of Natural Resources, "Water Quality Modeling of the Upper Wisconsin River for Wasteload Allocation Development (Water Quality Data), Madison, 1982.

(Storet).[41] General indicators of water quality sampled in the ambient monitoring network -- nutrients, biochemical oxygen demand, and suspended solids -- are presented in the following comparison. The mean of the data collected for each parameter was determined for two two-year periods, water years 1980-81 and calendar years 1977-78. Selection of these two distinct periods of time provides a "gap" and allows comparison of ambient quality over time.

This profiles again in parameters beyond dissolved oxygen the major changes occurring in Wisconsin streams[42]

Organic nitrogen

One of the limiting factors of plant growth is nitrogen. Organic nitrogen is the form that is tied up in organic compounds by plants and animals. It is also significant in that it eventually decomposes into ammonia which can be toxic to aquatic life. Monthly ambient monitoring indicates that 27 percent of the stations show a decrease in organic nitrogen levels over the past three years. The greatest reductions in the parameter occur in the Lake Michigan tributaries. Some monitoring stations, such as Oconto and the Upper Fox River at Berlin and Neenah-Menasha, indicate a drop of 25-50 percent in organic nitrogen. Other stations show a lesser drop in organic nitrogen.

Neillsville, on the Black River, shows a major reduction in organic nitrogen levels. There is no conclusive reason for the reduction, although owner 200 manure storage facilities have been established in the area since 1977. In most other parts of the state, organic nitrogen levels in rivers remain fairly constant. The majority of the state monitoring stations indicate values of 1 milligram per liter (mg/l) or less. The problem area of the lower Rock River is still apparent; nonpoint source loading compounded by municipal discharge may be the primary reasons.

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[41]This federal information STO-RAGE and RET-RIEVAL system is used for the compilation of data from fixed station monitoring sites and provides a "snap shot" look at the chemical condition of state's waters over time.

Wisconsin first established a network of monthly water quality monitoring stations in 1961. A new revised monitoring network, was established in 1977 (Table 9). Accumulated stream flow records were a major site selection criterion. Stations selected were located in water use areas or particular land use areas representing a broad spectrum of sites, from paired stations around problem areas to clean water streams. Stations on the Fox and Wisconsin Rivers provides a broad, inclusive data base.

[42]The data presented here is from: Wisconsin Department of Natural Resources - Wisconsin Water Quality: 1982 Report to Congress (Madison, 1982).

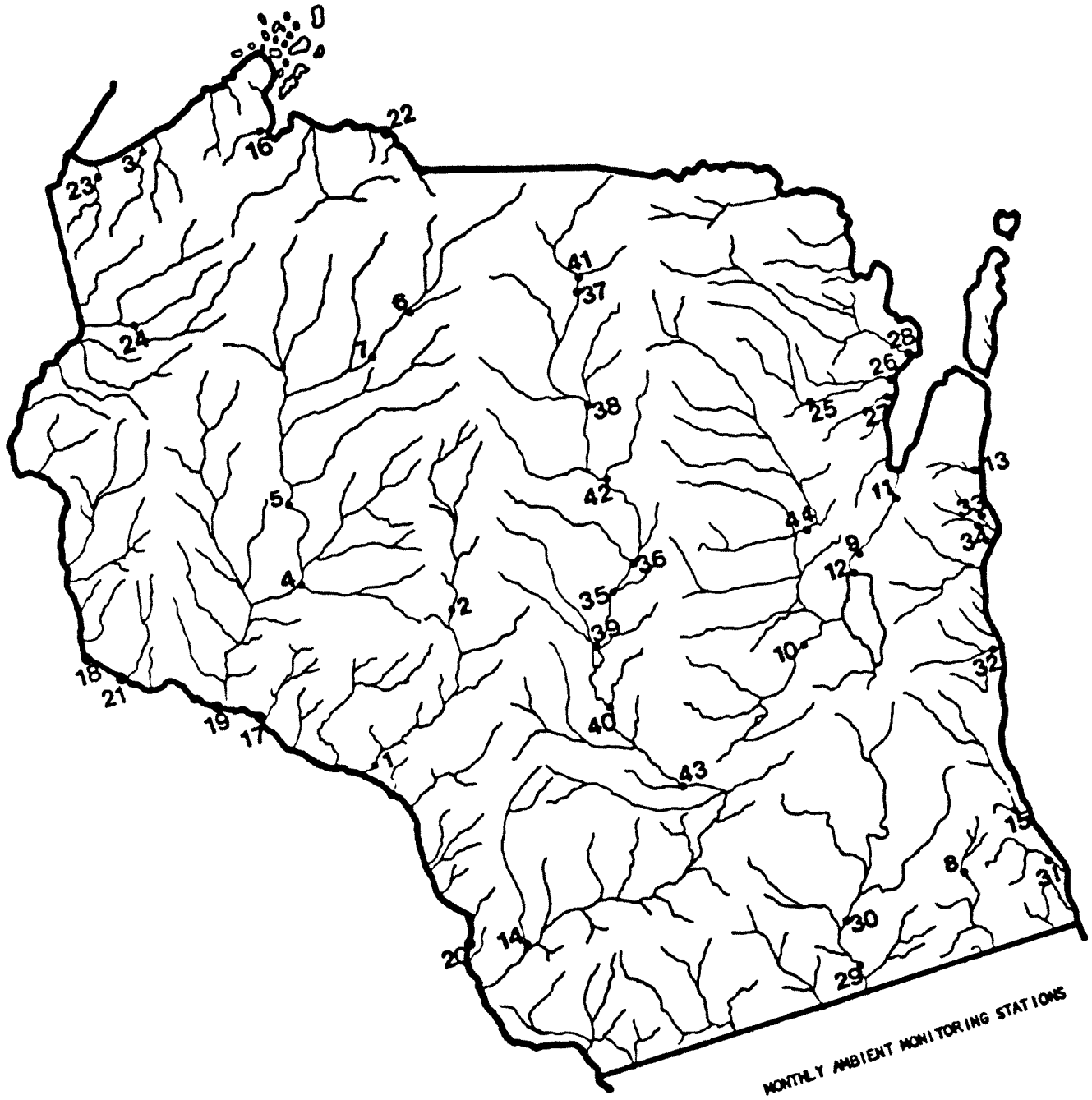
WISCONSIN'S MONTHLY MONITORING PROGRAM

TABLE 9

Station in Figure	Station Location
1.	Black River (Galesville)
2.	Black River (Nellisville)
3.	Bois Brule River (Brule)
4.	Chippewa River (Eau Claire)
5.	Chippewa River (Holcombe)
6.	Flambeau River (Park Falls)
7.	Flambeau River (Pixley)
8.	Fox (Illinois) River (Waukesha)
9.	*Fox River (Appleton)
10.	Fox River (Berlin)
11.	*Fox River (De Pere)
12.	*Fox River (Neenah-Menasha)
13.	Kewaunee River (Kewaunee)
14.	Kickapoo River (Steuben)
15.	Kinnickinnic River (Milwaukee)
16.	Lake Superior (Ashland)
17.	Mississippi River (Alma)
18.	Mississippi River (Hastings)
19.	Mississippi River (Lake Pepin)
20.	Mississippi River (Lynxville)
21.	Mississippi River (Red Wing)
22.	Montreal River (Saxon)
23.	Nemadji River (Superior)
24.	Namekagon River (Riverside)
25.	Oconto River (Gillett)
26.	Oconto River (Oconto)
27.	Pensaukee River (Pensaukee)
28.	Peshtigo River (Peshtigo)
29.	Rock River (Afton)
30.	Rock River (Indianford)
31.	Root River (Racine)
32.	Sheboygan River (Sheboygan)
33.	Twin River-east (Two Rivers)
34.	Twin River-west (Shoto)
35.	*Wisconsin River (Biron)
36.	*Wisconsin River (Du Bay Dam)
37.	Wisconsin River (Hat Rapids)
38.	Wisconsin River (Merrill)
39.	*Wisconsin River (Nekoosa)
40.	*Wisconsin River (Petenwell)
41.	Wisconsin River (Rhinelander)
42.	*Wisconsin River (Wausau)
43.	Wisconsin River (Wisc. Dells)
44.	Wolf River (New London)

* denotes continuous monitoring station

TABLE 9: MAP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MONTHLY AMBIENT MONITORING STATIONS

Ammonia

To a small extent, ammonia is naturally present in surface waters as a waste product of fish and as a breakdown product of organic nitrogen compounds. But additional amounts from wastewater may raise inriver concentrations to a point of toxicity to aquatic life.

High ambient values indicated by the 1977-78 statewide monitoring have dropped markedly. The Oconto River at Oconto and the Root River show the most dramatic reduction in ammonia concentration, each over 75 percent. Also, Afton, downstream from municipal sources on the Rock River, shows a significant decrease in ammonia.

Reduction trends are also apparent on the Wisconsin and Mississippi Rivers where stations average at least a 25 percent reduction. Overall, more than 40 percent of the monitoring stations show decreases in ammonia concentrations over the past three years.

Total phosphorus and dissolved phosphorus

Phosphorus, in addition to nitrogen, is a primary limiting factor in plant growth. Excessive concentrations in surface waters may cause algal blooms and weed growth. Phosphorus occurs in surface waters in a variety of forms. Total phosphorus measures the sum of orthophosphate, an organic phosphate applied to agricultural land and carried to surface waters in runoff; organic phosphorus, a form tied up in biological processes; and condensed phosphates, such as detergent phosphates which eventually change into orthophosphate in surface waters. Dissolved phosphorus or orthophosphate is phosphorus in the form most readily available to plants.

Statewide, there has been a downward trend in mean total phosphorus concentrations. Approximately one-half of the monitoring sites show decreasing concentrations, while the remaining stations show relatively no change. Along the Mississippi and Fox Rivers, stations average a decrease of over 25 percent and the Rock River stations each show about a one-third reduction in total phosphorus. The majority of the Great Lakes tributaries also show reductions in mean total phosphorus levels. Lake Superior stations all show negligible amounts, below 0.05 mg/l. Lake Michigan tributary monitoring indicates higher levels but the degree of reduction from the 1977-78 mean concentration averages 20 percent at most of the stations. Total phosphorus levels are still relatively high in the very southern, most populous part of the state; the Fox (Illinois) and the Root Rivers' sites monitor no appreciable change since 1977. Throughout most of the state, though, mean total phosphorus levels at the problem sites have been significantly reduced.

The existing statewide pattern of mean dissolved phosphorus levels is similar to that of total phosphorus -- relatively high concentrations in the southern and south central regions and low levels around the rest of the state. Previous problem areas are still apparent, but at a greatly reduced level.

Since 1977-78, the trend of dissolved phosphorus levels has been downward at over half of the monitoring stations; the remainder have been relatively static conditions.

Overall, each of the major rivers shows a drop in dissolved phosphorus concentrations. Many of the Lake Michigan tributaries average at least a 25 percent reduction, and concentrations at the Rock River stations are down by more than one-third. Mississippi River monitoring stations do not show any pattern in changing dissolved phosphorus levels. A significant decrease has been recorded at Neillsville on the Black River. The other major state river system, the Wisconsin River, also shows many lowered mean values, all less than 0.04 mg/l.

Biochemical oxygen demand

Biochemical oxygen demand is a measure of the amount of oxygen used by microorganisms as they break down organic matter in the river. The BOD values used in this analysis are based on the standard 5-day analysis and indicate water quality degradation caused by point sources.

Seventy-four percent of the stations show a decrease in BOD concentrations. The succession of seven stations on the Wisconsin River indicates that inriver concentrations have been reduced by about one-third over the past three years. The Fox River stations have also shown a marked reduction in the amount of BOD (about one-third). A major cutback in organic waste discharge by cities and paper mills along these rivers has been a primary factor in the resulting improvement.

Two paper mill discharges which caused major problems in the 1970s (on the Flambeau and Oconto Rivers) show a trend of major improvement. Since a reduction in mill discharge wastes, downstream concentrations are reduced more than 50 percent. Measurement of BOD on the Mississippi River at Hastings, monitoring the water quality impacts the Minneapolis, Minnesota Twin Cities urban area upstream, also shows a reduction. Biochemical oxygen demand is not monitored at stations where organic loadings cannot be attributed to a specific point source.

Suspended solids

Suspended solids, a primary cause of inriver turbidity and siltation may also exert an oxygen demand if the mean concentration is high. The problem can be naturally occurring, or compounded by poor agricultural practices and wastewater discharge from industries and municipalities.

In general, high concentrations of suspended solids are found in the peripheral rivers of the state, and lower amounts in the central area. The majority of the monitoring sites show low to medium concentrations, under 20 mg/l.

Over the past four years, about one-third of the stations have shown a decrease in suspended solids levels. Considerable decreases were noted at sites with a high degree of soil erosion -- the Kickapoo and Nemadji Rivers.

In each case, monitoring shows inriver concentrations down by over one-third. Other trends are variable though substantial. The Kinnickinnic and Kewaunee stations each indicate over a 50 percent drop in solids levels while the Root and Peshtigo Rivers show increases of 23 percent and more. Fifty percent of the stations statewide show no appreciable change in the magnitude of the suspended solids mean.

Water Quality Management - Facility Status

The history of both municipal and industrial wastewater facility construction in Wisconsin has been a slow one with very modest progress in the early days.

In 1873, Milwaukee was the first place in Wisconsin to have a public water system. Legislation for public sewers was passed in 1869 and in 1889 sewerage plans were required of cities in Wisconsin. In 1925, there were only 60 municipal wastewater plants of any character in Wisconsin. There were less industrial ones. By 1940, 213 treatment facilities were in operation and in 1947, 47 more had been added.[43] Beginning in the 1950's municipal facility construction was largely stimulated by grant programs from either the federal or state government and industrial construction was carried out under penalty of law.

Industrial

Shortly after the passage of P.L. 92-500, in 1972 with its statutory deadlines industrial discharges because the leaders in the clean-up of Wisconsin's waters.

As the data noted for the Lower Fox and Upper Wisconsin rivers suggests, the massive loading reductions from principal dischargers has contributed greatly to the rebound of usable water. During the spring of 1983 90-95% of all 1,034 industrial permittees in the state were meeting their permit requirements.

Obviously, process and groupings of processes vary according to treatment needs and stream requirements. Yet a compilation of current processes shows the broad range of industrial permittee process usage. Table 10 is a summary compilation of processes by permitted water use type.

Municipalities

Wisconsin's municipalities because of their generally less organized character, slower decision making processes and historical reliance on outside grants initially lagged far behind in carrying out the requirements of the 1972 Federal Act.

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[43]Murphy, Water Purity, pp. 60, 86, 88

TABLE 10
Wisconsin Industrial Dischargers
Water Treatment Processes
1983

	Process Wastewater	Contact Cooling Water	Non-Contact Cooling Water		Domestic Wastewater	Other
			With Additives	Without Additives		
Absorption Pond	94	31	5	4	1	
Activated Sludge Cont STB	1			1		
" " Conventional	4	7	3			
" " Ext. Aeration	6	3	1	1		1
" " Two Stage	1	6				
" " Pure Oxygen		1	1	1	1	
Aerated Lagoon	28	8	2	1	1	
Aerobic Digestion	2			1	1	
Anaerobic Digestion	6	2	1	1		1
Belt Press	1	2	2	3	1	4
Chemical Addition	3	3	5	7	2	2
Chemical Conversion	3	3	6		2	
Chlorine	9	3	8	7	3	
Coagulation-Fluctuation	6	14	3	2	1	2
Dechlorination	1		1			1
Disinfection-General	1	1			1	
Drainfield	21	20	2	1		
Dissolved Air Flot		2	3		3	1
Filters General	11	6	2	2		
Gravity Thicken	1	1		5	2	2
Ion Exchange	1					
Land Disposal General	9	1		1		
Land Spreading	75	23	5	2		1
Nitrification	1					
Oil and Grease Removal	44	9	1	1	2	
pH Control	12	18	10	4	3	2
Pressure Filters	1	2		1	6	1
Primary Gravel Sediment	158	18	8	1		
Prim/Prel	20	2	1		1	
RBCs	1	3				
Reverse Osmosis	2					
Ridge and Furrow	79	15	1	1		
Screening	21	11		1		
Solids Removal	2	1	1	1		
Solids Treatment	1	4	1	1		1
Spray Irrigation	38	37	18	5	3	1
Stabilization Pond	10	19	9	4	1	1
Trickling Filters	3	1	2			
Tube/Plate Set.	1		3	4	3	
Ultra Filtration	1					

	Process Wastewater	Contact Cooling Water	Non-Contact Cooling Water		Domestic Wastewater	Other
			With Additives	Without Additives		
Secondary Biological	1					
Secondary Clarification	2	9	16	12	6	
Multi Media Filter		2	1	3	1	2
Oxidation Ditch		1				2
No Treatment	299	13	2		2	
Packed Towers		1	1			
Floccul Basin			1	1		
Wet Oxidation			1	2		
Vacuum Filters			3	2		2

This latter factor, analysis suggests, was the largest disincentive to compliance, and until the advent of the Wisconsin Fund in 1978, Wisconsin municipalities were not succeeding in seeking compliance.

Federal Program

Between 1955 and 1966, 225 federal grant offers were made, totalling $51,904,183. Between 1966 and 1972, 89 projects (construction cost of $181,834,174.00) were funded at the 50% level or less and 166 projects (construction cost of $101,875,000.00) received the maximum grant share of 55.4%.[44] Since 1972 federal grants totalling $581,017,970.00 have been awarded to Wisconsin communities by the U.S. Environmental Protection Agency.[45]

State Funding

The first Wisconsin State water pollution abatement grant program was originated in August, 1966.[46]

In January of 1970, $144 million was allocated for a ten year period to assist municipal water pollution control facility development. An additional $13,085,000 was added in 1977. Over 600 grants were funded during this program.

In 1977 as earlier efforts ended the US-EPA federal program became mired in its own web of administrative ineptitude and congressional inaction, The Wisconsin legislature began to grapple with the reality of how to achieve compliance with the Federal Act within the time limits prescribed by law. In this introspection, certain conclusions were evident:

1. The lack of timely and consistent federal grant allocations destroyed the ability to expeditiously plan and build needed projects.

2. A shortage of planned and designed projects ready for construction caused unnecessary construction delays when money was made available.

3. Given the needs and rapidly accelerating inflation, the federal grants alone were not sufficient to carry out the federal law.

4. A single system of reviews, irrespective of funding source and requirements, for all municipal wastewater projects, was essential to prompt project planning and completion.

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[44]"Status of State Wastewater Grant Program," Unpublished Paper, Wisconsin Department of Natural Resources, July 18, 1975.

[45]"Step 1--393 grants totalling $116,929,578.00; Step 2--199 grants, $27,702,615.00; Step 3--163 grants, $436,385,777.00 (as of 1-3-83).

[46]Section 144.21, Wis. Stats.

5. To promote expeditious community action, a single priority system, equitably designed, evenly administered and essentially stable was necessary to provide local governments with a sense of progress.

6. Uniform statewide enforcement of Clean Water Act violations would need to be maintained to encourage timely completion of projects.

The Wisconsin Fund[47]

From these conclusions, the Wisconsin Legislature established the Wisconsin Fund to make possible a coordinated statewide program of construction. The fund was the glue to hold the program together; to provide adequate resources in a timely manner; to make planning and design adhere to a time schedule by having monies available on time; to consolidate review staffs from both the federal and the state programs with the assistance of delegation under the Cleveland-Wright Amendment[48] and to coordinate planning and design with consistent and applicable priority setting.

The Wisconsin Fund's philosophical premise was simple: that because of the vicissitudes of the federal program, with its stops and starts, changing rules and laws, funding cuts and rescissions, etc., no soundly conceived long-term project scheduling could be achieved and as a result, costs were uncontrollable.

As a grant program, the statute is simple. It provides for a system of grants to municipal government's similar in nature to the federal grant program, and while some differences in eligible costs were enacted[49] basic project concepts were maintained. Conditions such as the adoption of user fee systems and Operation and Maintenance plans were required and the federal priority system was adopted. Grant shares of 60% for step 3 and 75% for steps 1 and 2 grants were adopted. Since May, 1978, 248 step 3 grants have been awarded

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[47]144.24 Wis. Stats: The complete Wisconsin Fund program as proposed by the Governor and enacted by the Legislature included several elements including nonpoint source abatement grants, solid waste siting and planning monies, septic tank system rehabilitation and point source grants. For the purposes of this paper only the point source program is referred to by the term Wisconsin Fund.

[48]Section 205(g)(2), Federal Clean Water Act

[49]For example: 10 years of treatment plant reserve capacity, 1985 capacities for interceptors and collector systems eligibility only in previously unsewered communities.

totalling $265,603,369.[50] In addition, another $147,168,270 has been committed for 1983 construction.

As a result primarily of the Wisconsin Fund, by the end of the 1983-84 budget year it is expected that 90% of Wisconsin's 557 municipal facilities will comply with required discharge limits or be under construction.

Currently, a detailed inventory is underway as to processes in use in Wisconsin facilities.[51] Preliminary information shows, for example, that of the 557 facilities, 234 (42%) utilize Activated Sludge processes, 220 (36%) utilize ponds and lagoons and 123 (22%) use trickling filters or RBC processes. In terms of processes utilized, with approximately 60% of all permittees surveyed, Table 11 lists the results to date.

Current Municipal Construction Needs

Under the Federal Water Pollution Control Act, municipal treatment plants are required to achieve certain water quality objectives. Estimates of total expenditures necessary to reach these levels have been difficult to develop,

and in the early days of the Act, they were very bad. However, over time a basic system of biennial cost estimation developed by the U.S. EPA has begun to serve as a good data source for decision makers.

This system, while hampered on the national analysis level of political decisions changing the need definitions to fit particular national government budgetary purposes, has served Wisconsin well because of the state's decision to complete the 1976 survey on a 100% on-site survey basis.

Each subsequent survey has been designed as a correction from that base, modified to reflect changing conditions, court decisions and final cost estimates. In 1977, it was estimated that $2.3 billion in 1976 dollar costs would be necessary to meet pollution abatement standards between 1977 and 1990. It was also estimated that these costs would grow by $324 million during the same 13 years because of increased treatment need and capacity generated by population growth. Current estimates, based on 1982 data show for the first time an impact from the Wisconsin Fund expenditure and a <u>real</u> lessening of need.

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[50]Projects totalling $6,634,829 for step 2, and 14 projects of $550,047 have also been awarded. These lesser amounts reflect the tactical decision to fund most step 1's and step 2's out of federal monies. Thus, $419,856,515 of State Funds have been committed since 1978.

[51]Final data is expected to be available by August 1983 and will be supplied as a supplement and attachment for this section.

TABLE 11
Wisconsin Municipal
Wastewater Processes
1983-
PARTIAL DATA (60% of Permittees)

TREATMENT PROCESS	No. of Times Utilized	Percentage Total Processes Utilized
Secondary Biological		
Activated Sludge-Conventional	10	1.6
Activated Sludge-Contact Stabilization	20	3.2
Activated Sludge-Extended Aeration	8	1.3
Oxidation Ditch	6	1.0
Trickling Filters	53	8.6
Packed Towers	2	.3
RBC's	16	2.6
Aerate Lagoon	44	7.1
Stabilization Lagoon	50	8.1
Septic Tanks	2	.3
Activated Sludge-General	70	11.3
Post Aeration	20	3.2
Polishing Pond	23	3.7
Holding Pond	11	1.8
Nitrification	7	1.1
Sand filters	17	2.8
Chemical Addition	31	5.0
De Chlorination	2	.3
Filters-General	5	.8
Multi-Media Filters	1	.1
Land Disposal-General	3	.5
Absorption Pond	27	4.4
Spray Irrigation	2	.3
Ridge and Furrow	2	.3
Chlorine	182	29.4
Ultra Violet	5	.8

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Analysis: Current Municipal Construction Needs (Table 12)[52]

In viewing future pollution abatement needs in Wisconsin, it is necessary to review current estimates. And to fully interpret the fiscal situation for Wisconsin communities, it is necessary to separate "needs" into two categories: the Milwaukee Metropolitan area project and the balance of the state.

As table 12 indicates, by the end of 1983 construction year, outstate Wisconsin will have approximately $623,910,000 in unconstructed backlog projects. Of this amount $188,288,000 were identified as "needed" new collector systems; elements that are not generally necessary for immediate compliance with the Clean Water Act.[53] The data further identifies some need for combined sewer work ($125,829,000) and a significant amount of new interceptor activity. Much of this latter category would be in conjunction with new treatment facilities under construction or planned.

For the Milwaukee area, the situation is more pressing than in the balance of state. As of December 31, 1981, unmet Milwaukee project needs are estimated at $1,037,437,000. At the end of 1983 it is further estimated that $1,086,299,000 will remain.

In terms of the types of construction, Milwaukee represents the classic larger city. Treatment facilities in need of major work consist of 43% of the total cost. Major sewer rehabilitation (5%) is less extensive than one might expect, but new interceptors (31%) largely to accommodate already existing and overflowing volumes and to interconnect the two treatment facilities, is a major cost. And lastly, correction of ancient and overflowing combined sewer/storm water systems of the older city is estimated at 18%

Overall in fiscal and management terms Wisconsin's most serious challenge will be in the Milwaukee area.

The Milwaukee Program

In order to achieve compliance in Milwaukee the following elements are necessary:

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[52]Data for this paper is derived from: US EPA, Office of Water Programs Operations, 1982 Needs Survey, Cost Estimates for Construction of Publicly-owned Waste Treatment Facilities, December 31, 1982. As corrected for interpretation errors, costs and needs are shown as of December 31, 1981.

[53]Except in a few unique cases where severe pollution from an unsewered area is flowing directly into a body of water

Table 12
Wisconsin Municipal Needs - 1982 (000's)

Project Category	Milwaukee		Balance of State		
	Backlog	Year 2000	Backlog	Year 2000	
Secondary Treatment	564,883	564,883	220,000	331,000	
Advanced Secondary			86,000	143,000	
Advanced treatment Inflow/Infiltration	120	120	32,000	32,000	
Major Sewer Rehabilitation	71,265	71,265	7,735	8,735	
New Collectors	21,712	21,712	188,288	266,288	
New Interceptors	406,262	406,262	125,829	213,738	
Combined Sewer Overflow	243,195	243,195	125,829	125,829	
TOTALS (000's)	$1,307,437	1,307,437	799,590	1,120,590	2,107,027 2,428,027

Wisconsin Municipal Needs* - 1982
Adjusted

	Milwaukee	Balance of State
12/31/81 Backlog "needs"	1,307,437	799,590
1982 Construction	- 78,138	- 73,680
1982-83 Fundings	- 143,000	- 102,000
Net Balance FY 1984 Backlog	1,086,299	623,910

* To reflect 1982, 1983 construction activity for treatment plants
Milwaukee costs are court ordered expenditures

- Rehabilitate sewer lines to reduce clear water inflow and infiltration
- Construct relief sewers to abate overflow including 17 miles of large diameter storage/tunnels.
- Rehabilitate and expand Jones Island Wastewater Treatment Plant
- Rehabilitate and expand the Southshore Wastewater Treatment Plant
- Improve waste solids handling
- Improve and expend interceptors to all areas of the district and to eliminate some small inefficient local treatment facilities.
- Construction of storage facilities and interceptors to eliminate bypassing in local systems

The estimated total cost of the Milwaukee program is $2.1 billion. (1983 dollars):

ELEMENTS	ESTIMATED COSTS
- Jones Island	$478,824,000
- Southshore	$266,800,000
- Interceptors and relief sewers	$750,049,000
- Sewer rehabilitation	$791,011,000
- Off site solids management	$35,412,000
- Combined sewage overflow	$478,83,000
- Hydraulic and controls	$18,890,000
TOTALS	$2,107,878,000

By and large by the middle of this decade outstate communities will have completed their new construction. In Milwaukee design and construction must continue for 13-15 years.

Management and Structure

Management for wastewater projects in communities other than Milwaukee has traditionally been very simple. Each community (or group of communities where joint facilities are planned) hires a private consulting engineering firm to plan and ultimately design adequate treatment facilities. The municipality then submits its plans and designs to the Department of Natural Resources (WDNR) for review and approval. If grant funds are being requested, this process may be tailored to the special needs of a particular grant program. In all cases, facilities are designed to meet enforceable permit limits under the Federal Clean Water Act and comparable Wisconsin State Statutes.

Generally, the Department of Natural Resources through its field offices and its Division of Environmental Standards provides technical assistance and advice to municipalities throughout the process. In the case of failing older facilities, the Department may do existing plant analyses, to aid in determining the causes of old plant failure.

In local communities the oversight of planning and design consultant varies, from almost no technical oversight to detail project management. Generally, in smaller communities, facility planning and construction is totally entrusted to the consulting engineering who manages the project. Municipal

oversight is essential fiscal and political - providing the funds when necessary.

In larger communities where public works departments or sewerage commissions/districts exist -- more give and take occurs and the planning and design firm is essentially adjunct staff.

In Wisconsin at present, there are at least nine or more distinct types of statutorily approved municipal units of government providing sewer services.[54] Each of these type units operate somewhat differently and each type has a unique organic enabling statute.

And, while each and every one of these operating systems are appropriately permitted and regulated for pollutant discharge, the lack of consistency and structure does cause problems. This is most especially true in terms of management and fiscal capacity.

As treatment facilities and systems are constructed and become more sophisticated and more costly to build, operate and maintain, many of these districts will become hard pressed to function effectively and efficiently.

In practical terms, a legally organized district is viewed in today's managerial and fiscal world as rebuttally presumed to have the capacity to carry out its chartered activities. In many instances, the elimination of this presumption will only occur at the point of crisis or nonperformance.

To some, such a risk of large scale public capital investments with inadequate operating entities is concerning, but when coupled with the obvious environmental risk associated with nonperformance, it is critical.

In Wisconsin, the structure of managing agencies needs serious attention. Such a concern is further highlighted by the increased need to install adequate operation and maintenance practices in recently completed the facilities. Fundamental to this effort is the need to establish local community wastewater treatment as an "important" public responsibility. No longer is it possible for the small community to staff its treatment facilities with the unused time of more traditional employees such as street workers, snowplow operators or park employees. Modern facilities, require persons with training to manage the facilities and to regulate operations.

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[54]These include: joint sewer districts (pursuant to Section 144.07, Wis. Stats.); metropolitan sewerage districts (Section 66.02); city and village governments (Section 62.18); town sanitary districts (Section 66.072 and 60.30); town utility districts (Section 66.027); county utility districts (Section 59.083); intergovernmental contract districts (Section 66.30); and the Milwaukee metropolitan district (Chapter 282, Laws of 1981

Conclusion

Since October 18, 1972 it has been the policy of the United States to seek to "restore and maintain the . . . integrity" of the nation's waters and at the same time to "recognize, preserve, and protect the primary responsibilities and rights of states to prevent, reduce and eliminate pollution . . ."[55]

The management challenge of both pursuing pollution elimination and protecting the rights of the fifty states has been one of great difficulty and it has resulted in various results from state to state.

In Wisconsin, the pursuit of Clean Water has reached a higher than typical degree of success because of several unique circumstances. The current accomplishments in water quality improvement have been the result of advantageous historical circumstance, the coming together of administering agencies into a single management unit, the implementation of comprehensive management approaches, the enforcement of new and invigorated federal and state laws and the investment of billions of dollars by local and state government and private industries in new treatment facilities.

Wisconsin is, at present, within sight of achieving many of the treatment requirements of the Clean Water Act.

To finally achieve the ultimate goals of the Act and to eliminate the discharge of all pollutants will require much more than traditional point source treatment.

To this end, in the next decade, Wisconsin will see increased efforts at taming agricultural and urban run-off problems, eliminating toxic hazards, both air and water, encouraging nonpoint pollutant problem solving and building an understanding with all citizens of the impacts of run-off in the fundamental water quality of the State.

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[55]Public Law 92-500 Section 101

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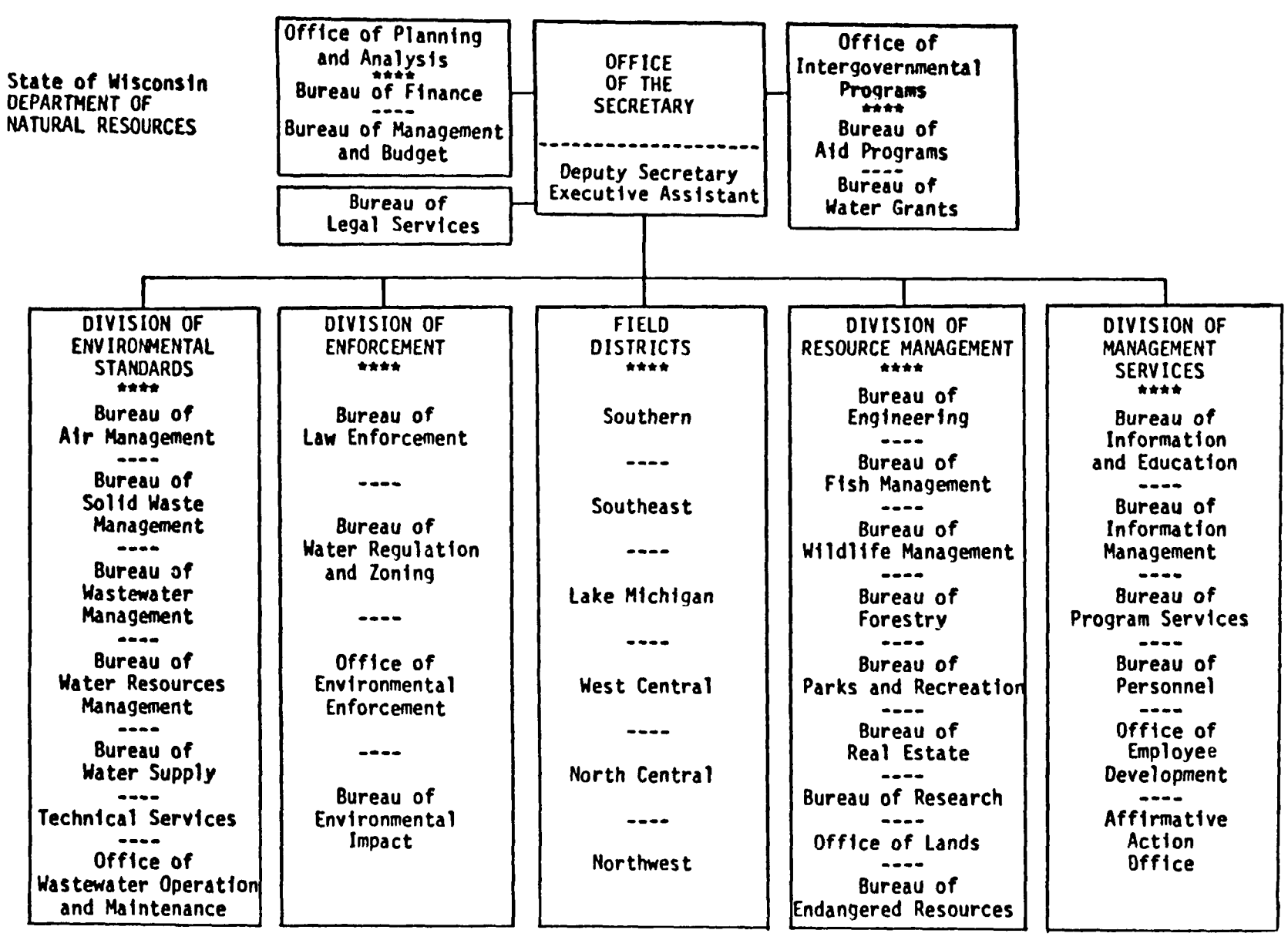
ATTACHMENT A



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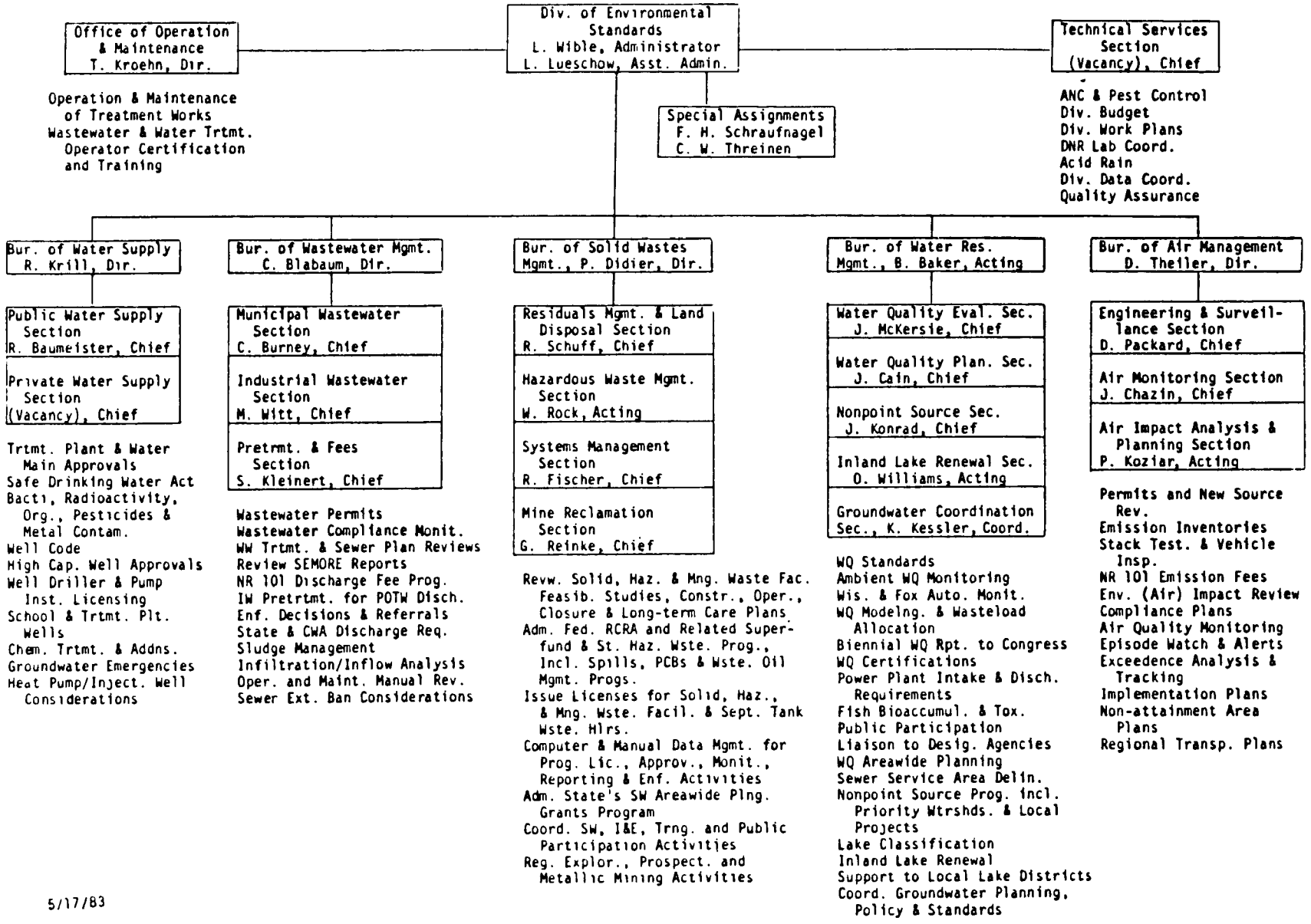
DIVISION OF ENVIRONMENTAL STANDARDS - ORGANIZATIONAL CHART

Air, Water, Wastewater, Solid Wastes and Water Resources Management Programs

ATTACHMENT B



5/17/83

REMOVAL AND PARTITIONING OF VOLATILE ORGANIC PRIORITY POLLUTANTS IN WASTEWATER TREATMENT

by

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REMOVAL AND PARTITIONING OF VOLATILE, ORGANIC PRIORITY POLLUTANTS IN WASTEWATER TREATMENT

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ABSTRACT

A pilot-scale study was conducted to evaluate the removal and partitioning of 16 volatile, organic priority pollutants in conventional waste water treatment processes. The apparatus used consisted of two, parallel, 2.2 l/s treatment sequences of primary clarifiers, aeration basins, and secondary clarifiers. The aeration basins were covered to permit quantitation of stripping losses. One treatment sequence was used as a control, to provide data at ambient concentrations; the other system was continuously spiked with a methanol solution containing the compounds investigated.

Substantial removals of all compounds were observed, with nine compounds having removals >99 percent; 1,1,2-trichloroethane had the lowest removal, 80 percent.

Desorption from the primary clarifier accounted for approximately 24 percent of the losses but was observed to be functionally dependent on the Henry's Law constants of the compounds studied. Stripping from the aeration basin was a major removal mechanism. The stripping losses observed in the study were compared with losses predicted by a clean water stripping model. The observed losses were lower than the predicted losses, and the difference was attributed to sorption to the biomass.

This paper has been reviewed in accordance with the U.S. Environmental Protection Agency's peer and administrative review policies and approved for presentation and publication.

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INTRODUCTION

Controlling the discharge of toxic materials to the environment is essential for the protection of our natural resources. The technological advances of the last 70 years have made possible the manufacture of a wide variety of synthetic materials for various end uses. Some of these materials exhibit mutagenic, teratogenic, or carcinogenic characteristics, and a significant number of the xenobiotic chemicals persist in the environment for protracted periods.

The U.S. EPA has principal responsibility for limiting the entry of toxic materials to the environment, and during the past several years, the Agency has conducted a number of investigations concerning alternate control strategies and technologies. One area of concern is the ability of the municipal wastewater treatment plant to protect receiving waters from the intrusion of toxic materials.

The Agency's Municipal Environmental Research Laboratory (MERL) has been conducting both extramural and in-house research to evaluate the removability and partitioning of toxic materials in publicly owned treatment works (POTW's). This paper presents the results of a pilot-plant study conducted at the Agency's Test and Evaluation Facility (T&E) in Cincinnati, Ohio to evaluate the removal and partitioning of volatile organic priority pollutants in POTW's. Similar research concerning the semivolatile organic priority pollutants and the metals has been reported previously (1,2).

EXPERIMENTAL PROTOCOL

PILOT PLANT OPERATIONS

The principal objective of this project was to quantitate the removals of the volatile, organic priority pollutants that could be expected in a conventional POTW. Other objectives were to develop additional data relative to the partitioning and stripping of the materials in conventional wastewater treatment processes.

The treatment sequence selected as being most representative of a typical POTW was primary clarification followed by conventional, plug-flow, activated sludge. Two, parallel, 2.2 l/s (35 gpm) pilot plants were used for this study and are shown in a simplified schematic diagram in Figure 1. The degritted and comminuted raw wastewater was pumped from the Cincinnati Mill Creek Wastewater Treatment Plant to a head tank at the T&E Facility. To avoid plugging of process piping, gross solids were removed by a 1.27 mm static screen at the project site. The wastewater was then pumped to the influent splitter box. Nominal operating conditions for the pilot-scale treatment sequences are presented in Table 1.

The primary clarifiers were 2.95 m in diameter and had a side water depth (SWD) of 3.6 m. At 2.2 l/s, the overflow rate was 28 $m^3/m^2 \bullet$ day.

The aeration basins were 5.4 m long, 3 m wide, and had a SWD of 3.6 m, which resulted in a theoretical residence time of 7.5 hrs at a flow of 2.2 l/s. Conventional Sanitare® coarse bubble diffusers were used for the study, and the nominal air flow was 57 l/s. The secondary clarifiers were 3.6 m in diameter and had a SWD of 3.6 m. At 2.2 l/s the overflow rate was 18.4 $m^3/m^2 \bullet$ d.

Every four hours the operations staff monitored the influent and return activated sludge (RAS) flows and the level of the spike stock solution, which was fed into the influent of the experimental system with a metering pump. Furthermore, grab samples were collected for monitoring mixed liquor (ML) and RAS oxygen uptake rates (OUR's), ML and RAS centrifuge volumes, ML settled sludge volume, pH, turbidity, and alkalinity.

Routine analyses for total suspended solids (TSS), the nitrogen series, total phosphorus, and chemical oxygen demand (COD) were performed on 24-hr composite samples.

ORGANICS SAMPLING AND ANALYSIS

The compounds that were studied are shown in Figure 2, and are given in Table 2, along with some of the more important physical/chemical properties for the compounds. The spike solution, containing the 16 compounds that were studied, was prepared weekly; methanol was used as the solvent. A one-liter, stoppered, graduated cylinder was used as the spike solution reservoir, and pumping rates were monitored by recording the volume in the reservoir every four hours.

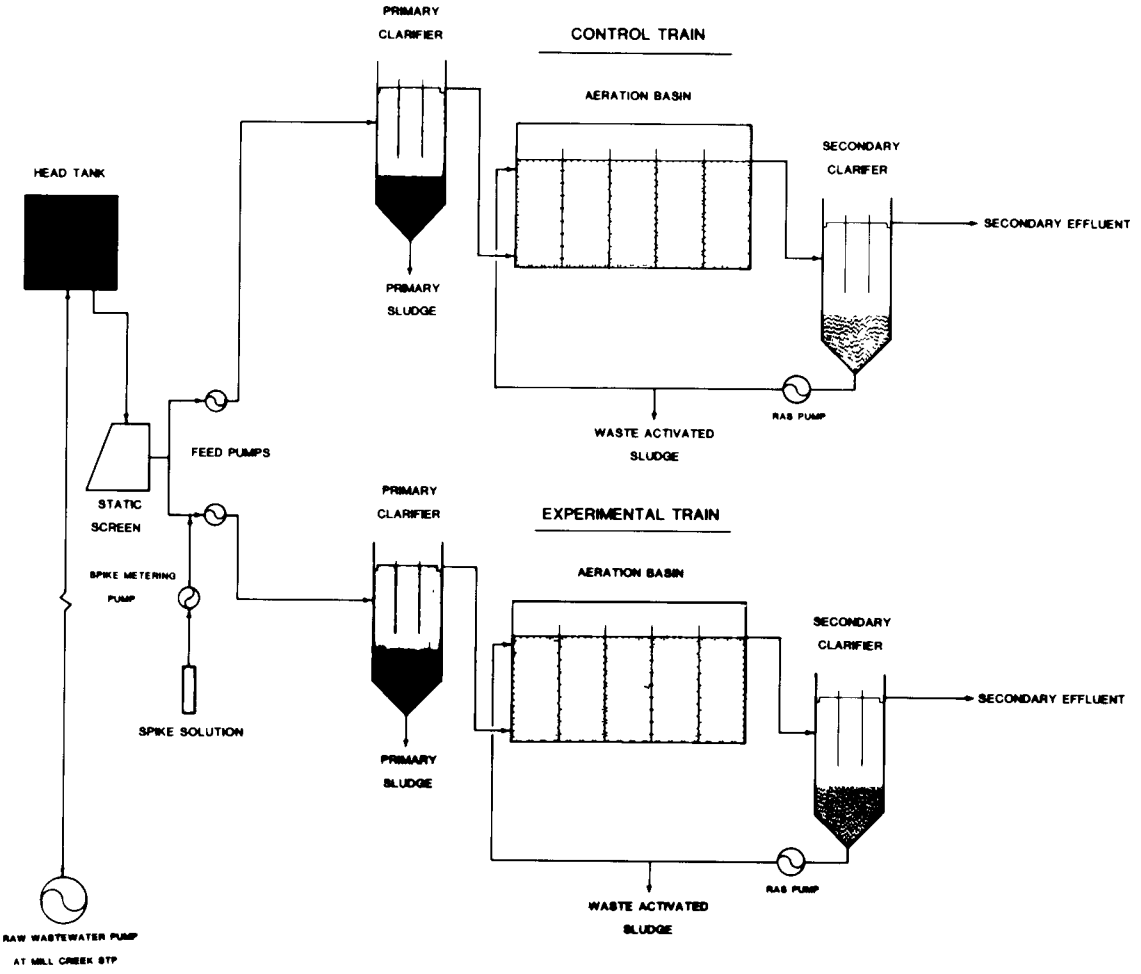

Figure 1. Simplified process schematic diagram of volatile priority pollutant pilot plant.

TABLE 1. NOMINAL OPERATING CONDITIONS FOR THE A AND B SYSTEMS USED ON THE VOLATILE PRIORITY POLLUTANT PROJECT

I. Design Flow, Qd = 35 gpm
= 50,400 gpd

II. Primary Clarifiers -
Diameter = 9'-8"
Weir Diameter = 9'-1"
SWD = 12'-0"
Surface Area = 73.4 ft^2
Surface Overflow Rate = 687 gpd/ft^2

III. Aeration Basins -
L:W:D = 17'-7":10'-0":12'-0"
Surface Area = 175.8 ft^2
Volume = 2,110 ft^3
= 15,780 gal.
Residence Time (Qd) = 7.5 hrs.

IV. Secondary Clarifiers -
Diameter = 11'-11"
SWD = 12'-0"
Surface Area = 111.5 ft^2
Surface Overflow Rate = 452 gpd/ft^2

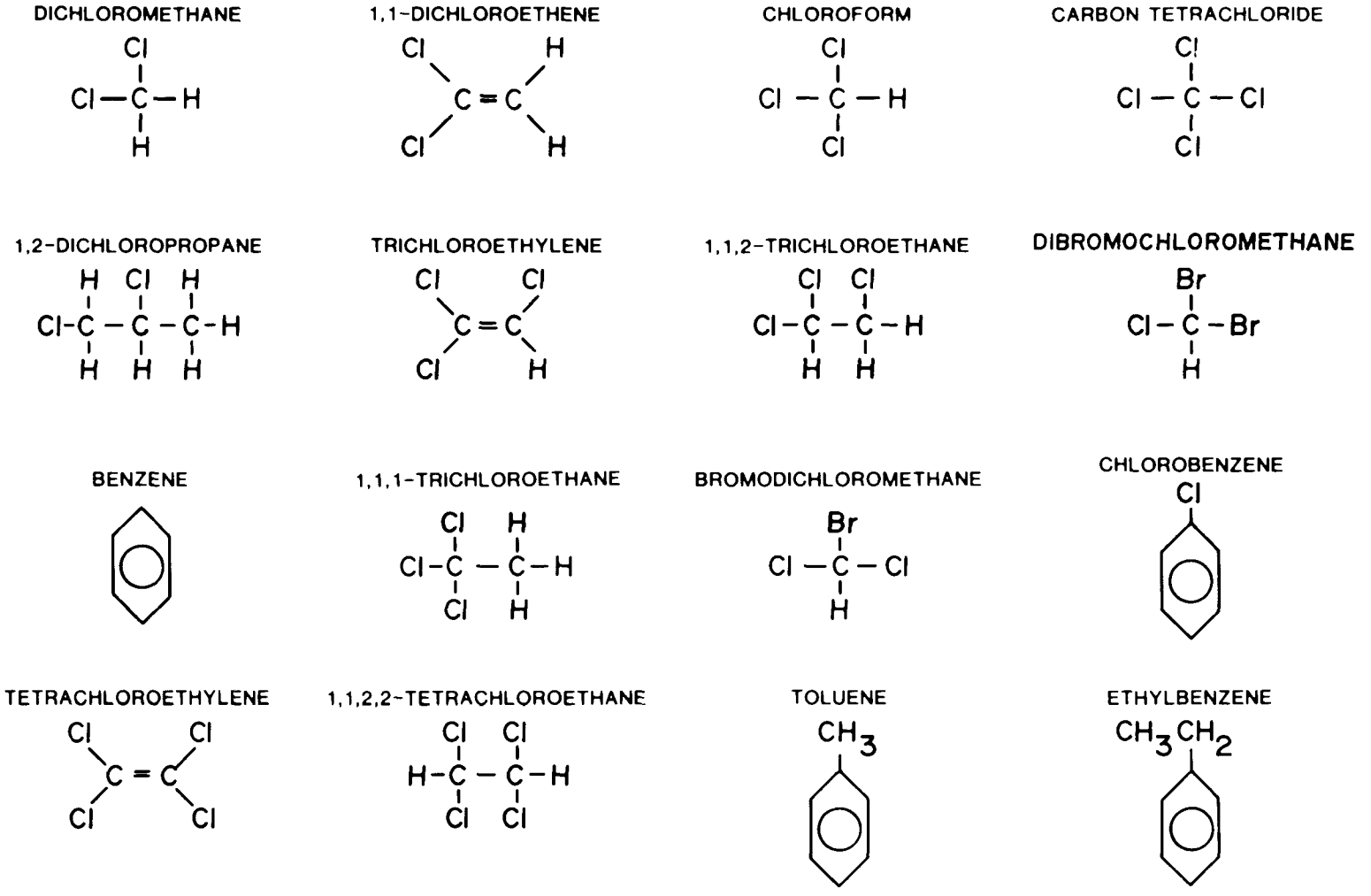

Figure 2. Volatile priority pollutants studied.

TABLE 2. COMPOUNDS STUDIED AND SELECTED PHYSICAL/CHEMICAL PROPERTIES

Compound	Henry's Law Constant H_c ($g \cdot m^{-3}$/ $g \cdot m^{-3}$)	Henry's Law Constant H ($M^3 \cdot atm$/ mol. x 10^{-3})	K_{ow} (Octanol/ Water Partition Coeff.)
Methylene Chloride	0.13	3.19	18
1,1-Dichloroethene	0.62	15.0	30
Chloroform	0.16	3.93	93
Carbon Tetrachloride	1.26	30.2	437
1,2-Dichloropropane	0.12	2.82	190
Trichloroethylene	0.49	11.7	195
1,1,2-Trichloroethane	0.03	0.74	148
Dibromochloromethane	0.03	0.78	123
Benzene	0.23	5.55	135
1,1,1-Trichloroethane	0.20	4.92	148
Bromodichloromethane	0.09	2.12	76
Chlorobenzene	0.16	3.93	691
Tetrachloroethylene	1.19	28.7	758
1,1,2,2-Tetrachloroethane	0.16	0.38	363
Toluene	0.25	5.93	490
Ethylbenzene	0.27	6.44	1412

In order to circumvent the problems associated with the compositing of samples containing volatile organic compounds, grab samples were used for these analyses. The samples were collected in 40-ml glass vials with Teflon®-sealed screw caps.

The off-gases from the aeration basins were sampled with Tenax traps. All gas samples were collected in duplicate, at a flow rate of 100 cc/min for 10 min. The one-liter sample volume was determined from breakthrough studies conducted on the Tenax traps.

The compounds were analyzed by EPA Method 624 (3). The GC column was 1% SP-1000 on Carbopack B, 180 cm by 2 mm ID, which was programmed at 8° C/min after a 2-min hold at 50° C.

Samples were added to the column using the Bellar-Lichtenberg purge and trap technique with a Tekmar LSC-1 apparatus. A 5-ml sample was purged for 11 minutes with 40 ml/min of helium onto a Tenax/silica gel trap. The trap was desorbed onto the column, which was at room temperature, by heating the trap to 180° C and backflushing with the GC carrier gas for three minutes. The column was then heated to 50° C, beginning the previously-described program. The mass spectrometer was turned on and data acquisition initiated.

Between runs the trap was baked at 225° C with a helium flow.

The mass spectrometer was a Finnigan 3300 which scanned from 35-400 amu every three seconds. The instrument was tuned to meet the p-Bromofluorobenzene (BFB) criteria. This compound was also used as the internal standard in each run.

RESULTS

CONVENTIONAL PARAMETERS

During this 12-month study, the average mixed liquor suspended solids (MLSS) concentrations for the control and experimental systems were 2,400 and 2,900 mg/l, respectively. The average sludge retention time (SRT) for the control system was 5.3 days, and for the experimental system was 5.9 days. The mean ML-OUR was 36 mg/l•h for the control and 40 mg/l•h for the experimental process; other operational parameters are presented in Table 3.

Table 4 summarizes the performance of both treatment sequences with respect to the conventional water quality parameters. Data are presented only for the control system primary clarifier. After two months of almost identical, parallel operation, the other primary effluent sample was deleted to reduce laboratory workload.

TSS and COD removals were good for both systems, averaging 93 and 86 percent, respectively. At the relatively high SRT's, substantial nitrification occurred during much of the project, as the nitrogen series data indicate. The effluent TSS concentrations and turbidity values for both systems indicate that good treatment was obtained during the study.

VOLATILE ORGANIC COMPOUNDS

During this study, a total of 20 sets of samples were collected for volatile organics analysis. The mean concentrations observed in the control treatment sequence are presented in Table 5, along with percent removal data for the primary clarifier and the entire treatment sequence.

The influent data represent ambient concentrations in the raw wastewater after static screening, and a "less than" indicates that at least one sample analysis was below detection limits. Toluene and chlorobenzene were both present in relatively high concentrations, 160 and 102 µg/l, respectively. Additionally, methylene chloride, 1,1-dichloroethene, chloroform, 1,1,1-trichloroethane, ethylbenzene, tetrachloroethylene, and tetrachloroethane were all present at concentrations greater than 10 µg/l.

The removal data for the primary clarifier vary considerably from compound to compound, but in general, substantial removals were observed, with an average of 36 percent for all compounds. However, the concentrations observed in the primary sludge were quite low, which indicates that partitioning to the primary sludge was not a significant removal mechanism.

Very low concentrations were observed for all compounds in the RAS and the activated sludge effluent. The removals observed in the control treatment sequence were very good for most of the compounds studied. However, in several instances the influent concentrations were too low to properly assess removability. The influent to the experimental system was spiked with the compounds being studied to improve the quantitation of removability, and Table 6 presents the mean concentrations observed in the

TABLE 3. SUMMARY OF PROCESS OPERATION DURING THE VOLATILE PRIORITY POLLUTANT STUDY

Parameter	System			
	Control		Experimental	
Influent Flow - l/s (gpm)	2.33	(37.0)	2.11	(33.5)
RAS Flow - l/s (gpm)	0.62	(9.8)	0.64	(10.2)
WAS Flow - l/d (gpd)	1980	(523)	1998	(528)
Primary Sludge Flow - l/d (gpd)	1889	(499)	2290	(605)
Air Flow - l/s (scfm)	56.6	(120)	56.6	(120)
MLSS - mg/l		2,439		2,886
RAS - mg/l		10,987		11,889
ML Cent. Vol. - percent		3.1		3.7
RAS Cent. Vol. - percent		15.5		14.9
ML - OUR - mg/l•h		35.8		40.4
RAS - OUR - mg/l•h		101.6		88.2
Normalized ML - OUR - hr^{-1}		0.015		0.014
Normalized RAS - OUR - hr^{-1}		0.009		0.007
SVI - ml/gm		122		153
SRT - days		5.3		5.9

TABLE 4. SUMMARY OF PROCESS PERFORMANCE DURING THE VOLATILE PRIORITY POLLUTANT STUDY

Parameter	Influent (mg/l)	Primary Effluent (mg/l)	Primary Clarifier Removal (percent)	Activated Sludge Effluent		Total Plant Removal	
				Control (mg/l)	Experimental (mg/l)	Control (percent)	Experimental (percent)
TSS	416	202	51	28	30	93	93
COD	573	329	43	88	77	85	87
Total-P	9.3	6.3	32	3.2	3.1	66	67
TKN	42.4	36.2	15	11.9	11.4	72	73
Organic-N	16.6	11.1	33	3.9	3.7	76	78
NH_3-N	25.8	25.1	3	8.0	7.7	69	70
NO_2-N	0.1	0.1	-	1.1	0.65	-	-
NO_3-N	0.18	0.3	-	13.1	15.1	-	-
Total-N	42.7	36.6	14	26.1	27.2	39	36
Alkalinity (as $CaCO_3$)	-	251		91	90	-	-
Turbidity (NTU)	-	199		11	10	-	-

TABLE 5. MEAN CONCENTRATIONS OBSERVED IN THE CONTROL TREATMENT SEQUENCE

Compound	Influent (µg/1)	Primary Effluent (µg/1)	Removal by Primary Clarifier (percent)	Activated Sludge Effluent (µg/1)	Removal by Treatment Sequence (percent)	Return Activated Sludge (µg/1)	Primary Sludge
Methylene Chloride	<30.6	<22.3	27	<2.5	>92	<1.4	<40.4
1,1-Dichloroethene	<10.7	<3.7	65	<0.2	>98	bdl	bdl
Chloroform	10.8	7.2	33	<0.4	>96	<3.4	<7.5
Carbon Tetrachloride	<6.2	<3.3	47	<0.2	>97	bdl	<1.5
1,2-Dichloropropane	<0.2	<0.3	-	<0.2	-	<1.2	<3.6
Trichloroethylene	<4.2	<2.9	31	<0.2	>95	bdl	<23.3
1,1,2-Trichloroethane	<2.9	<0.5	83	<0.6	>79	<6.5	<3.2
Dibromochloromethane	<0.6	<0.3	50	<0.2	>67	<1.3	bdl
Benzene	<2.7	<2.7	-	<0.3	>89	<1.5	<9.0
1,1,1-Trichloroethane	<65.0	34.2	48	<0.9	>99	<1.5	<37.2
Bromodichloromethane	<0.2	<0.2	-	<0.2	-	bdl	bdl
Chlorobenzene	102	87.3	14	<0.5	>99	<1.6	648
Tetrachloroethylene and Tetrachloroethane	<24.0	<2.8	88	<0.2	>99	<3.0	164
Toluene	160	114	29	7.3	95		654
Ethylbenzene	<24.5	7.3	70	<0.2	>99	bdl	283

bdl = below detection limit

TABLE 6. MEAN CONCENTRATIONS OBSERVED IN THE EXPERIMENTAL TREATMENT SEQUENCE

	Influent (µg/l)	Primary Effluent (µg/l)	Primary Sludge (µg/l)	RAS (µg/l)	Activated Sludge Effluent (µg/l)	Aeration Basin Off-gas (ng/l)	Removal by Primary Clarifier (percent)	Removal by Treatment Sequence
Methylene Chloride	118	89	<143	<21	<4.0	-	25	>97
1,1-Dichloroethene	79	34	<40	<1	<0.2	425	57	>99
Chloroform	137	143	<208	<7	3.6	838	0	>97
Carbon Tetrachloride	60	32	<14	<1	<0.2	322	47	>99
1,2-Dichloropropane	309	295	<461	<1	<6.0	1628	5	>98
Trichloroethylene	107	68	389	<1	<1.5	609	36	>99
1,1,2-Trichloroethane	133	155	<219	31	28.0	663	0	80
Dibromochloromethane	58	49	<10	<2	<7.0	1291	16	>88
Benzene	73	61	121	<1	<0.2	225	16	>99
1,1,1-Trichloroethane	132	96	<220	<1	<0.3	1022	27	>99
Bromodichloromethane	89	64	<25	<1	<0.2	1155	28	>99
Chlorobenzene	197	163	953	<5	<1.3	149	17	>99
Tetrachloroethylene and Tetrachloroethane	252	263	2033	25	16.0	1227	0	94
Toluene	255	198	974	<2	<0.6	672	22	>99
Ethylbenzene	82	69	766	<1	<0.2	243	16	>99

experimental system during this investigation. Due to the ubiquitousness of methylene chloride in the organics laboratory, the concentrations of that compound found in the aeration basin off-gas were considered unreliable and they are not reported.

The data for the primary clarifiers indicate that substantial decreases in concentration occurred for most compounds, the average decrease being 24 percent. However, the concentrations observed in the primary sludges were low, which indicates that partitioning to the sludge was not the principal removal mechanism. Losses due to volatilization at the clarifier surface, the weir plate, and in the effluent channel appear to be the major points at which the volatile compounds leave the primary clarifiers.

With the exception of 1,1,2-trichloroethane and dibromochloromethane, very good removals were observed in the spiked process. The average removal for all compounds was greater than 96 percent; 60 percent of the compounds were present in the effluent at a concentration less than 2.5 µg/l, and 45 percent were found at concentrations less than 1.0 µg/l.

TOXICITY REDUCTION

During this investigation, static acute toxicity tests were conducted using fathead minnows (Pimephales promelas), an invertebrate (Daphnia magna), and a bacteria toxicity assay (Microtox® by Beckman Instruments, Inc.). The procedures used followed the guidelines of Peltier (4) and have been reported in detail by Neiheisel (5). The data presented in Table 7 summarize the results obtained when all three organisms were being used for the studies, and represent six sets of toxicity tests. The values reported are the percent of sample in the test container necessary to cause the desired organism response.

For all three sample locations, the samples from the spiked, experimental system were no more toxic than these from the control system. Additionally, for all three organisms there was no significant difference in the toxicity of the influent and primary effluent samples. The LC50 values for the minnows and the Daphnia magna ranged from 30 to 40 percent and for the Microtox® from 8 to 10 percent. The secondary effluent samples were not acutely toxic to the organisms used.

TABLE 7. SUMMARY OF ACUTE TOXICITY DATA

Sample		Fathead Minnow 96-hr LC50 in %[a]	Daphnia magna 48-hr LC50 in %[a]	Microtox® 15-min EC50 in %[a]
Influent	Cont.	32.1	33.8	9.7
	Expt.	31.9	31.0	7.8
Primary Effluent	Cont.	36.3	39.8	9.6
	Expt.	32.4	35.6	8.6
Secondary Effluent	Cont.	>100	>100	>100
	Expt.	>100	>100	>100

[a] percent of sample in dilution

DISCUSSION

The data presented in Table 8 summarize the distribution of the volatile compounds in the experimental treatment sequence. The primary sludge did not contain appreciable amounts of most of the compounds. Some partitioning to the solids is evident, especially for chlorobenzene, ethylbenzene, toluene, tetrachloroethylene, and tetrachloroethane.

Several investigators have suggested that the octanol-water partition coefficient, K_{ow}, is a useful tool for estimating the sorption of organic compounds to particulate material (6,7,8). Figure 3 is a plot of primary sludge concentration factors (F_s) versus the K_{ow}'s. The sludge concentration factors, for each compound studied, were computed by dividing the concentrations in the primary sludge (µg/l) by the influent concentrations (µg/l). The correlation between F_s and K_{ow} is reasonably good.

The major losses of the volatile organic compounds during primary clarification are due to desorption. Thibodeaux (9) has discussed desorption from basins and rivers, and has presented desorption data for 11 compounds from aerated basins. Liss and Slater (10) presented a model of gas exchange at the air-sea interface, based on the two-film theory. They calculated overall mass transfer coefficients (K_L) ranging from 10.6 to 20 cm • h^{-1}, with the liquid phase transfer coefficients (k_l) ranging from 10.7 to 20 cm • h^{-1} and the gas phase transfer coefficients (k_g) varying from 1,030 to 1,900 cm • h^{-1}. Mackay and Leinonen (11) extended previous work on desorption of low solubility compounds (12) to include the liquid film resistances reported by Liss and Slater (10). Calculated parameters for 14 compounds were reported, with K_L's ranging from 1.5×10^{-4} m • hr^{-1} for Lindane to 0.144 m • h^{-1} for benzene.

Dilling, et al. (13) conducted a study of the evaporation rates of five chlorinated compounds. The studies were conducted in deionized water with initial concentrations of 1.0 mg/l. In subsequent work, Dilling (14) presented evaporation rate data for 27 chlorohydrocarbons. The experiments were conducted in 250-ml stirred beakers (200 rpm). The observed rates were generally first order, and the experimental data agreed reasonably well with rates calculated by using the methods of Liss and Slater (10) and Mackay and Leinonen (11).

Thibodeaux (9) has presented models for desorption from both plug-flow and completely-mixed basins. The fraction desorbed from a plug-flow basin can be calculated by:

$$F_p = 1 - e^{(-K_L a_v T)} \qquad (1)$$

where F_p is the fraction desorbed from the basin, K_L is the overall liquid phase mass transfer coefficient (L • t^{-1}), a_v is the interfacial area per unit volume (L^{-1}), and T is the mean residence time (t).

TABLE 8. SUMMARY OF MASS BALANCES ON THE EXPERIMENTAL SYSTEM

Compound	Percent of Each Compound Found at Indicated Sample Location			
	Primary[a] Sludge	Activated[b] Sludge Effluent	Waste[b] Activated Sludge	Aeration[b] Basin Off-gas
Methylene Chloride	1.4	<0.5	<0.3	-
1,1-Dichloroethene	<0.7	<0.1	<0.1	72
Chloroform	2.0	<0.2	<0.1	34
Carbon Tetrachloride	<0.3	<0.1	<0.1	59
1,2-Dichloropropane	<2.0	<0.2	<0.1	32
Trichloroethylene	4.7	<0.3	<0.1	41
1,1,2-Trichloroethane	2.1	<1.8	0.4	25
Dibromochloromethane	<0.2	<1.4	<0.1	>100
Benzene	2.3	<0.1	<0.1	15
1,1,1-Trichloroethane	2.1	<0.1	<0.1	62
Bromodichloromethane	<0.4	<0.1	<0.1	>100
Chlorobenzene	<6.6	<0.1	<0.1	5
Tetrachloroethylene and Tetrachloroethane	10.7	0.6	0.2	27
Toluene	5.1	<0.1	<0.1	20
Ethylbenzene	12.3	<0.3	<0.1	21

[a] based on influent concentrations

[b] based on primary effluent concentrations

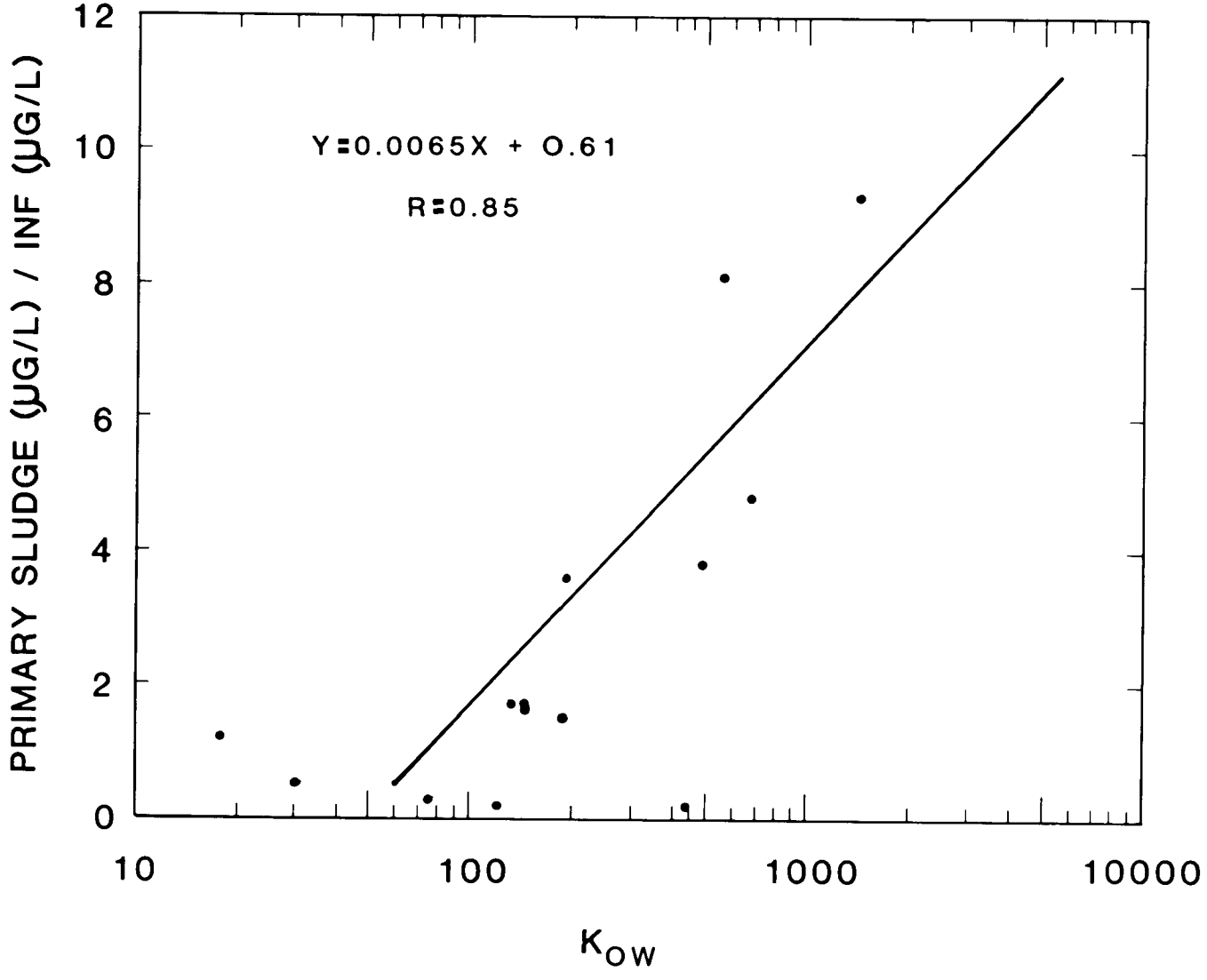

Figure 3. Primary sludge concentration factors, primary sludge concentration divided by influent concentration, versus K_{ow} for compounds studied.

Other investigators (15,16,17) have shown that the gas and liquid phase mass transfer coefficients can be related to K_L by:

$$\frac{1}{K_L} = \frac{1}{k_1} + \frac{1}{k_g H_c} \quad (2)$$

where k_1 and k_g are the liquid and gas film mass transfer coefficients ($L \bullet t^{-1}$), respectively, and H_c is the Henry's law constant ($m \bullet l^3/m \bullet l^3$).

Henry's law constants, H_c, are dimensionless ($g \bullet m^{-3}/g \bullet m^{-3}$) partitioning ratios which are related to the more frequently-encountered data in the literature by:

$$H = H_c\ RT \quad (3)$$

where H is the Henry's law constant ($atm \bullet m^3 \bullet mol^{-1} \bullet K^{-1}$); R is the universal gas constant, $R = 8.206 \times 10^{-5}$ ($atm \bullet m^3 \bullet mol^{-1} \bullet K^{-1}$); and T is the absolute temperature (°K).

As shown by Equation 2, the overall transfer coefficient, K_L, is a function of the H_c for any given compound. Therefore, the amount desorbed from a primary clarifier will be functionally dependent on the H_c of the particular compound. Figure 4 is a plot of the fraction of each compound desorbed, F, versus the H_c for each compound. The data exhibit considerable scatter; however, the general trend is apparent.

The fraction of each compound desorbed was computed from the data in Table 6, and Equation 1 was used to compute K_L's for each compound. In order to determine values for k_1 and k_g, K_L^{-1} was plotted versus H_c^{-1} (Equation 2). The intercept then gives k_1^{-1} (k_1 = 37.9 cm $\bullet$ h^{-1}), and the slope is k_g^{-1} (kg = 1400 cm $\bullet$ h^{-1}).

Liss (17) has defined the total resistance to mass transfer, R_t, as K_L^{-1} and the individual liquid and gas phase resistances (r_1 and r_g) as k_1^{-1} and $(H_c k_g)^{-1}$, respectively. Therefore, Equation 2 may be rewritten as:

$$R_t = r_1 + r_g \quad (4)$$

For those compounds with H_c's of about 0.024, the resistances in both the gas and liquid phases are approximately equal. For compounds with Henry's constants greater than 0.03, the liquid film resistance will control mass transfer, since r_1 will be much greater than r_g.

The values obtained for k_1 and k_g, and the Henry's constants, were used to compute overall mass transfer coefficients, K_L's, for each compound. The K_L's are presented in Table 9, along with overall mass transfer coefficients reported by other investigators. With the exception of this study, all of the values in Table 9 are based on either tap water or distilled water as the matrix.

The K_L's determined for compounds with low H_c's are in good agreement with the other data. However, many of the other K_L's are two to three

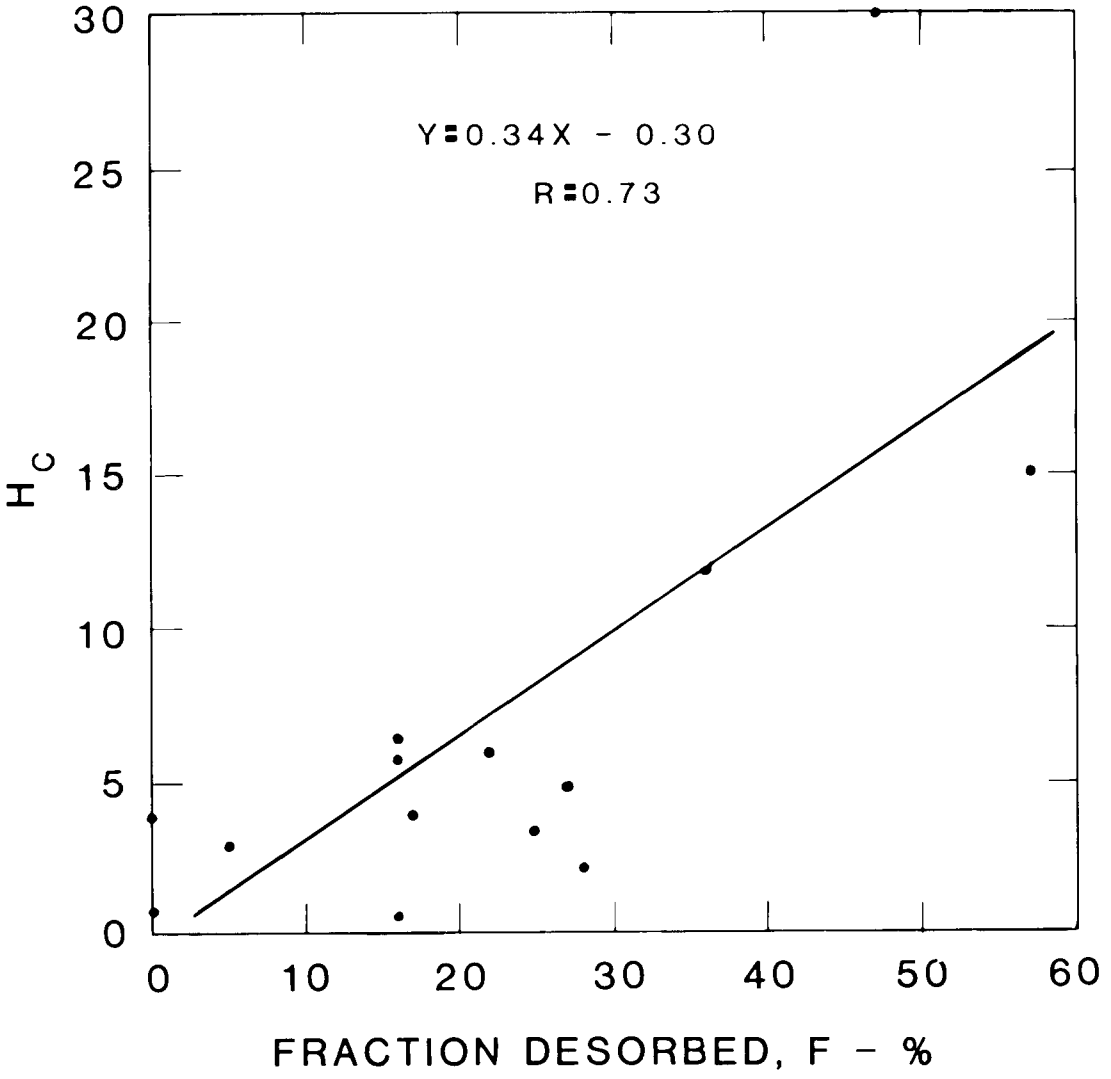

Figure 4. Influence of H_c on desorption from the primary clarifier of the experimental system.

TABLE 9. COMPARISON OF REPORTED VALUES OF THE OVERALL MASS TRANSFER COEFFICIENT, K_L, FOR DESORPTION

Compound	Reported Values Of K_L (cm • h^{-1})				
	Mackay and Leinonen (11)	Dilling (14)	Liss and Slater (10)	Kyosai, et al. (20)	This Study
Methylene Chloride		13.1			31.4
1,1-Dichloroethene		13.4		17	36.3
Chloroform		11.4			32.4
Carbon Tetrachloride		10.6	10.7		37.1
1,2-Dichloropropane					30.9
Trichloroethylene		11.3		11	35.9
1,1,2-Trichloroethane				13	19.9
Dibromochloromethane				17	19.9
Benzene	14.4				33.9
1,1,1-Trichloroethane		11.4		15	33.4
Bromodichloromethane				15	29.1
Chlorobenzene					32.4
Tetrachloroethylene		10.2			37.1
Tetrachloroethane		6.7		13	32.4
Toluene	13.3			10	34.2
Ethylbenzene				7	34.4

times the previously-reported values. Many factors may effect desorption, so the differences are not surprising. The presence of other compounds, electrolytes, oils and greases, and sorbents in the wastewater will result in non-ideal behavior. Furthermore, the hydrodynamics of the various experimental systems have not been well-defined. The energy input to any system will influence the film renewal rate and, therefore, the desorption which will occur.

Equation 1 indicates that K_L and a_v will predominantly influence the desorption from primary clarifiers. The residence time is clearly an important variable, but in practice it generally ranges from 1.5 to 2 h; therefore, it is relatively constant. The interfacial surface area per unit volume, a_v, is the clarifier depth for units with uniform cross sections. Figure 5 presents several curves of the fraction that can be expected to be desorbed from primary clarifiers as a function of the clarifier depth and the H_c of the particular compound.

The data in Table 8 indicate that substantial amounts of all compounds were found in the off-gas from the aeration basin of the spiked system. Interpretation of the off-gas data is, however, quite involved since the three mechanisms of sorption, biodegradation, and stripping may have been occurring simultaneously for several of the compounds studied.

Roberts, et al. (18) have presented a model to predict stripping by bubble aeration, in completely-mixed, clean water systems, in which the reciprocal of the removal may be computed as:

$$\frac{C_{LI}}{C_{LE}} = 1 + \frac{H_c Q_G}{Q_L} [1 - e^{-\emptyset Z_s}] \quad (5)$$

where C_{LI} and C_{LE} are the influent and effluent liquid phase concentrations, $M \bullet L^{-3}$; H_c is the Henry's law constant, $g \bullet m^{-3}/g \bullet m^{-3}$; Q_G is the gas flow rate, $m^3 \bullet s^{-1}$; Q_L is the liquid flow rate, $m^3 \bullet s^{-1}$. $\emptyset Z_s$ is defined as given below:

$$\emptyset Z_s = \frac{K_{La} \bullet V_L}{H_c \bullet Q_G} \quad (6)$$

where K_{La} is the overall mass transfer coefficient, s^{-1}, and V_L is the liquid volume, m^3.

The fraction stripped, η, may be computed by:

$$\eta = 1 - \frac{C_{LE}}{C_{LI}} \quad (7)$$

The model was used to estimate stripping losses from the aeration basin utilized during this study. A computer program was used to model the basin as four completely-mixed reactors in series and compute effluent concentrations for values of H_c ranging from 0.01 to 2.1. Two different

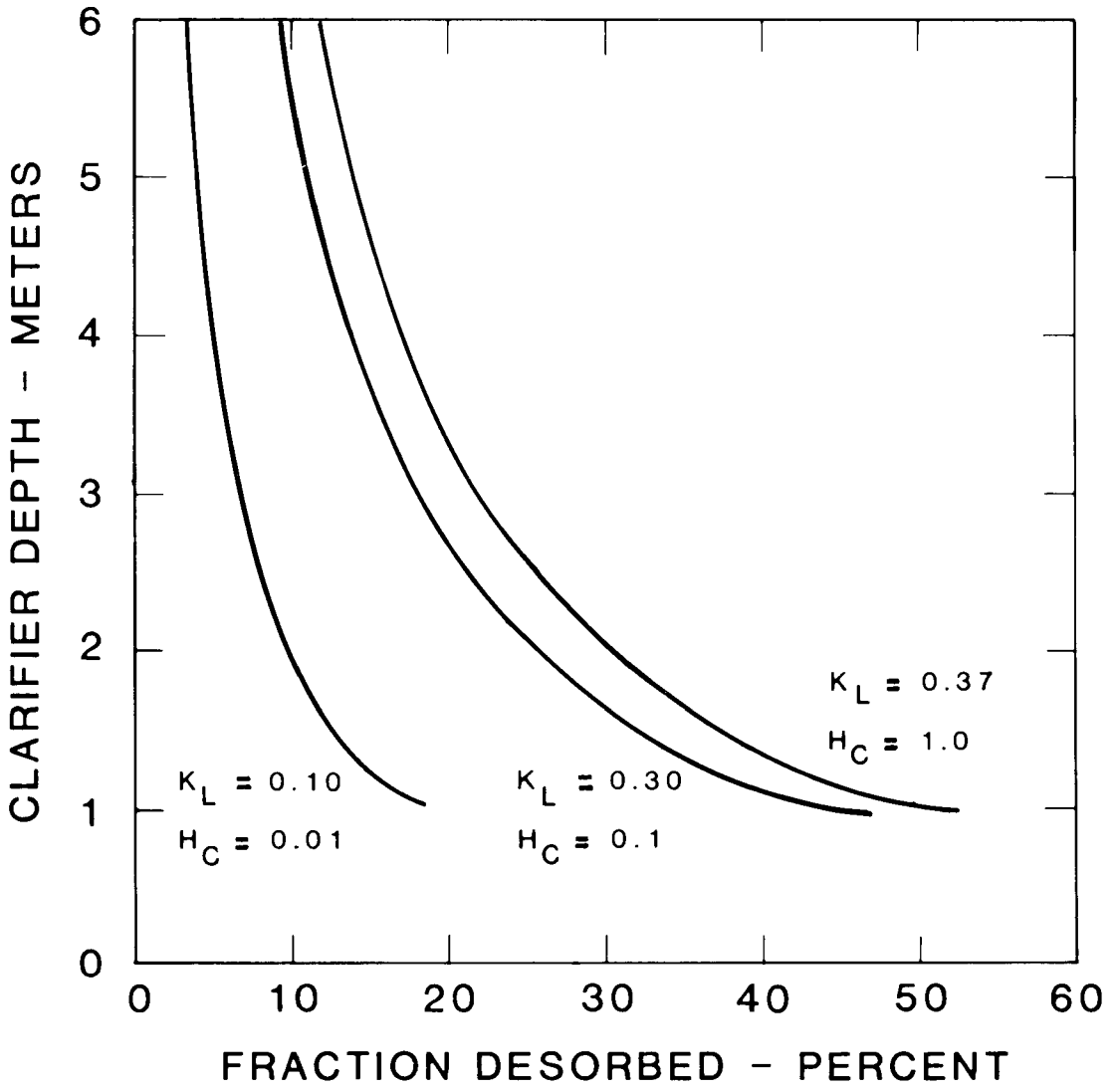

Figure 5. Predicted desorption from primary clarifiers as a function of clarifier depth, H_C, and mass transfer coefficients.

sets of conditions were evaluated; one with high gas to liquid flow ratios and high K_{La}'s, the other with low values for the same variables. The fraction desorbed, η, was then plotted as a function of the different H_c's.

The results are presented in Figure 6, where curves A and B represent the high and low values of Q_G/Q_L and K_{La}, respectively. For the case with high K_{La}'s, 90 percent removal due to stripping is predicted for compounds with H_c greater than 0.03, and for low K_{La} conditions, 90 percent removal is predicted at H_c of 0.07.

The fraction of each compound removed by stripping, observed during this study, is plotted in Figure 6 versus the compound's H_c. The observed removals due to stripping are considerably lower than the removals predicted by the model of Roberts, et al. (18). It is important to note that the slopes of the experimental data and the predicted removals are almost identical. The data indicate that some mechanism is reducing the stripping from the aeration basin.

Sorption onto the biomass is the principal mechanism which will reduce the amount of material stripped from the aeration basin, although other factors, such as matrix effects on solubility, may be important. Matter-Muller, et al. (19) have discussed the role of sorption as it relates to the removal of organic compounds in the activated sludge process. A positive correlation between the adsorption constant, the milligrams of a compound adsorbed per kg of total solids, and the K_{ow} was observed.

Some preliminary investigations were conducted during this study to evaluate the rate at which sorption to the biomass occurs. A 1.5 l sample of mixed liquor was placed in a 2 l beaker, and then spiked with a mixture of several of the compounds being studied. A Teflon® plug, with a 1/4 in. stainless steel sample line, was then placed on top of the sample. The plug produced a zero-headspace, air-tight sample container. The sample was then mixed with a magnetic stirrer.

Samples were withdrawn, by depressing the Teflon® plug, after 5, 10, 15, 30, and 60 minutes. The samples were immediately centrifuged and the liquid phase organic compound concentrations determined by GC/MS using the purge-and-trap method previously described.

The concentration versus time curves indicated that the sorption was first order, and the adsorption coefficients were computed accordingly.

Sufficient data are not yet available to properly quantitate the adsorption coefficients, K_s, for the compounds studied. Additionally, the influence of variables, such as the MLSS concentration and the SRT, have not been evaluated. The observed values of K_s have, however, been large enough to account for the differences between the predicted and the observed stripping removals. Figure 7 is a plot of the K_s's obtained for one study as a function of the K_{ow}'s. The correlation is quite good ($r = 0.95$), but additional data, gathered under varying conditions, are required before any attempt at generalization can be made.

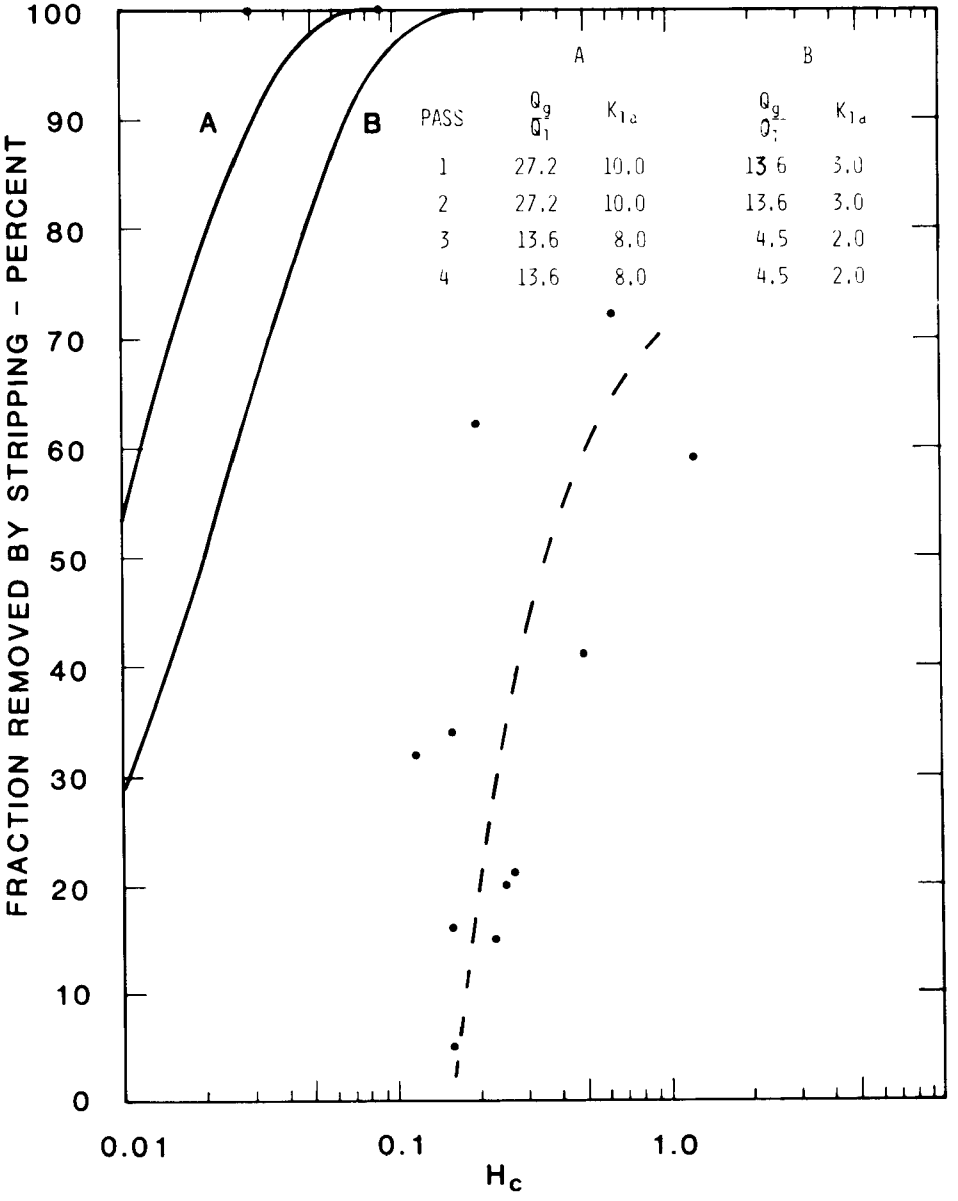

Figure 6. Removals due to stripping in aeration basins as a function of H_c, predicted by "clean water" model (Curves A & B) and data from this study.

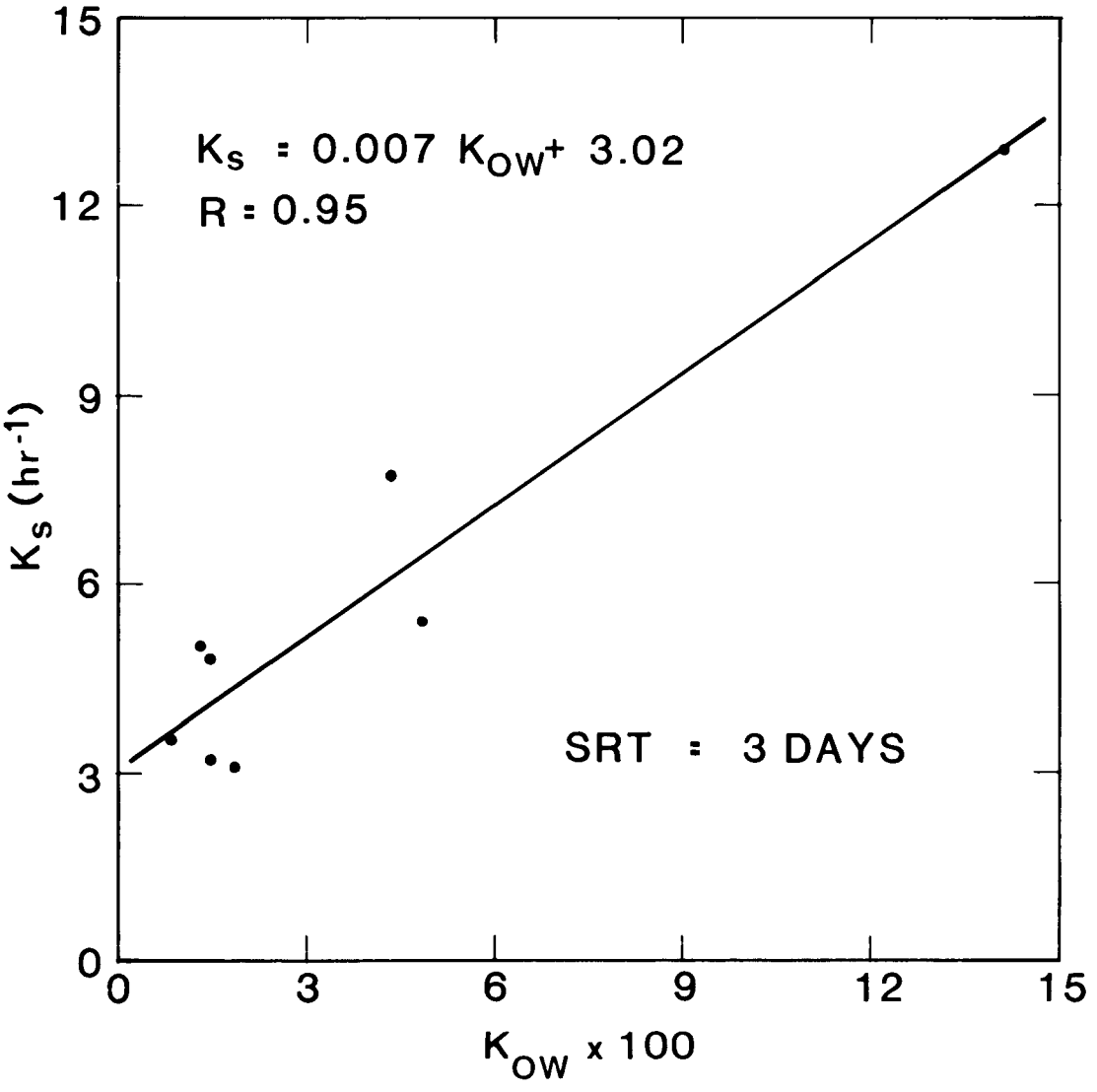

Figure 7. First-order sorption coefficient, K_s, to the mixed liquor as a function of the K_{ow}.

The biodegradability of a compound will obviously influence the extent to which the compound is removed by stripping. One intuitively expects the biorefractory compounds to be more strippable than these materials which are easily degraded. For the volatile compounds, which are structurally simple molecules, the extent of halogenation should influence the relative biodegradability of the compound.

Figure 8 is a plot of the percent halogen by weight of each compound versus the percent of each compound found in the aeration basin off-gas.

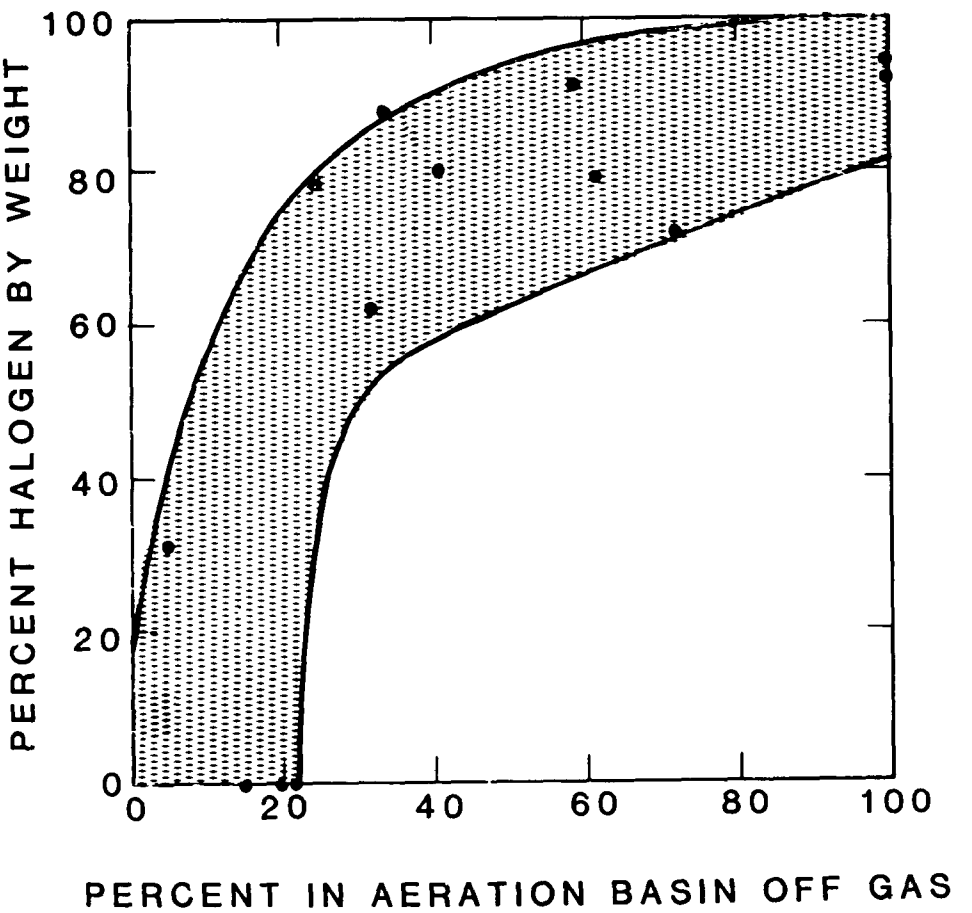

Figure 8. The influence of the degree of halogenation on the amount of a compound removed by stripping in the aeration basin.

CONCLUSIONS

This study was conducted to evaluate the removal of volatile priority pollutants in a conventional POTW and investigate the factors that determine how the compounds partition in the treatment processes. Several important conclusions can be drawn from the results of this investigation:

1. The majority of the volatile priority pollutants studied were removed very effectively by the pilot-scale POTW. Removals greater than 99 percent were observed for 9 of the 16 compounds investigated. Only 1,1,2-trichloroethane and dibromochloromethane had mean removals less than 90 percent.

2. Appreciable removals can be expected to occur in the primary clarifier due to desorption. The removals follow existing two-film models reasonably well, and gas and liquid phase transfer coefficients have been determined.

3. Partitioning to the solids during primary clarification was not a significant removal mechanism. However, the partitioning that did occur was observed to be functionally related to the K_{ow}.

4. For many of the compounds studied, stripping from the aeration basin was the principal removal mechanism.

5. A model which predicts stripping from clean water systems was compared with the stripping observed in this study. The predicted values were higher than the observed values, which suggests that some mechanism was reducing the stripping losses. Sorption to the biomass was found to be sufficiently rapid to account for the observed differences.

6. The adsorption coefficient, K_s, for organics to the biomass was related to the K_{ow}, which is consistent with previously-reported findings.

7. The biodegradability of a compound will infuence the extent to which the compound is removed by stripping. The more biorefractory the compound, the more likely the compound is to be stripped.

8. The percent of the molecular weight contributed by the halogens was shown to be a relative indicator of biodegradability.

9. Those materials which do partition to the biomass are biodegraded, or otherwise altered, since only low concentrations were observed in the waste activated sludge.

RECOMMENDATIONS

1. At the present time, the partitioning of the volatile priority pollutants in primary clarifiers appears to be predictable with reasonable accuracy. Some additional work should be conducted to confirm the existing models.

2. Stripping from aeration basins is not predictable at this time. Additional research is needed to quantitate the sorption to the biomass and the influence of biodegradability on stripping.

3. In order to fully understand the removal mechanisms, research should be conducted to determine whether metabolic intermediates are produced, or whether the compounds are directly mineralized.

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Energy Use at Municipal Wastewater Treatment Plants
Overview and Case Studies

by
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ENERGY USE AT MUNICIPAL WASTEWATER TREATMENT PLANTS-OVERVIEW AND CASE STUDIES

Serious concern for the cost and availability of energy began in the U.S. in the early 70's. Energy utilization continues to be a major consideration in the pursuit of affordable and cost-effective solutions to wastewater and sludge collection, treatment and disposal problems in the U.S. making it necessary to seriously study ways to maximize the utilization of energy in the design and operation of wastewater treatment facilities. It is the purpose of this paper to: first provide a brief review and perspective of the energy problem in the U.S. as it relates to municipal wastewater treatment and second, to provide a series of three case studies of innovative energy saving technologies (1) presently being planned (2), under construction and (3) under operation.

Information from the 1982 EPA Needs Survey (1) which provides some first order approximations regarding number, type, costs, etc. of municipal wastewater treatment facilities, is presented in Table 1. According to the survey, there were a total of 15,425 municipal wastewater treatment plants operating in the U.S. in 1980 designed to treat a flow of 1.34 X $10^8 m^3/d$ (35.3 bgd) (actual treated flow equalled 1.02 X $10^8 m^3/d$ (26.9 bgd)) from a service population of 163,000,000 (71% served). The survey projected that a total of 21,011 municipal wastewater treatment plants would be operational by the year 2000 with design capacity of 1.62 X $10^8 m^3/d$ (42.7 bgd) to handle wastewater from a sevice population estimated to increase to 257,000,000 (92% served). This represents an increase of 36% in number of operating plants, 21% in design flow capacity and 58% in service population. The net increase of 5,586 operational plants reflects about 1,480 plants to be abandoned, 446 plants under construction (as of 1980) and 6,620 plants to be built by the year 2000.

Overall effluent quality is also projected to improve with vastly reduced use of primary and advanced primary treatment - down to a total of 12 plants treating 5.54 X 10^6m3/d (1,464 mgd); and an overall increase of 8,693 in number of secondary, advanced secondary, advanced wastewater tertiary treatment, and no discharge facilities to treat an attendant increase in flow of 4.69 X 10^7 m^3/d (12,400 mgd).

It is clear that energy expended for the treatment of municipal wastewater in the U.S. will necessarily continue to increase for two basic reasons: increase in actual number of facilities and amount of wastewater treated and increase in the overall level of effluent quality.

According to the EPA publication, "Energy Conservation in Municipal Wastewater Treatment," (2) in 1977, about 150.73 X 10^{12}kJ/yr (142.87 X 10^{12} Btu/yr) were used in 1977 for treatment of municipal wastewater in publicly owned treatment works. This amounted to 0.17% of the annual U.S. energy use estimated to be 86 quads (1 quad = 1.055 EJ (1.0 X

TABLE 1

NUMBER, TYPE, AND SIZE OF PUBLICLY OWNED MUNICIPAL WASTEWATER TREATMENT FACILITIES IN THE U.S. IN 1980 AND PROJECTED FOR THE YEAR 2000(1)

	CATEGORY OF TREATMENT						
	No Discharge	Primary	Advanced Primary	Secondary	Advanced Secondary	AWT Tertiary	Total
1980 Facilities							
Count	1600	1036	2083	7946	2529	231	15,425
% of total	10.4	6.7	13.5	51.5	16.4	1.5	100
Act. '80 flow*	491	2474	2825	11,008	9376	714	26,891
% of total	1.8	9.2	10.5	40.9	34.9	2.7	100
Design flow*	759	3047	3391	14,291	12,288	1215	35,265
% of total	2.2	8.6	9.6	40.6	35.6	3.4	100
Year 2000 Facilities							
Count	2736	3	9	11,748	5845	670	21,011
% of total	13.0	<0.1	<0.1	55.9	27.8	3.2	100
Design flow*	1540	220	1244	17,474	19,288	2922	42,691
% of total	3.6	0.5	2.9	40.9	45.3	6.8	100
Change							
Count	+1136	-1033	-2074	+3802	+3316	+439	+5586
% increase	+71.0	-99.7	-99.6	+47.8	+131.1	+190.0	+36.2
Design flow*	+781	-2827	+2147	+3183	+6729	+1707	+7426
% increase	+102.9	-92.8	-63.3	+22.3	+53.6	+140.5	+21.1

* All Flows in mgd, 1 mgd = 3785 m^3/d

10^{15} Btu)) at the time. For 1990, the report projected that 271.02 X 10^{12} kJ/yr (256.91 X 10^{12} Btu/yr) would be required for wastewater treatment or 0.23% of the national energy use projected to be 114 quads. A more recent EPA report, "Energy Requirements of Present Pollution Control Technology" (3) indicates operating energy required for municipal wastewater treatment plants was 284.85 X 10^{15} kJ (270 X 10^{12} Btu) in 1977 with an estimated error bound of 95-570 X 10^{15} kJ (90-540 X 10^{12} Btu). This amounts to 16% of the energy required for pollution control among all stationary sources. Other stationary sources include: industry, 58%; power plants, 20%; municipal solid waste disposal, 6%. This estimated usage includes net primary electrical and fuel energy as well as secondary energy for production of chemicals, which should range from 9.8 - 13% of this total as pointed out in the EPA report, "Total Energy Consumption for Municipal Wastewater Treatment" (4). This leaves about 248-257 X 10^{12} kJ (235-244 X 10^{12} Btu) in net primary energy requirements for the operation of municipal wastewater treatment plants.

The 1977 energy estimate of 248 X 10^{12} kJ (235 X 10^{12} Btu) is extrapolated using 1982 Needs Survey information to arrive at the energy estimate of 258 X 10^{12} kJ (245 X 10^{12} Btu) for the year 1980 as presented in Table 2. Similarly, information from the 1982 Needs Survey, reference 2 and from Owen (5) is extrapolated to determine the projected energy required in the year 2000 of 460 X 10^{12} kJ (436 X 10^{12} Btu/yr) reflecting a 78% increase in energy requirements over 20 years. This is 371%, 134% and 217% of the increase in projected flow, population and number of wastewater treatment facilities, repsectively. Based on an assumed value of 140 quads national energy use in the year 2000, 0.31% of the U.S. energy use would be for municipal wastewater treatment representing an 11% increase in total energy budget share.

According to some energy forecasters (6), projections of year 2000 U.S. energy usage from 90 to 100 quads are considered to be realistic. Projected energy use for wastewater treatment could therefore be as high as 0.48% of the national figure-particularly if conservation in treatment does not continue.

Even though the energy required to treat wastewater is a small portion of the total energy use of the U.S., this is not the case in terms of the annual operating budget devoted to wastewater treatment. The cost of energy continues to be an increasing proportion of total operation and maintenance costs. As an example, Figure 1 presents power and labor cost index information extracted from the "Innovative and Alternative Technology Assessment Manual" (7) as updated by the most recent EPA published operation and maintenance cost index information (8). The power and labor indices both increased at a similar rate until 1973 when they were 1.326 and 1.48, respectively. In 1974, the power index increased to 1.77 while the labor index increased in a more moderate fashion to 1.60. As of the 3rd calendar quarter of 1982, the power index was 4.76, increasing 1.8 times as much as the labor index in a time period of nine years. This has occurred in spite of strong conservation measures which have taken place. According to the

TABLE 2. NET PRIMARY ENERGY REQUIREMENTS FOR THE OPERATION OF PUBLICLY OWNED MUNICIPAL WASTEWATER TREATMENT FACILITIES IN THE U.S. IN 1980 AND PROJECTED FOR THE YEAR 2000

	1980	2000	% Increase
No. of Wastewater Treatment Plants	15,425	21,011	36
Design Flow (bgd)	35.3	42.7	21
Population Served	163,000,000	257,000,000	58
Energy Required (10^{12} Btu)	245	436	78
National Energy Budget (quads)	87	140	61
% Municipal Wastewater Energy of National Energy Budget	0.28	0.31	11

1bgd = 3.785 X 10^6 m^3/d
1Btu = 1.055 kJ

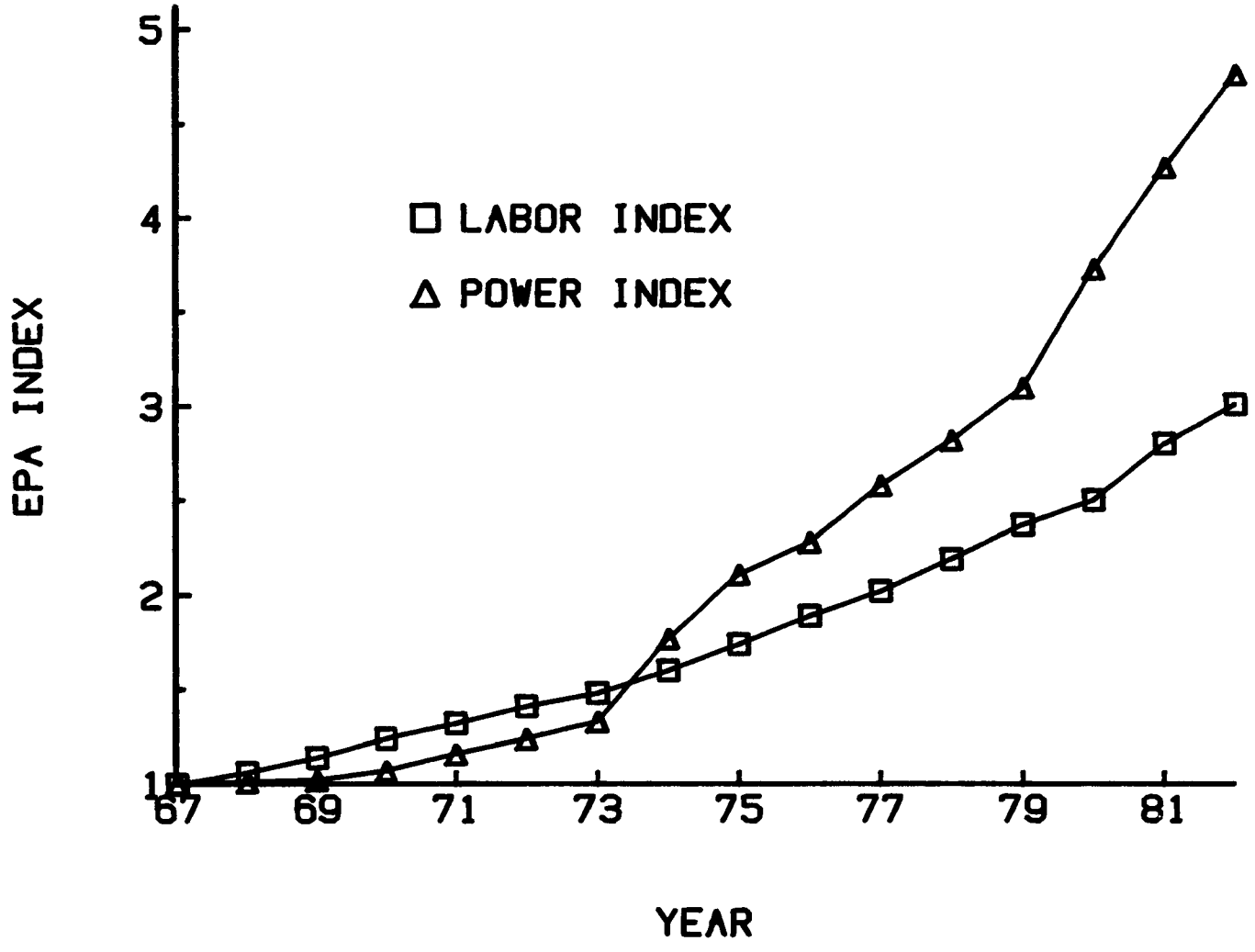


FIGURE 1. RELATIVE LABOR AND POWER COST INCREASES (7)

same operation and maintenance cost index information, in 1967, the cost of power comprised 19.5% (8) of total operation and maintenance costs in a wastewater treatment plant and has increased to 26 - 27% as of 1982.

Recent U.S. Department of Energy projections indicate that the costs of power and fuel will continue to rise. For example, in the commercial sectors, the cost of electricity is projected to increase to $\$20.00/10^6$kJ ($\$21.10/10^6$ Btu) by 1990, a 23% rise compared to 1981 prices. In a similar fashion, the cost of natural gas is projected to increase to $\$5.70/10^6$kJ ($\$6.01/10^6$ Btu), a 51% increase.

Information from the 1980 EPA Needs Survey (10) indicates that about $3.3 billion (1980 dollars) were expended for operation and maintenance of 15,250 publicly owned municipal wastewater treatment plants estimated to be treating an actual flow of $0.97 \times 10^3 m^3/d$ (25.7 bgd) (This is equivalent to $9¢/m^3$ treated (35¢/1000 gal)). The annual energy requirement for operation and maintenance of these facilities is estimated to be about 248×10^{12}kJ (235×10^{12} Btu) or about 1.45×10^{10} kWh for power (65% of the total) and 8.68×10^{13}kJ (8.23×10^{13} Btu) for fuel (35% of the total) (4). Applying adjusted energy cost information to these figures, energy costs are estimated to be about $0.69 B (1978 dollars), which is 26% of the total. It is fair to conclude that, on the average, energy costs comprise from 20%-30% of the total budget for operation and maintenance.

As a case in point, in 1982, the City of Cincinnati, Ohio (11) expended $6,535,000 for energy, which amounted to 20.4% of a total budget of $31,992,000 to operate and maintain wastewater treatment facilities. About 39% of the Cincinnati energy budget is directed to wastewater collection and treatment. In other specific cases, energy costs can be much higher.

Energy usage and cost estimates for operation and maintenance of municipal wastewater treatment facilities in the U.S. are useful for purposes of problem definition; but from the standpoint of targeting conservation, the energy requirements of individual unit processes and components is of fundamental importance. An estimate of energy used for plant operation is shown in Table 3 (4) for activated sludge treatment of municipal wastewater. Energy used to operate the treatment plant includes: electrical energy and heat energy for digester heating, building heat, sludge incineration and hauling of sludge to land disposal. Electrical energy represents 63% - 76% of the energy used for plant operation. Building heat requirements are estimated at only 5%-10% of the operating energy. Approximate estimates for the electrical energy requirements for various unit processes is shown in Table 4(4). At the 3785 m^3/d (1 mgd) size, a trickling filter requires 80% more energy than a primary plant; and the activated sludge plant requires roughly three times as much energy as the primary plant. An economy of scale is apparent whereby $3.8 \times 10^5 m^3/d$ (100 mgd) primary, trickling filter and activated sludge plants respectively utilize about 50%, 57%, 76% of the unit electrical energy consumption as with a 3785 m^3/d (1 mgd) facility(4).

TABLE 3. ESTIMATED OPERATING ENERGY BUDGET FOR MUNICIPAL WASTEWATER TREATMENT PLANTS (kWh/mg) (4)

	Size of Plant		
	1 mgd	10 mgd	100 mgd
Electrical Energy	1100	893	835
Digester Heating	168	168	168
Building Heat	160	59	86
Sludge Hauling	20	--	--
Sludge Incineration	--	230	230
Total	1448	1350	1319

In terms of kilowatt-hours per million gallons of wastewater treated. Estimates are based on activated sludge plants with anaerobic digestion. Sludge disposal is by incineration at the $3.8X10^4$ m^3/d (10 mgd) and $3.8X10^5$ m^3/d (100 mgd) sizes by hauling dewatered sludge 64.4km (40 miles) one-way to land spreading at the 3785 m^3/d (1 mgd) size. Energy recovery is by generation of electrical power with IC engine using digester gas as fuel. The waste heat from the IC engines is used to heat the digesters. Energy wheels are utilized to conserve building heat. 1mg = $3785m^3$

TABLE 4. ESTIMATED ELECTRICAL ENERGY CONSUMPTION FOR OPERATION OF MUNICIPAL WASTEWATER TREATMENT PROCESSES (kWh/mg)*(4)

	Size of Plant		
	1 mgd	10 mgd	100 mgd
Preliminary Treatment	18.5	6.6	2.5
Influent Pumping (30 ft TDH)**	153.0	145.1	129.3
Primary Sedimentation	30.6	12.2	7.3
Recirculation Pumping			
-Trickling Filters (Q_r/Q = 3.0)	183.0	174.0	155.2
-Activated Sludge (Q_r/Q = 0.5)	45.0	42.3	31.3
Diffused Air Aeration (AEF = 6%)	532.0	532.0	532.0
Mechanical Aeration (2 lb O_2/hp-hr)	404.0	404.0	404.0
Final Sedimentation	30.6	12.2	7.3
Chlorination	0.7	0.7	2.7
Sludge Pumping	2.7	2.7	2.7
Gravity Thickening	10.2	2.0	0.4
Air Flotation Thickening	70.0	60.8	46.9
Anaerobic Digestion	123.6	45.6	19.1
Vacuum Filtration	58.5	34.6	36.4
Incineration	65.0	28.7	25.9
Lights and Misc. Power	57.0	21.0	24.0

*1 mg=3785m^3

**TDH = total dynamic pumping head, Q_r/Q = return flow rate/average daily flow rate
AEF = aeration efficiency in percent for diffused air and lb O_2/hp-hr for mechanical aeration

In activated sludge plants, about 57% of the electrical energy is used for aeration, about 20% for pumping influent and return flows and 6% for mixing and heating the anaerobic digester.

The radically changing relationships in cost and availability of energy resources over the past decade have provided the driving force to seek energy saving solutions to wastewater problems at the national level. The Clean Water Act of 1977 (PL95-217), as amended by the Municipal Wastewater Treatment Construction Grant Amendments of 1981 (PL97-117), contains a number of provisions to encourage the development and implementation of innovative and alternative (I/A) wastewater technology that provide for greater use of energy recovery and conservation (7, 13).

In development of the Clean Water Act, the Congress recognized that energy self sufficiency is possible through use of alternate energy sources in publicly owned treatment works constructed with Federal support. In fact, a 115% cost-effectiveness preference was established which encourages construction of I/A technology systems in order to reduce the demand for non-renewable energy resources over the long term.

The EPA's I/A Technology Program widens the range of available solutions to address wastewater and sludge problems. Since the inception of the I/A Technology Program in October 1978, over 2,000 supporting grants have been awarded by the EPA to municipalities which have adopted I/A technology solutions (13, 14). Over 25% of the innovative technology projects funded resulted in direct savings of net primary energy and over 10% of the alternative technology projects funded were for energy recovery or conservation. These have included: co-disposal of sludge and refuse; anaerobic sludge digestion with productive use of recovered methane; self sustaining and starved air sludge combustion; use of alternate energy such as active and passive solar energy including photovoltaic cells, windmills, low head hydroelectric, and heat pumps; use of efficient aeration devices such as draft tube aeration; use of energy efficient unit processes such as inchannel clarification in an oxidation ditch.

Energy is also conserved indirectly by utilizing energy efficient I/A treatment technology such as land application and subsurface disposal systems. It has been previously estimated (13) that 17.3×10^8 kWh/yr in energy requirements could be saved through adoption of I/A technology in new construction through the EPA Construction Grants Program. This is 7%- 8% of the estimated wastewater treatment energy budget for 1980.

In addition to these savings, energy will be conserved through future retrofit modifications to existing treatment facilities, improved energy management in ongoing operations and a projected increase in the use of trickling filters, oxidation lagoons, etc. as secondary treatment technology in certain cases.

The remainder of this paper presents three examples, or case histories, of alternative energy technologies at municipal wastewater treatment plants:

1. Use of low-head hydroelectric generation at Bonney Lake, Washington

2. Use of active/passive solar, heat pumps at Wilton, Maine

3. Use of windmill derived power at Southtown, N.Y.

BONNEY LAKE, WASHINGTON (15)

BACKGROUND

Wastewater is generated from residences and businesses around Lake Tapps near the City of Bonney Lake, Washington. Lake Tapps is a storage reservoir for the White River Hydroelectric Plant. According to initial planning efforts, wastewater collected from the area is to be lifted over a ridge and dropped approximately 122 m (400 ft) vertical distance over approximately 1,097 m (3,600 ft) horizontal distance to the Puyallup River flood plain after which it will flow by gravity to an existing sewage treatment plant at Sumner, Washington.

Initially proposed was use of a gravity sewer interceptor to transmit flow to another treatment plant site on the floor of the Puyallup Valley at Alderton, Washington. This was not considered to be environmentally acceptable by the residents of the region, who preferred routing the wastewater to the City of Sumner Wastewater Treatment Plant. The Sumner option has one disadvantage: it is energy-intensive and requires a large lift station to pump wastewater over a hill and into the valley. The energy necessary for pumping provided an incentive to find a way of reducing energy costs.

SYSTEM DESCRIPTION

The wastewater collection area lies near a glacially-formed escarpment rising from the Puyallup River flood plain. The interceptor crosses the escarpment en route to the Sumner Regional Wastewater Treatment Plant. Various methods were considered for dissipating energy across the escarpment. Deep-drop manholes presented difficult construction and maintenance problems. A pressure sewer with energy dissipation features presented erosion, cavitation, and foaming problems. Reclaiming energy with a turbine generator was the most cost-effective solution with energy revenues offsetting the lift station energy costs. This system would provide a shelter against ever-increasing lift station energy costs. Under the EPA innovative and alternative technology program, wastewater transmission and treatment systems that effect significant reduction in net power consumption are encouraged. The Lake Tapps energy recovery station fulfilled these criteria and was approved by EPA in December 1980.

Planning and preliminary concept design for the Bonney Lake project has been conducted by the firm Philip M. Botch, Inc., Bellevue, Washington. Design of this innovative project was underway; however, due to recent problems of implementing a regional alternative, the project may not reach construction.

COMPONENT DESCRIPTION AND DESIGN CRITERIA

The proposed innovative alternative includes the use of 1,095 m (3,592 ft) of 45.7-cm (18-in) ductile iron (DI) penstock (hydrostatically-pressurized force main) and a hydroelectric generating station that would tie directly into the utility power grid. The generation station would include a two-jet impulse turbine and induction generator. A schematic of the project is shown in Figure 2. Characteristics and performance information follows:

Penstock diameter	45.7 cm
Penstock length	1,095 m
Verticle drop	112.8 m
Turbine/generator rating	125 kW
Average flow -- 1982	2,575 m^3/d
Average flow -- 2002	8,325 m^3/d
Yearly power production -- 1982	217,800 kWh
Yearly power production -- 2002	697,600 kWh
Value of power in 1982 at $0.04/kWh	$ 8,712
Value of power in 2002 with a 9 1/3 percent annual compounded escalation rate	$166,132
Turbine type	Pelton-impulse
Number of nozzles	One
Governor	None
Generator type	Induction
Method of operation	Automatic, unattended
Type of plant	"Run of river"

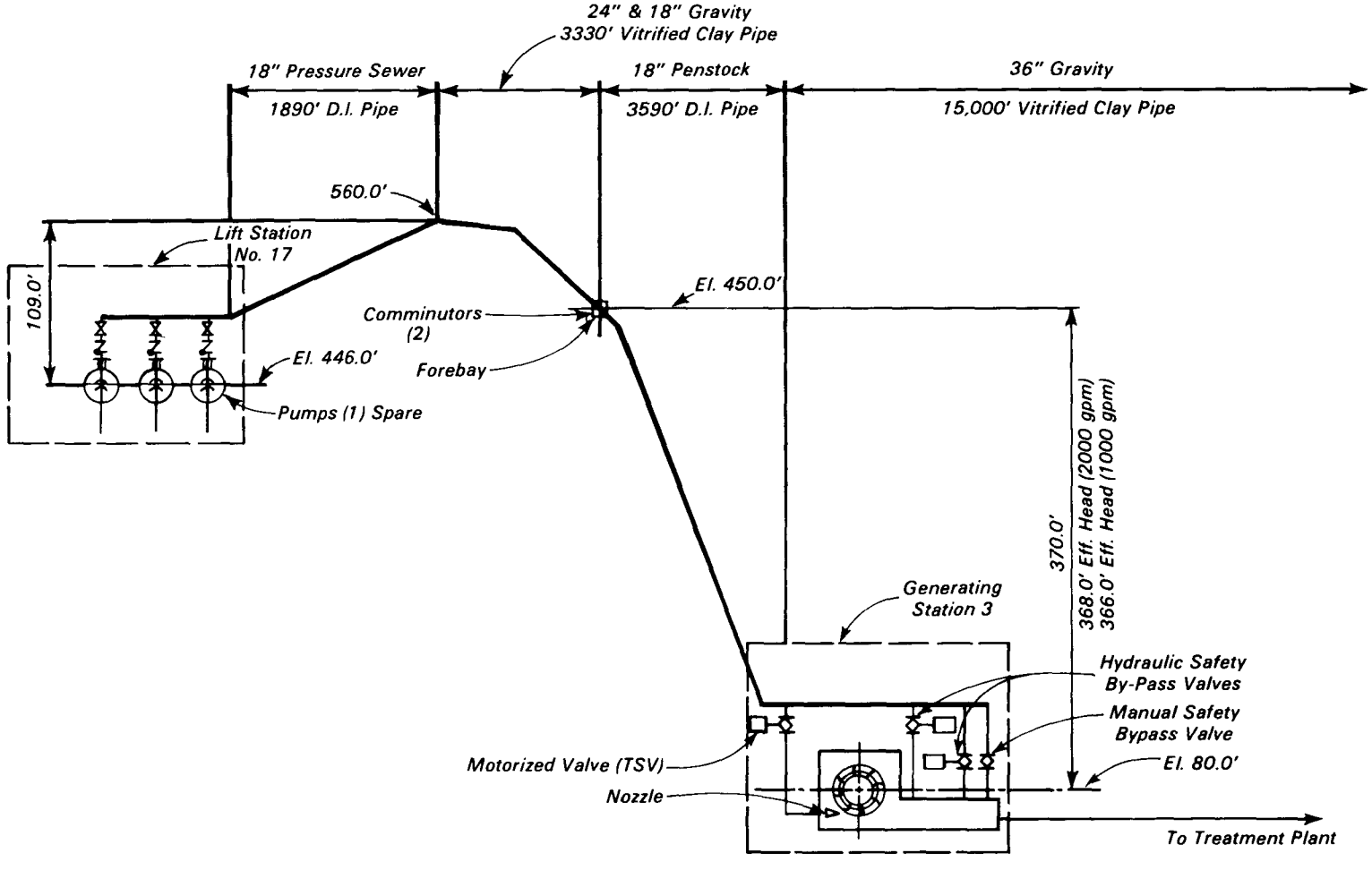


Figure 2. *Schematic-energy recovery system.*
Source: P.M. Botch and Associates, Inc.

A preliminary treatment station consisting of a comminutor, grit collector, and screen, would be located in the interceptor ahead of the penstock. The turbine would have an automatic jet deflector to prevent overspeed, and a surge-relief valve would protect against inadvertent flow surges. A blowout rupture disc is provided in the event the surge relief valve malfunction. If the turbine is down for repair, bypass throttling valves will automatically maintain the proper water level in the forebay.

A special variable opening nozzle would be designed to substitute for the nozzle commonly used in a Pelton turbine. The nozzle commonly used in a Pelton turbine employs a controlled needle opening. The small annular space around the needle at low loads and the preceeding straightening vanes presented potential clogging problems. The nozzles would be fixed in an open position and operated without needles. If the project is implemented, the special nozzle will be tested in a hydraulic laboratory before incorporating it into the turbine design.

ENERGY SYSTEM PERFORMANCE DATA

The energy analysis indicates that the five lift stations will require 137,984 kWh/yr at startup (2,575 m^3/d or 0.68 mgd)and 461,925 kWhr/yr in the year 2000 (8,325 m^3/d or 2.19 mgd). The generating station will produce 218,665 kWh/yr and 704,230 kWh/yr at these same flows. Therefore, the net energy production of the proposed innovative alternative is 80,681 kWh/yr at start up and year 2000, respectively.

SOUTHTOWN SEWAGE TREATMENT CENTER (WOODLAWN, NEW YORK) (16)

BACKGROUND

The new wastewater treatment facilities at the Southtown Sewage Treatment Center in Woodlawn, New York, will be powered by a wind turbine generator. The Southtown facility is located on the shore of Lake Erie, near Buffalo, New York, which is in an area of strong persistent winds.

Initial construction of the treatment facility began at Southtown in the fall of 1977 with support from the EPA Construction Grants Program, and all construction, including the innovative wind turbine generator system, is expected to be finished in Spring 1984. Design flow for the Southtown Sewage Treatment Center is 60,560 m^3/d (16 mgd). Major unit processes include pure oxygen activated sludge, chemical addition for phosphorus removal, sand filtration, and chlorination. Sludge will be incinerated. The effluent will be discharged to Lake Erie.

WTG Energy Systems, Inc., Buffalo, New York, developed and designed the wind power system. URS Engineers, Montvale, N.J., is the designer of the treatment plant.

The use of wind-generated electricity at the Southtown Sewage Treatment Plant will place Erie County in a unique position as one of the first commercial installations utilizing a wind generator at a municipal wastewater treatment facility in the U.S. The only other known application of windmills is at Livingston, Montana where four 25 kW rated Carter Wind System (Burkburnett, Texas) units have been operating since December 1981 to provide the wastewater treatment plant with power.

When the Southtown Sewage Treatment Center reaches design operation, it is estimated that the power utilization will be about 12,570,000 kWh/yr. The use of one 600 kW wind turbine generator will provide from 980,000 to 1,215,000 kWh/yr depending upon wind regime. Based on a cost-effectiveness analysis, net annual power cost savings from the wind turbine generator system are estimated to be a minimum of $27,000 (October 1982 $).

SYSTEM DESCRIPTION

One 600-kW wind turbine generators (model MP1-600, supplied by WTG Energy Systems, Inc.) will provide the power. A typical wind turbine generator is shown in Figure 3.

The model MP1-600 consists of: a 38.1 m (125 ft) diameter, three blade upwind rotor driving a 600 kW, 2400-V, 60 cycle AC generator through a 1 to 48 ratio speed-increaser; and a rotor, gear drive assembly, generator and hydraulic system mounted on a rotating base and enclosed within a machine cabin. This assembly is mounted on top of a pinned truss steel tower, and is yawed by a hydraulically-controlled bull-gear unit that provides 360° positioning to ensure maximum upwind efficiency of the rotor.

The rotor blades are a fixed pitch GA(w)-1 airfoil design incorporating blade-tip drag flaps that are automatically activated to stop the rotor under conditions of excessive wind speed, vibration, or any system malfunction such as icing. This "fail-safe" system permits unattended operation of the wind turbine, and prevents any aggravated system failures.

Electrical generation begins at wind velocity of 3.0 m/s (10 mph) at hub height. The rated generator output of 600 kW is achieved at 13.4 m/s (30 mph), and the maximum generator output of 750 kW is reached at a wind velocity of 15.2 m/s (35 mph). Shutdown occurs at 24.6 m/s (55 mph). The survival wind velocity is 67.1 m/s (150 mph). Throughout the operating range, the rotor of the MP1-600 system will maintain a constant 25 rpm for the production of constant frequency 60-Hz power. The wind regime at Southtown is expected to be 5.5 m/s (12.4 mph) providing an output of 980,000 kWh/yr for the wind turbine system.

The control unit of the MP1-600 system is a solid-state microprocessor located in the control house at the base of the tower. This

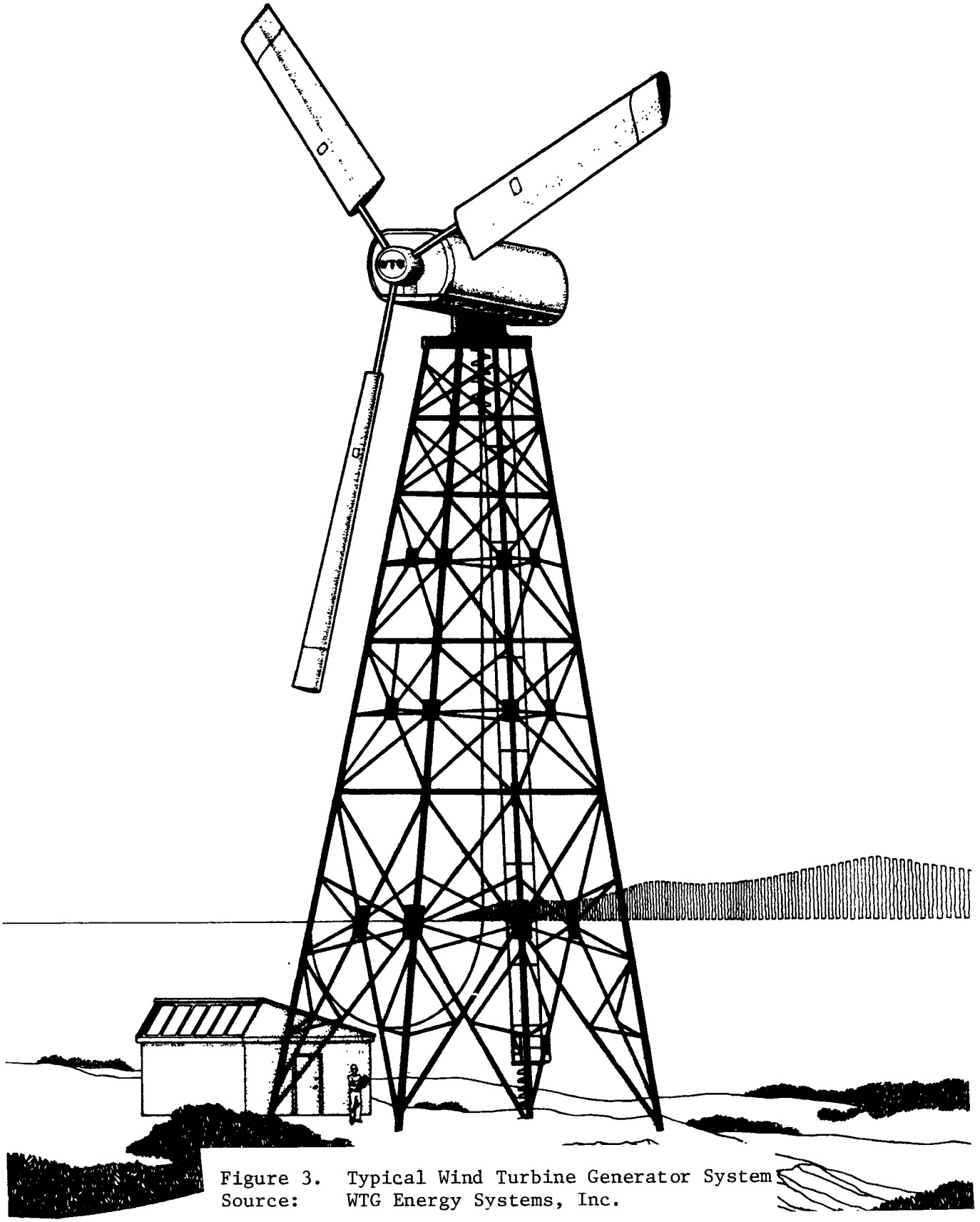


Figure 3. Typical Wind Turbine Generator System
Source: WTG Energy Systems, Inc.

preprogrammed computer continually monitors and controls all generating and operational parameters including wind velocity and range; and the maximum possible power output is introduced into the utility grid system at precision-controlled voltage and frequency. Even if the MP1-600 system is the only generator on the transmission line system, precise voltage and frequency will still be maintained.

COSTS AND REQUIREMENTS FOR OPERATION AND MAINTENANCE

The capital and installation costs (October 1982 $) for the wind turbine generator system, consisting of one 600-kW unit, are as follows:

One 600 kW wind turbine generator	$1,100,000
Foundation/Erection	$ 186,000
480-V distribution (305 m)	$ 58,000
Testing	$ 3,350
Total installed cost	$1,347,350

The estimated manpower requirements are based on operating experience from other WTG, Inc. installations. The system is designed for unattended operation; however, the system will require an annual allocation of approximately 78 man-hours for scheduled maintenance. The estimated annual maintenance requirement is based on data obtained from a wind analysis program conducted for the Southtown facility indicating annual machine availability at 90 percent (minimum) and annual machine operating time at 5,804 hr (66 percent of the year).

The estimated operation and maintenance requirements and costs are as follows:

Lubricants and spare parts	$4,500
Labor	$2,340
Total annual operating costs	$6,840

WILTON, MAINE USE OF INTEGRATED ALTERNATE ENERGY SYSTEMS (17)

BACKGROUND

The Wilton, Maine wastewater treatment plant was designed by Wright-Pierce Architechts and Engineers, Topsham, Maine. The 1,700 m^3/d (0.45 mgd) facility began operation in September 1978 and was one of the first municipal wastewater treatment plants in the U.S. to use alternative energy technology. A research project supported by the Municipal Environmental Research Laboratory was conducted from May 1977 to March 1981

to monitor and evaluate the performance of the energy systems installed at the Wilton treatment plant. As of March 1980, actual flow treated at the plant was 450 m^3/d (0.12 mgd) or 26% of design.

A primary objective of the design of the Wilton facility was to reduce net primary energy consumption by use of (1) alternate energy sources, (2) less energy intensive process equipment, (3) an overall architechtural and engineering energy saving design.

SYSTEM DESCRIPTION

The Wilton Wastewater Treatment Plant is located at 45° north latitude. Monthly average ambient temperatures range from a low of -4.4°C in January to a high of 21°C in July.

The Wilton plant was totally enclosed in two structures, and unit processes were built in close proximity to each other to reduce exposed surface area and to reduce length of hydraulic runs. Flow by gravity was used wherever possible. The control building was well insulated, and zone heating is practiced. Concrete block and brick with interspace insulation were used for construction materials to provide a large heat retaining mass.

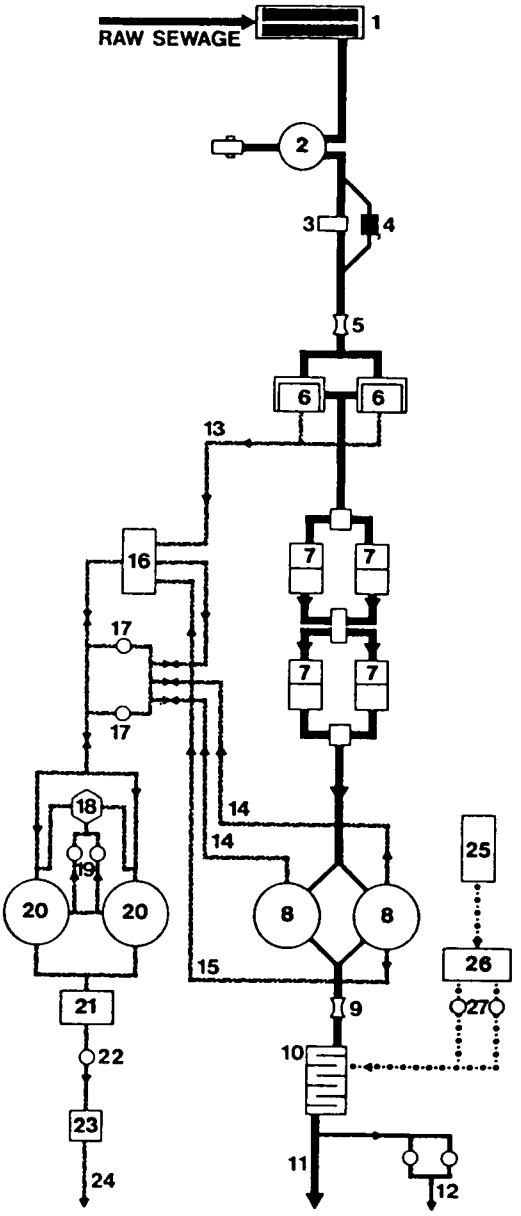
The roof design provides for retention of snow as a natural insulator. The building was built into a hillside with little northern exposure to minimize outside surfaces.

In order to reduce the need to import energy, active and passive solar energy producing systems were designed. A southern exposure was used to maximize production. The primary purpose of the active solar system is anaerobic digester heating but is also used for building heat and the hot water supply. Passive solar is used to provide heating of the clarifier room.

The heating system also included: a water-to-water heat pump to extract energy from the 7° - 10°C effluent; a digester gas fired boiler; and air-to-air heat exchanger to recover 60% of the heat from exhaust ventilation system air to preheat incoming cold air; and electrical generator cooling jacket heat recovery. The anaerobic digester system consisted of primary and secondary digesters with attendant high and low pressure gas recovery, storage and utilization. The design approach intended to provide active solar digester heating thus freeing digester gas to be stored in the form of a compressible, combustible high grade energy source for use in a boiler for building heating, running the electric generator and long term storage.

Figure 4 shows the plant process flow diagram and design criteria. Wastewater is lifted into the plant by use of screw pumps which use less energy than conventional pumps. Wastewater flow is by gravity throughout the remainder of the plant. Pretreatment is accomplished by comminution and grit removal. Rotary screens are used to remove solids and secondary treatment is provided by rotating biological contactors

1 SCREW PUMPS
2 GRIT CHAMBER
3 COMMINUTOR
4 BAR SCREEN BY-PASS
5 FLOW MEASURING & SAMPLING
6 PRIMARY SCREENS
7 ROTATING BIO-CONTACTORS
8 SECONDARY CLARIFIERS
9 FLOW MEASURING & SAMPLING
10 CHLORINE CONTACT CHAMBER
11 PLANT EFFLUENT
12 SPRAY IRRIGATION
13 PRIMARY SLUDGE
14 SECONDARY SLUDGE
15 SECONDARY SKIMMINGS
16 SLUDGE HOLDING
17 SLUDGE TRANSFER
18 SLUDGE HEATER
19 SLUDGE RECYCLE PUMPS
20 ANAEROBIC SLUDGE DIGESTERS
21 SLUDGE HOLDING
22 SLUDGE DEWATERING PUMP
23 DEWATERING UNIT
24 DEWATERED SLUDGE
25 HYPOCHLORITE GENERATION
26 HYPOCHLORITE STORAGE
27 METERING PUMPS



DESIGN CRITERIA

Quantity of sewage
0.02 m^3/s
Influent BOD
200 mg/l (340 kg/d)
Influent suspended solids
200 mg/l (340 kg/d)
Effluent BOD
20 mg/l (34 kg/d): 90% removal
Effluent suspended solids
20 mg/l (34 kg/d): 90% removal
Sludge quantity to digesters
9.5 m^3/d at 3.5% solids
Methane yield
110 to 125 m^3/d
Methane heat value
22,400 kJ/m^3 or 2.4 to 2.7 GJ/d

Figure 4. Wilton, Maine Process Flow Diagram (17)

which use less energy as compared to activated sludge processes. Secondary clarifiers provide final solids removal. Combined primary screened and secondary settled solids are anaerobically digested prior to dewatering and land application.

A conceptual energy flow diagram is shown in Figure 5 which illustrates the unique interdependency of energy sources and users at the Wilton plant. In the design, use of solar energy is maximized as the primary energy source; digester gas is the intended secondary energy source; and recovered effluent heat is intended as a back-up and final supplementary source. The generator was to provide stand-by heating in the event of a power failure and to provide power for general use when excess methane became available.

The active solar system is a hydronic type comprised of flat plate collector panels mounted as part of the building roof, an ethylene glycol/water collection loop, a heat exchanger and storage system. The collector panels cover 139.4 m^2 (1500 ft^2) gross area and consist of an extruded aluminum plate and frame, copper tubing to transport the collector fluid and two panes of low iron content tempered glass. The backs of the collectors are insulated with 114 mm thick rigid foam board insulation. The antifreeze solution is pumped through the panels and heated to 48.9°C to 60°C (120°F - 140°F). Energy from the heated ethylene glycol is exchanged with the plant's circulating hot water system. The array of 54 double glazed panels face 2° west of south at an angle of 60° to the horizontal. The resultant effective collection area is 119.5 m^2 (1,286 ft^2).

The passive solar system is built into the south wall at two levels in order to provide heating and lighting to the clarifier room. The array consists of 83.2 m^2 (812 ft^2) of glazed insulated, translucent fiberglass panels. These panels are also oriented 2° west of south at 60° to the horizontal. The effective collection area is 75.4 m^2 (812 ft^2). The transmissivity of the panels is 66%. Panel construction consists of four layers of fiberglass sheeting supported with aluminum bracing.

The water to water heat pump is used for hot water heating when digester gas is not available and solar production is inadequate. Due to problems with anaerobic digestion including start up difficulties and operation at under loaded conditions, and due to the fact that solar energy proved to be inadequate, the heat pump was the major source of heating during winter. The heat pump recovers heat from treatment plant effluent prior to discharge. Specification information for the heat pump is shown in Figure 6.

ENERGY AUDIT FINDINGS

Projected energy requirements for building heat, digester heating and electrical equipment serve as the basis for evaluating the actual observed performance of the respective systems. These are summarized in Table 5. Based on an equivalent energy comparative analysis, 86% of

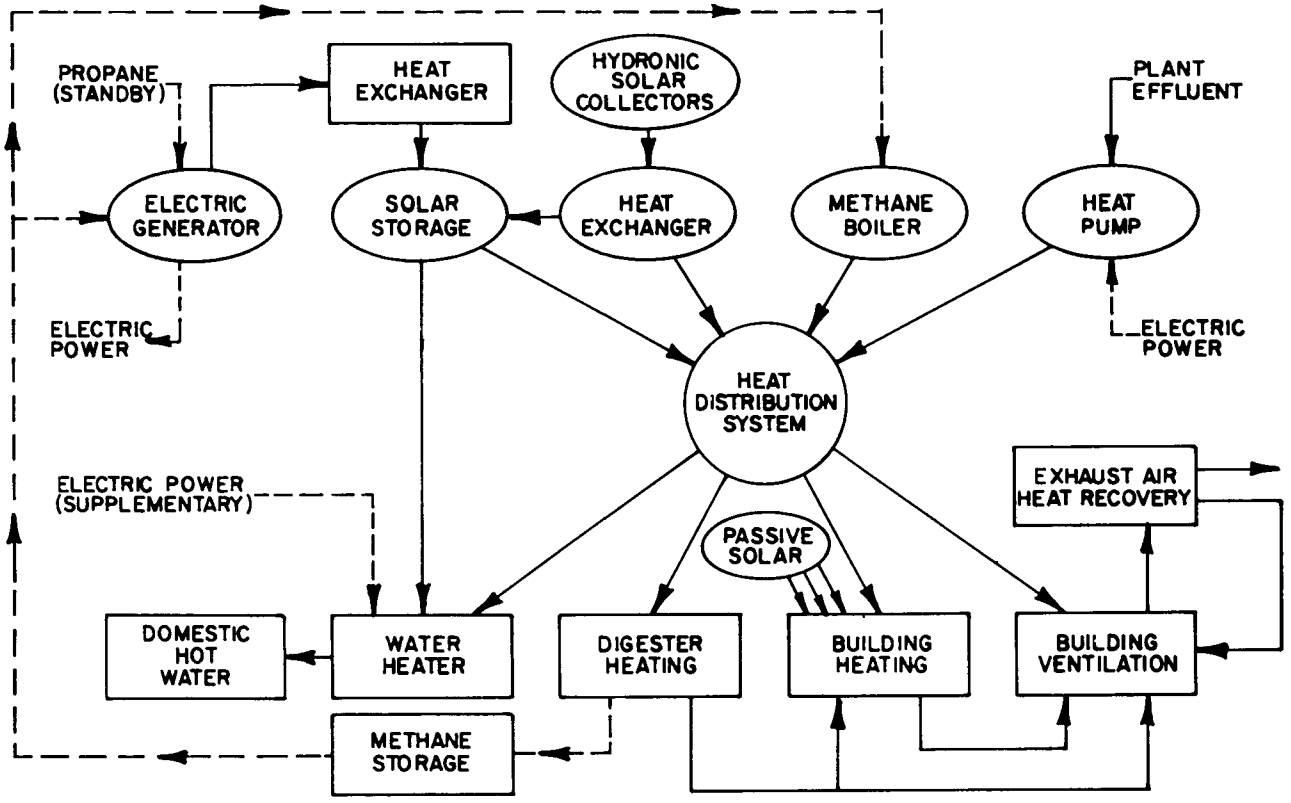


Figure 5. Conceptual Energy Flow Diagram for Wilton, Maine (17)

FIGURE 6 (17)

HEAT PUMP SPECIFICATION DATA

A. Manufacturer: Carrier

B. Total Heat Output Capacity: 320,000 Btu/hr

C. Condenser side:
   1. 40 gpm heating system water.
   2. Leaving water temperature: 130°F.
   3. Entering water temperature: 114°F.
   4. Water pressure drop: 13.5 ft.
   5. Refrigerant saturated discharge temperature: 140°F.
   6. Electric input at full load: 28.4 KW.
   7. COP = $\frac{\text{Heat Output}}{\text{Electrical Input}}$ = 3.3

D. Evaporator Side:
   1. Fluid: Sewage effluent with 10 ppm chlorine residual and minimal suspended solids.
   2. Fluid Flow: 60 gpm
   3. Entering Water Temperature: 50°F.
   4. Leaving Water Temperature: As required.
   5. Maximum water pressure drop: 5 PSI

E. Refrigerant: R-22

F. Saturated suction temperature: 36°F

TABLE 5 (17)

PROJECTED ENERGY REQUIREMENTS SUMMARY

Month	Bldg. Heating and Ventilating (GJ)	Digester Heating (GJ)	Total Heating (GJ)	Electrical (kWh)			
				Process Equipment	Heating and Ventilating Equipment	Lighting	Total Electrical
Jan.	78.85	33.43	112.28	19,860	14,560	520	34,940
Feb.	74.57	31.09	105.66	19,860	12,850	520	33,230
March	53.27	31.92	85.19	19,860	10,460	520	30,840
April	27.70	27.76	55.46	19,860	8,340	520	28,720
May	4.26	24.72	28.98	19,860	6,560	520	26,940
June	0	20.66	20.66	19,860	6,550	520	26,930
July	0	17.63	17.63	19,860	6,540	520	26,920
Aug.	0	17.86	17.86	19,860	6,540	520	26,920
Sept.	0	18.82	18.82	19,860	6,540	520	26,920
Oct.	14.91	22.81	37.72	19,860	6,550	520	26,930
Nov.	38.37	25.92	64.29	19,860	9,370	520	29,750
Dec.	53.27	30.51	83.78	19,860	11,710	520	32,090
ANNUAL	345.20	303.13	648.33	238,320	106,570	6,240	351,130

the total was projected for electrical energy leaving 14% for building and digester heating. Of significance is that 58% of the electrical energy requirement was projected for operation of process equipment.

Table 6 indicates the projected sources of energy to satisfy the heating energy demands shown in Table 5. Table 7 presents the actual energy used and produced during the ten month study. All data for energy users was monitored at the point of use. Distribution losses in addition to accuracy in monitoring account for the differences in total energy produced and total utilized.

Table 8 summarizes actual power used and actual produced heat energy as monitored during the study. Total power used over the ten month period equalled 258,440 kWh which was only 87.5% of the anticipated use . At an average flow of 454 m^3/d (0.12 mgd), this is equivalent to 1.79 kWh/m^3 (6,759 kWh/mg) which is a rather high rate of use. Total energy used at the facility was 3,403 GJ, 84% of which was for electric power. As actual flow treated reaches design capacity, the energy requirements will approach a more reasonable figure of 0.48 - 0.50 kWh/m^3 (1800 - 1900 kWh/mg) treated.

Shown in Figure 7 is a bar chart with the total energies produced by each heating system component during the ten month study.

Figure 8 illustrates estimated design and observed actual energy use and production on a system component basis.

Projected and actual percent energy source contribution for heating is presented in Table 9. It is notable that the heat pump provided 60% of the total annual energy produced by all five systems. During the months of October 1977 to March 1978, the heat pump provided the primary source of heating energy. The coefficient of performance (COP) of the heat pump varied with the temperature of the effluent. Table 10 presents a summary of heat pump performance information. The average COP observed during the study was 2.9.

The only months during which the actual collected energy by the active solar system equalled or exceeded the projected total were September and December 1981. The calculated long-term collection efficiency averaged 23% which is the net energy collected divided by total incident energy available. This is significantly lower than anticipated. The total solar energy collected was 122 GJ which was only 64% of that estimated. However, because overall actual heating demand was lower than expected, the active solar energy systems provided 82% of its expected share of energy. Reasons offered for the observed lower collection efficiency included:

1. Data/instrumentation error.

2. Collector heat loss factor
   a. Inadequate thermal isolation.
   b. Possible convective losses between the absorber plate and the rigid insulation.

TABLE 6 (17)

PROJECTED HEATING ENERGY SOURCES (GJ)

Month	Passive Solar Contribution	Active Solar Contribution	Emergency Generator Contribution	Methane Boiler Contribution	Heat Pump Contribution	Total Heating Contribution
Jan.	12.5	10.23	1.2	15.25	73.19	112.37
Feb.	15.6	14.60	1.2	14.13	60.16	105.69
March	13.9	17.74	1.2	15.25	37.12	85.21
April	7.0	15.64	1.2	14.69	17.01	55.54
May	0	15.14	1.2	12.67	0	29.01
June	0	14.90	1.2	4.60	0	20.70
July	0	17.12	0.5	0	0	17.62
Aug.	0	17.86	0	0	0	17.86
Sept.	0	18.82	0	0	0	18.82
Oct.	2.2	19.26	1.2	15.09	0	37.75
Nov.	8.4	10.98	1.2	16.71	26.99	64.28
Dec.	10.0	8.18	1.2	15.25	49.18	83.81
ANNUAL	69.6	108.47	11.3	123.64	263.65	648.66

TABLE 7 (17)

ACTUAL HEATING ENERGY CONTRIBUTION SUMMARY - USERS AND PRODUCERS (GJ)

USERS	JUNE	JULY	AUGUST	SEPT	OCT	NOV	DEC	JAN	FEB	MARCH
Digester Heating	9.774	7.603	11.561	19.107	22.335	21.177	31.363	33.531	22.917	33.624
Bldg. Htg/Ventilating	0.23	.06	.03	0.32	3.51	12.67	36.91	46.12	39.75	29.44
TOTAL	10.01	7.66	11.59	19.43	25.85	33.85	68.27	79.66	62.67	63.07
PRODUCERS										
Active Solar	10.315	8.932	12.907	25.821	10.663	8.225	11.341	7.453	13.254	14.067
Passive Solar	0	0	0	0	2.93	6.82	10.08	11.27	13.41	11.07
Generator Heat Recovery	0.327	1.047	0.445	0.457	0.345	0.548	1.192	1.065	1.333	1.665
Boiler	5.760	0.007	0.020	0	0.045	0	12.053	8.292	0.007	0.017
Heat Pump	0.559	0.005	3.784	3.057	32.594	31.653	52.778	73.548	64.159	64.728
TOTAL	16.96	9.99	17.16	29.34	46.58	47.25	87.44	101.63	92.16	91.55

TABLE 8 (17)
TOTAL ELECTRICAL ENERGY USAGE
AND HEATING ENERGY PRODUCED

Month	Measured Power (kWh)	Equivalent Elec. Heat Energy (GJ)	Produced Heat Energy (GJ)
January	29,600	328	102
February	29,920	331	92
March	30,200	335	92
April	-	-	-
May	-	-	-
June	22,680	251	17
July	22,120	245	10
August	26,200	290	17
September	21,000	233	29
October	25,840	286	47
November	23,000	255	47
December	27,880	309	87
TOTAL	258,440	2863	540

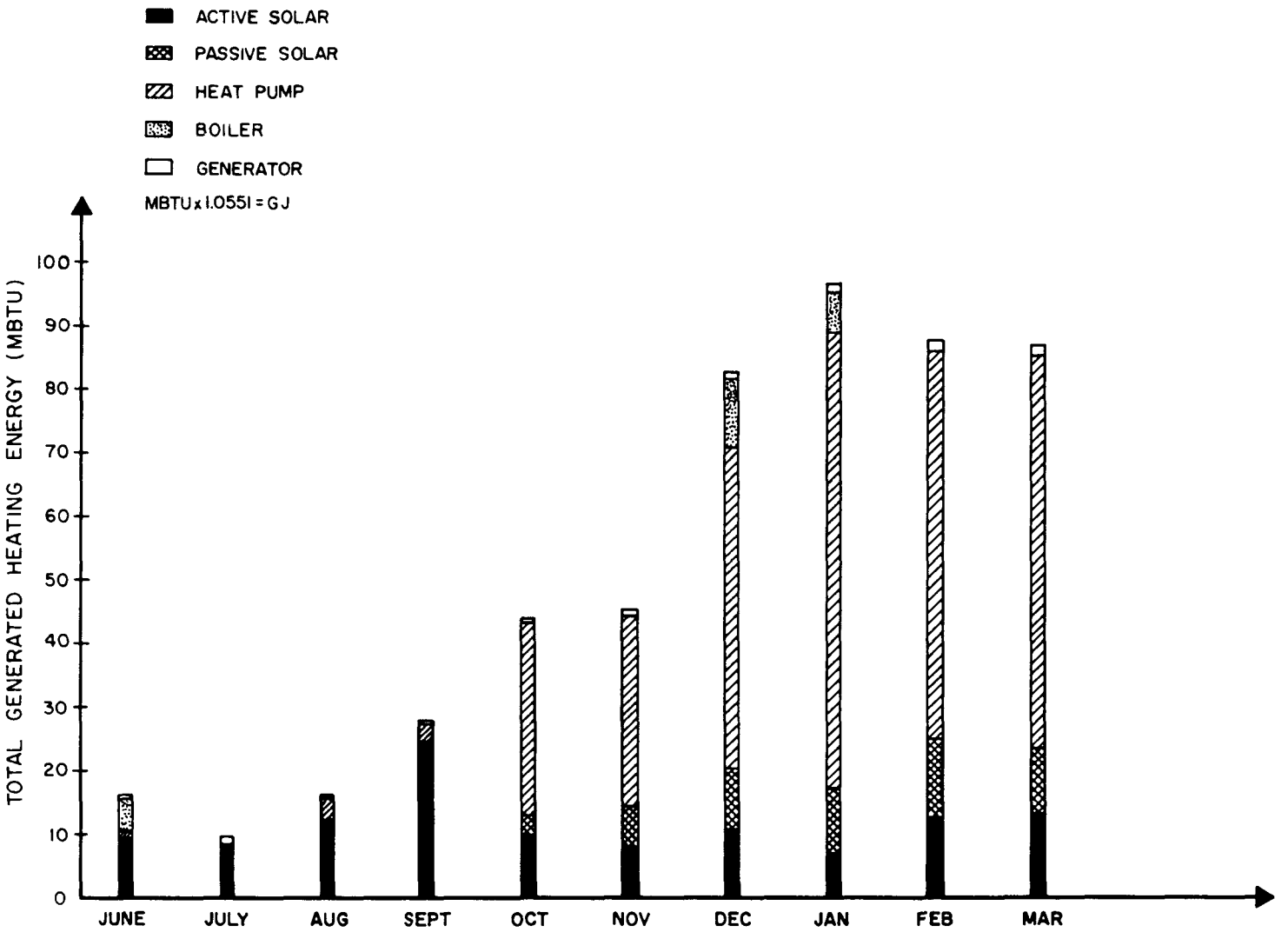


Figure 7. TOTAL ENERGY CONTRIBUTION BY EACH HEATING SYSTEM COMPONENT (17)

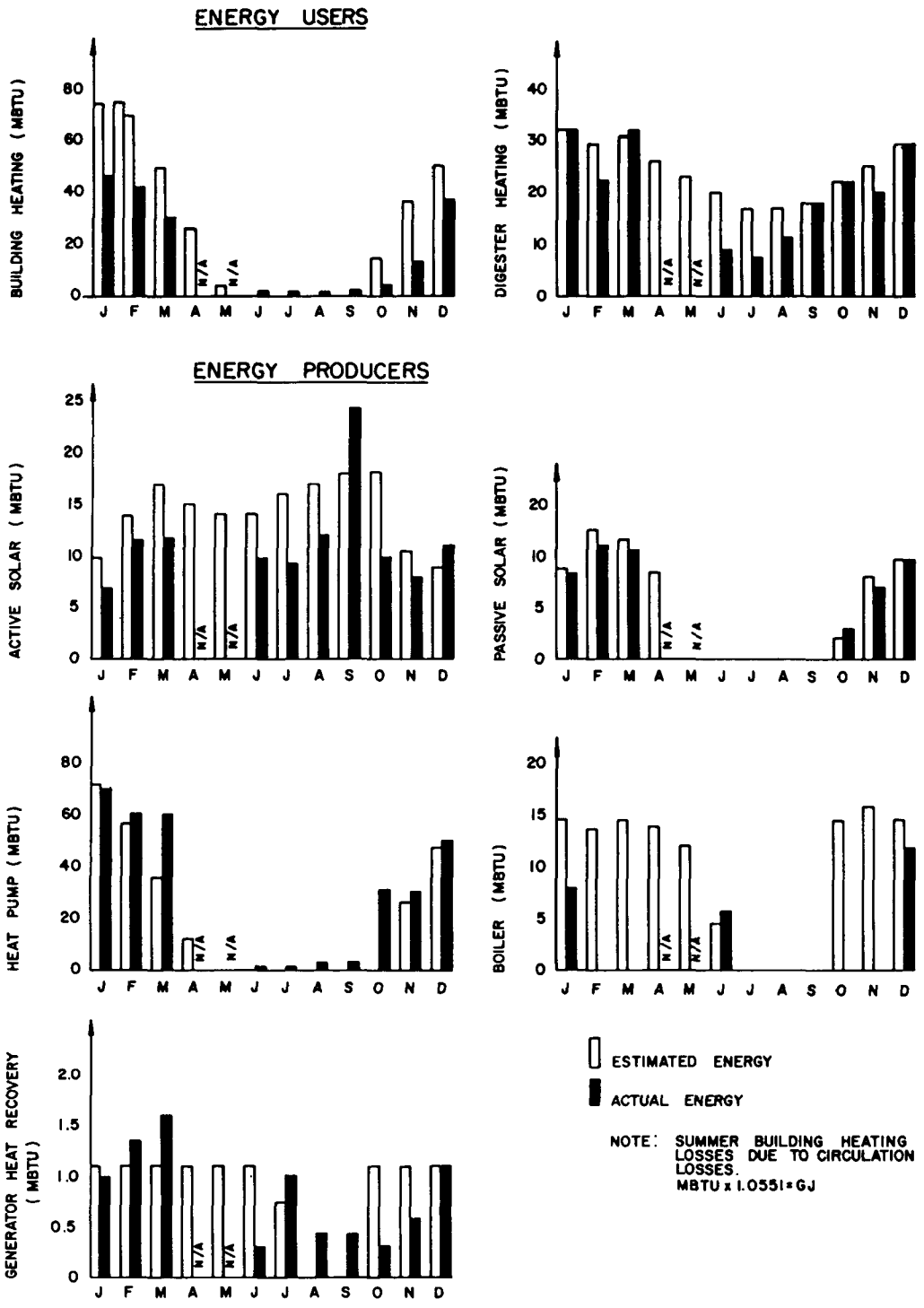


Figure 8.

Estimated Design and Observed Actual Energy Use and Production For Wilton, Maine (17)

TABLE 9 (17)

PROJECTED AND ACTUAL PERCENT ENERGY SOURCE CONTRIBUTION FOR HEATING

Month	Passive Solar		Active Solar		Emergency Generator		Methane Boiler		Heat Pump	
	Proj.	Act.	Proj.	Act.	Proj.	Act.	Proj.	Act.	Proj.	Act.
Jan.	11	11	9	7	1	1	14	8	65	73
Feb.	15	15	14	14	1	1	13	0	57	70
Mar.	16	12	21	15	2	2	18	0	43	71
Apr.	13	-	28	-	2	-	27	-	31	-
May	0	-	52	-	4	-	44	-	0	-
June	0	0	72	61	6	2	22	34	0	3
July	0	0	97	89	3	11	0	0	0	0
Aug.	0	0	100	75	0	3	0	0	0	22
Sept.	0	0	100	88	0	2	0	0	0	10
Oct.	6	6	51	23	3	1	40	0	0	70
Nov.	13	15	17	17	2	1	26	0	42	67
Dec.	12	12	10	13	1	1	18	14	59	60
ANNUAL	11	10	28	23	2	2	19	5	40	60

TABLE 10 (17)

HEAT PUMP PERFORMANCE SUMMARY

Month	Effluent Temp (°C)	Energy Input (kWh)**		Energy Output (GJ)	COP
		Compressor	Pump		
January	7.8	6807	204	68.80	2.73
February	6.8	6717	202	64.16	2.58
March	6.1	6388	192	62.64	2.64
April*	-	(2112)	(63)	(25.3)	-
May*	-	(2327)	(70)	(28.4)	-
June	15.7	40.5	1	0.51	3.43
July	18.3	0	0	0	-
August	20.0	241.9	7	3.54	3.95
September	19.3	224.9	7	2.75	3.29
October	16.7	2327	70	28.39	3.29
November	13.7	2112	63	25.32	3.23
December	10.2	4363	131	51.08	3.16
Annual		33,660	1010	360.89	2.90

* Estimated
** 1kWh = 3.6×10^6 J

3. Collector heat transfer losses
   a. Air within the fluid loop.
   b. Effect of the glycol solution.

4. Control sequencing and response.

5. Collector response sensitivity.

6. Collector efficiency losses due to dirt accumlated during construction.

The discrepancies between design predictions and operating data are attributed to the lack of a reliable design procedure to accurately distinguish between instantaneous collector efficiency and year long collector efficiency. Reliance on instantaneous efficiencies results in an over optimistic long-term performance expectation due to the phenomena of threshold temperature activation levels in relation to available insolation, cloud cover, etc. Presently available procedures of applying mean expected percent sunshine to heat harvest tables for prediction of available solar insolation have been found to be over optimistic because of the highly variable nature of sunshine. For example, on a partly cloudy day, even though the sun may shine, for 30% of the day, this may be in intermittent bursts which may not be of sufficient sustained intensity to reach threshold activation temperature levels. Also, percent sunshine tables fail to indicate time of day or intensity i.e., sunshine during midday is more productive than at other times.

Calculations indicated that the passive solar energy system provided a total of 10% of the actual annual energy used for heating which is 91% of the projected share. The manufacturer indicated a transmissivity for the panels to be 66%. The observed peak noon time transmissivity ranged from 35% in July to 57% in January. Part of the reduction is attributed to the overhang used for shading during summer. The remainder is attributed to other factors such as accumulated dirt and surface reflection.

The purpose of conserving energy at the Wilton facility was to ultimately reduce operating costs for electricity and fuel. Each energy producing component was evaluated for energy conservation as well as cost-effectiveness using a payback period approach. The results of this evaluation are shown in Table 11. Each component, except for digester gas production, was a net energy producer and net fuel cost saver. The use of an effluent heat pump and air to air heat recovery are most definitely cost-effective with good payback periods of 11 and 8 years, respectively. However, use of active solar energy at the Wilton facility is difficult to justify on a cost-effectiveness basis - the calculated payback period is 44 years.

Use of the passive solar energy system to heat the clarifier room at Wilton is judged to be marginally cost-effective. Use of passive solar to heat occupied areas where there is no exposure to water surface would improve cost-effectiveness and is a preferred method.

TABLE 11 (17)

ENERGY AND COST EFFECTIVENESS SUMMARY

Component	Output GJ	Output ($)	Input GJ	Input ($)	Initial Investment ($)	Energy Output/Input Ratio	Value Output/Input Ratio	Simple Payback (Years)
Active Solar	153.7	1,302	13.4	368	41,000	11.5	3.5	44
Passive Solar	34.26	290	0	0	7,000	N/A	N/A	24
Heat Pump	360.89	3,057	124.85	2,060	11,000	2.9	1.5	11
Generator Heat Recovery	10.13	86	0.09	1	2,900	113	86	34
Air-to-Air Heat Recovery	58.13	492	6.46	81	3,300	9.0	6.1	8
Digester Gas Production	325	2,750	551	3,837	N/A	0.59	0.72	N/A

The findings at Wilton regarding cost effectiveness of active solar energy can be generalized to an extent. Based on a technology assessment of solar thermal energy applications in wastewater treatment (18), active solar heated anaerobic digestion proved uneconomical throughout the U.S. A sensitivity analysis indicates that collector system cost is the single most significant cost item. In order to be economically viable on a payback basis, collector system costs would have to be reduced from $538/m^2$ ($50/ft^2$) to a range of $162/m^2$ to $323/m^2$ ($15 to 30/ft^2$).

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ENERGY RECOVERY, REUSE, AND CONSERVATION AT METRO CHICAGO

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ABSTRACT

In the aftermath of what, in 1973, was termed an "energy crisis," energy prices in the United States and elsewhere began to escalate rapidly. In an effort to contain costs and conserve limited natural resources, the Metropolitan Sanitary District of Greater Chicago (District) began a detailed investigation of its energy uses and costs. This paper presents a series of case studies which describe how the District reduced energy usage at its John E. Egan, O'Hare, and West-Southwest treatment facilities without any concomitant reduction in treatment efficiency.

At the John E. Egan facility the major energy saving steps were conversion from a two-stage to a single-stage nitrification system, modification of the sludge thickening process, and increased digester gas utilization. At the O'Hare facility, the major energy saving steps were conversion from a two-stage to a single-stage nitrification system and elimination of tertiary filtration. At the West-Southwest facility, the major energy saving steps were conversion of steam-driven blowers and pumps to electrically driven blowers and pumps, increased use of digester gas, elimination of the heat-drying process for sludge disposal, and reduction in air usage in the aeration tanks.

Total energy savings at these three facilities will be approximately $15.4 million/year once all of the modifications are completed.

INTRODUCTION

Historians looking back at the decade of the 1970s will probably conclude that the end of an era of inexpensive energy supplies was the dominant event of the period. In the aftermath of what, in 1973, was termed an "energy crisis," energy prices in the United States and elsewhere began to escalate rapidly. Prices of energy sources rose somewhat erratically between 1973 and 1978, and in 1979 again increased at an accelerated rate.

Although petroleum prices were the first to rise, various market forces soon caused the prices of virtually all other energy sources to rise in varying degrees. Consequently, most energy users, including the Metropolitan Sanitary District of Greater Chicago (District), experienced significant increases in their operating costs. For example, the estimated costs for natural gas and electricity at the District's West-Southwest Sewage Treatment Works increased about 39 percent from a total of $13.7 million in 1977 to $19 million in 1979. In an effort to contain costs and conserve limited natural resources, the District began a detailed investigation of its energy uses and costs.

The District serves a large area and operates numerous facilities; thus, its approach was to investigate all its installations and prepare an inventory of energy use. Although the development of such an inventory may appear obvious, it was a rather complex task for a body responsible for the collection and treatment of wastewater throughout the Chicago metropolitan area, encompassing virtually all of Cook County, Illinois. The District's service area extends over approximately 860 sq miles (2,230 km^2), and embraces a population of nearly six million people; and its various facilities receive and treat, to secondary or tertiary levels, wastewater flows that, in 1978, averaged nearly 1,400 MGD (61.0 m^3/s). What follows is a series of case studies which describe how the District focused its attention upon areas that could potentially result in cost and energy savings without any concomitant reduction in efficiency or service to its constituency.

CASE STUDIES

ENERGY SAVINGS AT THE JOHN E. EGAN WATER RECLAMATION PLANT

The John E. Egan Water Reclamation Plant (WRP) one of the seven District facilities, is located in northwest Cook County in an unincorporated area of Schaumburg and serves an area of approximately 44 square miles (114 km^2). This area encompasses most of the upper Salt Creek drainage basin and includes all or parts of Palatine, Schaumburg, Hoffman Estates, Arlington Heights, Roselle, Elk Grove Village, Rolling Meadows and Inverness. Con-

struction of the plant began in 1971 and the plant started treating sewage on December 16, 1975. The plant was constructed at a cost of about $43 million.

The plant is designed as a 30-MGD (1.31 m^3/s), two-stage nitrification system with dual media filtration for tertiary treatment. The plant consists of control, maintenance, pretreatment, filter, digester, laboratory, and thickener buildings; three pump houses; four aeration tanks; four digesters; and twelve settling tanks. The plant is capable of providing complete treatment for flows as high as 50 MGD (2.19 m^3/s). Primary treatment can be provided for an additional 75 MGD (3.28 m^3/s). All effluent flows are chlorinated.

The facilities provided (Figure 1) comprise coarse screening; pumping; fine screening; grit removal; two stages of aeration,each followed by settling; gravity filtration through dual-media filters; and chlorination.

Facilities for handling waste-activated sludge from the two aeration stages include flotation thickeners and anaerobic digesters. A centrifuge building to provide dewatering of the digested sludge was recently completed.

One of the design criteria aims for the John E. Egan WRP was to provide the maximum amount of automation to minimize the labor needed for plant operation and to provide steady and efficient treatment. To implement this criterion the plant contains a total of 1,177 instruments, transmitters and sensing elements. Most of the signals from the local instrumentation are transmitted to the control room instrumentation panel which contains a schematic of the plant with indicator lights for the major valves and equipment, as well as a multitude of recorders, indicators, and controllers. All of the analog and digital signals from the panel are entered into a dual computer system (one computer for backup).

An automatic dissolved oxygen control system maintains the oxygen content in each of the aeration tank's three passes. A typical dissolved oxygen pattern is 1.0, 1.5, and 2.0 mg/L in the first, second, and third pass, respectively. Air for the aeration system can be controlled by remote valve set point, flow set point, or cascaded via a dissolved oxygen set point from the dissolved oxygen probes. All control commands can be directed from the control room at the operator's selection. The plant's blowers, although turned on and off locally, are flow controlled via influent vanes which are varied automatically to maintain a set pressure.

The operational philosophy for the John E. Egan WRP follows the District's policy of providing the maximum degree of treatment to the maximum possible sewage quantity at minimum cost. To this end, the blowers and mechanical aeration devices are operated to maintain system air pressure at minimum levels. Air to distribution channels is reduced to minimum levels necessary to prevent solids settling. The overall air management procedure is to maintain sufficient aeration rates to achieve effluent standards without over aerating.

Plant personnel began a study in late 1979 to determine where energy

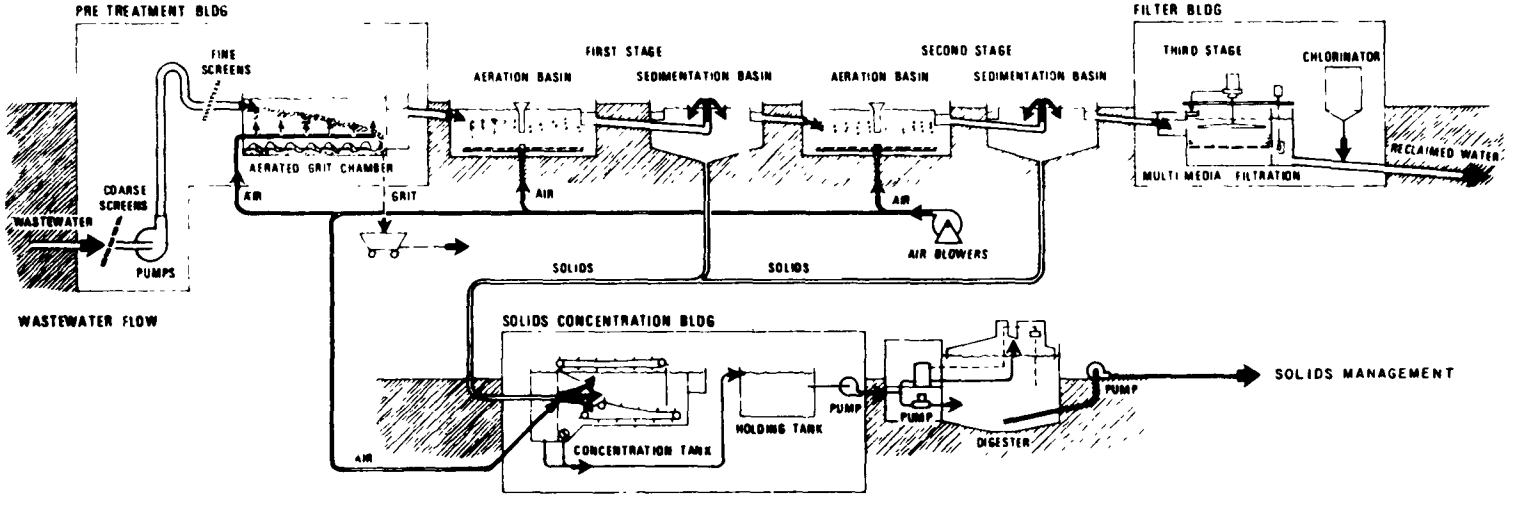


Figure 1. Schematic of the John E. Egan Water Reclamation Plant.

consumption could be reduced at the John E. Egan WRP. Process changes and methods suggested by the study were reviewed in terms of cost effectiveness and impact on treatment and plant environment. Where possible, energy consumed during each phase of the treatment process was determined. This study resulted in a series of operational changes and process flow modifications to sewage treatment and sludge handling processes that significantly reduced energy consumption.

In July 1981, the basic treatment process was changed from a two-stage nitrification process to a single-stage nitrification process, with the first-stage final settling tanks converted to primary tanks. Since the plant design and piping are arranged so that conventional activated sludge, contact stabilization, step aeration or any combination of the above process schemes can be utilized, no construction of any kind was required. It was believed that the change to a single-stage nitrification system could be successfully accomplished, since the plant was currently operating at only two-thirds of design capacity, thereby allowing sufficient hydraulic detention time for a single-stage nitrification system.

The change to a single-stage nitrification process was accomplished without any deterioration in effluent quality. Table 1 shows the final effluent suspended solids concentration and number of backwashes for 1981 and 1982. The 1981 data represents effluent quality before filtration as a two-stage operation, and 1982 data represents effluent quality before filtration as a single-stage activated sludge operation. As can be seen from this data, there was a decrease in effluent suspended solids and number of filter backwashes for relatively the same volume of sewage treated.

Table 2 shows annual averages of air used in the aeration tanks from 1980 through 1982. There was a decrease in air requirements of 3.7 MCFD (104,784 m^3/day). In addition, the air-to-sewage ratio decreased by 0.26 ft^3/gal (1.9 m^3/m^3) for the same period. These air usage decreases are due to conscientious use of the automatic dissolved oxygen control system and lower oxygen requirements for the single-stage system.

In conjunction with the change to the single-stage activated sludge process, the sludge thickener operation was eliminated. The waste-activated sludge is now pumped to the head of the plant and removed in the primary settling tanks. These primary tanks are operated with a high sludge blanket to allow for greater sludge concentration. The sludge is withdrawn at approximately 2.5 percent solids concentration and pumped directly to the anaerobic digesters.

Natural gas consumption has been reduced through modified digester operation at the John E. Egan WRP. Improved steam trap maintenance has led to increased heat exchanger efficiency in the digester. In addition, a digester gas utilization system to supply 500,000 therms per year (5.275×10^{10} kJ) for use in the John E. Egan WRP boilers was installed in 1982.

Cost saving associated with energy conservation measures mentioned above are shown in Table 3. The major saving in the aeration system is accounted for through reduction in air usage and power consumption for blower operation.

All of the operating costs for the dissolved air flotation unit, except sludge pumping cost, were saved. Digester gas utilization is expected to supply 500,000 therms/year (5.275 x 10^{10}kJ), which is 30 percent of the plant requirements for boiler fuel. At a 1981 natural gas cost of $0.37/therm ($3.51/10^6 kJ), a saving of $185,000 per year is anticipated.

TABLE 1. MONTHLY AVERAGES OF EFFLUENT SUSPENDED SOLIDS PRIOR TO FILTRATION AND NUMBER OF FILTER BACKWASHES FOR THE JOHN E. EGAN WATER RECLAMATION PLANT

Month	No. of Backwashes	Flow MGD	SS mg/L
		1981	
January	10	17.72	14
February	15	22.70	17
March	12	20.13	17
April	10	21.76	10
May	13	22.41	11
June	2	21.18	4
Average	12.4	20.98	12.2
July	Plant modified to single-stage activated sludge process		
August	NA*	NA	4
September	NA	NA	4
October	NA	NA	6
November	NA	NA	5
December	NA	NA	3
		1982	
January	4	18.47	5
February	4	18.63	7
March	12	21.11	10
April	7	24.48	6
May	3	19.39	4
June	2	17.34	4
July	4	20.19	5
August	3	17.63	5
September	3	17.29	5
October	3	16.27	6
November	9	23.41	2
December	16	26.93	8
Average	5.8†	20.60†	5.2

*NA = Not Available.
†Average for 1982 only.
Note: MGD x 0.0438 = m^3/s.

TABLE 2. AIR USAGE IN THE AERATION TANKS AT THE JOHN E. EGAN WATER RECLAMATION PLANT

Annual Average			Amount Decrease
1980	1981	1982	1980-1982
----------	----------	MCFD*	----------
13.60	11.60	9.90	3.70
----------	----------	ft^3/gal	----------
0.78	0.59	0.52	0.26

*MCFD = Million Cubic Feet Per Day.
Note: cu. ft. x 0.02832 = m^3.
ft^3/gal x 0.1336 = m^3/m^3.

TABLE 3. COST SAVINGS ASSOCIATED WITH ENERGY CONSERVATION AND PROCESS MODIFICATION AT THE JOHN E. EGAN WATER RECLAMATION PLANT

	Process Changes Made	Anticipated $ Savings/Year
	Wastewater	
1. a.	Change from two-stage to single-stage A/S system	$ 50,000
b.	Reduction in total air used	
c.	Reduction in air used/unit volume of sewage treated (air-to-sewage ratio)	
2.	Reduction in filter backwash frequency	4,000
	Sludge	
3. a.	Elimination of dissolved flotation thickeners	80,000
b.	Pump W/AS to head end of plant for removal and concentration to approximately 2.5%	
4.	Increased use of digester gas to supply approximately 30% of boiler fuel demand	185,000
TOTAL		$319,000

With the energy conservation measures at the John E. Egan WRP, a total saving of $319,000 per year is expected. Substantial savings like these will become a factor in meeting the challenge of maintaining high effluent quality while controlling operating costs, even in the face of ever-increasing fuel prices.

ENERGY SAVINGS AT THE O'HARE WATER RECLAMATION PLANT

The O'Hare Water Reclamation Plant (WRP) is the newest of the District's seven sewage treatment facilities. Located in the city of Des Plaines, the O'Hare WRP serves a 65.2 square mile (168.8 km^2) area which includes the communities of Arlington Heights, Mount Prospect, Prospect Heights, Wheeling, Elk Grove Village, Rolling Meadows, Buffalo Grove, and a part of the City of Des Plaines.

Sewage is conveyed to the plant through a 4.5-mile (7.2 km) long, 20-foot (6.1 m) diameter rock tunnel. The tunnel invert is 160 ft (48.8 m) below grade at its entrance to the plant pumping station, and serves as both a dry weather flow intercepting sewer and combined sewer overflow storage reservoir. The O'Hare system is the first fully functional segment of the District's Tunnel and Reservoir Plan (TARP) for storage and treatment of combined sewer overflows.

The O'Hare WRP was designed to treat 72 MGD (3.15 m^3/s), utilizing a two-stage activated sludge process for ammonia removal and dual media sand filtration to meet effluent standards of 4 mg/L BOD, 5 mg/L suspended solids, and 1.5 mg/L NH_4-N. The O'Hare WRP began operation in May 1980, and presently treats an average daily sewage volume of 29 MGD (1.27 m^3/s).

The O'Hare WRP is a particularly energy-intensive facility, as all sewage must be pumped from the rock tunnel-combined sewer overflow reservoir. The normal discharge head on the raw sewage pumps is 195 feet (59.4 m). Raw sewage pumping alone accounts for over 40 percent of the plant electrical consumption. In addition, the plant design requires pumping of the activated sludge plant effluent an additional 34 feet (10.4 m) up to the dual media sand filters. Prior to plant start-up, electrical energy consumption was projected at 2,500,000 KWH per month. Natural gas consumption for plant heating was projected at 900,000 therms (1 therm = 100,000 Btu = 105,500 kJ) annually.

Since any plant energy conservation program should consider all potential areas of reduction in order of power demand, the initial priorities were assigned to process equipment in order of magnitude of power consumption. Thus, an area of high potential for large energy savings is the retiring of capacity not currently in critical demand. This is especially true for a new facility in which flows and loadings are not yet up to design limits.

A decision was made in May 1980 to operate the O'Hare WRP as a single-stage activated sludge system, even though it was designed as a two-stage system. This mode of operation will continue as long as effluent standards are maintained. In 1981, the average ammonium nitrogen concentration of the raw sewage was 13.7 mg/L and the average concentration of the final effluent

was 0.2 mg/L. These figures represent an average ammonium nitrogen removal efficiency of 98 percent.

The major benefits of operating as a single-stage system are the reduction of process air requirements and the associated cost for power consumption. Table 4 shows annual average power consumption in KWH/day at the O'Hare WRP for 1980 through 1982. There was a reduction of 6,700 KWH/day for blowers used between 1980 and 1982.

TABLE 4. AVERAGE DAILY POWER CONSUMPTION AT THE O'HARE WATER RECLAMATION PLANT

	Annual Average KWH/Day*			(Increase) Decrease
	1980	1981	1982	1980-1982
Raw Sewage Pumps	23,000	24,500	25,500	(2,500)
Blowers	30,000	26,000	23,300	6,700
Sand Filtration	6,100	6,600	0	6,100
Sludge Disposal	2,000	2,400	2,400	(400)
Heating and Ventilation	10,200	5,500	3,900	6,300
Process Water	1,400	1,100	800	600
Lighting	2,500	1,700	1,300	1,200
Miscellaneous	8,000	5,000	4,700	3,300
TOTAL	83,200	72,800	61,900	21,300

*KWH = Kilowatt Hours.

Table 5 summarizes how air use has been reduced since 1980. As can be seen, the largest reduction was realized by reducing air rates to the raw sewage, return sludge, and mixed liquor channels. Total channel length requiring aeration is over 3,000 feet (914.4 m), with channel widths ranging from six feet to 14 feet (1.83 to 4.27 m). Solids deposition in these channels was a major concern; therefore, air rates were reduced on a trial-and-error basis to present rates. Aeration tank air rates were also reduced by changing the dissolved oxygen set point on a trial-and-error basis. Presently, the set point is maintained at 2.0 mg/L approximately 100 feet (30.48 m) from the effluent end of the aeration tanks.

On October 27, 1981, dual media filtration was discontinued. The second-stage settling tanks were put in service to provide a second settling of the first-stage effluent. There was no significant reduction in treatment

efficiency, and effluent quality continued to exceed standards.

TABLE 5. AIR DISTRIBUTION AT THE O'HARE WATER RECLAMATION PLANT

Area	Annual Average, MCFD* 1980	1981	1982	Increase(+) Decrease(-) 1980-1982
Aeration Tanks	15.70	14.70	12.20	-3.50
($ft.^3$/gal. Sewage)	(.64)	(.58)	(.50)	(-0.14)
Air Lift Pumps	2.20	2.20	2.50	+0.30
Distribution Channels	14.20	10.60	6.50	-7.70
Grit Channel	1.00	1.00	0.40	-0.60
Post Aeration	1.00	1.00	0.20	-0.80
(Overall, $ft.^3$/gal.)	(1.31)	(1.06)	(0.76)	(-0.55)
Total Air Use (MCFD)	34.10	29.50	21.80	-12.30

*MCFD = Million Cubic Feet Per Day.
Note: cu. ft. x 0.02832 = m^3; $ft.^3$/gal. x 0.1336 = m^3/m^3.

As shown in Table 6, the average plant effluent quality with filtration during 1980 and 1981, was 2.4 and 2.2 mg/L BOD, 2.0 and 1.4 mg/L suspended solids, and 0.7 and 0.2 mg/L NH_4-N, respectively. The average plant effluent quality, without filtration in 1982, was 2.3 mg/L BOD, 2.1 mg/L suspended solids, and 0.3 mg/L NH_4-N.

TABLE 6. ANNUAL AVERAGE EFFLUENT QUALITY AT THE O'HARE WATER RECLAMATION PLANT

Year	Treated Flow, MGD	Suspended Solids(mg/L)	BOD(mg/L)	NH_4-N(mg/L)
1980	26.1	2.0	2.4	0.7
1981	27.7	1.4	2.2	0.2
1982	28.8	2.1	2.3	0.3

Note: MGD x 0.0438 = m^3/s.

Annual reductions in electricity and natural gas consumption since 1980 are depicted in Figures 2 and 3. Plant electrical consumption has been reduced 26 percent from 2,546,674 KWH per month in 1980 to 1,882,654 KWH per month in 1982. Natural gas consumption has been reduced 70 percent from 456,620 therms (4.82×10^{10} kJ) in 1980, to 137,500 therms (1.45×10^{10} kJ) projected for 1983. It must be pointed out that these reductions are due to numerous energy conservation measures initiated during the two and one-half years since plant start-up. However, a large percent of the reductions are attributed to taking the second-stage aeration system and dual media filtration out of service.

The dollar savings realized through conservation efforts have been substantial. Electricity bills averaged $105,050 per month in 1980 and $98,783 per month in 1982, and natural gas cost $137,846 in 1980, and $88,509 in 1982, representing an annual energy cost savings of approximately $125,000 in 1982 compared to 1980. Table 7 shows the cost savings for the major process modifications at the O'Hare Water Reclamation Plant. It should be remembered, however, that the O'Hare WRP is currently operating at only 40 percent of design flow, which allows for greater operating flexibility than one might find at a more fully loaded plant. Therefore, energy cost savings may have been easier to achieve than at a more fully loaded treatment plant.

TABLE 7. COST SAVINGS ASSOCIATED WITH ENERGY CONSERVATION AND PROCESS MODIFICATION AT THE O'HARE WATER RECLAMATION PLANT

	Process Changes Made	Anticipated $ Savings/Year
1.	a. Change from two-stage to single-stage A/S system	$122,000
	b. Reduction in total air used	
	c. Reduction in air used/unit volume of sewage treated (air-to-sewage ratio)	
2.	Elimination of dual media filtration	111,000
3.	Decrease in process water usage	26,000
TOTAL		$259,000

ENERGY SAVINGS AT THE WEST-SOUTHWEST SEWAGE TREATMENT WORKS

The energy inventory that the District conducted indicated that the West-Southwest Sewage Treatment Works (STW) is the District's largest energy con-

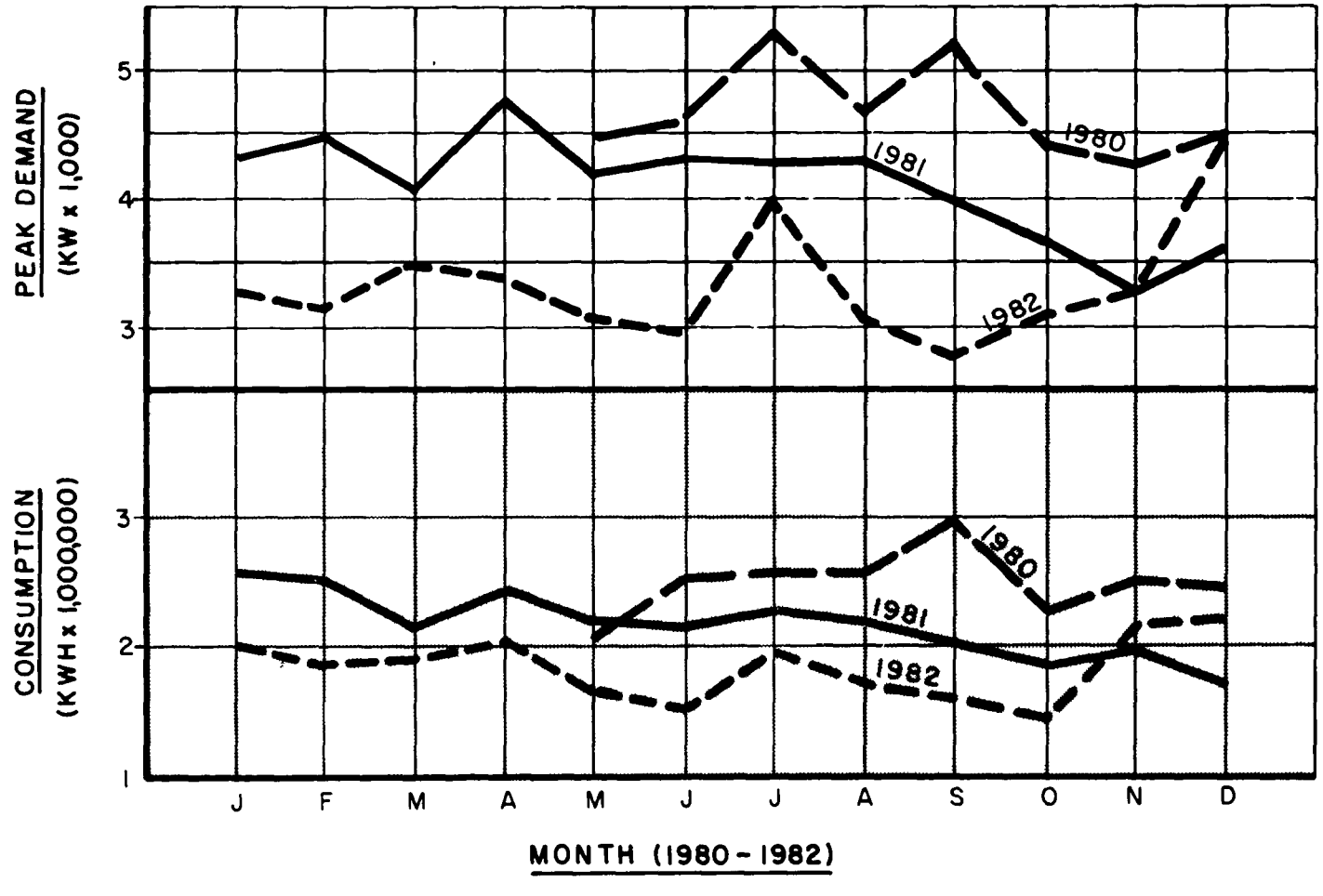


Figure 2. Monthly electrical usage at the O'Hare Water Reclamation Plant.

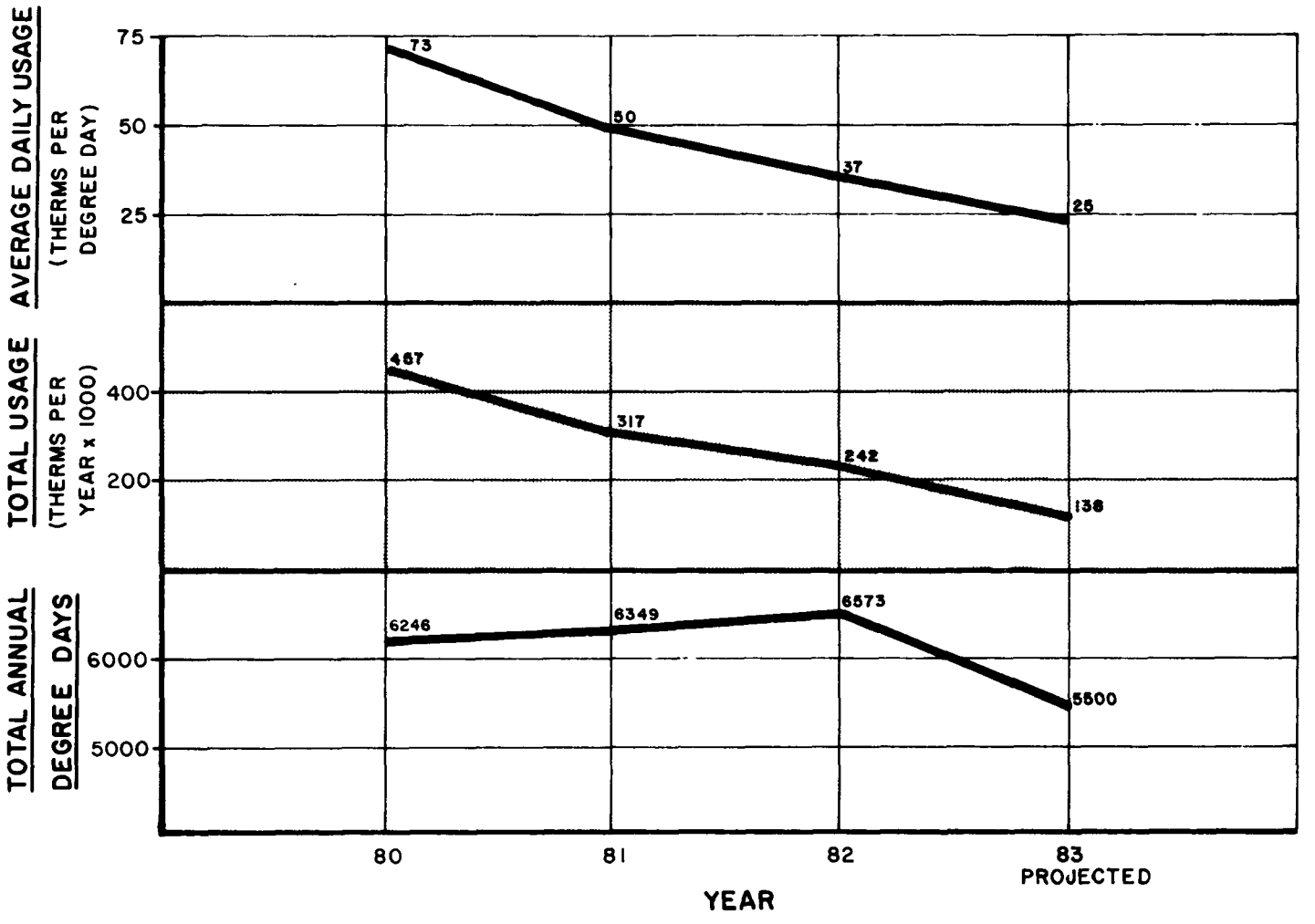


Figure 3. Natural gas usage at the O'Hare Water Reclamation Plant.

sumer. The West-Southwest STW, believed to be the largest treatment facility in the world, with a design capacity of 1,200 MGD (53.0 m^3/s), is a conventional activated sludge process facility, designed and constructed in the late 1930s. The sewage treatment process train consists of course screening, grit removal, fine screening, preliminary settling, secondary aeration, final settling, and effluent chlorination. The sludge processing and disposal systems consists of a combination of sand bed dewatering, anaerobic digestion, vacuum filtration, heat-drying, centrifugation, and land application.

Since the late 1970s, four major operating changes have been made to conserve energy at the facility. These four changes are described in the following sections.

Conversion of Steam-Driven to Electric-Driven Equipment at the West-Southwest Sewage Treatment Works

The West-Southwest STW currently uses a combination of steam and electricity to meet its energy requirements. Seven natural gas-fired boilers supply moderate pressure 425 psig (2,930 kN/m^2) steam to turbine-driven pumps and blowers. In addition, approximately 80 percent of the average 15,000 kW electrical demand is generated by turbine-driven generators, which permits the West-Southwest STW to generate most of its energy needs from natural gas. The boiler facility is also an integral part of the heat-dried fertilizer process in which vacuum-filtered wastewater solids are contacted with boiler flue gases in flash-drying units. Natural gas-fired afterburners are used to incinerate aerosols resulting from the heat-drying process.

The investigations indicated that the boilers, steam turbine, and heat-drying systems of the facility were very energy intensive and comprised the largest uses of energy in the District. Of a current 271,200 kW total energy demand at the West-Southwest STW, approximately 99 percent, or 267,600 kW, is provided by natural gas; and one percent, or 3,600 kW is furnished by purchased electricity. Of the 267,700 kW furnished by natural gas, approximately 78,400 kW is related to heat-drying; 36,400 kW is related to wastewater treatment operations; and the balance, 152,800 kW, is lost in energy conversion. In 1977, the annual energy costs for the West-Southwest STW included $13 million for natural gas and $620,000 for electricity; for a total of about $13.6 million. The present and future installed capacity and operating demand at the West-Southwest STW are contained in Tables 8 and 9, respectively.

TABLE 8. PRESENT INSTALLED CAPACITY AND OPERATING DEMANDS (1977)

Facilities	Installed Capacity (kW)	Average Demand (kW)
Electric motors	36,700	15,000
Steam-driven turbines	39,900	25,500
TOTAL	76,600	40,500
Steam-driven electric generators	18,000	11,000 (Average Output)

TABLE 9. FUTURE ENERGY CHARACTERISTICS (1995)

Facilities	Installed Capacity (kW)	Normal Load* (kW)	Heavy Load† (kW)
WS wastewater pumps‡	9,000	7,500	9,000
SW wastewater pumps§	11,200	2,100	4,300
Air compressors	58,600	32,200	47,700
Outfall pumps	5,670	3,000	4,300
Misc. equipment	32,000	10,100	10,100
TOTAL	116,500	52,200	75,400

*Average daily flow of 903 MGD (39.6 m^3/s), 205 days per year.
†Average daily flow of 903 MGD (39.6 m^3/s) plus Tunnel and Reservoir Plan (TARP) dewatering flow; total flow of 1,360 MGD (59.5 m^3/s) for 160 days per year.
‡WS = West Side (Imhoff Tank Facility).
§SW = Southwest (Primary Tank Facility).

With a view toward reducing the energy intensiveness of the facility, eight alternative systems were proposed and studied in detail. They are as follows.

Alternative 1 - Retain Existing System--The present steam turbine system would be retained to meet the existing energy requirements. Additional electricity would be purchased from the local utility to meet future energy requirements.

Alternative 2 - Purchased Electric Energy--All existing and future prime movers would be electrically driven. The total electrical demand would be supplied by purchased electricity.

Alternative 3 - District-Generated Electricity--All existing and future prime movers would be electrically driven. A new multi-fueled electric generating plant would supply the total electrical demand.

Alternative 4 - Modernized Steam-Drive--All existing and future prime movers would be turbine-driven. A new multi-fueled boiler system would supply steam for the facility.

Alternative 5 - Gas Turbine Drive--The existing steam turbines would be replaced with natural gas turbines. All future prime movers would be electrically driven. The energy demand would be supplied by a combination of natural gas and purchased electricity.

Alternative 6 - Purchase Peaking Steam--All prime movers would be electrically driven. Electricity would be derived from two sources. Electricity for the base load would be purchased at a constant rate from a local utility,

but additional power demands would be generated from in-house turbine-driven generators. Steam purchased over-the-fence would be used to drive the electric generators.

Alternative 7 - Purchase Peaking Electricity--This alternative is similar to Alternative 6, except that the steam purchased over-the-fence would be used to generate base load requirements and electricity purchased from a local utility would provide the balance of the demand.

Alternative 8 - Purchase Total Steam--This alternative is similar to Alternative 4, modernized steam drive, with the exception that the steam would be purchased over-the-fence.

The alternatives were subjected to an economic analysis involving determination of annual and present worth costs. Because the analyses were prepared over several years, all costs were related to the "Engineering News Record" (ENR) Cost Index for Chicago of 2,600, corresponding to price levels prevailing in 1977. Furthermore, all costs for in-house energy generation were developed on the basis of using natural gas as the primary fuel. The cost analysis, as presented in Table 10, indicates that Alternative 2 has an annual cost of approximately $13.1 million, as compared to the analogous cost of nearly $19 million for Alternative 1, which involves retaining the present system. Thus, the implementation of Alternative 2 at the West-Southwest STW would result in a projected annual savings of about $5.9 million.

In view of the uncertainty regarding the effect of inflation, any estimate of future costs is subject to question. For this reason, a sensitivity analysis was prepared to evaluate the effects of varying the future values of various economic parameters on the total present worth costs of each of the alternatives. The annual escalation rate for operation and maintenance (O & M) and energy costs was varied to determine if changes in the inflation rate could alter the economic analysis. The calculated data for four of the eight alternatives that derive energy from a single source are presented in Figure 4. Examination of Figure 4 clearly indicates that Alternative 2 will result in the least present worth costs in all cases studied. A typical result of the present worth analysis is presented in Table 11.

The efforts to conserve energy should deal not only with monetary costs, but also with the conservation of our primary energy resources. In the spirit of conservation, the primary energy requirements of the eight aforementioned alternatives were estimated, and are presented in Figure 5. The figure indicates that implementation of Alternative 2 would result in an energy demand of approximately 182,600 kW, as compared to 245,500 kW by the existing system. Thus, a shift to Alternative 2 would yield a saving of 62,900 kW of primary energy demand. Plans for implementing Alternative 2 are currently underway and the system should be operational in 1985.

TABLE 10. ECONOMIC COMPARISON OF ALTERNATIVES, ANNUAL COST BASIS ($1,000/YEAR)

	Alt. 1 Retain Existing System	Alt. 2 Purchase Elec. Energy	Alt. 3 District-Generated Elec.	Alt. 4 Modern Steam Drive	Alt. 5 Gas Turbine Drive	Alt. 6 Purchase Peak Steam	Alt. 7 Purchase Peak Elec.	Alt. 8 Purchase Total Steam
Amortized capital*	617	1,282	4,868	4,907	1,812	3,845	3,845	3,565
Operation and Maintenance	3,602	986	2,918	3,356	3,602	1,814	1,687	3,172
Electricity†	5,619	10,830	--	--	5,619	6,785	4,327	--
Natural gas‡	9,130	--	10,501	11,230	8,076	--	--	--
Steam§	0	--	--	--	--	10,857	14,546	21,613
Digester gas credit#	0	--	--	--	--	(788)	(788)	(788)
Total	18,968	13,098	18,287	19,493	19,109	22,513	23,617	27,562

*Amortized capital cost; term = 30 years, interest = 6.625%.
†Electricity purchased at rate of $0.0214/KWH (January 1977).
‡Natural gas purchased at rate of $0.167/therm ($0.106/10^6 kJ), (January 1977).
§Steam purchased from outside vendor at the following rates:
$3.15/$10^6$ Btu ($2.99/$10^6$ kJ) for Alternate 6.
$2.70/$10^6$ Btu ($2.55/$10^6$ kJ) for Alternate 7.
$2.90/$10^6$ Btu ($2.75/$10^6$ kJ) for Alternate 8.
#Digester gas sold at annual credit of $788,000 to steam producer.

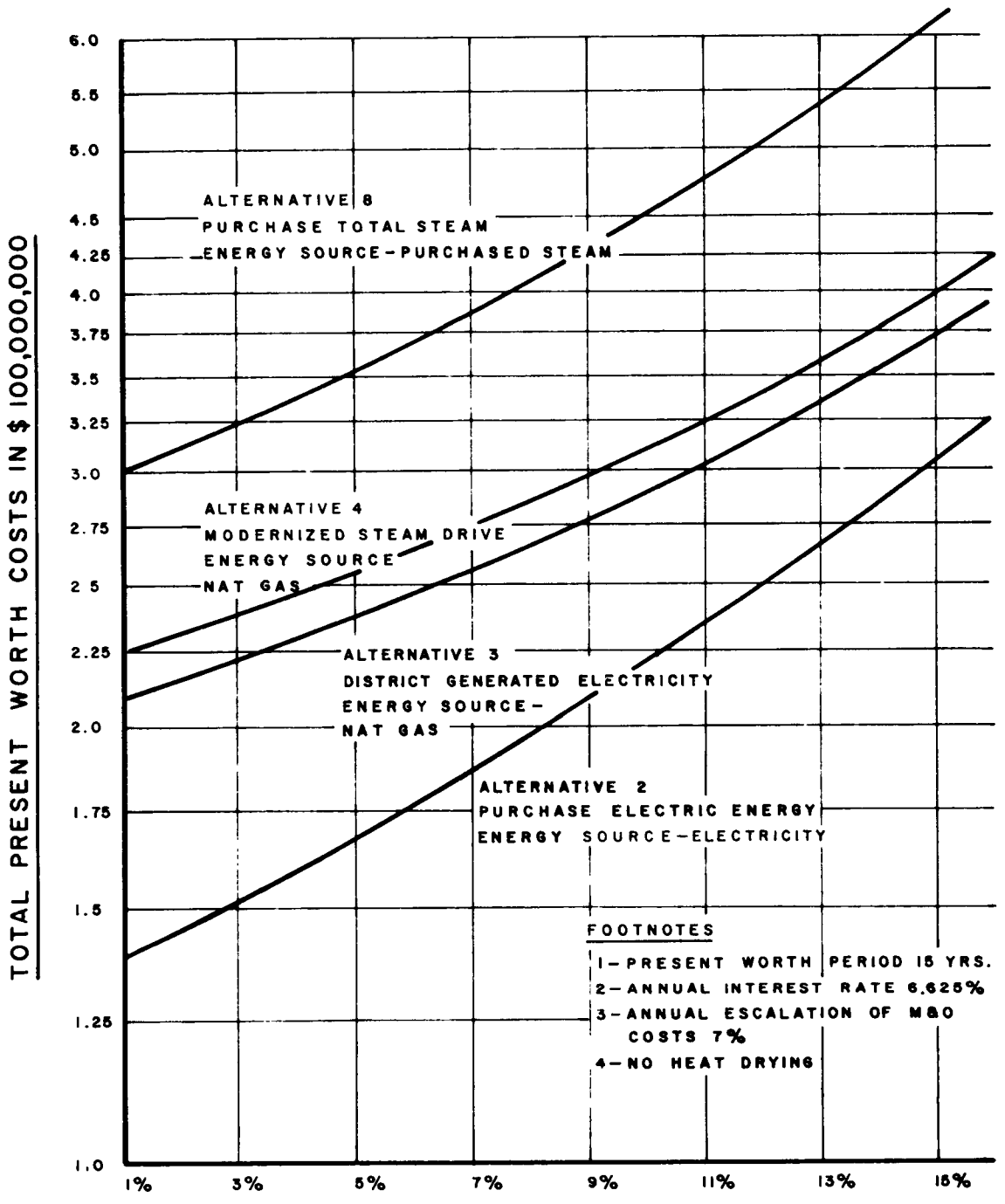
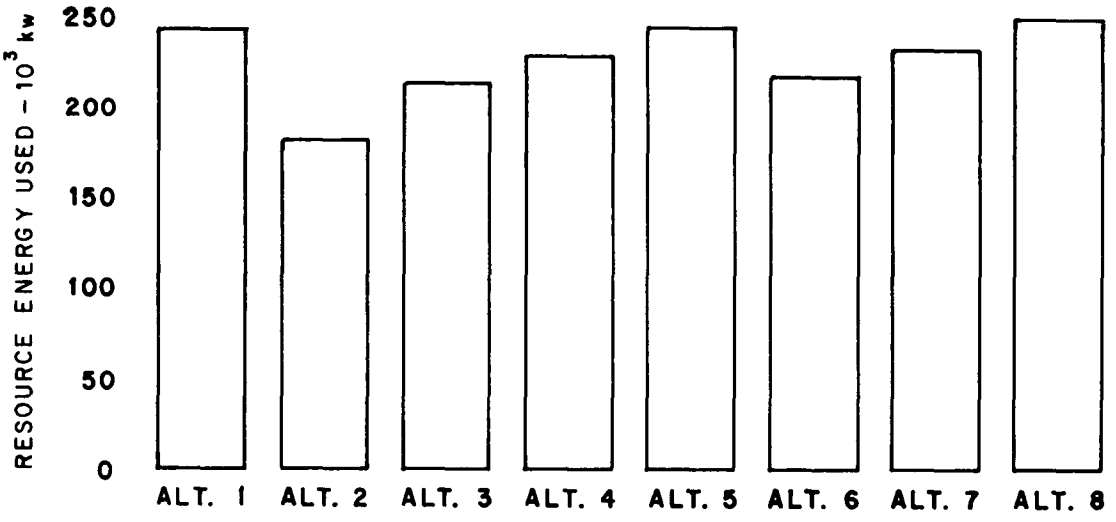


Figure 4. Total present worth costs as a function of the annual escalation rate of energy cost.

TABLE 11. ECONOMIC COMPARISON OF ALTERNATIVES, PRESENT WORTH BASIS ($ MILLION)*

	Alt. 1 Retain Existing System	Alt. 2 Purchase Elec. Energy	Alt. 3 District-Generated Elec.	Alt. 4 Modern Steam Drive	Alt. 5 Gas Turbine Drive	Alt. 6 Purchase Peak Steam	Alt. 7 Purchase Peak Elec.	Alt. 8 Purchase Total Steam
Capital cost	8.0	16.5	62.8	63.3	23.4	49.6	49.6	46.0
Operation and Maintenance†	51.9	14.2	42.1	48.4	51.9	26.2	24.3	45.7
Electricity‡	106.3	204.9	--	--	106.3	128.4	81.9	--
Natural gas§	199.0	--	228.9	244.8	176.0	--	--	--
Steam#	--	--	--	--	--	156.6	209.8	311.7
Digester gas credit**	--	--	--	--	--	(11.4)	(11.4)	(11.4)
Total	365.2	235.6	333.8	356.5	357.6	349.4	354.2	392.0

*Present worth calculated for term of 15 years and interest rate of 6.625%.
†Annual escalation rate for operation and maintenance costs is 7%.
‡Annual escalation rate for electricity purchased is 11%.
§Annual escalation rate for natural gas purchased is 13%.
#Annual escalation rate for purchased steam is 7%.
**Annual escalation rate for digester gas credit is 7%.



Note: ALT. 1 = RETAIN EXISTING SYSTEM
ALT. 2 = PURCHASE ELECTRICAL ENERGY
ALT. 3 = DISTRICT GENERATED ELECTRICITY
ALT. 4 = MODERN STEAM DRIVE
ALT. 5 = GAS TURBINE DRIVE
ALT. 6 = PURCHASE PEAK STEAM
ALT. 7 = PURCHASE PEAK ELECTRIC
ALT. 8 = PURCHASE TOTAL STEAM

Figure 5. Resource energy utilization, comparison of alternatives.

Digester Gas Utilization Program at the West-Southwest Sewage Treatment Works

The West-Southwest STW has an anaerobic digestion complex consisting of twelve 2,500,000-gallon (9.5×10^6 l) floating-cover high-rate digestion units. Each digestion unit is rated at 25 dry tons/day (22,680 kg/day) at a 3.3 percent solids feed. Digester operational data for 1977, based on monthly averages, are: (1) solids feed--4.8 percent to 5.7 percent; (2) organic loading--0.082 lb/cu ft/day to 0.130 lb/cu ft/day (1.31 kg/m^3/day to 2.08 kg/m^3/day); (3) volatile solids destruction--18 percent to 40 percent; (4) detention time--15 days to 25 days; and (5) gas production--13.1 cu ft/lb to 18.1 cu ft/lb (0.82 m^3/kg to 1.13 m^3/kg) of volatile solids destroyed.

Sludge heating and mixing is accomplished by continually circulating the sludge through water tube heat exchangers. Hot water is supplied to the heat exchangers from digester gas-fired fire-tube boilers. There are sixteen boilers for twelve digesters. Feed to the digesters is predominately waste-activated sludge.

The first four digesters at the West-Southwest STW were built and placed into service in 1964. At that time, the use of digester gas consisted only of that amount required to maintain mesophilic conditions in the digesters and to provide heat for the digester building. The economics at that time indicated that the excess digester gas be flared off in waste gas burners. Two additional sets of four digesters each were subsequently built in 1969 and 1973, and again the waste gas was flared off.

Since 1974, with the Middle East oil embargo and the shortage of domestic energy, the cost of energy has skyrocketed and made the utilization of this waste gas not only economical, but mandatory.

Digester Gas Production and Dollar Value--The gas produced in each digester is metered and recorded by a chart and totalizer. The average total gas production in 1977 was 2.7×10^6 cu ft/day (0.076×10^6 m^3/day). Figure 6 shows the gas produced on a monthly basis in 1977, and the gas utilized to maintain the digesters at 95°F (35°C). The difference between the two is the digester gas wasted, or the amount of gas available for further utilization. An average of 1.0×10^6 cu ft/day (0.028×10^6 m^3/day) of digester gas is available for additional utilization.

Digester gas is composed of approximately 65 percent methane, 30 percent carbon dioxide, and five percent varying amounts of nitrogen, hydrogen, carbon monoxide, and oxygen. This gas is capable of supplying 650 Btu per standard cu ft (24,214 kJ per standard m^3). Thus, the 1.0×10^6 cu ft/day (0.028×10^6 m^3/day) of excess digester gas has the energy equivalent of 650×10^6 Btu/day (686×10^6 kJ/day).

The major fuel source at the West-Southwest STW is purchased natural gas. In 1979, the price of the natural gas averaged approximately $0.25/therm ($2.37/10^6 kJ). Thus, the 6,500 therms (686×10^6 kJ) of unused digester gas is worth approximately $1,625/day or $593,125/year at 1979 prices.

Method of Utilization--In order to actually realize this dollar savings, a method had to be chosen to convert the digester gas to usable energy. Methods of utilizing digester gas studied at the West-Southwest STW included gas turbine generators, internal combustion engines, and water tube boilers.

Gas turbine-generating systems require a large capital investment. In our particular case, this meant $2,500,000 to $3,000,000. Also, there was much concern about cleaning the digester gas, which is a necessity for keeping the combustion chamber clean.

Internal combustion engines, as observed in other facilities, had high noise levels and an abundant amount of supporting equipment, such as lubrication, ignition, carburetion, and cooling systems. It appeared that there would be a problem using digester gas on an internal combustion engine since some plants had to redesign component parts of the engine to deal with the corrosive nature of the gas. Additionally, there was no convenient direct application to a pump or blower in the plant.

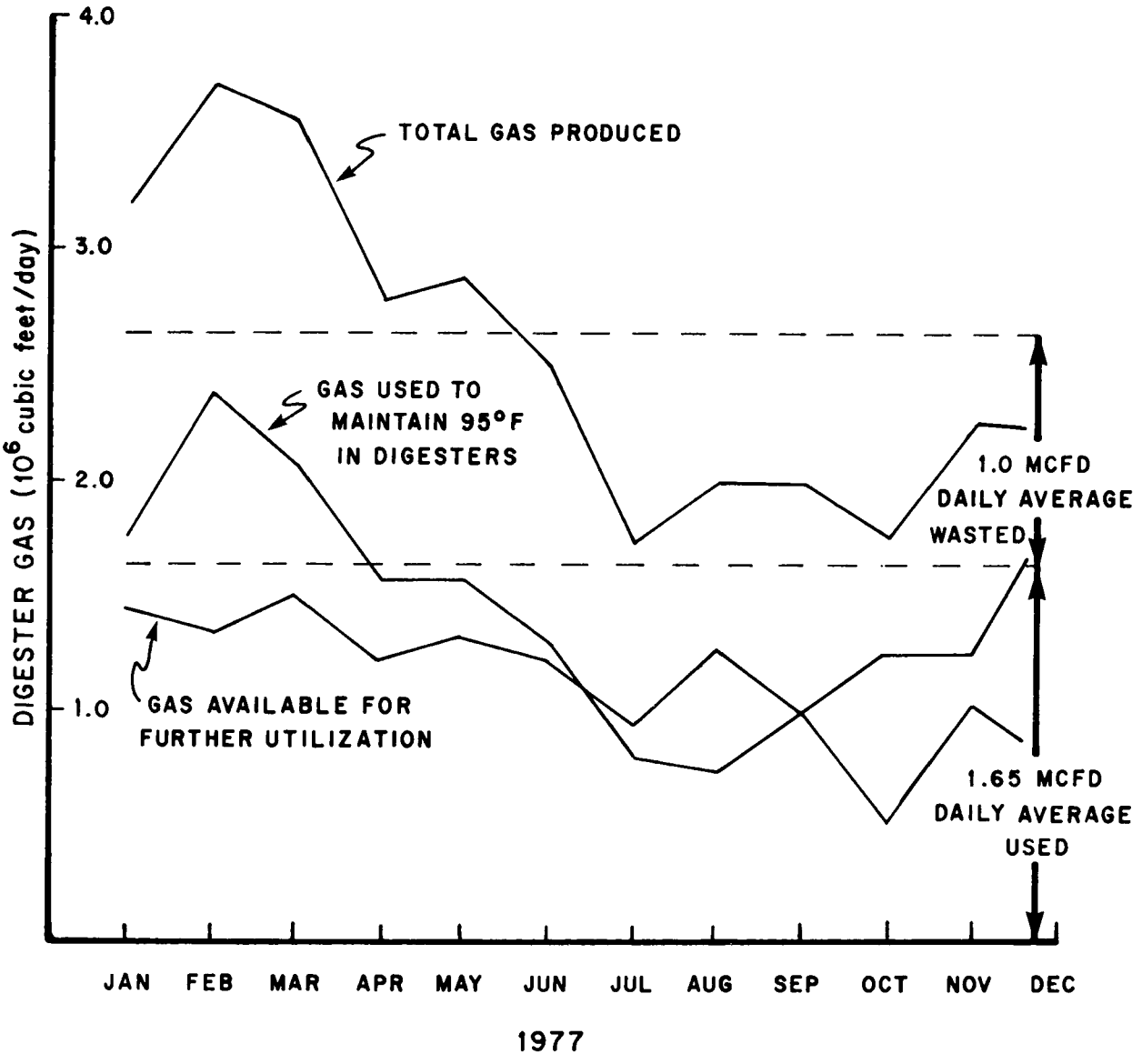


Figure 6. Digester gas production and utilization in 1977 at the West-Southwest STW.

The third alternative--ie., water tube boilers--was studied and became our first choice for three reasons:

1. Low capital cost and quick return on investment. It was estimated that the cost of material and labor to complete the project would be $870,000 (1979 prices). The cost of operation, including maintenance, is estimated at $150,000/yr. As the value of the digester gas has been calculated to be $593,125 annually, it can be seen that the project cost would be amortized in less than three years.

2. Complete utilization of low-pressure steam. The low-pressure steam requirements and the source of steam were studied along with the amount of steam generated from the excess digester gas. It was found that at no time would the amount of low-pressure steam available from the excess digester gas exceed the demand for low-pressure steam at the West-Southwest STW. Since the demand for low-pressure steam is present, a consistent boiler operation would be expected.

3. By 1985, the West-Southwest STW expects to replace six turbo-pumps and seven turbo-blowers with electric drives. This equipment is being phased out along with seven high-pressure steam generators due to their age and high cost of maintenance and operation. Elimination of the high-pressure steam generators will necessitate the installation of a new steam-generating system.

Boiler Selection--Once it was decided to utilize the excess digester gas by producing steam, it was necessary to determine the size and type of boiler best suited for this application. An engineering analysis of the daily variation in excess gas production and the steam-generating efficiency of various boilers led to the conclusion that 22,200 lb/hr (10,070 kg/hr) of steam would be the average output of the boiler required to burn all the excess digester gas. The maximum and minimum loads were calculated to be 32,000 lb/hr (14,515 kg/hr) and 10,000 lb/hr (4,536 kg/hr), respectively. From this it was decided to use two 35,000 lb/hr (15,876 kg/hr) steam generators. Two units were chosen to provide the facility with a backup unit to be used during repairs and also because there is a plan to increase the digester complex by 50 percent.

Proposed Installation--A schematic of the overall digester gas utilization system and the associated pressure control system designed to control this system is shown in Figure 7. Two gas pressure transmitters are installed in the gas header, one for each compressor. The transmitter puts out electrical signals (4 mA-20 mA) to the d-c variable speed controller which regulates the speed of the motor in a manner that maintains the pressure in the digester header at a 5.25-inch (133 mm) water column, plus or minus 0.5 in. (12.7 mm).

At the steam-generating end, the gas at line pressure is stored in the 2,000-cu ft (56.64 m^3) vessel which will function as a buffer for the system. Due to the variable rate of gas production, it was felt that this vessel would dampen the fluctuations in fuel supply to the boilers, thereby having a more uniform steam output from the boilers.

Incorporated into the controls is an alarm system designed to provide safety to the operation. The alarms, visual and audio, monitor such things as low gas header and compressor suction pressure, high discharge pressure and temperature, and the presence of methane gas in the area. In these situations, the compressors are automatically shut down.

As of 1982, this proposed installation has been completed and is operating satisfactorily, providing the District with a substantial cost savings.

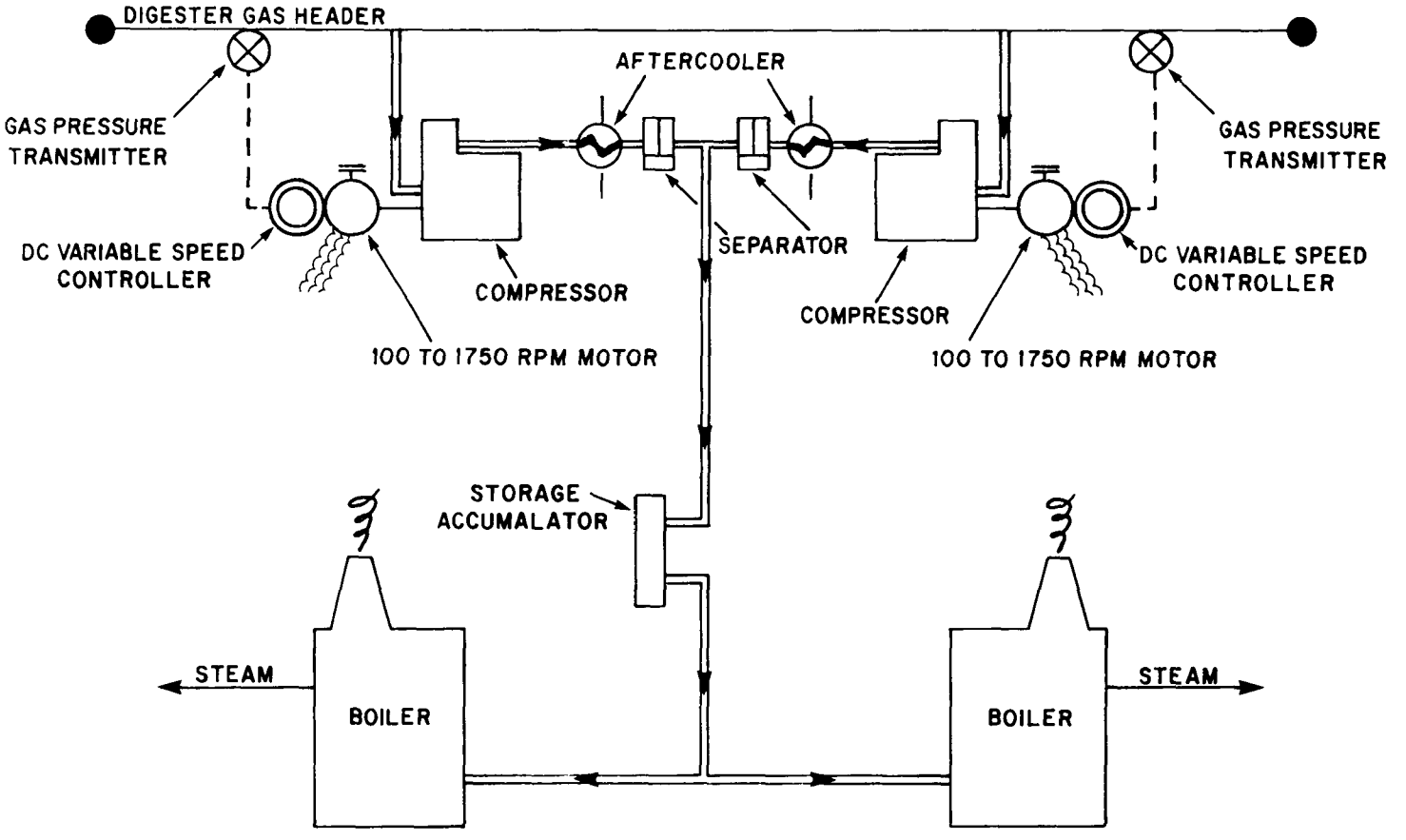


Figure 7. Schematic of the digester gas utilization system.

Elimination of Heat-Drying at the West-Southwest Sewage Treatment Works

The West-Southwest STW is, as the name implies, a combination of two facilities, operating together now, on the same site. The older West Side Sewage Treatment Works, which was placed into operation on June 6, 1930, is a primary treatment facility providing primary settling and digestion in 108 Imhoff tanks. Imhoff tank sludge is air-dried on sand beds and then placed into storage at the Harlem Avenue Solids Management Area (HASMA), two miles west of it. The West Side STW operation continues today essentially as it has in the past. The primary treatment provided by the West Side STW, however, proved inadequate for the increasing sewage flows generated in the growing Chicagoland area in the 1930s. To meet the needs of the community and to protect the water quality of the Chicago Sanitary and Ship Canal, an activated sludge facility was needed; and thus, the Southwest STW was built and placed into operation in 1939. The combined West-Southwest STW treats the Imhoff tank effluent produced at the West Side STW, along with the primary tank effluent from the Southwest Side STW itself. After a number of expansions through the years, the existing West-Southwest STW has a secondary treatment design capacity of 1,200 MGD (52.6 m^3/s).

The Southwest STW was designed, in part, around a heat-drying process, the solids disposal method selected in 1939 for processing waste-activated sludge. Hot gases used for flash-drying sludge are produced in boilers which also produced steam. This steam is then used to power all of the raw sewage pumps, the blowers, and three generators used to produce plant electricity.

The heat-drying process works as follows (Figure 8). Waste-activated sludge solids concentrated to approximately one to two percent solids content in gravity concentration tanks are conditioned with ferric chloride and sent to a bank of vacuum filters. At the vacuum filters, the conditioned sludge is dewatered partially to produce a filter cake of approximately 15 percent solids. The filter cake then drops onto a conveyor belt and is transported to a mixer where it is combined with dried solids from the drying line to obtain a mixture of approximately 55 percent solids, which is fed to the drying tower. Here hot gases of approximately 1,300°F (700°C) are mixed with the solids, resulting in an instantaneous evaporation of water. The dried solids are cooled, screened, and transmitted by conveyor belt to the fertilizer storage facilities. The material is an excellent soil conditioner and fertilizer base and is presently sold to a broker on a competitive bid basis. The broker in turn has developed a market of buyers who apply the solids to land.

The drying equipment itself consists of 14 units, each having a capacity to evaporate 25,000 lbs of water per hour (11,340 kg/hr) from sludge by using a blend of combustion gases and sludge vapor. Each unit consists of a sludge mixer, drying tower, flash dryer, sludge mixer air lock, two cyclone separators having double flap discharge valves, combined surge and fuel bin, dry return conveyor, dryer induced draft fan, and gas scrubber unit. Effluent water is used for cleaning gases passing through the gas scrubber.

Since 1939, the heat-drying plant had run continuously except for brief

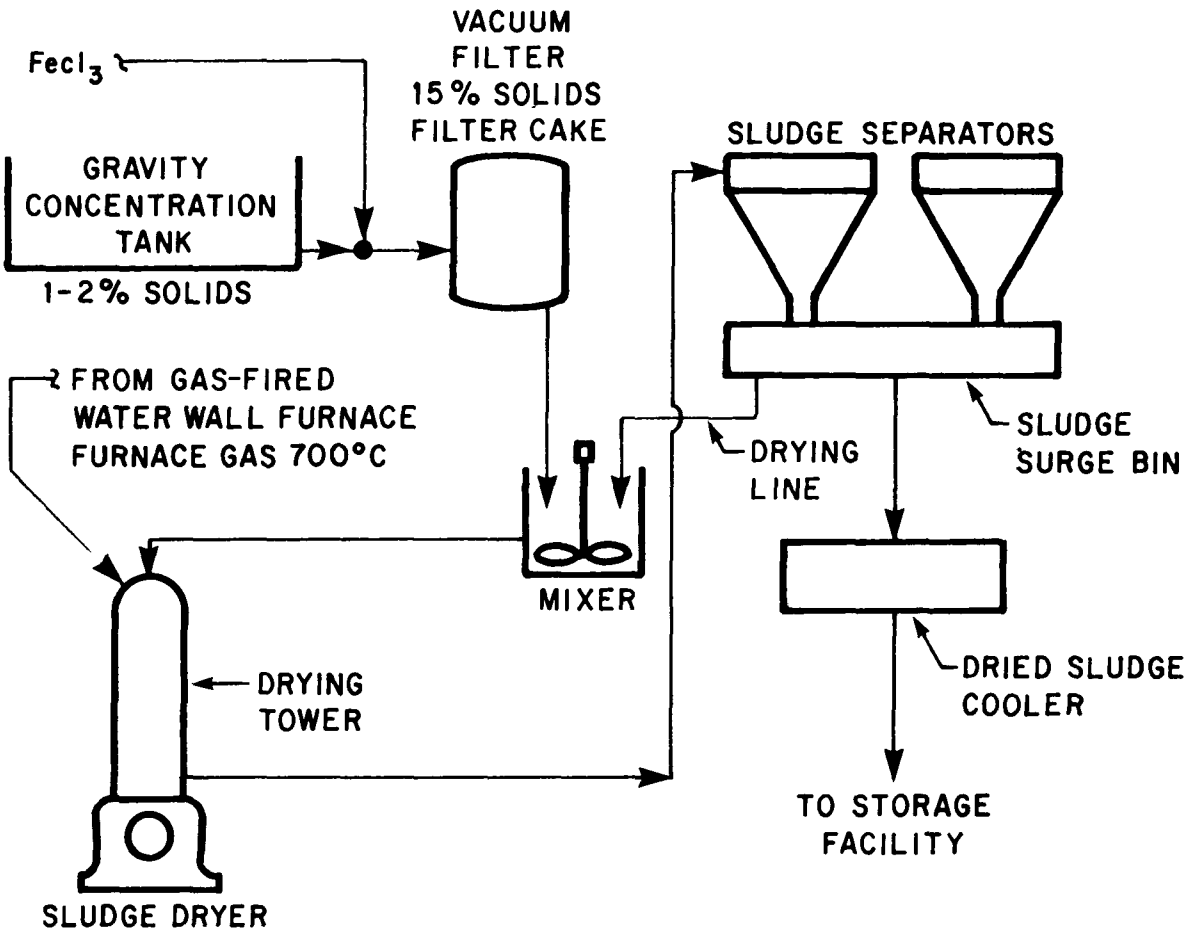


Figure 8. West-Southwest Sewage Treatment Works heat-dried fertilizer processing plant.

periods of time when the main conveyor belt needed repairs or replacement. Table 12 shows the input solids for various years since 1940.

The load on the heat-drying plant was partially reduced when anaerobic digesters were placed into operation in the 1960s and early 1970s. Even with the construction of twelve digesters, however, heat-drying continued because the average solids load in the plant (including recycle flows) exceeded the capacity of the anaerobic digesters.

To understand the operational changes made to accomplish the cessation of heat-drying without adding any new sludge processing facilities, it is necessary to review the sludge handling situation at the West-Southwest STW.

TABLE 12. SOLIDS INPUT TO THE HEAT-DRYING PLANT THROUGH THE YEARS

Year	Dry Tons/Day
1940	156
1950	163
1960	500
1970	415
1976	205
1978	165
1980	175
1981	90
1982	0

Note: Tons x 907.2 = kg

Figure 9 shows monthly averages of the suspended solids loadings to the aeration tanks coming from the West Side Imhoff tanks and the Southwest Side primary tanks from 1979 to 1982. The West Side contributed 54.3 percent of the load in 1979 and 60.9 percent in 1980. Furthermore, the West Side Imhoff tanks spilled over great amounts of solids in the spring when they became overloaded and could not be emptied because the winter cover of sludge on the sand drying beds had not yet been dug. As a result, the recycles occurred in the plant, raising the reported waste-activated sludge quantities. At the same time the input solids to heat-drying and digestion in March thru June went up 20 percent over the annual figure, and the plant became sludge-bound at times. Ironically, at other times of the year, excess sludge handling capacity existed in the plant.

To break this cycle, it was decided that ending the practice of sending Southwest primary solids to the West Side Imhoff plant year-round would end the overloading of the Imhoff tanks and would dampen the spring solids peaking. Further, it was felt that the heat-drying operation itself was contributing to in-plant recycles via the gas scrubbers, and that a reduction in overall reported solids would occur once heat-drying ceased. This phenomenon had occurred in July and August of 1980 (Figure 9) when repairs to the main fertilizer conveyor prompted a one-week shutdown. The shutdown was extended when it was observed that apparent plant solids had decreased markedly while the heat-drying plant was out of service. Beginning in July of 1981, Southwest primary sludge was sent to the digesters along with thickened waste-activated sludge; and on August 4, 1981, the heat-drying plant was shut down. As Figure 9 shows, the aeration tank loadings immediately changed; and now more solids were coming from the Southwest plant than from West Side. In the spring of 1982, the usual overflow of solids from West Side was greatly dampened; and although the solids loading to the digesters increased to

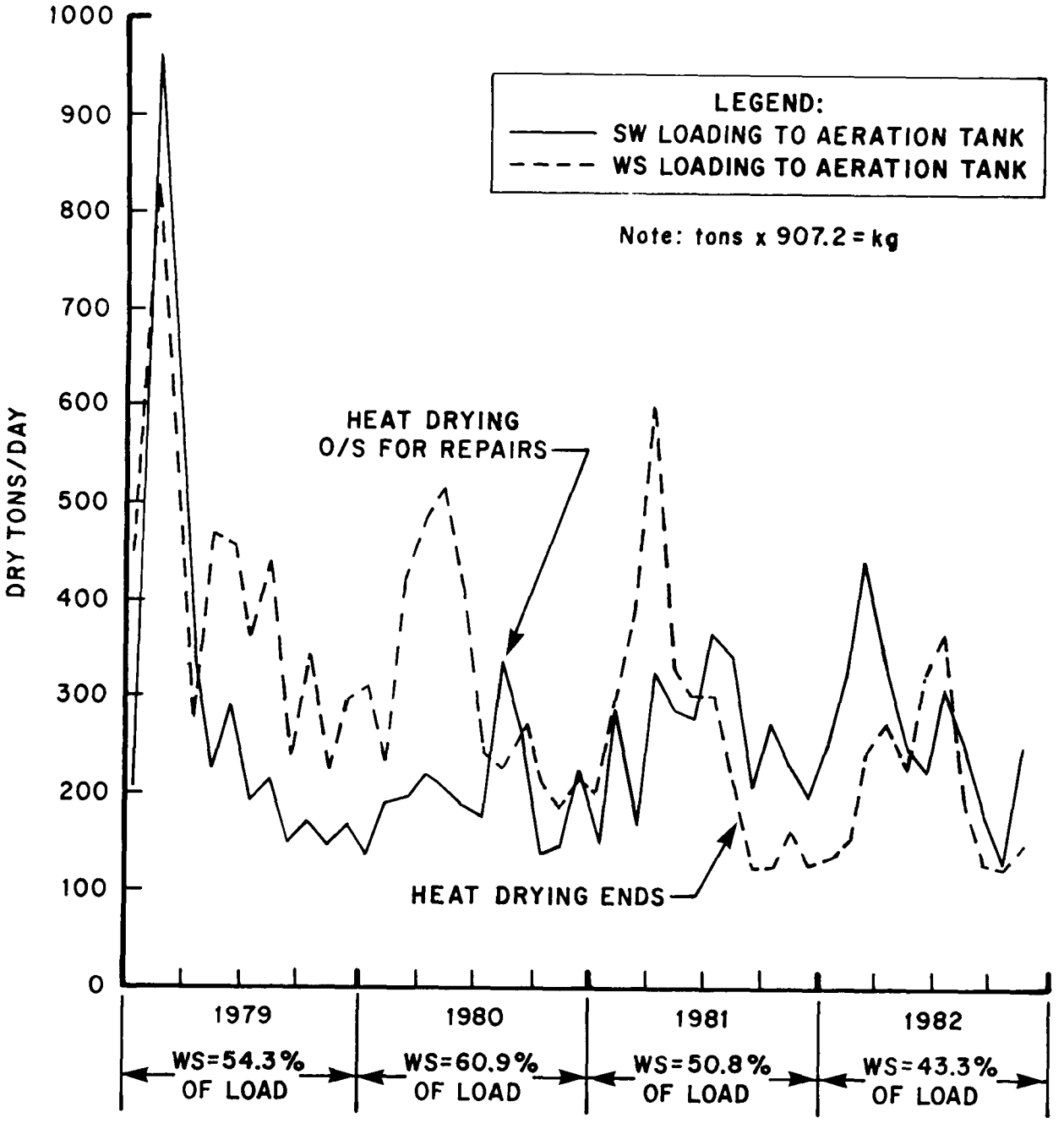


Figure 9. Aeration tank solids loading from West Side (WS) Imhoff tanks and Southwest Side (SW) Primary tanks.

capacity, there was no need to heat-dry.

The reduction in plant solids recycle has enabled the existing digester facilities to handle all solids at the West-Southwest STW since August 1981; and the heat-drying plant, the former mainstay for processing waste-activated sludge, serves as a standby process.

Table 13 shows the changes in sludge production and processing at the West-Southwest STW from 1976 to 1982, and the yearly natural gas usage for the heat-drying system. The table shows that as the solids input to the heat-drying plant was reduced beginning in 1981, the amount of sludge production also was reduced, resulting in the anaerobic digesters having sufficient capacity to handle the entire sludge production. In addition, natural gas usage for heat-drying has been reduced from over 20 million therms/yr (2.11×10^{12} kJ/yr) to zero. The elimination in August of 1981, of the heat drying of sludge has resulted in a large savings in energy and money. Based on average 1982 gas costs of \$0.35/therm (\$3.32/10^6 kJ), it is estimated that it would have cost the District approximately \$7.4 million in 1982 to heat-dry sludge at the input rate of 175 dry tons of solids per day (158,760 kg/day) which occurred in 1980, the last full year of heat-drying operation. This is a major savings, since the total 1982 gas bill at the West-Southwest STW was \$12.7 million.

TABLE 13. SOLIDS PRODUCTION, PROCESSING, AND NATURAL GAS USAGE AT THE WEST-SOUTHWEST SEWAGE TREATMENT WORKS (YEARLY AVERAGES)

Year	Total Sludge* (Dry Tons/Day)	Dry Tons/Day To Heat Drying	Dry Tons/Day to Digesters	Gas Usage for Heat Drying (Therms/Yr)
1976	497	205	292	25,495,000
1977	514	210	314	21,950,000
1978	494	165	329	17,205,000
1979	523	236	287	24,665,000
1980	517	175	342	21,161,000
1981	461	90	371	8,159,000
1982	417	0	417	0

*Excluding Imhoff tank sludge.
Note: Tons x 907.2 = kg; Therms x 105,500 = kJ.

Reduction of Air to the Aeration Tanks at the West-Southwest Sewage Treatment Works

The West-Southwest STW is a conventional air-activated sludge treatment facility consisting of four similar batteries of aeration tanks. Each battery is composed of eight tanks, with each tank consisting of four channels in series. The channels are 434 feet (132 m) long, 34 feet (10.4 m) wide,

and 15 feet (4.6 m) deep. Porous plate diffusers located along one side of each channel produce a spiral-type flow pattern. The diffuser plates are 80-permeability.

Compressed air is supplied by seven centrifugal blowers having a total capacity to compress 780,000 cfm (368 m^3/s) of free air to 8.5 psi (58.6 kN/m^2). The blowers are driven by steam turbines, with the steam being produced by seven gas-fired boilers. The boilers are two-drum, vertical, bent-tube type with superheater, and will deliver from 70,000 to 110,000 lbs (31,752 to 49,896 kg) of steam per hour each at 425 psi (2,928 kN/m^2).

The energy cost to power the blowers is one of the largest operating costs of the treatment process. In 1979, a total of 79.4 million therms of natural gas was used at the West-Southwest STW (Note: 1 therm = 100,000 Btu = 105,500 kJ). The average cost of natural gas in 1979 was $0.25/therm ($2.37/10^6 kJ), resulting in a total gas cost of $19.8 million. Of this amount, approximately $7.9 million, or 40 percent of the total, was spent to produce steam to power the air blowers.

Keeping the rapidly rising natural gas costs experienced in the late 1970s in mind, a program was begun in 1979 to determine whether the amount of air supplied to the aeration tanks could be reduced without having a detrimental effect on the excellent effluent quality which the West-Southwest STW was then achieving. Of special interest, was the ability of the facility to maintain year-round nitrification, which although not a current permit requirement, could be a requirement in the future. The major operational change which took place was to modify the distribution of air to the four-channel aeration tanks. Prior to 1979, the air was distributed fairly uniformly to each channel resulting in low dissolved oxygen levels at the head end of the aeration tanks. In 1979, a tapered aeration approach was implemented, in which proportionately more air was supplied to the first channel of each aeration tank, while less air was used in the remaining three channels. By doing this, an overall reduction of approximately ten percent in air usage was achieved while nitrification efficiency was actually increased.

Table 14 presents yearly averages of final effluent BOD, suspended solids (SS) and NH_4-N for the West-Southwest STW for 1977 to 1981. As can be seen, final effluent BOD has stayed in a 5 to 7 mg/L range and final effluent SS in a 6 to 8 mg/L range. Final effluent NH_4-N has decreased from an average of 2.4 mg/L in 1979 to 1.4 mg/L in 1981.

TABLE 14. YEARLY AVERAGES OF FINAL EFFLUENT BOD, SUSPENDED SOLIDS, AND NH_4-N AT THE WEST-SOUTHWEST SEWAGE TREATMENT WORKS

	Final Effluent		
Year	BOD (mg/L)	SS (mg/L)	NH4-N (mg/L)
1977	5	7	2.7
1978	6	8	1.8
1979	7	8	2.4
1980	6	6	1.6
1981	6	6	1.4

Table 15 presents yearly averages of air and steam usage for 1977 to 1981. As can be seen, steam usage in the blowers has been reduced from an average of 4.5 million lbs/day (2.0 million kg/day) in the 1977 to 1978 period to an average of 3.9 million lbs/day (1.8 million kg/day) in the 1980 to 1981 period. In 1981, the energy cost for producing steam was approximately $5.00/1,000 lbs steam produced ($11.00/1,000 kg). Thus, the average energy savings in reduced steam usage has been approximately $3,000/day or $1,095,000 /year.

TABLE 15. YEARLY AVERAGES OF AIR AND STEAM USAGE AT THE WEST-SOUTHWEST SEWAGE TREATMENT WORKS

Year	Sewage Flow (MGD)	Total Air Usage* (10^6 ft.^{3}/Day)	Steam Usage in Blowers (10^6 lbs./Day)
1977	795	745	4.48
1978	819	735	4.52
1979	873	690	4.32
1980	808	665	4.09
1981	807	677	3.80

*Approximately 85% of the air used goes to the aeration tanks. The remaining 15% is used for the return sludge airlift pumps and the aerated grit chambers.
Note: MGD x 0.0438 = m^3/s; ft.^{3} x 0.028 = m^3; lbs x 0.454 = kg.

Total Cost Savings at the West-Southwest STW

Table 16 presents a summary of the cost savings achieved at the West-Southwest STW since the late 1970s through process modifications and energy conservation measures. As can be seen, these savings amount to over $14 million/year, indicating the magnitude of the savings which can be achieved at a major facility through a careful analysis of operating procedures.

CONCLUSIONS

In a time of rapidly changing energy prices, it is prudent to critically review the design basis and assumptions that entered into the engineering development of existing facilities. In many cases, current economic conditions may dictate that substantial modifications be made to processes or operations that are otherwise technologically advanced. A realization of a changing economic environment caused the District to review all its facilities, to prepare an inventory of its energy uses, to initiate a procedure to focus in those areas where energy conservation would most probably be fruitful, and, finally, to implement a plan that would reduce energy consumption and related costs.

The work described in this paper was not funded by the U.S. Environmental Protection Agency, and therefore, the contents do not necessarily reflect the views of the Agency and no official endorsement should be inferred.

TABLE 16. COST SAVINGS ASSOCIATED WITH ENERGY CONSERVATION AND PROCESS MODIFICATIONS AT THE WEST-SOUTHWEST SEWAGE TREATMENT WORKS

Item		$ Savings/Year	
1. Convert from steam driven turbines to purchased electricity			$5,900,000
2. Digester gas utilization			
	a. Amortized capital cost*	- 86,000	
	b. O & M cost	-150,000	
	c. Energy production	+593,000	
Net Savings			357,000
3. Elimination of heat-drying			
	a. Energy savings	7,400,000	
	b. O & M cost savings	1,500,000	
	c. Lost profit from fertilizer sale	-1,469,000	
Net Savings			7,431,000
4. Savings due to reduction of air to the aeration tanks			1,095,000
5. Total savings at the West-Southwest STW			14,783,000

*Term = 20 years; Interest = 7.625%.

GLOSSARY OF TERMS

A/S = activated sludge

Btu = British thermal units

cfm = cubic feet per minute

cu ft = cubic feet

d-c = direct current

ft^3/gal = cubic feet per gallon

kJ = kilojoules

kN = kilonewtons

kW = kilowatts

KWH = Kilowatt-hours

lbs = pounds

mA = milli-amperes

MCFD = million cubic feet per day

psi = pounds per square inch

SS = suspended solids

W/AS = waste-activated sludge

PROGRESS IN SEQUENCING BATCH REACTOR TECHNOLOGY

by

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This paper has been reviewed in accordance with the U.S. Environmental Protection Agency's peer and administrative review policies and approved for presentation and publication.

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ABSTRACT

Sequencing batch reactor (SBR) technology is being implemented at various municipal sites in both the United States and abroad. Total life-cycle cost savings, ease of operation and reliability favor this technology at facilities sized up to 19,000 m^3 per day (5 mgd).

Current research investigations on SBR's concern controlling sludge settleability, low energy options, and biological phosphorus removal.

This paper has been reviewed in accordance with the U.S. Environmental Protection Agency's peer and administrative review policies and approved for presentation and publication.

INTRODUCTION

This paper will discuss progress of municipal implementation and laboratory research on batch reactors since the October 1981 United States/Japan Conference on Sewage Treatment Technology. In the United States and several other countries sequencing batch reactors (SBR) are being designed and constructed for municipal facilities to meet a variety of effluent requirements. Laboratory studies are being conducted to seek operational patterns to achieve nitrification, denitrification, and biological phosphorus removal.

As a refresher, since the last presentation on the SBR process, Figure 1 provides an elevation view of an SBR reactor operating in a secondary treatment sequence (1).

Because sequencing batch reactor technology is new to both designers and treatment plant operators, the major applications are currently for facilities of 19,000 m^3 per day (5 mgd), or less. This lack of experience on large size facilities is no barrier to implementation. Between the years 1983 and 2000, it is anticipated that 3,000 municipal facilities with design flows less than 7,500 m^3 per day (2 mgd) will need to be constructed in the United States. About 2.5 billion dollars will be expended on these sized treatment plants.

MUNICIPAL IMPLEMENTATION

The following list shows the locations where seqeuncing batch reactors are being constructed to provide municipal wastewater treatment.

Location	Size, m^3/d (mgd)
Poolesville, Maryland	2,200 (0.6)
Grundy Center, Iowa	3,200 (0.8)
Sabula, Iowa	1,900 (0.5)
LeClaire, Iowa	1,900 (0.5)
Union City, Tennessee	12,900 (3.4)
Jenuau, Alaska	19,000 (5.0)
Teullahoma, Tennessee	19,000 (5.0)
LaPaz, Mexico	200 (0.06)
Puerto Escondido, Mexico	1,000 (0.3)
Staten Island, New York	1,900 (0.5)
Taupo County, New Zealand	County code for all new facilities

The Grundy Center, Iowa SBR selection saved 18 percent construction cost compared to the least costly conventional alternative, and a savings of 9 percent in electrical cost. The Sabula, Iowa facility plan showed a 22 percent energy savings, and a 15 percent life-cycle cost savings over conventional activated sludge. Both facilities received an extra 10 percent federal share of construction funds under the Innovative and Alternative provisions of Public Law 95-217.

Taupo County, New Zealand has provided for SBR selection in all new plant construction for several reasons (2). Chief among these reasons was the need to provide economical treatment for nitrogen removal to protect Lake Taupo from eutrophication. The County Engineer evaluated several options for capital and operating costs and arrived at the following ranking for the Acacia Bay portion of Lake Taupo:

Acacia Bay, New Zealand, 2280 m^3/d (0.6 mgd)		
Process Option	Total Capital Cost	Annual Operating Cost
Oxidation Pond	$ 624,000	$ 9,800
Sequencing Batch Reactor	$ 634,900	$23,400
Activated Sludge	$ 746,800	$28,100
Biological Filtration	$ 755,000	$17,900
Rotating Biological Contactor	$ 956,000	$17,700
Oxidation Channel	$1,006,000	$27,000

The decision analysis showed that the oxidation pond, activated sludge, biological filter and rotating biological contactor processes could not be easily managed for nitrogen removal; and all but the oxidation pond were more capital expensive than the SBR. While the oxidation channel could be operated for nitrogen removal, the capital cost was prohibitive. Even though the analysis showed the SBR operating cost to be greater than three of the options, on a cost per kilogram of nitrogen removal, the SBR was most cost effective.

Whang and Hao developed computer simulation models for the process design of the Poolesville, Maryland SBR. Software developed uses an iterative process, from input data, to size reactors, give temporal concentrations of substrate and active biomass, and oxygen utilization rates (3).

OPERATIONAL CONSIDERATIONS

Since the majority of the SBR's being constructed are of small size a recent discussion of energy utilization at municipal facilities by Smith is of interest (4). This discussion shows that in the 3,785 m^3 per day (1 mgd) size range energy consumption is only 8.1 percent of the operating cost and labor accounts for 68.2 percent of the cost. Therefore, the SBR process with automated valving, and a process controller which eliminates major labor time, is a logical selection at small facilities. In Australia, where modifications of the SBR have been employed for several years, it is common to have one operator manage four facilities in a several hundred square kilometer area. Also, designers are beginning to recognize the attributes of the SBR in many situations. Barth has noted flexibility, equalization, quiescent settling, elimination of recycle pump, and process reliability, as criteria that direct selection of batch systems for many installations (5).

Bathija, has described various SBR tank designs and operational schemes (6). Racetrack, rectangular (length to width ~ 4:1), and circular SBR facilities have been constructed. Recommended side-water depth is 3 to 4.5 m (10 to 15 ft). If both aeration and mixing are independently necessary, a

jet type aeration device is preferred. If separate mixing only is not a process requirement any non-clogging aeration system can be specified.

Bathija also suggests SBR solutions to two common problem areas in municipal treatment; reduced hydraulic flow during the early design life of a facility, and increased hydraulic flow during storm events. During the early design life period of reduced flow, liquid level sensors can be set at a fraction of tank capacity. In this way, the length of treatment cycles is the same as design, and power is not wasted by over-aeration during an excessively long cycle time. There are several approaches to manage increased hydraulic flow during storm flows. If level sensors are left in their usual position, increased flows will cause the number of cycles per day to increase. If this should cause a reduction in efficiency the level sensors could be raised, within free-board limits, to keep the cycle time near design requirements. Either of these two management approaches could be coupled with modification of the time periods within a cycle. For instance, the time devoted to IDLE could be eliminated and that time period shifted to increase REACT time, yet not alter the overall design cycle time.

The SBR process has an outstanding reliability feature during increased hydraulic flows. The mixed liquor solids always remain in the reactor and cannot be washed out by hydraulic surges. Also, there is no recycle flow from a separate clarifier and, therefore, an SBR process does not rely on return sludge capability to maintain an adequate level of mixed liquor solids during increased hydraulic flow.

EQUIPMENT SELECTION

Because the SBR process is rather new, several types of instrumentation are being pursued. Some designs are employing photoelectric level sensors, others use float level switches for controlling reactor liquid level. Automated valves for control of reactor influent and effluent range from pneumatic devices to motorized gate valves.

Effluent discharge devices can be submerged standpipes, tipping weirs, float supported collector pipes, or submberged pumps.

In all but the smallest systems, commercial process controllers are used to time and control the process sequence. In systems under 20 m^3/day (5000 gpd) simple time clock and electrical solenoid valves would be recommended.

INDUSTRIAL APPLICATION

The Springfield Creamery in Springfield, Oregon, produces a waste that has a large concentration of soluble oxygen demand, intermittent flow, and extreme pH fluctuation. Flow is 40 m^3/day (10,000 gpd). An adjoining meat packing plant has a large concentration of suspended oxygen demand, intermittent flow, and stable buffered pH of about 7. Flow is 8 m^3 per day (2000 gpd).

Upon learning of the inherent equalization of an SBR process, ease of operation, wide flexibility of treatment options, and simple design con-

siderations, the two companies plan to mix these waste streams for combined treatment. The SBR will be an earthen basin with dimensions of 7.5 m x 4.5 m x 2.4 m (25 ft x 15 ft x 8 ft). Aeration will be provided by a submerged pump with an air blower exhaust line tapped into the intake side of the pump. Since the flow is intermittent, a single tank system will be adequate.

Due to the facts that SBR's can be operated to produce a defined residual concentration of a pollutant with no possibility of short circuiting, and that quiescent settling produces very clear effluent, the process has been selected to treat a combination of landfill leachate and chemical manufacturing wastes in Niagara, New York.

The plant wastes are generated from the industrial operations and the leachate is collected from a remote landfill area via collection underdrains and hauled by truck to the plant site. Both type wastes contain several of the Consent Decree priority pollutants. Daily design flow is 600 m^3.

The existing treatment at the industrial site consists of a storage tank, neutralization tank and carbon absorption columns; with discharge of the column effluent to the city sewer. Operation has shown that with this physical/chemical process, excessive soluble organic matter quickly saturates the absorption capacity of the carbon columns requiring frequent thermal regeneration of the carbon. Upgrading construction will add two biological SBR's, to reduce the soluble organics, and provision for adding ammonia and phosphate to serve as nutrients for the SBR organisms.

Many of the organic components of the mixed waste have slow rates of biodegradation. To insure efficient degradation, the SBR contents will be monitored for total organic carbon during REACT until a predetermined concentration is achieved. Thus, the SBR will decrease the organic loading to the carbon columns and efficient settling will not overly burden the columns with suspended solids. Waste activated sludge from the SBR's will be incinerated to destroy any sorbed priority pollutants on the sludge surface.

Pilot SBR operation at a scale of 50 m^3/day (13,000 gpd) has shown a 90 percent reduction of soluble organic carbon from the influent mixed waste. It is therefore anticipated that the useful life of the carbon, before regeneration is necessary, will be increased by almost a factor of 10. Figure 2 is a schematic of the full-scale process as it will exist in 1985.

LABORATORY RESEARCH ON SBR's

Control of Sludge Bulking

Working with 4 L SBR's, fed a soluble synthetic wastewater, Chiesa has correlated sludge volume index (SVI) with aeration time during the FILL period (7). He used the following strategies to manage the substrate tension in the SBR's:

Control Strategy	Unaerated/Unmixed FILL, hr	Aerated FILL, hr	Aerated REACT, hr	SETTLE, DRAW and IDLE, hr
A	2.00	0.00	4	2
B	1.75	0.25	4	2
C	1.50	0.50	4	2
D	1.25	0.75	4	2
E	1.00	1.00	4	2
F	0.00	2.00	4	2

Very spectacular results were obtained as shown in Figure 3. To confirm this observation, an SBR operated under control strategy F, with an SVI of 600 mL/g, was switched to strategy A; and in 15 days the SVI decreased to 100 mL/g.

The researchers explain these observations on the basis of competitive growth of one group of organisms in opposition to other groups. They postulate that strategy A, where substrate tension surrounding the organisms reaches a high level during FILL, and subsequent oxidation of organics and endogenous metabolism during REACT, which places the organisms in periodic starvation, selectively encourages the growth of floc forming organisms. Filamentous organisms survive best when the substrate tension is constantly maintained at a low level such as in strategy F.

The sludge developed under strategy A had a greater specific oxygen uptake rate than did the F strategy sludge; and the A sludge was much more efficient in substrate removal rate than the F sludge.

The authors point out the great flexibility of the SBR process to control substrate tension at any desired level. It can be noted in the above strategies that A would be equivalent to contact stabilization and F would be similar to a completely mixed activated sludge process.

Unaerated SBR

Irvine has demonstrated with bench-scale SBR's that a weak domestic wastewater can be treated without major power input for aeration (8). He used 4 L SBR's that were mixed with a stirrer, with just enough energy to keep the mixed liquor in suspension, during FILL and REACT. The organic loading was 0.02 kg BOD_5 per day per kg mixed liquor solids. A 24 hour cycle consisting of FILL 9 hr, REACT 12 hr, SETTLE 2 hr, DRAW 0.5 hr, and IDLE 0.5 hr; was used.

In this lightly loaded SBR process the aeration provided by surface renewal due to stirring was sufficient for the oxygen requirements of the organisms. The authors conclude that this low energy SBR process is analogous to a wastewater lagoon system. With the added advantage that algal growth will not occur, as is normal for conventional lagoons, because the mixed liquor turbidity prevents sunlight penetration. Mixed liquor concentration was 1800 mg/l, and the 2 hour SETTLE period allowed excellent liquid solids separation.

Results for this low energy system for removal of BOD_5 are shown in Figure 4. Suspended solids removal was equally as effective and averaged 10 g/m^3 over the 60 day study period.

Biological Phosphorus Removal in an SBR

Discussions at the June 1982 "Workshop on Biological Phosphorus Removal" showed a similar process feature for three proprietary phosphorus removal systems (9). This feature is shown conceptually in Figure 5. When activated sludge mixed liquor is subjected to an anaerobic or anoxic environment* with influent organic substrate, there is coincident leaching of phosphorus from the cellular material and sorbtion of organic matter. When environmental conditions are changed to an aerobic state, phosphorus is removed from solution and organics are oxidized.

The following several theories have been proposed to explain this finding (9). Anaerobic-aerobic staging can result in selection of a biological population that is capable of storing polyphosphates. The actual mechanism for storage and release and the basis of population selection is not known.

The regulation of the adenosine diphosphate ratio to adenosine triphosphate (ADP/ATP) controlled by the reversible enzyme polyphosphate kinase has been suggested as a mechanistic factor. When cells are placed in a "stress condition" such as lack of oxygen during an anoxic period, large amounts of polyphosphates are then stored in the cells when subsequent aerobic conditions are provided and cellular growth is rapid.

Another theory suggests that polyphosphate storage ability of certain bacteria allow them to compete and survive in an alternating anoxic-aerobic environment. The energy derived by hydrolysis of polyphosphate (ATP) may be used directly for transport of substrate across the cell membrane in the anoxic condition.

Polyhydroxy butric acid (PHB) has also been suggested as an important consideration. It has been noted that PHB is formed during anoxic conditions, and organisms such as Acinetobacter can accumulate both PHB and phosphates.

These various mechanistic theories seem to show that some type conditioning occurs during the anoxic period; whether enzymatic hydrolysis, uptake of a metabolite or environmental stress that confers a competitive advantage to certain phosphate storing organisms when conditions are switched to an aerobic state.

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*The term anaerobic refers to environments which have no measurable concentrations of either oxygen or oxidized nitrogen. Often times appreciable concentrations of carbonaceous materials must be present to maintain this condition. Anoxic refers to environments which have no dissolved oxygen but have oxidized nitrogen present. However, these two terms were used almost interchangeably in the workshop discussions.

The PhoStrip process* utilizes this anaerobic or anoxic leaching to remove phosphorus from a portion of mixed liquor in a side stream stripper tank. Both the A/O** and Bardenpho*** processes remove phosphorus by baffling the biological reactor to maintain the mainstream flow environmental conditions suitable for each process' operation.

Also, all three processes were reported to be subject to inhibition of phosphorus leaching during the anaerobic or anoxic phase if nitrate ion was present. If nitrate was present, provision for denitrification before phosphorus leaching had to be provided.

All of these applications for phosphorus removal are for continuous flow systems. The Municipal Environmental Research Laboratory is employing an SBR for studying biological phosphorus removal because the batch approach should allow close control of environmental conditions. A schematic of the 4 L bench-scale reactor used for these studies is shown in Figure 6. A vast number of operational schemes can be employed using a microprocessor controlled SBR. In order to achieve the release and uptake of phosphorus noted in Figure 5 the following cycle scheme was employed:

Cycle Event	Time	Purpose	Note
FILL+	1 hr	Add substrate	No aeration
STRESS	4.5 hr	Cause phosphorus leaching and denitrification	Mixer on No aeration (Time overlaps with FILL time)
AERATE	1 hr	Cause phosphorus uptake, and oxidize organics	Aeration
WASTE SLUDGE	4 sec	Remove phosphorus from reactor, and control SRT	Aeration
SETTLE	15 min	Form clear supernatant	No aeration

*Biospherics, Inc., 4928 Wyaconda Rd., Rockville, MD 20852

**Air Products, Inc., Box 538, Allentown, PA 18105

***EIMCO-PMD, 3839 S. West Temple, Salt Lake City, UT 84115

+Primary effluent from Cincinnati's Muddy Creek treatment plant used.

Cycle Event	Time	Purpose	Note
DRAW	5 min	Discharge effluent	No aeration
IDLE	10 min	Adjustment of cycle length	No aeration

The total cycle time was 6 hours and therefore the SBR emptied and filled 4 times each day.

The results achieved are shown in Figure 7. During FILL the phosphorus concentration in the reactor is affected by the phosphorus content at the end of the last cycle event (either DRAW or IDLE), the concentration in the feed entering the reactor and the leaching of phosphorus from mixed liquor solids. In the case depicted, the residual phosphorus in the SBR was diluted as primary effluent, with a smaller concentration entered, and a gradual increase to 33 g/m^3 occurred as mixed liquor cellular leaching occurred. During the STRESS period a plateau concentration existed, until aeration was commenced, and cellular uptake reduced the concentration to 2.5 g/m^3. Effluent filtered orthophosphorus concentrations approaching 0.0 g/m^3 have occasionally been noted.

Nitrate nitrogen remained at about 1 g/m^3 during STRESS due to the occurrence of denitrification. A dramatic increase of nitrate formation occurs once aeration is commenced and dissolved oxygen increases.

The SBR was operated under this regime for 30 days (120 cycles) using the Muddy Creek primary effluent. SBR effluent residual orthophosphorus was routinely between 2.5 and 0.5 g/m^3. Influent total phosphorus varied between 20 g/m^3 and 15 g/m^3, with orthophosphorus between 20 g/m^3 and 6 g/m^3. Muddy Creek wastewater is primarily domestic waste.

The process management to control biological phosphorus removal can be illustrated by referring to Figure 7. During FILL and STRESS, conditions have to be provided to cause denitrification of any nitrate and insure that leaching from the cellular mass occurs. STRESS periods ranging from 2 to 6 hours have been studied. Three hours appears to be the minimum time that can be employed. An aeration period of at least 1 hour is necessary to reincorporate the leached phosphorus and oxidize organics. Aeration times greater than this lead to less efficiency due to lysis of cells during endogenous respiration. Wasting of sludge from the SBR must be done when the biological solids have the highest content of phosphorus. This would be at the end of AERATE, which yields a dilute waste activated sludge of the same solids content as the mixed liquor. Or, wasting can be done near the end of SETTLE, which provides a thickened waste activated sludge. Caution is required here in case low dissolved oxygen in the sludge blanket causes leaching of phosphorus. Thus, overall efficiency of biological phosphorus removal dictates that SETTLE time be as short as possible, yet provide good solids separation. A minimum of 15 minutes and a maximum of 30 minutes has been satisfactory.

Time devoted to DRAW is not critical to biological phosphorus uptake directly. Mainly, on full-scale SBR's this time period would be related to effluent weir design to insure proper hydraulics within the reactor, and hydraulics of discharge appurtenances. Provision for an IDLE period is optional in SBR operation for biological phosphorus removal. A short IDLE period, either quiescent or aerated would have no impact on overall efficiency. IDLE is simply a convenience of adjusting cycles between reactors if two SBR's are operated alternately. The same SBR process cycle described above was employed with wastewater from Cincinnati's Little Miami facility. This waste contains a major fraction of wastes from a variety of industrial processes. Consistent biological phosphorus removal could not be achieved. Over a three month period variations to FILL, STRESS, and AERATE were attempted without success. Efficient removal with residual orthophosphorus values of 0.5 to 1.0 g/m^3, would be noted for several days; then deterioration of efficiency would occur.

These SBR studies, on two wastewaters, have not revealed the controlling parameters to insure consistent, highly reliable biological phosphorus removal. This same conclusion was noted for the three continuous flow biological phosphorus removal processes discussed at the Annapolis workshop (9).

Presently it would be recommended that any facility plan for biological phosphorus removal be guided by pilot plant studies prior to final selection. Additionaly, in those cases where a high reliability of phosphorus removal is necessary, it would be recommended that a chemical addition system be installed as backup equipment.

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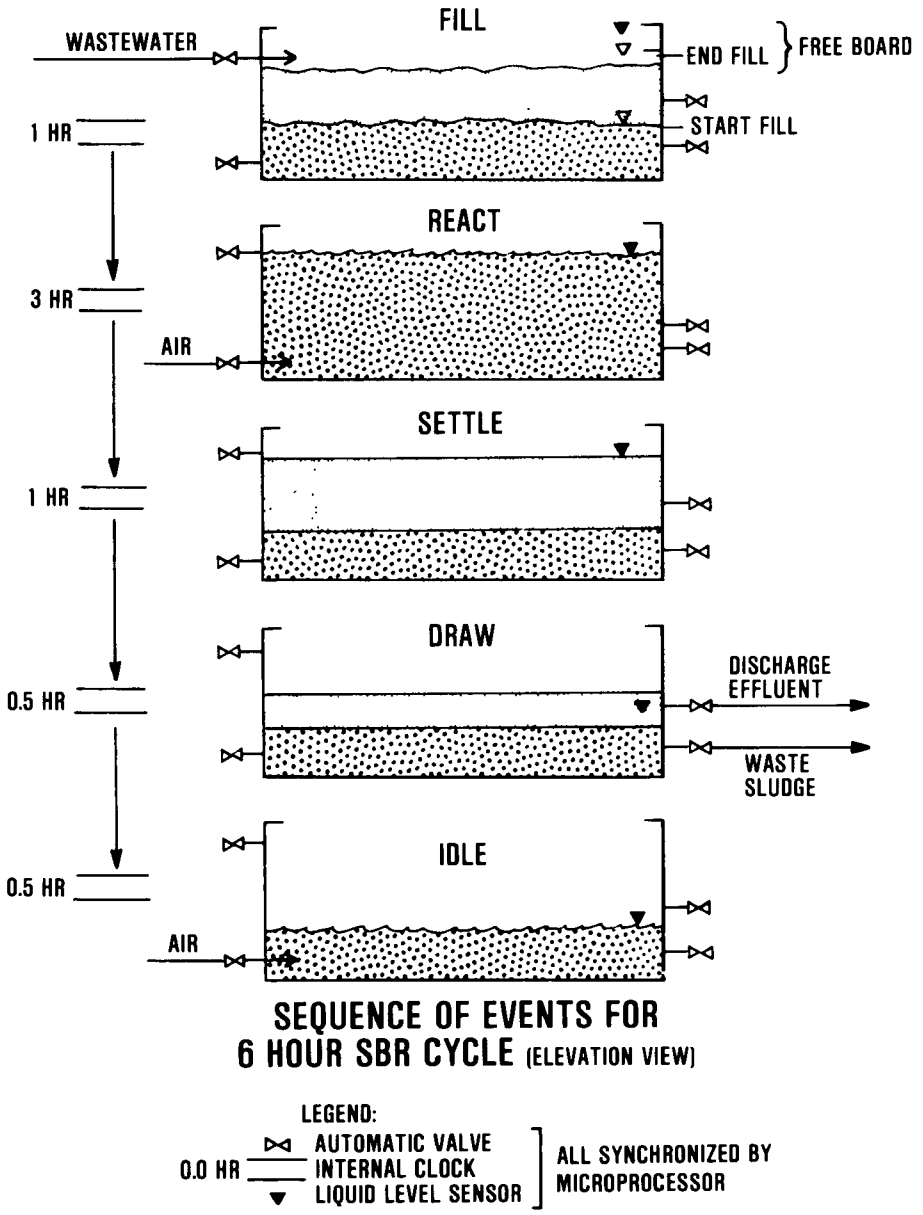

Figure 1. Sequence of events for 6 hr SBR cycle. (elevation view)

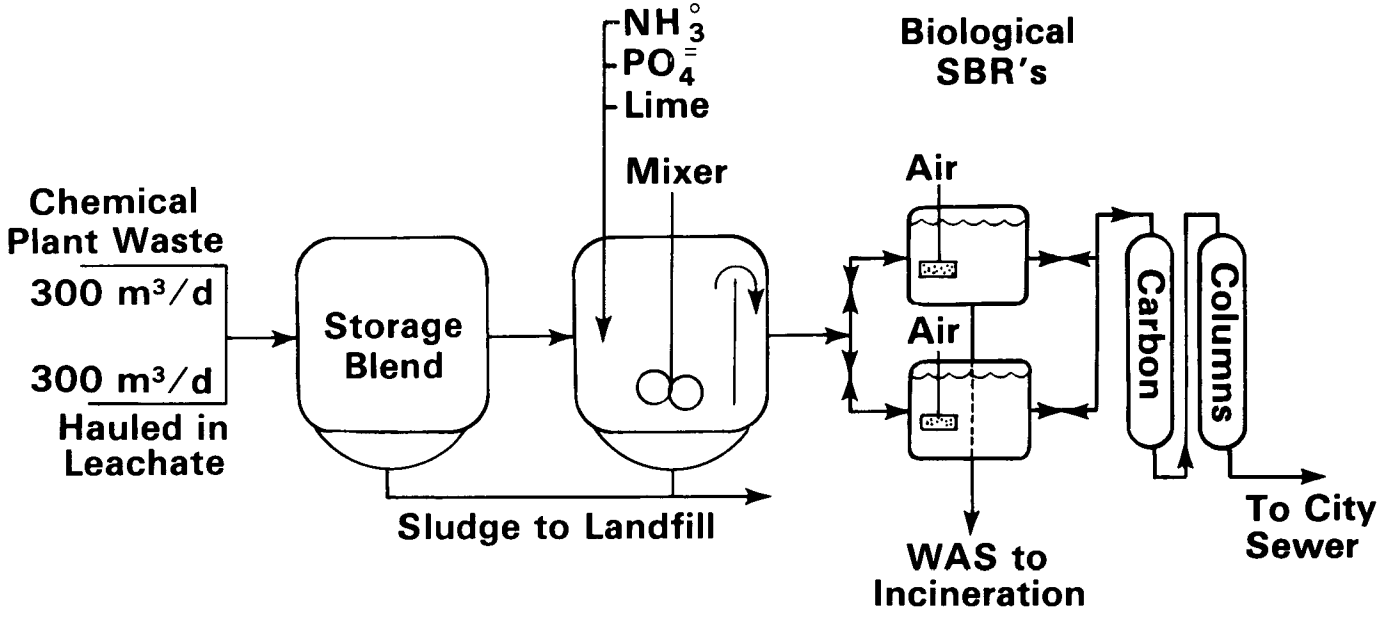

Figure 2. SBR-Carbon adsorption for industrial waste.

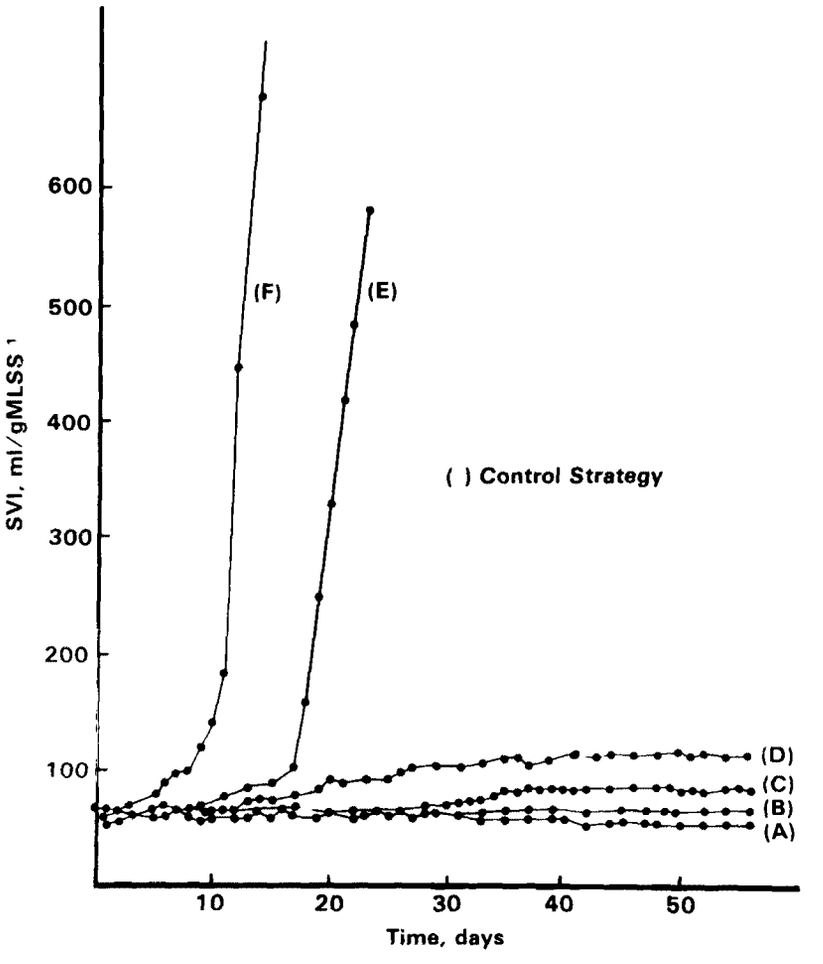

Figure 3. SVI as a function of control strategy. (From Ref. 7).

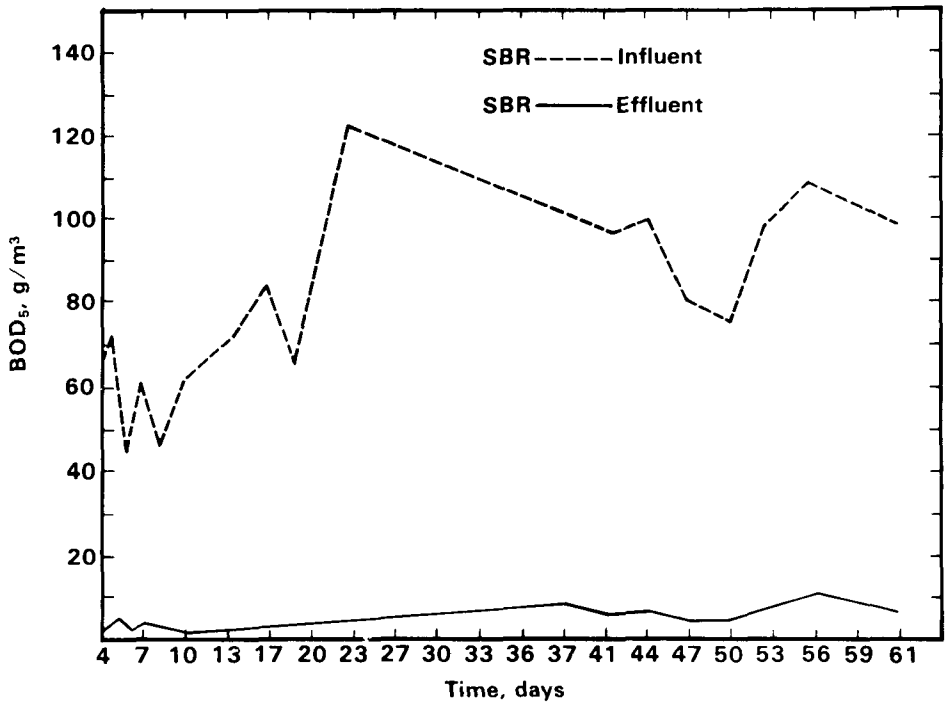

Figure 4. BOD_5 versus time for low energy SBR. (From Ref. 8).

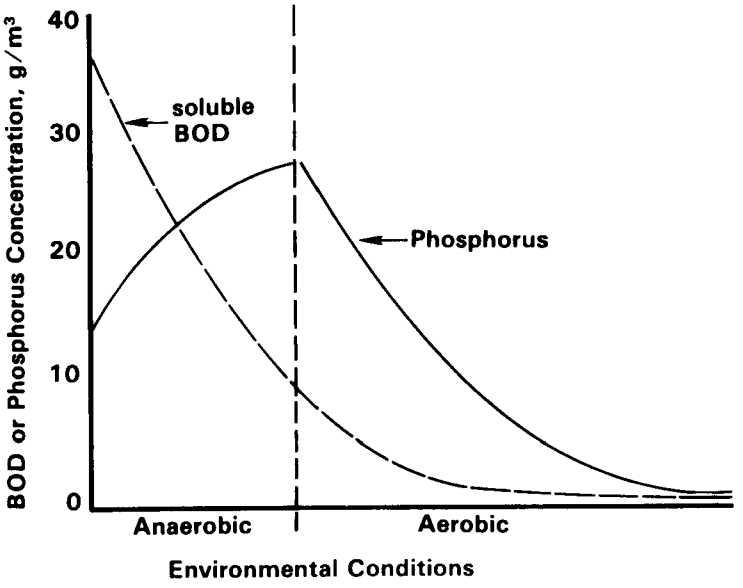

Figure 5. Biological phosphorus and BOD_5 removal due to anaerobic-aerobic contacting. (Ref. 10)

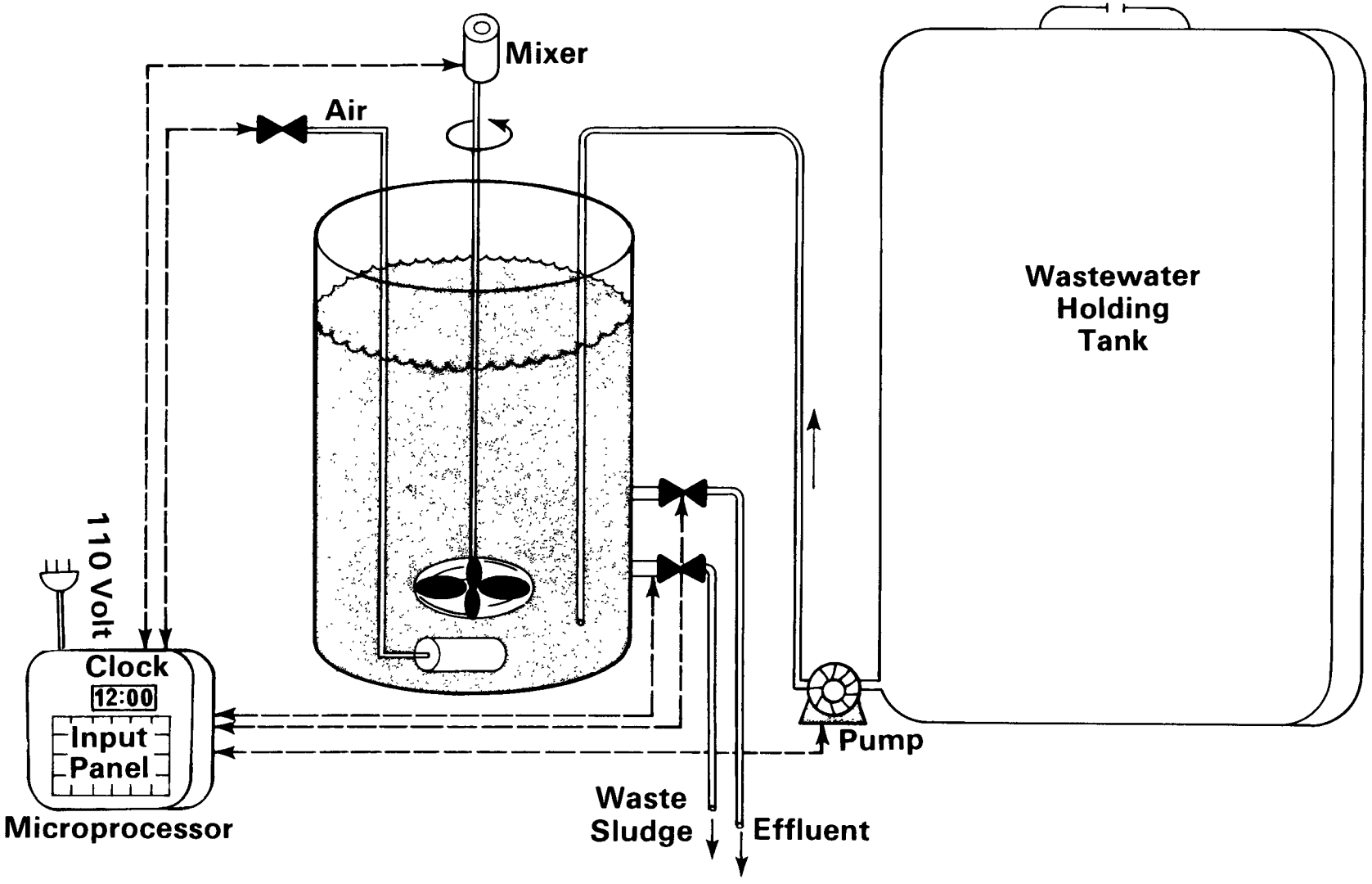

Figure 6. 4 L Bench-scale SBR.

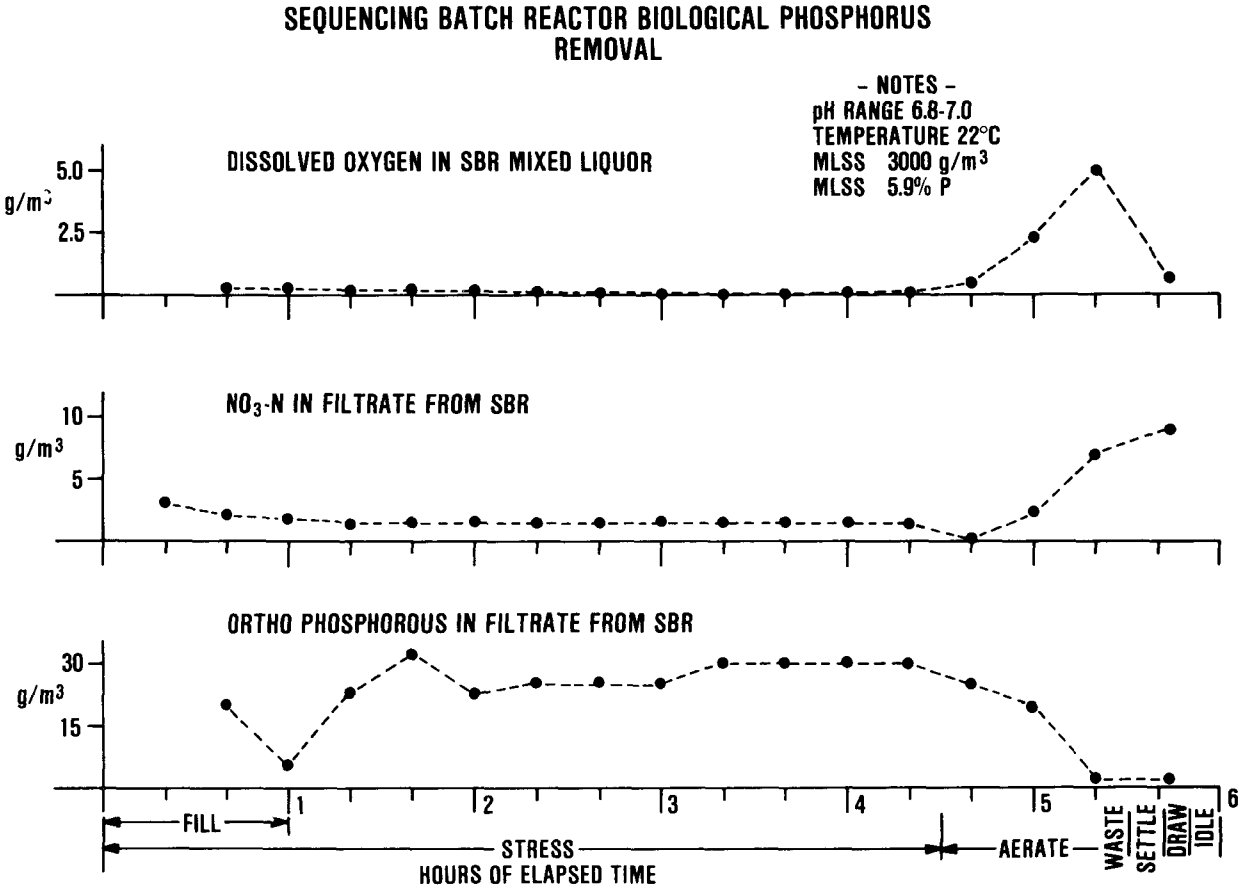

Figure 7. SBR Biological phosphorus removal.

THE BIOLOGICAL AERATED FILTER-
PROGRESS OF DEVELOPMENT IN THE U.S.

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THE BIOLOGICAL AERATED FILTER-PROGRESS OF DEVELOPMENT IN THE U.S.

INTRODUCTION

Current projections (1) indicate that, by the year 2000, 21,027 municipal wastewater treatment facilities will be operating in the United States (U.S.) to treat a flow of 161,778,000 m^3/d (42.7 bgd) (See Table 1). This compares to the 15,431 facilities presently operating. The net increase of 5596 operating facilities is based on abandonment of 1477 existing plants, 447 plants under construction (as of 1980) and 6627 new plants to be constructed by the year 2000. Therefore, more than 30% of the plants to be operating by the year 2000 are yet to be built. A total of 5832 existing facilities are to be enlarged and/or upgraded. Of the 21,027 facilities to be operating in the year 2000, more than half of the design flow to be handled will be treated to better than secondary levels. More than 80% of the total number of facilities will be for design flow capacity less than 4000 m^3/d (1.06 mgd); yet only 3% of the facilities will be treating more than 65% of the flow.

In order to address the present and future needs in these areas, the Wastewater Research Division of the U.S. EPA Municipal Environmental Research Laboratory conducts a national program to support research, development and evaluation of municipal wastewater and sludge collection, treatment and disposal technologies. Focus is on solutions that reduce capital and operating costs, use energy and resources more efficiently and improve operational reliability. Presently, particular program interest is directed to problems of small communities, operation and maintenance, technology suitable for retrofit and testing and engineering evaluation of conventional as well as "innovative and alternative" technologies.

As a result of these efforts, the range of available solutions will be widened to address: continuing changes in cost and energy relationships experienced in the economy; changes in water quality issues; and changes in affordability and funding scenarios-particulary as they relate to the recently passed "Municipal Wastewater Treatment Construction Grant Amendments of 1981" (PL97-117, 12/29/81).

THE BIOLOGICAL AERATED FILTER

As reported previously (2), a promising new technology development for wastewater treatment appears to be the biological aerated filter (BAF). The proprietary BAF process was developed in France by Omnium de Traitments et Valorisation (OTV), a company involved in engineering, equipment fabrication, and construction of wastewater treatment facilities. The BAF process is presently marketed in the U.S. by the Eimco

TABLE 1

NUMBER OF PUBLICLY OWNED MUNICIPAL WASTEWATER TREATMENT FACILITIES IN THE U.S. ACCORDING TO SIZE RANGE IN 1980 AND PROJECTED FOR THE YEAR 2000*

	FLOW (MGD) (0-.105)	(.106-1.05)	(1.06-10.5)	(10.57-50.2)	(50.2+)	Total
1980 Facilities						
Count	5120	7031	2760	421	99	15,431
Total Flow	261.8	2671.6	9114.9	9,333.4	13,889.0	32,270.7
% Flow	.7	7.5	25.8	26.4	39.3	100
% Facilities	33.2	45.5	17.9	2.7	0.7	100
2000 Facilities						
Count	8701	8460	3213	524	129	21,027
Total Flow	437.5	3063.7	10,737.9	11,211.4	17,290.9	42,741.4
% Flow	1	7.1	25.1	26.2	40.4	100
% Facilities	41.4	40.2	15.3	2.5	0.6	100
Change						
Count	3581	1429	453	103	30	5596
% of total	64.0	25.5	8.2	1.8	0.5	
Flow	175.7	392.1	1623.0	1878	3401.9	7470.7
% of total	2.4	5.2	21.7	25.1	45.6	
% incr. count	69.9	20.3	16.4	24.5	30.3	36.3
% incr. flow	67.1	14.7	17.8	20.1	24.5	21.2

* 1982 Needs Survey

Process Equipment Company, Salt Lake City, Utah, under an exclusive licensing arrangement.

Research interest in the U.S. is due to apparent simplicity of operation, reduced reactor volume and space requirements, positive effluent solids control eliminating the need for a secondary clarifier, associated sludge control problems with activated sludge processes, competitive power requirements and flexibility in treatment capability as well as size range applicability. An attractive feature of the BAF is the reduced land area requirement, which is about 20% of that for a plastic media trickling filter and secondary clarifier and 10% of that for conventional activated sludge systems.

GENERAL BAF DESCRIPTION

The BAF is a high rate, fixed film biological/solids filter. Figure 1 presents a process flow elevation view of the BAF.

In the BAF, primary effluent, which is free of coarse and flotable materials, is introduced to the top of the filter unit and flows by gravity downward through a packed bed about 1.5 m to 1.8 m deep (5.0 to 6.5 ft deep) of granular, porous media, which is a 3 to 6 mm kiln fired clay mined in the U.S. and which costs about $0.22/kg ($0.10/lb). Filter media loss is reported as 2% a year, and media life is expected to exceed 20 years. Free oil and grease must be reduced below about 20 mg/l to avoid media plugging. A high biomass concentration is established on the media, reportedly equivalent to 15,000 to 20,000 mg/l of mixed liquor volatile suspended solids (MLVSS). This is attributed to a high media specific surface area of 1312 to 1640 m^2/m^3 (400 to 500 ft^2/ft^3). Oxygen for biological growth is supplied by sparging compressed air directly into the packed media bed about 0.3 m to 0.5 m (1.0 to 1.5 ft) above the underdrains through a PVC pipe distribution system. The system is designed to ensure uniform air distribution across the bed. The air moves upward, countercurrent to the wastewater, but not at a sufficient velocity to expand the bed. Aeration continuously loosens the filter bed to allow penetration of solids, avoid plugging and make maximum use of the filter depth. Solids filtration and BOD absorption/oxidation take place throughout the volume of the filter, but the bed volume below the air sparger is undisturbed and serves as a polishing zone for suspended solids (SS) removal. The dissolved oxygen (DO) of the BAF effluent is reported to be about 0.5 mg/l. The BAF can potentially be used to achieve BOD and SS removal as well as nitrify without the use of final clarification.

The air bubbles are 1 to 3 mm in diameter compared to the 3 to 6 mm media size. As a result, contact time for the air bubble is extended due to the circuitous path that must be followed through the media and

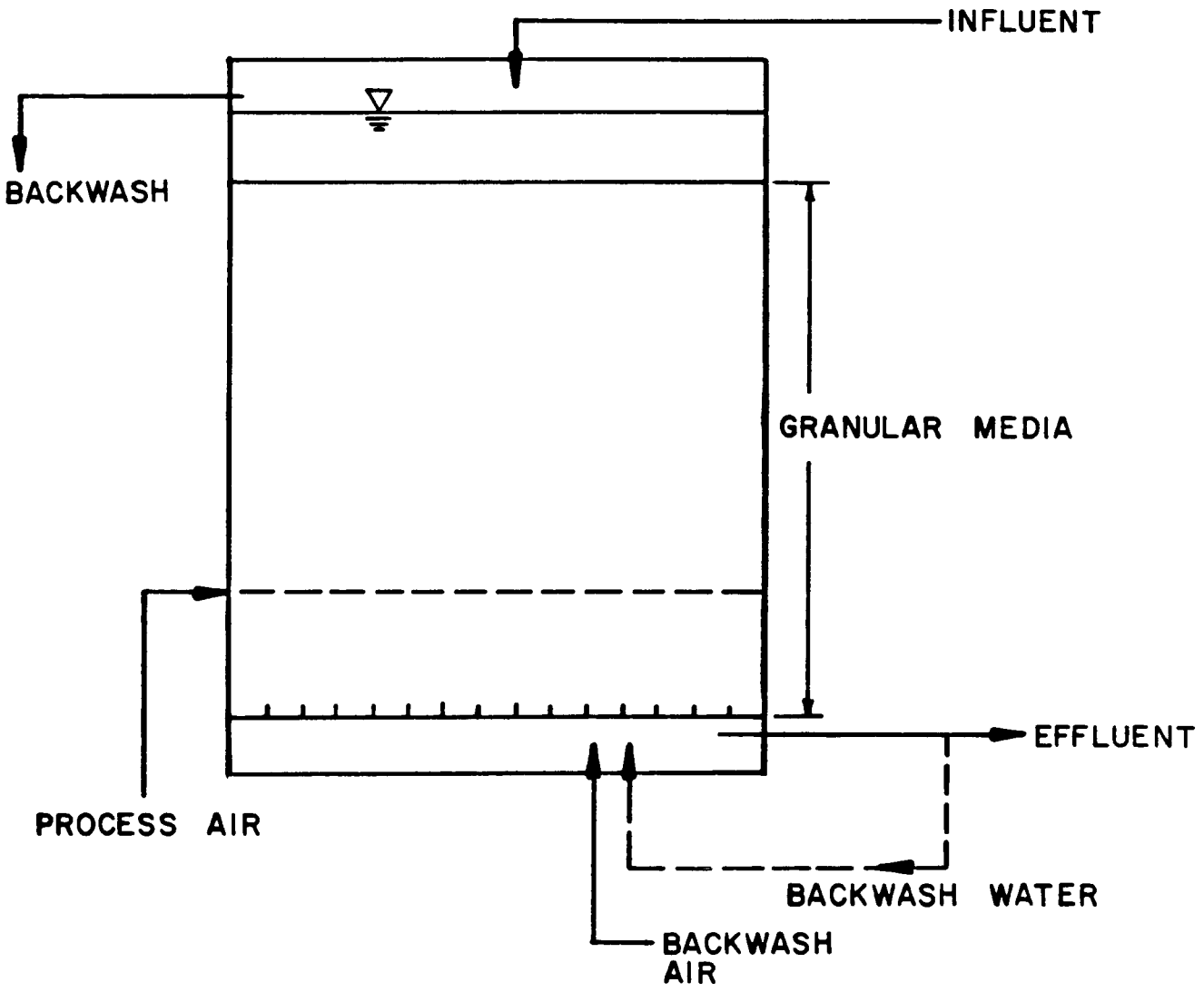

FIGURE 1 (6)

BIOLOGICAL AERATED FILTER SYSTEM

biofilm. Because of the low operating head of the unit, the manufacturer claims a relatively low blower discharge pressure of about 4.5 to 4.7 psi, offering the possibility of a low energy unit operation.

The BAF will typically be constructed above grade and requires approximately 3.5 m to 3.7 m (11.5 to 12.0 ft) of head to enter the module. Pumping may be required to feed the BAF. About 2.4 m (8 ft) of head, however, can be recovered for downstream units such as cascade aeration.

The total hydraulic residence time (HRT) in the packed bed will generally range from 30 to 80 minutes dependent upon influent strength and effluent required. Ninety-percent BOD_5 removals reportedly can be achieved at HRT's from 40 to 60 minutes at loadings of 3.2 to 5.0 kg $BOD_5/ m^3/d$ (200 to 310 lb $BOD_5/1000 ft^3/d$).

Accumulation of biological growth and trapped biological solids causes the head above the media bed to gradually increase from about 0.15 m (0.5 ft) under clean conditions to 0.61 m (2 ft) when a level sensor (timer control can also be used) will initiate a periodic backwashing procedure using water wash and air scour. Treated effluent is used for the water wash at a rate of 0.65 m/min (16 gpm/ft^2). Air is introduced to the bottom of the bed at a rate of 0.9 m/min (3 scfm/ft^2) to scour excess solids from the media. It is claimed that from 4 to 6 complete cycles are normally necessary during backwash. The backwash water is removed by a siphon located above the bed and returned to the primary clarifiers. The settling characteristics are similar to trickling filter slough with a sludge volume index (SVI) of 40 to 60. The estimated sludge production is 0.4 to 0.7 kg/kg BOD removed. The total volume of backwash water normally required is claimed to be about 2 to 2.5 times the bed volume or 5% of plant flow. The frequency of backwash required is claimed to be 15 to 24 hr for BOD removal units achieving secondary treatment and once every 24 hr for nitrification units. A microbial slime layer remains on the media to resume treatment. One backwash system can reportedly accomodate up to 20 BAF module units.

The BAF is presently made available through Eimco in the U.S. in the form of modules which are pre-fabricated, shop assembled units including air distribution piping, underdrain nozzles, backwash system, media and controls. Each module is made of steel and consists of a series of cells. A typical module is 3.7m x 12.8m x 3.7m (12ft x 42ft x 12ft) high and is 46.5 m^2 (500 ft^2) in cross section and contains a volume of 171.3 m^3 (6048 ft^3). Operation of the BAF is automatically controlled by use of a programmable controller.

FRENCH DEVELOPMENT OF THE BAF

As reported by OTV and Eimco (3)(4)(5)(6), OTV first piloted the

BAF concept at the Paris, France, Columbes treatment plant in 1975 for both BOD removal and nitrification. Both granular activated carbon and clay media (mined in France) were used in columns 0.11 m (4.4 in.) diameter ranging from 1.6 to 3.0 m (5.2 ft to 9.8 ft) deep. Selection of the media appropriate for such a filter would consider: the need for a high surface area with a high pore volume to maximize bacteria population, particle size, particle shape and density for hydraulic and air distribution headloss considerations, resistance to abrasion during backwashing, availability, and cost. Hydraulic application rates in the pilot work were varied from 0.5 to 5.0 m/hr (0.2 to 2 gpm/ft^2) of primary effluent. The COD, BOD_5, SS and NH_3-N of the primary feed averaged 200 mg/l, 85 mg/l, 75 mg/l and 22 mg/l, respectively. The organic loading rate varied from 2.5 to 15 kg COD/m^3/d (66 to 400 lb BOD_5 /1000 ft^3/d), and liquid retention times based on total bed volume ranged from 19 to 115 minutes. Reported BOD_5 effluent concentrations averaged less than 10 mg/l for loadings less than 5.1 kg BOD_5/m^3/d (318 lb BOD_5/1000 ft^3/d) (Figure 2). Effluent suspended solids averaged less than 5 mg/l for loadings over the full range. Further, the pilot study indicated that the clay and carbon media performed similarly (Figure 3). The primary effluent was increased in strength with beef extract in order to study a stronger wastewater characterized with average total COD, BOD_5, SS and NH_3-N as follows: 475 mg/l, 280 mg/l, 75 mg/l and 50 mg/l. As indicated on Figure 4, BAF treatment of the higher strength wastewater resulted in only a marginal increase in effluent BOD_5 at the same loadings.

At Colombes, France, while operating in a nitrification mode at 15°C at an organic loading rate of 2.2 kg BOD_5/m^3/d (140 lb BOD_5/1000 ft^3/d), BOD_5 was reduced from 110 mg/l to less than 10 mg/l and NH_3-N from 20 mg/l to less than 1.0 mg/l. For BOD_5 loadings less than 5.1 kg BOD_5/m^3/d (318 lb BOD_5/1000 ft^3/d), effluent SS averaged less than 5 mg/l.

A full-scale, 1893-m^3/d (0.5-mgd) system was built in LeHavre, France, and began operating in October 1978. The system consisted of four cells with total bed surface area of 30.2 m^2 (325 ft^2) to treat a primary effluent to provide advanced secondary industrial recycle water. A media depth of 2.0 m (6.6 ft) is used. Loadings during an August-September 1980 survey were 3.5 kg BOD_5/m^3/d (220 lb BOD_5/1000 ft^3/d) and 1.8 m/hr (0.83 gpm/ft^2). At 1514 m^3/d (0.4 mgd), BOD_5 was reportedly reduced from 150 mg/l to 16 mg/l and SS from 130 mg/l tO 7.5 mg/l. Wastewater temperature was 15°C.

While operating as a second-stage nitrification mode, BOD_5 was reported to be reduced from 25 mg/l to less than 10 mg/l and NH_3-N reduced from 20 mg/l to 3 mg/l at a loading rate of of 641 kg NH_3-N/m^3/d (40 lb NH_3-N/ 1000ft^3/d) and a temperature of 15°C. The LeHavre facility has also been reported to reduce NH_3-N to 1.0 mg/l. The EPA also has access to some raw as opposed to summary data from the LeHavre facility treating a wastewater with a strong industrial component including high organic nitrogen concentrations. Two cells were used for a nitrification test from October 1978 to March 1979 indicating that the units

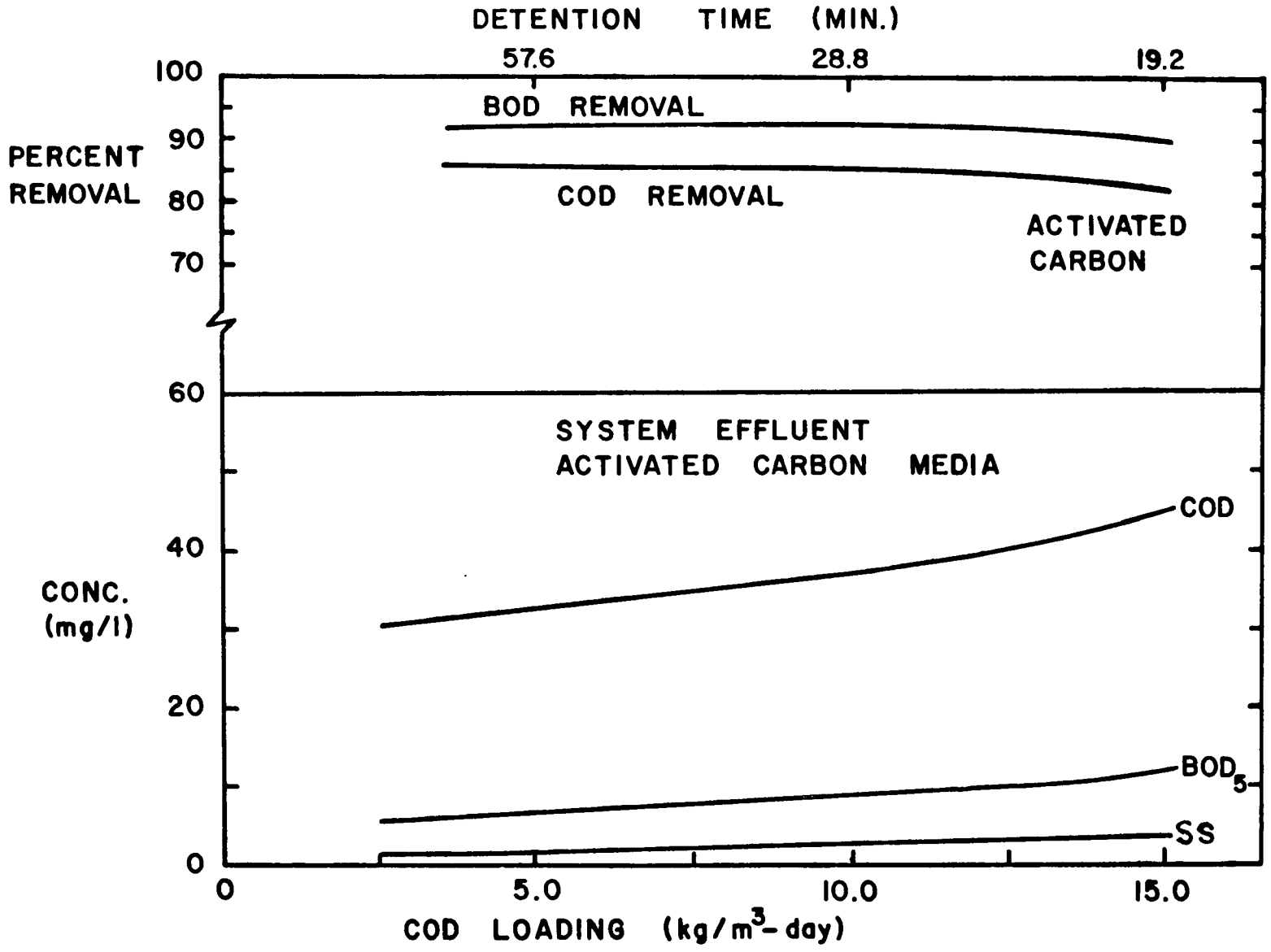

FIGURE 2 (6)

AVERAGE PERFORMANCE OF BIOLOGICAL AERATED FILTER FUNCTION OF ORGANIC LOADING - DOMESTIC WASTEWATER USING THE FRENCH PILOT UNIT (ACTUAL DATA NOT AVAILABLE)

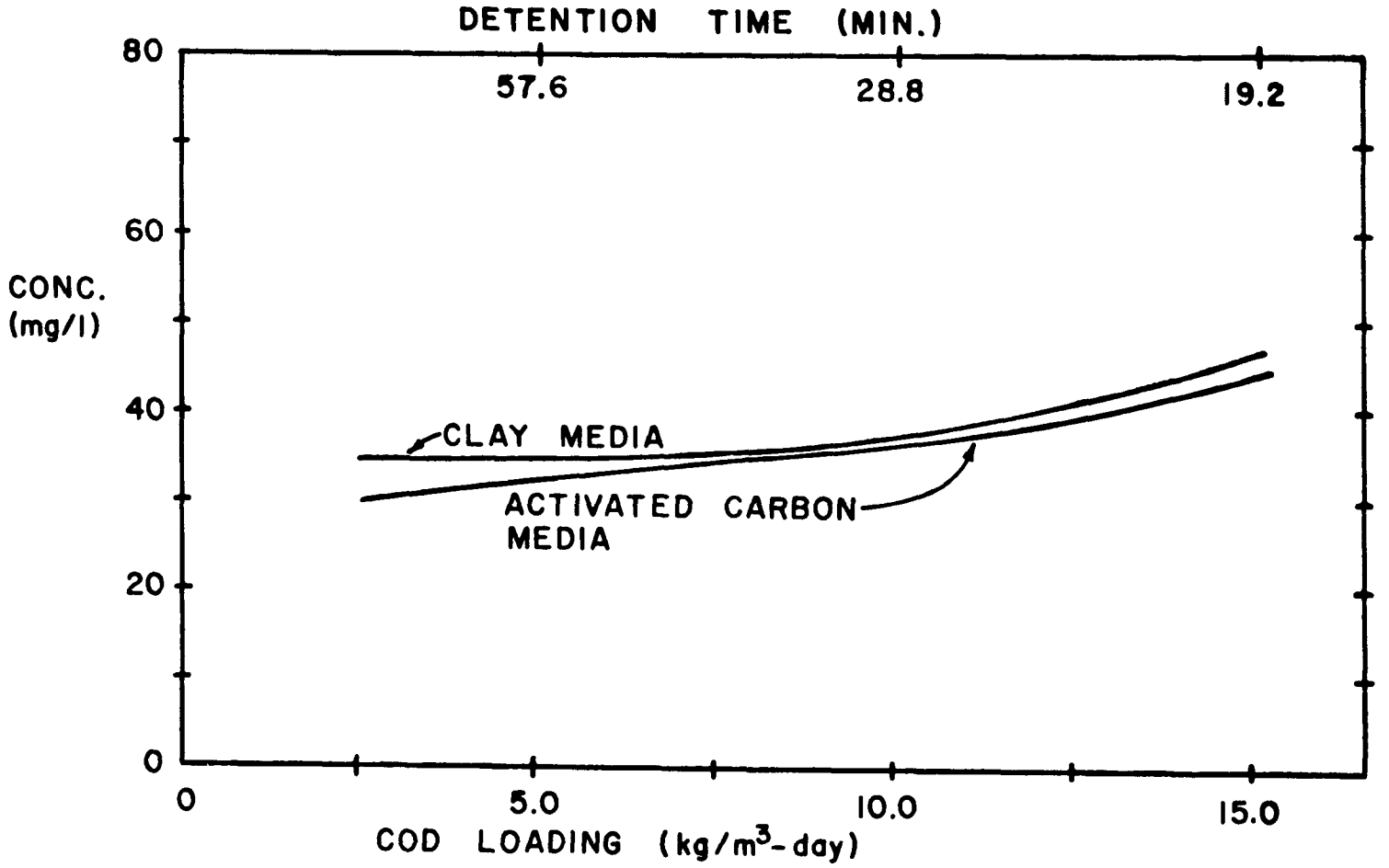

FIGURE 3 (6)

EFFECT OF MEDIA ON EFFLUENT COD CONCENTRATION USING THE FRENCH PILOT UNIT (ACTUAL DATA NOT AVAILABLE)

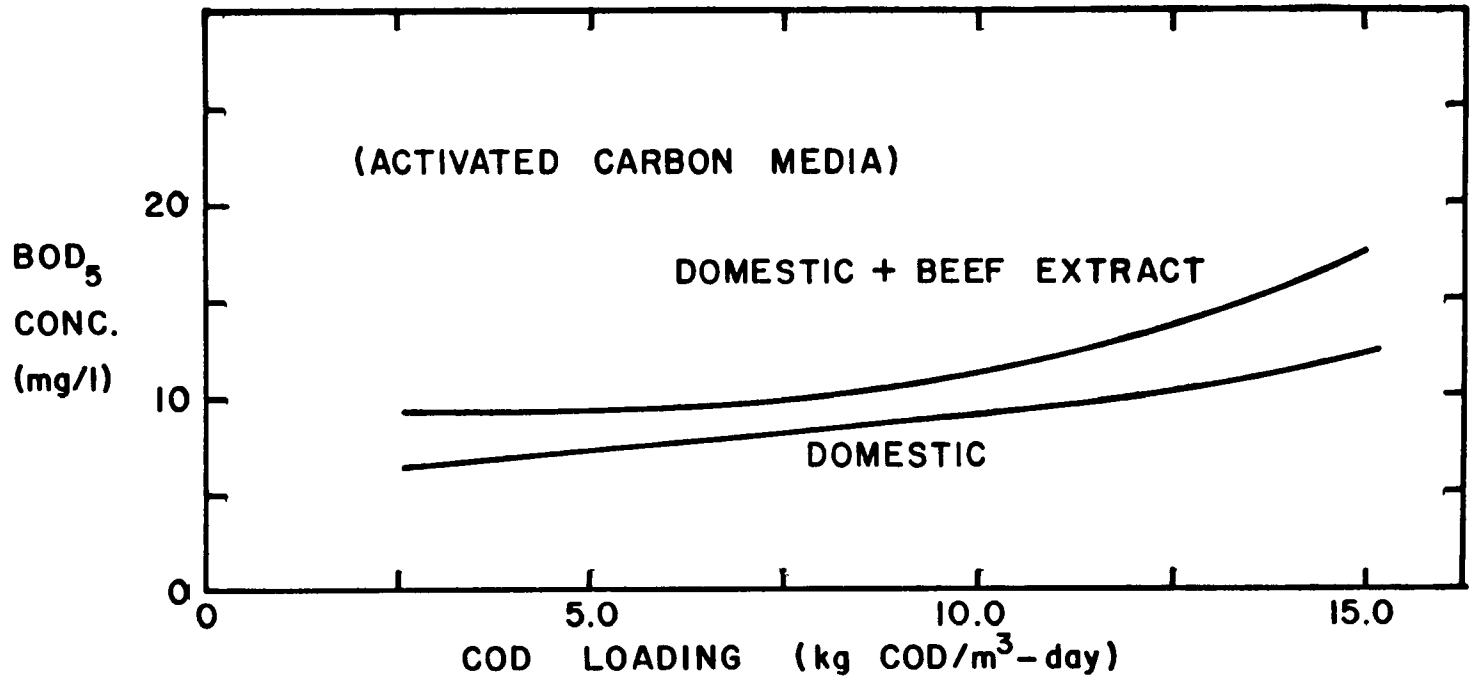

FIGURE 4 (6)

AVERAGE PERFORMANCE OF BIOLOGICAL AERATED FILTER USING HIGHER STRENGTH DOMESTIC - SYNTHETIC WASTEWATER USING THE FRENCH PILOT UNIT (ACTUAL DATA NOT AVAILABLE)

were capable of achieving an effluent NH_3-N of 7 mg/l, which was the objective of this particular test.

To date, in addition to the 1893-m^3/d (0.5-mgd) LeHavre and 757-m^3/d (0.2-mgd) Columbes facilities, 15,519-m^3/d (4.1-mgd) and 9463-m^3/d (2.5-mgd) facilities are in operation at Soissons and Valbonne, France. Other facilities are under construction including 18,925-m^3/d (5.0-mgd) and 13,248 m^3/d (3.5-mgd) facilities at Grasse and Hockfelden, France, respectively.

DEVELOPMENT IN THE U.S.

During the late 60's and early 70's the use of packed bed reactors (PBR) (a predacessor to the BAF) for nitrification progressed from the laboratory stage to pilot-scale and full-scale commerical availability (7). A PBR consists of a bed of media upon which biological growth occurs overlaying an inlet chamber. Wastewater is evenly distributed across the floor of the PBR by baffles, nozzles, or strainers. Several types of media have been successfully used in PBR reactors for nitrification including 2.5- to 3.9-cm stones, 5.0-mm gravel, 1.8-mm (effective size) antracite and 9-cm plastic media. The BAF differs from the PBR in several areas. The BAF is capable of removing significant BOD and solids as well as nitrifying. Wastewater flow is downward rather than upward. Air is supplied via an in-bed sparger rather than via predissolution of oxygen or bubbling air across the floor of the reactor.

A BAF proof-of-concept pilot plant study was carried out by Eimco during the latter part of 1979 to treat primary treated wastewater at Salt Lake City, Utah (6). The pilot reactor was 19 cm (7-1/2 in) in diameter with a media depth of 1.6 m (5 ft-3 in). When run at 12.54 m^3/d (2.3 gpm), the unit reportedly achieved BOD_5 and SS less than 10 mg/l.

At the University of Utah, a nitrification bench-scale study has recently been completed as part of work by Ms. Barbara Campion under the direction of Dr. H. David Stensel (8). Using a 0.17-m (6-1/2-in.) diameter reactor, 4.0 m (13 ft) high, effluent NH_3-N concentrations less than 1 mg/l were achieved at 17°C when treating an influent NH_3-N concentration ranging from 20 to 50 mg/l in a 1.5-m (5-ft) deep anthracite media bed at loadings of 399 kg NH_3-N/m^3/d (24.9 lb NH_3N/1000 ft^3/d). During the study, it was noted that BOD_5 dropped below about 20 mg/l before significant nitrification occurred. Effective size of the anthracite is the same as the media to be used in the BAF although specific surface area will vary due to differences in porosity and shape.

More recently, four other field test pilot studies in the U.S. have been completed at Park City, Utah (9), and Asheville, N.C.(10), as well as at Homestake Mining Company, Lead, S.D., and Spanish Fork, Utah.

The Park City, Utah, wastewater treatment facility is located near Salt Lake City, Utah, at an elevation of 1966 m (6450 feet). Process

water temperatures range down to 8°C. The effluent requirements of the 7192-m^3/d (1.9-mgd) facility require nitrification with effluent seasonal ammonia nitrogen concentrations ranging from 2.0 mg/l in summer up to 7.0 mg/l in winter months.

A two-column pilot unit (rated at 1 gpm average flow per column) (Figure 5) was tested during the spring of 1982 under the direction of PM Engineers, Inc., Salt Lake City, Utah. The pilot study and results have been reported by PM Engineers (9) and are summarized here. Each column was 30.5 cm (12 in.) in diameter and constructed to represent the actual vertical full-scale BAF unit including media depth, location of underdrains, influent air, etc. Some mechanical and equipment problems ocurred during the effort, but there were no problems encountered that related to process feasibility. For example, during the shakedown period, a compressor failure caused a 2-week delay until a replacement could be installed. Also, primary effluent was delivered by pumping to a 208-L (55-gal) drum surge tank. Pump suction was frequently lost causing flow to the BAF unit to terminate. In another instance, the primary clarifier, which was providing the feed to the BAF unit, was removed from service for 3 days. During this period, air only was fed to the BAF unit to maintain aerobic conditions. When primary effluent feed was resumed, the pilot unit was performing as desired within only 1 day. Problems were also encountered with foulants plugging feeder valves and blinding of the bed. The bed, therefore, required backwashing sooner than the regular 24-hr cycle. To solve this problem, the filter was backwashed with a single cycle (as opposed to 3 to 6 cycles in a full sequence). This had the effect of providing partial cleaning of the filter bed and extending the filter run length until the full backwash cycle could take place. As a result, the following procedure has been adopted by Eimco for recommended full-scale design and operation. A timer control activates a full backwash cycle on a regular 24-hr basis at night during low flow, and a level sensor detects the need for and initiates the shorter pre-emptive cycle.

The wastewater at the Park City plant is atypical. Primary effluent can be characterized as follows: BOD_5 = 50 mg/l, NH_3-N = 15 mg/l, Flow ranges from 2650 to 14,005 m^3/d (0.7 to 3.7 mgd) due to presence of infiltration/inflow. The pilot studies were conducted from April 19 through September 24, 1982 to determine if the Park City, Utah wastewater could be treated for BOD and ammonia removal to achieve permit limits without secondary clarification.

The most significant part of the study occurred from August 20 to September 24 when loading rates were most consistant. Figure 6 presents the results of the testing during this period when effluent BOD ranged from 5 to 15 mg/l and averaged about 10 mg/l and effluent NH_3-N^5 ranged from <1.0 mg/l up to 7.0 mg/l and averaged about 2.5 mg/l. Total suspended solids were always below 5 mg/l. The consulting engineer concluded that the following design loading rates should be used in

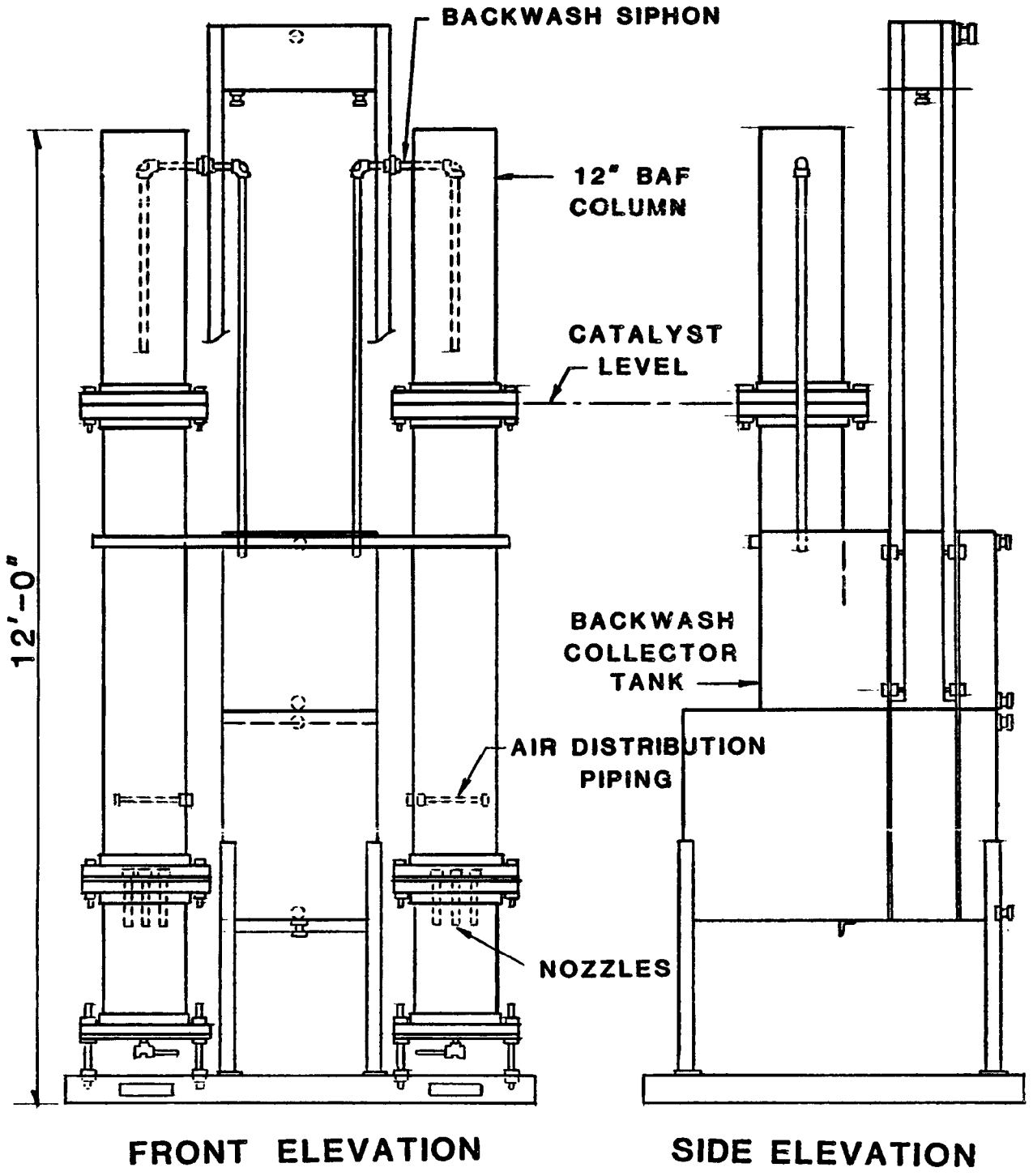

FIGURE 5

BAF PILOT PLANT AT PARK CITY, UTAH
SOURCE: PM ENGINEERS, INC. (9)

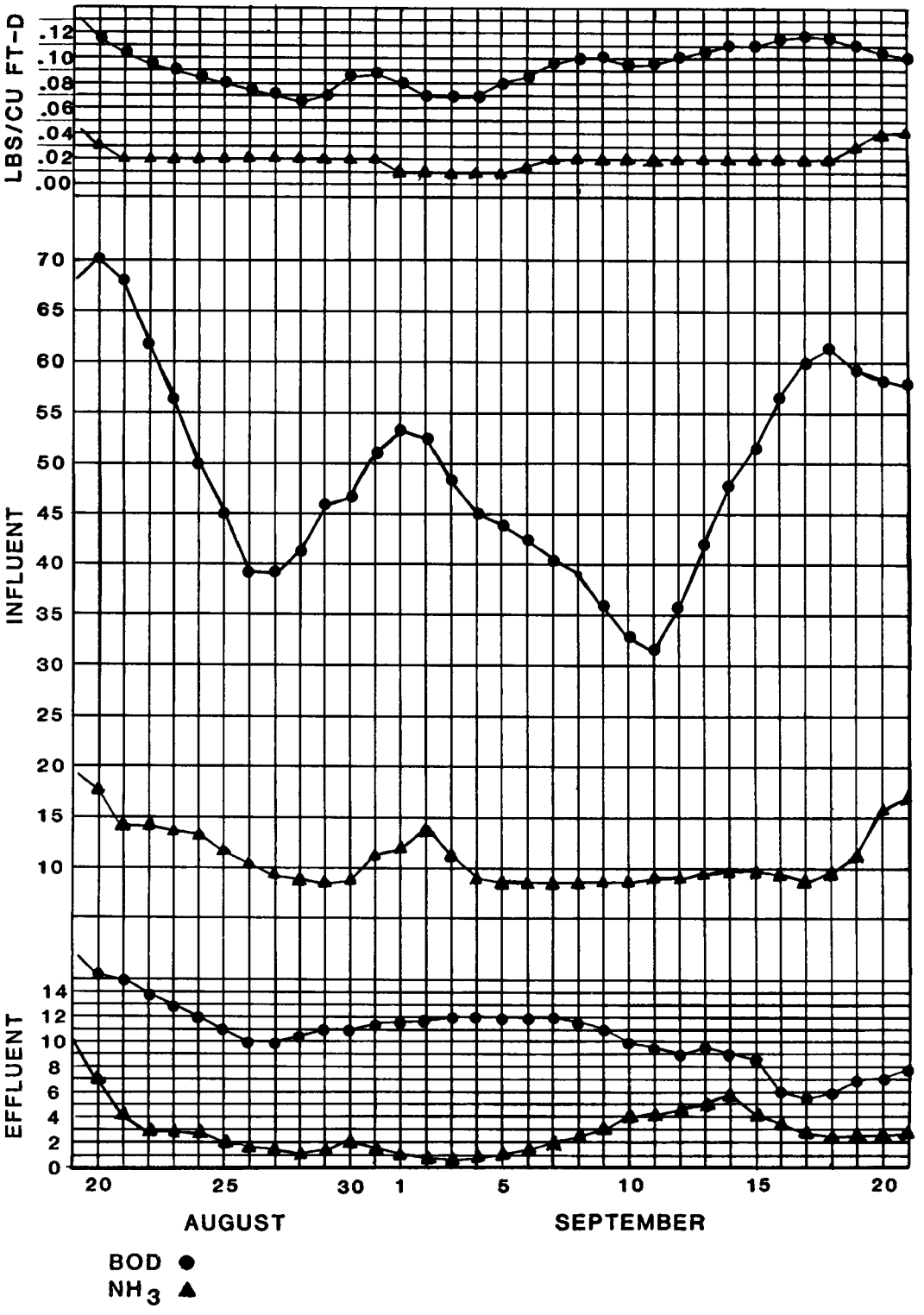

FIGURE 6.
PARK CITY PILOT TEST RESULTS
SOURCE: PM ENGINEERS, INC. (9)

full-scale design to meet the required effluent limits: 1.44 kg BOD_5/m^3/d (90 lb BOD_5/1000 ft^3/d) and 0.32 kg NH_3-N/m^3/d (20 lb NH_3-N/1000 ft^3/d).

In Asheville, N.C.(10), present plans are to expand the existing 94,625-m^3/d (25-mgd) step aeration wastewater treatment plant to 170,325 m^3/d (45 mgd). The plant treats a significant industrial wastewater component. The City of Asheville and their consulting engineer, Harry Hendon and Associates, Inc., Birmingham, Alabama, decided to seek a treatment process which would handle industrial shock loads and which would meet limited space requirements at the existing site. In May 1982, representatives of the treatment authority, the consulting engineer and Eimco made on-site inspections of several BAF installations in France. Subsequently, the BAF unit process was evaluated as part of a cost-effective analysis. A decision was made to pilot the BAF process at the treatment plant to determine if it would meet a BOD_5/SS requirement of 30/30 mg/l. The pilot study and results have been reported by Harry Hendon and Associates and Eimco (10)(11) and are summarized here.

The BAF pilot unit used comprises four independently operated cells, each measuring about 0.6 m x 1.1 m (2 ft x 3.5 ft) with a media bed depth of 1.8 m (6 ft). The pilot unit was manufactured by Eimco for such studies, is fully automated and includes a clean water effluent storage tank, a dirty water backwash storage tank and the pumps, blowers and control instrumentation needed for operation. As opposed to the manual backwashing conducted at Park City, Utah, the Asheville pilot unit was to backwash each cell automatically 4 to 6 times every 24 hr. Each backwash cycle was to include the following procedures:

1. Feed, process air and effluent valves in the cell were closed.

2. Air scouring proceeded at approximately 0.9 m^3/min/m^2 (3 cfm/ft^2) for 45 seconds.

3. Simultaneous air scour and water wash at 16 to 18 gpm/ft^2 proceeded until the liquid level in the cell reached a point just below the siphon invert.

4. A 30-second quiescent period followed to allow media settling.

5. Washwater was again introduced and continued until the liquid level in the cell reached an elevation just above the top of the siphon.

6. Backwash water was siphoned to the dirty water backwash sump.

A counter automatically kept track of the total number of backwash cycles for each cell in the pilot unit. These are summarized in Table 2. The consulting engineer estimates that, at design flows and loadings, about 7% of influent to the BAF would be used as backwash water. Due

TABLE 2 (10)(11)

SUMMARY OF BAF BACKWASHING CYCLES AT ASHEVILLE, N.C.

Operation	Cell Number 1	2	3	4
Loading (m/hr)	1.71	2.44	3.18	1.22
Number of Total Daily Timed Cycles	5	5	6	4
Number of Daily High Level Cycles	2	2	2	1
Number of Total Daily Cycles	7	7	8	5
Backwash Volume, m^3	3404	3404	3890	2431
Backwash Water (% Of Influent)	10	7.1	6.5	10.2

to the fact that the siphon in the pilot unit was mounted 0.2 m (8 in.) lower (for transportation purposes) than would be the case in full scale design, the actual number of cycles expected may be lower in a full-scale operation.

A survey of plant operating records indicated the maximum monthly average primary effluent BOD_5 to be 175 mg/l, which served as the basis for the proposed full-scale BAF design loading rates of 5.6 kg $BOD_5/m^3/d$ (348 lb $BOD_5/1000\ ft^3/d$), 0.09 m/min (2.24 gpm/ ft^2) under maximum hydraulic conditions and 0.04 m/min (0.99 gpm/ft^2) under average conditions of flow. The pilot plant was set up to operate at 5.7 kg $BOD_5/m^3/d$ (355 lb $BOD_5/1000\ ft^3/d$) and 0.09 m/min (2.26 gpm/ft^2) maximum hydraulic loading rate and 0.04 m/min (1.01 gpm/ft^2) average hydraulic loading rate.

During the total time of the pilot plant operation, the influent BOD_5 to the BAF ranged widely from 30 mg/l to 189 mg/l and averaged 107 mg/l, substantiating the proposed maximum design value of 175 mg/l in order to meet permit requirements at all times. Total oil and grease averaged 4.1 mg/l and ranged from 0.5 to 8.5 mg/l. After the startup period, BAF unit performance was monitored for 38 days under constant flow conditions at average flow in cell 2 and sustained overloading of 126% of average flow in cell 3 (Table 3). Average influent BOD_5 during this period was 87 mg/l.

The effluent BOD_5 averaged 20.3 mg/l and 22.3 mg/l in cell 2 and cell 3, respectively. Range of effluent BOD_5 was from about 6 mg/l to 41 mg/l in the cell loaded at average conditions and from 6 mg/l to 50 mg/l in the overloaded cell. Effluent TSS were consistently low, averaging less than 12 mg/l in all cells. During this time period, the plant's step aeration process was upset due to several industrial shock loadings. For example, pH ranged from 6.1 to 9.1 over 24 hr and high pH values of 11.0 were observed. As a result, the treatment plant was unable to comply with permit limitations whereas the BAF pilot unit continued acceptable performance. It is notable that, in spite of higher hydraulic and organic loadings experienced by overloaded cell 3, BOD_5 removal performance was within 10% of cell 2 when using effluent BOD_5 as the measure.

Due to the lower-than-average strength primary effluent, organic loadings never reached more than 62% of the actual design load during the study. The consulting engineer conducted a linear regression analysis of the data which indicated, as would be expected, higher effluent BOD_5 concentrations as organic load increased. The analysis also predicts that for the Asheville wastewater, at design loadings of 5.7 kg $BOD_5/m^3/d$ (355 lb $BOD_5/1000\ ft^3/d$), effluent BOD_5 would be less than 28 mg/l.

The pilot unit was briefly tested over a 3-day period from December 6 through December 8, 1982, to ascertain effects of diurnal

TABLE 3 (10)(11)

LOADING RATES AND OBSERVED PERFORMANCE OF THE ASHEVILLE, N.C. PILOT UNIT AT CONSTANT FLOW RATES

Cell No.	Flow (m^3/d)	Hydraulic Loading Rate (m/hr)	% of Design Hydraulic Loading Rate	Organic Loading Rate (kg BOD/m^3/d)	% of Design Organic Loading Rate	Average Effluent BOD (mg/l)	Average Effluent TSS (mg/l)
1	27.3	1.7	72	2.1	37	-	9.0
2	38.2	2.4	100	2.9	51	20.3	8.0
3	48.0	3.2	126	3.5	62	22.3	11.0
4	19.1	1.2	50	1.4	25	-	7.0

loading conditions. Cell 2 was operated at a constant flow rate of 2.4 m/hr (1 gpm/ft^2), the average hydraulic design low rate. Cells 1 and 3 were alternately operated at flow rates of 1.8 m/hr (0.72 gpm/ft^2) and 3.0 m/hr (1.25 gpm/ft^2). Flow was adjusted in a step wise fashion once each in the morning and at the end of the working day. The results of this testing are summarized in Table 4. There is no distinguishable difference in performance between cell 2 and cells 1 and 3.

UNIVERSITY OF UTAH/EPA FULL-SCALE DEMONSTRATION OF THE BAF

Design information is needed to determine BAF tank volume, air requirements, blower horsepower and sludge production. Tank volume is based on recommended volumetric organic loading to achieve a desired effluent quality as a function of temperature and wastewater characteristics. The hydraulic loading rate is used to determine the filter surface area. The required process air feed rate is based on an assumed value of 0.8 to 1.0 kg O_2/kg BOD_5 removed, an averge DO of 2 mg/l, a mid-depth saturation DO concentration and an oxygen dissolution efficiency of about 10.5% at standard conditions. Assumed values for alpha and beta are required.

The present design approach originates from Eimco, is empirical and is based on reported operation and performance data from OTV through Eimco and the limited U.S. pilot work.

Due to the impacts of the U.S. EPA Innovative/Alternative Technology Program, there are two U.S. locations where the BAF is presently under design based on this empirical approach for use as part of a full-scale wastewater treatment facility. These include: 1) a 2270-m^3/d (0.6-mgd) facility at Wallace, N.C., to treat a clarified trickling filter effluent for nitrification to achieve a 3.0-mg/l NH_3-N effluent and 2) a 3859-m^3/d (1.0-mgd) facility at Clinton, Arkansas to treat an equalized, pretreated domestic/poultry waste for nitrification to achieve a 1.0-mg/l NH_3-N effluent.

Other locations where the BAF system has been considered in planning include:

Location		Size	Treatment
1.	Asheville, N.C.	20.0 mgd	BOD/nitrification
2.	Wilmington, Ohio	2.3 mgd	BOD/nitrification
3.	Glendale, Ohio	0.5 mgd	BOD/nitrification
4.	Fairmont, N.C.	0.5 mgd	Nitrification
5.	Greene County, Ohio	4.5 mgd	BOD/nitrification
6.	Lewisburg, Ohio	0.3 mgd	Nitrification
7.	New Lebanon, Ohio	0.5 mgd	BOD/nitrification
8.	Park City, Utah	1.9 mgd	Nitrification

TABLE 4 (10)(11)

DIURNAL FLOW TEST RESULTS FOR ASHEVILLE, N.C.

	Average Influent Concentration (mg/l)	Effluent Concentration (mg/l)		
		Cell 1	Cell 2	Cell 3
December 6, 1982				
BOD (mg/l)	55	6.8	7	7.5
TSS (mg/l)	32	5	6	7
December 7, 1982				
BOD (mg/l)	90	11	8	13
TSS (mg/l)	40	10	10	8
December 8, 1982				
BOD (mg/l)	110	10	12	11
TSS (mg/l)	36	5	6	4
Average Results				
BOD (mg/l)		9	9	10
TSS (mg/l)		7	7	6

In general terms, there are areas of risk in utilizing the BAF in the U.S. stemming from differences in French and U.S. experiences including media, wastewater characteristics and flows. Long-term full scale operating experience in the U.S. has not substantiated projections of media life and loss, performance, frequency of backwash and affect of recycle on primary clarifier performance.

A more fundamental understanding of the process is desirable based on fixed film mass transfer information including media surface area, biomass per unit volume, liquid velocity, substrate type and concentration and bulk liquid DO concentration.

The EPA entered into a cooperative agreement with the University of Utah in May 1982 to conduct a 1893-m^3/d (0.5-mgd) (nominal flow rate) demonstration project through March 1984 to independently assess process feasibility and application of the BAF for treatment of a primary effluent. The Wastewater Technology Center for Environment Canada is also providing support for the project. Dr. H. David Stensel is the principle research investigator for the University of Utah. Progress of the demonstration project has been reported (12)(13) and is summarized here.

The pilot unit is located at the South plant of the South Davis Sewer Improvement District about 11.2 kilometers from Salt Lake City, Utah. The South Davis South plant includes primary and trickling filter treatment and is presently processing 9463 m^3/d (2.5 mgd) of combined domestic and industrial wastes. Domestic flow comprises 70 to 80% of the total with the remaining industrial contribution originating from an oil refinery, a tannery and a laundry. Primary treatment is accomplished with two 16.8-m (55-ft) diameter clarifiers operated at an average overflow rate of 21.4 m/d (526 gpd/ft^2). Secondary processes include a plastic media trickling filter followed by an intermediate clarifier, final biological treatment using two parallel-operated rock trickling filters and final settling. Underflow from the final clarifier is returned to the head of the plant. Figure 7 presents the physical layout of the BAF demonstration project. Primary effluent is pumped from the primary clarifier effluent wet well to the BAF through a 15.2-cm (6-in.) feed line. Two self-priming centrifugal pumps rated at 3 hp and 5 hp operate independently to supply primary effluent to each cell of a 2 cell BAF half-size module (3.7 m (12 ft) x 6.4 m (21 ft) x 3.7 m (12 ft)). Each cell includes 1.8 m (6 ft) of media and 11.5 m^2 (124.3 ft^2) of cross sectional area. Variable drive feed pumps are used to provide a variety of flow rates to each cell. A specially built DC signal generator unit automatically controls the flow rates on an hourly basis so as to simulate diurnal flow conditions. The signal generator also controls influent and effluent composite sampling pumping. Eimco provided the BAF tank with process air headers and a plastic nozzle underdrain system, a 25-hp backwash air blower, two 10-hp process air blowers, a 25-hp backwash water pump, a 1-1/2-hp com-

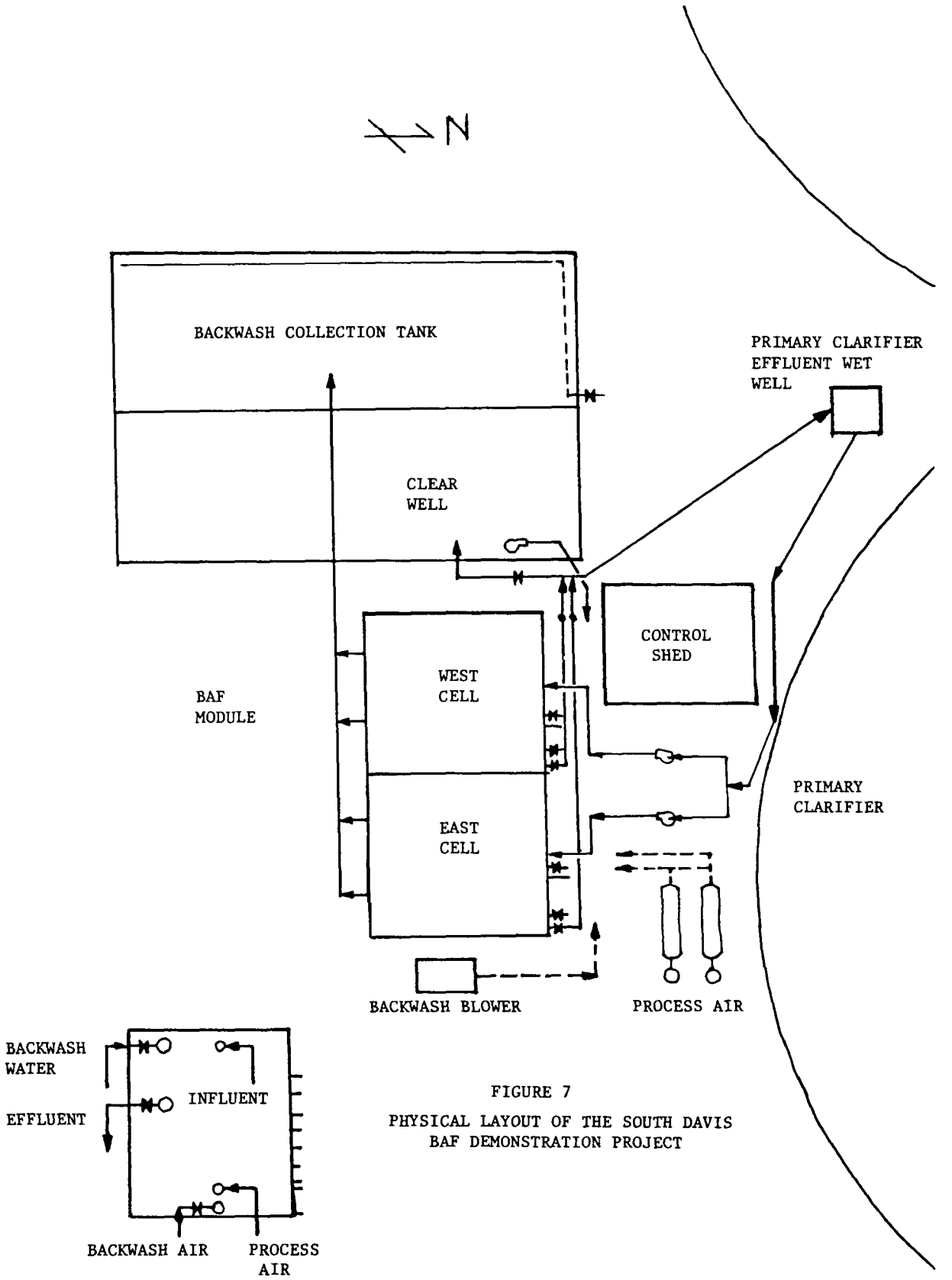

FIGURE 7

PHYSICAL LAYOUT OF THE SOUTH DAVIS
BAF DEMONSTRATION PROJECT

pressor, a microprocessor controller and necessary pneumatic operating valves. Sample taps were added at 0.3-m (1-ft) intervals through the height of the bed and below the underdrains. Propeller meters are used on the effluent lines to provide instantaneous and cumulative flow measurements on a continous flow recorder. An existing aerated grit chamber was divided for purposes of providing a backwash collection tank and a backwash water supply clear well to receive effluent. The dirty backwash water is returned to the head of the South Davis South plant.

Objectives of the project include assessment of treatment performance, operational and maintenance requirements and equipment reliability. Each cell is to be operated independently. Initially, loadings used reflect Eimco recommended loading rates of 3 and 5 kg $BOD_5/m^3/d$ to achieve effluent BOD_5/SS of 10/10 mg/l and 20/20 mg/l, respectively. Diurnal flows will be simulated ranging from peak to average flow factors of 1.25 to 1.75.

At each loading condition, BOD and SS removals will be determined as well as oxygen requirements, energy requirements, solids production and characteristics and bed and media characteristics, including attrition, film thickness, density and kinetics. A separate 16.5-cm (6-1/2 -in.) diameter column will be operated in parallel with one of the cells in order to assess scale-up factors. If successful, current plans are to use the column to evaluate nitrification performance of the BAF process.

Daily composite samples are collected from the influent and effluent for analysis of total COD, soluble COD, total BOD_5, soluble BOD_5, total suspended solids, volatile suspended solids and effluent turbidity. In-bed profiles of COD and suspended solids removals are obtained. Effluent and in-bed measurements of DO are taken daily. Off-gas samples of spent air are collected to determine oxygen uptake efficiencies. Temperatures are monitored. After each backwash cycle, dirty backwash water is measured for volume and sampled from an aerated, homogeneous mix collected in the backwash water collection tank.

Sludge settling tests are run to determine SVI and zone settling velocity. Sludge age is also determined. Headloss is measured across sample tap locations using a manometer. Tracer studies with lithium chloride will be conducted to define bed hydraulic characteristics.

Design of the project began in June 1982, and installation of the BAF unit was finally completed by the end of November 1982. During December, Eimco checked out the BAF equipment. About 19.5 bags of media were then added to each cell. Each bag contained about 1.1 m^3 (40 ft^3) of media. As delivered, the media contained a significant amount of fines which had to be removed by backwashing. Prior to start-up, backwashing was conducted using tap water obtained by filling the clear well using a garden hose. This cumbersome procedure limited the number of backwashes to four and twelve for the west and east cells, respectively.

Startup occurred December 27, 1982, at a hydraulic loading of about 327 m^3/d (60 gpm). After only 4 days of operation, shutdown was necessary due to a series of mechanical problems with the air supply compressor, flow meter, pneumatic control values and pressure release vent pipe. After repairs were completed, the unit resumed operation on January 14, 1983, but further operating problems were encountered due to 1) inadequate backwash air rates caused by a faulty pressure release valve in the backwash blower, 2) need for manual backwashing more frequently than every 24 hr because the microprocessor control had not been programmed, 3) media blinding causing influent to inappropriately exit the backwash siphon and causing carryover of media into the backwash tank, 4) incorrect bypass air rate through the process air header during backwash, 5) inadequate backwash air distribution and 6) the need to replace circuit breakers for the process air blowers, replace cracked PVC sample taps and correct air flow measurement equipment.

Excessive headloss due to media blinding were observed to occur in the top part of the bed. This headloss could have been due to fines plugging the bed, high biomass concentrations or air binding and/or inadequate backwashing; but exact causes have not yet been identified. Based on sieve analyses, it is noted that the effective size of the media changed since startup from 2.3 to 3.0 mm as measured on April 5, 1983, indicating removal of fines. The above problems were corrected by February 14 when the plant began fully automatic operation. Data collected during the period from January 14 through February 14 is not considered to completely represent BAF performance due to interruptions in flow and other problems encountered. Through March 8, the flow fed to each cell was maintained at 327 m^3/d (60 gpm) resulting in average organic loadings of 1.2 to 1.4 kg $BOD_5/m^3/d$ (75 to 87 lb BOD_5/1000 ft^3/d). Hydraulic loading was 1.2 m/hr (0.5 gpm/ft^2) during this period. The run times between timed backwashing for the east cell improved to a 24-hr regular interval. The west cell frequently required backwashing every 10 to 12 hr, possibly due to a sag in the underdrain plate causing a non-uniform backwash air rate. This problem has been solved by increasing the backwash air rate. Beginning on March 8, 1983, the flow rate to each cell was increased and a diurnal flow pattern was to be initiated simulating peak to average flow factors of 1.25 and 1.75, but it was discovered that the process air distribution piping holes were drilled too small causing high headlosses in order to provide the necessary increase in air flow for the attendant increase in organic load.

Media was removed from both cells, and the hole size was increased to 1.5 mm by drilling in order to provide sufficient air flow. This task was accomplished by April 8, 1983. The media was replaced by April 11, and startup was again initiated at about 1.2 m/hr (0.5 gpm/ft^2), resulting in a loading of about 1.0 kg $BOD_5/m^3/d$ (62.5 lb $BOD_5/1000 ft^3/d$). On April 22, loadings were increased to about 2.2 kg $BOD_5/m^3/d$ (137 lb $BOD_5/1000 ft^3/d$) and 3.5 kg $BOD_5/ m^3/d$ (218 lb BOD_5/

1000 ft^3/d) on the east and west cells, respectively. Diurnal flow was simulated using a peak to average flow factor of 1.3.

Organic loading rates and influent/effluent BOD_5 data for January through March 1983 are presented in Figures 8 and 9 for the east and west cells, respectively. Figures 10 and 11 present hydraulic loading rate and effluent suspended solids for the east and west cells, respectively. As mentioned previously, data collected up through February 14 do not reflect representative BAF performance. Performance after automatic operation is considered more representative.

This is illustrated by Table 5, which presents information on ranges and averages of loading rates and effluent characteristics for the east and west cells, both before and after automatic operation. The smaller operating ranges of loadings after automatic operation reflect improved process control resulting in significantly smaller ranges in effluent BOD_5, incidental improvement in the range of effluent solids removals and significantly improved effluent BOD_5 concentrations.

During the period January through March 1983, the average BOD_5 and total suspended solids effluent concentrations and removal efficiencies are essentially the same for both cells in spite of the fact that the west cell was loaded at higher organic and hydraulic rates. Tables 6, 7, 8 and 9 present average process performance results of the west and east cells both before and after automatic operation. After automatic operation, 88% BOD_5 removal and 91% SS removal efficiencies were achieved resulting in 10 mg/l BOD_5 and 10 mg/l SS in the effluent. These results must be viewed in consideration that the system was loaded at less than 50% of the 3 to 5 kg $BOD_5/m^3/d$ (187-312 lb $BOD_5/1000$ ft^3/d) rates to be studied as part of the demonstration project and less than the average hydraulic loading rate of 2.4 m/hr (1 gpm/ft^2) recommended by Eimco for typical conditions.

As with the Asheville, N.C., pilot work, the South Davis project has been conducted to date under the constraint of a relatively weak and a typical primary effluent concentration of 85 to 90 mg/l BOD_5.

Summaries of performance for the east and west BAF cells during the 3-week period of April 25 - May 15, 1983, are shown in Tables 10 and 11, respectively. At the increased loading rates during this time period, effluent BOD's averaged 11 mg/l and 17 mg/l for the east and west cells, respectively, reflecting removals of 87% and 79%. Similarly, effluent total suspended solids were 10 mg/l and 18 mg/l reflecting removals of 91% and 85%. Table 12 presents a comparison of performance data for the time periods February 14 - March 8, 1983 (period 1) and April 25 - May 15, 1983 (period 2). In the east cell, the organic and hydraulic loading rates were increased by 1.9 times and observed performance was about the same for both time periods. In the west cell, however, at organic and hydraulic loading rates 2.5 to 2.6 times

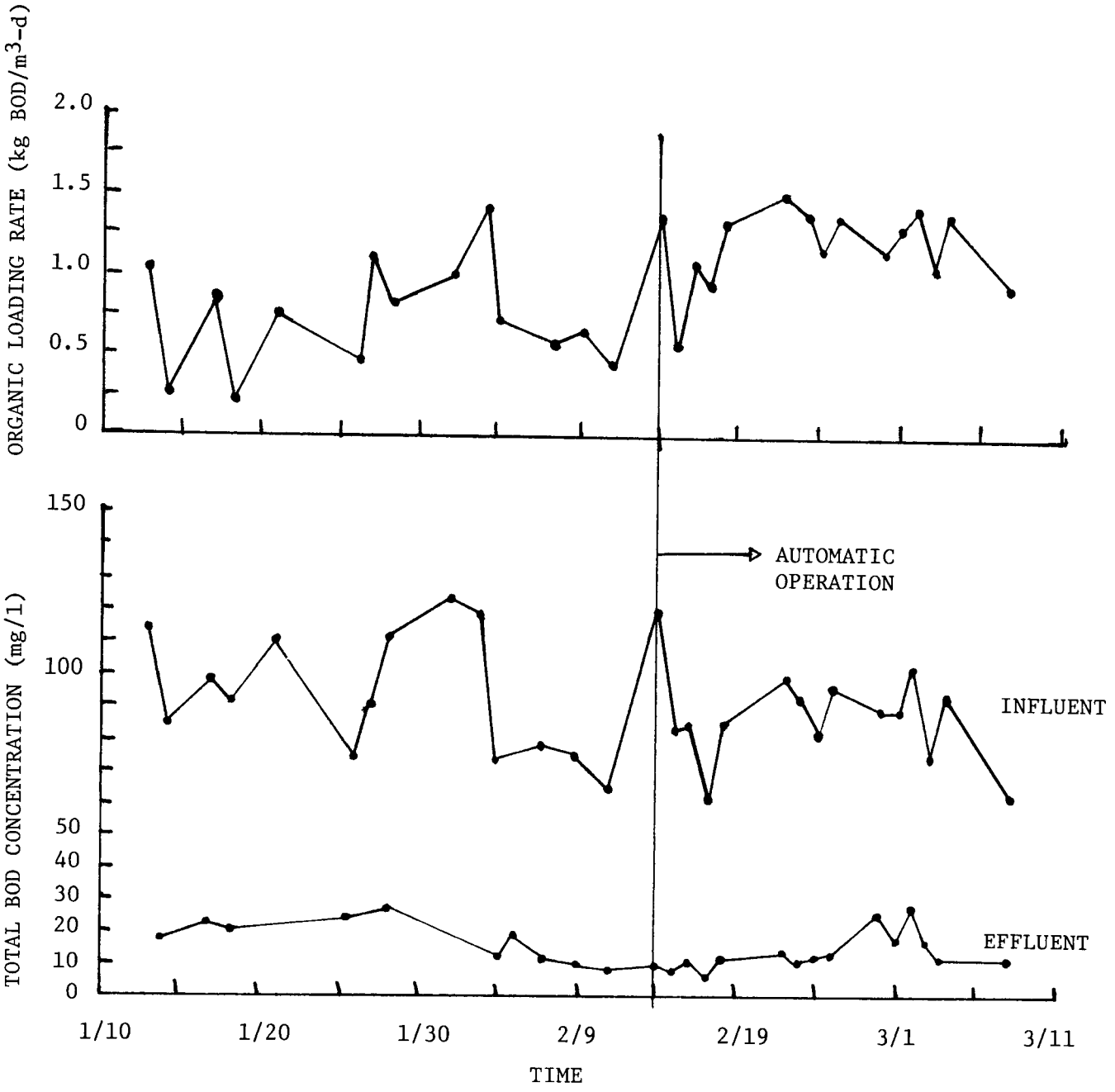

FIGURE 8

ORGANIC LOADING AND BOD REMOVAL IN EAST CELL OF THE SOUTH DAVIS BAF DEMONSTRATION PROJECT

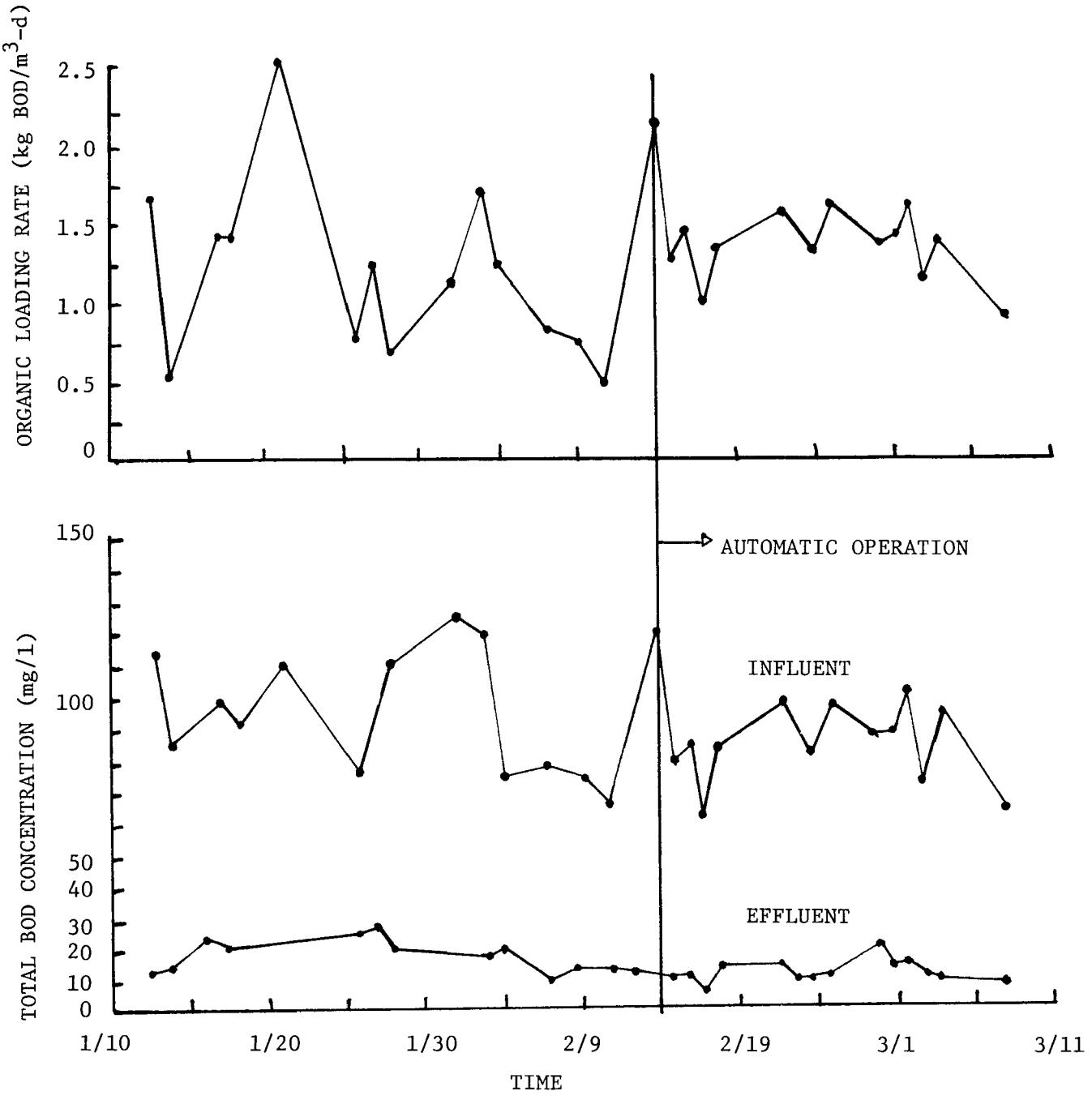

FIGURE 9
ORGANIC LOADING AND BOD REMOVAL IN WEST CELL OF THE SOUTH DAVIS BAF DEMONSTRATION PROJECT

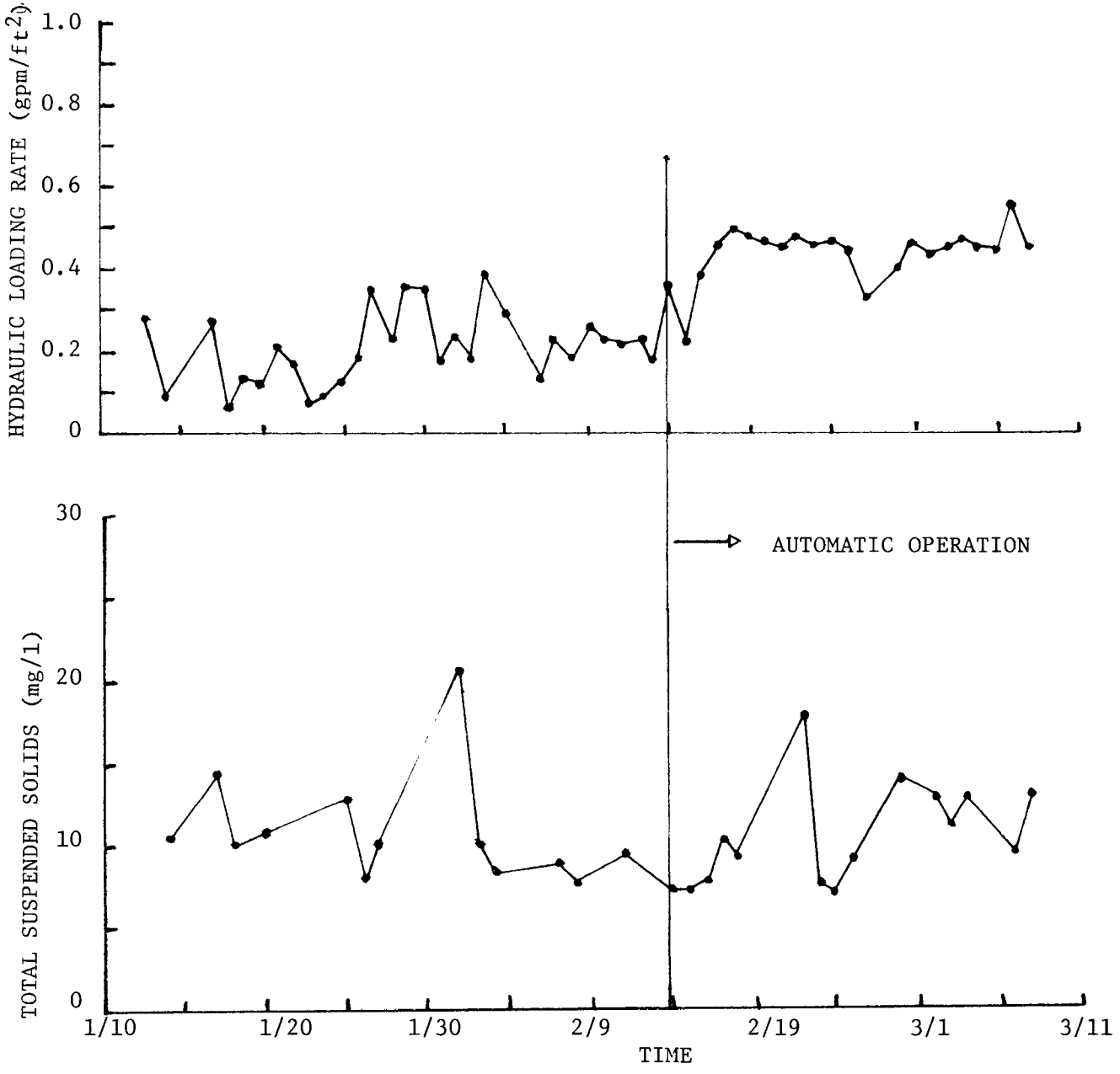

FIGURE 10

HYDRAULIC LOADING AND EFFLUENT SUSPENDED SOLIDS IN EAST CELL OF THE SOUTH DAVIS BAF DEMONSTRATION PROJECT

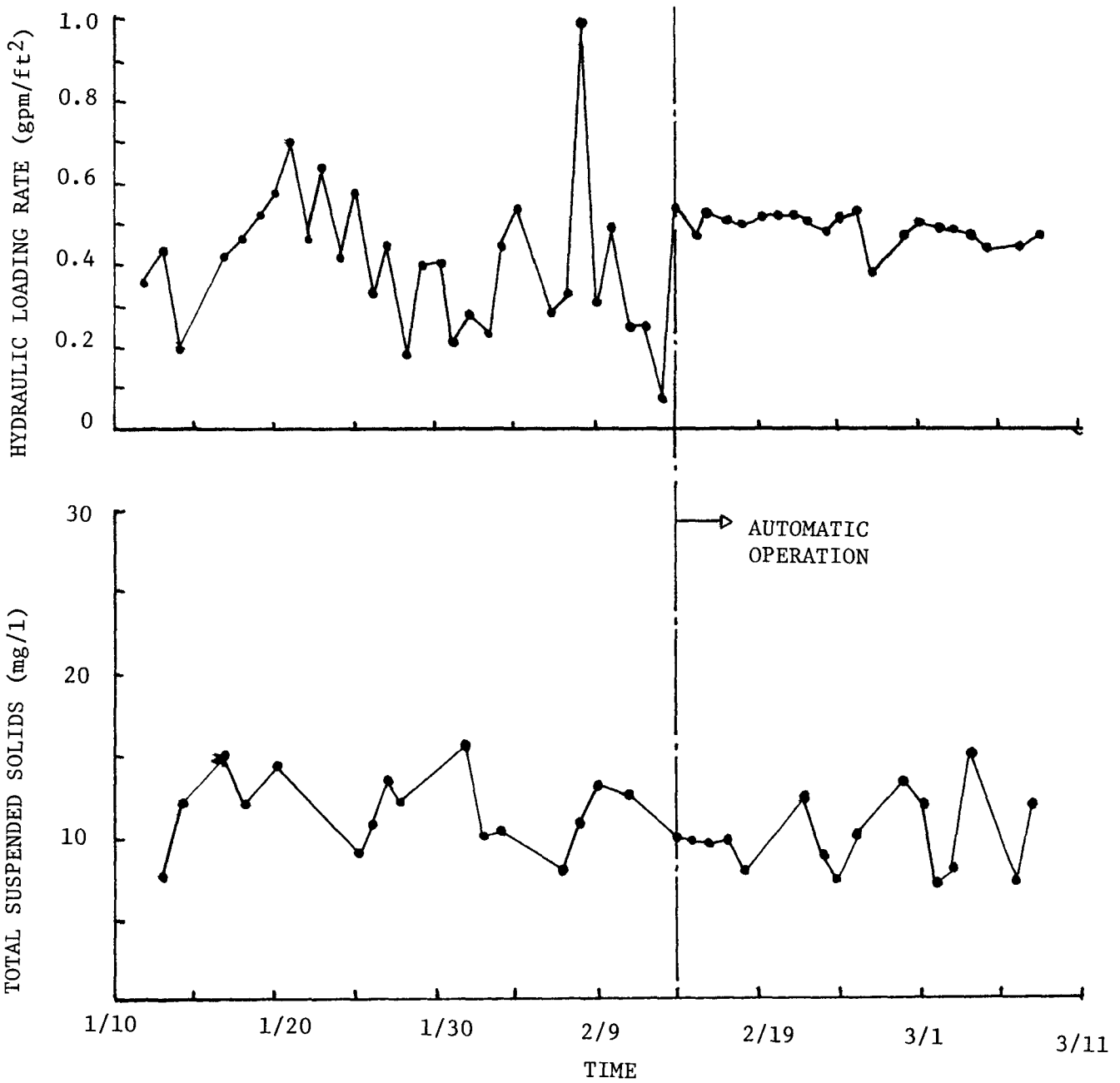

FIGURE 11
HYDRAULIC LOADING AND EFFLUENT SUSPENDED SOLIDS IN WEST CELL OF THE SOUTH DAVIS BAF DEMONSTRATION PROJECT

TABLE 5

PERFORMANCE OF BAF BEFORE AND AFTER AUTOMATIC OPERATION
AT THE SOUTH DAVIS BAF PROJECT

	East Cell		West Cell	
	Before	After	Before	After
Ranges				
Organic Load Rates ($kgBOD/m^3/d$)	0.22-1.41	0.93-1.5	0.5-2.52	0.92-1.5
Hydraulic Load Rates (m/hr)	0.15-0.95	0.81-1.37	1.86-2.44	0.93-1.30
Effluent BOD (mg/l)	9-28	7-17	10-27	6-14
Effluent TSS (mg/l)	7-21	7-18	7-16	7-15
Averages				
Organic Load Rates ($kgBOD/m^3/d$)	0.73	1.19	1.17	1.39
Hydraulic Load Rates (m/hr)	0.51	1.08	1.00	1.20
Effluent BOD (mg/l)	17.4	12.1	17.4	11.6
Effluent TSS (mg/l)	10.8	10.5	11.8	10.1

TABLE 6 (12)(13)

SOUTH DAVIS BAF
PROCESS PERFORMANCE IN WEST CELL BEFORE
AUTOMATIC OPERATION (1/12/83-2/13/83)

Parameter	Avg. Influent Concentration (mg/l)	Avg. Effluent Concentration (mg/l)	Avg. Removal Efficiency (%)
TBOD	93	17	81
SBOD	26	10	63
TCOD	220	66	70
SCOD	90	50	44
TSS	116	12	90
VSS	83	9	90

Avg. Hydraulic Loading Rate = 1.00 m/hr
Avg. Organic Loading Rate = 1.17 kg $TBOD_5/m^3/d$
Avg. Organic Loading Rate = 3.16 kg $TCOD/m^3/d$

TABLE 7 (12)(13)

SOUTH DAVIS BAF
PROCESS PERFORMANCE IN EAST BAF CELL BEFORE
AUTOMATIC OPERATION (1/12/83-2/13/83)

Parameter	Avg. Influent Concentration (mg/l)	Avg. Effluent Concentration (mg/l)	Avg. Removal Efficiency (%)
$TBOD_5$	93	17	81
$SBOD_5$	26	9	65
TCOD	220	62	72
SCOD	90	50	44
TSS	116	11	91
VSS	83	7	91

Avg. Hydraulic Loading Rate = 0.51 m/hr
Avg. Organic Loading Rate = 0.73 kg $TBOD_5/m^3/d$
Avg. Organic Loading Rate = 1.59 kg $TCOD/m^3/d$

TABLE 8 (12)(13)

SOUTH DAVIS BAF
PROCESS PERFORMANCE IN WEST BAF CELL AFTER
AUTOMATIC OPERATION (2/14/83-3/8/83)

Parameter	Avg. Influent Concentration (mg/l)	Avg. Effluent Concentration (mg/l)	Avg. Removal Efficiency (%)
$TBOD_5$	97	12	87
$SBOD_5$	27	6	79
TCOD	234	51	78
SCOD	85	39	54
TSS	114	10	91
VSS	69	7	91

Avg. Hydraulic Loading Rate = 1.2 m/hr
Avg. Organic Loading Rate = 1.39 kg $TBOD_5/m^3/d$
Avg. Organic Loading Rate = 3.62 kg $TCOD/m^3/d$

TABLE 9 (12)(13)

SOUTH DAVIS BAF
PROCESS PERFORMANCE IN EAST BAF CELL AFTER
AUTOMATIC OPERATION (2/14/83-3/8/83)

Parameter	Avg. Influent Concentration (mg/l)	Avg. Effluent Concentration (mg/l)	Avg. Removal Efficiency (%)
$TBOD_5$	97	12	85
$SBOD_5$	26	6	74
TCOD	234	51	78
SCOD	85	39	54
TSS	114	11	91
VSS	69	7	90

Avg. Hydraulic Loading Rate = 1.08 m/hr
Avg. Organic Loading Rate = 1.19 kg $TBOD_5/m^3/d$
Avg. Organic Loading Rate = 3.16 kg $TCOD/m^3/d$

TABLE 10 (12)(13)

BAF PROCESS PERFORMANCE SUMMARY
FOR EAST CELL 4/25-5/15/83

	Time Period			
	4/25-5/1/83	5/2-5/8/83	5/9-5/15/83	AVG.
Flow Treated (m^3/d)	511	596	595	566
Organic Loading Rate (kg BOD/m^3/d)	1.92	2.13	2.63	2.24
Average Hydraulic Loading Rate (m/hr)	1.86	2.18	2.15	2.05
Average Detention Time (min)	61.3	50.7	50.9	54.5
Effluent BOD_5 (mg/l)	9	12	12	11
Effluent TSS (mg/l)	10	12	9	10
Effluent DO (mg/l)	1.8	2.4	-	2.0
BOD Removal (%)	88	85	87	87
TSS Removal (%)	91	90	92	91
Wastewater Temperature (°C)	12.2	15.3	11.6	12.9

TABLE 11 (12)(13)
BAF PROCESS PERFORMANCE SUMMARY
FOR WEST CELL 4/25-5/15/83

	Time Period			
	4/25-5/1/83	5/2-5/8/83	5/9-5/15/83	AVG.
Flow Treated (m^3/d)	858	810	861	843
Organic Loading Rate (kg BOD/m^3/d)	3.42	3.39	3.75	3.53
Average Hydraulic Laoding Rate (m/hr)	3.13	3.15	3.13	3.13
Average Detention Time (min)	36.4	35.0	35.1	35.5
Effluent BOD_5 (mg/l)	17	19	16	17
Effluent TSS (mg/l)	19	17	17	18
Effluent DO (mg/l)	2.2	1.6	3.0	2.2
BOD Removal (%)	78	78	82	79
TSS Removal (%)	84	85	85	85
Wastewater Temperature (°C)	12.6	14.1	11	12.4

TABLE 12
COMPARISON OF BAF PERFORMANCE IN THE EAST AND WEST CELLS DURING 1) 2/14-3/8/83 and 2) 4/25-5/15/83

	East Cell		West Cell	
	Period 1	Period 2	Period 1	Period 2
Organic Load Rates (kg BOD/m^3/d)	1.19	2.24	1.39	3.53
Average Hydraulic Load Rate (m/hr)	1.08	2.05	1.20	3.13
Effluent BOD (mg/l)	12	11	12	17
BOD Removal %	85	87	87	79
Effluent TSS (mg/l)	11	10	10	18
TSS Removal %	91	91	91	85

higher, effluent BOD_5 and total suspended solids increased to 17 mg/l and 17.5 mg/l, respectively (time period 2), from 12 mg/l and 10 mg/l (time period 1) indicating that the west cell is beginning to approach the limits of performance capability. The spring of 1983 has been an unusually wet one for the Salt Lake City area. This has been due to higher-than-normal rainfall and snow melt causing serious flooding in the area. As the study proceeds, the wastewater strength is expected to increase. Present plans are to increase loadings to about 3 kg $BOD_5/m^3/d$ (187 lb $BOD_5/1000\ ft^3/d$) and 5 kg $BOD_5/m^3/d$ (312 lb $BOD_5/1000\ ft^3/d$) on the east and west cells, respectively, and run at these constant loadings for a time.

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SLUDGE MANAGEMENT PLANS AND PRACTICES AT THE METROPOLITAN SANITARY DISTRICT OF GREATER CHICAGO

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ABSTRACT

The Metropolitan Sanitary District of Greater Chicago (District) services the city of Chicago and over 120 suburban communities. The combined industrial and domestic loading, equivalent to 11 million people, ultimately results in 635 dry megagrams (Mg) of sludge solids per day that must be managed.

The District has long believed that sludge utilization represents one of the best sludge management alternatives. Since the late 1960s the District has initiated projects throughout the state of Illinois utilizing sludge for agricultural and land reclamation purposes. Two of these projects include: (1) A 48-hectare (ha) farm on the site of a 0.35 m^3/s (8.1 MGD) water reclamation plant in a residential suburb, utilizing the 800 dry Mg of sludge produced annually to grow corn; (2) Strip-mined land, 322 km southwest of Chicago in Fulton County, reclaimed by the addition of sludge solids.

At the Fulton County site, from 1972 to 1981, an average of 477 dry Mg have been applied per hectare, increasing the organic carbon content of the minespoil from 0.67 to 4.6 percent. Crops grown are sold for livestock feed.

Current sludge processing methods at the District include new dewatering methods that result in a sludge of over 60 percent total solids. This soil-like product is being used to cover a closed, 91-hectare municipal landfill, and to construct a winter recreational area.

Competitive bidding for contracts to handle sludge have been a management option used by the District. Currently, an innovative contract for a private firm to handle 20 percent of the District's raw sludge is being prepared.

INTRODUCTION

The Metropolitan Sanitary District of Greater Chicago (District) service area encompasses almost all of Cook County, Illinois, which extends over 2,227 km^2 (860 sq miles) and includes the city of Chicago and more than 120 suburban communities. Wastewater is collected and treated from a domestic population of 5.5 million people. Added to this domestic loading, the nondomestic and combined sewer load contribute an additional load equivalent of 5.5×10^6 people, bringing the total population equivalent served to 11×10^6. Approximately 635 megagrams/day (700 tons/d) of organic solids are processed for disposal.

Municipalities through the years have attempted to dispose of their recovered wastewater solids through various combinations of long-term storage in lagoons or stockpiles, landfilling, ocean disposal, and incineration. The relatively recent public awareness of such environmental issues as air quality, land usage, and energy conservation is forcing a closer look at this age-old problem--how to effectively process and dispose of wastewater solids without causing environmental damage.

During the past 50 years, while significant advances have been made in the field of wastewater treatment and solids processing, the area of solids disposal is just beginning to be explored intensively. No matter how much processing is given to the recovered wastewater solids, there remains some fraction that must be disposed of or utilized.

Faced with increasing solids production rates, escalating fuel costs, and limited land availability for disposal, the Metropolitan Sanitary District of Greater Chicago Board of Commissioners in the late 1960s adopted a policy of solids utilization. Today, the District believes that use rather than disposal of the recovered solids from wastewater treatment represents one of the best management alternatives.

The District has always endeavored whenever economically justified to adopt the concept of solids utilization into its sludge management programs. In this paper, the present and some of the past sludge management techniques used by the District will be discussed.

AGRICULTURAL USE - FOOD CHAIN CROPS

HANOVER PARK LAND APPLICATION SITE

The District's Hanover Park Water Reclamation Plant (WRP) located in the Village of Hanover Park, Illinois, treats the wastewater from a 31-km^2 (12-mi^2) area which includes portions of 4 villages in the northwestern part of Cook County, Illinois. In 1981, an average raw sewage flow of 0.35 m^3/s (8.10 MGD) was treated.

In order to make the Hanover Park WRP a totally independent treatment facility, a 48.5-ha (120-acre) farm was constructed in 1978 to utilize all

the sludge generated. Sixteen fields were developed thusly:

1. Each field was graded to have a gentle slope so liquid sludge could be applied at the high end through gated irrigation pipe;

2. Fields have terraces and dikes to divert surface runoff to the lower end of each field; and

3. A tile drainage system in each field collects infiltration and surface runoff and conveys this water back to the Hanover WRP.

In the winter when no sludge application is made to fields, digested sludge from the WRP is stored in two lagoons. During the application season, both the lagooned sludge and digester drawoff are applied, so that by the end of the application season, ideally the storage lagoons are empty.

Sludge is pumped to the farm fields through a buried pipeline to sixteen distribution points (one in each field) to which the above ground slotted irrigation piping is connected.

The location of the plant and farm are shown in Figure 1 which also shows the proximity of residential housing.

Sludge application began in 1979 when nine of the sixteen fields received sludge. Table 1 presents the sludge application quantities for each field in 1979, 1980, and 1981. Total dry megagrams (Mg) applied each year were 127 (140 tons), 550 (607 tons), and 822 (906 tons) for 1979, 1980, and 1981, respectively. By 1981, the Hanover Park WRP farm was more than able to accommodate the total annual sludge production from the WRP.

Digester output during 1981 totaled 784.5 dry Mg (865 tons) or 2.14 dry Mg/d (2.37 tons/day). During 1981, a total of 557 Mg (614 dry tons) of digested sludge at 2.0 percent solids and 265 dry Mg (292 dry tons) of lagoon sludge at 1.6 percent solids were applied to the Hanover Park WRP farm fields.

Table 2 shows the mean chemical analysis of the sludge (digester drawoff and lagooned sludge) applied to the fields for the first three years of operation (1979-1981).

The fields are planted each year to either corn or grass. During the first year of application (1979) all fields were planted to grass. In 1980 and 1981, several fields were planted to corn with averaged yields of 53.6 hectoliters/ha (61.6 bu/acre) in 1980 and 13.4 hectoliters/ha (15.5 bu/acre) in 1981. Wet conditions were responsible for the low yield in 1981. All harvested corn is sold to a local grain elevator.

Grain samples from each field were analyzed for total metal content. These results are shown in Table 3 for the 1980 grain and Table 4 for the 1981 grain. As Tables 3 and 4 show, Cd content of corn grain from the Hanover Park fields ranged from <0.02 (6 samples) to 0.13 mg/kg.

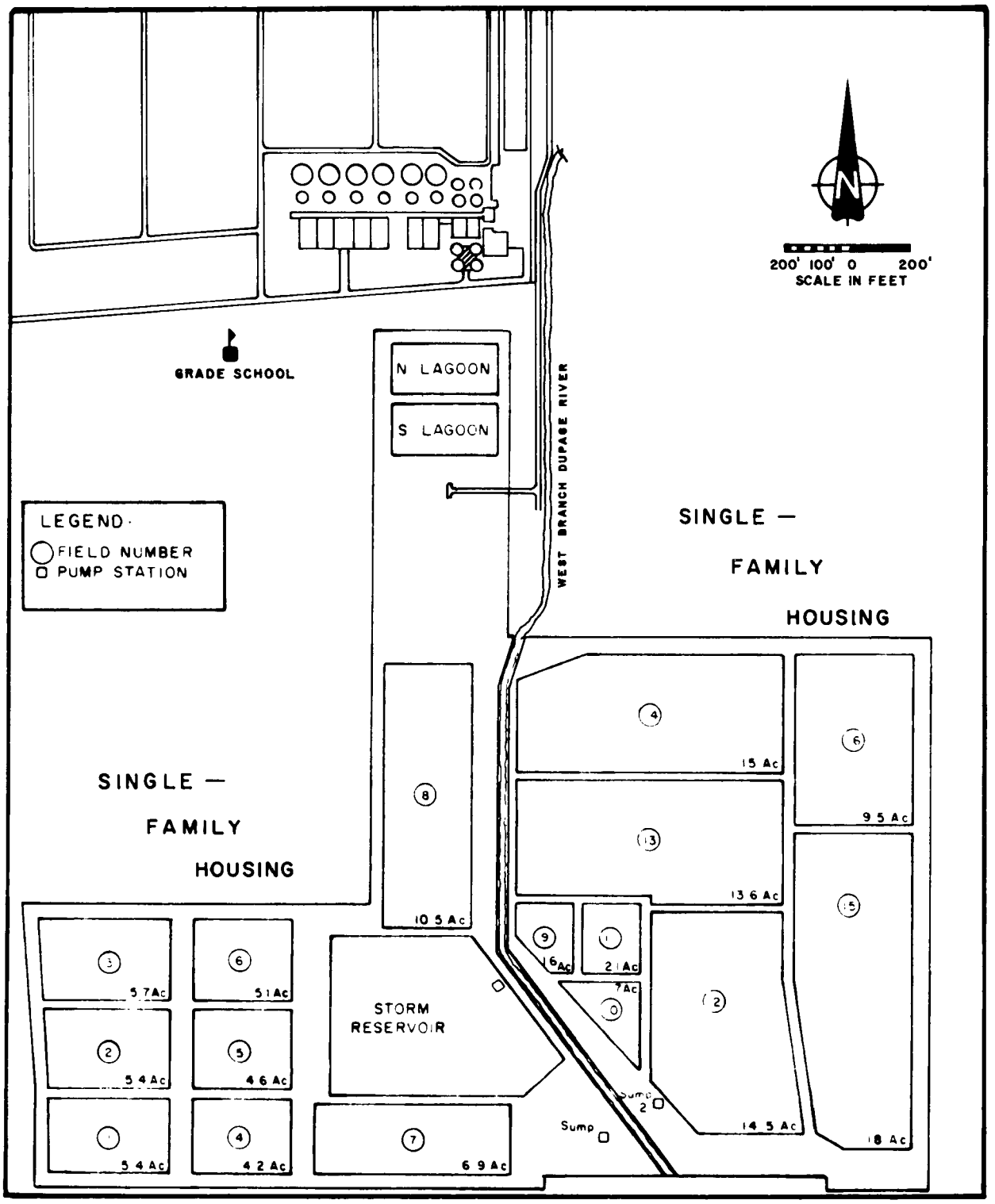


Figure 1. Location of fields utilized for sludge application at the Hanover Park Water Reclamation Plant.

TABLE 1. HANOVER PARK WATER RECLAMATION PLANT FARM SLUDGE APPLICATION FOR 1979 THROUGH 1981, IN DRY MEGAGRAM OF SOLIDS PER HECTARE

Field	1979*	1980†	1981‡	Total 1979-1981§
	Mg/ha			
1	2.7	11.0	26.4	40.1
2	0	4.9	0	4.9
3	0	5.8	18.8	24.6
4	0	5.8	50.2	56.0
5	2.0	2.0	0	4.0
6	11.6	11.6	7.4	30.7
7	8.7	6.9	0	15.7
8	0	4.3	23.3	27.6
9	7.4	32.5	33.6	73.5
10	0	14.3	14.6	28.9
11	7.6	0.0	13.9	21.5
12	0	15.0	34.0	49.1
13	0.7	15.7	4.5	20.8
14	2.5	6.9	17.9	27.3
15	0	21.7	9.9	31.6
16	9.9	5.6	18.8	24.4

*In 1979, the total solids application was 127 Mg.
†In 1980, 550 Mg.
‡In 1981, 822 Mg.
§The cumulative total for 1979-1981 was 1,499 Mg.

Costs associated with the 48.5-ha (120-acre) Hanover Park WRP farm include land purchase of $1,675,000 in 1970, construction (including flood control system) of $3,299,800 in 1971, and annual operations and maintenance (O and M) costs as follows:

1) 1979, $74,832 expended, or $593 per Mg ($538/dry ton) applied.

2) 1980, $64,000 expended, or $116 per Mg ($105/dry ton) applied.

3) 1981, $69,882 expended, or $84.9 per Mg ($77/dry ton) applied.

These above unit cost figures indicate that the O and M cost has decreased as experience was gained and greater quantities of solids were applied (Table 1) to the farm fields.

The Hanover Park WRP farm represents a sludge application to land project in extremely close proximity to nearby single-family residences

TABLE 2. ANALYSIS OF THE SLUDGE APPLIED TO FARM FIELDS AT THE HANOVER PARK WATER RECLAMATION PLANT FROM 1979 THROUGH 1981. RESULTS ARE BASED ON TWENTY-ONE MONTHLY DIGESTER DRAWOFF COMPOSITES AND FOURTEEN MONTHLY LAGOON SLUDGE COMPOSITES.

Constituent	Digester Drawoff			Lagoon Sludge		
	Mean	Min.	Max.	Mean	Min.	Max.
	----------%----------					
Total solids	2.07	1.6	2.8	2.47	0.13	4.8
Total volatile solids	60.8	57.4	65.6	51.8	40.4	57.6
	----------kg/Mg solids----------					
Total P	18.6	5.8	33.3	16.6	3.44	23.5
N-Kjeldahl	65.8	56.6	83.2	51.7	39.2	89.5
N-NH_3	24.8	15.8	35.0	20.5	8.19	60.5
Al	8.42	2.92	12.0	7.48	0.77	12.8
Ca	35.2	19.2	47.5	37.5	10.5	70.0
Cd	0.058	0.009	0.214	0.063	0.015	0.243
Cr	1.37	0.34	2.44	0.77	0.19	1.07
Cu	1.96	0.64	2.58	1.48	0.23	2.05
Fe	11.7	4.50	17.5	15.6	3.46	18.6
K	2.94	2.40	3.88	5.79	2.16	20.0
Mg	7.76	1.75	11.8	15.1	4.22	38.5
Mn	0.63	0.26	0.88	0.46	0.18	0.58
Na	9.41	6.43	12.5	15.7	3.75	83.1
Ni	0.20	0.08	0.37	0.15	0.08	0.28
Pb	0.19	0.08	0.24	0.22	0.03	0.27
Zn	1.27	0.56	1.84	1.81	0.38	2.98
	---------- g/Mg solids----------					
Hg	5.25	1.87	7.48	4.97	0.85	10.8

(Figure 1). Some homes are within 30 m (100 ft) of sludge application fields. The District has found that the public accepts the fact that a sewage sludge farm is located almost in their backyards. Complaints from neighbors have been minimal and in some years no complaints at all are received by plant operating personnel. The Hanover Park WRP farm shows that a well-managed and properly designed sludge application to land site is acceptable to the general public.

TABLE 3. SELECTED METALS ANALYSIS OF CORN GRAIN GROWN IN 1980 ON FARM FIELDS AT THE HANOVER PARK WATER RECLAMATION PLANT. VALUES ARE MEANS OF DUPLICATE SAMPLES.

Field	Zn	Cd	Cu	Cr	Fe	Ni	Pb	K	Ca	Mg	Mn
	mg/kg (oven-dried, 65°C)										
2	17.9	<0.02	4.63	1.51	68.8	0.50	0.09	3,539	221.0	1,471	4.57
3	20.5	<0.02	2.03	0.84	41.5	0.26	0.04	3,966	59.8	1,435	4.58
5	20.5	<0.04	1.57	1.06	51.0	0.34	0.04	3,922	67.7	1,453	4.98
6	18.7	<0.02	1.47	0.62	44.4	0.34	<0.01	3,584	53.8	1,254	4.58
7	19.4	<0.02	1.42	1.82	20.8	0.34	<0.04	3,678	40.0	1,519	5.60
8	19.2	<0.02	1.74	1.99	49.5	0.52	0.03	3,731	108.0	1,516	5.39
14	18.3	<0.02	1.89	2.24	63.2	0.44	0.19	4,151	107.0	1,728	5.16
16	19.2	<0.02	1.98	1.92	92.2	0.40	0.09	3,751	172.0	1,556	7.38

TABLE 4. SELECTED METALS ANALYSIS OF CORN GRAIN GROWN IN 1981 ON FARM FIELDS AT THE HANOVER PARK WATER RECLAMATION PLANT. VALUES ARE MEANS OF DUPLICATE SAMPLES.

Field	Zn	Cd	Cu	Cr	Fe	Ni	Pb	K	Ca	Mg	Mn
	mg/kg (oven dried, 65°C)										
3	14.6	0.03	1.03	0.30	12.8	1.01	0.09	5,999	57.2	1,579	4.54
6	12.6	0.04	1.18	0.35	13.6	1.12	0.06	7,212	59.1	1,261	3.94
14	17.2	0.05	1.90	0.26	18.0	1.05	0.45	7,314	69.4	1,645	5.95
16	20.7	0.13	1.85	0.20	20.1	1.24	0.08	7,771	61.2	1,479	3.94

FULTON COUNTY LAND APPLICATION SITE

The District's Fulton County project is the embodiment of the concept of the agricultural utilization of sewage sludge as a sludge management alternative. As at the Hanover Park WRP farm site, sludge is applied as a fertilizer; but at the Fulton County site, this sludge utilization operation has the added benefit of reclaiming previously surface-mined land.

Fulton County is located in West Central Illinois, approximately 300 km (200 mi) southwest of Chicago. From the 1920's to the 1960's, 16,000 ha (40,000 acres) of former row crop land in Fulton County was strip-mined for coal before the enactment of reclamation laws. The gently rolling fields of fertile prairie soil were replaced by a corduroy topography of steep spoil mounds and long, narrow lakes. While many strip-mined areas are characterized by acidic soils and surface waters, Fulton County has neutral to alkaline calcareous soils. This characterization increases the suitability of the site for reclamation by sludge application, as the availability of metals for plant uptake is substantially reduced in an alkaline environment.

Description of the Site

In 1970, the District initially purchased 2,898 ha (7,156 acres) of land in Fulton County and currently owns 6,289 ha (15,528 acres). Over 60 percent of the site had been strip-mined and was being used for livestock pasture. It is the District's intention to reclaim the strip-mined spoil areas so that they can be suitable again for row-crop agriculture. This will be accomplished by regrading of the land and increasing the nutrient and organic matter content of the spoil by additions of sludge solids.

To store the sludge that would be barged down the Illinois River from Chicago, holding basins were constructed. In 1971 two basins, each lined with a two-foot layer of compacted clay, were formed with a combined capacity of approximately 3.8 million cubic meters (5,000,000 yd^3). During 1973, holding basins #3A and #3B with a combined capacity of 2.3 million m^3 (3,000,000 yd^3) were constructed to store the supernatant from the other two basins.

During this same period, extensive regrading was done to form fields suitable for sludge application and row-crop agriculture. Each application field was bermed, and drains to a runoff retention basin designed to capture the regional 100-year storm event of 15.5 cm (6.1 in). The runoff water is retained until it meets the State of Illinois permit standards which are currently TSS$\leq$99 mg/L, BOD$\leq$33 mg/L,[1] and fecal coliform$\leq$494 counts/100 ml. Extensive surface-water and groundwater environmental monitoring operations were also initiated at the beginning of the project. Figure 2 shows an overview sketch of the Fulton County site and Figure 3 a typical application field. By 1981, 110 fields of approximately 2,300 ha (5,700 acres) were being utilized for row-crop or forage production.

[1]TSS ≡ Total suspended solids; BOD ≡ Biochemical oxygen demand.

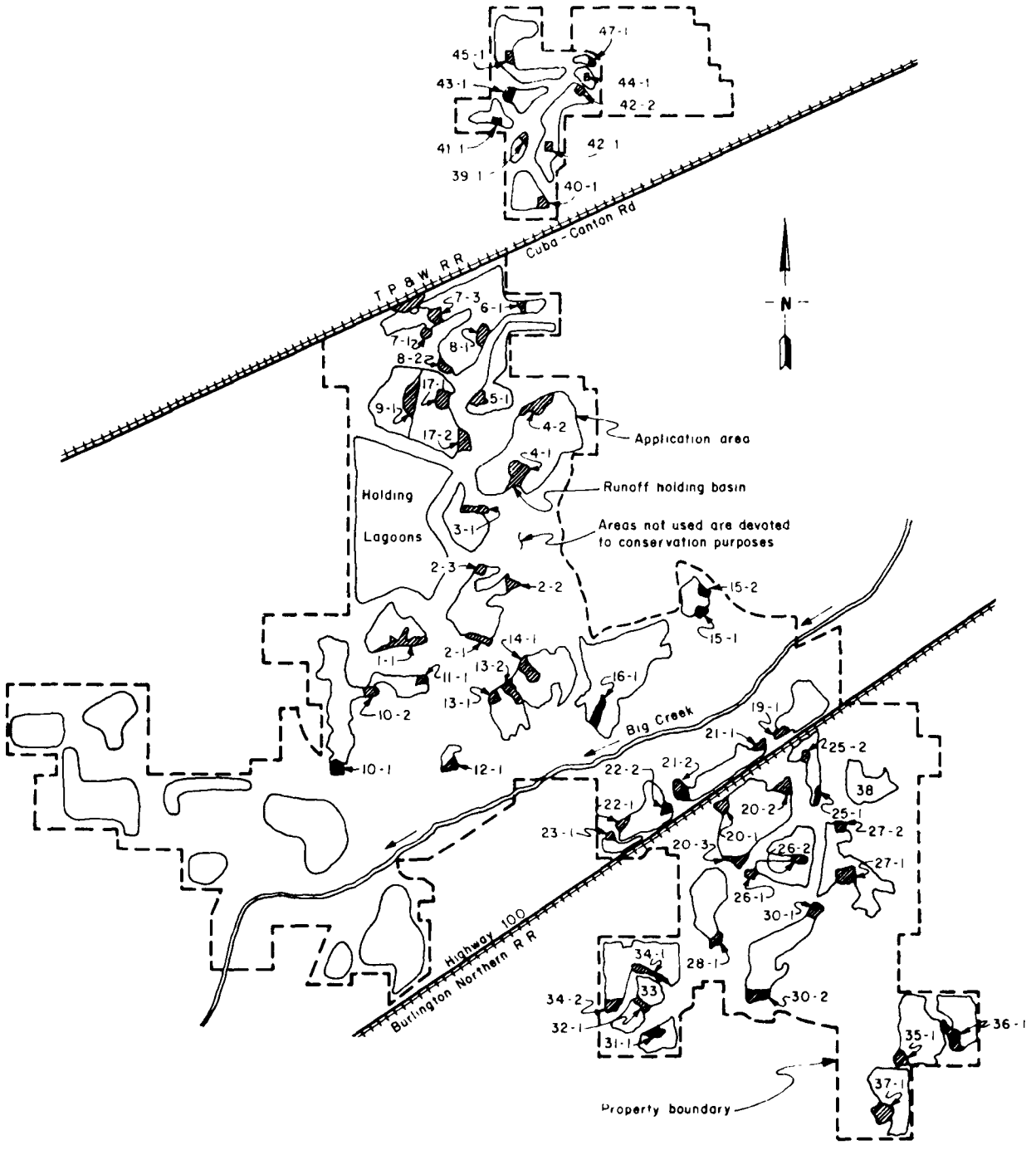


Figure 2. Farm fields and runoff basins at Fulton County, Illinois.

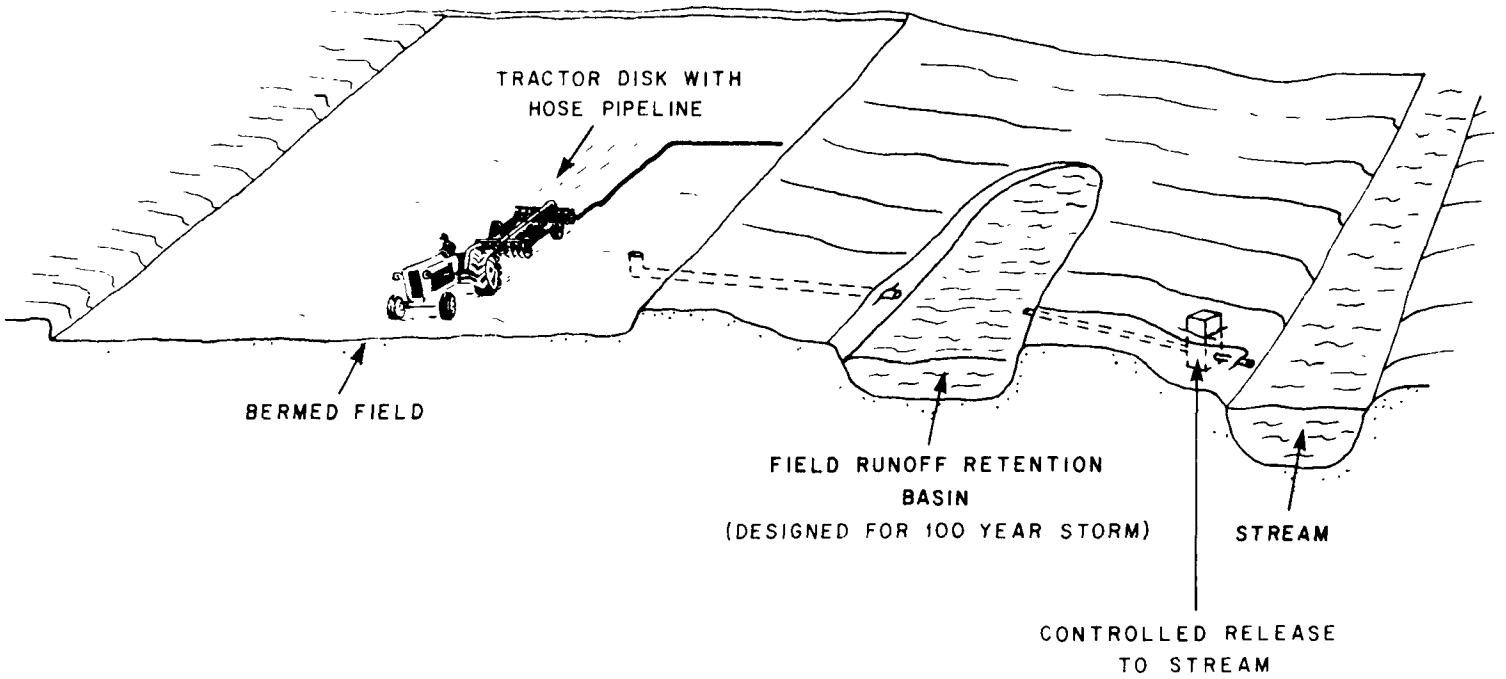


Figure 3. Typical field design with runoff water capture system at Fulton County, Illinois.

Operation of the Fulton County Site

The site became operational in August of 1971 when the first barge of liquid sludge docked in Liverpool, Illinois. From 1971 to 1981 an average of 1.27 million wet Mg (1.4 million tons) of sludge at 4-6 percent total solids have been barged to the Fulton County site each year from the District's West-Southwest Sewage Treatment Works (STW). The sludge shipped to Fulton County is normally drawn from heated (35°C) anaerobic high-rate digesters at the West-Southwest STW treating waste-activated sludge and a small percentage of primary sludge. This digested sludge had also been stored at the Lawndale Avenue lagoons at the West-Southwest STW for many years; and during the summer, barges can be filled with lagooned sludge or with a mixture of both lagooned and freshly digested sludge. Since 1981 when a new centrifuge facility at the West-Southwest STW became operational, the sludge barged has been a mixture of liquid polymer-dosed centrifuged digested sludge (15 percent TS) and digested sludge (3-4 percent TS) blended to 6-8 percent total solids.

From the barge dock at Liverpool, Illinois, the sludge is pumped 17 km (10.4 mi) through a pipeline to the holding basins. During warm weather, the sludge is pumped from the basins via a surface pipe distribution system to the fields. At the fields, the methods of sludge application have continually evolved over the years. Initially, spraying by a traveling sprinkler was the main method; but it was totally replaced by 1977 by incorporation of the sludge-using tractor-drawn tandem-disk incorporators with a sludge distribution manifold, which directs sludge to each disk blade as it tills the soil. Incorporation became the method of choice for the following reasons: (1) a greater amount of sludge can be applied in a shorter period of time; (2) applications are more uniform as varying winds are not a factor; (3) the reduced visibility of the sludge is aesthetically more acceptable; and (4) the potential for odors is greatly reduced.

Since 1980, dewatered sludge solids at 55 percent from the holding basins have been hauled to fields by means of a hydraulic ram ejector truck and self-loading scraper. A bulldozer then spreads the sludge, which is then incorporated with a heavy disk.

The supernatant from the settling of the sludge in the holding basins has been applied as an irrigant and fertilizer since 1977 to pasture fields via gated pipe. An approximate average rate of 1,000,000 liters per hectare (100,000 gal/acre) has been applied each year.

Sludge Quality and Application Rates

From 1972 to 1981 an average of 1.1 million wet Mg (1.3 million tons) of liquid sludge have been applied each year to the Fulton County farm fields. This sludge is typified by the 1981 analysis given in Table 5. The sludge is well-stabilized at 46.9 percent volatile solids. One dry megagram contains 28 kg total phosphorus and 46 kg total Kjeldahl nitrogen. Sludge has been applied at an approximate yearly rate of 50 dry Mg/ha (22 tons/acre) to minespoil fields. Since sludge is being applied for reclamation, the rates have been higher than typical agronomic rates based on crop nitrogen requirement.

TABLE 5. MEAN VALUES AND RANGE OF THE PRINCIPAL CONSTITUENTS OF THE SLUDGE APPLIED AS FERTILIZER TO FULTON COUNTY FARM FIELDS FROM MAY 26 TO OCTOBER 30, 1981 AND THE MEAN CONTENT PER DRY MEGAGRAM. RESULTS ARE BASED ON TWENTY WEEKLY COMPOSITE SAMPLES

	Minimum	Maximum	Mean	Mean Content Per Dry Mg	
pH	7.4	7.9	7.6	-	
EC, dS/m	3600	5800	4975	-	
	----------mg/L----------			kg/Mg	(lb/ton)
Total P	724	2020	1455	27.9	(55.7)
N-Kjeldahl	1387	2888	2398	45.9	(91.9)
$N-NH_3$	542	1428	1038	19.9	(39.8)
Alk. as $CaCO_3$	3270	4660	3790	73.0	(145.0)
Cl	264	508	339	6.5	(13.0)
Fe	1900	3740	2351	45.0	(90.1)
Zn	133	259	181	3.47	(6.93)
Cu	77.8	135	95.6	1.83	(3.66)
Ni	18	26	22	1.68	(0.84)
Mn	19.1	33.0	25.1	0.48	(0.96)
K	8	50	30	0.6	(1.1)
Na	7	31	18	0.3	(0.7)
Mg	470	810	647	12.4	(24.8)
Ca	1120	3950	1854	35.5	(71.0)
Pb	31.1	66.0	45.0	0.86	(1.72)
Cr	114	241	163	3.12	(6.24)
Cd	8.0	17.9	12.4	0.24	(0.48)
Al	520	830	662	12.7	(25.4)
Hg	0.104	0.533	0.311	0.006	(0.012)
	----------%----------				
Total solids	4.40	6.00	5.22	1000.00	(2000.00)
Total volatile solids	44.6	49.3	46.9	469.00	(938.00)

Table 6 shows the annual application rates and cumulative applications of sludge after ten years to the 21 minespoil fields that have grown at least one crop of corn. An average of 477 dry Mg/ha (213 tons/acre) of sludge have been applied as of the end of 1981.

Effect of Sludge Application on Mine-Spoil

In the spring of each year prior to sludge application, the plow layer (0-15 cm) of soil is sampled and analyzed. Table 7 presents the yearly averages and ranges of pH, electrical conductivity (EC) and organic carbon found for the same 21 sludge-amended mine-spoil fields listed in Table 6.

TABLE 6. ANNUAL AND CUMULATIVE SLUDGE APPLICATION RATES FOR MINE-SPOIL FIELDS THAT HAVE GROWN CORN AT THE FULTON COUNTY, ILLINOIS LAND RECLAMATION SITE FOR 1972-1981

Field	1972	1973	1974	1975	1976	1977	1978	1979	1980	1981	TOTAL
	dry Mg/ha										
1	0	0	43	65	121	0	116	0	94	52	491
2	0	0	54	41	129	83	36	107	51	60	560
3	13	3	55	23	83	0	83	14	90	55	419
4	0	1	55	86	117	67	0	94	48	56	524
5	0	1	52	90	81	0	53	75	73	70	495
7	0	1	48	50	13	64	59	0	64	68	368
8	0	2	27	57	72	0	64	58	76	58	414
9	5	1	37	15	46	28	67	0	93	54	345
11	0	0	66	79	119	0	109	105	60	63	600
12	0	0	38	43	61	58	0	91	26	150	467
13	0	0	36	33	40	81	44	82	74	67	457
14	0	0	15	30	23	64	75	20	85	133	445
15	0	0	65	67	133	91	19	130	70	47	622
17	5	4	43	47	34	84	19	120	26	54	436
25	0	0	22	65	0	112	85	27	85	41	438
26	0	0	61	28	98	108	88	17	97	57	555
28	0	0	39	82	95	0	79	135	37	68	535
30	0	0	45	43	101	126	23	115	104	33	590
32	0	0	8	49	64	0	75	50	16	45	308
33	0	0	9	69	91	0	98	124	32	74	497
47	0	0	0	0	46	85	90	88	74	65	448
$\bar{X}w$*	8	2	41	53	78	81	67	81	65	65	477

*The weighted mean ($\bar{X}w$) is the mean of only those fields that received sludge that year.

TABLE 7. MEANS (X) AND RANGES OF pH, ELECTRICAL CONDUCTIVITY (EC) AND ORGANIC CARBON FOUND IN SOIL (0-15 cm) FROM 21 SLUDGE-AMENDED MINE-SPOIL CORN FIELDS AT THE FULTON COUNTY, ILLINOIS ALND RECLAMATION SITE FROM 1972-1981

Year	pH				EC* (dS/m)				Organic C† (%)			
	$\bar{X}$	N‡	Min.	Max.	$\bar{X}$	N	Min.	Max.	$\bar{X}$	N	Min.	Max.
1972	7.54	11	7.20	7.80	1.47	11	0.45	3.00	0.67	22	0.24	1.56
1973	7.70	6	7.50	7.90	1.30	6	0.67	2.50	1.05	6	0.78	1.25
1974	7.65	40	6.50	8.40	1.53	40	0.16	3.40	0.67	52	0.16	1.80
1975	7.11	42	6.50	8.00	2.07	42	0.86	4.60	0.86	84	0.28	1.81
1976	7.14	40	6.10	7.55	1.85	40	0.50	3.50	1.24	80	0.59	2.90
1977	6.79	42	6.30	7.30	2.87	42	0.74	4.80	2.07	84	0.81	3.29
1978	6.80	42	6.19	7.18	2.04	42	0.49	3.60	3.07	84	1.96	4.88
1979	6.70	60	5.26	7.41	2.86	42	0.99	4.50	3.53	84	1.89	5.83
1980	6.64	42	5.59	7.18	2.65	42	0.60	4.40	3.85	84	2.02	5.40
1981	6.49	42	6.03	6.94	2.28	42	0.64	3.68	4.60	84	2.35	6.59

*Determined on 1:1 soil/water solution.
†Determined according to Walkley-Black method (1).
‡= Number of analyses.

Over a ten-year period, the pH has decreased from 7.54 to 6.49; EC has increased from 1.47 to 2.28 dS/m; and organic C has increased from a low 0.67 percent in 1972 to 4.60 percent in 1981. If the pH of a farm field gets below 6.5, agricultural limestone is applied to keep soil pH above this value.

For comparison, Table 8 shows the results from four mine-spoil fields that did not receive sludge application, although supernatant was applied one year (1979 or 1980). These fields were inorganically fertilized with commercial N-P_2O_5-K_2O at a rate of 140-112-112 kg/ha (125-100-100 lb/acre) for a corn crop. As these fields were not put into service until 1976, the table only presents data from 1976 to 1981. The three parameters have all shown a slight increase over six years. The pH has increased from 7.03 to 7.29; EC from 1.78 to 2.05 dS/m; while organic carbon has increased only 0.36 percent from 0.68 to 1.04 percent, likely due to incorporation of crop residues.

The application of sludge has increased the fertility of the mine-spoil field soil as measured by nitrogen (N), phosphorus (P), and potassium (K) levels. Table 9 gives the yearly means and ranges of these elements for the sludge-amended mine-spoil fields. The available mineral N, as measured by exchangeable NH_4-N and NO_3+NO_2-N, was 21 mg/kg in 1972 and 189 mg/kg in 1981. Available P has increased from 24 mg/kg to 299 mg/kg of soil in 1981. The exchangeable K has not increased as dramatically since the sludge is not high in K. For example, the 1981 sludge applied contained 0.06 percent K. Exchangeable K increased from 122 mg/kg in 1972 to 326 mg/kg in 1981.

In comparison, the fertility of the inorganically fertilized mine-spoil as measured by the same three parameters has not changed much. Table 10 shows a change of 7.3 to 34 mg/kg available mineral N, 7.5 to 12 mg/kg available P, and 98 to 220 mg/kg exchangeable K from 1976 to 1981.

In addition to the essential plant nutrients just discussed and other essential trace elements, such as zinc, copper and iron, sewage sludge also contains such metals as cadmium (Cd), Nickel (Ni), and lead (Pb). The sludge fertilizer applied to fields from 1974 to 1981 has mean concentrations of 290 mg Cd, 410 mg Ni and 900 mg Pb per dry kg of sludge. Using these concentrations with the average cumulative sludge application of 477 Mg/ha, approximately 138 kg Cd, 196 kg Ni and 429 kg Pb per ha have been applied to an average mine-spoil field after ten years. Assuming that the 15-cm plow layer of soil weighs 2.24 million kg (2,000,000 lb/acre-6 inch), concentrations expected to be found with the above metal loadings would be 62 mg Cd, 88 mg Ni and 192 mg Pb per kg of soil. Table 11 gives the yearly means and ranges of 0.1 N HCl extractable Cd, Ni, and Pb. The average concentrations found in 1981 of 60.1 mg Cd, 56.7 mg Ni and 175 mg Pb/kg compare favorably with the above estimates. As the soil analysis is an extraction and not a total digestion, lower values would be expected. Table 11 also shows the yearly increases in concentration of the three elements Cd, Pb and Ni. As would be expected, concentrations of these elements in mine-spoil fields with commercial fertilization have not changed over the years as shown in Table 12. Any differences are probably due to sampling and analytical variability.

TABLE 8. MEANS (X) AND RANGES OF pH, ELECTRICAL CONDUCTIVITY (EC) AND ORGANIC CARBON FOUND IN SOIL (0-15 cm) FROM FOUR INORGANICALLY FERTILIZED CORN FIELDS AT THE FULTON COUNTY, ILLINOIS LAND RECLAMATION SITE FROM 1976-1981

	pH*				EC*(dS/m)				Organic C† (%)			
Year	$\bar{X}$	N‡	Min.	Max.	$\bar{X}$	M	Min.	Max.	$\bar{X}$	N	Min.	Max.
1976	7.03	6	6.75	7.45	1.78	6	0.27	3.00	0.68	12	0.51	0.80
1977	6.77	2	6.47	7.07	0.18	2	0.16	0.19	0.62	4	0.41	0.81
1978	7.35	8	7.04	7.65	1.53	8	0.33	2.80	0.77	16	0.51	1.15
1979	7.67	8	7.47	8.07	1.32	8	0.26	2.66	0.77	16	0.60	0.94
1980	7.40	8	7.17	7.64	1.71	8	0.40	2.98	0.86	16	0.64	1.10
1981	7.29	8	7.01	7.41	2.05	8	0.34	3.08	1.04	16	0.84	1.33

*Determined on 1:1 soil/water solution.
†Determined according to Walkley-Black method (1).
‡N = Number of analyses.

TABLE 9. MEANS ($\bar{X}$) AND RANGES OF AVAILABLE MINERAL NITROGEN AND PHOSPHORUS AND EXCHANGEABLE POTASSIUM FOUND IN SOILS (0-15 cm) FROM 21 SLUDGE-AMENDED MINE-SPOIL CORN FIELDS AT THE FULTON COUNTY, ILLINOIS LAND RECLAMATION SITE FROM 1972-1981

Year	Mineral N* (mg/kg)				Available P† (mg/kg)				Exchangeable K‡ (mg/kg)			
	$\bar{X}$	N§	Min.	Max.	$\bar{X}$	N	Min.	Max.	$\bar{X}$	N	Min.	Max.
1972	21	22	0	69	24	22	11	33	122	22	98	173
1973	32	12	14	59	13	12	8	18	92	12	79	102
1974	14	80	0	123	10	52	0	22	72	48	26	121
1975	54	84	4	255	23	84	7	89	130	84	71	222
1976	63	80	29	119	59	80	10	135	122	80	66	205
1977	146	84	24	360	146	84	46	320	171	84	103	563
1978	43	82	16	136	214	84	108	400	219	84	104	527
1979	217	84	23	530	393	84	194	617	225	84	103	421
1980	179	84	49	418	269	84	170	347	281	84	148	454
1981	189	84	63	370	299	84	188	403	326	84	112	469

*Available mineral N consists of exchangeable NH_4-N and NO_3+NO_2-N extracted with 2N KCl according to Bremner (2).

†Available phosphorus was determined by extraction with 0.03 N NH_4F + 0.025 N HCl (3).

‡Exchangeable potassium was determined by extraction with 1N NH_4OAC (4).

§N = Number of analyses.

TABLE 10. MEANS ($\bar{X}$) AND RANGES OF AVAILABLE MINERAL NITROGEN AND PHOSPHORUS AND EXCHANGEABLE POTASSIUM FOUND IN SOILS (0-15 cm) FROM FOUR INORGANICALLY FERTILIZED CORN FIELDS AT THE FULTON COUNTY, ILLINOIS LAND RECLAMATION SITE FROM 1976-1981

	Mineral N* (mg/kg)				Available P† (mg/kg)				Exchangeable K‡ (mg/kg)			
Year	$\bar{X}$	N§	Min.	Max.	$\bar{X}$	N	Min.	Max.	$\bar{X}$	N	Min.	Max.
1976	7.3	12	3.2	12	7.5	8	0.4	20	98	12	80	132
1977	28.0	4	25.0	32	7.1	3	0.4	16	81	4	76	88
1978	16.0	16	8.8	37	3.5	16	0.4	16	125	16	104	149
1979	11.0	16	6.1	15	27.0	16	7.5	56	110	16	39	220
1980	43.0	16	9.1	80	20.0	16	7.7	43	196	16	112	326
1981	34.0	16	12.0	72	12.0	16	0.4	40	220	16	143	428

*Available mineral N consists of exchangeable NH_4-N and NO_3+NO_2-N extracted with 2N KCl according to Bremner (2).
†Available phosphorus was determined by extraction with 0.03 N NH_4F + 0.025 N HCl (3).
‡Exchangeable potassium was determined by extraction with 1N NH_4OAC (4).
§N = Number of analyses.

TABLE 11. MEANS ($\bar{X}$) AND RANGES OF EXTRACTABLE* CADMIUM, NICKEL AND LEAD FOUND IN SOIL (0-15 cm) FROM 21 SLUDGE-AMENDED MINE-SPOIL CORN FIELDS AT THE FULTON COUNTY, ILLINOIS LAND RECLAMATION SITE FROM 1972-1981

Year	Cd (mg/kg)				Ni (mg/kg)				Pb (mg/kg)			
	$\bar{X}$	N†	Min.	Max.	$\bar{X}$	N	Min.	Max.	$\bar{X}$	N	Min.	Max.
1972	0.23	22	0.03	1.50	6.7	22	3.2	10.0	3.63	22	0.33	8.50
1973	1.00	8	0.33	1.52	6.4	8	3.0	10.0	2.44	8	0.33	6.12
1974	3.19	65	0.23	21.4	6.6	54	3.0	10.3	5.58	54	3.97	7.24
1975	5.91	84	0.13	13.5	10.2	84	1.6	19.1	16.1	84	5.70	35.6
1976	11.2	80	3.36	26.2	13.4	80	5.3	29.6	27.0	80	10.8	59.8
1977	19.3	84	7.43	42.7	18.7	84	6.0	35.5	50.2	84	16.8	114.0
1978	39.5	84	21.2	68.3	37.2	84	17.8	54.7	101.0	84	63.0	151.0
1979	46.3	84	23.0	133.0	41.5	84	19.8	111.0	136.0	84	78.3	219.0
1980	49.2	84	23.4	72.0	41.6	84	16.4	69.9	129.0	84	53.9	189.0
1981	60.1	84	30.7	73.5	56.7	84	35.2	70.2	175.0	84	110	215.0

*Metals were determined by extraction with 0.1 N HCl (5).
†N = Number of analyses.

TABLE 12. MEANS ($\bar{X}$) AND RANGES OF EXTRACTABLE* CADMIUM, NICKEL AND LEAD FOUND IN SOILS (0-15 cm) FROM FOUR INORGANICALLY FERTILIZED CORN FIELDS AT THE FULTON COUNTY, ILLINOIS LAND RECLAMATION SITE FROM 1976-1981

Year	Cd (mg/kg)				Ni (mg/kg)				Pb (mg/kg)			
	$\bar{X}$	N†	Min.	Max.	$\bar{X}$	N	Min.	Max.	$\bar{X}$	N	Min.	Max.
1976	0.52	12	0.22	0.98	11.0	12	4.3	16.5	4.69	12	2.32	6.37
1977	0.43	4	0.26	0.67	3.3	4	3.3	3.3	3.58	4	3.10	4.02
1978	0.41	16	0.05	1.32	7.0	16	4.3	10.1	5.86	16	0.99	7.65
1979	0.40	16	0.33	0.95	6.7	16	3.3	9.9	5.59	16	0.33	15.50
1980	0.69	16	0.33	1.54	6.4	16	3.1	14.5	1.18	16	0.33	4.16
1981	0.39	16	0.33	0.97	6.5	16	3.2	11.6	3.95	16	0.33	6.62

*Metals were determined by extraction with 0.1 N HCl (5).
†N=Number of analyses.

Effect of Sludge Application on Corn

As the major agricultural crop in Illinois, corn is grown at the Fulton County site along with other crops such as soybeans, wheat and forage. A three-year field rotation schedule is generally used--for two years, sludge is applied; and in the third year a crop is grown to utilize the available N and provide economic return. Corn yields have been variable as shown in Table 13 and 14 for the sludge-amended and non-amended mine-spoil fields. Yearly yield averages of sludged corn fields ranged from 0.7 to 6.1 Mg/ha (11 to 97 bu/acre). No corn was grown on inorganically fertilized mine-spoil until 1977.

TABLE 13. CORN GRAIN YIELDS FROM SLUDGE-AMENDED MINE-SPOIL FIELDS AT THE FULTON COUNTY, ILLINOIS LAND RECLAMATION SITE FROM 1972-1981

Field	1972	1973	1974	1975	1976	1977	1978	1979	1980	1981
	Mg/ha* (15.5% moisture)									
1						0.9		3.7		
2				3.0			1.4			6.2
3	3.3	3.0	0.6	4.0		2.0		5.7		
4							2.2			
5						1.2			1.9	
7					1.6			3.0		
8						1.3			1.3	
9	2.7	2.4								
11						0.7			0.6	
12							1.8			
13			0.4						2.5	
14								5.1		
15			1.0				1.9			
17	2.3	2.1								
25					2.4			4.1		
26				2.8				4.2		
28						0.7			0.7	
30				1.6			2.0			
32									1.3	
33									0.6	
47										6.0
$\bar{X}$	2.8	2.5	0.7	2.8	2.0	1.1	1.9	4.3	1.3	6.1

*Mg/ha x 15.7 = bu/acre.

TABLE 14. CORN GRAIN YIELDS FROM INORGANICALLY-FERTILIZED MINE-SPOIL FIELDS AT THE FULTON COUNTY, ILLINOIS LAND RECLAMATION SITE FROM 1972-1981

Field	1972	1973	1974	1975	1976	1977	1978	1979	1980	1981
	----------Mg/ha* (15.5% moisture)----------									
29							3.9	4.2	4.3	
50							5.1	3.8		
64						2.4			0.5	
65						1.0			1.3	3.6
145								5.8	4.7	
$\bar{X}$						1.7	4.5	4.6	2.7	3.6

*Mg/ha x 15.7 = bu/acre.

From 1977 to 1981 the yearly means, as shown in Table 14, ranged from 1.7 to 4.6 Mg/ha (27 to 73 bu/acre). The average yield for non-mined land throughout all of Fulton County from 1972 to 1981 was 6.4 Mg/ha (101 bu/acre) and ranged from 4.7 to 7.7 Mg/ha (74-122 bu/acre). The years 1974, 1977 and 1980 were considered least favorable for corn production in Fulton County (6). The average yields for these three years were only 0.7, 1.1 and 1.3 Mg/ha on sludged mine-spoil. Favorable weather conditions seem to be of greater importance on mine-spoil than normal agricultural soils. No trend of yield increase, as soil fertility increases from sludge addition, is discernible; but good yields are possible as shown in 1981.

The corn crops grown from 1972 to 1981 were sampled from trace metal analysis. Table 15 presents the mean concentrations found each year in corn grain grown on the same 21 sludge-amended mine-spoil fields previously discussed. It is obvious from the table that corn is an excellent crop for sludge-fertilized land, as no evidence of metal accumulation in the grain is shown.

TABLE 15. MEAN CONCENTRATIONS OF TRACE METALS FOUND IN CORN GRAIN GROWN ON SLUDGE-AMENDED MINE-SPOIL AT THE FULTON COUNTY, ILLINOIS LAND RECLAMATION SITE FROM 1972-1981

Year	Zn	Cd	Cu	Cr	Ni	Pb	Mn	Fe
	----------mg/kg (oven-dried, 65°C)----------							
1972	91	--	2.8	1.7	1.4	--	10.0	98
1973	27	0.2	3.4	0.2	0.6	0.3	7.2	21
1974	28	0.1	2.5	1.0	1.5	0.2	8.6	45
1975	34	0.3	2.4	0.5	1.4	0.5	8.9	32
1976	30	0.3	2.1	0.1	0.8	0.1	4.8	23
1977	36	0.5	1.9	0.3	1.1	0.1	6.7	38
1978	38	0.5	2.9	0.2	1.7	0.1	6.6	31
1979	42	0.6	4.0	0.5	5.7	0.2	7.0	28
1980	33	0.2	2.3	3.7	0.7	<0.1	7.2	32
1981	26	0.1	2.1	0.9	0.8	<0.1	4.1	28

Table 16 gives the concentrations found in the corn grain from 1977 to 1981 for inorganically fertilized land. The Zn, Cd, Cu, Ni, Mn and Fe overall means are slightly lower than the sludge-amended corn concentrations for the same period; but the differences are insignificant.

TABLE 16. MEAN CONCENTRATIONS OF TRACE METALS FOUND IN CORN GRAIN GROWN ON INORGANICALLY-FERTILIZED MINE-SPOIL AT THE FULTON COUNTY, ILLINOIS LAND RECLAMATION SITE FROM 1972-1981*

Year	Zn	Cd	Cu	Cr	Ni	Pb	Mn	Fe
	mg/kg (oven-dried, 65°C)							
1977	25	0.1	1.1	0.1	0.7	<0.1	4.4	24
1978	21	0.3	2.3	0.2	0.6	0.1	5.6	18
1979	21	0.1	2.9	1.0	1.6	0.2	4.5	27
1980	28	0.1	2.0	3.8	0.5	0.1	7.8	26
1981	31	0.1	1.7	0.6	0.7	<0.1	5.4	20

*No corn was grown on inorganically-fertilized mine-spoil from 1972-1976.

Federal Regulations Concerning Sludge Application To Land

On September 13, 1979, the United States Environmental Protection Agency (7) published "Criteria for Classification of Solid Waste Disposal Facilities and Practices; Final, Interim Final, and Proposed Regulations" in the Federal Register. These regulations allow two alternatives for sludge application projects:

1. For land used to produce food chain crops other than animal feed:

   a. Soil pH must be maintained at 6.5 or higher;

   b. By 1987, the annual application of Cd cannot exceed 0.5 kg/ha (0.44 lb/acre); and

   c. The cumulative application of Cd is limited by the soil cation exchange capacity.

2. For land used to produce animal feed:

   a. Soil pH must be maintained at 6.5 or greater;

   b. A facility operating plan must show that the crops will be used strictly for animal feed and if alternative land uses arise, what measures will be taken to safeguard against possible Cd health hazards; and

   c. The land record or deed must stipulate that the property has received high Cd application from solid waste, and food chain crops should not be grown.

The average Cd concentration of the District's sludge applied to the Fulton County site was 0.29 kg/dry Mg (0.58 lb/ton). Therefore, under the

first alternative, only 1.7 dry Mg/ha (0.8 ton/acre) could be applied to keep the annual Cd application below 0.5 kg/ha. As this low application rate is unfeasible, the second alternative is followed by the District. Since 1979 all crops grown on sludge-amended fields have been grown for animal feed only and have been sold to Norris Farms in Fulton County, a livestock feeding operation.

Cost of the Fulton County Site

The 1981 costs for the Fulton County project are contained in Table 17. The total cost per dry megagram was $346.45 ($314.23/dry ton). Transportation accounts for 52 percent of the costs and increased 23 percent over 1980. Revenues from crop sales were $198,227; while 63,387 dry Mg (69,886 tons) of solids were utilized, for a return of $3.13/Mg ($2.84/ton).

TABLE 17. COSTS FOR THE FULTON COUNTY LAND APPLICATION SITE FOR 1981

Item	Cost	
	$/dry Mg	$/dry ton
Capital	28.36	25.72
Land Costs and Taxes	11.02	10.00
Application	94.86	86.04
Barge Transportation	180.96	164.13
Anaerobic Digestion	18.13	16.44
Centrifugation	16.25	14.74
Monitoring	7.11	6.45
Revenue from Crop Sales	-3.13	-2.84
TOTAL	353.56	320.68

LAND RECLAMATION FOR RECREATIONAL USE AND NON-FOOD CHAIN AGRICULTURE

SLUDGE PROCESSING

On-Site Sludge Processing

Centrifuge Operation--A portion of the sludge produced at the District is processed through continuous solid bowl centrifuges which are at the District's John E. Egan, West-Southwest and Calumet sewage treatment facilities. These facilities also contain a liquid and dry polymer handling system and

screw and belt conveyors for unloading the centrifuged sludge for truck and rail transport. In these centrifuge facilities, liquid anaerobically digested sludge with a solids content of 3 to 6 percent solids is concentrated to a dewatered cake with a solids content of 14 to 21 percent.

Sand Bed Dewatering--At the West-Southwest STW, Imhoff digested sludge is processed on sand drying beds. The Imhoff sludge is applied to the sand drying beds in liquid form at a solids concentration of about 7 percent solids after being dosed with a liquid polymer at a rate of 8.5 kg/Mg (17 pounds per dry ton) of sludge solids. The dewatered sludge is typically removed from the sand drying beds in three to nine weeks time at a solids concentration of about 40 percent. In 1981, the sand bed dewatering system processed an average of 48.9 dry Mg (54 dry tons) per day.

Heat Drying--In 1939 the heat-drying plant at the West-Southwest STW was put in operation. Over the years capacity was increased; while air pollution emissions were greatly reduced through conversion from coal to natural gas and installation of gas scrubbers and afterburners. The system is as follows (Figure 4):

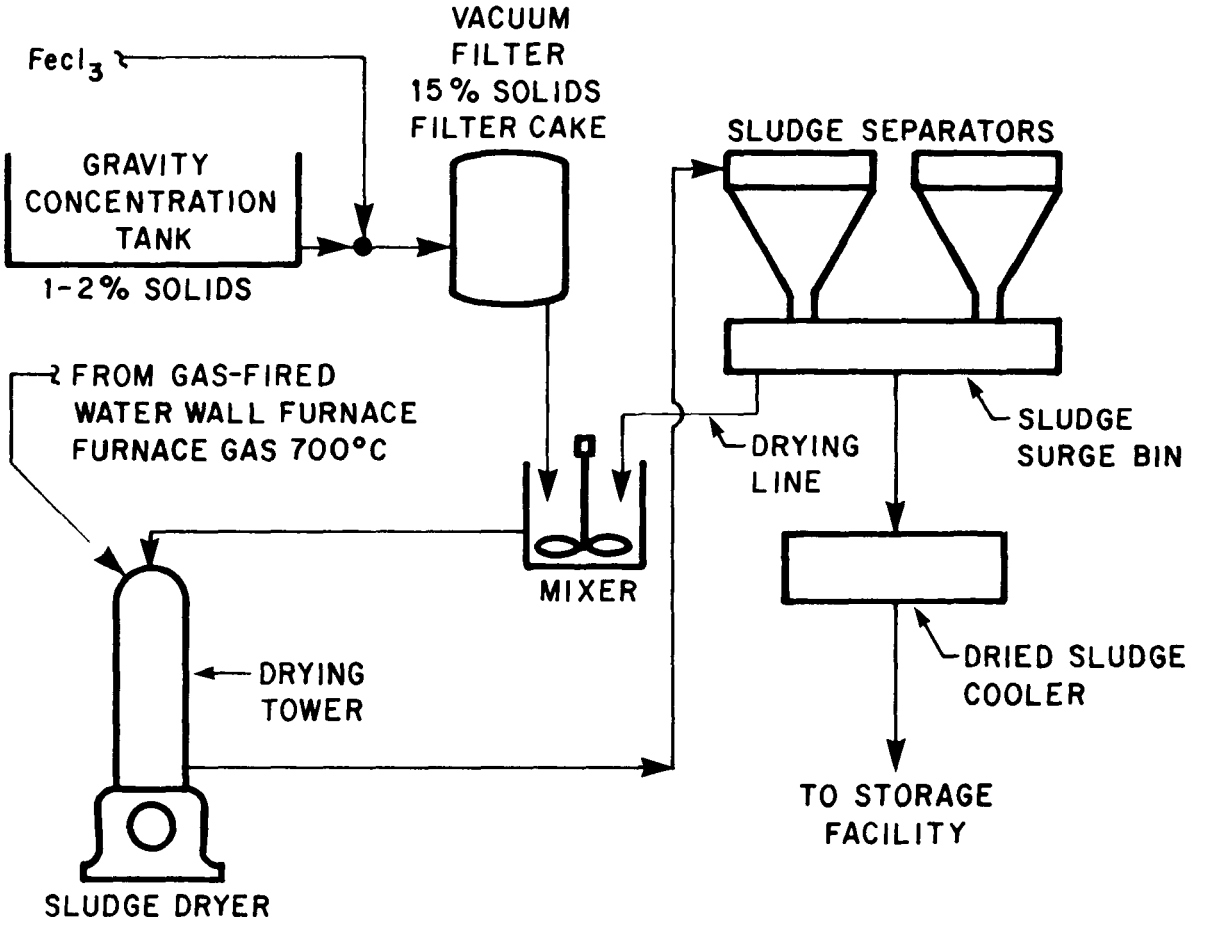


Figure 4. Heat-dried fertilizer processing plant.

Waste-activated solids concentrated to approximately 1 to 2 percent solids content in gravity concentration tanks are conditioned with ferric chloride and sent to a bank of vacuum filters. At the vacuum filters the conditioned sludge is dewatered partially to produce a filter cake of approximately 15 percent solids. The filter cake then drops onto a conveyor belt and is transported to a mixer where it is combined with dried solids from the drying line to obtain a mixture of approximately 55 percent solids, which is fed to the drying tower. Here hot gases of approximately 700°C (1,300°F) are mixed with the solids, resulting in an instantaneous evaporation of water. The dried solids are cooled, screened, and transmitted by conveyor belt to the fertilizer storage facilities. The material is an excellent soil conditioner and fertilizer base and is presently sold to a broker on a competitive bid basis. The broker in turn has developed a market of buyers who apply the solids to land.

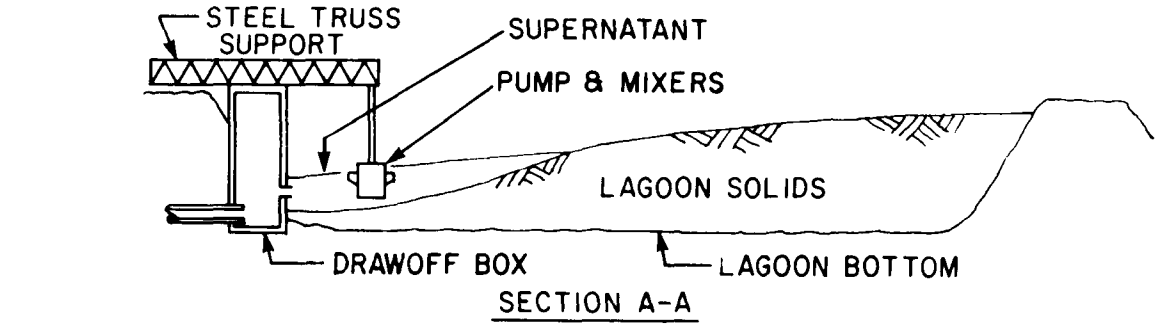
Discontinuance of Heat-Drying Program--The use of the heat-drying process was reduced considerably in 1981. During this year, input to the system averaged 81.6 dry Mg (90 dry tons) per day; while in 1980, it averaged 158 Mg (175 dry tons) per day. On August 5, 1981, the heat-drying plant was put on standby service. Stockpiles of heat-dried sludge are being sold to a broker until exhausted. There are no current plans to start up the process.

The heat-drying process is an energy-intensive operation which uses nearly 12 kilowatt hours of equivalent electrical power to process 1 kg (2.2 lbs.) of sludge. Land application, including stabilization, transportation and application energy, at the Fulton County utilization site uses only about 1.30 kilowatt-hours per kg (0.59 kw-hr/lb.) of dry solids applied. With the present energy costs in the U.S. and the need to conserve energy resources, it is extremely unlikely that the process will be used again by the District.

Off-Site Sludge Processing

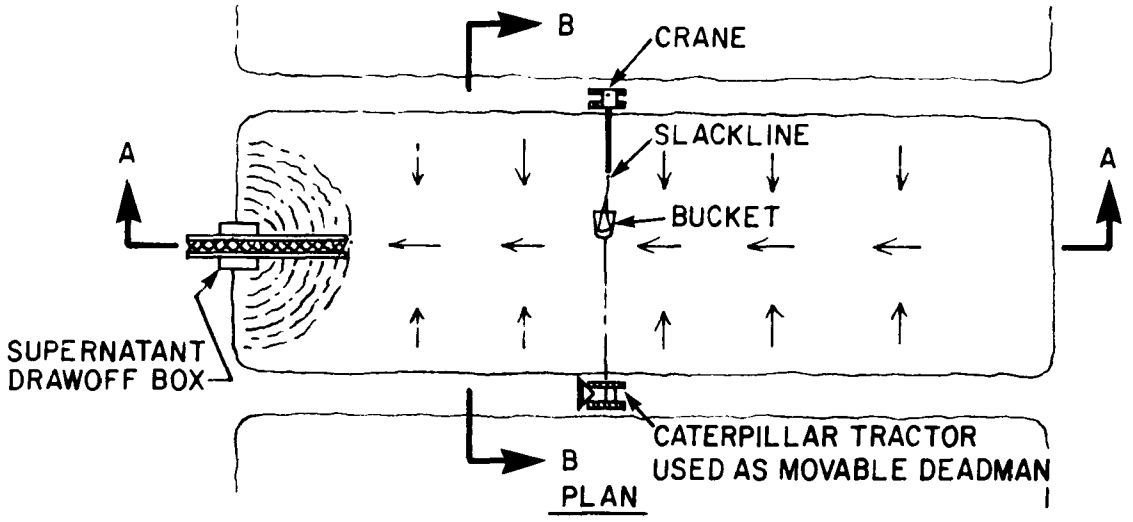
Lagoon Dewatering--Some of the solids from the heated anaerobic digesters at the District's West-Southwest STW is stored in large holding basins (lagoons). Lagoon dewatering consists (Figure 5) of first establishing a drainage profile within a lagoon to facilitate gravity dewatering of trapped water and rainfall runoff through the drawoff box. The advantage of gravity flow is significant in that pumping is not necessary and winter weather does not curtail drainage operations. The lagoon then is allowed to remain at rest with an established drainage profile through the following winter. A slackline cableway system is then used to further improve lagoon drainage. The system as used is intended to scrape material from the center upwards toward the sides of the lagoon. In this way it is intended that the following be accomplished:

- In addition to the initial drainage gradient established along the length of the lagoon, a drainage gradient is now established across the width of the lagoon.
- The slackline first scrapes one side of the lagoon and then the other.



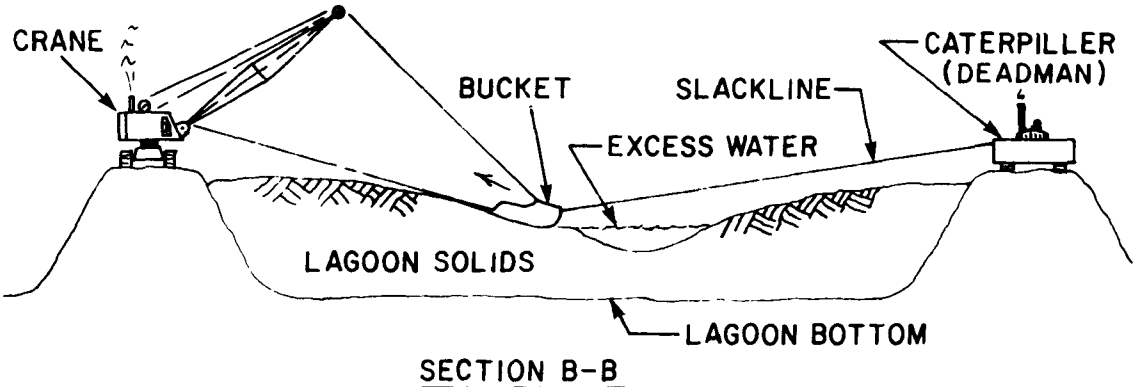
SECTION A-A

(a) PRECONDITIONING



PLAN

(b) STRESSING THE SOLIDS



SECTION B-B

(c) STRESSING AND SHIFTING OF DRIER MATERIAL TO THE SIDES OF THE LAGOON

Figure 5. Lagoon Solids Removal.

Approximately 15 cm (6 in) of lagoon crust [material near the top of the lagoon, the top 7 or 8 cm (3 in) of which has undergone appreciable evaporation] is scraped each time, and is piled up along the sides of the lagoon for further dewatering and storage before excavation and transportation. Stored material along the lagoon dikes is loaded by dragline onto dump trucks. The lagoon dewatered sludge typically has a solids concentration of about 33 percent.

Drying Cell Operation--A portion of the lagooned and centrifuged dewatered sludge is further dried at several sites by a process called agitation drying. The agitation drying process consists of first applying the sludge to clay-lined drying cells surrounded by soil berms. A bulldozer then spreads the sludge to a depth of about 40.6 cm (16 in). As the wet sludge surface dries in the atmosphere, a crust forms on the surface, inhibiting further drying. In order to achieve continued drying, a bulldozer is driven through the sludge daily, breaking the crust and exposing fresh wet material to the elements. The process continues until a dry product is formed. The dry product is then pushed into a windrow and removed in trucks. Depending upon weather conditions and initial sludge solids content, the drying process can take 40 to 70 days and can consistently achieve final solids concentrations over 60 percent.

LAND RECLAMATION

City of Chicago, 103rd and Doty Site

In 1979, the District and the City of Chicago discussed the possibility of a joint cooperative program to reclaim a City of Chicago owned landfill. The landfill was 91 hectares (225 acres) in size and was used for the disposal of municipal solid waste, construction debris, curb and gutter clearings and tree removal wastes. As the site was licensed and operated before the enactment of today's stricter environmental regulations, it did not have a final cover, lacked runoff control, and had no vegetative cover.

In 1979, the City of Chicago and the District agreed to conduct a demonstration to show how District sludge might be used to convert the land site at 103rd and Doty Avenue into an environmentally acceptable landfill which would be aesthetically pleasing to the surrounding neighborhood. The demonstration site was 14.9 ha (37 acres) in size. The site was covered with a 0.6 m (2 ft) clay seal, and then 1.52 m (5 ft) of sludge from District drying cells (60 percent solids or greater) was applied. The whole area was then seeded with a rye grass mixture to establish erosion control. The project was a complete success and a vigorous stand of grass was produced.

The success of the demonstration project led to an intergovernmental cooperative effort between the District and the City of Chicago. In 1982, the City Council of Chicago passed an ordinance that established the District as the city agent in the final closure of the 103rd and Doty site.

The closure plan will be performed in 3 phases. Phase I will involve

the development of a 23.4-ha (58-acre) winter recreation hill and parking area. The site will first be capped with a 0.6-m (2-ft) clay seal, and then a hill will be constructed from clean clay soil fill and District drying cell solids. The entire site will be bermed and runoff directed to drainage control ponds. The entire site will be landscaped with grass and trees.

Phase II will take place concurrently with Phase I. A 8.8 ha (22 acre) site which represents the most highly visible portion of the entire site will be capped with a 0.6-m (2-ft) clay seal and then 1.52 m (5 ft) of District drying cell solids. Again, the entire site will be landscaped with grass and trees and runoff directed to ponds.

It is anticipated that Phase I and II will be completed in late 1984.

Phase III will involve the reclamation of the remaining 58.6 ha (145 acres) of the 103rd and Doty site. The site will receive 1.5 m (5 ft) of drying cell solids followed by 0.6-m (2-ft) clay cap and 0.6 m (2 ft) of sludge as a base for vegetative cover. The District will contour the site as dictated by the City of Chicago who have indicated that the site would be used for a golf course, public park or tree nursery. It is anticipated that this phase will take eight years to complete.

In 1981, the cost of digestion, lagooning, drying, transportation and application of sewage solids to the 103rd and Doty site was $65/dry Mg ($59/dry ton).

Palzo Land Reclamation Site

In 1970, the U.S. Forest Service and the District became involved in a joint effort to reclaim portions of the Palzo Mines in southern Illinois. These acidic mine spoils, now part of the Shawnee National Forest, had virtually no vegetation since surface mining ceased in 1961. The acidic runoff had seriously affected a nearby receiving stream (Sugar Creek) and the solution to this problem became a primary goal of the U.S. Forest Service.

In 1977, the U.S. Forest Service reclaimed 52.6 (130 acres) of acidic soil in the Shawnee National Forest using liquid lagooned digested sewage sludge from the District's Calumet STW. The District supplied this sludge at no charge to the U.S. Forest Service who applied the sludge at an average rate of 657 Mg/ha (300 dry tons/acre). The total cost for this operation including land grading, sludge transportation, and environmental monitoring was $96/Mg dry sludge ($87/ton). District cost for sludge transportation was $78/Mg ($71/ton).

Libby-Owens-Ford Land Reclamation Site

In 1969, sludge supplied free of charge by the District was used to treat an alkali sand-filled lagoon with a pH of 10.5 near Ottawa, Illinois, which was owned by the Libby-Owens-Ford Company. This lagoon was totally devoid of vegetation, and during dry weather produced dust which was a nuisance to surrounding residents. Liquid digested sewage sludge was in-

corporated into the sand-filled lagoon by filling 0.6-m (2-ft) deep trenches with liquid sludge and covering this with sand. Using this application method, 380 Mg/ha (170 dry tons/acre) of sludge were mixed into the sand surface. Grass was planted on the soil and has grown vigorously ever since.

Liquid lagooned sludge was shipped from the District's West-Southwest STW to Ottawa, Illinois, and applied to the site at a total cost to the District of $85/dry Mg ($77.20/dry ton).

Ottawa Silica Sand Land Reclamation Site

The Ottawa Silica Sand Company owns a 60.7-ha (150 acres) strip-mined site approximately 125 km (77 miles) southwest of the city of Chicago. This site was strip mined by a coal company, and then sold to the Ottawa Silica Sand Company. The company had hoped to mine the site for sand, but abandoned such attempts because of the high cost of mining the rough strip-mine topography. The Ottawa Silica Sand Company was aware that the site created aesthetic and environmental problems. The site was barren and generated an acid runoff to the nearby Illinois River. The company wanted to vegetate the site to reduce soil erosion and acid runoff and make the site more aesthetically acceptable.

The District became aware of the Ottawa Silica Company site in the late 1970s and approached the company with a proposal to use some of the District's sludge as a soil amendment for vegetating the Ottawa site. It was agreed that a reclamation effort using sewage sludge would be made on a 0.81 ha (2 acre) portion of the 60.7-ha (150-acre) site owned by the Ottawa Silica Sand Company, and the District agreed to pay for the total costs for reclamation.

A total of 6,398 wet Mg (7,037 tons) of lagooned digested sludge was shipped to the Ottawa site with an average concentration of 29 percent. The actual application rate was about 2,000 Mg/ha (892 tons/acre). Mixing of the lagooned sludge with the mine spoil was accomplished by placing the sludge along the crests of the mine spoil ridges and after partial drainage, the sludge was mixed with and pushed into the valley between the ridges. The entire site was planted with grass. The grass was successfully established and growing two years after the sludge application.

The total unit cost to the District including sludge transportation and application was $81/dry Mg ($73/dry ton).

Nu Earth Program

Nu Earth is a wastewater solid that has been digested anaerobically in the digestion compartment of Imhoff tanks, followed by air drying on sand beds and haulage by rail to a stockpile area for further dewatering over a period of years. Within the past few years, the District has developed its very successful local Nu Earth distribution program. Individual users can order a minimum quantity of one 18.4 m^3 (24 cu yd) truckload that is delivered free of charge within an 80 km (50 mile) radius of

the distribution point. Nu Earth is a compost-like material resembling a rich soil (about 55 percent solids content) and has proved acceptable to the public for use in landscaping and flower gardening. It contains approximately 4 percent N, 6 percent P_2O_5, and 0.1 percent K_2O.

In 1981, nearly 35,373 dry Mg (39,000 dry tons) of Nu Earth was distributed for non-food chain uses in the greater Chicago area. The cost to the District for the Nu Earth program, including Imhoff digestion, sand bed dewatering, solids storage and transportation, is about $70/dry Mg ($63/ton).

LANDFILLING OF SLUDGE WHEN REQUIRED

Solids from the District's storage lagoons also are removed on a competitive bid contractual basis. The contractor is required to excavate and dispose of the solids in a legal manner, and most of it in recent years has been going to sanitary landfills. This method of solids disposal has been used when the quantity of solids desired to be disposed of exceeds the land application/utilization capacity of the existing market place, such as when it is desired to dispose of large quantities in a short time period to eliminate a lagoon to make space for plant expansion. Also, contract disposal will be chosen when physical constraints in the lagoon complex do not make slackline removal desirable. In 1981, 50,855 dry Mg (56,070 dry tons) of solids were removed from lagoons via lagoon cleaning contracts. At the present time there are no plans to utilize this mode of solids management in the future, but this option will be considered if conditions so dictate. This mode of solids management is one of the least expensive solids management options used and averages about $80 to $100 per dry Mg ($88.2 to $110.3 per dry ton).

USE OF THE PRIVATE SECTOR IN MUNICIPAL SLUDGE MANAGEMENT

The District believes that in certain circumstances private contractors can economically manage municipal sludge in a cost-effective and environmentally safe manner. The District is therefore currently preparing a contract by which private concerns can competitively bid for the right to manage a portion of the District's sludge production.

The bidding organization must be prepared to manage 108.8 Mg (120 dry tons) of District sludge using an option which is acceptable to local, state, and federal agencies. The sludge will be provided in a slurry of 1 to 3 percent solids and will not be stabilized. The vendor must show that he can manage the sludge for a 20-year period. Cost increases incurred by the vendor during the life of the contract will be reimbursed by the District according to a predetermined formula. The lowest qualified bidder will be awarded the contract, but the District has set a predetermined maximum capital and operation and maintenance cost and maximum energy usage which cannot be exceeded. Current plans call for bids to be opened in May of 1983 for this sludge management option.

SUMMARY

Wastewater solids management is the most difficult problem today facing most of the operators of publicly owned wastewater treatment facilities. With goals of the future being the elimination of combined sewer overflows and increased treatment efficiencies, solids quantities for ultimate disposal or utilization will increase, magnifying the already difficult solids management problems.

The philosophy of today's wastewater solids managers at the District is to maximize the degree of flexibility of each solids distribution program. Research, experimentation, and venturing into unexplored avenues holds the key to answering the solids management questions of the future.

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SLUDGE INCINERATOR FUEL REDUCTION PROGRAM

by
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ABSTRACT

The Hartford Metropolitan District Commission has successfully accomplished two major operational changes at its water pollution control plant to significantly reduce sludge incinerator fuel consumption. The Hartford Water Pollution Control Plant was one of the first municipal plant operations in the U.S. to use the continuous belt filter press. During the period from 1979 to 1981, the Hartford Plant converted from vacuum filters to all belt filter presses. The resulting improvement in solids production and drier sludge cake enabled a 65% reduction in specific fuel consumption. In 1982, the Hartford incinerators were operated with a new, more fuel efficient operating mode adopted by the plant engineering staff with the technical assistance of the Indianapolis Center for Advanced Research. Routine operations with the new incinerator operating mode during 1982 achieved an additional 51% fuel reduction. The net change in the incinerator specific fuel consumption from both of these operational changes between 1978 and 1982 was 104 gallons of oil per dry ton of sludge cake incinerated. The 1978 average fuel consumption of 125 gallons per dry ton was reduced to an average of 21 gallons per dry ton for 1982 for an overall reduction of 83%. The average operational cost savings from this reduction and related savings amounted to over $1,300,000 per year for 1982 as compared to 1978 operations. With these two major operational changes, the Hartford Water Pollution Control Plant has become one of the most energy efficient sludge handling operations by incineration in the U.S.

SLUDGE INCINERATOR FUEL REDUCTION PROGRAM

INTRODUCTION

The disposal of municipal sewage sludge by incineration most often requires the use of large amounts of auxiliary fuel. Increasing energy costs in recent years have made incinerator fuel consumption a major operational cost problem for many municipal operations. To reduce this operational cost, the Hartford Metropolitan District Commission conducted two major operational changes at its Hartford Water Pollution Control Plant. The Hartford plant was one of the first in the U.S. to convert to continuous belt filter presses for sludge cake dewatering. This conversion took place between 1979 and 1982 with the installation of four belt filter presses. The conversion to belt filter presses resulted in dramatic fuel savings of 65% and increased solids production. In late 1981, the Hartford plant engineering staff and incinerator operators adopted a new, more fuel efficient operating mode with the technical assistance of the Indianpolis Center for Advanced Research. The new incinerator operating mode has been exclusively used in routine operations since January, 1982, and has resulted in an additional 51% reduction in the specific fuel consumption. The combined result of these two major operational changes was an 83% reduction in the incinerator fuel consumption. These results are reviewed here as an example of how improved dewatering equipment and incinerator operating technology offer new options to those municipal plants which are currently bound, for various reasons, to sludge disposal by incineration.

PAST OPERATIONS

The Hartford Water Pollution Control Plant performs primary and secondary wastewater treatment for more than 45 million gallons of wastewater per day and generates in excess of 200 wet tons of filtered sludge cake per day. The sludge handling facility was designed in 1968 with four dissolved air flotation thickeners, five drum-type vacuum filters and three multiple (11) hearth incinerators. The incinerators are equipped for either gas or oil

operation and are rated at 12.5 wet tons per hour maximum capacity. In 1978, before conversion to belt filter presses, the vacuum filters average 13.8% cake solids. Production required continuous operation of three of the five vacuum filters with two of three incinerators operating around the clock, on a five-day work week. The plant operation experienced the typical production and maintenance problems associated with handling an extremely wet sludge cake, that is, clinkers, broken teeth, plugged drop holes, excessive refractory problems, broken shear pins, smoking, etc..... To compound problems, the Hartford plant began receiving sludge from satellite plants in East Hartford, Rocky Hill and from Glastonbury.

In view of these operating problems and the ever increasing cost of fuel, the Hartford Metropolitan District Commission initiated a major program effort to find new methods in plant operations and processes to regain "control" of operating costs. The belt filter press conversion program and the new incinerator operating mode development were two of the major projects among others which have been undertaken.

BELT FILTER PRESS CONVERSION

Testing and Operational Experience

Hartford began pilot testing belt filter presses in the spring of 1978. Test results showed that significantly drier sludge cake was produced at higher production rates as compared to the performance of the existing vacuum filters. The plant staff then conducted side-by-side performance tests of the most qualified machines to select the first press for procurement and installation. The first press was installed in 1979 with a carefully monitored start-up and shake-down period. Despite numerous mechanical problems and excessive downtime, the belt filter press quickly proved itself so economically beneficial, the approval for acquiring the second press was granted only four months after the first one was installed. The payback period for the first press was only six weeks!

In selecting the second press, performance tests were again conducted to evaluate overall performance, with emphasis placed on mechanical design, and maintenance features to incorporate these requirements in the bid specifications. The second press, which was supplied by a different manufacturer than the first, was installed in December, 1979, just eight months after the first one.

During the first half of 1980, the Hartford plant operated with two belt presses and one vacuum filter. The mechanical reliability sought with the second press was realized, as well as, continued fuel savings and sludge cake production improvements. With this continued operational success, the District purchased a third press from this same supplier and installed it in December, 1980. During 1981, the plant was able to operate almost exclusively with belt presses. A fourth press was placed into operation in June, 1982 to increase capacity and operational flexibility in pairing two presses to each operating incinerator.

Being one of the first plants to try the belt filter presses, the early operational experiences were expectedly problematic. The initial press operation was plagued with numerous mechanical problems with the bearings, rollers, spray water pump, filter screen cleaning, filter screen tracking, and filter screen seam closures. With assistance from the manufacturer, the first press was retrofitted and upgraded for more reliable operation. The second through fourth presses were from a different supplier, have had few mechanical problems, and have been satisfactory in operation.

As more operational experience was gained, improvements were made in several key operating conditions. The filter screen seam closure wearing problem was reduced by using higher molecular weight plastic scraper blades resulting in an increase in filter screen operating life from an average of 500 hours to an average of 1500 hours. A new filter link screen is being tested with a potential increase in cake solids and an operating life in excess of 2500 hours. Proper polymer conditioning of the sludge cake on all the presses has been a problem. A two component liquid polymer mix was developed from experiments with the polymer supplier to reduce the dosage requirements to the same level as was required for the vacuum filters. Changes in the sludge conditioning tank to improve polymer sludge mixing have also helped reduce dosage requirements and increased flexibility to adjust to varying sludge characteristics. Maintaining a constant blend in the mixing of the raw primary and waste activated sludges from four plants requires close operator control. Sludge blend variations of only 5% to 10% can cause the press screens to plug and the sludge to squeeze out the ends of the rollers with a resulting loss in percent solids and production. In spite of these operating problems associated with reducing a new operating technology to routine production line practice, the operational improvements and cost savings achieved with the belt filter presses in Hartford have been dramatic.

Fuel Reduction Results

To provide an accurate baseline for comparing the fuel reduction achieved by converting to belt filter presses and by improvements in the incinerator operating mode, a statistical analysis was made of the key operating performance data for past operations and each of the years during which changes were made. In addition, the correlation of specific fuel consumption (SFC) measured in gallons of oil per dry ton with the absolute sludge cake moisture to volatile ratio by weight (M/V) was computed to provide a more comprehensive measure of change for comparison.

The average specific fuel consumption for the incinerator operations in 1978 was 125 gallons of oil per dry ton. The sludge cake solids averaged 13.8%, and the volatiles averaged 77.1%. The sludge cake M/V ratio which is directly related to, and principally determines, the specific fuel consumption demand was an average 8.6 which is relatively high.

The savings resulting from the belt filter presses is reflected in the sharp reduction in the sludge cake M/V ratio, particularly in 1980 when the major fuel reduction was achieved. The net reduction of an average of almost 82 gallons of oil per dry ton would translate into a savings of over

EFFECT OF BELT FILTER PRESSES ON KEY OPERATING PERFORMANCE VARIABLES FOR 1978 TO 1981

VARIABLE	YEAR 1978	1979	1980	1981	1978 to 1981 PERCENT CHANGE
PERCENT SOLIDS	13.8	14.5	18.5	19.5	+41
SLUDGE CAKE M/V	8.6	8.1	5.8	5.4	-37
FUEL CONSUMPTION GALS./DRY TON	125.2	116.1	60.5	43.5	-65

TABLE I.

848,000 gallons of oil at the 1982 dry ton production level of 10,351 dry tons. Furthermore, the average incinerator-hours of operation per day also dropped from 46.5 in 1978 to 35.7 in 1981, a 23% decrease.

These substantial results were accomplished after a considerable amount of time and effort was invested by the Hartford plant management, staff and operating personnel. The experience of Hartford with the belt filter presses serves as a classic example of the opportunities that exist in many plants throughout the country to achieve internal improvements with the adoption and modification of new operating technologies. Most operations would have been contented at this point with the lower fuel consumption levels that had been achieved. However, in the case of the Hartford Metropolitan District Commission and Staff, the dramatic success with the belt filter presses encouraged us to pursue the adoption of other innovative operating technologies to improve our operations further. One of the technologies adopted was an improved incinerator operating technique which had been developed by the City of Indianapolis Department of Public Works with the Indianapolis Center for Advanced Research under sponsorship of the U.S. EPA Municipal Environmental Research Laboratory.

NEW INCINERATOR OPERATING MODE

The development and testing of the new operating mode was the result of a cooperative effort by the Hartford engineering and operating personnel with combustion engineers from the Indianapolis Center for Advanced Research (ICFAR). The new operating mode was derived from refined operating technology developed by ICFAR through a major operational research project conducted on the Indianapolis incinerators. On-the-job instruction and training in the use of the new operating mode was also performed to demonstrate the potential fuel reduction and upgrade operator performance.

Operational Testing and Analysis

An operational analysis was made of the Hartford incinerator operations which included airflow measurements, exhaust gas analysis, assessments of key instrumentation and controls, existing operator specific practices, load rate management, incineration/dewatering modes, airflow management, burner use profiles, combustion zone location and hearth temperature profiles. A kinetic incinerator analytical model was also used to determine the most optimum load rate and plant operating mode which would result in the minimum possible fuel consumption.

ICFAR made preliminary analysis of practices which were contributing to excessive fuel consumption. Some of these findings were: 1) combustion occurring too high in the incinerator, 2) high exhaust gas temperatures, 3) high draft settings and too much auxiliary air, 4) misuse of heated rabble arm cooling return air, 5) improper burner use profiles, and 6) improper techniques for controlling combustion zone location. Also contributing to the high fuel consumption were other problem areas associated with the lack of remote operator controls for airflow dampers and burners.

The airflow measurement results were used to help correct the problems found in the airflow management approaches being used, as well as, to assess the relative impact of various previously proven techniques on the Hartford incinerator set-up. A preliminary analysis indicated that through optimum airflow management, the potential fuel reduction from these measures alone was 70% with a sludge cake having a moisture to volatile ratio of 5.0 and with an incinerator loading rate of 6 wet tons per hour. Furthermore, the kinetic rate analysis for these conditions predicted that the potential fuel consumption for the Hartford operation with such a dry cake was nearly zero! This analytical result was in agreement with the preliminary estimate drawn from airflow management techniques, since an additional 30% could be reasonably expected from improved combustion zone location control, optimum burner use profiles, improved load management, and the synergistic effect of these operating mode techniques on fuel consumption. Based on these results and ICFAR's previous experience from similar programs in Indianapolis, Buffalo, and Nashville, periods of autogenous combustion were expected with the new operating mode. Autogenous combustion was achieved several times during the operational trial and demonstration test for as long as eight hours.

Based on the operational trial tests and analyses, a new operating mode with specific instructions and operating settings was developed. The new operating mode was then demonstrated in full plant operation for a two week performance demonstration test period. On-the-job operator training in the use of the new mode was also accomplished at the same time. After completion of the successful performance test, the operating mode was further refined for routine operational use.

The new operating mode was characterized by the following general operating guidelines:

- °Maximum use of heated rabble arm cooling air return
- °Lowest possible draft to minimize excess air and leakage
- °Combustion on hearth 7 or 8 to maximize drying area
- °Eliminate airflow to top hearth burners
- °Control combustion zone location with burner use profile
- °Slow center shaft speed to improve sludge drying
- °Discontinue use of hearth #5 burners

The specific operating instructions which constituted the new operating mode were given to the incinerator operators and included procedures for sludge load management, incinerator operational control, general operating settings, specific settings for normal operations, combustion zone location control, standby and start-up operations, and techniques to control sludge cake "burn-outs". The most effective incineration/dewatering mode was determined from the trial tests to be two belt filter presses paired to each operating

incinerator. The most optimum incinerator loading rate was found to be 6 wet tons per hour per incinerator based on analysis and trial tests of load rates between 4.5 and 7 wet tons. The 6 ton per hour load rate was the minimum rate associated with keeping up with the overall plant loading rate and still minimize fuel consumption. The improved operating mode also enabled a further reduction in fuel usage because the new mode allowed the presses to be slowed down resulting in a small increase in solids.

Fuel Reduction Results

The new incinerator operating mode was placed into routine operational use directly following the two week performance test conducted in January 1982. At this time, the new operating mode has been in continous use by Hartford for over a year. The operational data for the Hartford plant for 1982 was analyzed to measure and compare the fuel reduction achieved.

The next slide (graph) shows the average specific fuel consumption for the Hartford operations from 1978 through 1982 for comparison of the total historical change. The average specific fuel consumption for 1982 was 21.1 gallons per dry ton as compared to 43.5 for 1981, a 51.5% reduction. With this improvement the total fuel reduction achieved by Hartford between 1978 and 1982 amounted to 104 gallons per dry ton, or 83%, which at the 1982 production level represented a savings of 1,076,000 gallons of #2 fuel oil as compared to 1978.

In addition to reducing direct fuel consumption, the new operating mode provided increased operating flexibility with the equipment, since the incinerators could now be efficiently operated at load rates 50 to 60 percent of capacity which was not possible before without paying a tremendous penalty in excess fuel consumption. The incinerator operation is also now characterized by cooler maximum operating temperatures, more steady state control, less particulate emissions and reduced maintenance on internal incinerator parts. In particular, damage to the refractory has decreased, rabble teeth have not been failing due to the lack of clinker formations, the rabble arms are lasting longer, and the burner slagging problems has been drastically reduced.

In spite of the significant fuel reduction achieved with the new operating mode, the Hartford incinerator operation still has further fuel reduction potential which has yet to be realized from the new operating mode. There are several equipment related reasons which currently are limiting further gains.

The conveyor system has posed numerous problems. The sludge cake from the belt filter presses is delivered to the incinerators via a series of belt conveyors. Due to the arrangement of the presses, one conveyor feeds both operating incinerators. Diverting a fixed amount of sludge on a continuous basis off the conveyor is a difficult task. If dry cake builds on the front of the diverter, the amount of feed to both incinerators will be changed, upsetting the thermal equilibrium.

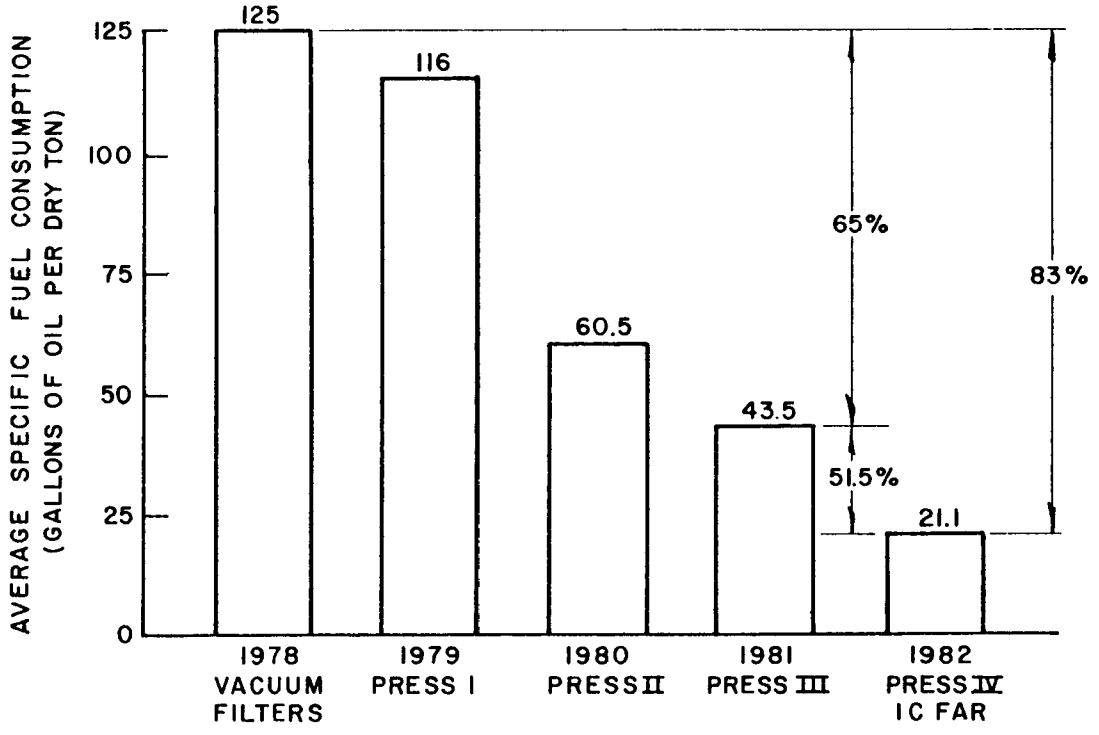
AVERAGE SPECIFIC FUEL CONSUMPTION
(GALLONS OF OIL PER DRY TON)
125
100
75
50
25
0
125
116
60.5
43.5
21.1
65%
51.5%
83%
1978
VACUUM
FILTERS
1979
PRESS I
1980
PRESS II
1981
PRESS III
1982
PRESS IV
IC FAR

TABLE IL

Another critical equipment related problem has to do with the inadequacy of the original rabble arm raking pattern which was originally set-up for a wetter sludge cake. The drier cake commands a far different profile for the number of rabble arms per hearth and the spacing between rabble teeth depending on what hearth your considering. During the operational tests an uneven sludge distribution problem was encountered several times. A distorted burning pattern resulted with sludge combustion occurring on three hearths simultaneously with uneven burning on one side of these hearths. This condition also creates an increase in incinerator "burnouts" where the dry sludge cake gets to a point of uncontrollable autogenous combustion and the incinerator steady state thermal equilibrium is lost in the process. Plans are underway to correct the rabbling pattern to adjust for the drier sludge cake. To a lesser extent, some improvement in the instrumentation and remote operating controls would also enable improved operator performance.

COST SAVINGS

The nominal cost savings from reducing incinerator fuel consumption on an annual basis was estimated from the change in the specific fuel consumption from 125 to 21 gallons of oil per dry ton. Based on 1982 production of 10,351 dry tons, the savings would be over $1,076,000 per year using an estimated price for #2 fuel oil of $1.00 per gallon.

In addition to the incinerator fuel savings, there were also other energy savings realized from the belt filter press conversion. Since the plant started up in 1972, the activated sludge mixed liquor suspended solids (MLSS) concentration had averaged 4,000 to 5,000 milligrams per liter (mg/l) requiring approximately 100 million cubic feet of dissolved air per day. With the belt filter presses, the sludge production increase has enabled the MLSS level to be lowered to a more desirable 2,000 mg/l range. The resulting decrease in the dissolved oxygen demand reduced the daily air usage to approximately 55 million cubic feet per day. This reduction, in turn, reduced the electrical energy requirements of a 3,000 hp air compressor by 20% which amounted to a $200,000 per year savings in electrical costs. Also, each vacuum filter had a 71.5 hp requirement as compared to 22 hp for each belt press. This reduction in electrical use resulted in an estimated savings of $25,000 per year. In addition, the elimination of the vacuum pumps which required potable water at a rate of 30 GPM/filter resulted in a savings of $6,000 per year. In total, these additional savings amounted to over $231,000 per year.

The total plant estimated annual energy cost savings from converting to belt filter presses and the new incinerator operating mode are shown on Figure II. The sharply rising dashed line represents the annual total gas, oil and electrical costs. The costs are actual to 1978 and have been estimated to reflect the subsequent yearly costs if vacuum filters remained in operation. The solid lower branch line represents the actual yearly costs since the retrofitting to belt filter presses and the new incinerator operating mode.

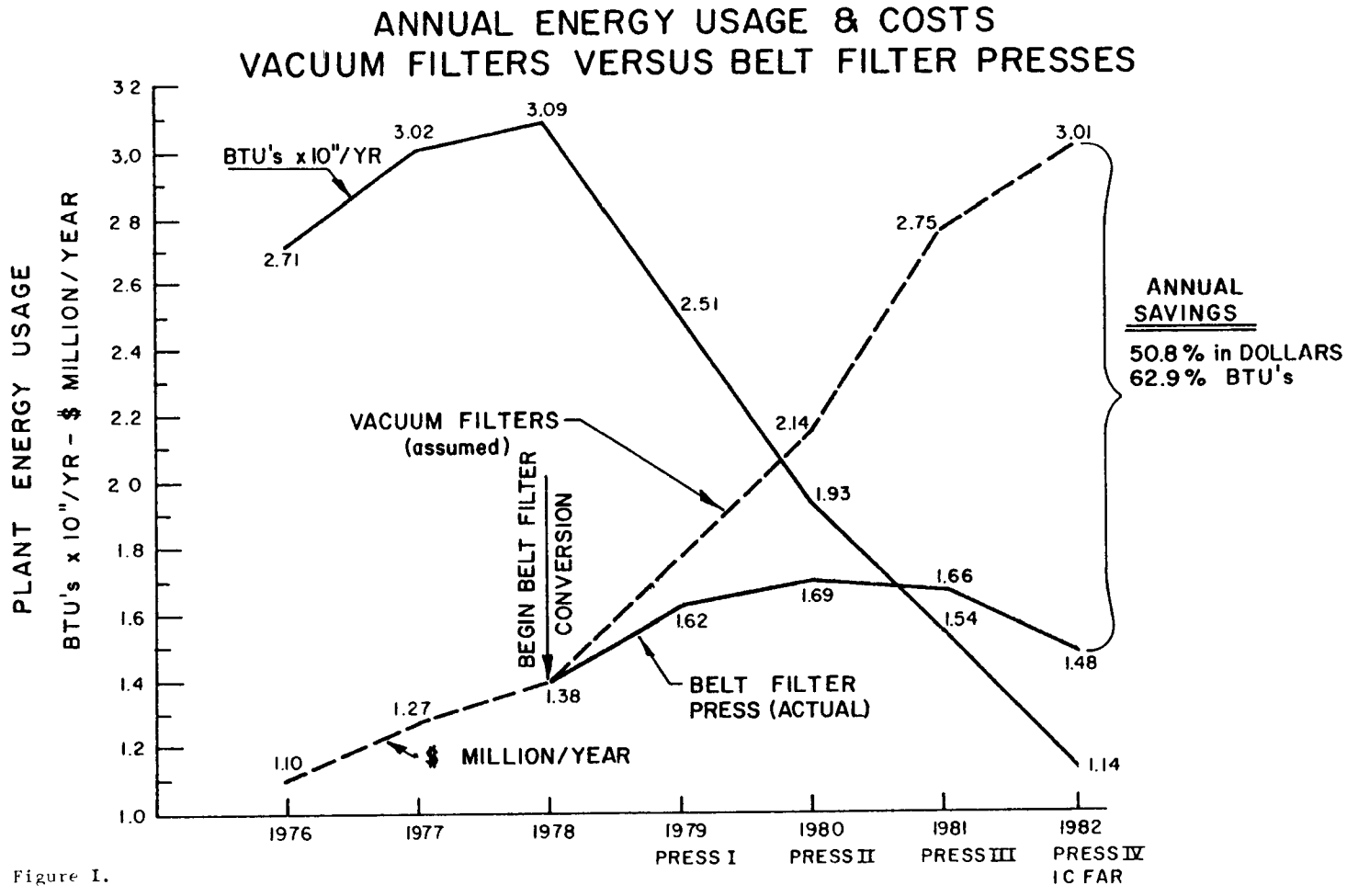
ANNUAL ENERGY USAGE & COSTS
VACUUM FILTERS VERSUS BELT FILTER PRESSES
PLANT ENERGY USAGE
BTU's x 10"/YR - $ MILLION/YEAR
BTU's x10"/YR
$ MILLION/YEAR
VACUUM FILTERS (assumed)
BEGIN BELT FILTER CONVERSION
BELT FILTER PRESS (ACTUAL)
ANNUAL SAVINGS
50.8% in DOLLARS
62.9% BTU's
3.2
3.0
2.8
2.6
2.4
2.2
2.0
1.8
1.6
1.4
1.2
1.0
2.71
3.02
3.09
2.51
1.14
1.10
1.27
1.38
1.62
1.69
1.66
1.54
1.48
2.14
2.75
3.01
1.93
1976
1977
1978
1979
PRESS I
1980
PRESS II
1981
PRESS III
1982
PRESS IV
IC FAR

Figure I.

The shaded area represents the annual cost savings. For example, our 1982 costs were cut in half by $1.5 million.

The upper increasing then sharply decreasing solid line, demonstrates the continuously rising energy demand then the constant decrease during the retrofit period. The total plant energy has been reduced by 62.9% between our peak year of 1978 and 1982.

CONCLUSIONS AND RECOMMENDATIONS

1. Fuel consumption in multiple-hearth sewage sludge incinerators can be significantly reduced by improved dewatering equipment and also by using more fuel-efficient operating modes.

2. Further applications of improved operating techniques need to be promoted as a cost effective approach to reducing incinerator fuel consumption.

3. Low cost investments for improved instrumentation and controls and operator training programs can result in major fuel savings for many existing operations.

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