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PROCEEDINGS

THE JAPANESE-GERMAN WORKSHOP ON WASTE WATER AND SLUDGE TREATMENT

Date:
October 4 and 5, 1982
Place:
Public Works Research Institute
Ministry of Construction
Tsukuba Science City, Ibaraki Prefecture, Japan

WATER QUALITY CONTROL DIVISION
PUBLIC WORKS RESEARCH INSTITUTE
MINISTRY OF CONSTRUCTION
TSUKUBA SCIENCE CITY, IBARAKI PREFECTURE, JAPAN

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Foreword

The Japanese-German Workshop on Waste Water and Sludge Treatment was planned at the preceded meeting of the Japanese-German Panel for Research and Development on New Environment Protection Technology to deepen the technical imformation exchange. Last year, both German and Japanese sides agreed that the first workshop would be held in Japan on October 1982.

I am very pleased as a member of the delegation to have been able to complete this workshop with a great success. I would like to express my hearty gratitude to the delegations of both sides, especially to the Head of German delegation, Prof. Dr. S. H. Eberle, Director, Division of Water Technology, Nuclear Research Center Karlsruhe for the greatest contribution to the successful workshop.

The presented papers to the workshop comprise a useful body of knowledge on sewage and sludge treatment in both countries. I am convinced that the Proceedings will be available not only for the delegation members but also for the engineers concerned in both countries.

T. Tamaki

Tsutomu Tamaki
Director General,
Department of Sewerage and Sewage Purification
Ministry of Construction

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Prof. Dr. S. H. Eberle
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Mr. Shunsoku Kyosai
Chief, Ultimate Disposal Section, WQCD, PWRI, MC

Dr. Ken Murakami
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Chief, Research and Technology Development Section, JSWA

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Sanitary Engineer, Research and Technology Development Section, JSWA

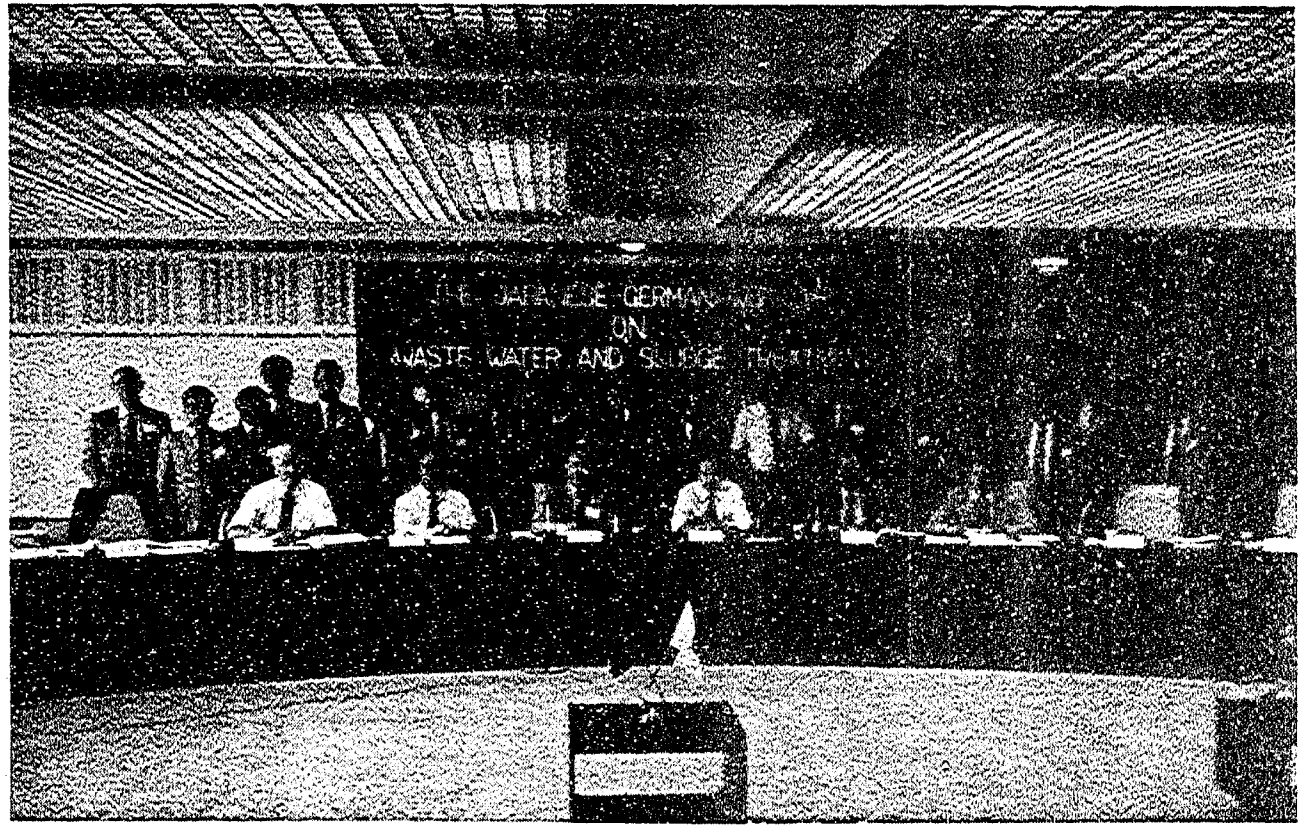
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Mr. Kazunari Matsunaga
Director, Construction Division, Sewage Works Bureau, Osaka City

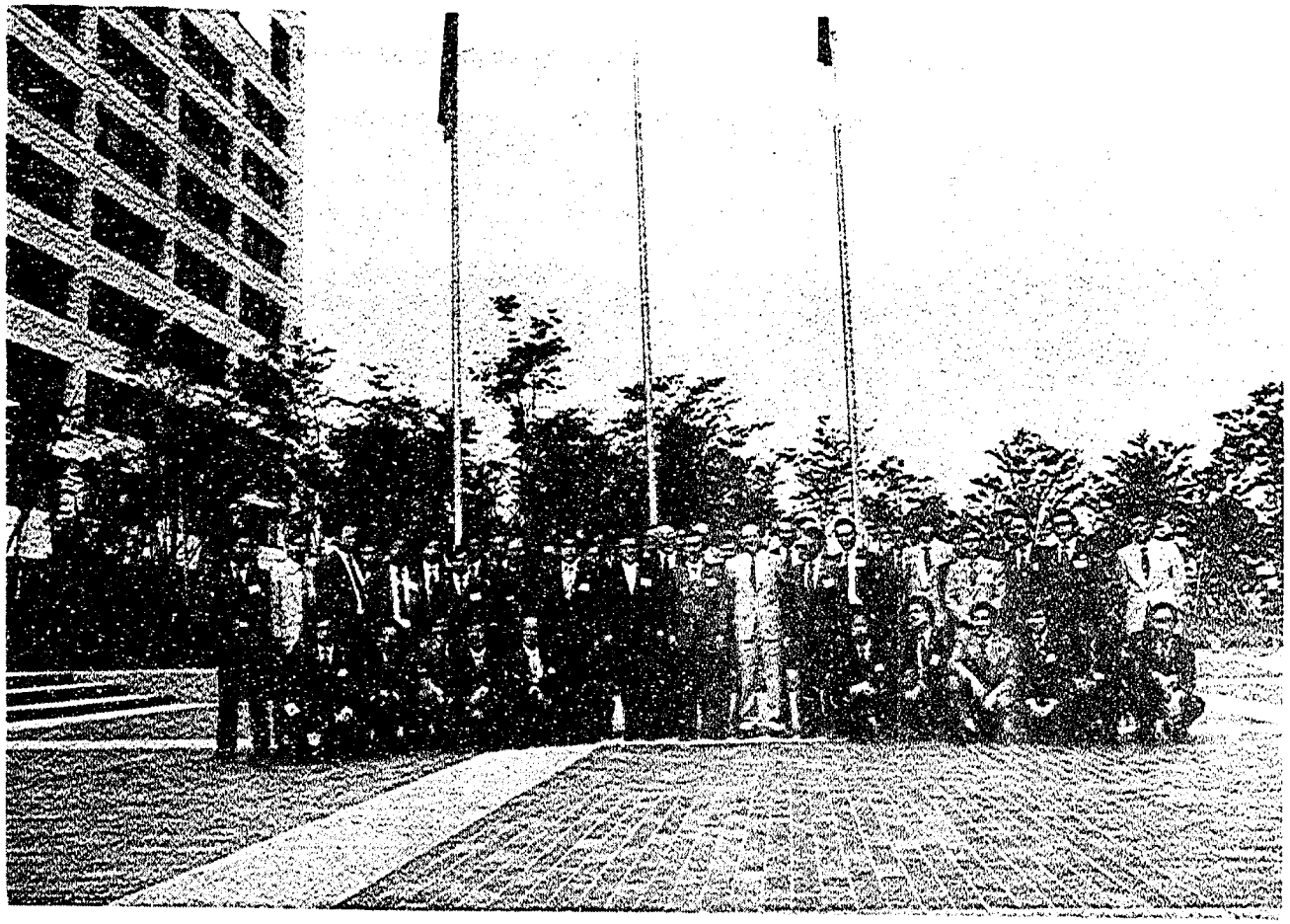
Dr. Nagaharu Okuno
Assistant Director, Section of Engineering Research and Development, Department of Sewage Works, Tokyo Metropolitan Government

Mr. Sakuji Yoshida
Chief, Plant Designing Division, Construction Department, Sewage Works Bureau, Yokohama City

Mr. Masaki Kamimura
Chief, Water Quality Laboratory, Sewage Works Bureau, Kyoto City



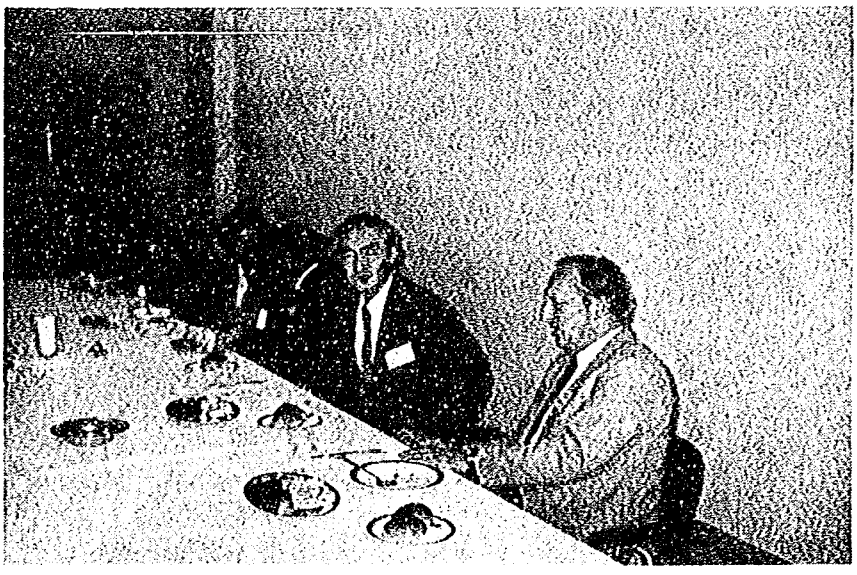
Germany and Japan Delegates to the Workshop, Public Works Research Institute, Ministry of Construction.



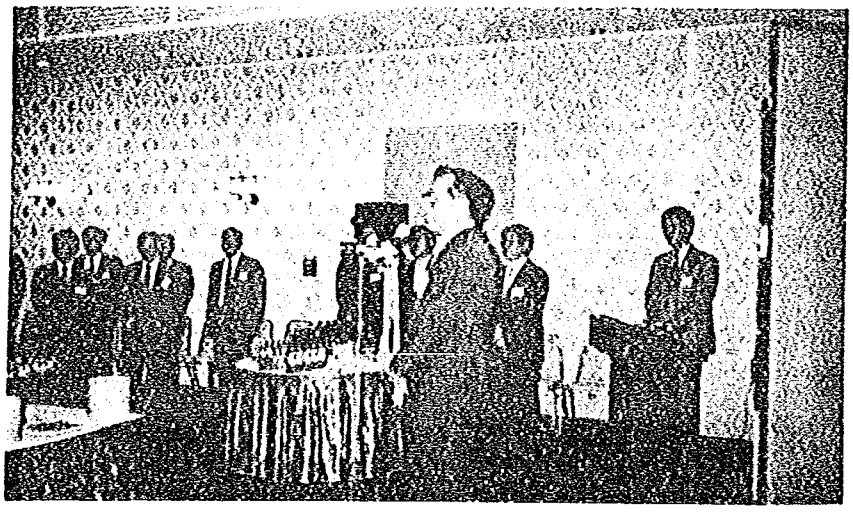
German and Japanese Participants to the Workshop, Public Works Research Institute, Ministry of Construction



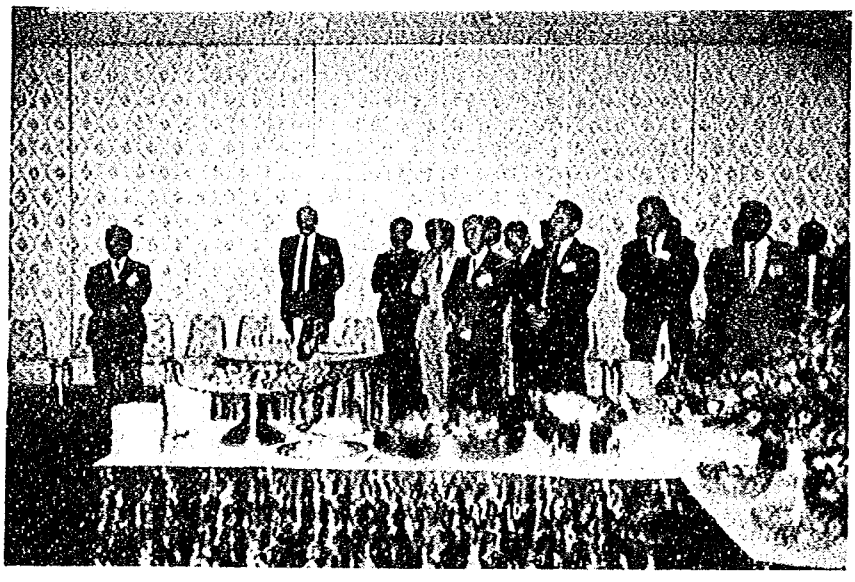
Snap Shot of the Workshop at the International Conference Room of PWRI



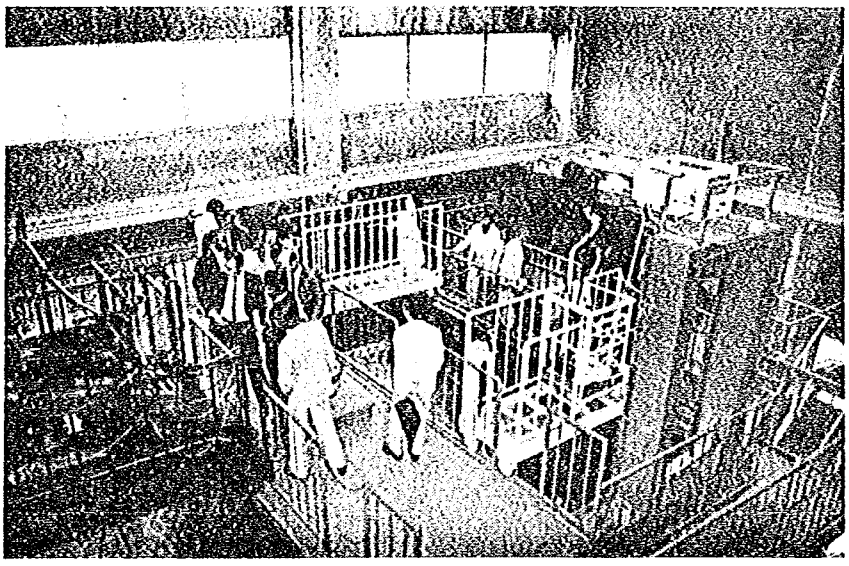
Dr. S. H. Eberle, the Head of German delegation and Mr. T. Tamaki, the Head of Japanese delegation.



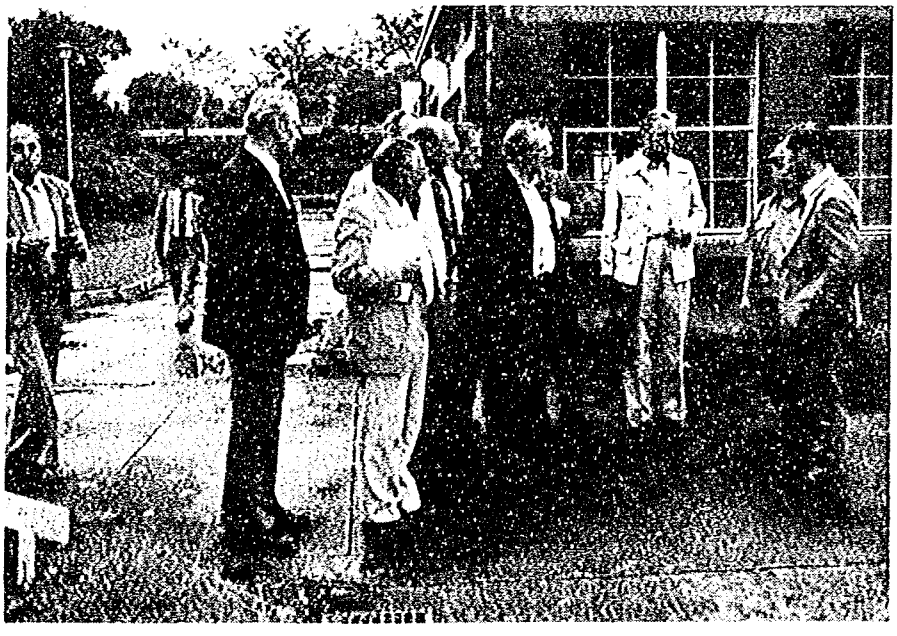
Dr. T. Okubo, Driector General of PWRI greets at the reception held in Tsukuba Science City.



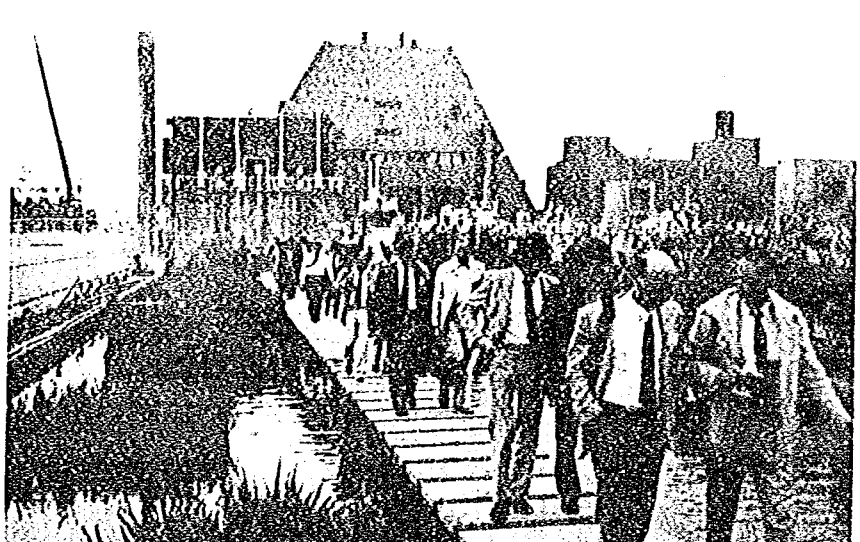
Snap Shot at the reception held in Tsukuba Science City.



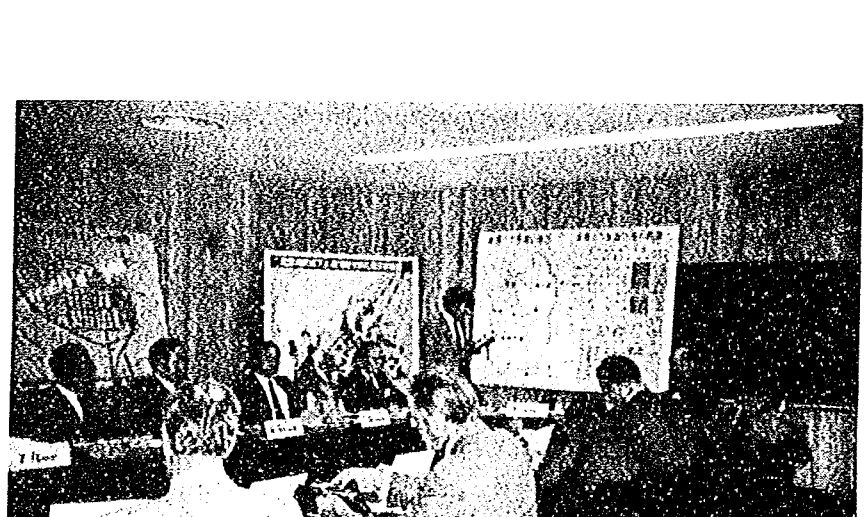
Study Tour at the Public Works Research Institute in Tsukuba Science City.



Study Tour at Mikawashima Sewage Treatment Plant in Tokyo Metropolitan.



Study Tour at Kanagawa Sewage Treatment Plant in Yokohama City.



Study Tour at Konan-chubu Sewage Treatment Plant in Shiga Prefecture.

SCHEDULE OF THE WORKSHOP AND ATTACHED PROGRAM

October 4 – 5, 1982	Workshop ● Public Works Research Institute, Ministry of Construction (Tsukuba, Ibaraki Prefecture)
October 6,	Study Tour ● Public Works Research Institute, Ministry of Construction ● Kohoku Purification Center Kasumigaura Kohoku River-Basin Sewerage (Ibaraki Prefecture) ● Mihawashima Sewage Treatment Plant (Tokyo Metropolitan)
October 7,	Study Tour ● Research and Technology Development Division, Japan Sewage Works Agency ● Kanagawa Sewage Treatment Plant (Yokohama City)
October 8,	Study Tour ● Konan-chubu Sewage Treatment Plant, Lake-Biwa River Basin Sewerage (Shiga Prefecture)
October 9,	Study Tour ● Toba Sewage Treatment Plant, ● Kishoin Sewage Treatment Plant (Kyoto City)

SUMMARY OF OPENING ADDRESS AND CLOSING ADDRESS

Summary of Opening Address

1. Dr. T. Okubo Director General of Public Works Research Institute, Ministry of Construction.
   He explained that the Japanese–German cooperation in the field of environmental protection technology was initiated in 1976 and this cooperation had carried out its activities since that time.
   And he expressed that exchange of results of research and technological information through this workshop was significantly important so as to promote the sewage works efficiently.
   He also hoped success of the workshop.
2. Mr. T. Tamaki the Head of Japanese delegation.
   He expressed welcome to German side delegation and explained the procedure how the workshop was planned. This was proposed at the fourth Japanese–German Panel meeting for Research and Development on New Environmental Protection Technology and reached to an agreement to hold the workshop in Japan at the sixth meeting.
   And he said that this workshop would be fruitful.
3. Dr. S. H. Eberle the Head of German delegation.
   He expressed gratitude to Japanese hosts and explained the German environmental protection policy and emphasized that sewage works played an important role for environmental protection.
   And he said that the workshop was very useful to exchange the knowledge of sewerage field of both countries.

Summary of Closing Address

1. Dr. S. H. Eberle the Head of German delegation.
   He expressed that both side members gained much information on the wastewater technology and got a lot of valuable presonal contacts.
   And at last he expressed his highest sincere thanks to Japanese colleagues.
2. Mr. T. Tamaki the Head of Japanese delegation.
   He said that the two days workshop was very fruitful, so Japanese side would like to make proceedings of this workshop in English and translate German papers into Japanese and publish them on the Journal of JSWA.
   And he expressed that Japanese side would like to make an effort so that the second workshop might be held in Germany in the near future.
   At last he expressed his sincere appreciation to the both sides delegations and the other stuffs.

JOINT COMMUNIQUE

The Japanese-German Workshop on Waste Water and

Sludge Treatment

(Municipal Waste Water Field)

Tsukuba-Science-City, Ibaraki Prefecture, October 5, 1982

1. The Japanese-German Workshop on Waste Water and Sludge Treatment was held in Public Works Research Institute of Ministry of Construction in Tsukuba-Science-City, Ibaraki Prefecture from October 4-5, 1982.
2. The Japanese Delegation headed by Mr. Tsutomu Tamaki, Director General, Department of Sewerage and Sewage Purification, Ministry of Construction was composed of six from the Ministry of Construction, one from the Ministry of Health and Welfare, four from the Japan Sewage Works Agency and one each from the Cities of Tokyo, Yokohama, Kyoto and Osaka.
   In addition to these members many experts from Ministry of Construction, municipalities, Japan Sewage Works Agency, universities and other organizations joined the Workshop.
3. The Federal Republic of Germany Delegation headed by Dr. S. H. Eberle, Director, Division of Water Technology, Nuclear Research Center Karlsruhe was composed of two federal government representatives, three representatives from local authorities and eight representatives from universities and institutes.
   One representative from the German embassy joined the Workshop.
4. The chairmanship of the Workshop was shared by Mr. Tsutomu Tamaki and Dr. S. H. Eberle.
5. Principal topics of the Workshop were:
   Administration and finance of sewerage, biological wastewater treatment, nitrogen and phosphorous removal, reuse of wastewater, treatment of sludge and smallscale municipal wastewater treatment facilities.
   The discussions which followed the presentations were also useful to both countries.
6. Study tours are planned from October 6–9 after these two days session at Public Works Research Institute of Ministry of Construction, Kohoku Sewage Treatment Plant of Kasumigaura-Kohoku Regional Sewerage, Mikawa-shima Sewage Treatment Plant of Tokyo Metropolitan Government, Research and Technology Development Division of Japan Sewage Works Agency, Kanagawa Sewage Treatment Plant of Yokohama City, Konan-Chubu Sewage Treatment Plant of Lake-Biwa Regional Sewerage, Toba Sewage Treatment Plant and Kissyoin Sewage Treatment Plant of Kyoto City.

SESSION 1

ADMINISTRATION AND FINANCE OF SEWERAGE

Industrial Wastewater Control and Water Quality Surcharge System in Osaka City
Kazunari Matsunaga 3
Organization and Economics of a German Waste Water Treatment District
G. Annen 47

INDUSTRIAL WASTEWATER CONTROL AND WATER QUALITY SURCHARGE SYSTEM IN OSAKA CITY

Kazunari Matsunaga
Director
Construction Division
Sewage Works Bureau
Osaka City

1. INTRODUCTION

Nowadays, many cities in Japan are positively promoting the construction of sewerage as a national minimum requirement for healthy and comfortable living and as basic facilities for water quality conservation of public water bodies.

In Osaka City, the ratio of sewered area to total city area now stands at 96.1%, as of fiscal year 1981, the next step being proper management, maintenance and operation of these facilities. For these purposes, many problems are to be solved as follows: to secure stable financial resources, establish a maintenance and operation system, reinforce industrial wastewater control, and establish a stable system of treatment and disposal of sludge. In this paper, the present status of industrial wastewater control and the water quality surcharge system (included in the sewer user charge system) adopted by Osaka City are presented.

2. PROFILE OF OSAKA CITY

Located in the center of the Japanese archipelago, Osaka City has prospered as one of the two major cities of Japan. It is the center of economy and culture in the western part of Japan.

The city faces the Bay of Osaka to the west and lies in the Osaka Plain which is bordered by the Settsu, Kawachi and Izumi mountain ranges to the east. The Uemachi Tableland runs south from slightly east of the business center of the city. The eastern side of the Tableland inclines gently, while the western side slopes notably. As a result, the eastern part of Osaka City has a high altitude, while the western part gradually descends to sea level.

The city of Osaka is often referred to as "the city of water". The River Yodo originating in Lake Biwa, the largest lake in Japan, and many other rivers flow through the city.

The total area of the city is 210.95km^2. The city is divided into 26 administrative wards. The population, according to the national census in 1980, stands at 2,648,200.

Outline of sewage treatment plants and rivers are shown in Fig. 1.

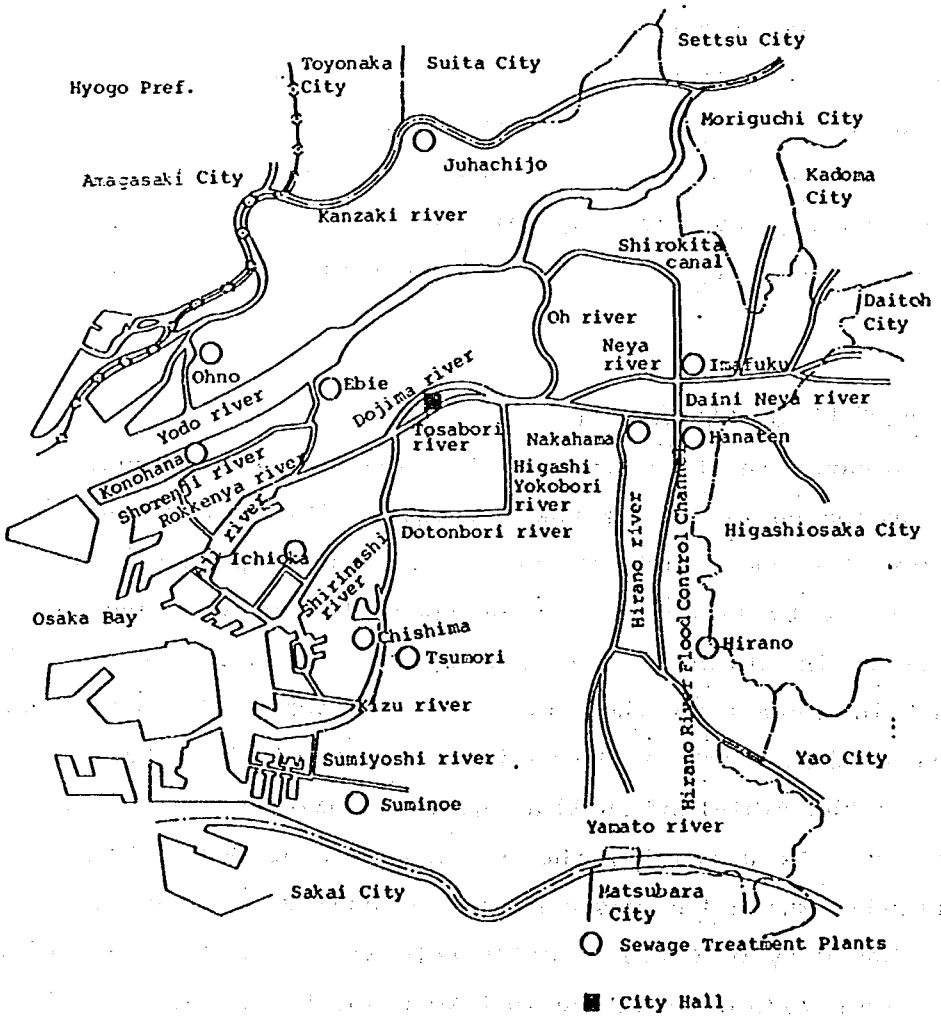


Fig. 1 Outline of Sewage Treatment Plants and Rivers in Osaka City

3. PRESENT STATUS OF SEWERAGE

A modern sewerage construction project began in 1894 to improve the City's sewerage systematically. The driving force was the wide spread of cholera in 1886 and 1890. On those occasions, the sewerage in the center of the city was improved, mainly to prevent cholera and other epidemics. Later, the sewerage in peripheral areas was improved. Since 1922, sewerage improvement has been carried out as part of the city's urban planning project. As water pollution in rivers was triggered, by the increase in the volume of sewage due to the rapid growth of the city, the concentration of population in the urban area and progress in industry, a sewage treatment plan was drawn up in 1928 based on a comprehensive urban plan. In 1940, two sewage treatment plants were constructed and started operation. However, the plan was temporarily suspended by the breakout of World War II.

After World War II, improvement of the sewer systems was promoted to cope with the increased industrial wastewater accompanying the industrial recovery of the post-war period, to solve problems of water pollution in the rivers and of night soil disposal, and to prevent storm surge damage in the western part and inundation in the eastern part. In 1960, a 10-year sewerage improvement plan was inaugurated. As a result of the positive efforts made since then, the sewered area in Osaka reached 45% of the city's total area in 1968.

The prosperity of the city and its economic progress were so remarkable that a need to renew the plan and to improve the sewer systems arose in quite a short time. The first 5-year sewerage improvement plan to construct sewerage throughout the city was established in 1968.

Later, several sewerage improvement plans were put into force, and today 96.1% of the city area is equipped with sewerage, and 98.1% of the city's population use sewerage. Parallel with the 5th National 5-year Sewerage Improvement Plan which was started in 1981 with a total projected cost of ¥11,800 billion, Osaka City established another 5-year plan for sewerage improvement and has

already started this undertaking. The present status of sewerage in Osaka City is shown in Table 1.

Table 1 Present Status of Sewerage in Osaka City

(As of March 31, 1982)

Ⓐ Total Population *Note		2,648,200	Extent of Sewer Piping		4,128km
Ⓑ City's Land Area (Approved area)		185.03km²	Pumping Stations	Number of Stations	59
Drained Area	Ⓒ Total Area	177.78km²		Rain Water Discharge Capacity	589.23m³/sec.
	Installation Ratio Ⓒ/Ⓑ	96.1%	Treatment Plants	Number of Plants	12
Treated Area	Ⓓ Total Area	177.74km²		Treatment Capacity	2,596,000m³/day (Excluding 102,600m³/day for other cities)
	Installation Ratio Ⓓ/Ⓑ	96.1%		Maximum Volume on a Sunny Day	2,315,000m³/day
	Ⓔ Population Receiving Treatment Service	2,597,500		Average Sludge Generated	860t/day
	Installation Ratio Ⓔ/Ⓐ	98.1%		Rain Water Discharge Capacity	417.68m³/sec.
Installation of Flush Toilets					
Ⓕ Estimated Number of Households in Osaka City		700,000			
Ⓖ Number of Households with Flush Toilet		688,110			
Diffusion Rate Ⓖ/Ⓕ		98.3%			

Note: Source
National census Oct. 1980

4. THE 4TH 5-YEAR SEWERAGE IMPROVEMENT PLAN OF OSAKA CITY

4-1 REASON FOR THE PLAN

In order to achieve and maintain the environmental water quality standards for public water bodies, and to eliminate or mitigate inundation, Osaka City has continued its efforts for qualitative improvement of sewerage facilities. By the end of this fiscal year, expansion of a certain sewege treatment plant is to be completed, and all the treatment plants will have been upgraded almost completely. To protect the eastern part of the city from inundation, Benten Pumping Station and the major part of the Tennoji-Benten trunk sewer were completed, and storm water began

flowing through the trunk sewer and being discharged from the Benten Pumping Station in this May.

These projects will surely help achieve the environmental water quality standards established for rivers in the city and contribute to the mitigation of damage caused by inundation, which otherwise spreads over the eastern part of the Uemachi Tableland and other areas.

Since the already improved area is dotted with locales subject to inundation damage due mainly to the increased storm runoff which has followed urbanization, the city's future challenges include construction of additional sewer piping and well-conceived, systematic reinforcement of pumping facilities. It is also necessary to maintain good water quality in public water bodies through stabilization of sewage treatment, and to rebuild out-dated facilities, and to improve other facilities for natural resources and energy conservation. These tasks need be carried out immediately.

For these purposes, Osaka City has come up with a new 5-year sewerage improvement plan, with major emphasis placed on measures against inundation, in harmony with the 5th National 5-year Sewerage Improvement Plan launched in 1981.

4-2 SCHEDULE AND COST

A. Schedule : 1981 ~ 1985

B. Total cost: ¥170 billion

- Breakdown

  Sewer piping : ¥82.8 billion

  Pumping stations: ¥27.6 billion

  Treatment plants: ¥59.6 billion

4-3 FOCUS AND CONTENTS OF THE PLAN

A. Inundation prevention/mitigation project

(Cost: ¥130 billion)

As the improvement of rivers progresses, sewerage improvement work is being carried out for areas most often damaged by inundation, with the objective that by the end of fiscal year 1985 about 80% of such areas will be free from inundation or at least inundation damage will be minimized.

As for installation of sewer piping, a total of 23 trunk sewers extending 26km and branch sewers extending 164km will be constructed. In addition, the pumping capacity of 13 major pumping stations (including 6 pumping stations in the treatment plant sites) will be upgraded.

B. Water quality conservation and other projects

(Cost: ¥40.0 billion)

° Water quality conservation project

By upgrading treatment facilities to stabilize the quality of effluent discharged from treatment plants, the environmental water quality standards for public water bodies in Osaka City can be maintained steadily.

For this purpose, sewage treatment facilities and sludge treatment facilities must be expanded and improved.

° Project for construction of regional sewer system

Construction work for the Northern Neya River Regional Sewerage in Osaka City will be virtually completed by the end of fiscal 1982. The work for the Southern Neya River Regional Sewerage and the Western Yamato River Downstream Regional Sewerage will be completed by the end of fiscal 1985. In this project, trunk and branch sewers extending a total of 82km will be constructed.

° Project for installation of flush toilets

The Sewerage Law stipulates that owners of houses or other structures have to install flush toilets within 3 years after the provision with sewerage system. But, we sometimes face with cases that flush toilets are not installed within 3 years because of different concern between owners and tenants. In such cases the mediation committee system can be utilized to spread 100% installation of flush toilets to the whole city area. The objective is to decrease the number of households without flush toilet to about 1,000 by the end of fiscal 1985, thereby increasing the installation ratio of flush toilets to 99.8%.

5. WATER QUALITY CONSERVATION OF PUBLIC WATER BODIES

The industrial development and urbanization of the high economic growth period which began in the early 1960's has improved the living standards of the people remarkably, while on the other hand it has exacerbated water pollution in public water bodies. To deal with this grave problem, the national government set up environmental water quality standards in 1970 as desired standards for various environmental conditions related to water pollution, based on the Basic Law for Environmental Pollution Control enacted in 1967, and launched various measures for meeting and maintaining these environmental standards, including industrial wastewater control, to which factories discharging wastewater directly into public water bodies are subject, as well as promotion of construction of sewerage systems.

The environmental standards related to water pollution are grouped into two categories. One is related to the protection of human health, as shown in Table 2. The standards of this category apply to all public water bodies, and must be met immediately.

Table 2 Environmental Standards for Human Health

(Toxic Substances)

Item	Cyanogen	Alkyl mercury	Organic phospho-rus	Cadmium	Lead	Hexavalent chromium	Arsenic	Total mercury
Standard Values	Not de-tecta-ble.	Not de-tecta-ble.	Not de-tecta-ble.	0.01ppm or less	0.1ppm or less	0.05ppm or less	0.05ppm or less	Not de-tecta-ble.

The other category is related to the protection of the living environment. The standards are based on the classification of rivers, coastal waters and lakes into several types. As shown in Table 3, classification is made according to the use of individual public water bodies, thereby setting specific environmental standards. As of March 1979, 2,236 rivers, 98 lakes and marshes, and 532 coastal waters (a total of 2,866 water bodies) were subject to this classification.

Table 3 Environmental Standards for the Living Environment

1. Rivers

(1) Rivers (Excluding Lakes and Marshes)

Item / Category	Use	Standard Values				
		pH	Biological Oxygen Demand (BOD)	Suspended Solids (SS)	Dissolved Oxygen (DO)	Number of Coliform Groups
AA	Water supply class 1 Natural environment conservation and uses listed in A∿E	6.5∿8.5	1ppm or less	25ppm or less	7.5ppm or more	50MPN/ 100ml or less
A	Water supply class 2 Fishery class 1 Bathing Uses listed in B∿E	6.5∿8.5	2ppm or less	25ppm or less	7.5ppm or more	1,000MPN/ 100ml or less
B	Water supply class 3 Fishery class 2 Uses listed in C∿E	6.5∿8.5	3ppm or less	25ppm or less	5ppm or more	5,000MPN/ 100ml or less
C	Fishery class 3 Industrial water class 1 Uses listed in D∿E	6.5∿8.5	5ppm or less	50ppm or less	5ppm or more	—
D	Industrial water class 2 Argicultural water Uses listed in E	6.0∿8.5	8ppm or less	100ppm or less	2ppm or more	—
E	Industrial water class 3 Environmental conservation	6.0∿8.5	10ppm or less	Floating matter such as garbage should not be vissible.	2ppm or more	—

(2) Lakes (Natural Lakes, Reservoirs, Marshes, and Artificial Lake or More Than 1 × $10^7 m^3$ Water)

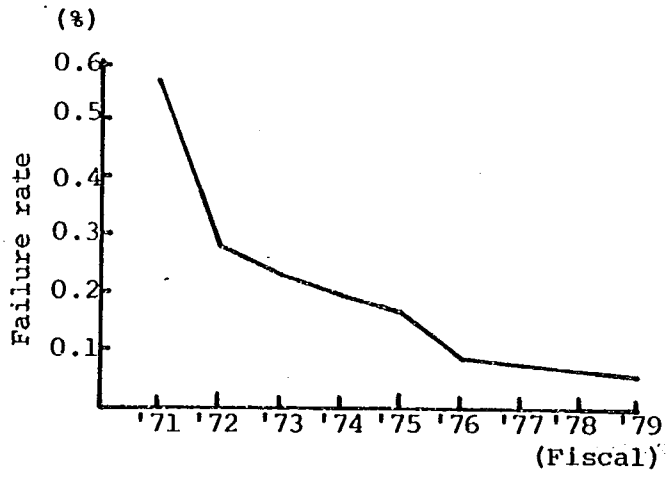
Item / Category	Use	Standard Values				
		pH	Chemical Oxygen Demand (COD)	Suspended Solids (SS)	Dissolved Oxygen (DO)	Number of Coliform Groups
AA	Water supply class 1 Fishery class 1 Natural environment conservation and uses listed in A ∿C	6.5 ∿ 8.5	1ppm or less	1ppm or less	7.5ppm or more	50MPN/ 100ml or less
A	Water supply classes 2 and 3 Fishery class 2 Bathing Uses listed in B ∿C	6.5 ∿ 8.5	3ppm or less	5ppm or less	7.5ppm or more	1,000MPN/ 100ml or less
B	Fishery class 3 Industrial water class 1 Agricultural water Uses listed in C	6.5 ∿ 8.5	5ppm or less	15ppm or less	5ppm or more	—
C	Industrial water class 2 Environmental conservation	6.0 ∿ 8.5	8ppm or less	Floating matter such as garbage should not be vissible.	2ppm or more	—

2. Coastal Waters

Item / Category	Use	Standard Values				
		pH	Chemical Oxygen Demand (COD)	Dissolved Oxygen (DO)	Number of Coliform Groups	N-hexane Extracts (Oils, etc.)
A	Fishery class 1 Bathing Uses listed in B ∿C	7.8 ∿ 8.3	2ppm or less	7.5ppm or more	1,000MPN/ 100ml or less	Not detectable.
B	Fishery class 2 Industrial water Use listed in C	7.8 ∿ 8.3	3ppm or less	5ppm or more	—	Not detected.
C	Environmental conservation	7.0 ∿ 8.3	8ppm or less	2ppm or more	—	—

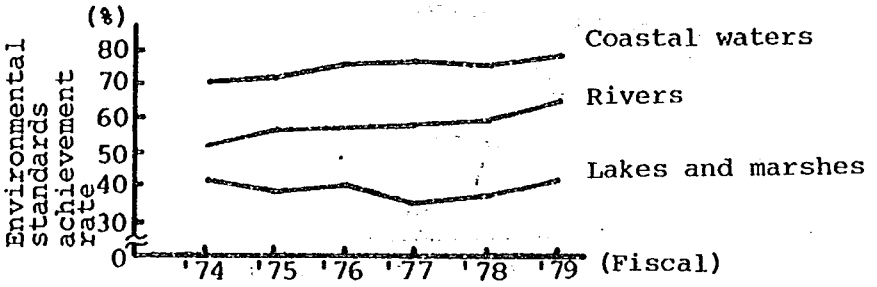
Now, let us see how these environmental standards are met. With regard to the environmental standards related to human health, only 0.06% of subjects tested at 4,996 points nationwide failed to meet the standards in fiscal 1979, as shown in Fig. 2, evidencing remarkable improvement.

With regard to environmental standards related to conservation of the living environment, 65.0% of rivers, 41.8% of lakes and marshes and 78.2% of coastal waters met the standards, showing a general trend of improvement (Fig. 3).



Note: The failure rate is the ratio of subjects failing to meet the standards to the total number of subjects tested.
Remarks: Studied by the Environmental Agency

Fig. 2 Change in Rate of Failure to Meet Health Item Standards (Toxic Substances)



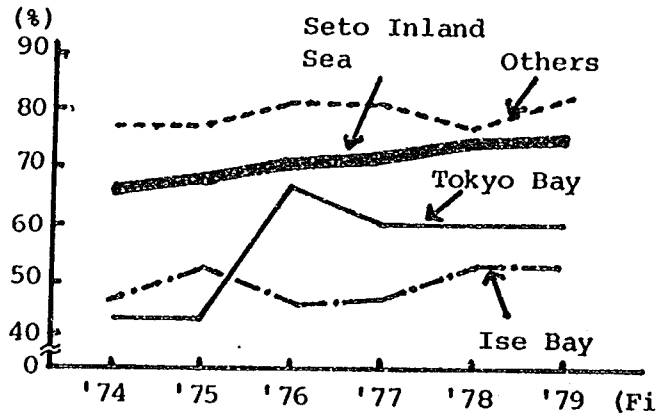
Note:

$$\text{Achievement rate} = \frac{\text{Number of water bodies meeting environmental standards}}{\text{Total number of water bodies subject to environmental standards}} \times 100(\%)$$

Remarks: Studied by the Environmental Agency

Fig. 3 Change in Achievement of Environmental Standards for the Living Environment (BOD (COD for Lakes, Marshes and Coastal Waters))

It must be noted, however, that the improvement of the water quality in stagnant coastal waters such as Seto Inland Sea, Tokyo Bay and Ise Bay flattened out as shown in Fig. 4, even in coastal waters with comparatively closer achievement of the target. Under such circumstances, an effluent mass control system for COD was introduced in 1978 for the first time for these three coastal waters. In these areas, the presence of nitrogen, phosphorus and other nutritive salts help breeding and growth of aquatic life such as plankton and algae, promoting eutrophication. As a result, red tides are generated, causing immeasurable damage to fishing resources.



Note: $\text{Achievement rate} = \frac{\text{Number of water bodies meeting environmental standards}}{\text{Total number of water bodies subject to environmental standards}} \times 100(\%)$

Remarks: Studied by the Environmental Agency

Fig. 4 Change in Achievement of Environmental Standards in Large Stagnant Water Bodies (COD)

Fig. 5 shows change in pollution of rivers flowing through the city of Osaka. Actually, almost all rivers have met the standards, with only a few exceptions. The main contributing factor is that nearly 100% of the city area is sewered. Yet, a few rivers have not met the standards because, we presume, upstream cities have inadequate sewerage.

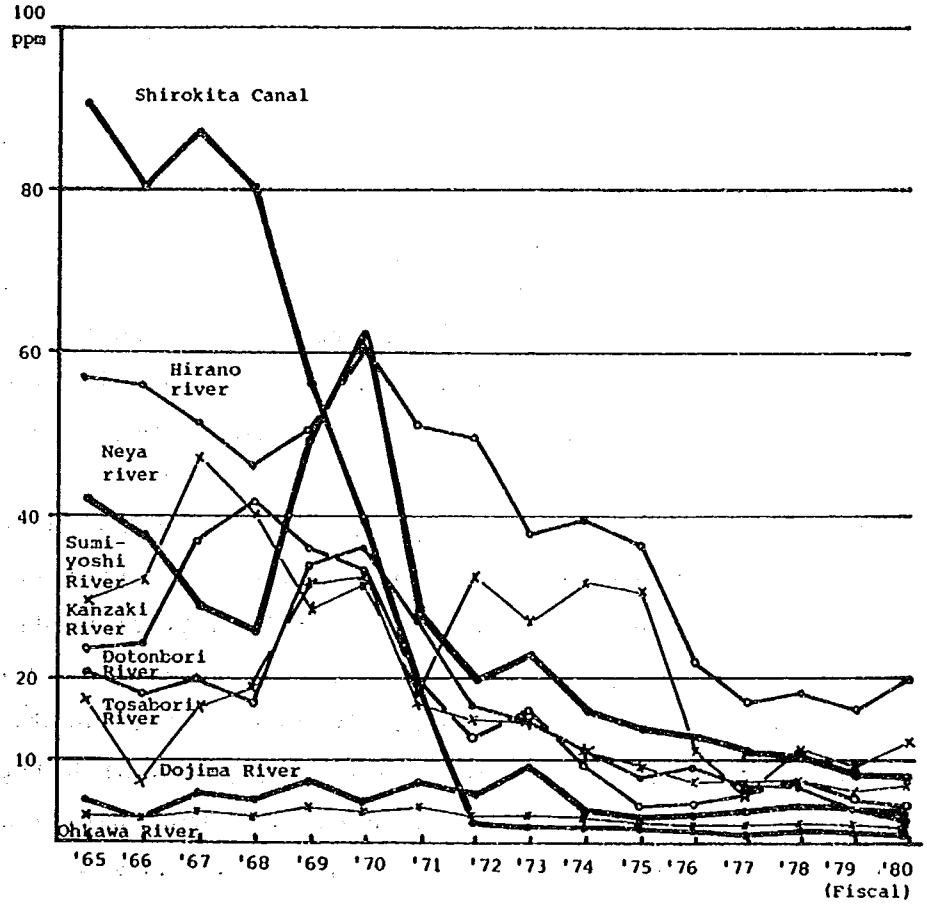


Fig. 5 Change in Pollution of Rivers in Osaka City (BOD)

6. INDUSTRIAL WASTEWATER CONTROL

As stated earlier, the fundamental construction work of public sewer systems is almost completed in Osaka City, although redoubled efforts are still required for measures against inundation and for qualitative improvement of municipal sewage treatment plants. At the same time, adequate maintenance and operation of completed facilities is also essential to conservation of water quality in public water bodies.

To fulfill the purpose of the sewerage system, the quality of wastewater flowing into the sewage treatment plant must be controlled properly, and industrial wastewater adversely affecting sewerage facilities must be strictly controlled.

Guidance for the control of wastewater discharged from industries in Osaka City has been given from the late 1960's, from the viewpoint of minimizing damage to sewerage facilities. Following the amendment of the Sewerage Law in 1970, Osaka City revised municipal ordinances in 1972 to require conformance to the effluent standards and installation of pretreatment facilities, which marked the beginning of thorough control of industrial wastewater. In 1976, the municipal ordinances were further revised in parallel with the amendment of the Sewerage Law in 1976. At present, Osaka City controls wastewater discharged from factories with the same strictness seen in the Water Pollution Control Law which controls wastewater discharged directly into public water bodies.

6-1 LAWS AND ORDINANCES RELATED TO INDUSTRIAL WASTEWATER CONTROL

The laws and ordinances for the control of industrial wastewater are divided into two categories, as shown in Table 4: one is for the prevention of damage to sewer piping and pumping facilities, and the other is to maintain the quality of effluent discharged from sewage treatment plants within the standards.

Table 4 Laws and Ordinances for Industrial Wastewater Control in Sewered Area

Subject	Law	Standards Applied	Water Quality Item	Quantity of Water	Governing Ordinance	Purpose
Specified factories	Article 12.2	Restrictive standards	Health items	Irrespective of quantity	Government ordinance	To maintain quality of effluent discharged from sewage treatment plants within the standards
			Environmental items (Excl. pH, BOD, SS and n-hexane extracts)	$50m^3$/day or more, in principle	Government ordinance	
			Environmental items (pH, BOD, SS, and n-hexane extracts)	$50m^3$/day or more, in principle	Municipal ordinance	
	Article 12.10	Pretreatment standards	Environmental items	Less than $50m^3$/day, in principle	Municipal ordinance	
Non-specified factories	Article 12.10	Pretreatment standards	Health items Environmental items	/ (In compliance with municipal ordinance)	Municipal ordinance	
Irrespective of types of all factories	Article 12	Pretreatment standards	Temperature, pH n-hexane extracts and iodine consumped	/ (In compliance with municipal ordinance	Municipal ordinance	To prevent damage to sewer piping and pumping facilities

6-1-1 CONTROL FOR PROTECTION OF SEWERAGE FACILITIES

The municipal ordinance requires industries, which discharge wastewater continuously into the public sewerage system, thus remarkably damaging or hampering the functions of the sewerage facilities, to install pretreatment facilities, in compliance with the standards stipulated by the government ordinance. Before beginning installation, such industries must submit their plan for installation of pretreatment facilities and other required items to a competent office. The standards for installation of pretreatment facilities are given in Table 5.

Table 5 Standards for Protection of Sewerage Facilities

Temperature	Less than 45°C
pH	Over 5 to 9 excl.
Normal-hexane extracts	
Mineral oils	5mg/ℓ or less
Animal and vegetable oils and fats	30mg/ℓ or less
Iodine consumed	Less than 220mg/ℓ

6-1-2 CONTROL FOR MAINTAINING QUALITY OF EFFLUENT DISCHARGED FROM SEWAGE TREATMENT PLANTS WITHIN THE STANDARDS

The standards are provided to control industrial wastewater to comply with the technical standards for quality of effluent discharged from sewage treatment plants stipulated by the government ordinance. Controls are grouped into two categories: Controls with direct penalties for specified factories, and those requiring installation of pretreatment facilities instead of direct penalty.

6-1-2-1 DIRECT PENALTY CONTROL STANDARDS FOR SPECIFIED FACTORIES (RESTRICTIVE STANDARDS)

In the Water Pollution Control Law, facilities likely to generate polluted water in various industrial processes are designated "specified facilities". The law controls factories with the specified facilities (referred to as "specified factories"). The Sewerage Law, likewise, provides standards for these specified factories and penalizes any violators directly.

The standards for direct penalties are classified into uniform standards relating to items which cannot be dealt with at sewage treatment plants and are therefore provided for by the government ordinance, and standards relating to items which can be dealt with at sewage treatment plants, which are provided for by entity having jurisdiction over the public sewerage in accordance with the standards set forth in the government ordinance and in consideration of the capacity of each plant.

The uniform standards provided by the government ordinance are as shown in Table 6.

In principle, the application of these standards must conform to those contained in the Water Pollution Control Law which is applied to industries discharging wastewater directly into public water bodies. Accordingly, items (1) through (9) are applied to all specified factories, irrespective of the volume of effluent discharged. On the other hand, items (10) through (16) shall not be applied to specified factories with an average discharge of less than $30m^3$/day.

The items with which sewage treatment plants have sufficient treatment capability are as follows.

These standards are, as mentioned earlier, established in the municipal ordinance by the party having jurisdiction over public sewerage in accordance with the standards contained in the government ordinance, so long as the functions of the sewage treatment plant will not be adversely affected.

Table 6 Uniform Standards by Government Ordinance

(1)	Cadmium and its compounds	Cadmium: 0.1mg/ℓ or less
(2)	Cyanogen compounds	Cyanogen: 1mg/ℓ or less
(3)	Organic phosphorous compounds	1mg/ℓ or less
(4)	Lead and its compounds	Lead: 1mg/ℓ or less
(5)	Hexavalent chromium compounds	Hexavalent chromium: 0.5mg/ℓ or less
(6)	Arsenic and its compounds	Arsenic: 0.5mg/ℓ or less
(7)	Mercury, alkyl mercury and other mercury compounds	Mercury: .0.005mg/ℓ or less
(8)	Alkyl mercury compounds	Not detectable
(9)	PCB	0.003mg/ℓ or less
(10)	Phenols	5mg/ℓ or less
(11)	Copper and its compounds	Copper: 3mg/ℓ or less
(12)	Zinc and its compounds	Zinc: 5mg/ℓ or less
(13)	Iron and its compounds (soluble)	Iron: 10mg/ℓ or less
(14)	Manganese and its compounds (soluble)	Manganess: 10mg/ℓ or less
(15)	Chromium and its compounds	Chromium: 2mg/ℓ or less
(16)	Fluorine compounds	Fluorine: 15mg/ℓ or less

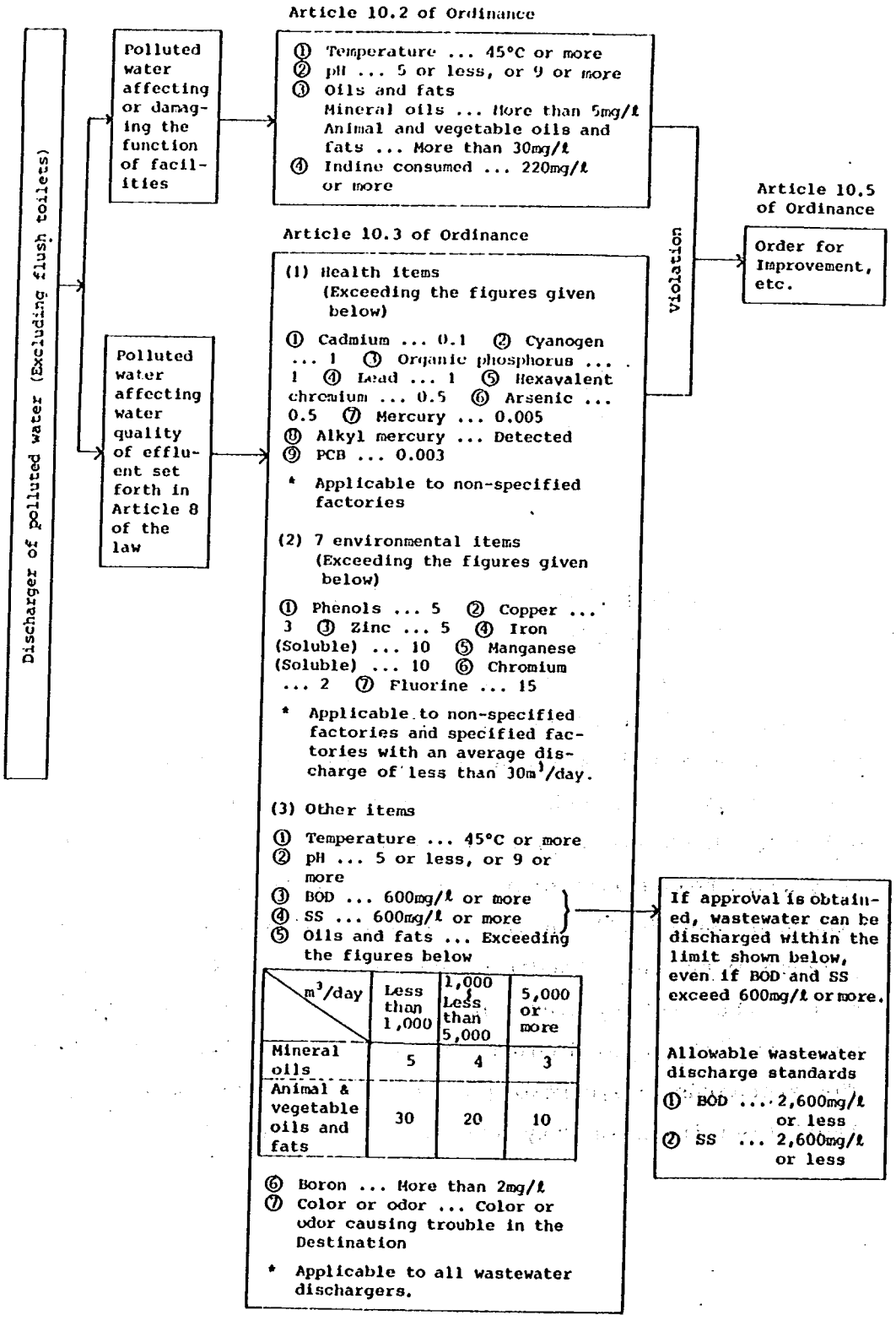
As in the case of the uniform standards, these standards shown in Table 7 shall not be applied to specified factories with an average discharge of less than $50m^3$/day, in compliance with the Water Pollution Control Law.

Table 7 Direct Penalty Control Standards for Specified Factories by Osaka Municipal Sewerage Ordinance

pH		Over 5 to 9 excl.
BOD		2,600mg/ℓ or less
SS		2,600mg/ℓ or less
Oils and Fats	Mineral Oils	5mg/ℓ or less
	Animal and Vegetable Oils and Fats	30mg/ℓ or less

6-1-2-2 STANDARDS FOR CONTROLS REQUIRING INSTALLATION OF PRETREATMENT FACILITIES (PRETREATMENT STANDARDS)

The standards for controls requiring installation of pretreatment facilities are applied to all factories to which the above-mentioned direct penalty control standards are not applied, contrary to the standards described in the immediately preceding 6-1-2-1, which are applicable to specified factories. If a factory is likely to discharge polluted water and fails to meet the standards set forth in the municipal ordinance, which is in compliance with the standards established by the government ordinance, the factory must install a pretreatment facility or take necessary steps to improve the quality of wastewater discharged. The standards for controls requiring installation of pretreatment facilities in Osaka City are as shown in Fig. 6.



m³/day	Less than 1,000	1,000 Less than 5,000	5,000 or more
Mineral oils	5	4	3
Animal & vegetable oils and fats	30	20	10

Fig. 6 Standards for Controls Requiring Installation of Pretreatment Facilities by Osaka Municipal Sewerage Ordinance

6-2 PRESENT STATUS OF MONITORING SYSTEM

For stringent control of industrial wastewater, the establishment of a monitoring system is essential. The proposal from the Fourth Sewerage Financing Study Committee pointed out: "In order to prevent the inflow of contaminated industrial wastewater into sewerage, to maintain sewerage facilities, and to maintain the quality of effluent discharged from sewage treatment plants, an effective monitoring system must be developed in close cooperation with the environmental administration". In addition, the Sewerage Law empowers local municipalities with extensive authority for the control of industrial wastewater. It is a nation-wide task to complete a system which can exercise this authority to the fullest extent.

In Osaka City, the monitoring system has been established step by step since 1973. Presently, the Water Pollution Control Department (comprising 4 sections) of the Administration Division is in charge of all industrial wastewater control and water quality analyses. In this department, 53 staff members work. Among them 32 people form 16 teams (each consisting of two people) so that at least 11 teams are on duty every day.

The functions of the four sections in the Water Pollution Control Department are as follows:

(1) Administration section (19 staff members)

Supervisory administration of sewage quality control; research for designation of factories subjected to water quality surcharge; and research on substances not yet controlled.

(2) First and second control sections (22 staff members)

Industrial wastewater control and provision of guidance for installation of pretreatment facilities for the entire city, divided into two areas.

(3) Water quality examination center (12 staff members)

Water quality examination of industrial wastewater; research on sewage treatment functions; and technical supervision of water quality examination

Since most of the city is sewered, this department also supervises water pollution control in factories and other entities which discharge their wastewater directly into rivers and other public water bodies subject to the Water Pollution Control Law and the Factory Pollution Control Ordinance of Osaka Prefecture.

6-3 FACTORIES SUBJECT TO CONTROL

Osaka City, as the center of the economy in the western half of Japan, has prosperous industrial activities. According to industrial classification, the number of manufactures totals 34,772 factories as shown in Table 8 (Industrial Statistics, Dec. 1980). More than 90% of these manufactures are small enterprises with not more than 19 employees. These factories are concentrated in areas where commercial and industrial activities and people's residences coexist, except for the coastal heavy chemical industrial complex. We conducted a fact-finding survey on about 18,000 factories (factories using water of 100m^3/month or more plus specified factories). As a result, we found that there were 5,869 specified factories (as of Oct. 31, 1981), among which 2,807 factories required installation of pretreatment facilities. In addition, among non-specified factories, 319 factories required installation of pretreatment facilities, which means that a total of 3,126 factories required some sort of wastewater treatment. Under the guidance for installation of pretreatment facilities, which began in around 1970, the installation rate now stands at 99.4%.

Table 8 Classification of Manufactures in Osaka City by Size

(Source: Osaka Municipal Statistics Bulletin No. 330, Dec. 1980)

Total	With 19 or Less Employees	With 20 or More Employees
34,772	31,513 (90.6%)	3,259 (9.4%)

Among specified factories, 1,099 were factories subject to direct penalties discharging toxic substances or with an average wastewater discharge of $30m^3$/day or more. All 848 factories discharging substances listed in Table 6 are now equipped with pretreatment facilities. (See Table 9.)

Table 9 Classification of Factories Requiring Pretreatment Facilities

As of Oct. 31, 1981

Subject	Law		Factories Requiring Installation	Factories Equipped with Pretreatment Facilities	Rate of Installation
Specified factories	Article 12.2		1,099	1,094	99.6%
		Section 1	848	848	100
		Section 5	503	498	99.0
	Article 12.10		1,708	1,696	99.3
	Article 12		—	—	—
	Sub-total		2,807	2,790	99.4
Non-specified factories	Article 12.10		319	316	99.1
	Article 12		—	—	—
	Sub-total		319	316	99.1
Total			3,126	3,106	99.4

Note: Section 1 and Section 5 overlap in Article 12.2.

The installation of pretreatment facilities classified by size of enterprise is shown in Table 10. As is obvious from this table, more than 92% of the factories requiring installation are medium, small and very small enterprises.

Table 10 Installation of Pretreatment Facilities Classified by Size of Enterprise (As of Oct. 31, 1981)

Installation of Pretreatment Facilities / Size of Enterprises	Factories Requiring Installation	Factories Equipped with Pretreatment Facilities	Rate of Installation %
Large enterprises	100	100	100
Medium & small enterprises	1,057	1,053	99.6
Very small enterprises	1,841	1,825	99.1
Public utilities and undertakings	128	128	100

Note: Classification of enterprises is in accordance with the Minor Enterprises Act.

Efforts are being made to bring the rate of installation to 100% for factories not yet equipped with pretreatment facilities, including technical guidance such as improvement of manufacturing processes and administrative guidance such as provision of good offices for funding.

6-4 SPOT INSPECTIONS

As the installation of pretreatment facilities has reached the stage of virtual completion, the emphasis of supervision has shifted to proper maintenance and operation of installed pretreatment facilities. At present 16 patrol teams, each consisting of 2 people, are organized so that 11 teams are on duty every day.

In spot inspections, the inspection team samples treated wastewater discharged from the final discharge outlet, checks all discharge outlets for domestic wastewater and rainwater for leakage of polluted water and abnormality, and enters the factory to check wastewater from the manufacturing process and operation of pretreatment facilities. If, as a result of water quality examination, the sampled wastewater is found in violation of the pretreatment standards, the cause is identified and strict warning given to the violator. For the worst violators, administrative measures, including an order for improvement and temporary suspension of wastewater discharge, are taken to ensure prompt action.

About 6,000 spot inspections are conducted annually. In fiscal 1981, intensive surveillance was given to four sewage treatment districts where factories discharging heavy metals are concentrated. The administrative measures taken for vicious violators are as shown in Table 11. In fiscal 1981, 29 violators were given orders for improvement and 4 others received orders for temporary suspension of wastewater discharge.

Table 11 Annual Change in Administrative Measures

Administrative Measure \ Fiscal Year	1973	1974	1975	1976	1977	1978	1979	1980	1981
Order for improvement	32	42	37	24	12	26	32	25	29
Temporary suspension of wastewater discharge	—	1	—	—	3	1	1	2	4

Furthermore, as a step to complement spot inspections, automatic samplers have been installed temporarily in the trunk sewer and pumping stations for continuous monitoring of wastewater. Based on the results of water quality examination of the samples, night patrols or simultaneous spot inspections in a certain area are conducted, as necessary, with the objective of reinforcing monitoring of wastewater treatment outside business hours.

6-5 FINANCING FOR POLLUTION PREVENTION FACILITIES

Huge funds are required for installation and improvement of pretreatment facilities, which often impose a heavy financial burden on small enterprises. Osaka City has established the Pollution Prevention Facilities Funding System to provide loans to medium and small enterprises and to aid them in terms of interest. Table 12 shows how this system is employed by enterprises. The system was most often utilized in 1973, 1974 and 1975. In recent years, the system is widely used for improvement and rebuilding of pre-treatment facilities.

Outline of Osaka City's Pollution Prevention Facilities Funding System (Fiscal 1982)

1. Loans

   ¥25 million per enterprise (with security)

   ¥ 6 million per enterprise (without security)

2. Term

   7 years (Unredeemable for 1 year)

3. Interest rate

   7.5% per annum

4. Aid by Osaka City

   Aid in interest rate — 6.5% per annum for small enterprises

   5.5% per annum for medium enterprises

5. Guarantee fee

   0.95% per annum (for loan of ¥1 million or more)

Table 12 Annual Change in Osaka City's Pollution Prevention Facilities Funding

Fiscal Year	1970	1971	1972	1973	1974	1975	1976	1977	1978	1979	1980
Number of Enterprises Using the System	12	14	40	100	147	90	46	60	35	41	36
Loans (Unit: Million Yen)	78.2	96.4	235.27	625	957.5	549.6	268.5	316.5	236.5	291.3	200

6-6 FUTURE TASKS

As stated earlier, remarkable improvement has been made in the installation of pretreatment facilities. Now, the functions of sewage treatment plants are rarely affected by the wastewater flowing into the plant throughout the year. Content of heavy metals in the sludge in sewage treatment plants has been reduced drastically, compared with 1973 and before. Yet, heavy metals in the sludge must be further reduced through various measures including thorough maintenance and operation of pretreatment facilities. The future tasks that Osaka City has to deal with are described below.

6-6-1 DEVELOPMENT OF INDUSTRIAL WASTEWATER TREATMENT TECHNOLOGY

Conventionally, pretreatment facilities have been constructed to maintain wastewater discharged from manufacturing processes within the pretreatment standards. In the future, however, it will be more necessary to develop wastewater treatment methods, such as improvement of manufacturing processes and adoption of a closed-loop system, in consideration of reduction of generation of industrial waste, the need to recycle wastewater and effective utilization of natural resources to prevent environmental pollution. As regards the closed-loop system, some metal plating factories have already adopted it using ion-exchange resin for the purpose of energy saving. In the future, less costly, technically simpler methods must be developed so that they can be utilized by small enterprises.

6-6-2 THOROUGHGOING EFFORTS FOR MAINTENANCE AND OPERATION

Most of the causes of violation we find as a result of water quality analysis of sampled wastewater in daily spot inspections are inadequate maintenance and operation of pretreatment facilities. Inadequate maintenance and operation of pretreatment facilities results in not only the failure to observe the pretreatment standards but also the damage to the facilities and shortening of facilities' life.

Basic maintenance and operation of pretreatment facilities consists in daily proper operation and daily inspection and maintenance.

For this purpose, Osaka City provides factories with thorough guidance, including the appointment of a supervisor in charge of pretreatment facilities, who is required to check water quality at all times, ensure that the pretreatment facility is operated and maintained normally (e.g. checking, cleaning and calibration of measuring instruments), and keep daily records of operation.

Medium and small enterprises, lacking space, have installed all or part of pretreatment facilities underground below the manufacturing process, or on top of the building or outdoors far away from the process. Although it must be admitted that in most small enterprises, the manager of the factory operates not only the management of the enterprise but also that of the pretreatment facility, despite the need for more stringent control, the failure to meet the standards seems to originate from insufficient understanding of the importance of water quality control and lack of social responsibility.

To deal with this situation, Osaka City has taken every opportunity to hold explanatory forums for enterprises to widen their correct knowledge of sewarage facilities and raise their awareness of the importance of water quality control. As a consequence, considerable efforts are now being made for better maintenance and operation of pretreatment facilities.

In Osaka, a system to appoint supervisors responsible for water quality control, which has already been instituted in some cities, is now under examination.

6-6-3 JOINT TREATMENT FACILITIES

The 4th sewerage Financing Study Committee pointed out that "joint treatment must be promoted through concentrated siting of enterprises to push forward measures for medium and small enterprises". This method seems viable, since industrial complexes and apartments have been tried out as general pollution preventing

measures, yielding favorable results. However, in promoting joint treatment through concentrated siting of factories, certain preconditions must be met. First, adequate sites must be secured. Second, huge funds are required for the construction of plants and wastewater pretreatment facilities. Third, old factories of individual enterprises must be sold after transfer, so that the proceeds can be used as funds for the transfer of factories (Osaka City has introduced a system to purchase land occupied by pollution-generating factories.). Since enterprises do not participate in such a project unless they have certain prospects regarding these problems, only enterprises with firm managerial footing have transferred their factories. In this regard, the national government and local public entities must provide positive financial aids, including construction funds, stop-gap funds, and purchase of land previously used by factories.

In some cities, sludge generated from pretreatment facilities and wastewater, waste acids and waste alkalis generated from various manufacturing processes are treated jointly by public entities and cooperatives. We believe that joint treatment or disposal of industrial wastes will be very effective, especially for medium and small enterprises. At present, Osaka City has the Waste Acids Treatment Center (cooperative) and Clean Osaka Center (for treatment and disposal of sludge, funded by Osaka City). The gist of these measures to deal with industrial wastes is centralized treatment/disposal by public entities, which does not trigger secondary pollution, especially for small and medium enterprises. This issue requires further examination.

6-6-4 DEVELOPMENT OF CONTINUOUS WATER QUALITY MONITORING SYSTEM

The purpose of industrial wastewater control is to determine the actual condition at a factory by spot inspection and to provide adquate instructions and guidance. However, spot inspections

alone, which are conducted twice a year on the average, cannot fulfill the objective. Under these circumstances, the development of a continuous water quality monitoring system using automatic analysers is necessary.

To ensure the enforcement of effluent mass control concerning COD, required by the Water Pollution Control Law, prefectural governments concerned have been promoting the establishment of a COD continuous monitoring system using telemeters. Before this, Osaka City carried out the model project of continuous water quality monitoring system using telemetry with the aid of subsidies from the Environment Agency in 1978, to examine whether it is technically possible to measure and record COD load by automatic instruments. The monitoring system was completed in fiscal 1981. And now, COD is monitored at all times by 9 telemeters installed at the Factory Monitoring Stations, 13 at the Sewage Treatment Plant Monitoring Stations, and 10 at the River Monitoring Stations.

Using this system as a model, a system to monitor wastewater discharged by factories into sewerage must be developed. The national government has already started developing automatic measuring instruments. However, technological development for automatic instruments to be employed in the continuous monitoring system for all the regulated substances is insufficient and is not yet put into practical use.

6-6-5 RESPONSE TO EFFLUENT MASS CONTROL SYSTEM

The major problem in industrial wastewater control is the response to the effluent mass control system for COD, which was put into force in July 1981. Direct dischargers to public water bodies with not less than 50m^3/day of wastewater among specified factories are subject to this system.

The control is based on the findings that restrictions on concentration are not sufficient to achieve the environmental standards in stagnant water bodies: Seto Inland Sea, Tokyo Bay and Ise Bay.

The system intends to reduce the total mass of organic pollutants affecting COD, which flow into each water body, as long as is practically available.

The outline of the system is as follows. The national government established fundamental policies to reduce total mass of organic pollutants for individual water bodies, assigned each water body and related prefectures reduction targets, and determined the range of effluent mass control standards. Based on these fundamental policies, the prefectural governments concerned worked out mass reduction plans to achieve the target and set the standards for effluent mass control.

The standards for effluent mass control at each sewage treatment plant, established by the national government, are as follows.

$$L = C \times Q \times 10^{-3}$$

Where L: Permissible pollution load (kg/day)

Q: Capacity of existing treatment plant (m^3/day)

C: COD concentration applied to each industry (mg/ℓ) (Shown in Table 13)

Table 13 Range of Value C

Treatment Process	Range of Value C
Activated sludge process, conventional trickling filter process, and other equivalent processes	30 ∿ 60mg/ℓ
High-rate trickling filter process, modified aeration process, and other equivalent processes	60 ∿ 80mg/ℓ
Sedimentation process	80 ∿ 110mg/ℓ
More advanced treatment process than activated sludge process or conventional trickling filter process	10mg/ℓ

Based on these guidelines, values C for sewage treatment plants in Osaka Prefecture are set forth as shown in Table 14.

Table 14 Value C for Sewage Treatment Plants in Osaka Prefecture

Classification of Industry			*1	*2	*3	Remarks
201	Sewerage works	(1) Plants where sewage can be treated by the activated sludge process, conventional trickling filter process, or other equivalent processes.	30	40	30	(1) For plants where sewage can be treated by more advanced treatment processes than activated sludge process, conventional trickling filter process, or other equivalent processes, values in Section 3 shall be 20, 20 and 20 in the order of listing. (2) Notwithstanding the provision in Section 3, values in this section shall be 40, 50 and 30 in the order of listing, for plants designated by the governor of the prefecture.
		(2) Plants where sewage can be treated by the high-rate trickling filter process, modified aeration process, or other equivalent processes.	60	70	30	

Note 1: Value C applied to industries founded before June 30, 1980, from July 1, 1984
2: Value C applied to industries founded before June 30, 1980, till June 30, 1984
3: Value C applied to industries founded after July 1, 1980, from July 1, 1980

This system also requires specified factories which discharge wastewater of not less than 400m^3/day to measure pollution loads using automatic measuring instruments. At sewage treatment plants in Osaka City, COD, TOC and UV automatic measuring instruments are used.

It must be pointed out that some sewage treatment plants have difficulties in responding to the introduction of the effluent mass control system. These plants employ the biological treatment process, which is capable of treating more than 90% of inflow BOD but only 65 ~ 80% of inflow COD. The major causes are high pollution load of some industrial wastewaters and the inclusion of substances which cannot be easily decomposed by the conventional biological process. Therefore, the situation emerges that even if adequate sewage treatment is carried out at some treatment plants, actual effluent loads of COD fail to meet the effluent mass control

standards, making it necessary to examine COD inflow restrictions for industrial wastewater flowing into the public sewerage system.

At present, the national government is examining methods to measure hardly biodegradable substances as well as procedures for their restrictions. The method of measurement of hardly biodegradable substances has almost been established.

7 WATER QUALITY SURCHARGE SYSTEM

7-1 CHANGE IN SEWER USER CHARGES

In Osaka City, sewer user charges began to be collected in 1940, when two sewage treatment plants started their operation. The collected money was used for maintenance and operation of the plants.

Thereafter, sewer user charges were revised several times. It was not until the early 1960's that the charges began to be regarded as important financial resources, as the sewer construction projects developed, and the diffusion rate reached 35%.

In 1968, a 5-year sewage improvement plan was made, and sewer user charges were revised, based on the principle that treatment costs for rainwater should be paid for by the public and sewage treatment costs by those who receive sewerage service. The purpose of this revision was to secure financial resources for maintenance and operation of the city's sewerage systems and for redemption of principal and interest when bonds are issued. Sewerage works can not operate unless huge financial resources are secured for maintenance and operation, construction funds are made available, and a financial base is stabilized so that principal and interest for borrowings can be duly redeemed. Under these circumstances, Osaka City for the first time revised sewer user charges, closely linked with the construction plan.

The revision of the Sewerage Law in 1970 dictated that water quality must be taken into consideration in collecting sewer user charges. Based on this concept, a rational water quality surcharge system in which surcharges are determined in accordance with quality of wastewater discharged was adopted.

In 1972, Osaka City revised the sewer user charges again, to review the conventional sewer user charge system comprehensively. As a result of this revision, a progressive-rate system was adopted as well as the water quality surcharge system to underline the "polluter-pays" principle and to retain fairness among users.

In 1974, the progressive-rate system was further reinforced, and further revised in 1977 and 1981. The existing sewer user charges is as shown in Table 15.

7-2 WATER QUALITY SURCHARGE SYSTEM

The water quality surcharge system sets forth water quality surcharges for different water qualities of highly polluted wastewater (exceeding concentrations of 200mg/ℓ), as long as the water does not adversely affect sewerage facilities, in consideration that the maximum concentration of domestic wastewater is 200mg/ℓ for both BOD (or COD) and SS, which are supposed to be treated in sewage treatment plants. At present, users discharging not less than 1,250m^3/month of wastewater whose BOD (or COD) and SS concentrations are within the range of 200 ~ 2,600mg/ℓ are charged water quality surcharge in accordance with the rank of concentration in addition to sewer user charges. Ranks of water quality surcharges are as shown in Table 16.

Investigation to determine water quality, upon which calculation of water quality surcharge is based, is carried out for 3 days per year, as a rule. Usually, an automatic sampler is installed at the discharge outlet of the factory so that average water quality representing water quality throughout all business hours can be determined. Average water quality is measured every day for three days, and the highest value is used for the calculation of the water quality surcharge for the specific factory.

Table 15 Sewer User Charges (Monthly General Charges)

Use	Classification	Unit Water Amount	Charge
General use	Basic charge	Up to 10m^3	¥200
	Excess charges	Per every 1m^3 from 11m^3 to 20m^3	¥30
		Per every 1m^3 from 21m^3 to 30m^3	¥40
		Per every 1m^3 from 31m^3 to 50m^3	¥50
		Per every 1m^3 from 51m^3 to 100m^3	¥60
		Per every 1m^3 from 101m^3 to 200m^3	¥70
		Per every 1m^3 from 201m^3 to 500m^3	¥85
		Per every 1m^3 from 501m^3 to 1,000m^3	¥100
		Per every 1m^3 from 1,001m^3 to 5,000m^3	¥125
		5,001m^3 and over	¥140
Common use	Basic charge	Up to 8m^3	¥100
	Excess charge	Per every 1m^3 from 9m^3	¥15
Public bath	Basic charge	Up to 10m^3	¥200
	Excess charge	Per every 1m^3 from 11m^3	¥9

Table 16 Water Quality Surcharges

Rank	Water Quality Classification ppm	BOD or COD ¥/m^3	SS ¥/m^3
A	201 ∿ 300	14	15
B	301 ∿ 450	31	38
C	451 ∿ 600	52	64
D	601 ∿ 850	80	100
E	851 ∿ 1,100	114	144
F	1,101 ∿ 1,350	149	188
G	1,351 ∿ 1,600	184	232
H	1,601 ∿ 1,850	219	277
I	1,851 ∿ 2,100	253	321
J	2,101 ∿ 2,350	288	365
K	2,351 ∿ 2,600	323	410

Notes: 1. BOD or COD, whichever is greater, shall be used.
2. The surcharges are applicable for users discharging not less than 1,250m^3 of wastewater monthly.

Example of calculation of Water Quality Surcharge

Discharge amount: 2,000m^3/month

Water quality : BOD 350ppm Rank B

classification SS 250ppm Rank A

Calculation:

BOD surcharge is ¥31/m^3.

So,

2,000m^3 × ¥31 = ¥62,000/month

SS surcharge is ¥15/m^3

So,

2,000m^3 × ¥15 = ¥30,000/month

Therefore, water quality surcharge in this case will be

¥62,000 + ¥30,000 = ¥92,000/month

7-3 PRESENT STATUS OF THE WATER QUALITY SURCHARGE SYSTEM

Industries which use not less than 1,000m^3/month of potable water, industrial water, river water and/or well water are extracted as the subjects to water quality surcharge system. The number of these users was about 1,400 in 1973 when the water quality surcharge system started, but has decreased to about 800 now, reflecting the general trend of decreasing water use. Preliminary study of water quality is carried out for these users, and then final investigation according to the above method is carried out for factories, either BOD (or COD) or SS of the wastewater discharged from which is expected to exceed 200 ppm.

Yearly changes in the number of users paying water quality surcharges and the amount collected are shown in Table 17.

Table 17 Number of Factories Subject to Water Quality Surcharges and Amount Collected

Item \ Fiscal Year		1973	1974	1975	1976	1977	1978	1979	1980
Number of users subject to surcharge		231	178	145	180	195	186	169	174
BOD or COD	Aggregate number of collections	1,547	1,405	1,294	1,426	1,516	1,488	1,453	1,389
	Amount (Unit: ¥1,000)	341,596	347,164	392,122	390,805	645,621	568,947	618,704	508,583
SS	Aggregate number of collections	1,176	877	521	604	606	660	449	422
	Amount (Unit: ¥1,000)	211,773	165,151	127,137	187,269	210,592	211,078	86,756	65,197
Total amount (Unit: ¥1,000)		553,369	512,315	519,259	578,074	856,213	780,025	705,460	573,780

The annual change in water use in Osaka City are shown in Fig. 7 and Fig. 8. Classified by source, industrial water and river and well water use have decreased drastically. In the classification by volume, the curve representing water consumption of 1,001m^3/month or more indicates decline year after year. This is the result of water saving efforts made by many factories.

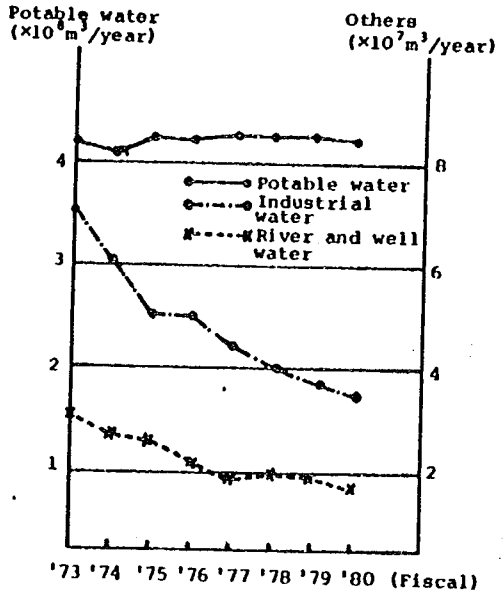


Fig. 7 Yearly Water Consumption Classified by Source

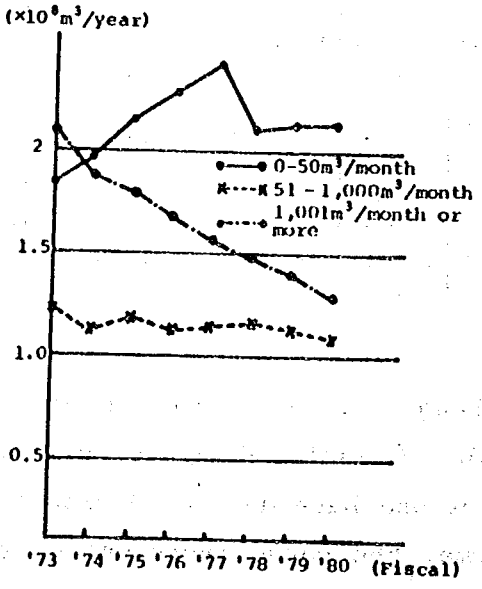


Fig. 8 Yearly Water Consumption

Now, let us look at the yearly change in the volume of effluent discharged by factories subject to the water quality surcharge system, classified by water quality items and water quality ranks. As shown in Figs. 9 and 10, the volume of effluent discharged in every water quality rank in terms of SS shows remarkable decline. With regard to BOD (or COD), the volume of effluent discharged in the relatively concentrated water quality rank (301 ppm or more decreased year after year.

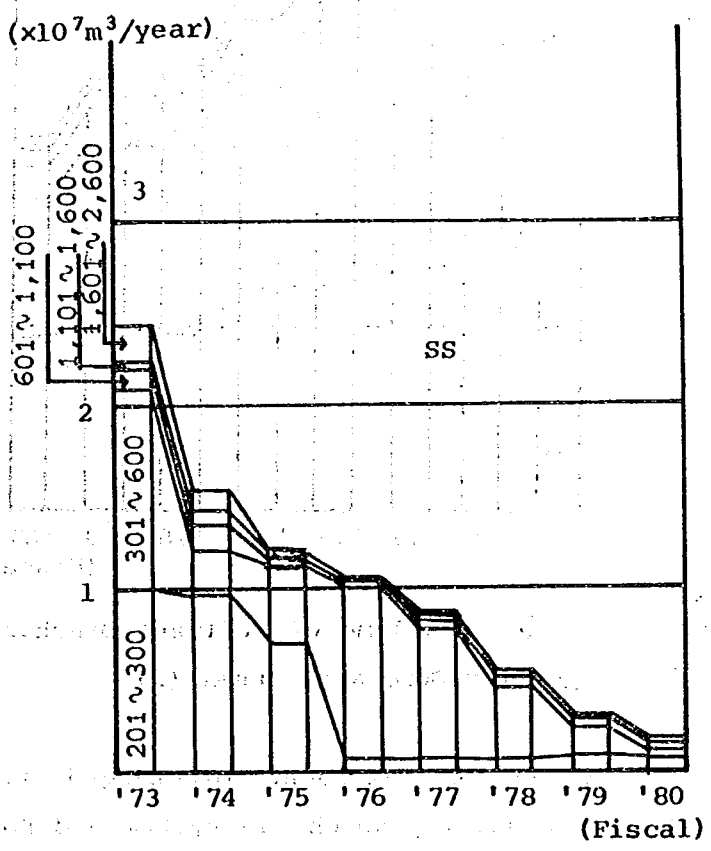


Fig. 9 Annual Change in Volume of Effluent Discharged according to Water Quality Surcharge Items (1)

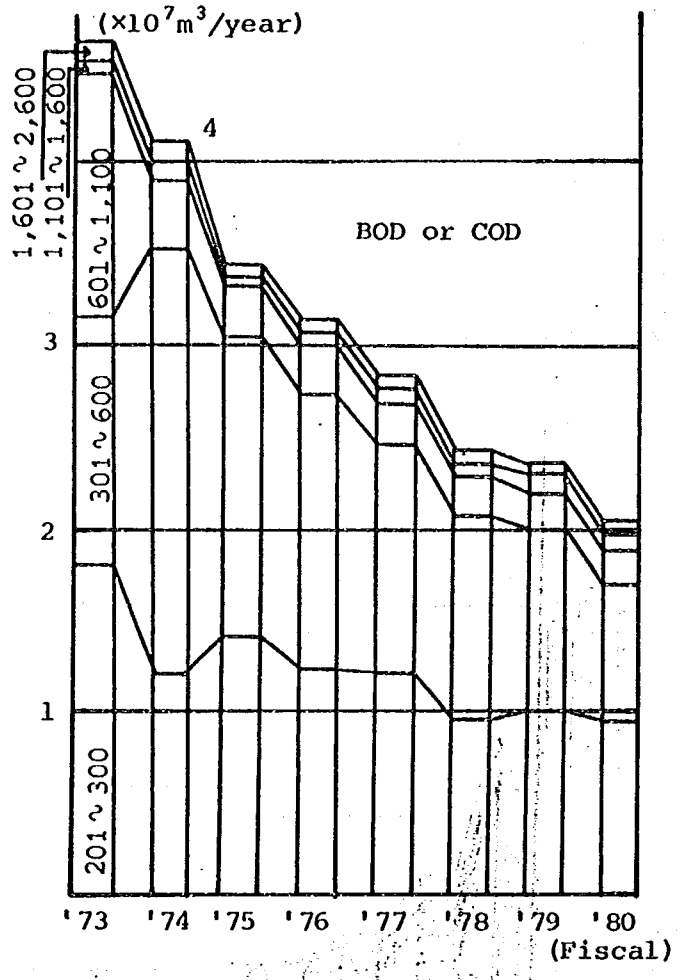


Fig. 10 Annual Change in Volume of Effluent Discharged according to Water Quality Surcharge Items (2)

The total volume of effluent discharged from factories subject to the water quality surcharge system and BOD, COD and SS loads have changed annually as shown in Fig. 11. A comparison between 1973 and 1980 reveals that the volume of effluent decreased by about 150,000m^3/day, BOD load by about 60t/day, COD load by about 44t/day and SS load by about 58t/day.

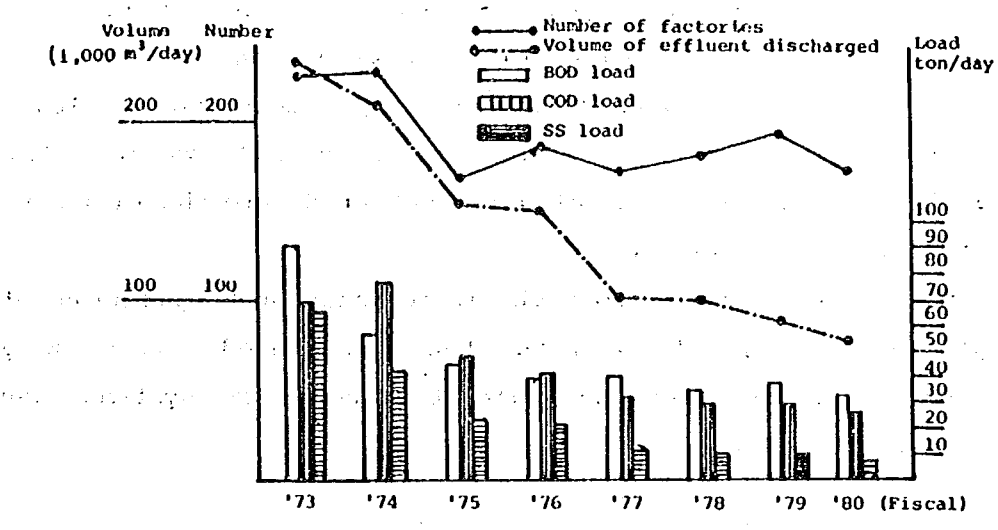


Fig. 11 Change in Volume of Effluent and Loads Discharged from Factories Subject to Water Quality Surcharge System

These phenomena are the results of positive efforts by enterprises, which had to bear considerable economic burdens imposed by the water quality surcharge system, especially when they were hit by the oil crisis in 1973 and the water shortage in 1974. These efforts included improvement of water utilization and discharge systems in factories; rationalization of water utilization, for example, by reuse (recycling) of indirect cooling water; pretreatment of separated highly-polluted wastewater; and improvement of water quality by means of improved processes.

These are testimony that the water quality surcharge system has contributed greatly to industrial wastewater control and conservation of water resources.

8. AFTERWORD

This paper has described the industrial wastewater control and the water quality surcharge system in Osaka City. Ten years have passed since the municipal ordinance was revised in 1972. Especially with regard to industrial wastewater control, laws and ordinances were revised during this period, completing the legal systems concerned and establishing adequate control systems. The water quality surcharge system also impenetrated into the industry, producing favorable results.

In the future, more thorough maintenance and operation of sewerage facilities, which have been completed after many difficulties, is required so that the sewerage systems can fulfill their purpose.

ORGANIZATION AND ECONOMICS OF A GERMAN WASTE WATER TREATMENT DISTRICT

Dr. Gunther Annen
Chief Executive
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and Lippeverband

1. INTRODUCTION

Normally, the producers of wastewater are responsible for its treatment as well. Cities and communities have their own sewage treatment plants, and major industrial factories that are not connected to municipal sewers clarify their water themselves.

But we observe cooperation among wastewater producers which can be accomplished in various ways. Industrial works can conduct their wastewater to a municipal treatment plant. On the other hand one town in Germany has its wastewater clarified in the treatment plant of a large chemical enterprise. Towns can also work together in the management of one or more common plants. This can be done in a rather loose way by contract or by founding a special organization with the towns concerned as its members.

The constructions just mentioned are of mainly local significance. Besides, there are examples of wastewater district organizations that include entire river basins. Wastewater associations of this kind don't restrict themselves to the problem of collecting the wastewater of a limited area and clarifying is as economically as possible. Beyond that, such an association is responsible for the construction and operation of all the sewage treatment plants in a praticular river basin. By these means, it is possible to take into most careful consideration the needs of the entire water region by centralized and homogeneous planning.

2. WATER ASSOCIATIONS

Wastewater associations can be based on voluntary agreement. In most cases, however, they are formed by a legal act of the state who also gives them certain rights in order to fulfil their duties lawfully. Often, the associations are responsible not only for pollution control, but also for water quantity, land drainage, river maintenance, flood protection, etc. These associations with an extensive competence can better be named water associations. Thus, the close connection between water quality and water quantity can be considered in the best way possible.

In Germany, water associations have a long history. At first, they weren't concerned at all with wastewater. In the Middle Ages already, water associations developed for the construction, maintenance and defence of dikes at the sea-coasts in northern Germany. Other associations arose in rural regions in order to improve the land drainage. One common characteristic for these water associations is the fact that everybody who owns land in the district of a water association must become a member of the association. Nobody is allowed to enjoy the advantages of the efforts of an association without taking part in the cooperative work through contributions or individual help.

In the second half of the last century, a large industrial district, based on coalfields, began to develop in western Germany. After the technical methods had been found, coal could be extracted in large quantities out of great depth. Steel-works and other industries followed and with them an immense increase in population by people from other countries or other parts of Germany who started settling there. This development also created water problems: wastewater infected local streams and rivers, and many diseases and epidemics could be traced back to the insanitary water condiditons.

The situation was especially serious in the centre of the industrial district, the Emscher river basin where 2,5 million people are living today. The individual cities are situated very closely together, thus forming almost one continuous city-area. The responsible people in the cities and industries realized that these difficulties could be overcome only in a joint effort. Thus, in 1904, the Emschergenossenschaft (Emscher River Association) was founded, the first large and multipurpose water association in Germany and probably in the world. Because its tasks and dimension by far exceeded those of earlier water associations, which were active mainly in rural districts, the state issued a special law for the foundation of the Emschergenossenschaft.

Following this model, other water associations were established: in the southerly adjacent Ruhr River Basin in 1913 and in the northerly adjacent Lippe River Basin in 1926 (Fig. 1). A few others were founded for further river catchment areas in western Germany, all of them designed in a similar way. In the following, the Lippeverband (Lippe River Association) will be described as one example.

The district of the Lippeverband includes an area of 2,780 km² (square kilometres) with a population of 1.28 million. The main industries are coal-mines, chemical factories, steel works and power stations.

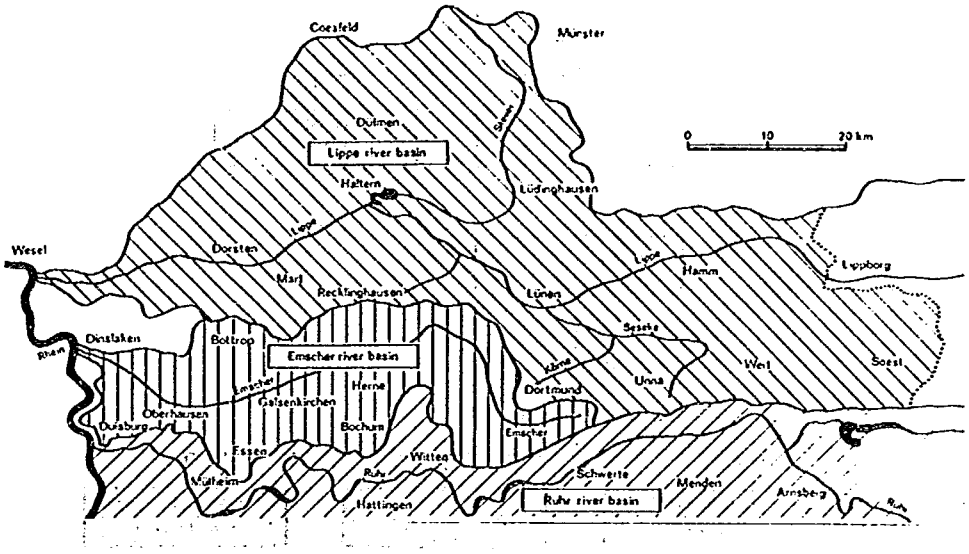


Fig. 1 Area of the river basins Emscher-Ruhr-Lippe

3. ORGANIZATION

Members of the Lippeverband are all the municipalities, communities and mines that are located in the association's district; furthermore industrial enterprises, waterworks, railways, waterways etc., but only if their membership fees reach a certain minimum amount. Smaller enterprises are included within the communities they belong to.

It has to be pointed out that membership doesn't result from voluntary enrolment but from the law for the Lippeverband (compulsory membership). The association is a self-administrating body of Public Law which means that for its plannings and actions it is responsible to itself. The governmental supervision is restricted to a control that the association works in accordance with the regulations of the law. The Lippeverband centainly has to act within the scope of governmental water quality policy.

The highest authority of the Lippeverband is the General Assembly, with those members entitled to vote whose charge reaches 1/1,000 of the total charge. The more dues a member pays, the higher is the number of his votes. The General Assembly makes all the fundamental decisions; it

determines the budget and the rules of charging and also elects the members of the Board (Fig. 2).

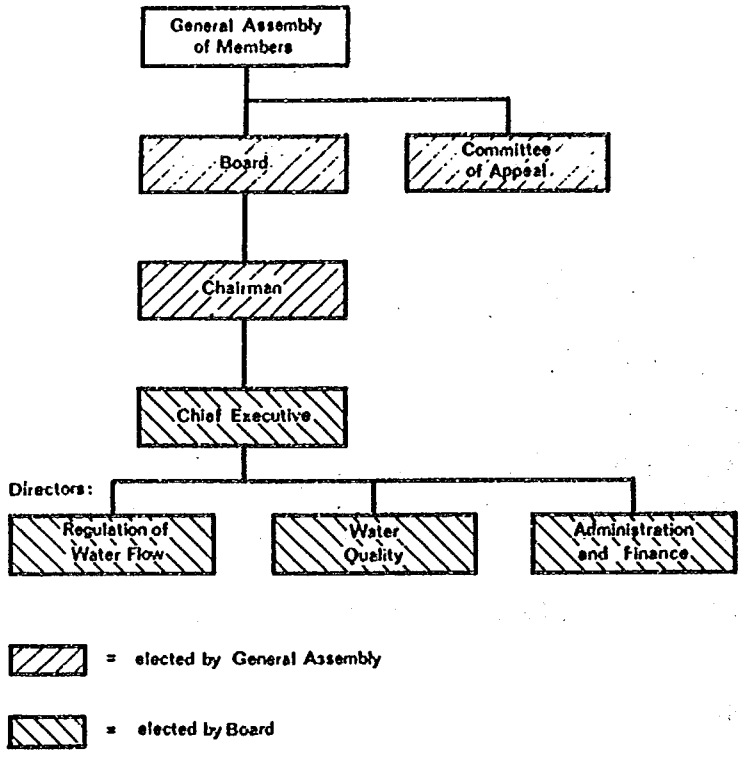


Fig. 2 Organization of the Lippeverband (Lippe River Association)

The Board consists of 13 members. It decides how the work of the association is to be carried out in particular. The Chairman of the Board is also the Chairman of the association. He is the leading personage of the association and its main representative.

The General Assembly also elects the members of a Committee of Appeal which meets whenever a member complains about the demanded charges. The Committe of Appeal investigates the case and hears the members and the Board. Only after the process, if the member is not satisfied with the decision of the commission, he has the right to appeal to a court of justice.

The staff of the Lippeverband headquarters in Essen and Dortmund, combined with that of the Emschergenossenschaft (Emscher River Association), is distributed between 3 directorates (regulation of water flow, water quality, administration and finance) with a total of 19 departments. The employees are engineers, scientists, lawyers and administrative personnel.

4. PROCEEDING PRINCIPLES

The Lippeverband plans, constructs and operates sewage treatment plants. The sewerage function is undertaken by the local authorities. At a certain point the association takes over the wastewater, clarifies it and then discharges it into the receiving watercourses.

For the treatment of storm water, a reasonable labour-division takes place. Storm water overflow tanks that are most useful within the sewerage systems are built and operated by the local authority. However, if it proves to be better to set up such tanks close to or within a sewerage treatment plant, the Lippeverband is responsible for that.

The wastewater of the industries is either received by the municipal sewerages or conducted to the sewage treatment plants by the Lippeverband. Some of the major industrial works, however, that are located along the Lippe River, clarify their wastewater in cooperation with the Lippeverband in plants of their own. This is an effective method because in these enterprises the treatment of wastewater already starts in the production and is completed by the final clarification. Therefore it is wise to keep all steps of the clarification in one hand.

When all wastewater treatment processes for an entire river basin are planned by one single association, the available money can be employed where it is of most use. After its foundation, the Lippeverband therefore built its first sewage treatment plants where the sources of the most wastewater lay. Thus, the treatment plants for bigger towns and industries became the major concern in this priority program. In that way a significant improvement of the water quality could already be achieved. Gradually the network was completed, including the smaller communities.

Today the Lippeverband operates 85 treatment plants where 98 % of the wastewater ist clarified biologically (Fig. 3). Another plant is under construction to deal with the remaining 2 %. But that doesn't mean the end of the Lippeverband's work. Some treatment plants are overburdened or old; they have to be extended and brought to a modern level. The introduction of a third stage of purification to eliminate the phosphorus is practised for only a few places, where the wastewater is discharged into a lake utilized to gain potable water.

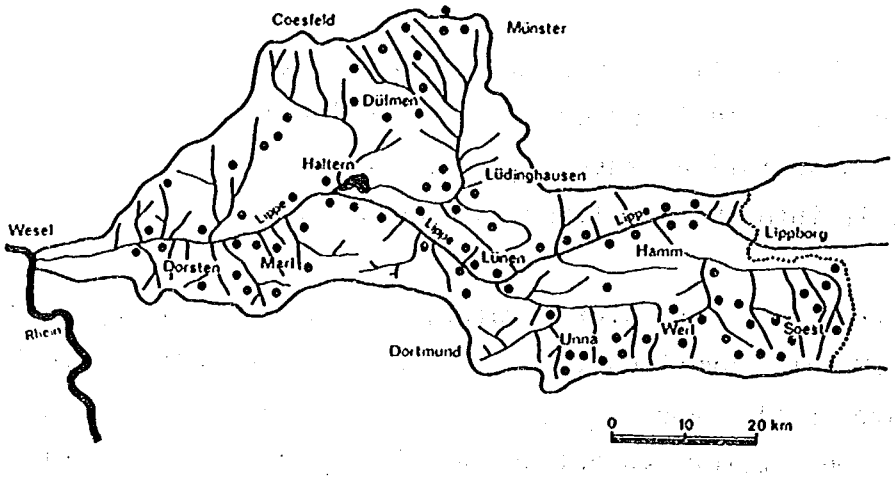


Fig. 3 Treatment plants of the Lippeverband

Besides the wastewater treatment the Lippeverband is engaged in "other water quality activities". Among these is the quality monitoring in numerous monitoring stations. They enable the prompt and effective reaction to pollution incidents in addition to the quality control field staff who deals with emergencies. At some points of the Lippe the river will be aerated artificially when in days of little water flow and warm weather the O_2-level threatens to go down beneath the limit necessary for fish life.

Another interesting solution for quality problems is the addition of water out of the system of west German shipping canals into the Lippe river. By this way nearly 50 % can be added to the low flow of the Lippe. This water will be pumped by special pumping stations, financed by the Lippeverband, out of the River Rhine from lock stage to lock stage to the shipping canals to the city of Hamm. There it overflows to the Lippe when the river doesn't reach a certain minimum flow. This is a good example for the close connection between water quality and water quantity policy.

5. ECONOMICS

Because of the centralized management in the operation and maintenance for many treatment plants the costs can be lower as if each plant had to be cared for by the local authority. Small plants are not manned but controlled from a larger plant in the neighbourhood. Special equipment i.e. the chemical and biological laboratory in the headquarter can work in a very effective manner within a water association.

The solidarity between the members of the association corresponds to the solidarity in the charging system. The costs of all treatment plants are summed up and distributed among all wastewater producers who use a treatment plant of the Lippeverband. This is an important principle derived from the already mentioned priority principle: when it is necessary to improve the effect of the pollution control system all wastewater producers contribute to the cost irrespective of the places of the improvements.

There is solidarity between the utilizers of large and small treatment plants, too. Small plants produce higher costs per capita and therefore the large plants subsidize the small ones.

The expenditure for new plants and installations is mainly financed by loans. Besides this money is available from a governmental construction fund, but only for domestic sewage, not for industrial wastewater. The capital service for interest and redemption and the costs of operation and maintenance are charged from the members. Along with the improvement of pollution control in the Lippe area the total charge for that purpose has risen considerably within the last years (Fig. 4).

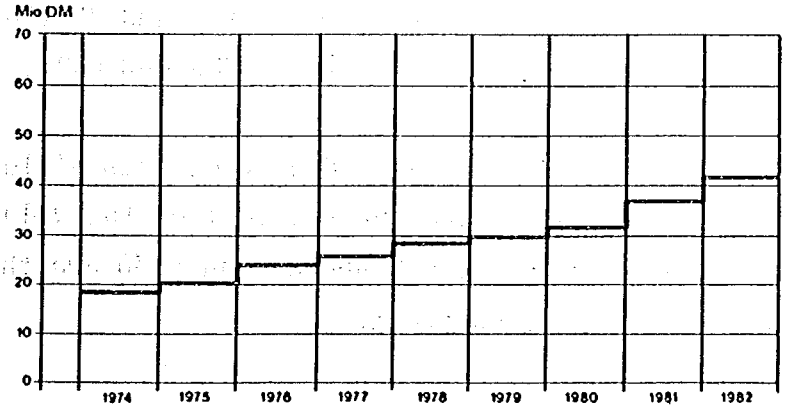


Fig. 4 Total charge for wastewater treatment and other water quality activities

The main problem of a charging system is how to devide the total charge among the members. If there was domestic sewage only, the distribution would be easy: the number of people in each member municipality would give an appropriate measure. It is more difficult however to assess the industrial wastewater. When converting the load into population equivalent it is not sufficient to calculate with the organic part (BOD or COD) only.

The question must be: which costs are caused by a certain wastewater treated in a biological purification plant. In this view the sewage flow and the settleable solids are of considerable influence, too (Fig 5).

$$B = \frac{Q}{0.0026}\left[0.40 + 0.25\ \frac{A - 0.3}{6.0 - 0.3} + 0.35\left(\frac{1}{2}\ \frac{BOD - 20}{250 - 20} + \frac{1}{2}\ \frac{COD - 80}{500 - 80}\right)\right]$$

B = population equivalent
Q = waste water flow (l/s)
A = settleable solids (mg/l)
BOD = BOD_5 in settled sample (mg/l)
COD = COD in settled sample (mg/l)

Fig. 5 Charging formula

The formular expresses that 40 % of the plant costs are caused by the wastewater quantity, 25 % by the settleable solids and 35 % by the organic matter. The latter part falls half-and-half to BOD and COD.

The treatment plants of the Lippeverband produce mean effluent values of 0,3 and 20 and 80. These values are subtracted from the values of the raw wastewater. The numbers 0,0026 and 6,0 and 250 and 500 are the mean values of domestic sewage or of one inhabitant.

The relevant values are not those of a year or a month but the mean values of 6 hours with the largest load by quantity and concentration within the production period. This is important, because a treatment plant must meet not only mean loads but has to remain within the legal effluent standards in times of peak load, too. This regulation gives incentives, especially for industrial wastewater producers, to equalize the wastewater flow by internal means. In most cases this is much more economical than the enlargement of a whole treatment plant.

The charging formula is not applied to the costs of the "other water quality activities". These activities (river aeration, low flow enrichment etc.) are necessary to avoid problems in the receiving watercourses caused even by treated wastewater. Therefore not only the users of the association's plants but also the industrial factories, that treat their wastewater themselves, have to participate in the costs of these activities. One can imagine that the effluent values are nearly the same at all treatment plants. So only the water quantity is the measure for charging the other water quantity activities.

6. SUMMARY

The water associations have developed to very important instruments of water pollution control in the industrialized part of western Germany with a high density of population. The members form a close community and feel a joint responsibility for an active water quality policy in "their" river basin. The principle of self-administration distributes rights and duties in a well balanced way. All groups of members are appropriately represented in the association. Up to now no contrasts of interests had to be observed, especially not between the towns and the communities on one hand and the industry on the other hand. There is also practised a good cooperation with the governmental administration on the base of mutual confidence.

SESSION 2

REUSE OF WASTEWATER

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PRESENT STATUS OF WASTEWATER REUSE IN JAPAN

Tsutomu Tamaki
Director General,
Department of Sewerage and Sewage Purification,
Ministry of Construction

1. INTRODUCTION

Wastewater recycling in Japan dates back to the fall of 1951 when the Tokyo Metropolitan Mikawashima Wastewater Treatment Plant conducted an experiment using effluent at the request of a nearby paper mill owner.

This occurred because the water available to the paper mill from the Arakawa River was contaminated with industrial effluents. At the same time, the available water became too saline due to the influence of tidal water as a result of ground subsidence caused by over-extraction of ground water in the Koto Delta Area.

The use of wastewater for actual production was tested for about 1 year, and it was found that the process was not affected by recycled wastewater and, if purified through a small sand filter, the wastewater improved paper quality.

There are many works in the Koto Delta Area, including paper mills, tanneries, furriers, dye plants, oil and fat plants, and fertilizer plants.

They all had industrial water shortage problems, and were interested in the results of the wastewater recycling tests at the paper mill.

Having heard about the success of wastewater recycling, some 70 plant owners filed a joint petition with the Tokyo Metropolitan Government in November 1952 for wastewater recycling services.

In response to this request, the Tokyo Metropolitan Government started construction of a rapid sand filter near the final settling tank of the activated sludge process in the Mikawashima Wastewater Treatment Plant in December 1953 and completed it in about 8 months. This was followed by the construction of recycled water receiving facilities at the applicant plants and subscription contracts for recycled water distribution. On February 10, 1955, the tertiary effluent was fed to the paper mill.

Later, the Minami-Senju Industrial Water Purification Plant was constructed in the Koto Delta, and it was decided to supply raw water to the Minami-Senju Industrial Water Purification Plant from the Mikawashima Wastewater Treatment Plant.

At present, there are about 480 wastewater treatment plants in Japan, which discharge about 6,500 million m^3 of effluents a year. At the majority of these plants, the effluents have been recycled in the form of foam-dispersing water, cooling water and cleansing water in treatment process, inplant cleaning water, toilet flushing water, defrosting water, etc.

Of the 480 wastewater treatment plants, 34 deliver about 50 million m^3 of effluents a year for industrial or waterscape purposes.

Table 1 Status of the use of wastewater treatment plant effluents outside the plants

Location	Wastewater treatment plant	Recycling rate (m^3/day)	Supplier	User	Application
Hakodate	Nambu	2,000	Hakodate Municipal Government	Sewage treatment plant	Dilution
Hitachi	Ikenokawa	50	Hitachi Municipal Government	Ikenokawa Sports Park	Flush toilets and sprinklers
Ashikaga	Ashikaga Municipal No. 1	8,000	Ashikaga Municipal Government	Shichikason Irrigation Water Cooperative	Agricultural irrigation
Sano	Sano Water Purification Center	6,000	Sano Municipal Government	Akagi Soil Improvement Area	Agricultural irrigation
Mobara	Kawanakajima	1,028	Mobara Municipal Government	Sewage treatment plant	Dilution
Funabashi	Nishiura	4,940	Funabashi Municipal Government	Sewage treatment plant	Dilution, process water
Ena	Ena Municipal Purification Center	1,000	Ena Municipal Government	Sewage treatment plant	Dilution
Tajimi	Ikeda	340	Tajimi Municipal Government	Sewage treatment plant	Dilution
Gamagori	Gamagori Municipal Wastewater Treatment Center	40	Gamagori Municipal Government	Minato Kyoritsu Rokuun K.K.	Sprinkler
Himeji	Chubu	450	Himeji Municipal Government	Himeji Port Aggregates Cooperative	Washing sea sand
Takasago	Takasago	300	Takasago Municipal Government	Chemical fertilizer plant	Washing and exhaust gas scrubbing
Ashiya	Ashiya	150	Ashiya Municipal Government	Incineration plant	Exhaust gas cooling
		300	Same as above	Park	Sprinkler (summer only)
Kurashiki	Mizushima	1,500	Kurashiki Municipal Government	Sewage treatment plant	Dilution
Ube	Tobu	82	Ube Municipal Government	Incineration plant	Cooling water
Imabari	Togaya	38	Imabari Municipal Government	Yunoura Beach Heights Garden	Waterscape and sprinkler
Matsuyama	Chuo	1,440	Matsuyama Municipal Government	Soil Improvement Area and Irrigation Cooperative	Irrigation of paddies
Iizuka	Iizuka	400	Iizuka Municipal Government	Sewage treatment plant	Defoaming and cooling
		130	Same as above	Sanitation Center	Car washing
Oita	Benten	3,000	Oita Municipal Government	Gravel Cooperative	Washing sea sand
Kumamoto	Rendaiji	23,000	Kumamoto Municipal Government	Irrigation Cooperative	Paddy irrigation
Okinawa Chubu Regional Sewerage Area	Isahama	1,500	Okinawa Prefectural Government	Sewage treatment plant	Dilution

Table 1 Status of the use of wastewater treatment plant effluents outside the plants (cont'd)

Location	Wastewater treatment plant	Recycling rate (m^3/day)	Supplier	User	Application
Tokyo	Shibaura	230	Tokyo Metropolitan Government	Tokyo Central Wholesale Market, Shibaura Plant of Meat Market	Cooling water
		500	Same as above	JNR's New Tokaido Line Tokyo Train Dispatching Station	Washing
	Mikawashima	60,400	Same as above	Minami-Senju Water Purification Plant, Tokyo Metropolitan Government Waterworks Bureau	Raw water for industrial use
	Sunamachi	1,500	Same as above	Sunamachi Sewage Treatment Plant, Tokyo Metropolitan Government Sanitation Department	Dilution, and so forth
	Morigasaki	670	Same as above	Ohi Incineration Plant, Tokyo Metropolitan Government Sanitation Department	Washing
Kawasaki	Iriezaki	952	Kawasaki Municipal Government	Sewage treatment plant, Kawasaki Municipal Government Sanitation Department	Dilution
Nagoya	Meijo	24	Nagoya Municipal Goverment	Scavengers	Cleaning of sewers
	Horidome	15	Same as above	Same as above	Same as above
	Sennen	12,000	Same as above	Nagoya Municipal Government Waterworks Bureau	Raw water for industrial use
Kyoto	Toba	1,465	Kyoto Municipal Government	Agricultural Cooperatives	Irrigation
Osaka	Nakahama	2,000	Osaka Municipal Government	Osaka Municipal Government Park Bureau	Outer moats of Osaka Castle Park
Kitakyushu	Hiakari	5,000	Kitakyushu Municipal Government	Incineration Plant, Kitakyushu Municipal Government Sanitation Department	Washing, gas scrubbing, dilution of sewage
	Kogozaki	3,651	Same as above	Mitsubishi Chemical Industries	Cooling water
		23	Same as above	Taiyo Sanso	Cooling water
		1,440	Same as above	Kitakyushu Municipal Government Sanitation Department	Washing, gas scrubbing
Fukuoka	Chubu	200	Fukuoka Municipal Government	Chuo Public Center, Fukuoka City Hall, and so forth	Flush toilets, and so forth
	Tobu	600	Same as above	Tobu Incineration Plant, Fukuoka Municipal Government Sanitation Department	Miscellaneous uses
		1,500	Same as above	Marine Products Processing Public Corporation	Same as above

2. NEED FOR REUSE OF WASTEWATER

According to the Third All-Japan Comprehensive Development Plan (3rd AJCDP), it is expected that the population will continue to increase in Japan, particularly in major cities, and that the area of densely inhabited district (DID) will increase.

Table 2 Population increase in DID

Item	Population (in thousands)							Mean annual growth rate (%)					
	1960	1965	1970	1975	1986	1990	2000	1965/1960	1970/1965	1975/1970	1985/1975	1990/1985	2000/1990
Population in all Japan	93,419	98,275	104,665	111,940	123,749	128,272	136,899	1.02	1.27	1.35	1.01	0.72	0.65
Population in DID	40,830	47,261	55,997	63,823	79,942	86,327	97,883	2.97	3.45	2.65	2.28	1.55	1.26
Population in all Japan/ Population in DID (%)	43.7	48.1	53.5	57.0	64.6	67.3	71.5						

Notes: 1. The data for 1960, 1965, 1970, and 1975 are based on national census statistics.
2. The data for 1985, 1990, and 2000 are estimated by the Planning and Adjustment Bureau of the National Land Agency.
3. Beginning with 1970, the data includes Okinawa.

Regarding water demand, it is forecasted that the growth of unit of water consumption will gradually level off as people become more conscious of water saving, but total demand will continue to rise steadily.

In November 1978, the Ministry of Construction issued a report titled "Water Resources Development Plan and Water Use for the Year 1990" in which it estimated, based on the population forecast in the 3rd AJNDP, that in 1990 the water supply-demand situation would be severe in 10 areas, including Southern Kanto, Kinki, and Northern Kyushu, and that the country as a whole would suffer a water shortage of 1,500 million m^3 a year.

While the water demand growth in recent years has been held below the forecast, the water demand is expected to grow steadily on a long-term basis. Domestic water consumption will rise in keeping with population increase and improvement in living standards, industrial water consumption will increase with economic developments, and the demand for irrigation water will rise as paddies and dry fields are improved for modernization in agriculture.

To stabilize socioeconomic development, it is of paramount importance to conserve both the quality and quantity of our water resources as a precious national asset, and to establish and maintain a balance between supply and demand, both at present and in the future. This requires integrated measures to stabilize the water supply by development and conservation of water resources.

On the other hand, the volume of wastewater treated as of FY 1981 was about 6,500 million m^3 and is expected to increase with the expansion of sewerage-served population. The voluminous effluents from the wastewater treatment plants are basically an important element of the natural water recycling system and are to be released into public waterways, such as rivers, for recycling.

Depending on the regional water circumstances, however, effluents from the wastewater treatment plants should be recycled directly, rather than relying on the natural recycling system.

Most cities suffering from water shortages are located in littoral zones.

The treated wastewater in these cities is adequate for most of the reuse purposes, and can be recycled directly.

In this respect, the wastewater treatment plants may be regarded as "dams in the cities."

The recycling of treated wastewater can be roughly classified into two systems: closed-loop and open-loop recycling systems. (See Table 3.) In the closed-loop recycling system, the treated wastewater is recycled directly, without passing through public waterways.

In the open-loop recycling system, the treated wastewater is discharged into the public waterways, then pumped back for reuse.

The closed-loop recycling system is subclassified into two types: one in which the user recycles water used for the same purpose within his own facility through a closed loop of the recycling system; and the other in which the user supplies treated wastewater for others use.

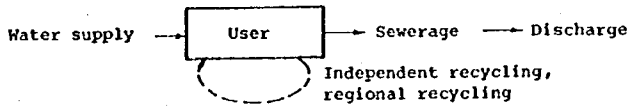
The open-loop recycling system is also subclassified into two types: the river-flow type, in which treated wastewater is turned back into rivers, and so forth, then pumped back for reuse whenever necessity arises; and the hydrological regime control type in which treated wastewater is turned back to the upper reaches of a river in the drought time only. The river-flow type has been widely practiced.

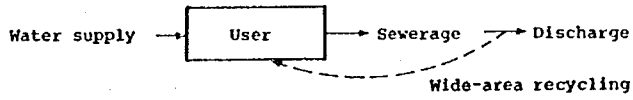
In drought-prone regions, it has become increasingly important to promote water saving and reuse, that is, to recycle water directly.

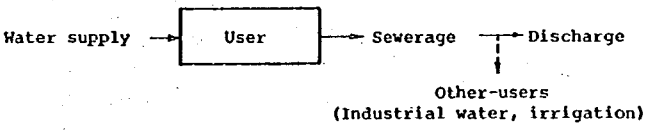
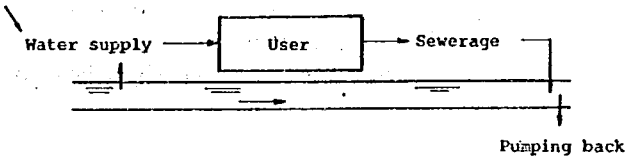
The use of treated wastewater for purposes other than drinking as a direct recycling method, however, is predicated on an in-depth study of and solution to the following problems: development of scale and slime, leading to corrosion of recycled water-distribution facilities; problems arising from misuse; level of water quality required for reuse; consensus of opinion about the use of treated wastewater; problems concerning public health and hygiene; and so forth.

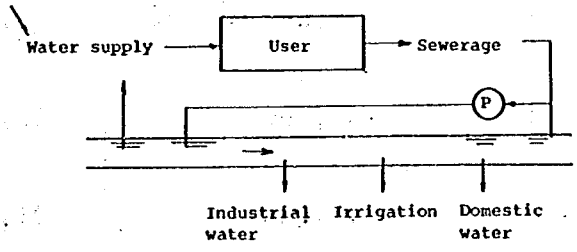
For the purpose of studying these problems, the Ministry of Construction commissioned the Japan Housing Corporation (renamed the Japan Housing and Urban Development Public Corporation) to conduct a 5-year survey from FY 1977. The Japan Housing and Urban Development Public Corporation formulated technical guidelines (draft) for the recycling of treated wastewater.

Table 3 Modes of treated wastewater recycling

<table>
<tr>
<td>Closed-loop recycling system</td>
<td>(1) Exclusive recycling (miscellaneous water recycling system)
a.
 

b.
 

(2) Split recycling
 
</td>
<td>Treated wastewater is directly recycled without passing through public waterways, such as rivers. This closed-loop recycling is divided into two systems: exclusive recycling and split recycling systems. In some cases, these two systems are used in combination.</td>
</tr>
<tr>
<td>Open-loop recycling system</td>
<td>(1) River-flow recycling (traditional recycling system)
 

(2) Hydrological regime control recycling (applied only in the drought-time)
 
</td>
<td>Treated wastewater is discharged into rivers and other public waterways and is pumped back for reuse at the lower reaches. This method is classified into the river-flow and hydrological regime control systems.</td>
</tr>
</table>

3. SURVEY OF RECYCLING TECHNOLOGY IN HEIJO NEW TOWN

Heijo New Town, which lies in the outskirts of Nara Prefecture, plans to have a population of 73,000. The Heijo Development Authority conducted a 4-year feasibility study of wastewater recycling from July 1978 to 1981, making use of treated wastewater available from the Heijo Purification Center in Heijo New Town. The Heijo Purification Center has an advanced treatment facilities. (see Table 4.)

Table 4 Particulars of Heijo Purification Center

Name:	Heijo Purification Center
Address:	259, Utahime-cho, Nara Prefecture
Area:	Approximately 12,000 m^2
Planned population coverage:	23,400 (by two existing series) 35,000 (by three series in the future)
Influent quality:	BOD, 200 mg/ℓ; SS, 200 mg/ℓ
Effluent quality:	BOD, less than 10 mg/ℓ; SS, less than 8 mg/ℓ
Wastewater treatment system:	Conventional activated sludge process (balancing tank installed)
Advanced treatment system:	Rapid sand filter and activated carbon adsorption system (anthracite, silica sand, activated carbon)
Sludge disposal system:	Thickener and vacuum dewatering process

Planned wastewater flow

Item	Unit flows, liter/person/day	Current project (23,400 persons)	Future project (35,000 persons)
Daily average sewage inflow	250	5,850 m^3/day	8,750 m^3/day
Daily maximum sewage inflow	350	8,190	12,250
Hourly maximum sewage inflow	900	21,060	31,500

Intended to provide a basis for formulating a draft of comprehensive standards to administer the wastewater recycling activities, the survey was conducted according to carefully designed experiments.

Socially accepted uses for recycled water:

(1) flush toilets; (2) sprinkler systems; and (3) waterscape.

Six types of raw water for recycling was selected after considering experimental works in the past and reality in use:

(1) secondary effluent, (2) filtrate of secondary effluent through a rapid sand filter, (3) tertiary effluent from activated carbon), and the product of each of these three processed by ozonizing. Chlorination was carried out for each type.

Figure 1 shows a flow diagram of the recycling system used for the survey. In the Center, the primary settling tank is equipped with a balancing tank to absorb sudden changes in influent loads peculiar to the housing complex; this is a satisfactory method of effluent stabilization.

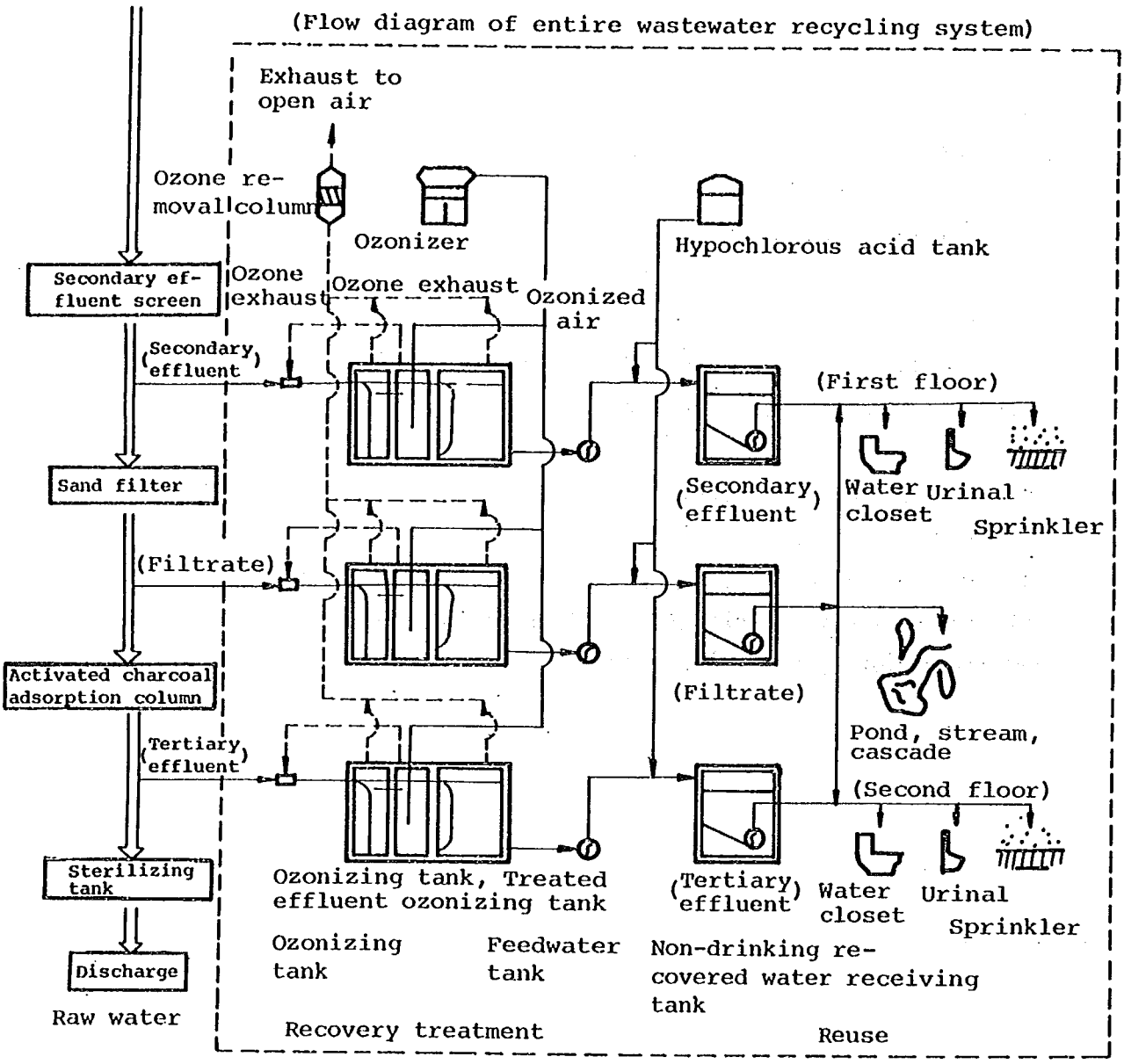


Fig. 1 Flow diagram of recycling system

(1) Conditions of treatment

Figure 2 shows that the BOD values of secondary effluents agreed with the exception of dispersions in spring and a slight decline in water quality in winter. In the tertiary treatment, the BOD was markedly reduced by the removal of SS through a sand filter, and the COD and colors were noticeably removed by activated carbon adsorption.

The chlorination process (injection rate: 1-5 mg/ℓ) and the ozonizing process (injection rate: 10-20 mg/ℓ) specifically installed for the survey produced a significant effect on the removal of colors and coliform group.

On average, the ozonization improved the degree of color to 11 for reclaiming secondary effluent and to 5.5 for reclaiming tertiary effluent because of the synergestic effect of activated carbon adsorption.

Coliform group and general microbes were almost totally sterilized in both the secondary and tertiary effluents by the combination of ozonization and chlorination. (See Figure 3.) Comparing with wastewater treatment plants of a similar size, the test results were too good. It was therefore considered necessary to eliminate the false-bottom effects, since the actual sewage influent flow was nearly half the planned daily average value and the balancing tank stabilized the treatment processes. In other words, it was feared that the test results might not be applicable in a general way.

For this reason, additional surveys have now been carried out using the supernatant (BOD: approximately 20 mg/ℓ) taken from the inlet of the final settling tank for test water from secondary treatment.

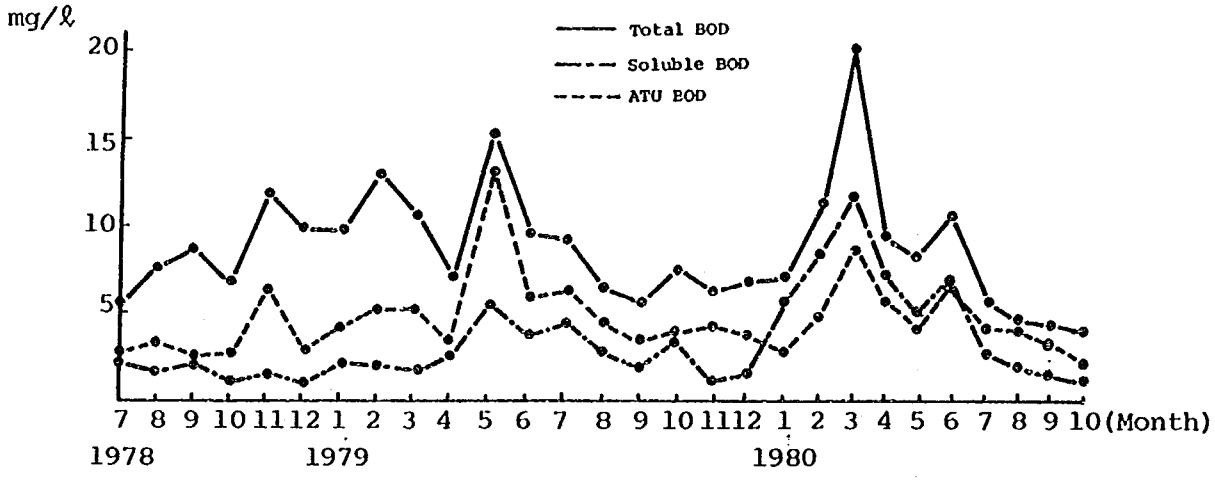
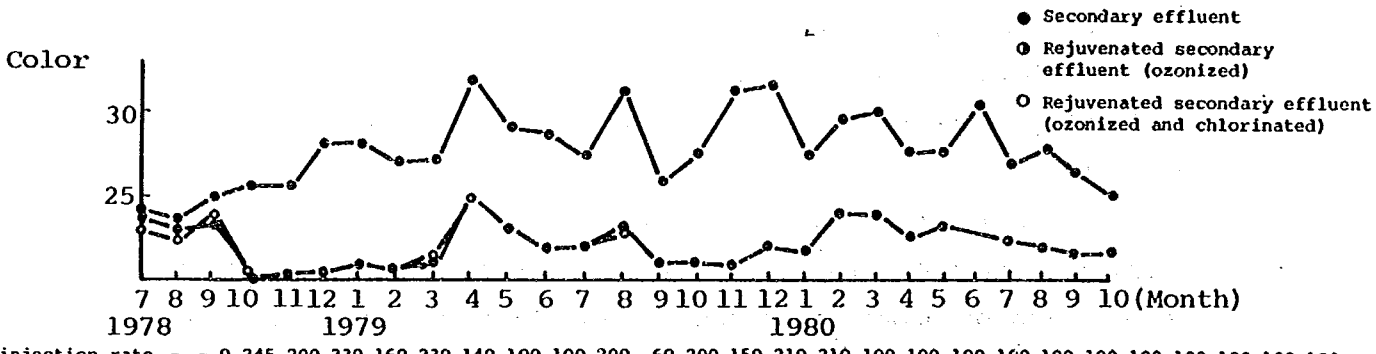
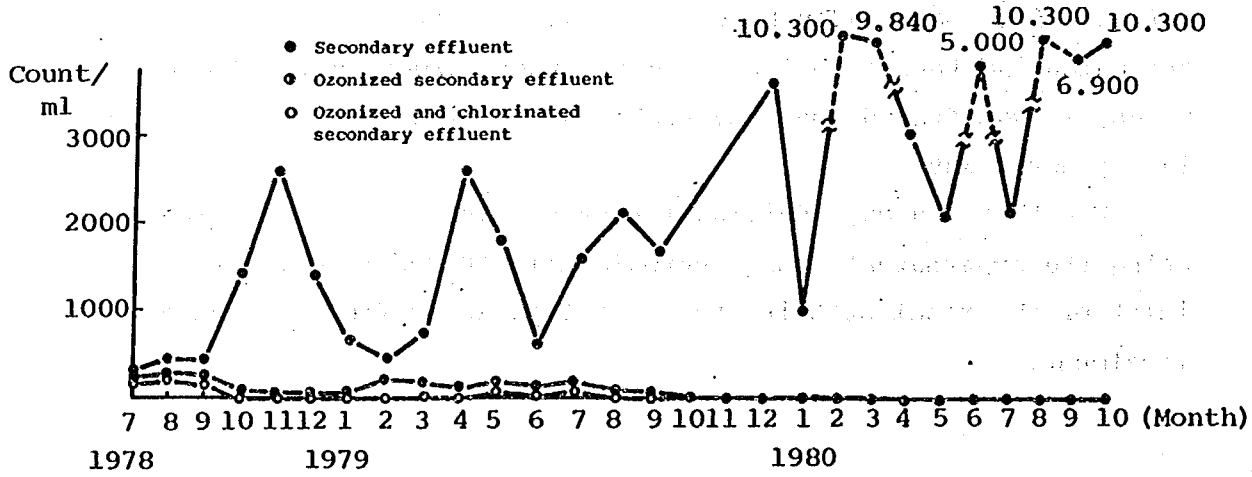


Fig. 2 Behavior of total BOD, soluble BOD, and ATU BOD



O_3 injection rate	-	-	0	245	200	230	160	230	140	100	100	200	60	200	150	210	210	100	100	100	100	100	100	100	100	100	100	100	100	100
Cl injection rate	20	-	0	-	0.5	4.0	2.9	2.3	3.6	8.3	0.0	2.6	2.0	4.2	2.9	5.1	4.9	5.2	2.0	3.7	3.6	3.1	2.6	2.6	3.7	3.6	3.9	4.7		

Fig. 3 Ozonization and chlorination of secondary effluent (color)



Ozonization and chlorination were conducted until September 1979.

Fig. 4 Ozonization and chlorination of secondary effluent (general microbes)

(2) System trouble

All the facilities, equipments and piping were checked for trouble.

a. Inspection of piping for corrosion and blocking

Stainless stell pipe (SUS), rigid polivinyl-chloride pipe (VP), rigid polivinyl-chloride lined steel pipe (VLP) and cast iron pipe (CIP) were checked for corrosion and clogging after a lapse of 700 days. It was found that only CIP had been affected by scale growth. Slime was formed in the secondary effluent, but not to the detriment of water flow. It was judged that the inner wall surface of CIP should be processed before use.

b. Contamination and failures of equipment

Contamination and failures sometimes developed in the ozonizing system under the influence of the contaminants in the reclaiming secondary effluent.

The delivery facilities and terminal user devices for reclaimed effluent, although clogged to some extent, were generally in good shape functionally.

(3) Storage tests

The reclaimed water was stored in 2 m^3 steel and reinforced concrete tanks to study the changes in water qualities and other conditions, and to identify problems involved in recycling. Residual chlorine in the sand filter filtrate was disappeared after 5 days of storage, and that in the reclaimed tertiary effluent disappeared in about nine days. But residual chlorine lasted longer in winter. It was found that coliform group and general microbes began to rally gradually just after residual chlorine was reduced to zero. It was found essential to totally seal the storage tank to prevent the growth of undesirable algae.

In these aspects, there was no significant difference among tanks of different materials.

(4) Influences on utilizing for waterscape

Foaming in ponds and waterfalls showed no significant difference due to the concentration of anionic surface-active agents and types of raw water.

In any case, foam-dispersing measures will be necessary for the production of waterfall effect.

As regards the transparency, turbidity, algae, and degree of color, the tertiary treatment facilities were acceptable.

(5) Running costs

The running cost for the ozonizing system was about ¥15 per cubic meter, accounting for nearly half of the total running cost of the recycling system. Since its effects are tremendous, the reduction of running cost is urgently required. The activated carbon process was also costly; about ¥20 per cubic meter.

As the scale merit for construction cost is remarkable, the recycling plants are of an economically feasible size. But the scale merit on maintenance and operation costs is not so significant. Judging from total cost basis, the recycling system should have a capacity of at least 1000 m^3 per day.

(6) Virus test

The virus test is historically new and difficult, and its methods have yet to be established. In the survey, tests were conducted once every month over a period of 1 year to check for viruses, and the results were supplemented with data available from reference works. In the survey, it was found that the viruses in secondary and tertiary effluents were almost totally inactivated by the injection of ozone at a rate of 8-20 mg/ℓ and with the level of residual chlorine held at 0.25-1.5 mg/ℓ.

(7) Ozonizing test

The ozonizing tests were conducted to evaluate the virtue of ozone and find the optimal ozone injection rate.

a. Sterilization effect

To sterilize coliform group in the secondary effluent, the necessary ozone dose rate was as high as 22 mg/ℓ partly because the secondary effluent contained SS. The filtrate through the sand filter called for 11 mg/ℓ, and the tertiary effluent 3 mg/ℓ for total inactivation. Considering the running cost of the ozonizing system, the secondary effluent should be ozonized after filtration through a sand filter. Chlorination and ozonization should be used in combination because of their high synergistic effect.

b. Decolorization

Ozone showed a high decolorization effect; the color was improved more than 10 degrees per milligram of ozone per liter. But the decolorization efficiency fell when the degree of color was below the 10 degrees required for waterscape.

c. Turbidity

Nearly 70% of turbidity could be removed from effluents containing SS by ozonization.

(8) Sensory test

Forty-eight males and females of the same age groups as the residents of New Town were sampled and polled for what degree of water quality is sensuously permissible. In the sensory test, the control-subject comparison method was used according to a nonrepetitive method in consideration of the unfamiliality of the testees with this kind of test.

Those who replied "odorless" accounted for 81% with respect to activated carbon-processed water, 77% for tap water, 52% for reclaimed tertiary effluent, and less than 40% for other water. The average requirements for color and odor, as found by the control test, are summarized in Figures 5 and 6.

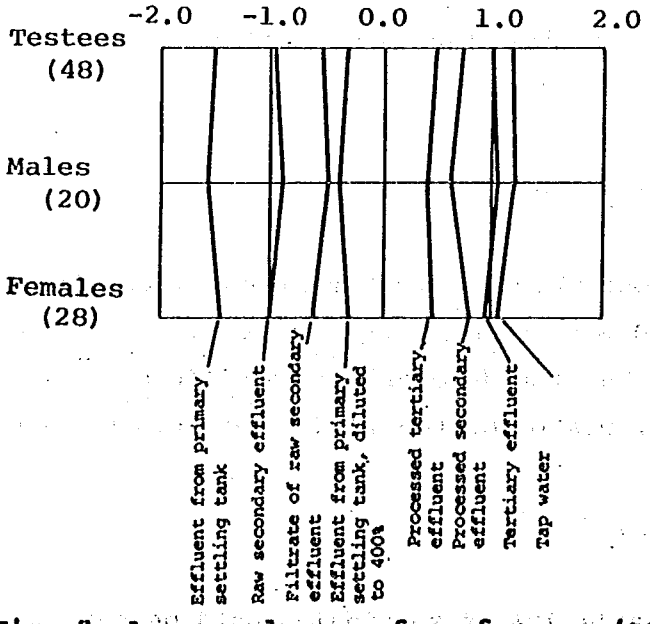


Fig. 5 Average degree of preference (color)

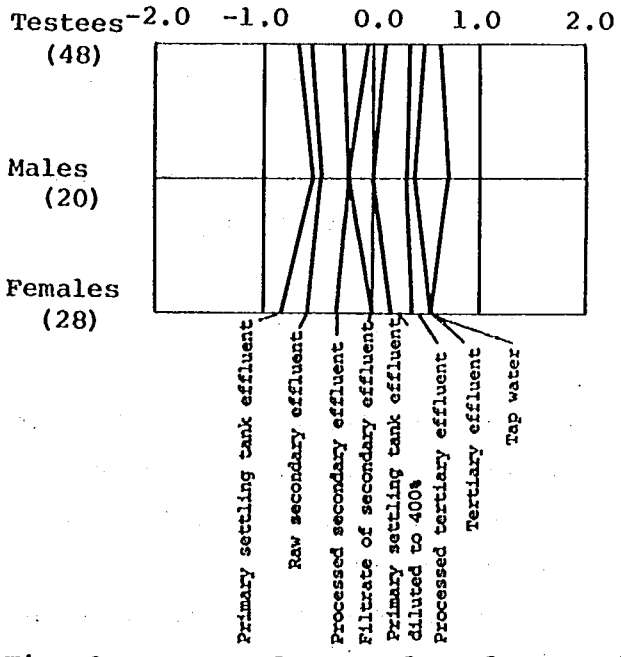


Fig. 6 Average degree of preference (odor)

As noted, likes and dislikes are governed by color rather than by turbidity and transparency. The degree of preference is likely to be positive if color is held below 20 degrees.

Concerning odor, a rotten smell (sewage smell) was never tolerated, and the secondary effluent was rarely accepted. It is hard to say that the reclaimed secondary effluent is not usable, because it is better in degree of color than the sand filter filtrate.

The secondary effluent was described as "not fit for any purpose" by 38% of the testees, the filtrate by 13%, and the reclaimed secondary effluent by 15%.

On the other hand, more than 80% of the testees allowed the filtrate and reclaimed secondary effluent to be used for flush toilets and sprinklers, suggesting that receptivity has little correlation with BOD concentration etc.

In waterscapes, it was found that recycled water was acceptable only after ozonization and chlorination.

4. TECHNICAL GUIDELINES (DRAFT) FOR WASTEWATER RECYCLING

The technical guidelines herein apply to the wide-area exclusive closed-loop recycling system. (See Table 3.)

(1) Guidelines for recycling of wastewater

The uses of recycled wastewater should be determined with due account given to social acceptability and safety. In principle, the uses of recycled waste-water are limited to flush toilets, sprinklers, and waterscapes.

Questionnaire surveys concerning social acceptance were conducted at four housing complexes, including Nara New Town. Those respondents who were receptive to the promotion of wastewater recycling accounted for 38-60%.

Including those respondents who were not absolutely opposed, the percentage in favor of recycling was as high as 89-97%.

The uses for which wastewater recycling was tolerated were limited to sprinklers, toilet flushing, waterscapes, car washing, and the like. The degree to which the recycled wastewater may come in contact with human bodies is qualitatively evaluated in Table 5.

Table 5 Classification of uses by susceptibility to contact with recycled wastewater

Susceptibility	Uses
Comparatively low	Toilet flushing, cooling, sprinklers, waterscapes
High	Car washing, cleaning, fire extinguishing defrosting
Unavoidable (cutaneous)	Laundry, bathing, handwashing
Unavoidable (oral)	Drinking

For safety, recycled wastewater should be limited to uses in which people will least be affected by contact.

(2) Water quality requirements by uses

The water quality of raw water to be recycled should be better than or equal to that of secondary effluent. Recycled wastewater should meet the water quality requirements specified in Table 6.

Table 6 Water quality standards by uses

	Characteristics	Flush toilets	Sprinklers	Waterscapes*1
Standard quality	Number of Coliform group (number/mℓ)	Less than 10	Not detectable	Not detectable
	Residual chlorine (combined) (mg/ℓ)	Retained*2	More than 0.4	-
Target quality	Appearance	Not offensive	Not offensive	Not offensive
	Turbidity (degree)	-	-	Less than 10
	BOD (mg/ℓ)	-	-	Less than 10
	Odor	Not offensive	Not offensive	Not offensive
	PH	5.8-8.6	5.8-8.6	5.8-8.6

*1. Waterscape refers to water used for man-made aesthetic creations such as ponds, fountains, rivulets, cascades, installed within housing area, etc.

*2. Chlorination is carried out for sanitation purposes. Recycled water around the receiving tank nearest the service end should contain residual chlorine.

The establishment of water quality standards is one of the most important items for the promotion of wastewater recycling. This is because water quality governs the scale and economic feasibility of the water reclamation facilities.

Wastewater recycling systems must supply reclaimed water of stabilized qualities; supply reclaimed water which is not hazardous to human health; and be free from corrosion, clogging, and other functional troubles.

In addition, reclaimed water must not be offensive to users, but maintain the aesthetic virtues of water. In particular, hygienic problems must be given more careful consideration than functional or sensory problems.

In the water quality standards set forth here, characteristics concerned with human health are classified as standard quality, and those concerning the functions of the recycling system are specified as target quality.

(3) Operation and maintenance

Operation and maintenance of the wastewater recycling system should be attentive to the following:

a. To assure water quality standards specified for respective uses, and to ensure a stabilized supply of reclaimed water.

b. To conduct periodic maintenance and inspection of each facility to prevent problems.

(4) Water quality control

For water quality control, the following should be strictly observed:

a. Water quality inspection should be conducted according to a schedule in which the inspection characteristics, inspection cycles, and inspection subjects are specified in compliance with raw water conditions, uses, and water quality standards established for the various uses.

b. To conduct water quality inspection according to the Ministrial Ordinance concerning the Inspection of Sewage Quality (Ordinance No. 1 promulgated by the Ministry of Health and Welfare and the Ministry of Construction on 17 December 1962) and other relevant rules, regulations, and standards.

Whether the populace accept the waste water recycling depends on the possibility of supplying the reclaimed wastewater of good and stable qualities. For this purpose, it is especially important to constantly check and control the qualities of reclaimed wastewater. Usually, water quality inspection is divided into regular and occasional inspection.

In the regular inspection, water quality is inspected at regular intervals. If degradation in the quality of raw water or functional deterioration of the reclaiming facilities is likely to spoil the quality of reclaimed wastewater, inspection should be carried out occasionally.

Table 7 lists the standard inspection characteristics, inspection cycles, and inspection subjects classified by types of inspection.

Table 7 Inspection characteristics, inspection cycles, and inspection subjects

Regular inspection			Occasional inspection		
Inspection characteristics	Inspection cycles	Inspection subjects	Inspection characteristics	Inspection cycles	Inspection subjects
Number of coliform group	At least once a month	At the outlet of the receiving tank nearest to the service end	The inspection characteristics should be selected as circumstances require.	Inspection should be carried out whenever the recycling system processing performance is degraded to the extent that required quality level may not be attained.	The inspection subjects specified for the regular inspection, and the main valves and other relevant components should be inspected.
Residual chlorine	At least once a day				
Appearance Odor Turbidity PH	At least once a day				
COD*1	At least once a week				

*1. The measurement of BOD is time-consuming. It may therefore be substituted by the measurement of COD if the COD/BOD correlation is established in advance.

(5) Safety control

The following safety measures should be provided:

a. Proper safety measures against emergencies such as degradation of reclaimed water quality.

b. Proper and viable measures for the prevention of misconnection and drinking by misuse.

c. Strict observance of the Labor Safety and Hygiene Law, and other related rules, regulations, and standards concerning jobs in the water reclamation plant.

5. MODEL PROJECT OF WASTEWATER RECYCLING IN FUKUOKA CITY

In 1978, Fukuoka suffered its worst water shortage due to the lowest rainfall ever recorded in the 89 year history of the Fukuoka Regional Weather Station.

As a result, the citizens of Fukuoka had to survive on the limitation of water supply for 10 months from 20 May 1978 to 24 March 1979.

Fukuoka lies in the northern part of Kyushu and is poor in water resources.

Its population of over 1 million has always suffered water shortages. The 1978 water shortage, however, defeated the citizens drumming into them the lesson that water is limited and must not be wasted.

For this situation, the Fukuoka Municipal Government has been working on a model project since 1979 for recycling wastewater for miscellaneous use.

The construction of a recycling system started in 1979 and part of it was put into use in June 1980.

(1) Processing system

The recycling plant is fed with secondary effluent available from the Chubu Wastewater Treatment Plant which is draining wastewater from the 3700 hectare of central area of Fukuoka and treating it at a daily rate of 350,000 m^3 according to the activated sludge process. In the model project, the recycling plant is planned to process the secondary effluent at a daily maximum rate of 200 m^3. According to the 1973-78 statistics, the secondary effluent shows on average 8.5 mg/ℓ in SS, 9.5 in degree of color, and 394/m in number of coliform group.

In the project, the target quality of reclaimed water was set for a turbidity of less than 20 degrees, a non-offensive color, a non-offensive odor, and in number of coliform group less than 1/mℓ.

To achieve the target quality, a sand filter installed for in-plant recycling was used, because it was judged from the statistical data of effluent quality that the filtration would be enough for achieving a maximum SS removal rate of 50% if turbidity was equal to SS.

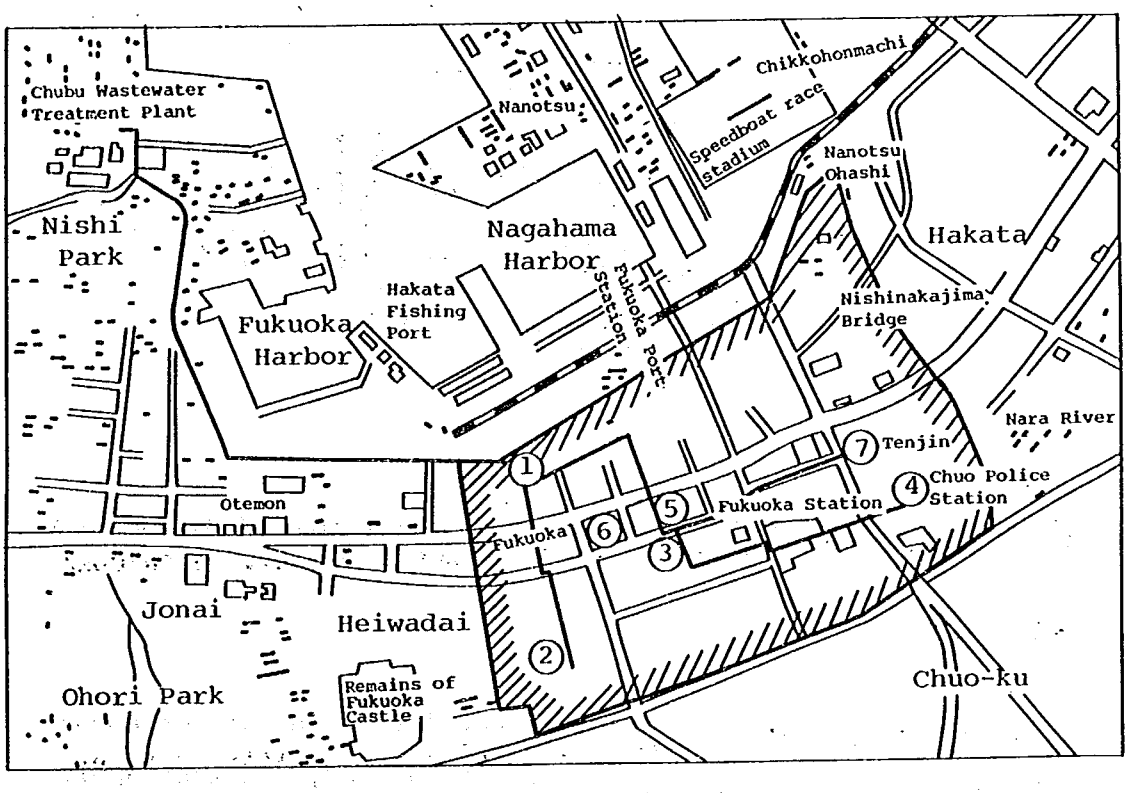
For removal of colors and odors, the activated carbon adsorption and ozonizing processes are usually conceivable. The ozonizing process, however, was used for the project because of its sterilization effect and simplicity in operation and maintenance. While the ozonizing process was considered to dispense with a sterilizing process, a hypochloric soda sterilizing process was added for the prevention of putrefaction of reclaimed water during delivery.

Figure 8 shows the process flow.

(2) Particulars of the recycling plant:

Processing capacity: 200 m^3/day
Sand filter (use of existing sand filter)
Ozonizing system: 200 m^3/day
Chlorinating system: 200 m^3/day

Delivery system:	pumping system, and automatic operation by pressure tank
Delivery pressure:	1.5 kg/cm^2 (min.) at the service end
Piping:	Mortar-lined cast iron pipe for straight runs; epoxy resin powder-coated cast-iron pipe for shapes



▨ : Area to which reclaimed wastewater is supplied.

Fig. 7 Site map

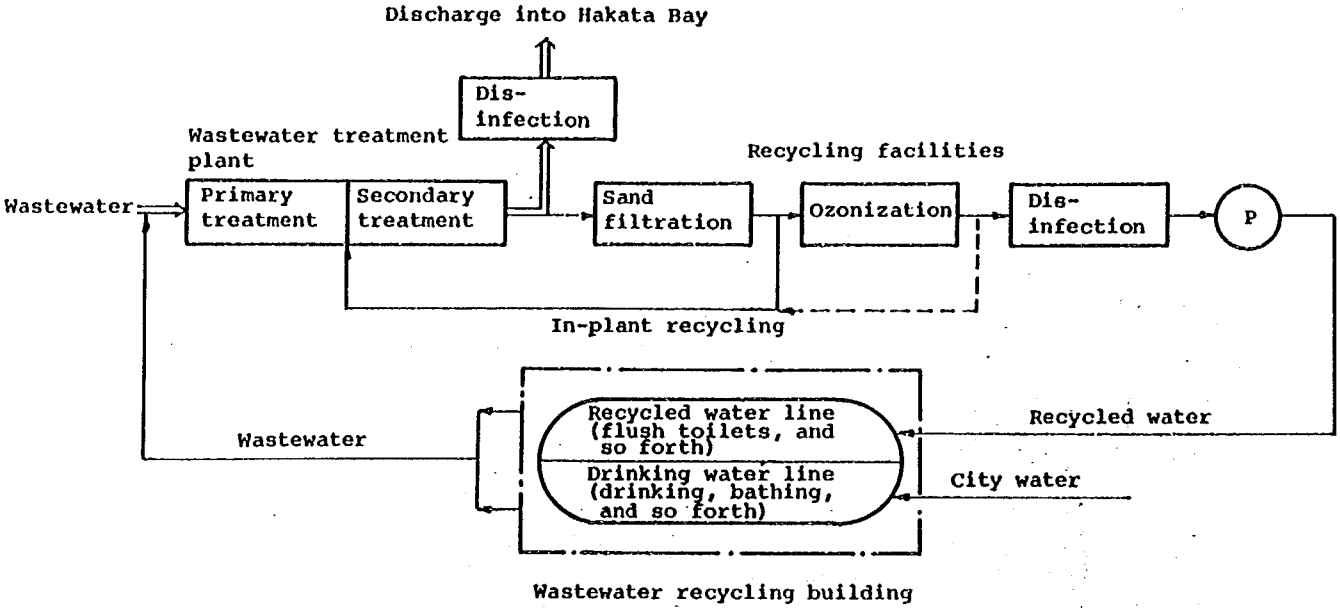


Fig. 8 Flow sheet of wastewater recycling model project in Fukuoka

(3) Supply cost

At present, reclaimed water is supplied at a rate of about 60 m^3 a day. The running cost is about ¥400/m^3, and the supply cost is about ¥850/m^3, including depreciation cost and interest.

If the recycling plant is expanded to its planned capacity of 200 m^3/day, the running cost will be reduced to ¥150/m^3, and the supply cost, including the depreciation cost and interest, will be reduced to about ¥290/m^3.

At Fukuoka Municipal Hall, which now consumes 50 m^3 of reclaimed water a day, reclaimed water may be a successful alternative to fresh water if the demand for miscellaneous water rises and the supply cost is reduced by the scale merit of the recycling operation, because fresh water presently costs about ¥210/m^3.

(4) User charge

In principle, user charge should be determined according to the supply cost. At present, however, the user charge for fresh water service is applied for the supply of reclaimed water.

6. WASTEWATER RECYCLING MODEL PROJECT IN SHINJUKU

The Tokyo Metropolitan Government has been pushing a project for recycling wastewater in Shinjuku, a subcenter of Tokyo, where an urban redevelopment project has also been promoted.

In anticipation of a sharp increase in water consumption and hence in sewage volume upon completion of the urban redevelopment project, the Tokyo Metropolitan Government has designated the central quarters of Nishi-Shinjuku 6-chome as a wastewater recycling model project site, and has advised high-rise building owners there to install double pipelines for the recycling of wastewater for flush toilets.

An outline of the wastewater recycling project

a.	Coverage:	50 ha within Shinjuku
b.	Planned capacity:	8000 m^3 a day for flush toilets in double-piped buildings and Chuo Park
c.	Installation:	Distribution mains (3.5 km), service lines, pumps, etc.

Wastewater reclamation center (related project)

a. Location: Basement of a high-rise building (B5, 39F) to be constructed within the urban redevelopment project site at Chuo District, Nishi-Shinjuku 6-chome

b. Source of raw water: Split from sewer pipes

c. Treatment: Advanced treatment

Project period: 1982 to 1984

Partial construction of the distribution facilities started in 1982.

Table 8 Breakdown of planned reclaimed wastewater flow

Classification of buildings, and others	Planned flow	Breakdown
Existing high-rise buildings	2650 m^3/day	Shinjuku Mitsui Building, Nomura Real Estate Building, Yasuda Fire and Marine Insurance Building, Shinjuku Center Building, Dai-ichi Life Insurance Building, Odakyu Century Hotel
Medium- to high-rise buildings under construction	860	South Annex of Keio Plaza Hotel, Nihon Seimei Sumitomo Building
Buildings to be constructed in future	4280	Tokyo Medical College, Nishi-Shinjuku 6-Chome Redevelopment Building, public spaces owned by the Tokyo Metropolitan Government
Shinjuku Chuo Park	210	
Total	8000 m^3/day	

Note: The buildings to which recycled wastewater is supplied are all double-piped.

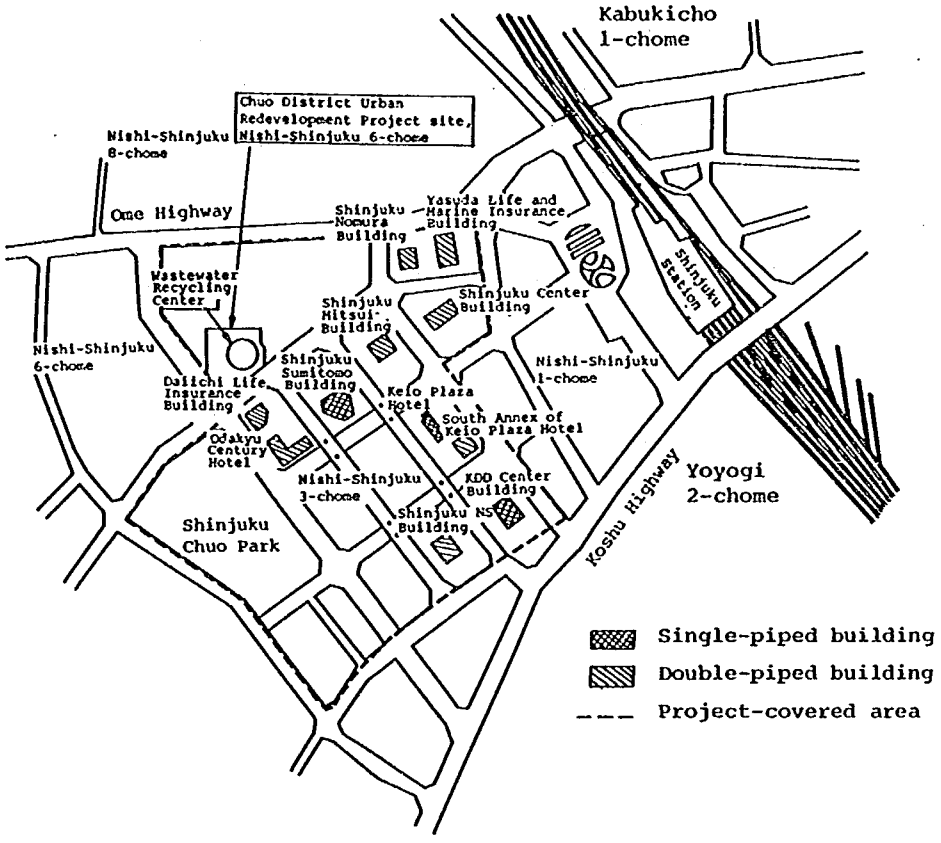


Fig. 9 Project site map

7. SUMMARY

(1) Wastewater recycling in Japan dates back to 1955 when the Tokyo Metropolitan Government first supplied recycled water to a paper mill for industrial use.

(2) In Japan, the effluents discharged from wastewater treatment plants amount to about 6,500 million m^3 a year, of which 50 million m^3 a year is recycled for ex-plant uses.

(3) In Japan, the long-term water demand-supply forecast dictates stepped-up efforts for direct recycling of wastewater.

(4) The Ministry of Construction has formulated technical guidelines (draft) for wastewater recycling.

(5) The Fukuoka Municipal Government and Tokyo Metropolitan Government have been executing model projects for wastewater recycling for flush toilet use, and so forth. In Fukuoka, the recycling plant is already in service.

(6) In the future, wastewater recycling will be actively promoted.

EXPERIENCES AND STUDIES FOR WASTE WATER REUSE INCLUDING GROUND WATER ENRICHMENT

Prof. Dr. H. Sontheimer
German Waterworks Association
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Experiences and Studies for Waste Water Reuse, including Ground Water Enrichment

Heinrich Sontheimer, Karlsruhe

Experience with ground water enrichment in Germany

For more than 100 years ground water enrichment has been one of the most important treatment steps, if surface water has to be used for drinking water purposes. Till today nearly all water works in our country using polluted surface waters have ground filtration as one part of the overall treatment. ..
A typical example for this is the river bank filtration, used along the Rhine river. As can be seen from Figure 1, the efficiency of this treatment step is very high regarding organics removal.

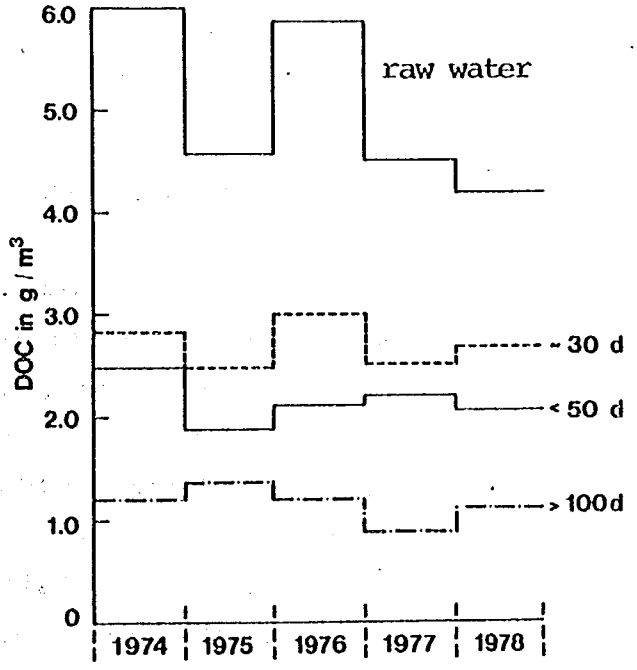


Figure 1: DOC removal for river bank filtration at the Rhine river with different retention times within the ground

One can see from this diagram that the removal efficiency for dissolved organics depends on the retention time and is fairly high while this treatment is now used for more than 60 years at the works studied here.

This positive experience with surface water treatment has initiated during the last few years a new way for treating polluted ground waters, too. As this example shows some important aspects of this type of treatment, some more details on this plant in Karlsruhe will be given in the following.

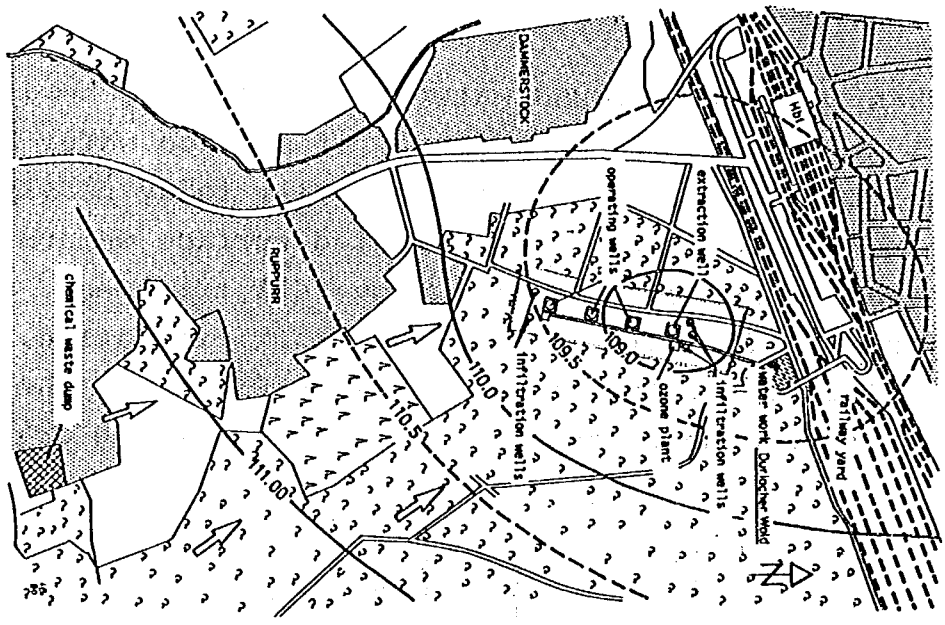


Figure 2: "Durlacher Wald" water work in Karlsruhe

One can see from Figure 2 that this oldest Karlsruhe water work is situated near the railway station and in between large housing areas. As the station has to be used for three oil refineries, an increasing ground water pollution with mineral oil products occurred slowly, and therefore a few years ago it became necessary to close down the well that was closest to the railway station area. Two years later pollution reached the next well and one had to decide on some sort of water treatment. The usual way would have been to use activated carbon filters with a preozonation ahead.

But referring to the experience made with ground filtration at the surface water works, it has been decided to reinfiltrate the ozonated water into the same aquifer at both ends of the four wells in line and to do this instead of carbon filtration. The scheme for this treatment is given in Figure 3.

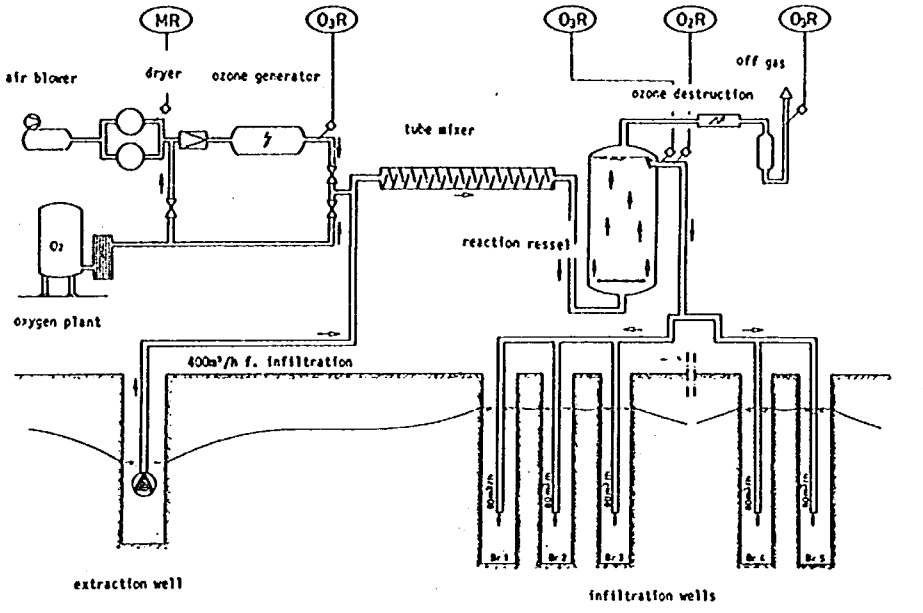


Figure 3: Process scheme for the "Durlacher Wald" water work

The ozone is introduced using a static mixing device and has about 10 min in the reaction tank before it is pumped into the ground again. The effects of this recirculation can best be seen by studying the oxygen concentration changes in the different wells.

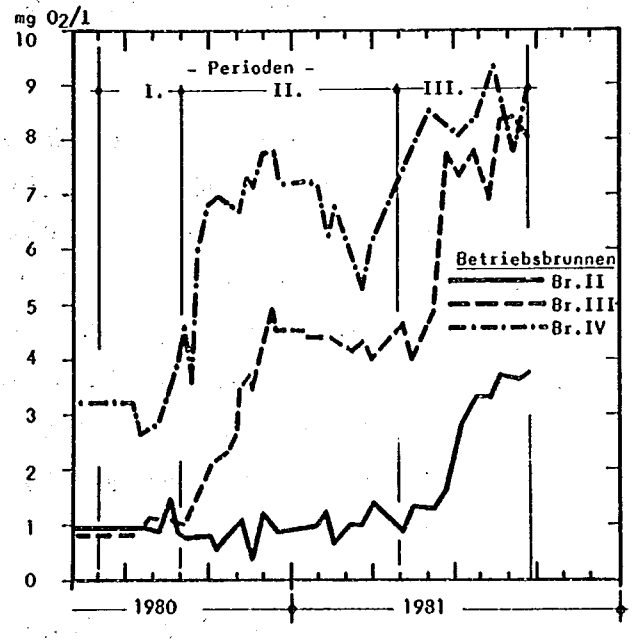


Figure 4: Oxygen concentration changes in the different wells of the water works
Perioden = periods
Betriebsbrunnen = operating wells

As could be expected when reinfiltrating oxygen containing water, the oxygen concentration increased with time in the wells used for drinking water, but there was still an oxygen consumption during the ground filtration of 2 - 3 mg/l indicating biological activity within the ground. The effects of this regarding DOC-removal can be seen from the results in Figure 5.

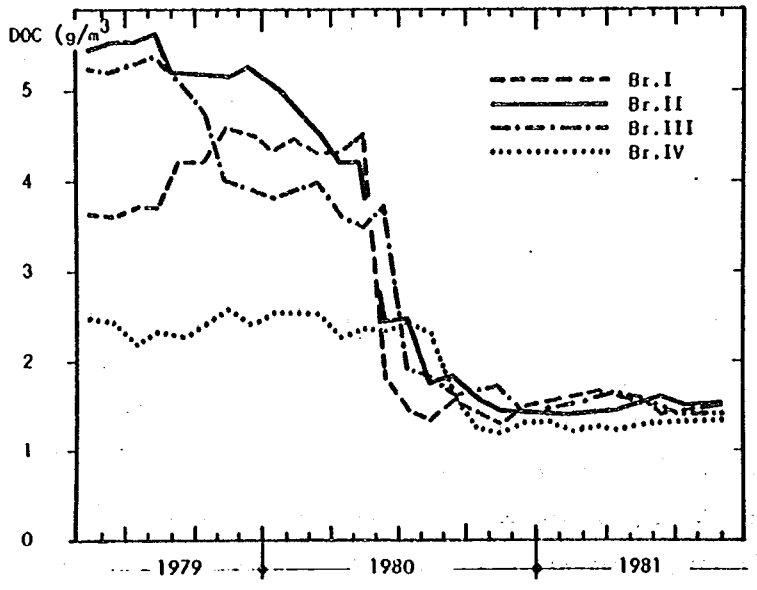


Figure 5: DOC-reduction through infiltration of ozonated ground water

The DOC-values found after starting the treatment in April 1980 showed a drastic reduction of the pollution within a very short time. Since this time the ground water can be distributed without any further treatment and without any disinfection, too, as it has now the same good quality as all the other ground waters in this area. Mineral oil products as well as taste- and odour-causing substances couldn't be found any more. There didn't even remain any other polluting substances in the water.

This example gives clear indication of the high efficiency of ground filtration for the removal of organic pollutants.

Having this in mind, one can understand that most studies on waste water reuse done till now in our country included some kind of ground infiltration. Instead biological activated carbon filters have been used, too.

Goals of waste water reuse studies

Considering the high amount of rain water and the many rivers with high water flow in our country, there doesn't seem to be a real necessity for waste water reuse. But there are some exceptions as, for example, the situation in West Berlin, where ground water renovation won't be sufficient for maintaining the necessary ground water level and thus waste water infiltration may be one possibility to solve this problem. There are some further considerations to use treated waste water for industrial purposes. In addition, there is no big difference between the treatment of polluted river water and advanced waste water treatment as it is necessary for reuse. It is for these reasons that some special problems in this field have been studied in our country. The aims of these experiments can be seen partly from the following Table, indicating the problems to be studied on a common German-Israelian research project. Here most of the experimental studies have been made at the Technion in Haifa.

Experimental studies for waste water reclamation at the Technion Haifa (Israel)

1.) Comparison between physico-chemical and biological processes for nitrogen removal.

2.) Effects of preozonation on the efficiency of activated carbon and slow sand filters.

3.) Removal efficiency of the treatment for pathogenous microorganisms.

4.) Toxicological evaluation of organic substances remaining after different treatment steps.

Figure 6:

As can be seen from these points two different ways for nitrogen removal have been compared as well as very intensive studies on the effect of ozone treatment for the efficiency of slow sand and activated carbon filters have been made. Very similar studies have been made in Berlin for groundwater infiltration and in Darmstadt together with GAC-treatment. Besides two different pretreatment steps have been tried at these two places while very intensive bacteriological and toxicological studies have been made in Haifa. The following paper will give a survey on the results reached at the different pilot plants.

Results from the Haifa pilot plant (WACHS, KOTT, YANNAI et al)

In order to study different possibilities for nitrogen removal, two pilot treatment streets have been installed and operated.

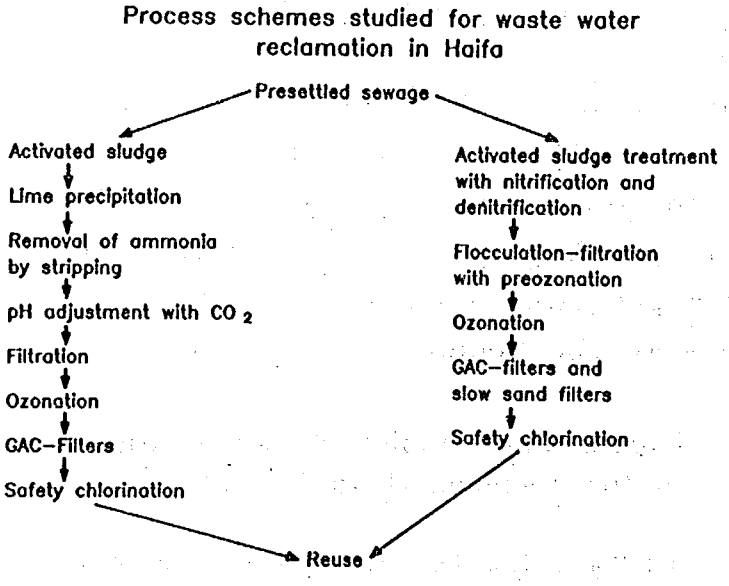


Figure 7: Process schemes for the pilot plant in Haifa

The most important difference can be seen in the processes used for the removal of nitrogen compounds. One pilot plant used lime precipitation and blowing out of ammonia while the other plant has used biological nitrification and denitrification in the same plant. Without going into more detail, it can be told that both types of treatment allowed to reach a nitrogen

concentration below 10 ppm, when starting with 50 - 80 ppm, but the biological plant has proven more economic, especially if sludge handling was taken into consideration. On the other side, lime precipitation gave a high efficiency in phosphorous removal while combined flocculation and filtration only resulted in a partial removal, though having the same effect in turbidity removal when preozonation was used. As there could be seen no necessity for such a fargoing phosphorous removal, many studies have been made without lime precipitation. A scheme of this pilot plant is given in Figure 8.

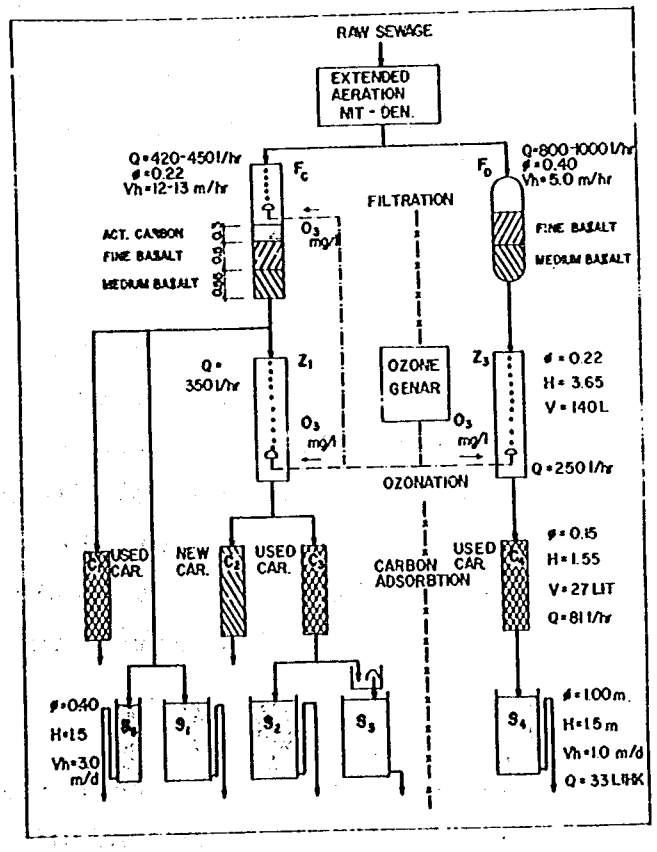


Figure 8: Scheme of the Haifa pilot plant

As one can see from this diagram, extended aeration with nitrification and denitrification has been used for biological treatment. After this a dual media filter with and without preozonation was used, followed by the main ozonation step. After this different carbon and slow sand filters have been used. A summary of the results is given in Figure 9.

Results of a pilot plant on waste water reclamation in Haifa (operational mode with nitrification and denitrification)

Sampling point	CSB g/m^3	DOC g/m^3	N g/m^3	Turbidity FTU
raw sewage	ca. 600	–	75	300
a. biolog. treatment	60–100	20–30	5–8	3–6
a. precip. and filtr.	20– 25	8–10	5–8	<1
a. biol. active carbon f.	10– 15	4– 5	5–8	<1
a. ozone, flocc. and filtr.	25– 35	10–15	5–8	<1
a. biol. active carbon f.	12– 15	6– 8	5–8	<1

Figure 9: Pilot plant results in Haifa

The Table indicates the high efficiency of the activated sludge plant with extended aeration and nitrogen removal. Furthermore one can see that the results with the lime treatment are somewhat lower than those with the small amount of flocculants added before the dual media filter. But after the biological activated carbon filters the results are very similar and they reach the same values of 2 - 3 ppm of DOC after the slow sand filters.

The positive effect of ozonation is still more pronounced, when analyzing dissolved organic chlorine before and after chlorine addition for disinfection purposes.

Concentration of organic chlorine compounds at the waste water reclamation plant in Haifa

Sample origin	Dissolved Organic Chlorine (DOCl)	
	with	without chlorine addition
Activated sludge effluent	32 ppb	472 ppb
GAC–effluent without preozonation	–	87 ppb
GAC–effluent with preozonation	–	57 ppb
Slow sand filter without preozonation	31 ppb	171 ppb
Slow sand filter with preozonation	7 ppb	58 ppb

Figure 10: DOCl-measurements at the Haifa pilot plant

The results allow the conclusion that a combined chemical and biological oxidation does not only remove nearly all originally present organic chlorine compounds, but the precursors for their formation, too, so that it will be possible to use chlorine for final disinfection while this will not be possible at all with the effluent of the extended aeration plant. In addition to this, it has been shown that ozone removes all pathogenic bacteria and viruses.

Most important are the results of the toxicological studies, of which an example is given in Figure 11.

Toxicity tests with the remaining organics (enrichment with XAD-resins) after different treatment steps affecting two different cell cultures

Testing method	cell culture	dist. H_2O	drinking water Haifa	reclaimed waste water after biological treatment and filtration		
				+10g/m³ Cl_2	+ AK-Filtrat.	+ AK-Filtrat. + Ozone
% of tissue cells with morphological damages	BgM	0	0	100	85	0
	MK	0	0	75	37	0
% of tissue cells surviving after 48h	BgM	100	100	0	23	100
	MK	100	89	7	22	80

Figure 11: Toxicity of the organics from different waters on tissue cultures

While nearly all tissue cells have damages, when using the organics from the effluent of the activated sludge plant after chlorination, there are no damages at all and a high survival percentage, when using water that was treated with ozone and activated carbon filters afterwards. Very similar results could be found by using the waters to feed mice over 3 generations. These results indicate very clearly the positive effects of using ozone instead of chlorine and to combine this with activated carbon filter treatment.

Results of pilot plant study in Berlin (SARFERT et al)

The high efficiency of ozone treatment for a better removal of dissolved organics could be seen during the Berlin studies, too. One example for this is given in Figure 12.

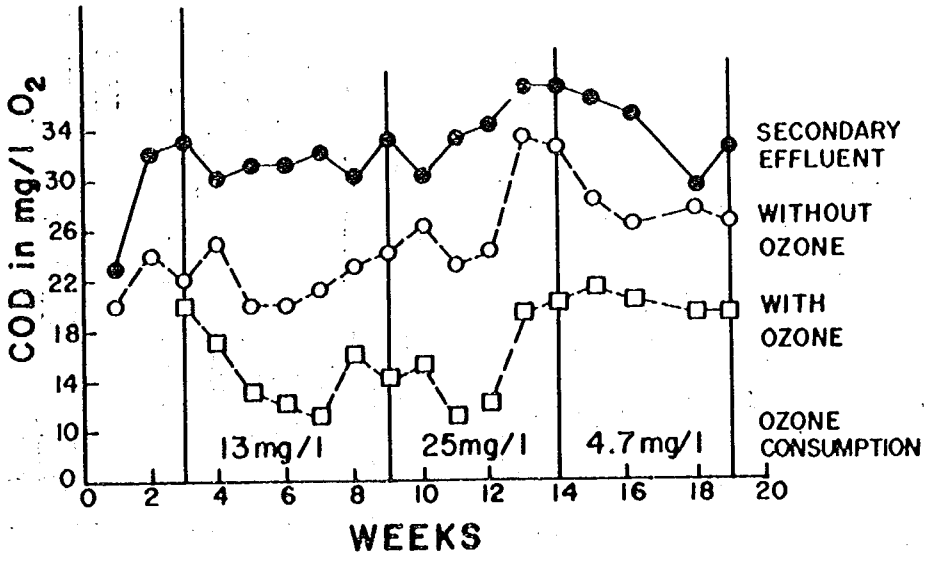


Figure 12: Effect of preozonation on DOC-removal in slow sand filters

During these studies secondary effluent at the Berlin sewage plant was treated in slow sand filters (0,1 m/h, 2 m height) with and without preozonation. One can see that there results a significant improvement in DOC reduction through the micro-organisms in the slow sand filter after the use of ozone. This is due to oxidation processes leading to better biodegradable substances.

Considering these results and other experiences it has been decided to use ozonation for waste water reuse, too, adding ozone before the infiltration step and after flocculation. The process scheme for the pilot plant, treating 2 x 100 m^3/h can be seen out of Figure 13.

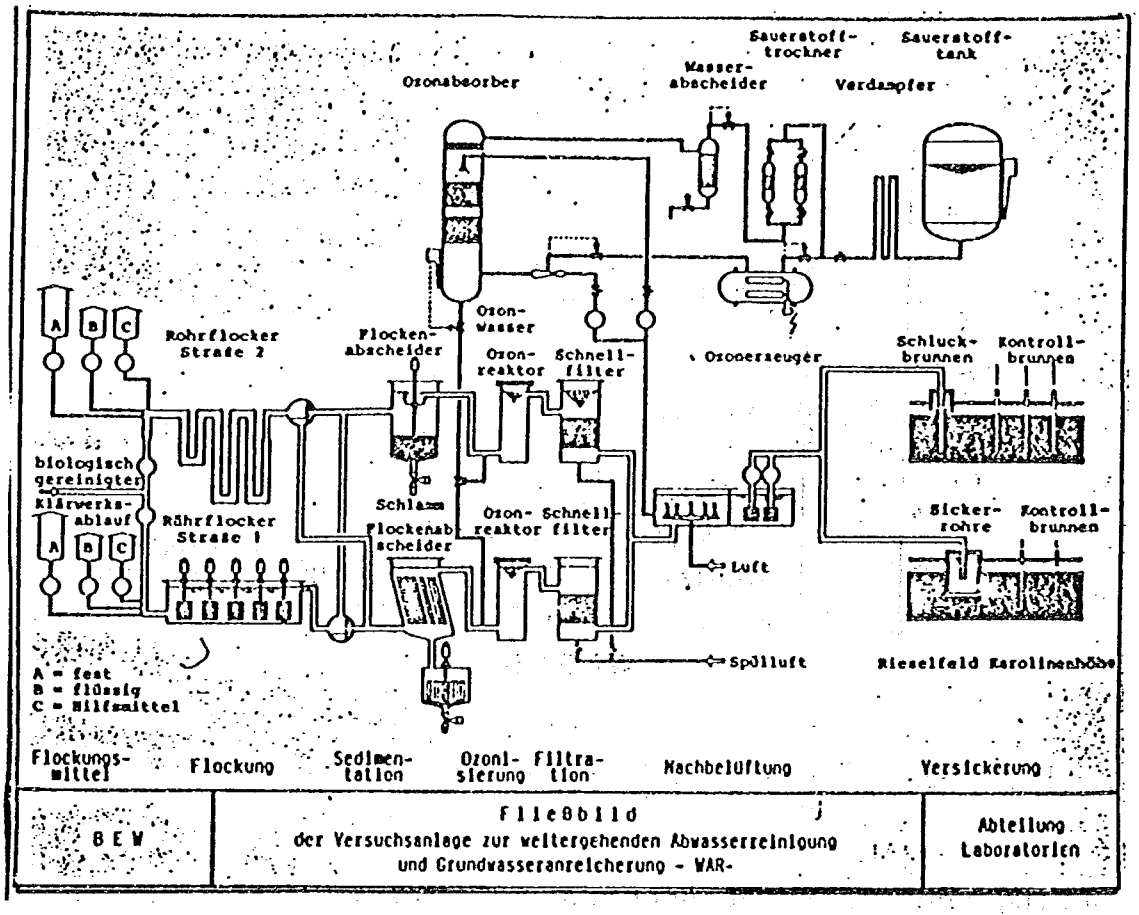


Figure 13: Process scheme of the waste water reuse pilot plant at Berlin
(Translation of legends see page 12a

As one can see from the diagram, there have been studied two ways doing the flocculation and sedimentation. One street is using long pipes for the flocculation step and a special vertical flow clarifier for the floc separation. The other street uses impellers for intensive mixing and flocculation and a parallel plate interceptor for floc separation. Besides this, different installations have been tried for the infiltration step. Typical results from the treatment plant are summarized in Figure 14.

Appendix

Translation of legends in Figure 13

Sauerstofftank	= oxygen tank
Verdampfer	= evaporator
Sauerstofftrockner	= oxygen
Wasserabscheider	= water separator
Ozonabsorber	= ozone absorber
Ozonwasser	= ozone water
Ozonerzeuger	= ozone generator
Rohrflocker, Str. 1	= tube flocculator, train 1
Rührflocker, Str. 2	= stirrer flocculator, train 2
biologisch gereinigter Klärwerksablauf	= secondary effluent
Schlamm	= sludge
Flockenabscheider	= floc separator
Ozonreaktor	= ozone reactor
Schnellfilter	= rapid sand filter
Luft	= air
Schluckbrunnen	= injection well
Kontrollbrunnen	= monitoring well
Sickerrohre	= subsurface drainage
Spülluft	= air wash
Rieselfeld Karolinenhöhe	= land treatment site Karolinenhöhe
fest,flüssig,Hilfsmittel	= solid, liquid, aids
Flockungsmittel	= flocculants
Flockung	= flocculation
Ozonisierung	= ozonation
Nachbelüftung	= aeration
Versickerung	= recharge

Results of waste water recovery plant in Berlin

	DOC	COD	UV(254nm)
secondary effluent	11,5g/m^3	35,5g/m^3	30,7m^{-1}
after flocculation and filtration without ozone	8,1 "	24,9 "	19,6 "
after ground infiltration	5,1 "	20,5 "	14,1 "
after flocculation and filtration with ozone	7,4 "	26,2 "	13,5 "
infiltration water	6,7 "	23,2 "	11,7 "
after ground filtration	4,0 "	17,0 "	8,1 "

Figure 14: Mean values for treatment efficiency of the Berlin pilot plant for waste water reuse

The results show that there can be reached a slightly lower DOC and COD as well as UV-absorption, when using oxidation with ozone. But the differences are not very great. Nevertheless, one has to consider that the ground water in the area used for these studies doesn't have a lower DOC than the waste water to be treated in the ground. This depends on the organic concentration in the aquifer and can't be changed very easily. The very similar concentrations of the treated waste water and the ground water on the other side didn't allow to study the effect of ozone pretreatment in more detail. But all results indicate that there is no reason for not using the pretreated water for ground water enrichment.

Results of studies in Langen (WOLTERS et al)

Very similar results could be found in an other pilot plant near Frankfurt in Langen. The process scheme of this plant can be seen in Figure 15.

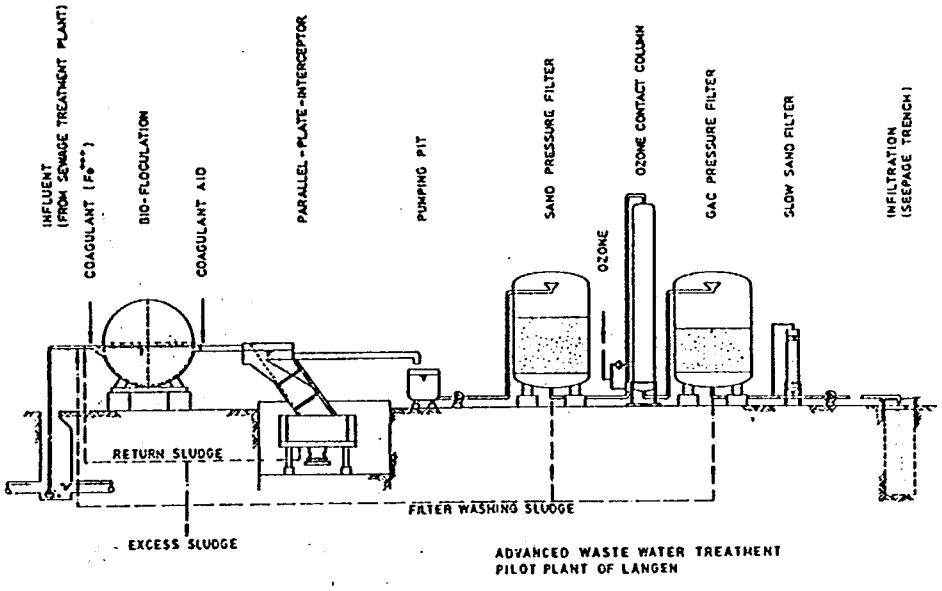


Figure 15: Process scheme of the pilot plant for advanced waste water treatment for reuse purposes in Langen (Germany)

Here instead of normal flocculation and sedimentation a process called "bioflocculation" was used. It consists of a rotating disc biological plant, where flocculants are added to get a better removal efficiency. A slurry recirculation from the sedimentation tank to the basin with the rotating discs has proven worthwhile as well as some polyelectrolyte addition before the sedimentation. The pretreated water has been filtered and then treated with ozone before the GAC pressure filters. After this slow sand filters as well as ground infiltration have been used in parallel installations.

Typical results giving COD-values, for the different treatment steps are given in Figure 16.

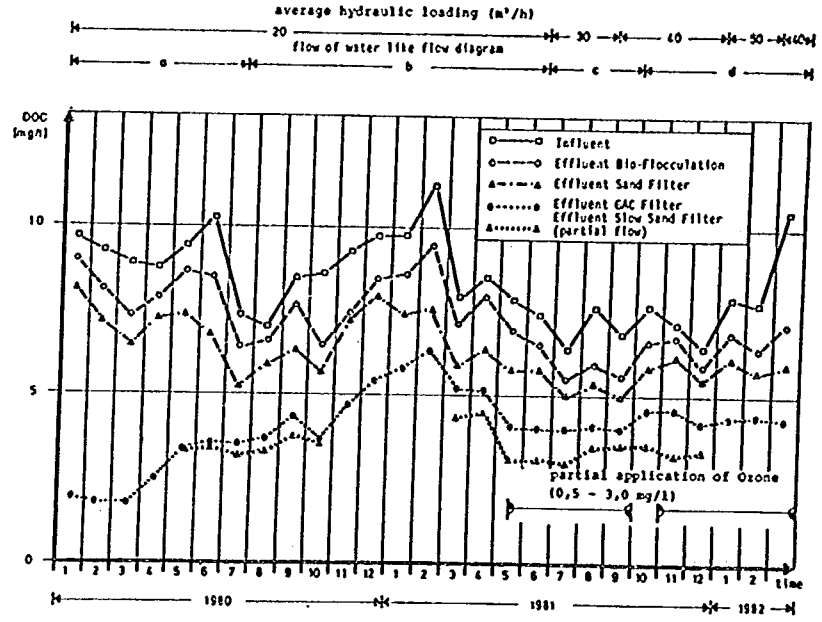


Figure 16: Results of the Langen pilot plant for waste water reuse

The diagram shows that each treatment step contributes to the reduction of the concentration of dissolved organics. Finally it was possible to reach COD values below 10 ppm and DOC values below 4 ppm. This agrees very well with the Berlin and the Haifa data.

Lower final DOC values could be got when higher ozone dosages have been used. Typical results for this effect can be seen from Figure 17.

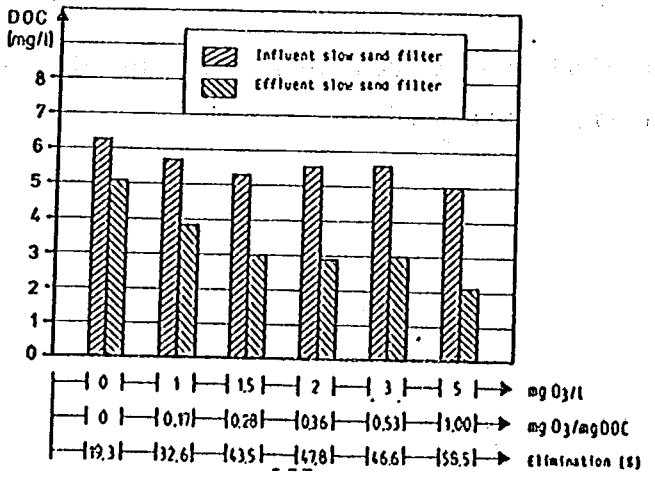


Figure 17: Effect of ozone dosage on treatment efficiency

The data given in this diagram confirm the high efficiency of ozone treatment in conjunction with different filters working biologically. Very similar observations have been made for surface water treatment. The combined chemical and biological oxidation using ozone for the first step and activated carbon filters after this for a combined adsorption and biological oxidation is now used in many water works with good success. The results from the Langen plant and from the other mentioned pilot studies prove the same positive effects for advanced waste treatment, too.

Conclusions

While waste water reuse is not a very urgent problem in our country, some special problems have been studied in different pilot plants. The results allow for the following special conclusions.

1.) Ozone pretreatment before activated carbon filters as well as before ground filtration is a very effective means for a high organics removal. Using high enough dosages DOC values can be reached similar to those in drinking water.

2.) More important than this effect are the results of the toxicological studies indicating an excellent water quality without any toxic effects, when using ozonation combined with carbon filters.

3.) In order to prevent flocculation and sedimentation, where high amounts of flocculants have to be used, a combined flocculation and filtration process removes all suspended and colloidal particles, if a small ozone dosage is added before the flocculants. In this case there is only a partial phosphate removal, but there seems to be no necessity for further removal, if the water will be reused for industrial or agricultural purposes or for infiltration and ground water enrichment.

The above mentioned three points of our studies differ from those made in other countries. All other observations are very similar to those described in literature. Using all this knowledge waste water reuse will be possible from the water quality side. But there may exist some other restrictions, especially those on the economics of this type of treatment. It is for these reasons that waste water recovery and reuse will be used only in very special cases in our country for the next 10 years.

LAND TREATMENT OF SEWAGE

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and
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1. INTRODUCTION

Eutrophication of lakes, reservoirs, and other landlocked waters is one of the most serious water pollution problems in Japan. It is therefore strongly urged that nutrients be removed not only at publicly-owned sewage treatment plants, but also at nonsewer sources of pollution, such as waste from rural households and livestock farms. For nonsewer sources of pollution, land treatment is one of the most promising methods and it has become quite common in recent years. Some local municipal governments subsidize construction of treatment facilities to promote domestic sewage treatment in sewerless areas. There are already several hundred land treatment facilities installed under such promotion plans.

Almost all land treatment facilities in Japan are based on the infiltration-percolation system in which subsurface pipes or trenches are used. One reason for this is that Japan's principal agricultural product is rice and treated domestic effluent contains too high a concentration of nitrogen for paddy irrigation.

Fig. 1 shows a cross section of a typical subsurface trench.

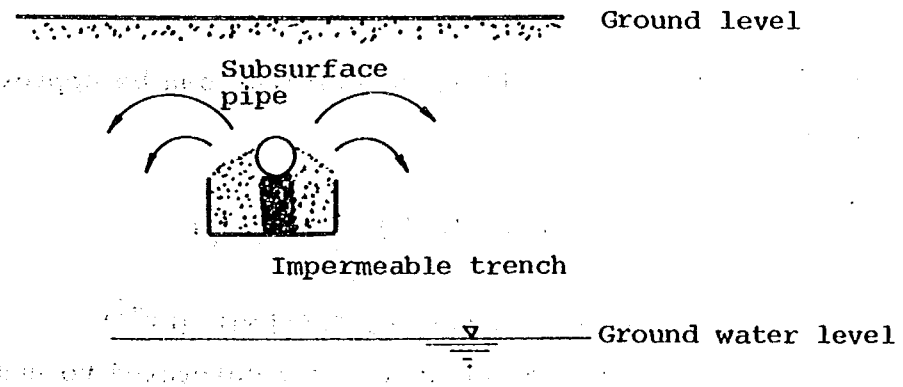


Fig. 1 Concept of subsurface trench for land application

The wastewater runs from a discharge pipe into the subsurface trench, then seeps into the soil by capillary action. It is claimed that this system rarely clogs, but the design parameters of land treatment facilities have not yet been fully clarified and such facilities are often out of service due to clogging.

This report concerns the infiltration rate and removal of phosphorus and nitrogen at land treatment facilities, including the one shown in Fig. 1.

2. POTENTIAL INFILTRATION RATE OF SUBSURFACE TRENCH

2.1 Infiltration of Clean Water

It is possible to theoretically analyze infiltration of clean water into soil via a subsurface trench; this has already been done[1], and the infiltration rate has been determined by numerical analysis.

In the case of unsaturated, steady-state infiltration, the flow of ground water can be determined by the following equation:

$$\mathrm{div}\ \{\rho K(\psi).\mathrm{grad}\ (\psi + Z)\} = 0$$

Where: ρ = density of water (ML^{-3})
ψ = Head (L)
Z = Vertical coordinate (L)
$K(\psi)$ = Unsaturated permeability coefficient (LT^{-1})

The unsaturated permeability coefficient can be approximated by the following formula:

$$K(\psi) = Ko\ (\psi_b/\psi)^{\eta} \quad (\psi \leqq \psi_b)$$

Where: Ko = Saturated permeability coefficient (LT^{-1})
ψ_b = Head in a transient state from saturated to unsaturated state (L)
η = Constant

For sandy loam (Ko = 5.6×10^{-4} cm/s; ψ_b = -39.8 cm; n = 3.5), the infiltration rate is calculated as shown in Fig. 3, when the subsurface trench width b is 20 cm, height d is 7 cm, trench water depth h is 3.5 cm, and the covering earth depth D is 58 cm.[1] (See Fig. 2 for symbols.)

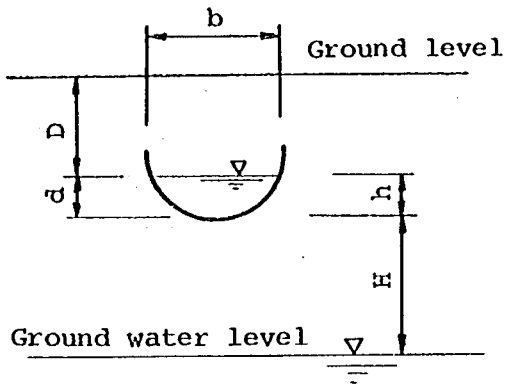


Fig. 2 Notation for numerical analysis

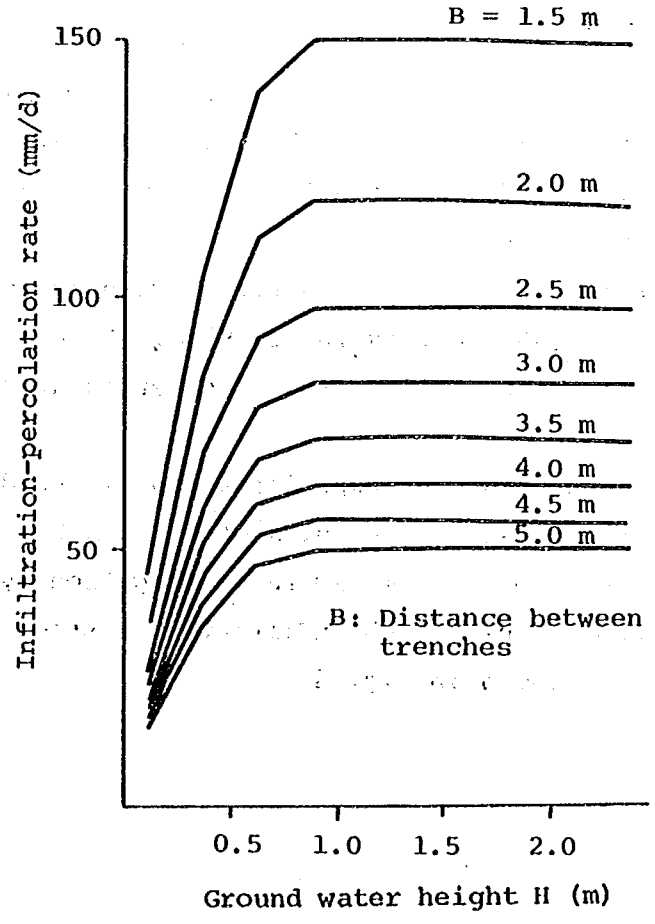


Fig. 3 Infiltration rate for clean water from subsurface trench

In Fig. 3 parameter B is the distance between subsurface trenches (or subsurface pipes). Fig. 3 shows that the infiltration rate was almost constant when the head difference from the ground water level exceeds 0.7 m. When installation spacing B is 2.5 m, the infiltration rate is nearly twice as fast as that when the spacing is 5 m. The infiltration per unit trench or pipe length becomes lower when the installation spacing is greater than 2-3 m.

2.2 BOD Loading VS Clogging

Soil clogging and resultant decrease in the application rate is one of the most critical factors in designing a land treatment system. This phenomenon is particularly crucial with a subsurface infiltration system. The causes of clogging include bacteria growth, solids in the wastewater, bubbles, formation of insoluble salts, etc. In a subsurface trench infiltration system, clogging is caused chiefly by bacteria growth. In this case, BOD loading serves as a clogging index. For this reason, the relationship between BOD loading and clogging was studied by analyzing the available data from existing land treatment facilities and the experimental facilities.

A subsurface trench infiltration system used to treat biologically treated swine wastes (BOD: approximately 90 mg/ℓ) at a rate of 2-3 mm/day has been free of clogging for 5 years. (The effluent BOD loading is 0.18-0.3 $g/m^2/day$.)

At present, land treatment experiments on filtered secondary effluent are conducted using four units of 5 x 5 x 2 m lysimer: one is used for an overland flow system (surface irrigation system) and the other three for subsurface trench infiltration systems with different types of trenches as shown in Fig. 4

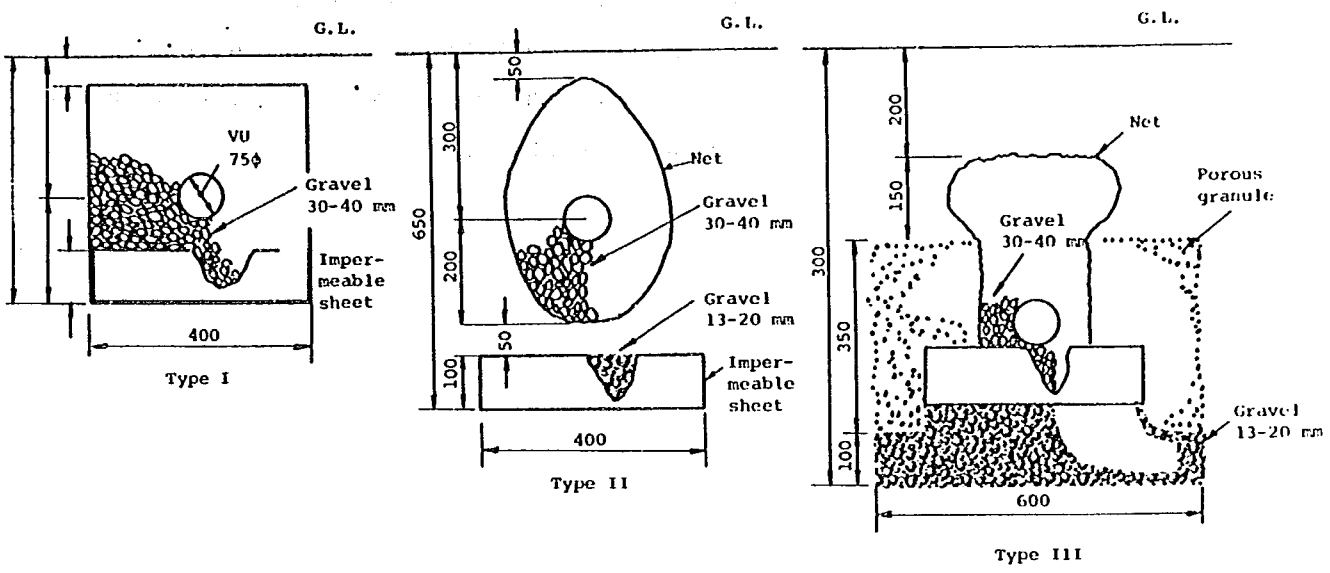


Fig. 4 Cross section of treches

Two trenches were installed in each lysimeter. At first, the application rate of filtered secondary effluent was set a 430 mm per 2 weeks (application for 9 days; pause for 5 days) or 31 mm/day on average. The operations have been continued on lysimeters with Type II and III trenches at the initially set application rate for about 1 year without clogging problems. In the case of the lysimeter with Type I trenches, however, the effluent seeped to the surface in several months because of clogging, and the application rate had to be reduced by half to about 15 mm/day. Effluent BOD was about 5 mg/ℓ on average, and BOD loading was 0.15 g/m^2/day at a 31 mm/day application rate.

According to tests on subsurface application using a lysimeter of the same dimension, it was found that when the settled sewage with an average BOD of 50 mg/ℓ was applied at a fixed water level in the subsurface trench, at first, the infiltration took place at a rate of 40 mm/day, but in 1 month, the infiltration rate had leveled off to 5 mm/day.[2] At the final filtration rate, BOD loading was about 0.25 g/m^2/day. Therefore, it is concluded that BOD loading less than 0.2-0.3 g/m^2/day would be appropriate to avoid clogging.

Table 1 lists the design features given in the design manual published by USEPA.[3]

In the subsurface infiltration process, BOD loading was about 0.7 g/m^2/day when the application rate was 2 in/week with the BOD concentration in the settled sewage at 100 mg/ℓ. This value is somewhat large compared with the value discussed above.

Table 1 Comparison of design features for land treatment processes

Feature	Principal processes			Other processes	
	Slow rate	Rapid infiltration	Overland flow	Wetlands	Subsurface
Application techniques	Sprinkler or surface [a]	Usually surface	Sprinkler or surface	Sprinkler or surface	Subsurface piping
Annual application rate, ft	2 to 20	20 to 560	10 to 70	4 to 100	8 to 87
Field area required, acres [b]	56 to 560	2 to 56	16 to 110	11 to 280	13 to 140
Typical weekly application rate, in.	0.5 to 4	4 to 120	2.5 to 6 [c] 6 to 16 [d]	1 to 25	2 to 20
Minimum preapplication treatment provided in United States	Primary sedimentation [e]	Primary sedimentation	Screening and grit removal	Primary sedimentation	Primary sedimentation
Disposition of applied wastewater	Evapotranspiration and percolation	Mainly percolation	Surface runoff and evapotranspiration with some percolation	Evapotranspiration, percolation, and runoff	Percolation with some evapotranspiration
Need for vegetation	Required	Optional	Required	Required	Optional

a. Includes ridge-and-furrow and border strip.

b. Field area in acres not including buffer area, roads, or ditches for 1 Mgal/d (43.8 L/s) flow.

c. Range for application of screened wastewater.

d. Range for application of lagoon and secondary effluent

e. Depends on the use of the effluent and the type of crop.

1 in. = 2.54 cm

1 ft = 0.305 m

1 acre = 0.405 ha

3. ADSORPTION OF PHOSPHORUS BY SOIL

3.1 Phosphorus Adsorbability of Soil in Terms of Langmuir Ultimate-Capacity Constant

The equilibrium state of phosphorus adsorption by soil is approximated by Langmuir equation:

$$\frac{C}{q_\infty} = \frac{1}{Kb} + \frac{C}{b} \qquad (1)$$

Where: q_∞ = Saturation capacity per unit weight of soil when the adsorbate concentration in the solution is C

K = Langmuir equilibrium-energy constant

b = Langmuir ultimate-capacity constant

Several attempts have been made to correlate various characteristics of soil with the adsorbability expressed in Langmuir ultimate-capacity constant (b).[4]-[6] Chemical factors that govern the phosphorus adsorbability of soil are said to include (a) aluminum and iron in the soil, (b) calcium in the soil, (c) organic substances in the soil, and (d) pH. Novotny et al proposed a multiple correlation formula expressing b in pH, clay content, and organic carbon content assuming that the fluoride-extractable aluminum in the soil is in inverse proportion to soil pH. Tondon expressed the adsorbability in terms of fluoride-extractable aluminum in soils.[6]

The authors tried to obtain a similar correlation formula by measuring Langmuir ultimate-capacity constant (b) of typical soils in Japan. It was found that calcium, organic substances, etc. contribute little. By making use of the fluoride-extractable aluminum in soil alone, a regression equation having a comparatively high correlation coefficient was obtained as follows:

$$b = 0.8288\ (FE.A\ell) + 0.6298 \qquad (2)$$

Where: b = Langmuir ultimate-capacity constant obtained after shaking for 1 day at 20°C (mg-P/g-soil)

(FE.Aℓ) = Quantity of aluminum extractable with 0.5N NaF solution (mg-Aℓ/g-soil)

For the measurement of b, 1 g of soil and 200 ml of dibasic potassium phosphate solution adjusted to pH 7 were used. For the measurement of FE.Aℓ, Kawaguchi's titration method [7] was used.

Thirty samples were used to determine Eq. 2. The correlation coefficient of the equation is 0.905. Fig. 5 shows a comparison between the observed values and the values calculated by Eq. 2.

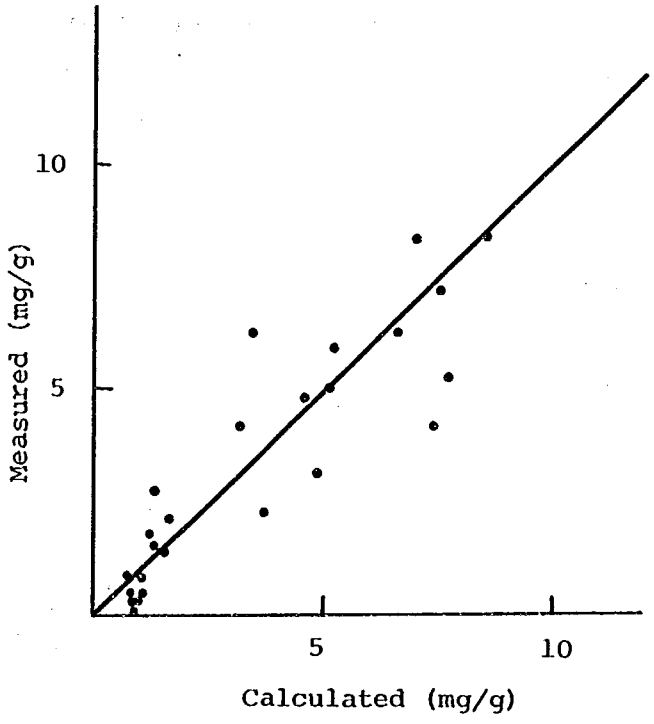


Fig. 5 Comparison of measured and calculated ultimate-capacity constants

The effect of iron in the soil is not considered significant. According to a multiple regression analysis using FE.Aℓ and iron as variables, the term for iron is one order smaller than that for FE.Aℓ, and the correlation coefficient is somewhat lower than that of Eq. 2.

It has been reported that pH has a significant effect on the phosphorus adsorbability of soil and that the adsorption capacity reaches a maximum when the pH is in the range of 3 to 4.[5] The maximum adsorption capacity at a pH value of 3-4 was about 1.5-2 times as much as that at a pH value of 7.[8]

The amount of organic substances in soils influence the adsorption capacity to some extent.[4] It was observed, however, that organic content influences the adsorption rate rather than the adsorption capacity. Fig. 6 shows b_1/b_5 (b_1 = Langmuir ultimate-capacity constant determined by 1 day adsorption test; b_5 = Langmuir ultimate-capacity constant determined by 5 day adsorption test) and the quantity of organic substances in soil relative to the aluminum content. In Fig. 6, C is the organic carbon content (mg-C/g) in soil, and FE.Aℓ is the fluoride-extractable aluminum content (mg-Aℓ/g). Fig. 6 shows that the increase in the content of organic substances decreases b_1/b_5, that is, it takes a longer time for the adsorption to reach a saturation level.

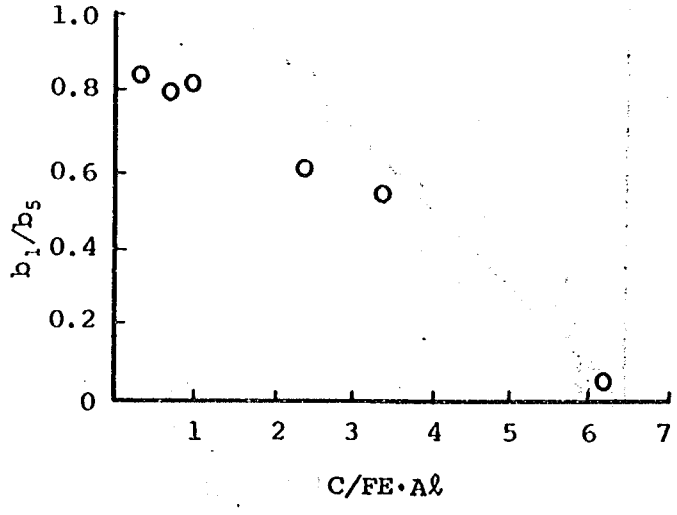


Fig. 6 Effect of organic carbon in soil on phosphorus adsorption

3.2 Relationship between Langmuir Ulitimate-capacity Constant and Phosphorus Adsorption Capacity Determined by Column Test

Column tests were carried out to determine whether Langmuir ultimate-capacity constant can be an index of phosphorus adsorption capacity in the actual application.

A 10 cm inside diameter and 80 cm high column was used. A weathered granite soil sample was filled into the column to a depth of 50 cm. The phosphorus adsorption capacity of the soil was very low with b at 0.32 mg-P/g. The column was fed with a dibasic potassium phosphate solution adjusted to pH 7, and the phosphate concentration was set at 5 mg/ℓ or 10 mg/ℓ as P. The test was conducted for 22 weeks (154 days).

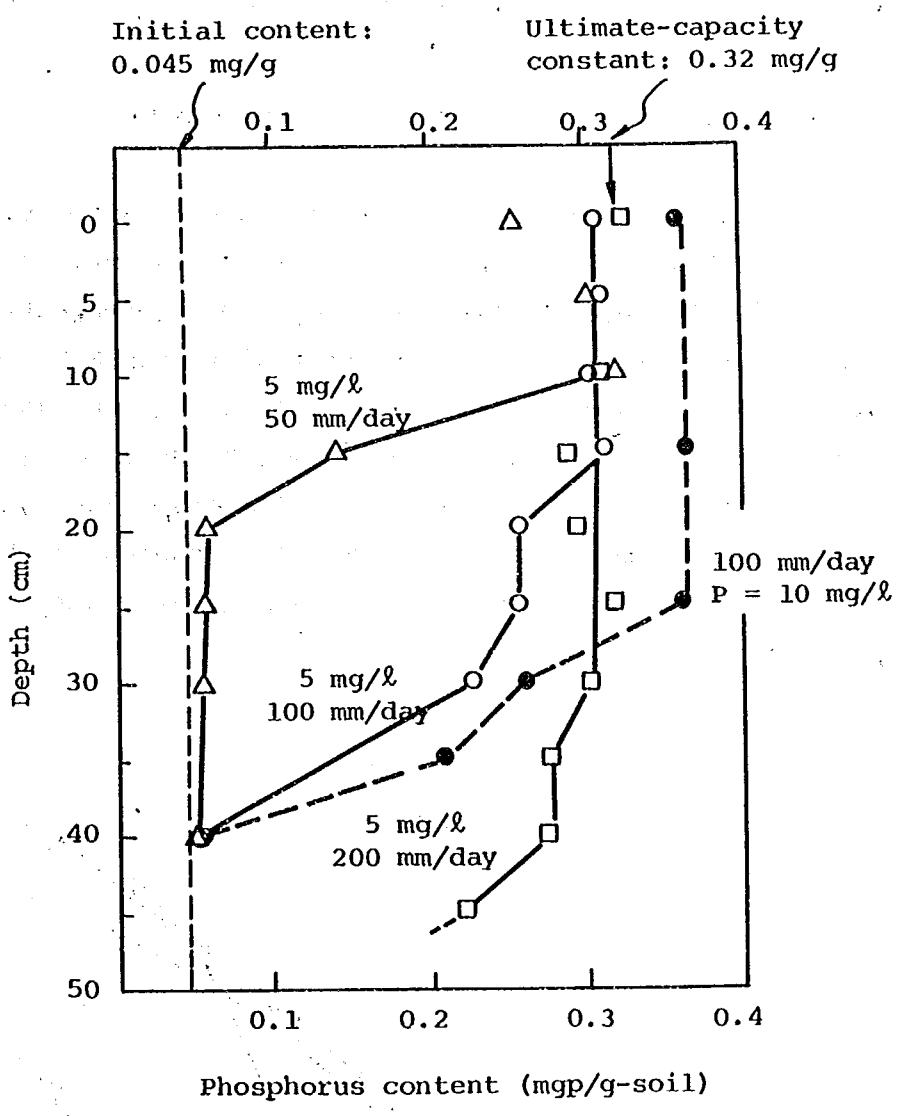


Fig. 7 Distribution of phosphorus content in soil

Table 2 Mass balance of phosphorus obtained from column tests

Run	Pin feed mg/ℓ	Application Rate mm/d	P loading mg	P accumulated in soil mg	P in effluent mg
A-1	5	50	330	408	-
A-2	5	100	660	839	-
A-3	5	200	1,320	1,279	36
A-4	10	100	1,320	1,186	-

Mass balance during 154 days of experiment

Fig. 7 shows the phosphorus distribution in the soil after 22 weeks. The mass balance of phosphorus determined from Fig. 7 is given in Table 2. According to Fig. 7, when the phosphorus concentration in the feed solution was 5 mg/ℓ, the phosphorus content in the soil was nearly equal to b. As the phosphorus concentration in sewage in Japan is nearly 5 mg/ℓ, Langmuir ultimate-capacity constant may be used to express the saturated adsorption capacity of soil. From the comparison between Run A-3 and Run A-4, it was found that even when the loading is the same, the higher the concentration and the lower the application rate, the higher the adsorption capacity.

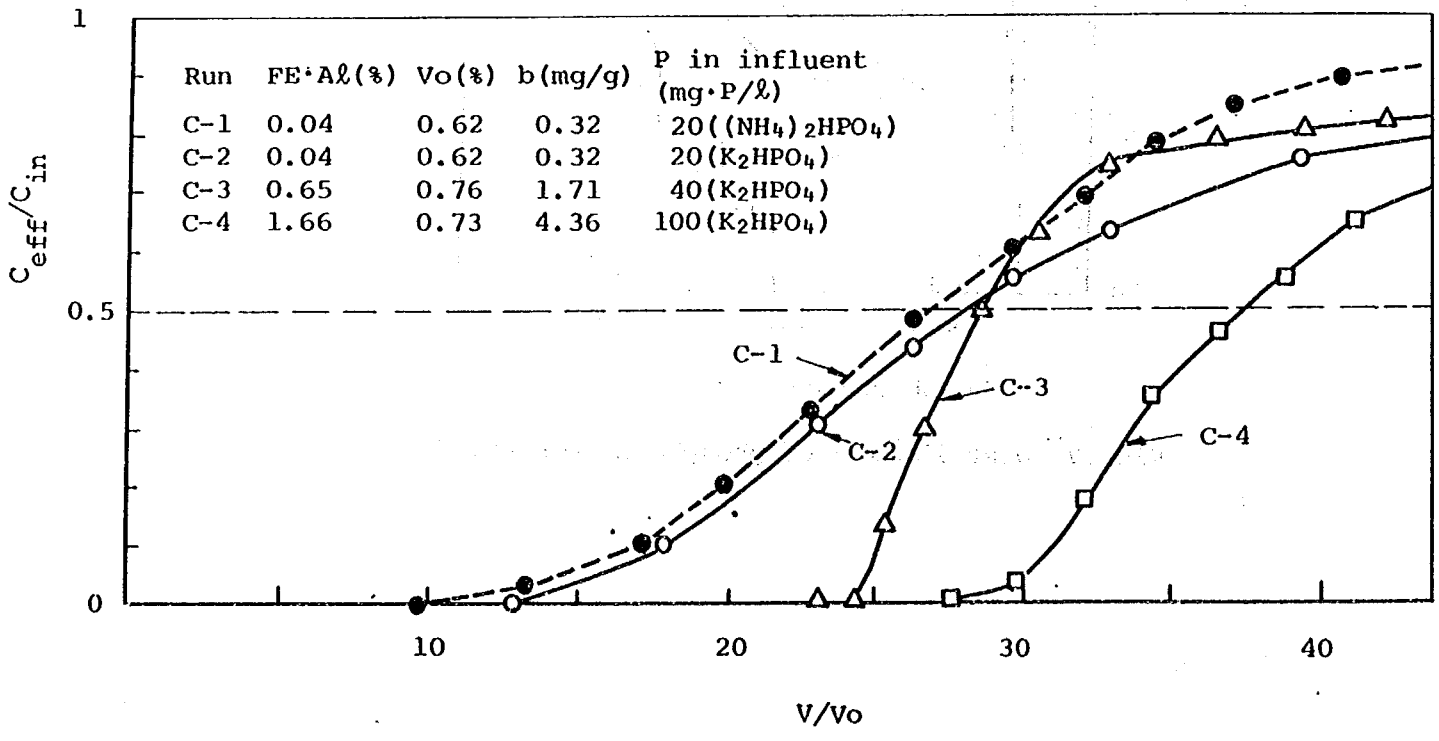


Fig. 8 Phosphorus breakthrough curve

Fig. 8 shows a phosphorus breakthrough curve obtained by similar experiments using a small column. The column used was 3.5 cm in diameter and 30 cm high. The soil was filled into the column to a depth of 13 cm. The abscissa shows the cumulative volume of applied feed expressed in multiples of the void volume of the soil.

The value of V/Vo (V = Cumulative volume of applied feed; Vo = Void volume of the soil) at the time when phosphorus was detected in the effluent was about 10 in Run C-1 and C-2, 23.1 in Run C-3, and 27.5 in Run B-4, respectively, and phosphorus loading by that time corresponded respectively to 26%, 71%, and 71% of the adsorption capacity calculated from Langumir ulitmate-capacity constant. This also suggests that the higher the concentration in the feed, the higher the adsorption. It should be noted however that the soils used in Runs C-1 and C-2 contain much less fluoride-extractable aluminum compared with the organic content, and that the organic substances may have interfered with phosphorus adsorption.

4. REMOVAL OF NITROGEN BY LAND TREATMENT PROCESS

It is known that when wastewater with low organic content, such as well-treated secondary effluent, is percolated through soils, nitrification of ammonia nitrogen takes place, but denitrification rarely happens. To determine the organic carbon concentration necessary for denitrification in the soils, tests were carried out using a 1.6 cm diameter, 30 cm long column filled with 50 g of loamy soil to a depth of 25.6 cm. The feed to the column was adjusted with sodium nitrate to make the nitrogen concentration 10 mg/ℓ. The feed was added with glucose to make up for organic carbon. The glucose concentration was set at 0, 20, 50, 100, and 150 mg/ℓ in terms of TOC.

The tests were carried out at an application rate of 75 mm/day, and the effluent quality stabilized in about 7 days. Table 3 gives the nitrogen balance after the steady state was attained. As the table shows, it is necessary to increased the C/N ratio to above 10 for satisfactory denitrification. Assuming that the oxygen utilized to oxidize organic substances is limited to the oxygen in nitrates, the C/N ratio is about 1.3 stoichiometrically. It is evident that in unsaturated percolation, the oxygen in air contributes much to oxidation of organic substances.

Table 3 Nitrogen removal and C/N ratio

Run	TOC in feed mg C/ℓ	NO_3-N in feed mg N/ℓ	C/N	N loading μg N/d	N in effluent μgN/ℓ	N removal %
D-1	0	10	0	150	137	8.8
D-2	20	10	2	150	127	15.7
D-3	50	10	5	150	90	40.0
D-4	100	10	10	150	16	89.2
D-5	150	10	15	150	0	100

The similar tests were conducted using actual sewage as a feed. The test arrangements are shown schematically in Fig. 9. The total nitrogen in the settled seqage used as a feed was 18-25 mg/ℓ, and TOC was 35-60 mg/ℓ. The majority of the nitrogen in the settled sewage was accounted for by ammonia nitrogen. The application rate was set in three stages: 50, 100, and 200 mm/day. For 3 days during the test period, distilled water was fed instead of settled sewage to determine the detention time by observing the changes in the nitrogen concentration in the effluent. The detention time was 6, 3, and 1.5 days for 50, 100, and 200 mm/day respectively. Fig. 10 shows the changes in the total nitrogen (TN) in the feed and effluent during the test period. The curve for the effluent is shifted by the detention time for direct comparison in TN between the feed and effluent.

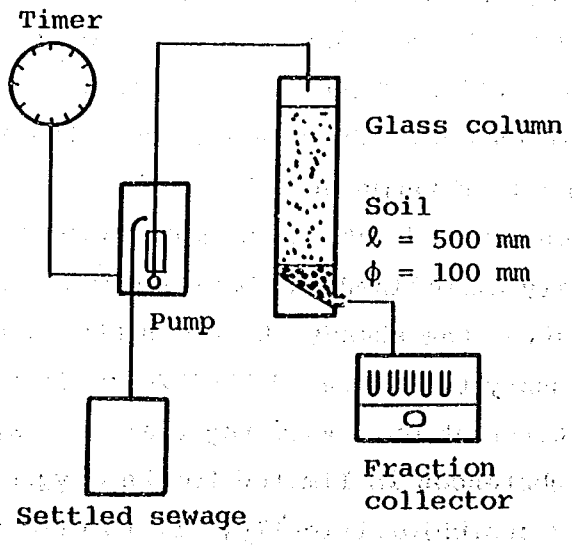


Fig. 9 Block diagram of experimental set-up

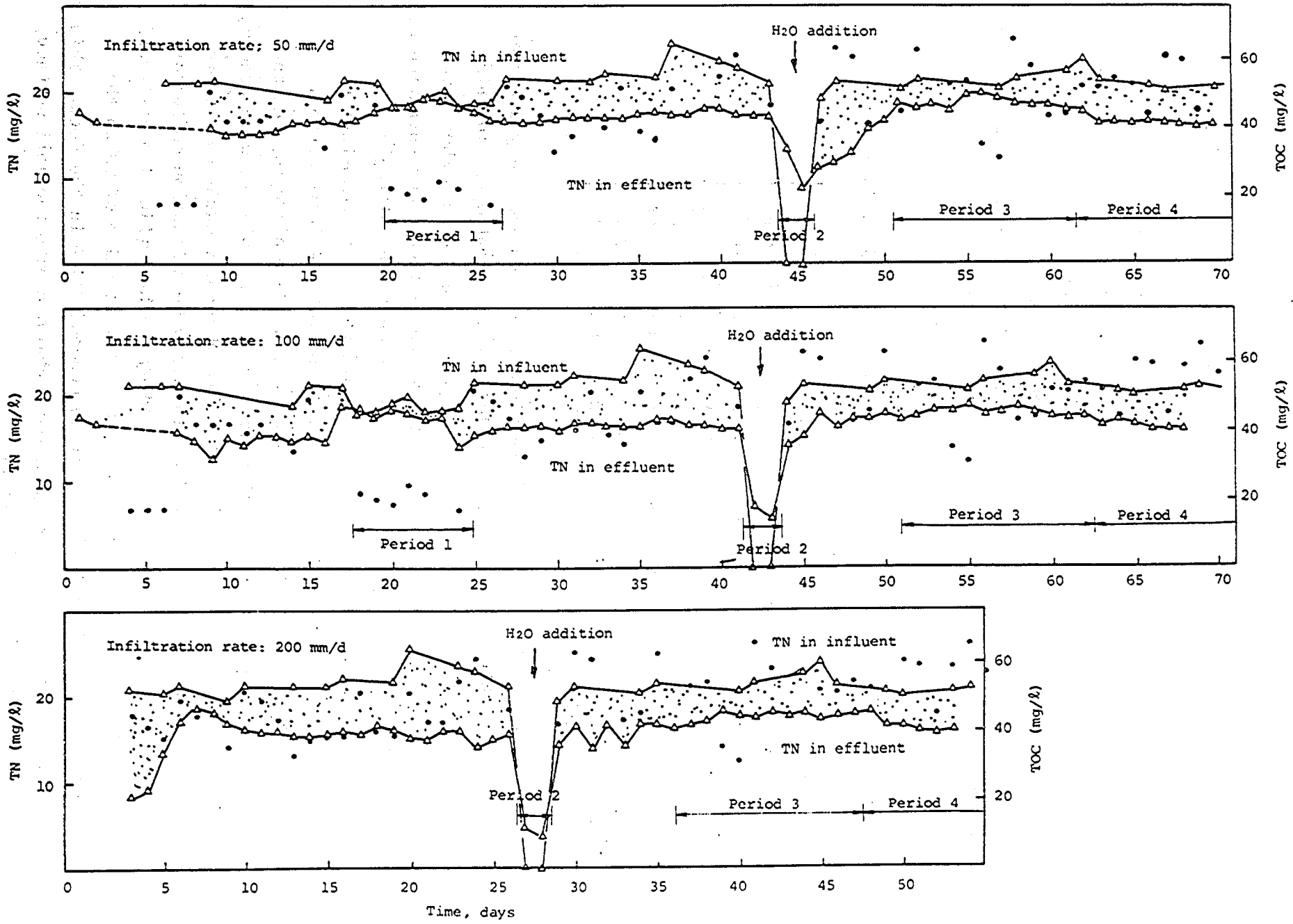


Fig. 10 Nitrogen concentrations in influent and effluent, Laboratory column experiment with sandy loam

In Fig. 10, the feed in Period 1 was the settled sewage and secondary effluent mixed at a ratio of 1-to-1, and that in Period 2, the distilled water. In Period 3, porous cinder granules were spread over the soil surface for uniform infiltration in order to reduce clogging. In Period 4, soil was spread over the porous granules to depress the infiltration of air. In periods other than Periods 1-4, the nitrogen balance was taken and listed in Table 4. The C/N ratio of settled sewage was 2-to-3, and the nitrogen removal in Table 4 is consistent with the results shown in Table 3 in which glucose was used as a carbon source.

Table 4 Nitrogen removal in case of settled sewage application

Application rate	mm/d	200	100	100
Retention time	day	1.5	3.0	6.0
Influent	mg/ℓ	21.0	21.0	21.0
	mg/d	33.0	16.5	8.25
Effluent	mg/ℓ	15.6	16.5	17.4
	mg/d	22.5	13.0	6.84
Removal	mg/d	10.5	3.5	1.41
	%	31.8	21.2	17.1

In Period 1 when a mixture of settled sewage and secondary effluent was used as a feed, the C/N ratio in the feed was about one. In this case little or no nitrogen removal took place. In Period 3 when porous granules were spread over the soil surface, the nitrogen concentration in the effluent was a little higher, but in Period 4, the nitrogen concentration decreased again. This implies that it is important to limit the supply of oxygen from the air to promote denitrification.

The nitrogen in the effluent was mostly nitrate nitrogen, but the nitrogen concentration in the effluent varied over a wide range depending on the application pattern. Fig. 11 shows TN (nearly equal to nitrate nitrogen) in the effluent sampled sequentially by a fraction collector. There is a marked periodicity corresponding to the application patterns. At an application rate of 200 mm/day, the maximum value of TN in the effluent became much higher than the TN concentration in the feed. This suggests that the ammonia nitrogen, once adsorbed onto the soil surface, will run off after being nitrified.

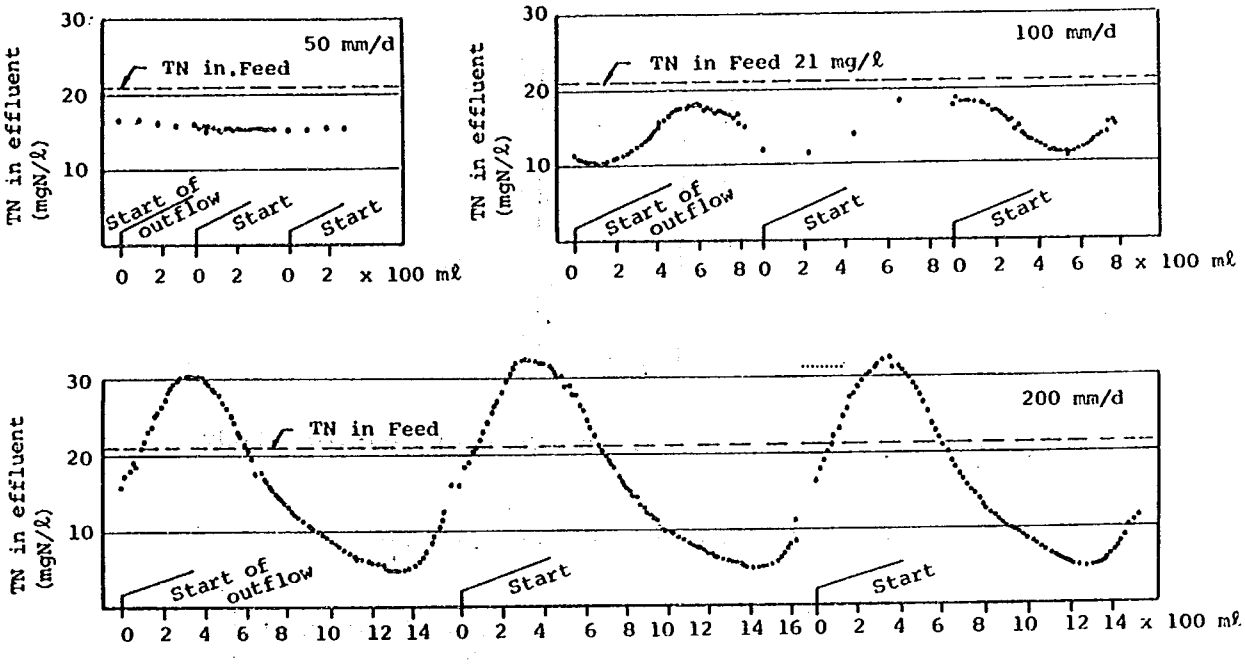


Fig. 11 Variation of TN in effluent

To achieve high nitrogen removal efficiency at a low C/N ratio, tests were conducted using the device shown in Fig. 12, in which the supply of oxygen from the air was limited by making the flow in the deeper layer a saturated flow. The column was filled with sand or soil, and the settled sewage or a mixtrue of settled sewage and secondary effluent (1:1 mixture) was used as a feed. Table 5 lists the experimental conditions. The application rate was fixed at 100 mm/day for every run. The cross-sectional area of the column was 56 cm^2 for sand, and 242 cm^2 for soil. The total nitrogen of the settled sewage was 20-30 mg/ℓ, and most of the nitrogen was ammonia nitrogen. TOC was 40-90 mg/ℓ and the C/N ratio averaged about 3.

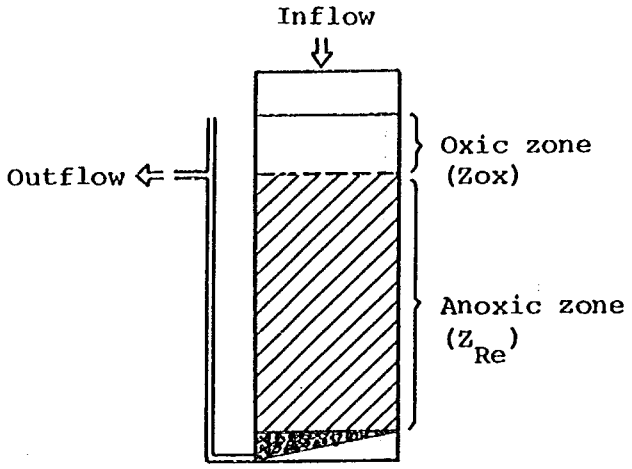


Fig. 12 Experimental setup for denitrification study

Table 5 Experimental conditions

Run	Media	Feed	Column depth (cm)	
			Zox	Zre
Q-1	Sand	Settled sewage	10	30
-2	Sand	Settled sewage	15	25
-3	Sand	Settled sewage	20	20
-4	Sand	Settled sewage	25	15
-5	Sand	Settled sewage	35	10
P-1	Soil	Settled sewage	15.5	20
-2	Soil	Settled sewage	25.5	10
-3	Soil	Mixed sewage	15.5	20
-4	Soil	Mixed sewage	20.5	15
-5	Soil	Mixed sewage	25.5	10

Fig. 13 shows the nitrogen balance determined from 100 days of experiment. The amount of nitrogen accumulated in the soil is neglected. When settled sewage was used, the nitrogen removal was 60-70% under favorable conditions. Installation of the saturated layer may have improved efficiency. When mixed sewage (C/N ratio = 1-to-2) was used, the nitrogen removal remained as low as 25-45%, probably due to deficiency in carbon supply. It may be concluded that when organics in the feed are the only available source of carbon, the condition of complete nitrification in the soil causes deficiency in carbon for denitrification.

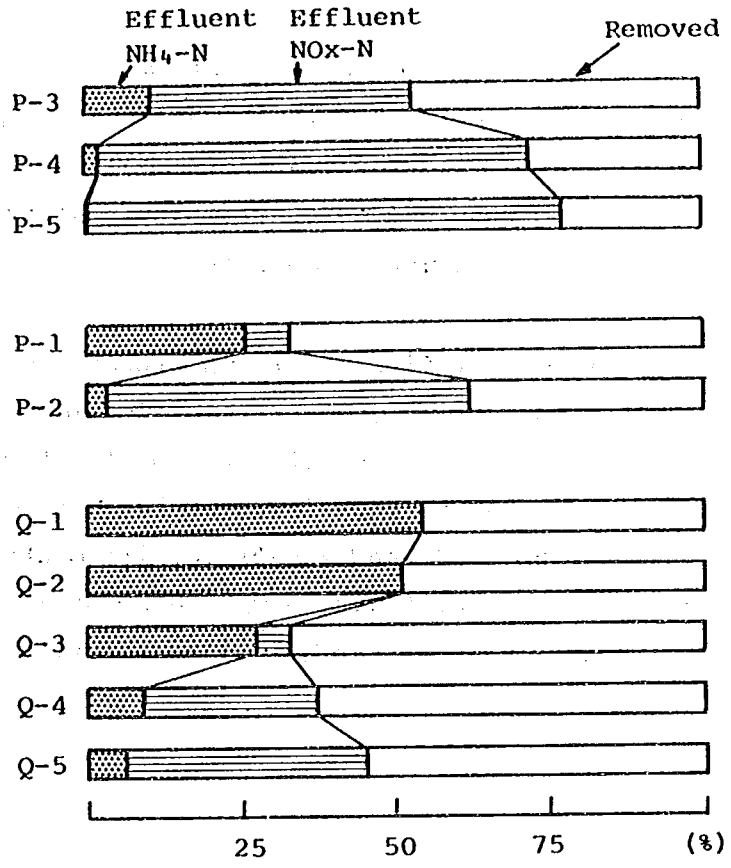


Fig. 13 Nitrogen balance during 100 days of experiment

Where already well-nitrified wastewater, such as well-treated secondary effluent, is to be subject to land treatment process, it seems advantageous to apply it in the form of saturated flow, although high nitrogen removal would not be expected since the C/N ration is usually very low.

5. SUMMARY

The land treatment process using mainly the subsurface trench system was studied.

(1) When a subsurface trench system is used to apply clean water into sandy loam, an infiltration rate of 50-150 mm/day can be expected. In the case of wastewater application, however, the clogging limits the filtration rate. The critical BOD loading in this respect will be 0.2-0.3 $g/m^2/day$.

(2) The phosphorus adsorption capacity of soil can be well approximated by Langmuir ultimate-capacity constant, when the phosphorus concentration in the applied wastewater is about 5 mg/ℓ. There is a strong correlation between Langmuir ultimate-capacity constant and the fluoride-extractable aluminum content in soil.

(3) To achieve complete denitrification in unsaturated percolation, the C/N ratio of the feed must be greater than 10. The provision of a saturated percolation layer under the unsaturated layer increases nitrogen removal. Nitrogen removal was 60-70% for settled sewage with a C/N ratio of about 3, and 25-45% for 1:1 mixture of settled sewage and secondary effluent with a C/N ratio of 1-to-2. When the secondary effluent is used as feed, little denitrification is expected.

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SESSION 3

BIOLOGICAL WASTEWATER TREATMENT

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EXPERIENCE OF PURE OXYGEN ACTIVATED SLUDGE TREATMENT IN KYOTO

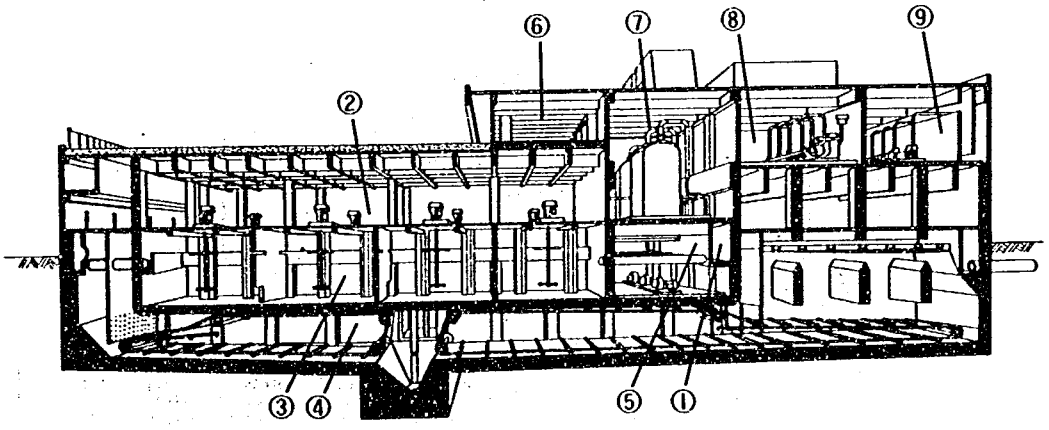
—JOINT TREATMENT OF MUNICIPAL AND DYEING WASTES—

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1. Preface

At the Kisshoin wastewater treatment plant in Kyoto City, the covered pure oxygen activated sludge treatment facilities with a capacity of 40,000 m^3/d (20,000 m^3/d X 2 lines) were constructed in 1977 in order to treat effectively the municipal wastewater containing dyeing wastes from the textile dyeing industry which is one of the traditional industries in Kyoto. Since then, the technical research for the pure oxygen system has been continuously carried out with the use of one of the two lines. The experimental research for the pure oxygen system has been conducted by the Kyoto City Office in cooperation with the Japan Sewage Works Agency to study the characteristics of treatment and the economics of operation. A part of the research results was reported at the 12th Essen Congress in 1979.

This report describes the results of studies on the performance of treating influent including wastewater from textile dyeing factories and on the power saving for the covered pure oxygen system from among many results of technical research enforced since the start of operation.



(1) Influent Conduit
(2) Aerator Room
(3) Aeration Tank
(4) Final Sedimentation Tank
(5) Sludge Pump Room
(6) Electrical Room
(7) Oxygen Generator
(8) Air Compressor
(9) Ventilation Fan Room

Fig. 1 Cross-Sectional View of Oxygen Plant

Table 1. Design Specification for Oxygen Plant

Design Capacity		40,000 m³/d (20,000 m³/d X 2 Lines)
Planned Water Quality		Influent / Primary Effluent / Final Effluent BOD (mg/ℓ): 260 / 210 / 11.3 SS (mg/ℓ): 200 / 130 / 26
Aeration Tank	Type	Covered 4-Stage Surface Aerator
	Dimension	Width Length Depth 10.4m X 11.0m X 5.14m
	Effective Capacity	2,350 m³/Line
	Number of Stages	4
	Number of Tanks	2
	Aerator	Surface Aerator with Bottom Impeller
	Type of Aerator	45° Pitched Turbine Blades
	Blade Impeller	32° Pitched Turbine Blades
	MLSS	4,000 mg/ℓ
	Return Sludge	15,000 mg/ℓ
	BOD-SS Loading	0.45 kg/kg MLSS·d
Final Sedimentation Tank	Type	Rectangular
	Dimension	Width Length Depth 10.0m X 81.85m X 3.5m
	Effective Capacity	2,865 m³/Line
	Number of Tanks	2
	Overflow Rate	24 m³/m²·d
	Sedimentation Period	3.4 hrs
	Sludge Collector	Chain Flight Type
Oxygen Generation Facilities	Type	Vertical Cylindrical 4-Bed PSA
	Dimension	Diameter Height 2.6m X 3.98m X 4-Bed
	Capacity	15 tons O_2/d
	Oxygen Purity	90%
	(Accessories) Liquid Oxygen Storage Facilities	15 tons O_2/d

Table 2. Operational Condition for Experiment

Phase	Experimental Period	Types of Influent (Mixing Rate of Influent B %)	Inflow Quantity (m³/d)	Aeration Period (Q base hrs.)	BOD Capacity Loading (kg/m³·d)	BOD-SS Loading (kg/kg sed)	Influent BOD (mg/ℓ)	MLSS (mg/ℓ)
I	Nov. 1~Nov. 30, 1977	A (27)	19,150	2.95	1.16	0.33	142	2,550
II	Dec. 14~Dec. 29, 1977	AB (50)	19,670	2.87	1.36	0.30	163	4,550
III	Feb. 1~Mar. 3, 1978	B (100)	17,550	3.21	1.74	0.33	233	5,310
IV	Mar. 23~Apr. 22, 1978	A (27)	30,120	1.87	2.00	0.41	156	4,920
V	May 17~June 17, 1978	A (27)	39,730	1.42	1.71	0.51	101	3,350
VI (A)	June 19~July 8, 1978	AB (>80)	21,350	2.64	0.91	0.22	100	4,100
VI (B)	July 12~Aug. 26, 1978	B (100)	21,120	2.67	1.40	0.33	156	4,290
VII	Sept. 5~Sept. 29, 1978	AB (40)	26,500	2.13	1.00	0.29	89	3,460
VIII	Oct. 2~Oct. 31, 1978	AB (50)	27,180	2.07	1.15	0.28	99	4,080
IX	Nov. 2~Nov. 30, 1978	AB (60)	26,230	2.15	1.23	0.36	110	3,420
X	Mar. 1~Apr. 7, 1979	A (27)	25,720	2.19	1.32	0.49	121	2,680
XI	Apr. 9~May 19, 1979	A (27)	25,420	2.22	1.21	0.57	111	2,100
XII	May 22~July 13, 1979	A (27)	25,170	2.24	1.06	0.71	101	1,530

2. Influent Quality and Treatment

At the Kisshoin plant, there are the primary sedimentation tanks for two lines A and B as shown in Fig. 2. Influent of A Line mainly contains domestic wastewater, and influent of B Line contains a large quantity of textile dyeing wastes, for which it is difficult to treat biologically. The flow rate of A and B Lines is about 73 : 27 on average. This plant has a conventional diffused air system with a capacity of 85,000 m^3/d and the pure oxygen system with a capacity of 40,000 m^3/d.

The technical research for the pure oxygen system has been carried out mainly with the use of one of the two lines in the system. For oxygen facilities, primary effluent of A-line (influent A) and primary effluent of B-line (influent B) can be mixed at various ratio (influent AB). But even in influent A, 27 % of influent B is contained. (Fig. 2)

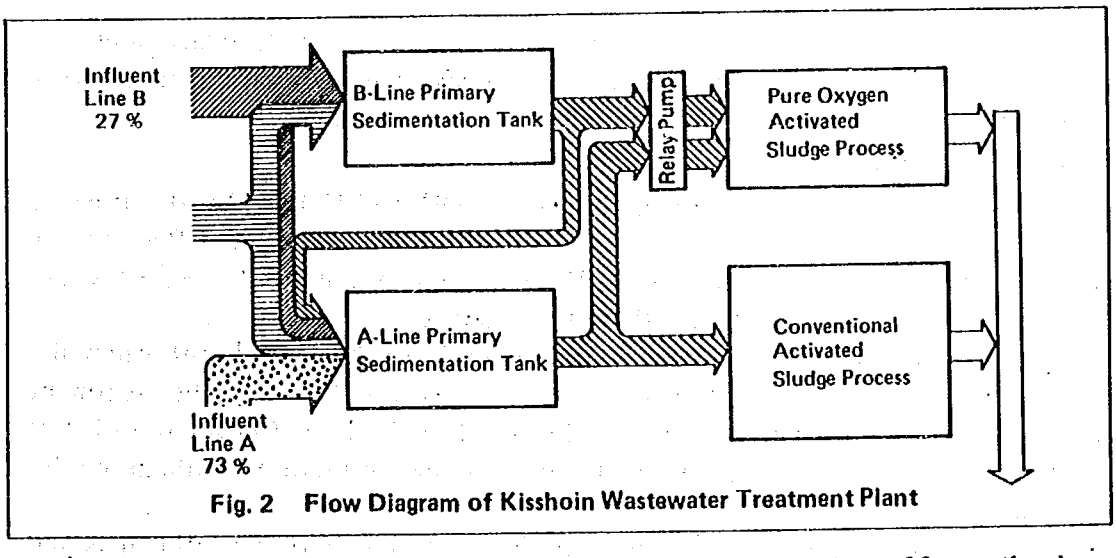


Fig. 2 Flow Diagram of Kisshoin Wastewater Treatment Plant

The research for treatment functions was carried out for about 20 months since November 1977 as shown in Table 2, by changing influent quality, flow rate and operational conditions. Based on the data of various experiments on three types of influent B, AB and A, this section discusses the effect of influent substrates including dyeing wastes on the performance of pure oxygen system.

2.1 Influent Quality

In the recent years, the quality of influent in B Line has considerably improved as a result of the industrial recession and the effective administrative instructions for industrial pretreatment plants. But it still contains hardly biodegradable substrates. Both the quantity and quality of dyeing wastes vary greatly because of the seasonal differences in the type of textiles, chemicals used and amount of products.

(1) Water Temperature

The temperature of influent B greatly varies because of the thermal wastes from dyeing factories. Shown in Fig. 3 is an example for the hourly variation of temperature of influent B on working day and off-day at dyeing factories. Such a remarkable variation of temperature may cause bad effects on the sludge activity especially in winter.

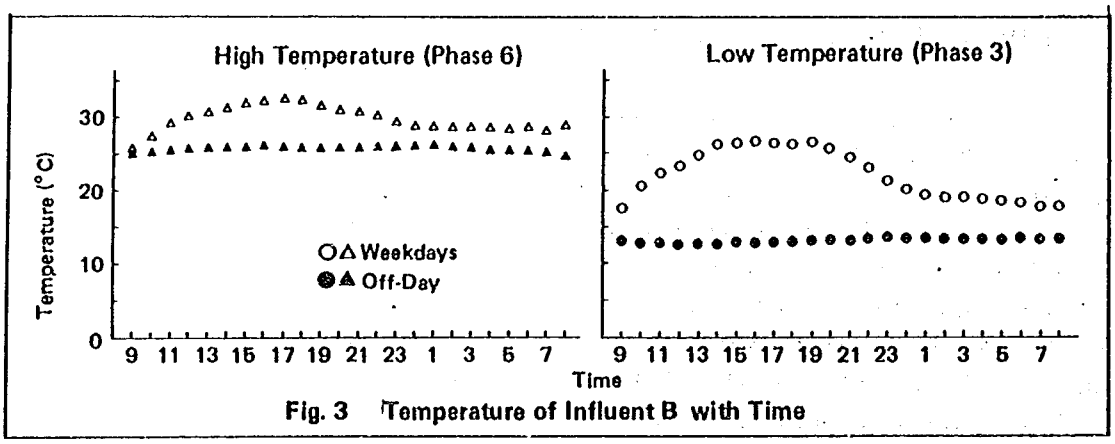


Fig. 3 Temperature of Influent B with Time

(2) Colour

Influent B is coloured by dyes remaining in dyeing wastes. In many cases, the colour changes from purplish red to deep brown-gray. The colouration still remains on effluent and causes worsening of transparency and appearance.

(3) BOD and COD_{Cr}

Fig. 4 indicates the relationship between BOD and COD_{Cr} by the type of influent during the research period. Both BOD and COD_{Cr} of influent B are higher than those of influent A. Although BOD of influent A and AB are almost same, COD_{Cr} of influent AB is higher than that of influent A.

It is reported that the biological treatment is easily carried out when the BOD/COD_{Cr} ratio is over 0.6 but when the ratio is less than 0.6, the treatment becomes hard. According to the figure, almost all of the BOD/COD_{Cr} ratio is less than 0.6 even for influent A. This fact indicates that influent B contains hardly biodegradable substrates.

In Fig. 4, the group of influent AB does not lay between influent A and

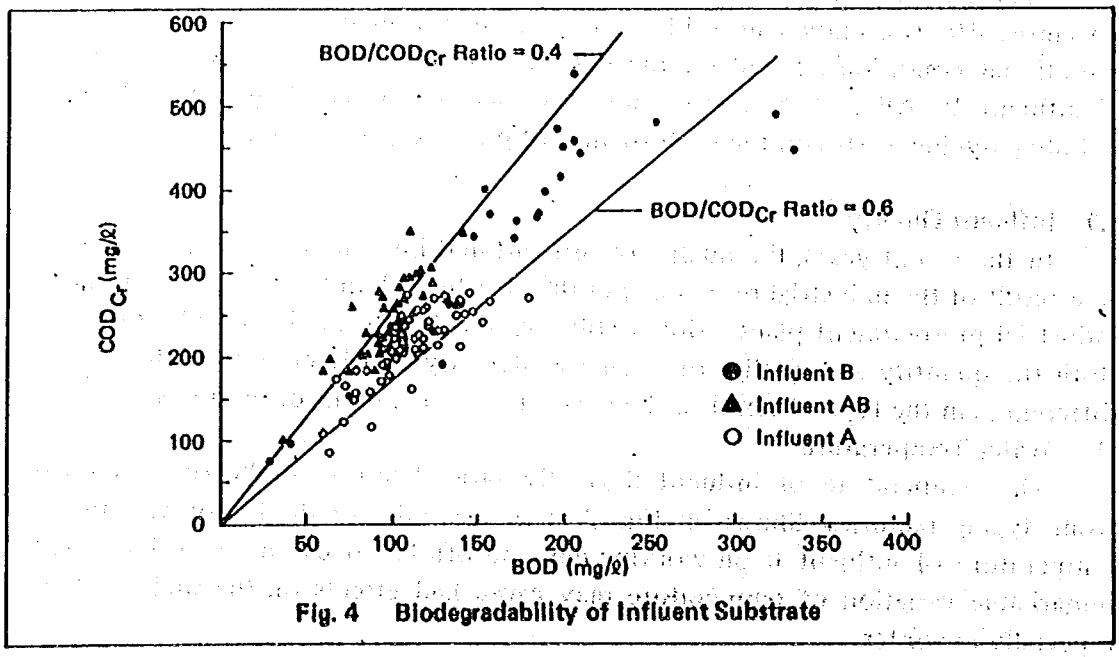


Fig. 4 Biodegradability of Influent Substrate

influent B because of the seasonal variation of substrates: At dyeing factories in this district, dyeing is usually made for natural textiles in winter and synthetic textiles in summer. Because wastewater from synthetic textile dyeing process mainly contains hardly biodegradable synthetic paste and surface-active agents, wastewater has low BOD and comparatively high COD_{Cr} in summer.

(4) Total Solids and Dissolved Solids

Shown in Fig. 5 is the relationship between total solids (TS) and dissolved solids (DS) by the type of influent. The ratio of DS and TS is on a straight line with the slope of about 0.9, and the concentration differs greatly, according to the mixing ratio of influent B. For instance, TS is 300 ~ 700 mg/ℓ in influent A and 900 ~ 1,300 mg/ℓ in influent B, while influent AB shows the medium value. It indicates the influence of dyeing wastes.

The volatile solids of influent B account for about 20 % of TS, and the remainders are mainly inorganic dissolved solids which are caused by inorganic chemicals such as sodium hydroxide, sodium sulphate and sodium carbonate in the dyeing process. The iodine consumption of influent B shows a high value of about 500 mg/ℓ which is about 10 times that of influent A. It is mainly caused by the discharge of the reducing agent such as sodium sulphite and sodium hydrosulphite used in the dyeing process.

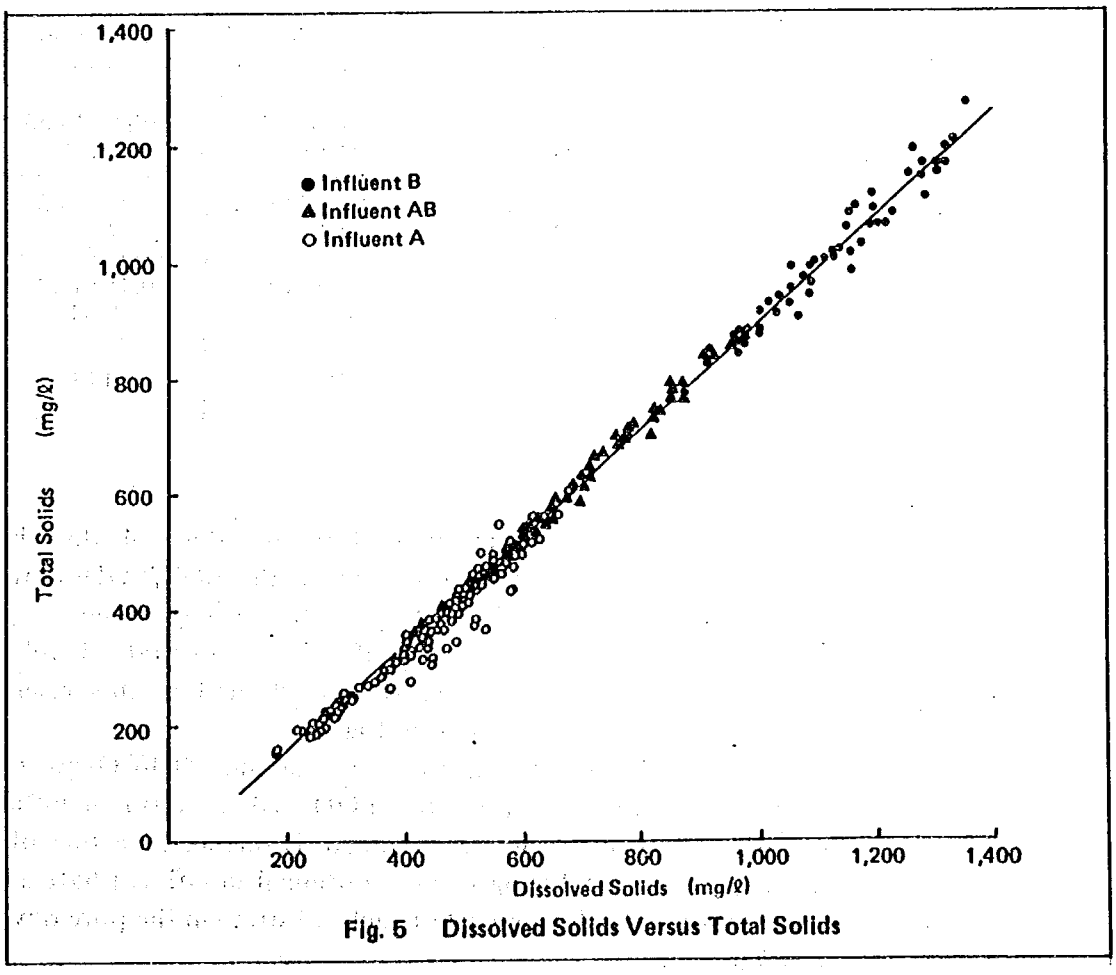


Fig. 5 Dissolved Solids Versus Total Solids

(5) Nitrogen and Phosphorus

As shown in Table 3, nitrogens of influent B are 62 mg/ℓ for total nitrogen and 41 mg/ℓ for ammonium nitrogen, both on the average value. It is remarkably larger than that of influent A. It is caused by urea and ammonium sulphate that are used as hydrotropic reagent for printing paste.

The ratio of BOD : N : P in influent is important for the biological treatment. Although the appropriate ratio is 100 : 5 : 1 for the activated sludge treatment, the ratio for influent B is 100 : 33 : 1 (as Total-N, Total-P). Nitrogen is in a large excess, but phosphorus is slightly scarce because some of total phosphorus are not always possible to be uptaken for cell growth.

Table 3. Characteristics of Influent (Range / Average)

		Influent B	Influent AB	Influent A
Total Nitrogen	(mg/ℓ)	53.40~71.49 62.28	22.62~56.38 33.57	18.80~35.82 27.25
Ammonium Nitrogen	(mg/ℓ)	37.79~50.32 41.29	9.05~37.60 17.67	8.93~20.47 13.39
Nitrite Nitrogen	(mg/ℓ)	0.02~0.14 0.08	0.01~0.27 0.08	0.01~0.23 0.08
Nitrate Nitrogen	(mg/ℓ)	0.17~0.42 0.28	0.15~0.52 0.36	0.08~1.09 0.39
Organic Nitrogen	(mg/ℓ)	14.97~25.56 20.62	13.10~19.61 16.06	10.15~19.53 13.32
Total Phosphorus	(mg/ℓ)	1.92~3.31 2.40	1.65~3.20 2.49	2.00~2.83 2.55
Ortho-phosphate	(mg/ℓ)	0.26~0.66 0.40	0.48~1.40 0.96	0.45~1.14 0.91
BOD	(mg/ℓ)	187	100	113

2.2 Effect on Treatment

Table 4 shows the experimental results by the type of influent in the whole period. Fig. 6 roughly indicates the relationship between the BOD/COD_{Mn} ratio of influent and the remaining rate of substrates with the average value in each experimental phase. Although the biodegradability of influent substrates is related to the BOD/COD_{Cr} ratio, the BOD/COD_{Mn} ratio was adopted in this reserach because COD_{Mn} and COD_{Cr} are closely related as in Fig. 7.

In Fig. 6, the remaining rate of substrates increases as the BOD/COD_{Mn} ratio of influent decreases. As mentioned before, the BOD/COD_{Mn} ratio of influent B, in which a large quantity of dyeing wastes is mixed, is smaller than that of influent A. As a result, it is clear that dyeing wastes contained in influent have a bad effect on the biological treatment. The removal of each substrate in the pure oxygen system is described as follows:

Table 4. Treatment Performance

			Influent		
			B	AB	A
Operational Condition	Inflow Quantity	(m^3/d)	19,600	26,700	32,600
	Recycle Sludge Ratio	(%)	47.8	35.9	37.6
	Aeration Period (Q base)	(hrs)	2.89	2.11	1.73
	BOD-SS Loading	(kg/kg MLSS·d)	0.34	0.32	0.55
	BOD-Capacity Loading	(kg/m^3·d)	1.54	1.11	1.27
	Overflow Rate	(m^3/m^2·d)	24	33	40
	Oxygen Consumption per BOD Removed	(kg O_2/kg BOD_R)	1.54	1.68	1.20
	MLSS	(mg/ℓ)	4,700	3,370	2,450
	MLVSS/MLSS	(%)	73.9	75.4	80.0
	Waste Sludge per BOD Removed	(kg SS/kg BOD_R)	0.34	0.46	0.54
	SRT	(days)	13.1	9.7	4.4
Influent Characteristics	Total Solids	(mg/ℓ)	1,100	725	511
	BOD	(mg/ℓ)	187	100	113
	Soluble BOD	(mg/ℓ)	110	58	61
	SS	(mg/ℓ)	103	72	72
	COD_{Mn}	(mg/ℓ)	163	90	75
	COD_{Cr}	(mg/ℓ)	407	289	221
	BOD/COD_{Mn} Ratio		1.15	1.10	1.51
	BOD/COD_{Cr} Ratio		0.46	0.40	0.51
Effluent Characteristics	BOD	(mg/ℓ)	18.9	18.1 (13.2)*	12.7
	Soluble BOD	(mg/ℓ)	10.6	6.5	6.6
	SS	(mg/ℓ)	21	16	10
	COD_{Mn}	(mg/ℓ)	70	41	29
	COD_{Cr}	(mg/ℓ)	142	86	67
Removal Rate	BOD	(%)	89.9	81.9 (86.8)*	88.7
	SS	(%)	79.6	77.8	86.1
	COD_{Mn}	(%)	57.0	54.4	61.3

* ATU-BOD

(1) BOD Removal

Shown in Fig. 8 is the effect of BOD-SS loading on effluent BOD by the type of influent. It is found that the operation with low BOD-SS loading is required for influent B in comparison with influent A. The critical BOD-SS loading for maintaining 20 mg/ℓ BOD for the effluent standard becomes 0.4, 0.6, 1.2 kg/kg SS·d for influent B, AB and A respectively from the regression line shown in the figure. Because plots are scattered, the normal operation shall be carried out with loading which is lower than the critical loading. It is necessary to treat influent B with less than a half of BOD-SS loading for influent A.

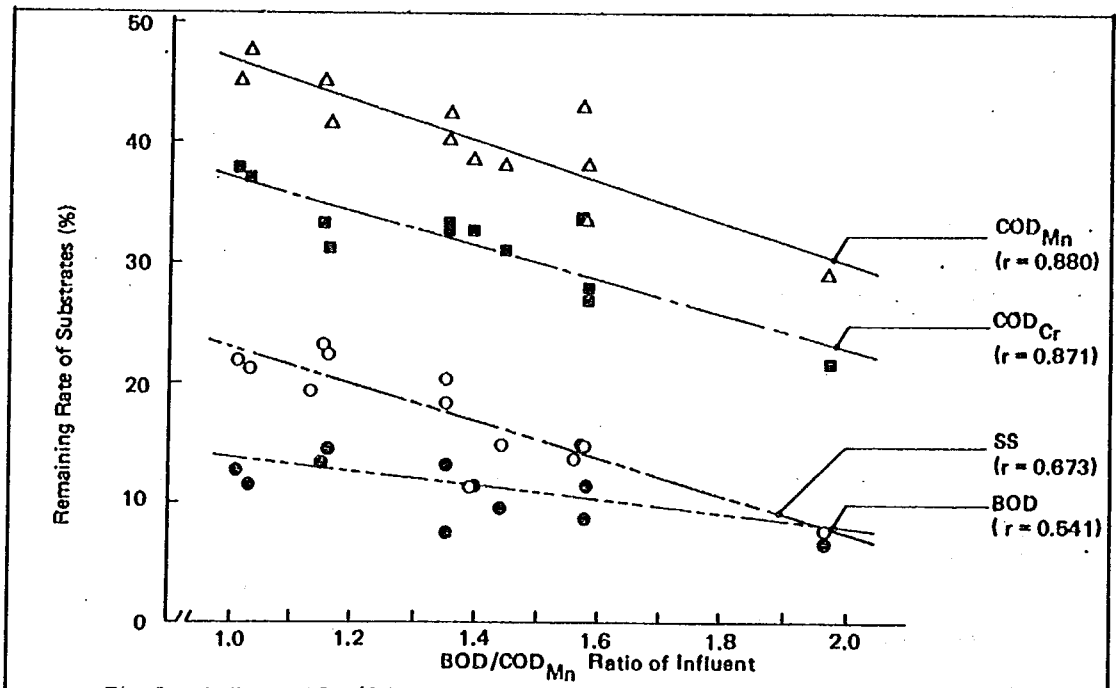


Fig. 6 Influent BOD/COD$_{Mn}$ Ratio versus Remaining Rate of Substrates in Effluent

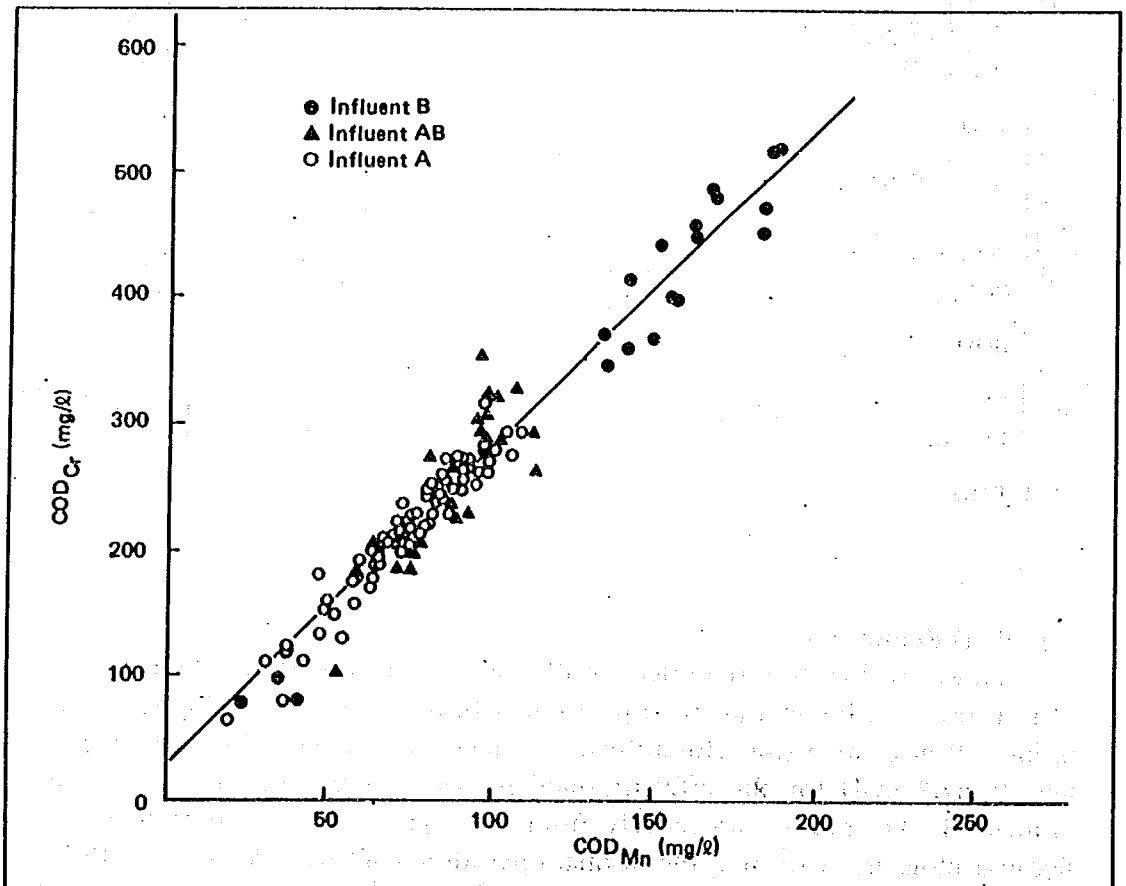


Fig. 7 Relationship between COD$_{Cr}$ and COD$_{Mn}$ of Influent

To evaluate the influence of BOD-capacity loading on effluent BOD, the effect of BOD-SS loading in the following experimental operation must be taken into consideration. In Fig. 8, the high BOD-SS loading of influent A was obtained by decreasing MLSS compulsorily because of the low concentration of influent BOD. So, the effect of BOD-capacity loading on effluent BOD by the type of influent is indicated in Fig. 9 only for the phase with the average BOD-SS loading of less than 0.5 kg/kg SS·d. Even if the BOD-capacity loading of influent A is more than 2 kg/m^3·d, the effluent BOD is less than 20 mg/ℓ. For influent B, the effluent BOD is over 20 mg/ℓ in some cases with the BOD-capacity loading of 1.0 ~ 1.5 kg/m^3·d. Accordingly, the mixing of dyeing wastes remarkably affects the BOD removal.

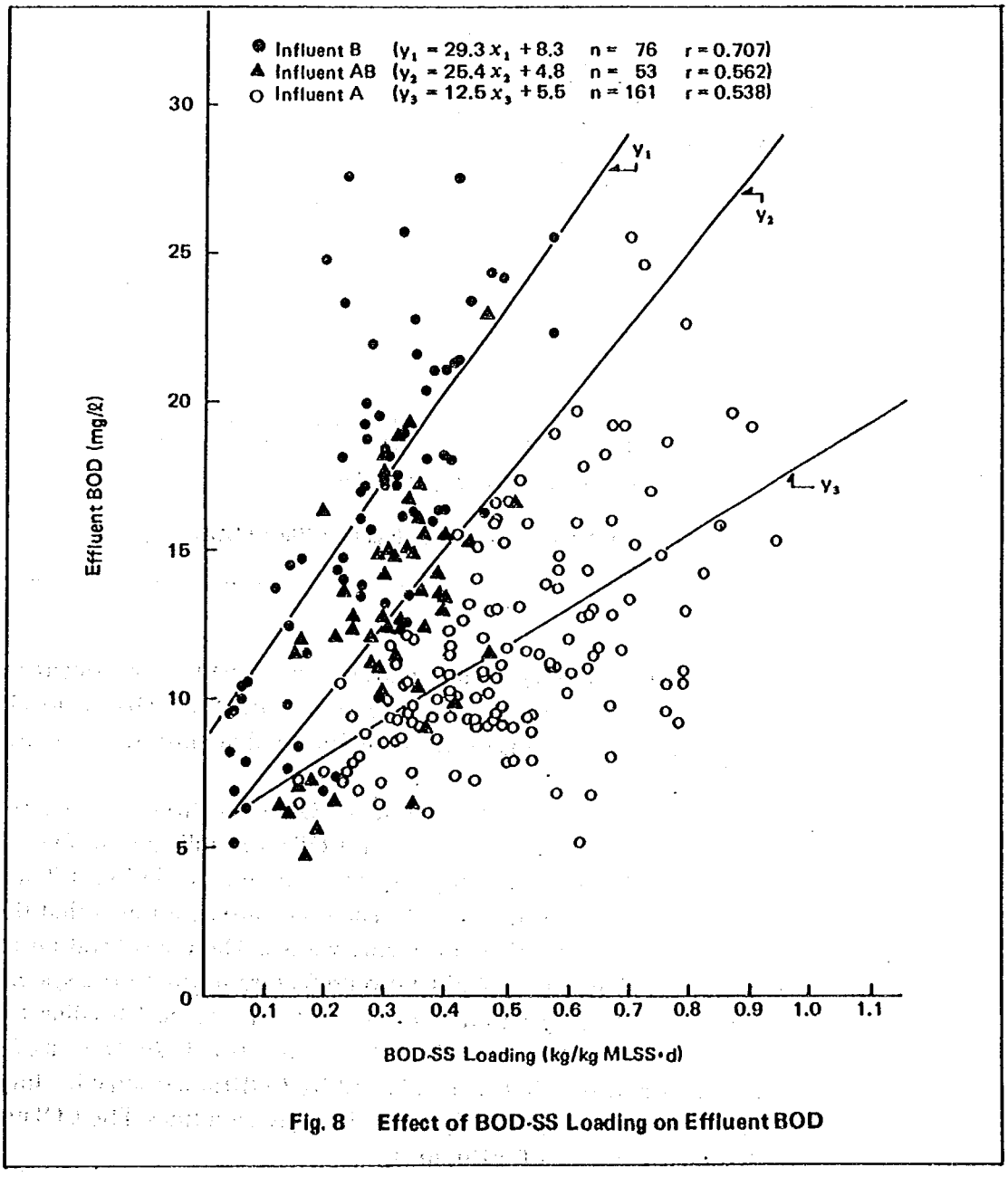


Fig. 8 Effect of BOD-SS Loading on Effluent BOD

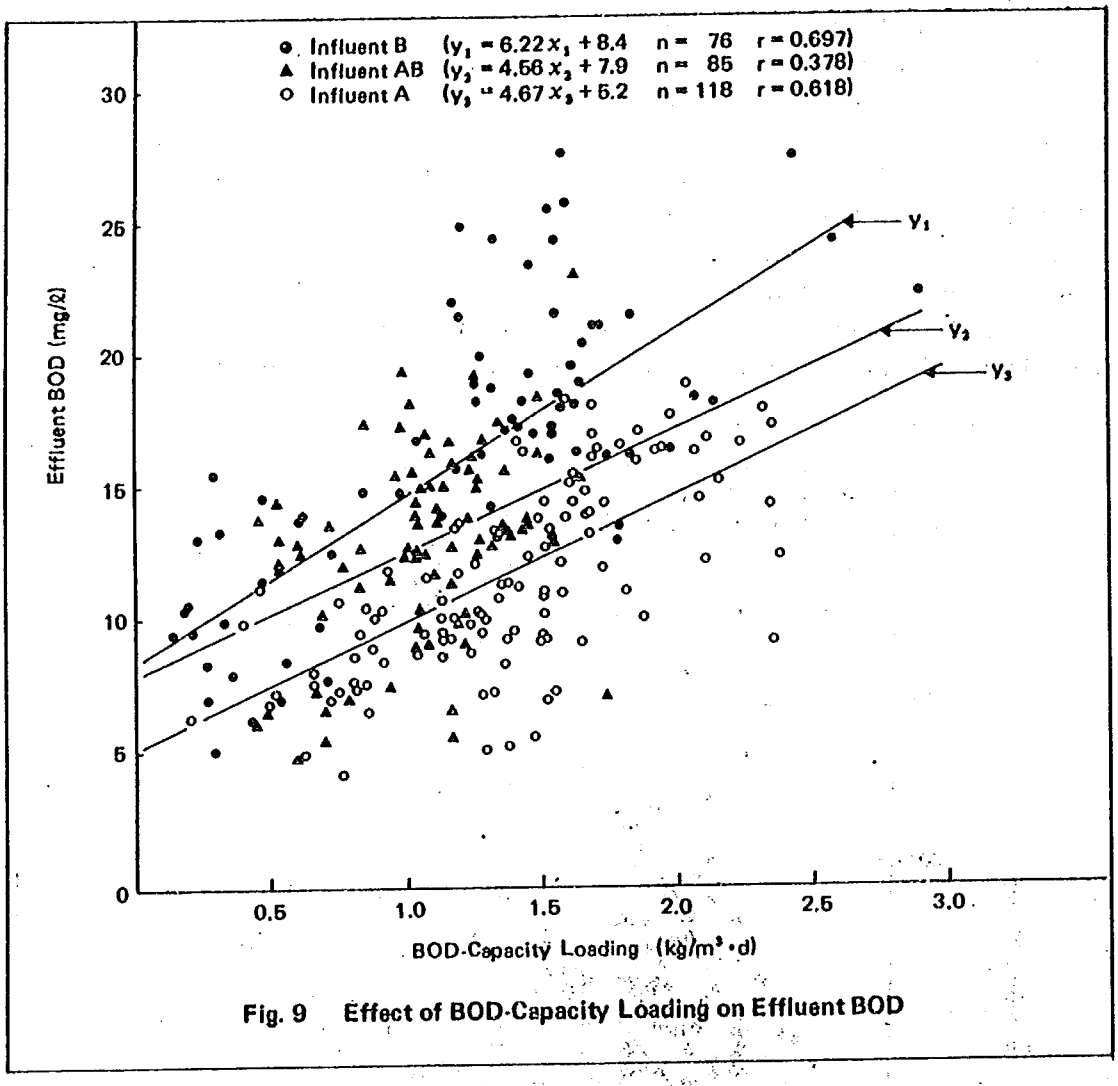


Fig. 9 Effect of BOD-Capacity Loading on Effluent BOD

(2) COD_{Mn} Removal

As COD_{Mn} includes hardly biodegradable matters, the COD_{Mn} component in influent has a big effect on the removal. In comparison with the BOD removal, the difficulty of COD_{Mn} removal is suggested as in Fig. 6 that indicates a sharp slope of regression line for COD_{Mn}.

The effect of COD_{Mn}-SS loading on the effluent COD_{Mn} is shown in Fig. 10, and the effect of COD_{Mn}-capacity loading on effluent COD_{Mn} is indicated in Fig. 11 by the type of influent. In each figure, the slope of regression line for influent B and AB is sharp as compared with that for influent A. The sharp slope indicates that the biological treatment is difficult for COD_{Mn} in dyeing wastes. The concentration of effluent COD_{Mn} is set at less than 40 mg/ℓ in Kyoto Prefecture based on the system for regulating the total amount of pollutants in effluent. The critical loading for maintaining less than 40 mg/ℓ COD_{Mn} of effluent becomes 0.1, 0.26, 0.55 kg/kg ss·d for COD_{Mn}-SS loading and 0.6, 0.9, 1.4 kg/m³·d for COD_{Mn}-capacity loading, both for the influent B, AB and A respectively from the regression lines. The COD_{Mn} removal is difficult for the treatment of influent B.

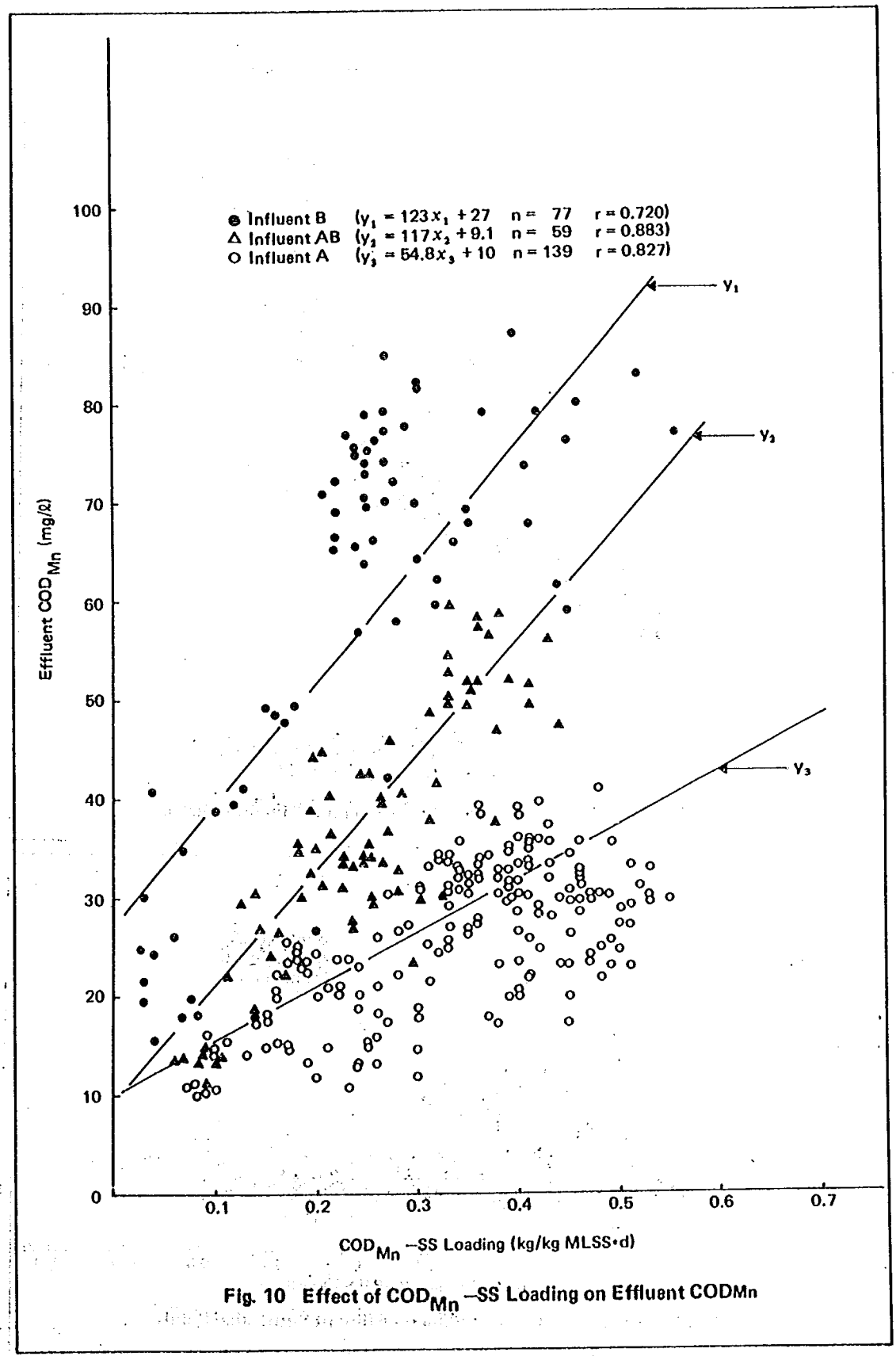


Fig. 10 Effect of COD_{Mn} –SS Loading on Effluent CODMn

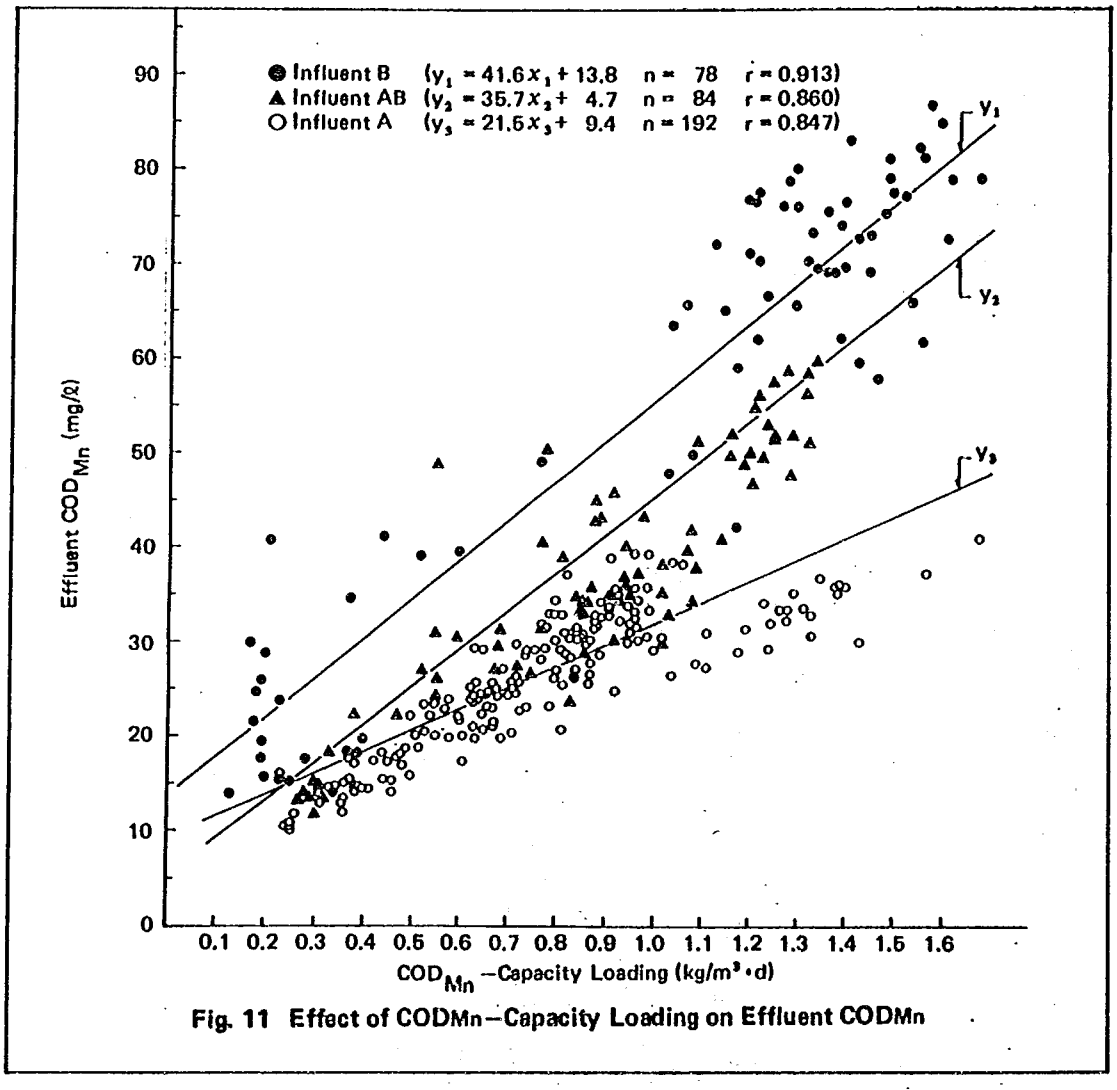


Fig. 11 Effect of CODMn–Capacity Loading on Effluent CODMn

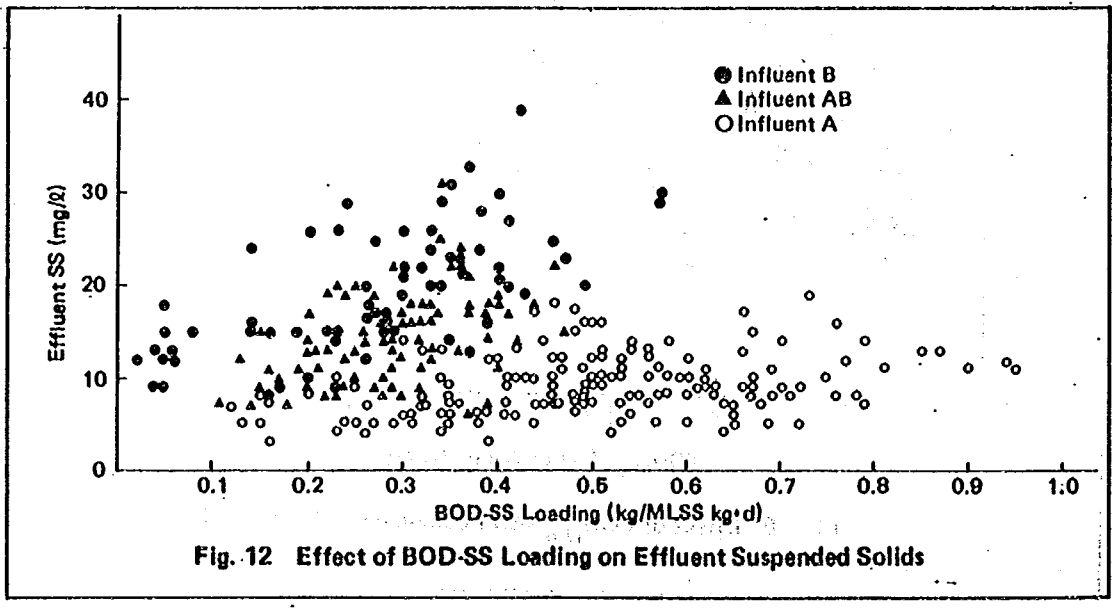


Fig. 12 Effect of BOD-SS Loading on Effluent Suspended Solids

(3) SS Removal

Fig. 12 shows the effect of BOD-SS loading on effluent suspended solids (SS) by the type of influent. For influent A, effluent SS does not increase to 20 mg/ℓ even in case of the high BOD-SS loading. The high mixing ratio of influent B makes effluent SS higher.

(4) N and P Removal

The removal of nitrogen and phosphorus is affected complicatedly by various factors such as the influent quality and the operational conditions, but the effect of those factors has not yet been probed. The average removal rates of each phase range 16.4 ~ 39.4 % for N and 48.2 ~ 81.0 % for P. The removal ratio of BOD : N : P is 100 : 3.2 ~ 10.1 : 0.9 ~ 1.4.

In the covered pure oxygen system, the CO_2 concentration in the mixed liquor is increased by the biological metabolism, and pH of mixed liquor comes down to around 6. It is reported that the nitrification is hard to occur due to low pH in comparison with the conventional air system. But sometimes the effluent contains 3 ~ 4 mg/ℓ of NO_3-N, and the effluent BOD increases more than 30 % as compared with ATU-BOD. It is found that the nitrification takes place to some extent under some operational conditions.

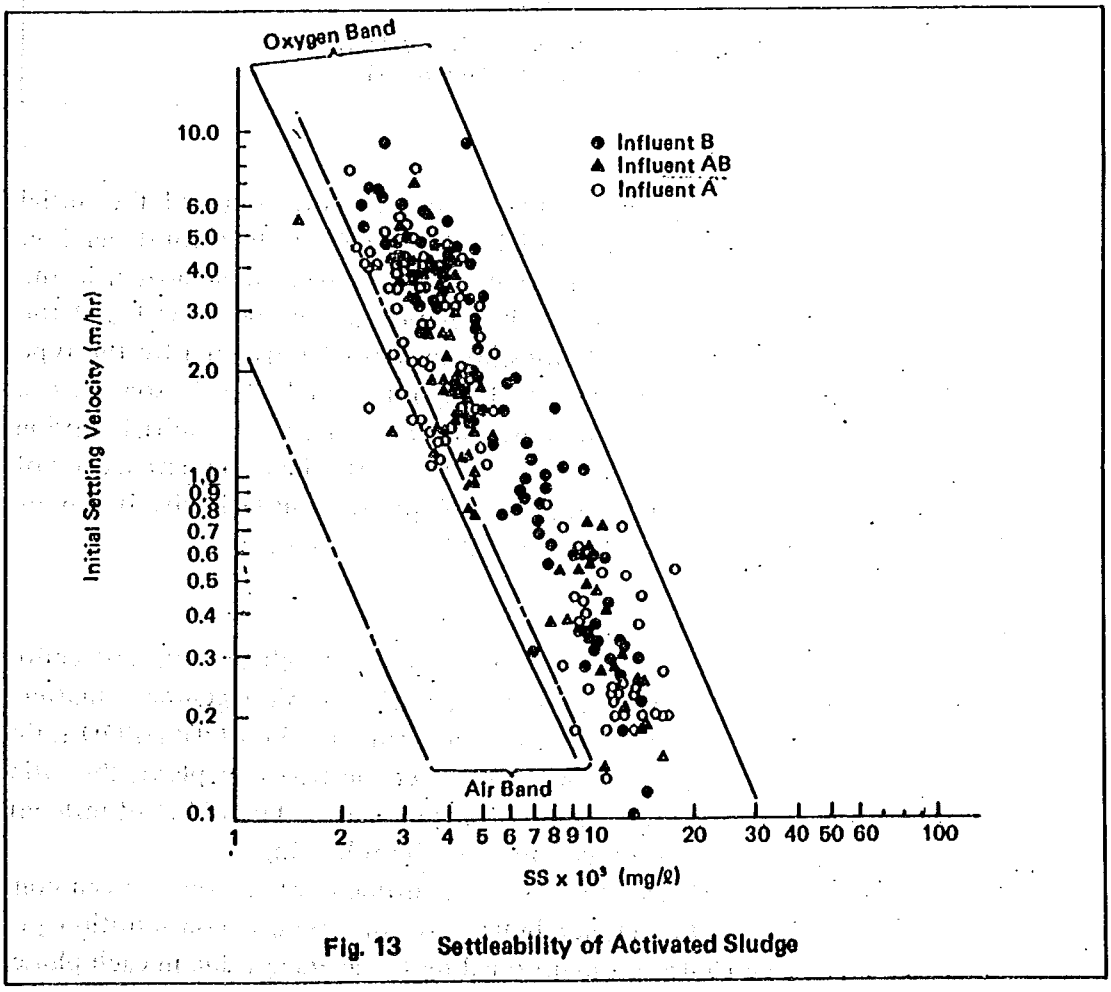


Fig. 13 Settleability of Activated Sludge

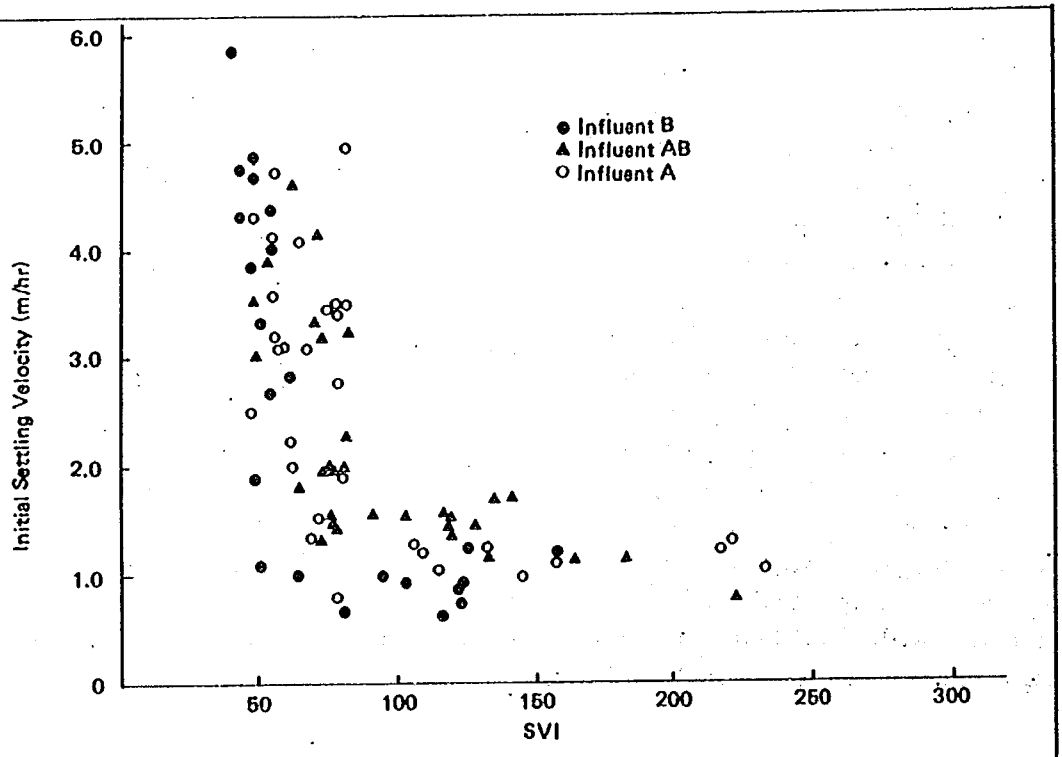


Fig. 14 SVI versus Initial Settling Velocity (MLSS 2830~7700 mg/ℓ)

(5) Activated Sludge

Fig. 13 indicates the relationship between SS concentration and the initial settling velocity of activated sludge. The initial settling velocity does not depend on the type of influent, and it is larger than that of the conventional system in the whole concentration range. Fig. 14 shows the relationship between SVI and the initial velocity of mixed liquor. The relationship does not be affected by the type of influent. The range of SVI is usually 50 ~ 100, and exceeds 200 in some cases. Also in the pure oxygen system, the bulking may be caused. The initial settling velocity is remarkably varied in the SVI range of 50 ~ 80. The dominant species of organisms in activated sludge for each influent are Aspidisca and Epistylis. It may be caused by the mixing of influent B at the rate of 27 % in influent A.

2.3 Effect on Oxygen Consumption

The oxygen consumption depends on the amount of cell growth and endogenous respiration of activated sludge, and the ratio of biodegradable matters (BOD) and the total organic matters (COD_{Cr}) in influent. The COD_{Cr}/BOD ratio is 1.5 ~ 2.0 for municipal wastewater in general. At the Kisshoin plant, the ratio is usually high, and exceeds 2.7 for infleunt B in some cases. The effect of influent substrates on the oxygen consumption is studied in the following:

Shown in Fig. 15 is the COD_{Cr}/BOD ratio of influent versus the oxygen consumption per BOD removed (kg O_2/kg BOD_R) or the oxygen consumption per COD_{Cr} removed (kg O_2/kg COD_{CrR}) mentioned by the average value in each phase.

The oxygen consumption per COD_{Cr} removed does not change even if the COD_{Cr}/BOD ratio increases. The oxygen consumption per BOD removed increases sharply as the COD_{Cr}/BOD ratio raises. The above two cases indicate that oxygen is consumed not only for the decomposition of BOD matters but also for the removal of COD matters in the aeration tank. The oxygen consumption increases in the treatment of influent B with the high COD_{Cr}/BOD ratio.

On the assumption that total oxygen are used for the biological oxidation, the oxygen consumption is studied according to the following W.W. Eckenfelder's equation:

$$O_2 \text{ (kg/d)} = a' \times (BOD_R \text{, kg/d}) + b' \times (MLVSS, \text{kg})$$

a' = BOD removed as energy for cell growth

b' = endogenous respiration rate

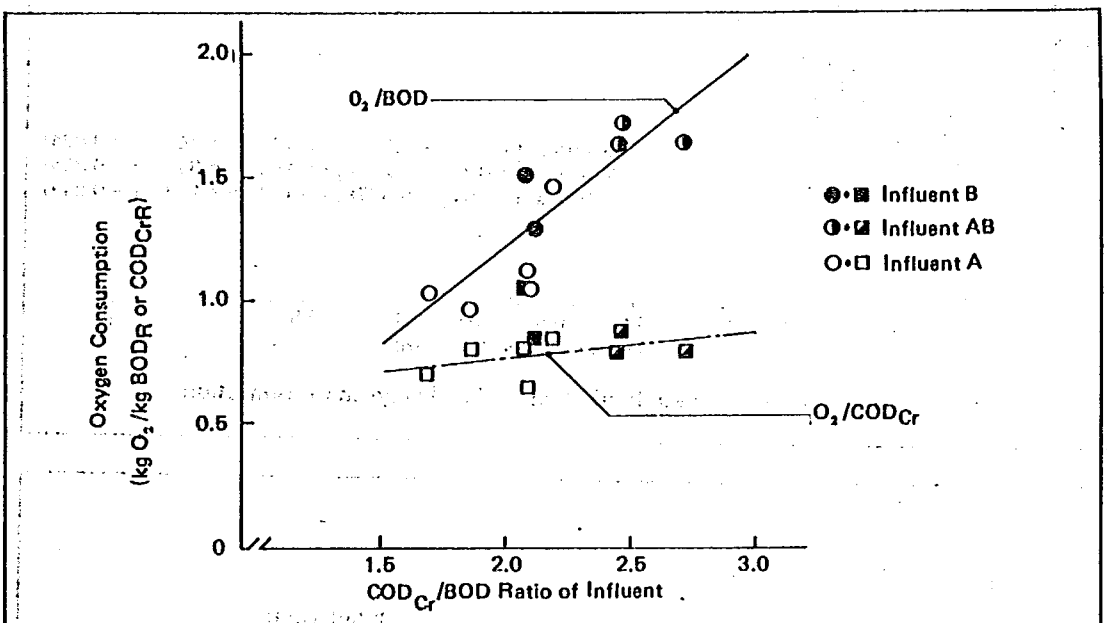


Fig. 15 Relationship between COD_{Cr}/BOD Ratio of Influent and Oxygen Consumption

According to the relationship between BOD removed per MLVSS (kg BOD_R /kg MLVSS·d) and oxygen consumption per MLVSS (kg O_2/kg MLVSS·d) shown in Fig. 16, a' is 1.11, 1.22, 0.88 and b' is 0.164, 0.143, 0.134 for influent B, AB and A respectively. The value of a' is 2 ~ 3 times that of a' for the conventional system which is generally said to be 0.4 ~ 0.6. On the other hand, b' can not be evaluated generally due to the effect of temperature.

Fig. 17 shows the relationship between the F/M ratio (kg BOD/kg MLVSS·d) and the oxygen consumption per BOD removed (kg O_2/kg BOD_R). The oxygen consumption decreases with the increase of the F/M ratio because the endogenous respiration ratio becomes lower. At the similar F/M ratio, the oxygen consumption for influent B is larger than that for influent A.

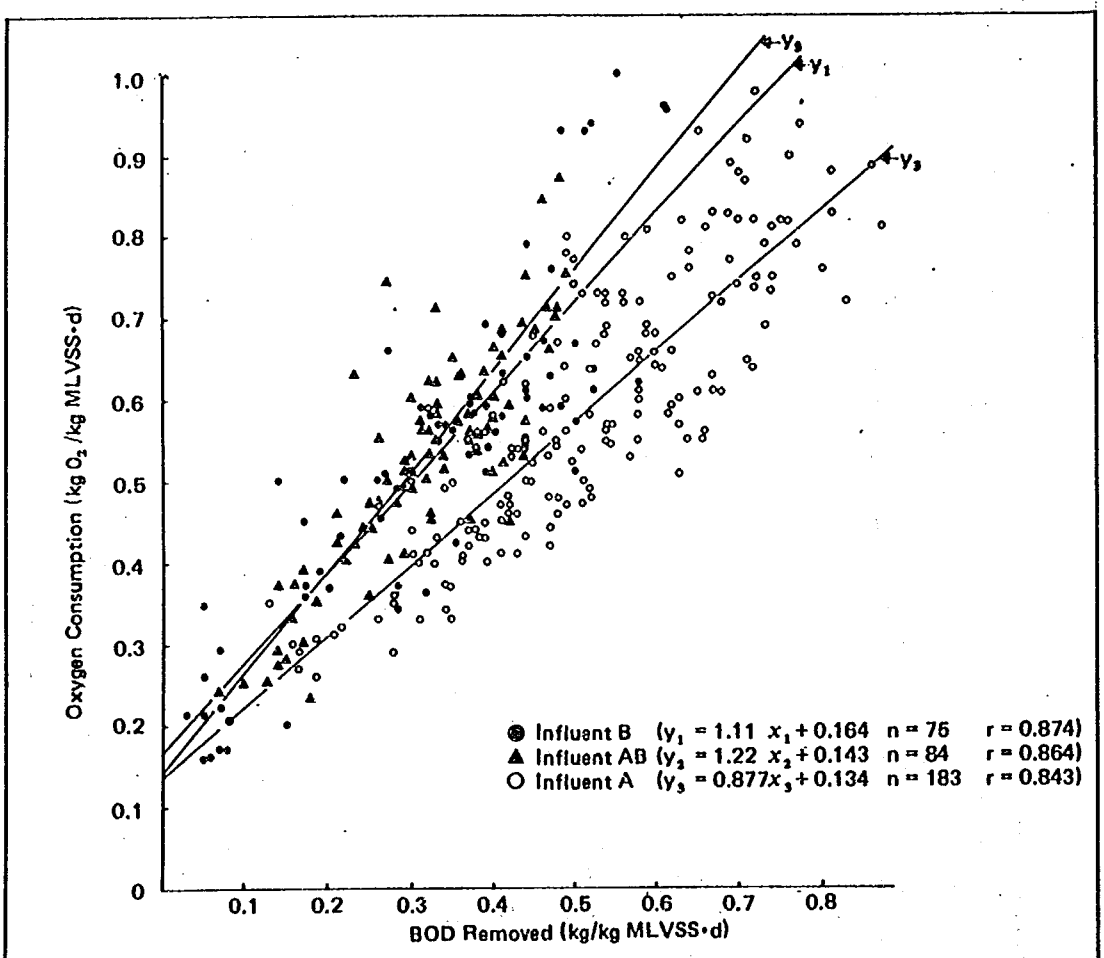


Fig. 16 Relationship between BOD Removed and Oxygen Consumption

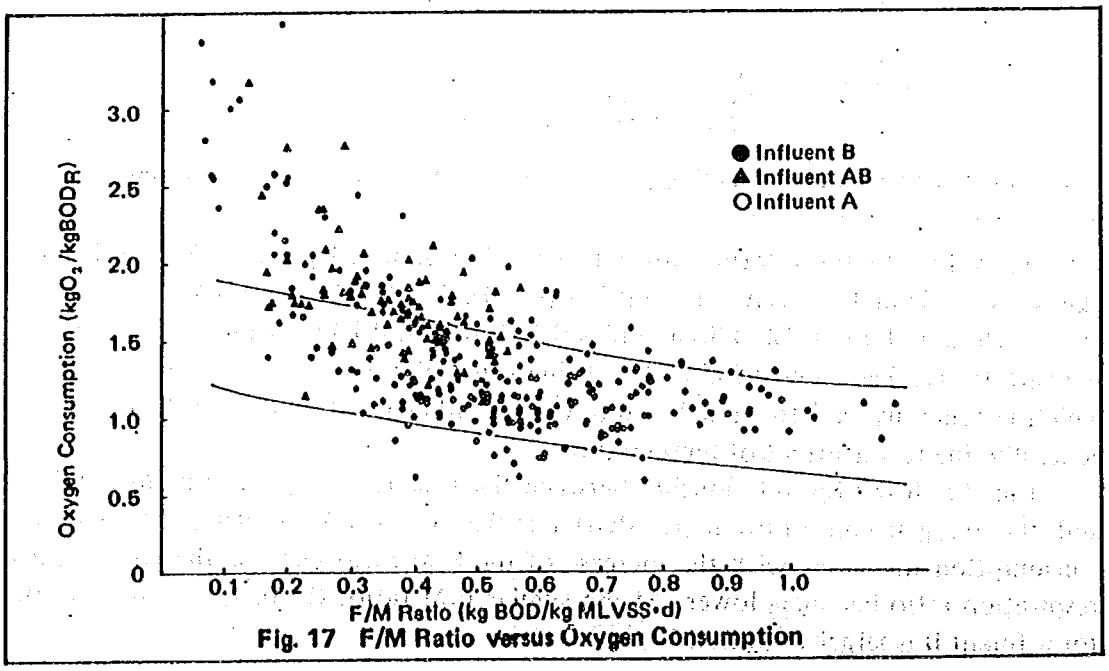


Fig. 17 F/M Ratio versus Oxygen Consumption

3. Power Consumption and Survey for Saving Power

3.1 Power Consumption

The total power consumption in the whole experimental period was about 8,000 kWh/d on average (Fig. 18). Of the total, 39.6 % accounts for air compressor in oxygen generator of PSA (Pressure Swing Adsorption) type and 26.4 % for mechanical aerator. The rate of 66 % for these air compressor and mechanical aerator constitutes a major part of power consumption in the covered pure oxygen system. The power consumption for the secondary treatment except that for ventilation fan and others is studied in the following:

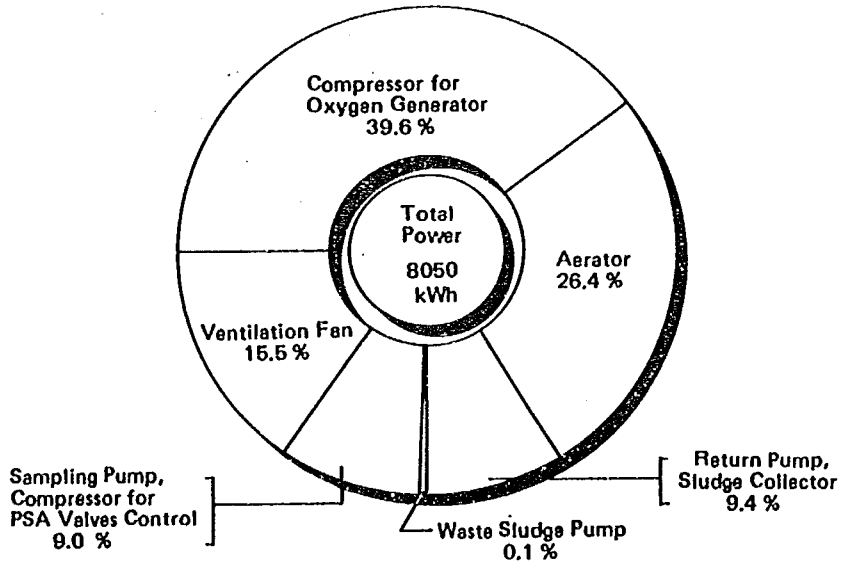


Fig. 18 Distribution of Power Consumption

Shown in Fig. 19 is the relationship between the BOD removed (kg/d) and the power consumption per BOD removed (kWh/kg BOD_R). The power per BOD removed becomes larger for influent B than that for influent A. But it differs to some extent because of the variation of operational conditions and influent substrates.

Oxygen is supplied to the aeration tank by the automatic control of air compressor (motor power : 220 kW) for oxygen generator in accordance with the oxygen consumption. The power consumption rate of influent B is high because of the large consumption of oxygen. It is clear in Fig. 20 showing the relationship between the power consumption for oxygen generator per BOD removed (kWh/kg BOD_R) and the BOD removed (kg/d).

The power consumption per BOD removed becomes large in case of the low BOD loading. It resulted from the increase of the endogenous respiration ratio for activated sludge and the unloading operation of air compressor. The power consumption in the unloading operation of air compressor is only about 15 % of that in the loading operation. In case of the low BOD loading, the power consumption is greatly affected by the unloading operation. Fig. 21 indicates the relationship between the power consumption for oxygen generator (kWh/d) and the amount of oxygen generated (kg O_2/d). The decrease of power consumption is not found in the low supply of oxygen.

The power consumption in the mechanical aerators is described. The aeration tank is divided into four stages. Each stage is equipped with one surface aerator with bottom impeller. The specification of aerator is; 1st stage – 2,540 m/m for surface aerator, 1,330 m/m for bottom impeller in blade diameter, revolution 37 rpm and motor power 37 kW, 2nd ~ 4th stages – 2,090 m/m, 1,220 m/m, 45 rpm and 30 kW. These aerators were usually operated with the constant revolution regardless of the fluctuation of influent loading. The power consumption reaches 26.4 % of the total as mentioned previously. It is further increased in case of the low loading that lowers the oxygen supply.

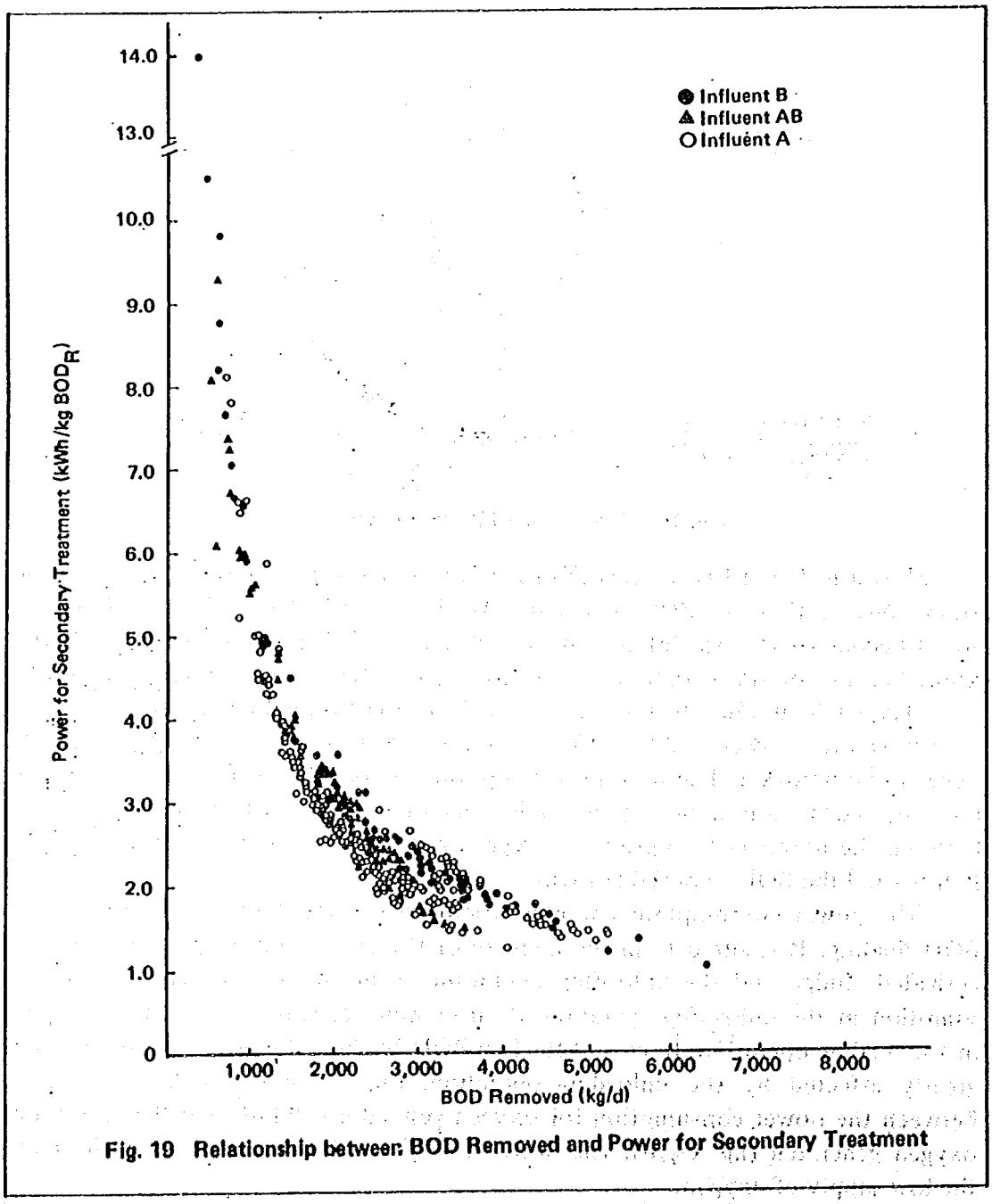


Fig. 19 Relationship between BOD Removed and Power for Secondary Treatment

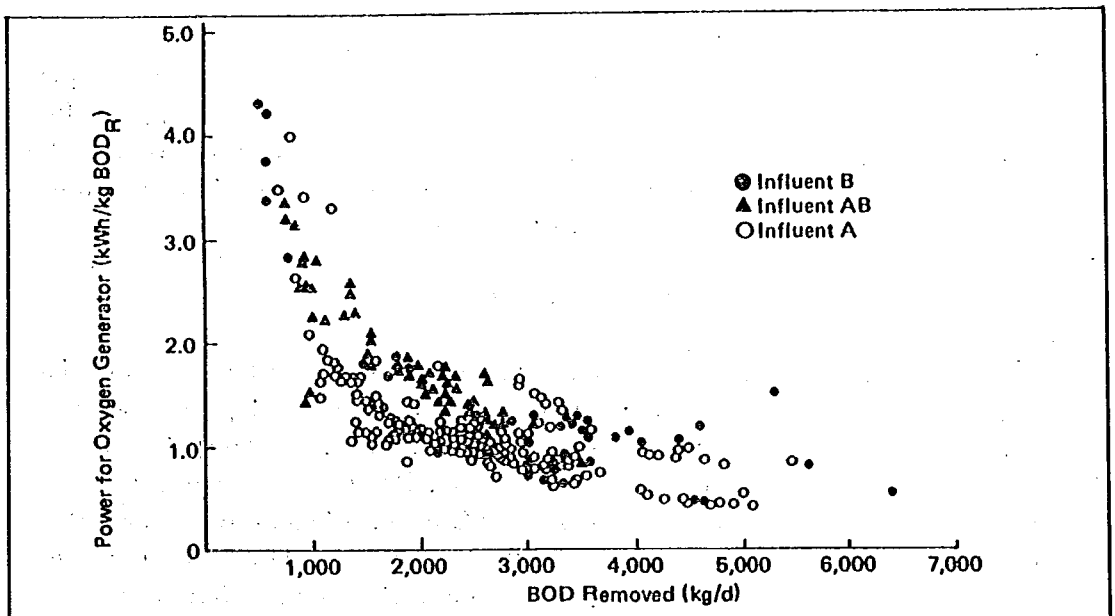


Fig. 20 Relationship between BOD Removed and Power for Oxygen Generator

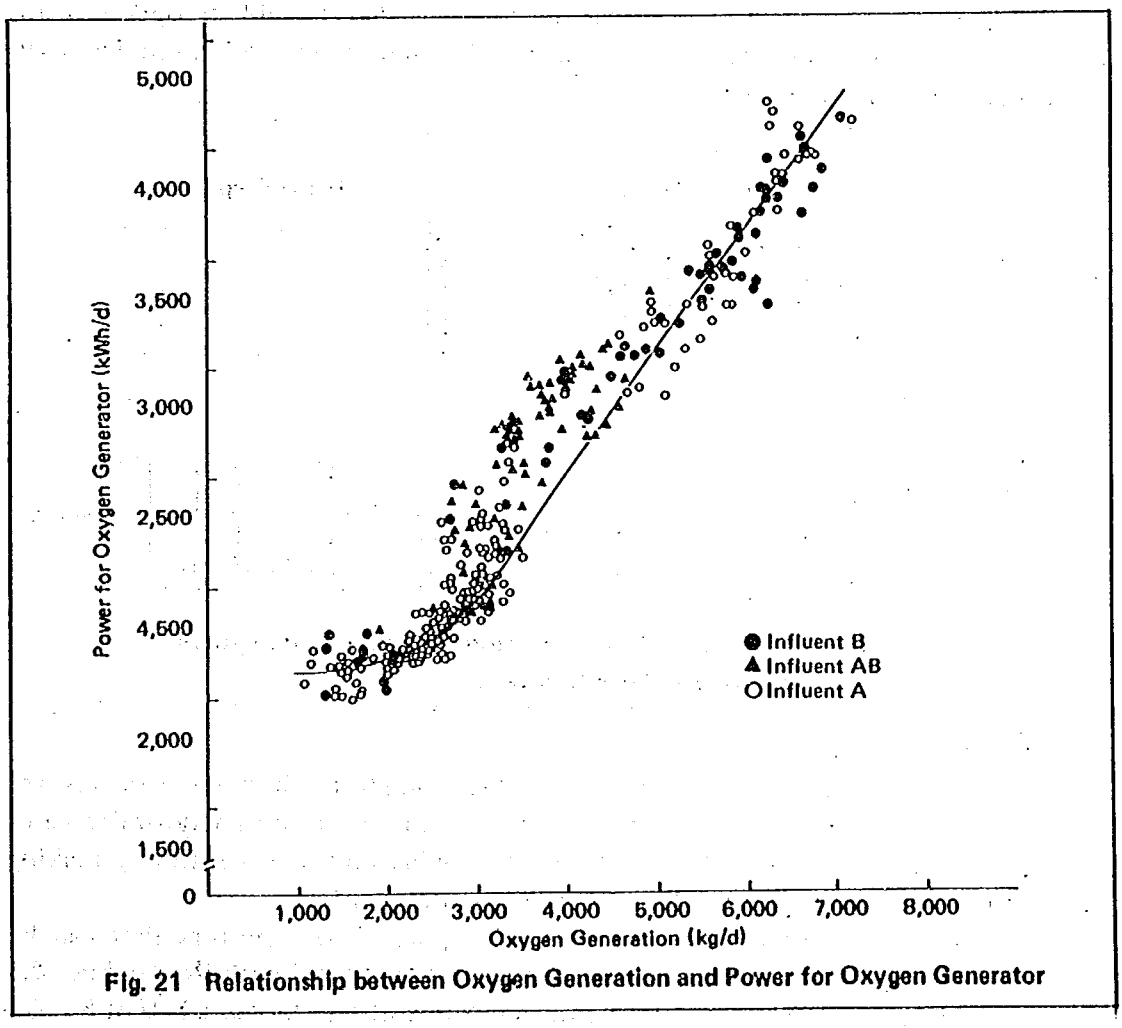


Fig. 21 Relationship between Oxygen Generation and Power for Oxygen Generator

3.2 Experiment for Saving Power

The air compressor for oxygen generator and the mechanical aerator occupies the major part of the power consumption in the covered pure oxygen system. For the reduction of the maintenance and operation cost, it is necessary to save the power consumption. A survey for saving power was carried out from September 1979 to October 1980 after the research for treatment.

3.2.1 Improvement of Facilities for Saving Power

(1) Oxygen Generator

For the PSA oxygen generator, there are the 3-bed and 4-bed systems. The 4-bed system was used in the previous research. The difference of power consumption in the two systems is shown in Fig. 22. In the 3-bed system, the pressure of adsorbent bed ranges from 0.7 to 2.8 kg/cm^{2}G. On the other hand, oxygen is generated at the constant pressure of 2.8 kg/cm^{2}G in the 4-bed system. The output pressure of air compressor becomes similar to the bed pressure. The power consumption in the 3-bed system becomes smaller than that in the 4-bed system because the power consumption is well proportional to the output pressure. But there is a problem as the purity of oxygen in the 3-bed system becomes lower than that in the 4-bed system. The oxygen generator was improved so that it is able to operate with the 4-bed or 3-bed system. The power consumption and the treatment efficiency of the two systems were surveyed.

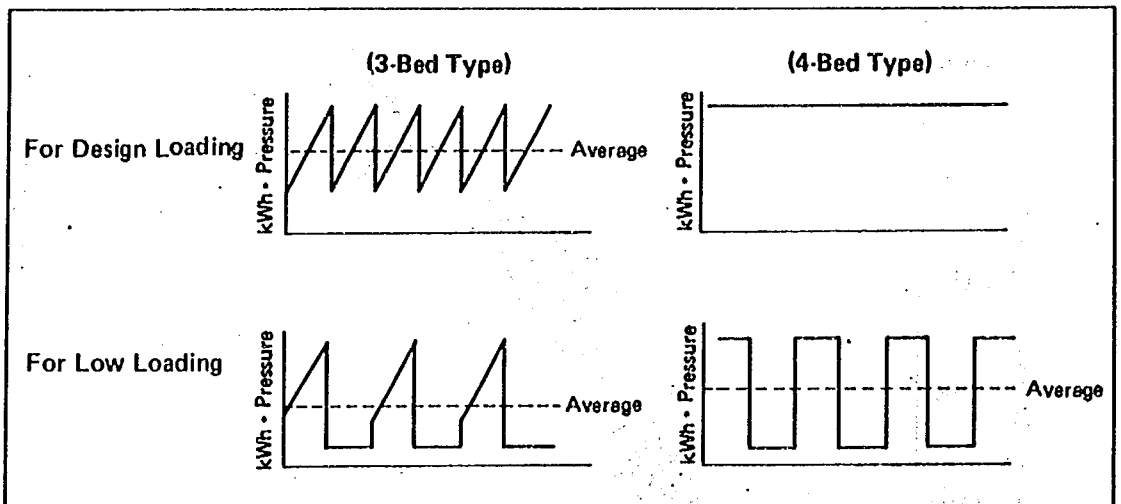


Fig. 22 Comparison of Out put Pressure (or Power Consumption) for 3-Bed and 4-Bed Generators

(2) Mechanical Aerator

Oxygen dissolve excessively in low influent loading when the aerators are operated with the constant revolution. The power consumption of aerator decreases with the slowdown of revolution. Attempt for saving power was made by making the oxygen dissolution appropriate with the revolution control.

The revolution control is conducted with pole change motors that can be changed manually for 3 stages of 4, 6 and 8 poles. As compared with the original, the revolution of motor is the same at 4-pole, two-thirds at 6-pole and a half at 8-pole.

Table 5. Result of Experiments for Saving Power

			Case 1 (Influent A)				Case 2 (Influent A)				Case 3 (Influent B 60%)			
Operation of Aerator			Constant Revolution		Revolution Control		Constant Revolution		Revolution Control		Constant Revolution		Revolution Control	
Temperature (Average °C)			Low (18.5)	High (23.9)	Low (15.2)	High (22.3)	Low (23.7)	High (25.3)	Low (20.5)	High (25.4)	Low (17.6)	High (22.2)	Low (20.1)	High (25.2)
Experimental Period			Nov.30,'79 ~Dec.20	Sep.9,'80 ~Oct.2	Jan.8.'80 ~Feb.8	Oct.7.'80 ~Oct.30	Sep.27,'79 ~Oct.20	Jul.29.'80 ~Aug..14	Nov.7.'79 ~Nov.22	Aug.19.'80 ~Sep.4	Feb.18.'80 ~Mar.13	Apr.22.'80 ~May.29	Mar.15.'80 ~Apr.18	Jun.18'80 ~Jul.17
Inflow Quantity (m^3/d)			21,040	20,900	20,920	20,660	32,660	32,880	32,800	32,900	23,710	23,740	23,570	23,480
Aeration Period (Q base, hrs.)			2.68	2.70	2.70	2.73	1.72	1.71	1.72	1.71	2.38	2.38	2.40	2.40
Oxygen Utilized Rate (%)			96.6	90.4	93.6	84.8	95.5	89.1	94.1	81.7	98.3	95.2	82.8	84.2
BOD-SS Loading (kg BOD/kg SS·d)			0.28	0.22	0.24	0.24	0.32	0.38	0.36	0.32	0.32	0.36	0.36	0.29
BOD-Capacity Loading (kg BOD/m^3·d)			0.88	0.82	0.65	0.77	1.07	0.99	1.09	1.02	1.07	0.96	1.17	0.88
Sedimentation Period (Q base, hrs.)			3.26	3.29	3.29	3.33	2.10	2.09	2.09	2.09	2.90	2.87	2.92	2.93
MLSS (mg/ℓ) (VSS %)			3,100 (81.1)	3,100 (81.1)	2,900 (76.2)	3,320 (77.6)	3,330 (79.6)	2,920 (76.7)	2,770 (79.6)	3,270 (75.6)	3,370 (78.6)	2,880 (78.2)	3,250 (78.0)	3,010 (77.1)
SVI			76	128	99	213	66	74	73	80	97	120	101	104
BOD (mg/ℓ)	Influent		99	73	93	88	77	71	78	75	106	102	116	88
	Effluent		13	4.8	7.6	9.0	17	10	14	7.6	12	10	15	8.1
SS (mg/ℓ)	Influent		57	51	53	56	54	50	56	45	65	78	76	69
	Effluent		5	4	5	4	8	8	7	5	7	12	8	10
COD_{Mn} (mg/ℓ)	Influent		70	55	69	63	52	89	58	50	86	84	96	72
	Effluent		21	15	22	18	17	19	20	16	31	33	33	29
Waste Sludge per BOD Removed (kg ss/kg BOD_R)			0.59	0.62	0.62	0.45	0.48	0.64	0.40	0.46	0.74	0.57	0.77	0.54
Power Consumption (kWh)	Aerator		2,420	2,329	892	1,181	2,380	2,348	1,257	1,625	2,447	2,420	965	1,635
	Oxygen Generator (PSA)	3-Bed	1,850	2,146	1,931	2,042	–	2,329	–	2,799	1,982	2,130	2,339	2,255
		4-Bed	–	2,490	2,184	2,811	2,723	3,565	2,475	–	2,566	–	2,869	–
Power Consumption	Per Inflow Quantity (kWh/m^3)	3-Bed	0.208	0.219	0.141	0.161	–	0.145	–	0.138	0.193	0.194	0.145	0.170
		4-Bed	–	0.235	0.152	0.197	0.160	0.145	0.117	–	0.224	–	0.165	–
	Per BOD Removed (kWh/kg BOD_R)	3-Bed	2.12	3.11	1.38	2.10	–	2.75	–	2.17	2.03	2.38	1.48	2.22
		4-Bed	–	4.40	1.66	2.51	2.85	2.68	1.86	–	2.74	–	2.59	–

3.2.2 Reulsts of Experiment

The research for saving energy was carried out by changing the operation of oxygen generator (3-bed or 4-bed system) and aerator (constant revolution or controlled revolution) for various influent loading. The pattern for revolution control in each case was decided to keep MLDO at about 6 mg/ℓ at the outlet of aeration tank. The average values of the experimental results are shown in Table 5.

(1) Power Saving in Oxygen Generator

Fig. 23 indicates the relationship between BOD removed (kg/d) and power consumption per BOD removed (kWh/kg BOD_R). It is the comparison of the 4-bed and 3-bed systems of oxygen generator with the constant revolution of aerator in all cases. The 3-bed system is better for the power consumption in any BOD loading. For instance, the power consumption is reduced by about 15 % for the 3-bed system as compared with the 4-bed system at 2,000 kg/d BOD removed in the figure.

The purity of oxygen is 85 ~ 88 % for the 3-bed system and 90 ~ 93 % for the 4-bed system. However, the lower purity of the 3-bed system does not affect the treatment efficiency although the oxygen utilization rate also lowers slightly.

(2) Power Saving with Revolution Control

The effect of revolution control in all cases can not be compared in a chart because the control pattern differs in cases. So only an example of Case 3 (influent AB 26,000 m^3/d) is shown in Fig. 24 that indicates the relationship between BOD removed (kg/d) and the power consumption per BOD removed (kWh/kg BOD_R). It is the comparison of the 4-bed and 3-bed systems of oxygen generator with the constant revolution and the controlled revolution of aerator. The power saving with revolution control is more effective than that with the oxygen generator. For instance, the power consumption at 2,000 kg/d BOD removed in the figure is reduced by about 28 % with the revolution control, and it is further saved by about 6 % in the 3-bed system of oxygen generator for the total of about 34 %.

Shown in Fig. 25 is the comparison of the power consumption per BOD removed between the constant revolution with the 4-bed system and the revolution control with the 3-bed system, regardless of the influent and operational condition. The figure indicates a larger effect of power saving in any BOD loading. Meanwhile, the treatment efficiency is not affected by the revolution control as seen in Table 5.

For reference, the comparison of the power consumption of pure oxygen system and the conventional diffused air system is summarized as follows:

In case of the constant revolution with the 4-bed system in the pure oxygen system, the power consumption per BOD removed was 4 ~ 41 % larger than that for the air system at the A-line of the Kisshoin plant. But in the revolution control with the 3-bed system, the power consumption in the pure oxygen system became 5 ~ 26 % smaller than that for the air system.

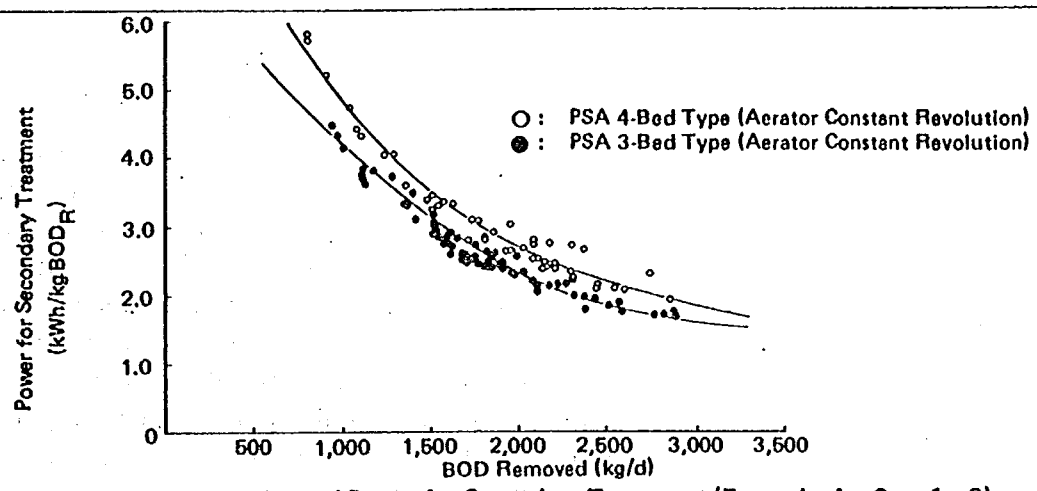


Fig. 23 Comparison of Power for Secondary Treatment (Examples for Case 1 • 2)
—— 4-Bed and 3-Bed PSA Oxygen Generator ——

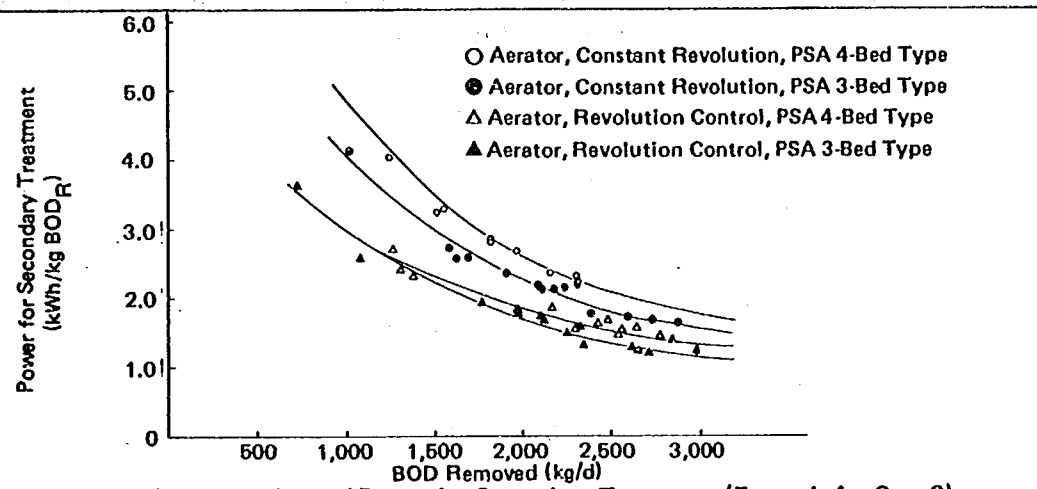


Fig. 24 Comparison of Power for Secondary Treatment (Example for Case 3)
—— Effects of Revolution Control and Improvement of Oxygen Generator ——

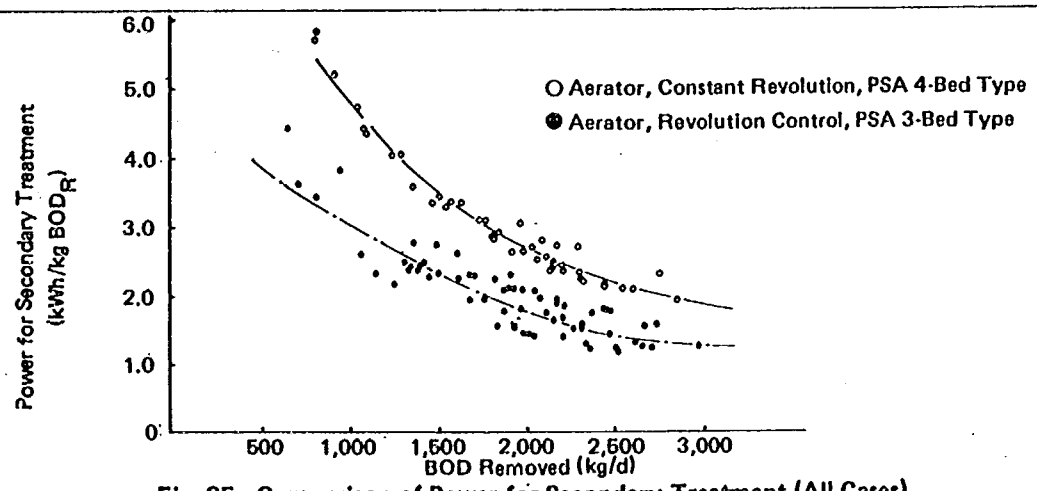


Fig. 25 Comparison of Power for Secondary Treatment (All Cases)
—— Total Effects of Revolution Control and Improvement of Oxygen Generator ——

4. Conclusion

This paper reported the technical research on the joint treatment of municipal and dyeing wastes in the covered pure oxygen system. Although the dyeing wastes include substrates which are hardly biodegradated, the effective treatment is available when the pure oxygen system is operated on the appropriate operational condition by mixing municipal wastes. The power consumption is also saved remarkably with the revolution control of aerator and the improvement of oxygen generator.

The technical research for the oxygen system has been continued. A survey is also under way on problems concerning the corrosion of concrete and steel, automatic control system including the revolution control, durability of molecular sieve adsorbent and removal of nitrogen and phosphorus in the pure oxygen system.

This research has been carried out so that the Ministry of Construction may evaluate the pure oxygen system from engineering viewpoint. We express our profound appreciation to Dr. Mamoru Kashiwaya, former head of the Research and Development Division of the Japan Sewage Works Agency, and those concerned for technical assistance and cooperation in our research.

GERMAN EXPERIENCES WITH THE PURE OXYGEN ACTIVATED SLUDGE PROCESS

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Institute for Sanitary Engineering
Aachen University

1. INTRODUCTION

Since the middle of the 70s also in Western Germany pure oxygen is used in wastewater treatment. The first pure oxygen aerated wastewater treatment plant in Western Germany went into operation in 1974. This has been "Oberding-Work" for treating a knackery's effluent.

After that a lot of pure oxygen aerated plants went into operation in Western Germany.

The first part of the following paper will give a short review about recent application of pure oxygen in Western Germany's wastewater treatment. In the paper's second part a new two stage activated sludge process with pure oxygen aeration in second step shall be described. The so called 'Adsorption Pure Oxygen Activated Sludge Process' (APOP) has been developed at the Institute of Sanitary Engineering at Aachen University.

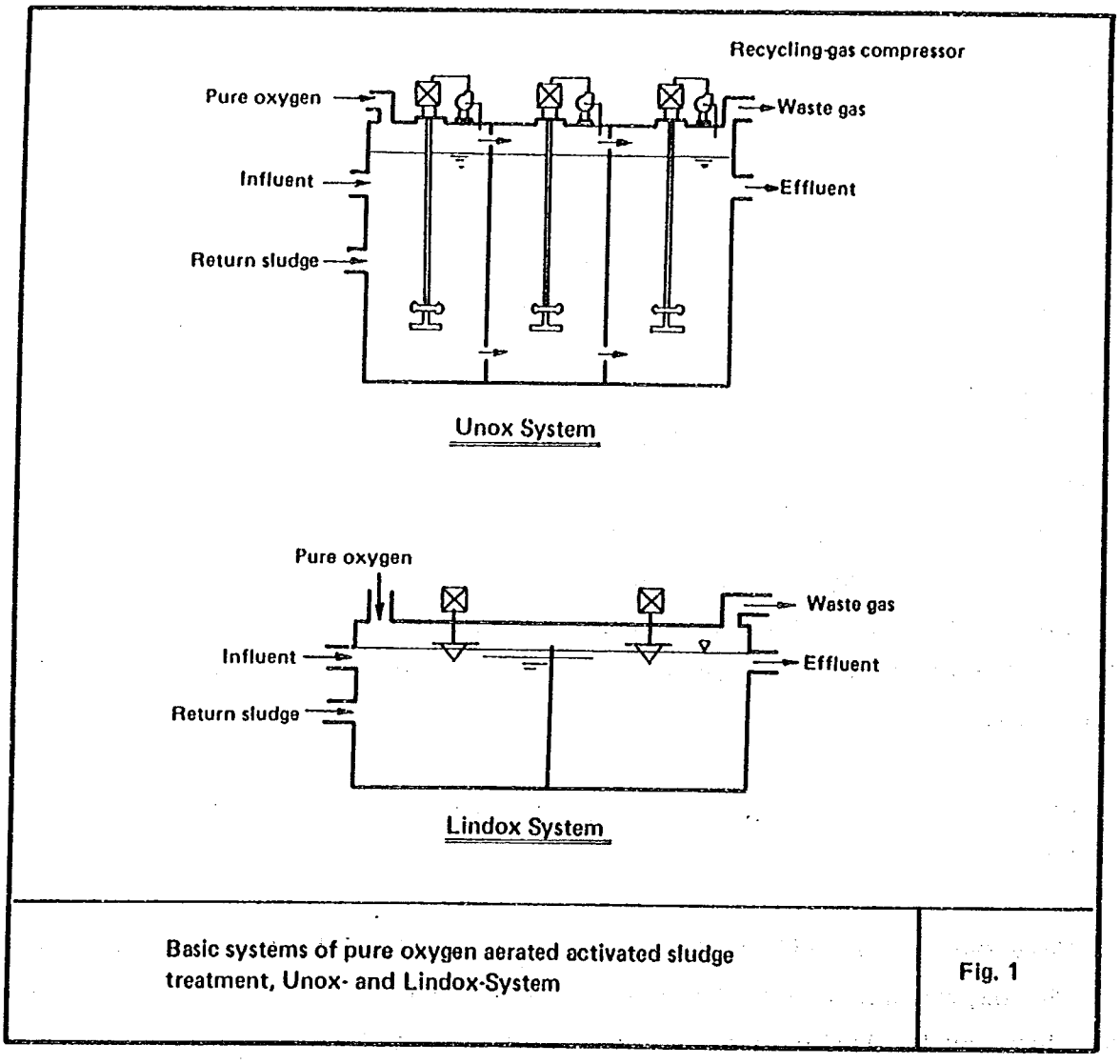
2. PURE OXYGEN AERATED WASTEWATER TREATMENT IN THE FEDERAL REPUBLIC OF GERMANY

Numerous possibilities in pure oxygen aerated plant design are known.

The oxygen aerated tanks may be constructed covered or uncovered and also they may be designed as a one-pocket or multi pocket type.

2.1 Basic systems

In Western Germany till today only covered oxygen aerated plants have been used. The majority is performed as multi pocket plants. Mostly a tank consists of three or four pockets which are divided by walls. The vapor space is covered. The tank is working like a cascade. Pure oxygen and wastewater enter the first pocket and reach the further pockets by crossing holes in the dividing walls. Figure 1 shows those pure oxygen systems which are used in Western Germany. It's the UNOX-System and the LINDOX-System. In UNOX-System pure oxygen is blowed into the tank throughout a hollow shaft. The agitation of mixed liquor is done by propellors. LINDOX-System uses surface aerators and bubble aeration respectively. These facilities achieve both, aeration and agitation. Oxygen-gas is recycled by blowers.



Basic systems of pure oxygen aerated activated sludge treatment, Unox- and Lindox-System

Fig. 1

Fig. 1 Basic Systems of pure oxygen aerated activated sludge treatment, Unox - and Lindox - System

2.2 Pure oxygen generation systems

Beside pipeline providing and liquid gas storage there are two major methods for on-site oxygen generation.

2.2.1 Cryogenic-process (low temperature-process)

For producing a large amount of pure oxygen the cryogenic-process has proved to be the most economical one. The method has been developed by LINDE. It uses the different vaporization points of oxygen and nitrogen for dividing these two phases. The purity of liquid oxygen is about 99.5%. The power consumption for oxygen generation and aeration ranges about 0.6 kWh/kg O_2.

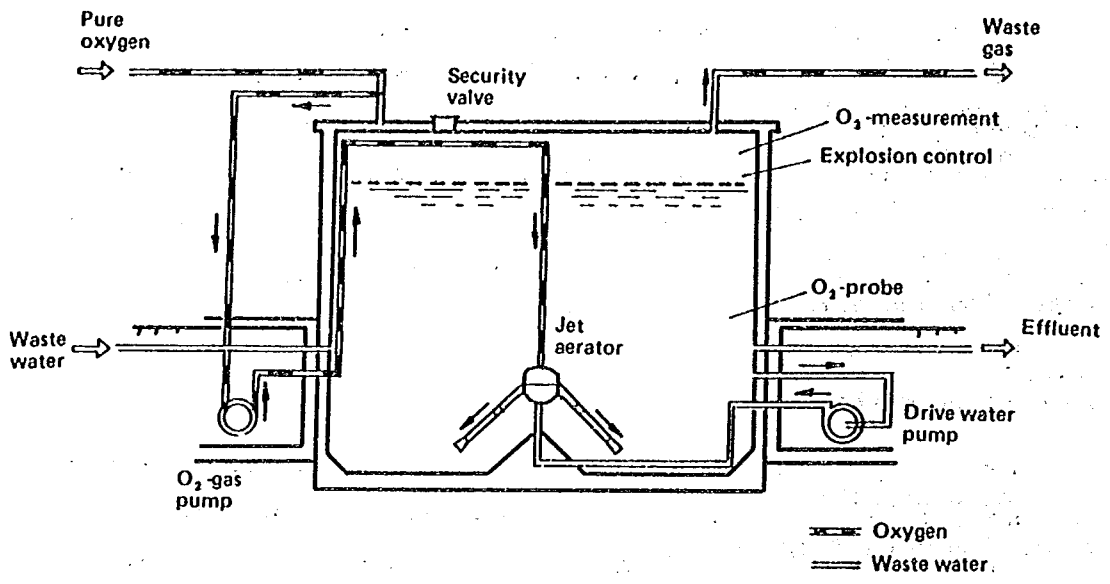
2.2.2 Adsorption-method

For generation of small amounts of pure oxygen the Pressure Swing Adsorption-method (PSA) mostly is used. It has proved to be convenient if oxygen demand ranges between 1.0 and 30.0 t/d. The compressed air is lead through a molecular sieve where the nitrogen is adsorbed. The oxygen is generated in gaseous state. The purity ranges at about 90%. The advantage of PSA is the possibility of oxygen generation in response to the plant's actual oxygen demand. Power consumption for oxygen generation and aeration ranges about at 0.7 kWh/kg O_2.

2.3 Aeration facilities

Mostly in Western Germany's pure oxygen aerated plants the aeration in done by surface aerators. They are sturdy and their reliability is rather high.

Recently there has been developed the so called 'jet - aeration' by BAYER AG and BASF AG respectively.



Jet Aeration | Fig. 2

Fig. 2 JET AERATION

If the aeration tank has to be designed in a more deep shape - for instance because of space lack - 'jet-aeration' as a volume aerating system may be recommended. The jet-aerator is a kind of water jet pump. The drivewater-pumps exhaust the mixed liquor from the activated sludge tank and push it through the injector. The mixed liquor leaves the nozzels together with small fixed and finely divided oxygen bubbles (Fig. 2). The nozzel diameter of BAYER-jet aerator is 8.0 mm the one of BASF-jet aerator 30.0 mm. Since 1977 the BAYER-jet aerator works on BAYER-treatment plant 'Wuppertal-Rutenbeck' without complaints. Also the BASF-jet aerator is working successfully on KNOLL-AG-treatment plant at Minden, since 1980.

3. REVIEW ABOUT THE NOWADAYS EXISTING PURE OXYGEN WASTEWATER TREATMENT PLANTS IN WESTERN GERMANY

The review will be given separately about municipal and industrial wastewater treatment plants by using two tables.

3.1 Municipal wastewater treatment plants

Table 1 describes Western Germany's nowadays existing municipal pure oxygen aerated wastewater treatment plants. It is to be seen that pure oxygen treatment of municipal wastewater in Western Germany is used only very recently. The BOD_5 - sludge load ranges between 0.30 kg BOD_5/kg MLSS/d and 0.55 kg BOD_5/kg MLSS/d. Despite of these more high sludge loading values, the BOD_5 - effluent concentrations are rather low. On 'Breisgauer-Bucht'-treatment plant nitrification occurs temporary.

While discussing pure oxygen aeration, concrete-corrosion due to concrete-agressive carbonic acid has to be mentioned. Slight concrete corrosion for instance has been observed on 'Breisgauer Bucht'-treatment plant. Mostly, however a concrete-cover of 4.00 cm is sufficient to avoid severe dawage of reinforcement.

Power-consumption of 'Goeppingen'-treatment plant and 'Luebeck-Herrenwyk'-treatment plant ranged at 1.0 kWh/kg $BOD_{5\text{-removed}}$ and 0.8 kWh/kg $BOD_{5\text{-removed}}$ respectively. On 'Breisgauer-Bucht'-treatment plant the power consumption was 2.4 kWh/kg $BOD_{5\text{-removed}}$ due to the temporary nitrification.

Western German Pure Oxygen Activated Sludge Plants/Municipal Wastewater								
Plant name	Flow	Influent BOD_5	Reactor volume	Sludge load	Effluent BOD_5	O_2-production System	O_2-production Capacity	Start of operation
	m^3/d	mg/ℓ	m^3	kg BOD/ kg/MLSS/d	mg/ℓ	-	t/d	-
Breisgauer Bucht	100,000	100	24,400	0.12-0.34	6[3]	Cryo.	78	1980
Goeppingen	50,000	190	6,272	0.40-0.55	13[4]	PSA[1]	16	1978
Luebeck-Herrenwyk	1,480	482	436	0.54	22[4]	Liquid Storage	33	1980
Lemgo	43,200	130	3,752	0.30	-	Cryo.	6.5	n.o.[2]
Bremerhaven	58,300	424	12,800	0.30	-	-	-	n.o.[2]
Bielefeld-Heepen	59,313	151	-	0.33	-	Cryo.	10	n.o.[2]
Bonn-Salierweg	67,500	237	8,000	0.27	-	Cryo.	17	1980
	1) Pressure Swing Adsorption 2) not yet in operation 3) BOD_{5C} 4) BOD_{5T}							
ISWW Aachen 1982	Plant Loading, Wastewater Quality, Oxygen-Production System							Table 1

3.2 Industrial wastewater treatment plants

Table 2 describes the nowadays existing Western German pure oxygen plants for treatment of industrial wastewater. Despite of the very high BOD_5-influent concentrations the majority of the listed plants reaches estonishing low BOD_5-effluent concentrations. Most of the german plants which are listed in Table 2 are treating knackery-effluents. Surely first of all ordor-problems occuring with knackery-wastewater treatment, have urged the responsibles to install the first Germany pure oxygen treatment plant already in 1974.

Western German Pure Oxygen Activated Sludge Plants/Industrial Wastewater									
Plant name	Type of industry	Flow	Influent BOD_5	Reactor volume	Sludge load	Effluent BOD_5	O_2-production System	O_2-production Capacity	Start of operation
		m^3/d	mg/ℓ	m^3	kg BOD/ kg/MLSS/d	mg/ℓ	-	t/d	-
BAYER AG Wuppertal	Chemical Industry	11,000	1,940	(I) 6,240 (II) 3,260	(I) 0.63 (II) 0.34	50 (T)	Cryo.	67	1977
KNOLL AG Minden	Pharmacy	6,700	1,050	1,250	0.22	24 (C)	Cryo.	15.5	1980
Oberding[1)]	Knackery	26	6,900	134	0.20	25 (T)	Liquid Storage	-	1974
St. Erasmus[1)]	Knackery	35	3,300	67	0.25	25 (T)	Liquid Storage	-	1976
Plattling[1)]	Knackery	272	5,900	1,194	0.20	25 (T)	Liquid Storage	-	1977
Gunzen-hausen[1)]	Knackery	27	4,500	67	0.39	25 (T)	Liquid Storage	-	1977
Scheide-mantel[1)]	Bones	481	3,750	625	0.58	50-200 (T)	Liquid Storage	-	1977
Roten-stadt[1)]	Knackery	20	4,000	66	-	25 (T)	Liquid Storage	-	1977
Heufeld[1)]	Chemical Industry	123	3,000	231	-	25-90 (T)	Liquid Storage	-	1978
Sprend-Lingen[1)]	Knackery	25	2,700	64	-	25 (T)	Liquid Storage	-	1978
Nieder-Wollstadt[1)]	Knackery	100	5,000	111	-	400 (T)	Liquid Storage	-	1978

1) Source: Bischofsberger, W: Belebungsanlagen mit Sauerstoffbegasung fuer industrielle Abwasser in Deutschland, GWA-Aachen, Bd. 42, 1980 609-634

ISWW Aachen 1982	Plant Loading, Wastewater Quality	Table 2

4. NEW DEVELOPMENT AT THE INSTITUTE FOR SANITARY ENGINEERING AT AACHEN-UNIVERSITY

Considering experiences with high efficiency and reliability of existing pure oxygen treatment plants at the Institute for Sanitary Engineering at Aachen University a new two stage activated sludge process had been developed. Second step of this process is pure oxygen aerated. The process is called 'Adsorption Pure Oxygen Activated Sludge Process' (APOP). Actually this system is a modification of a two stage air aerated activated sludge process, which has also been developed and tested at the Institute for Sanitary Engineering at Aachen University. APOP has been supposed to reach high efficiency and process stability even if raw wastewater would be of bad quality. For providing the advantages of APOP 20 months losting experiments had been implemented on 'Krefeld'-sewage works.

Experiments main objects have been the followings:

1. Evaluation of BOD_{5T} - and COD_{Cr}-effluent concentrations

2. As far as possible observation of nitrogen-removal, especially NH_4-N-decrease and NO_3-N-concentrations.

3. Investigations of process-stability with regard to BOD_{5T}-effluent concentrations

4. Estimation of sludge de-waterability of first stage sludge, second stage sludge and the mixture of first and second stage sludge.

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* BOD_5-analysis without ATU

4.1 Description of two stage pilot plant

Figure 3 shows the pilot plant's flow sheet. A short review about the tanks' volumes is listed below:

1st high loaded tank	0.60 m^3
intermediate clarifyer	1.20 m^3
2nd pure oxygen aerated tank	0.75 m^3
final clarifyer	3.00 m^3

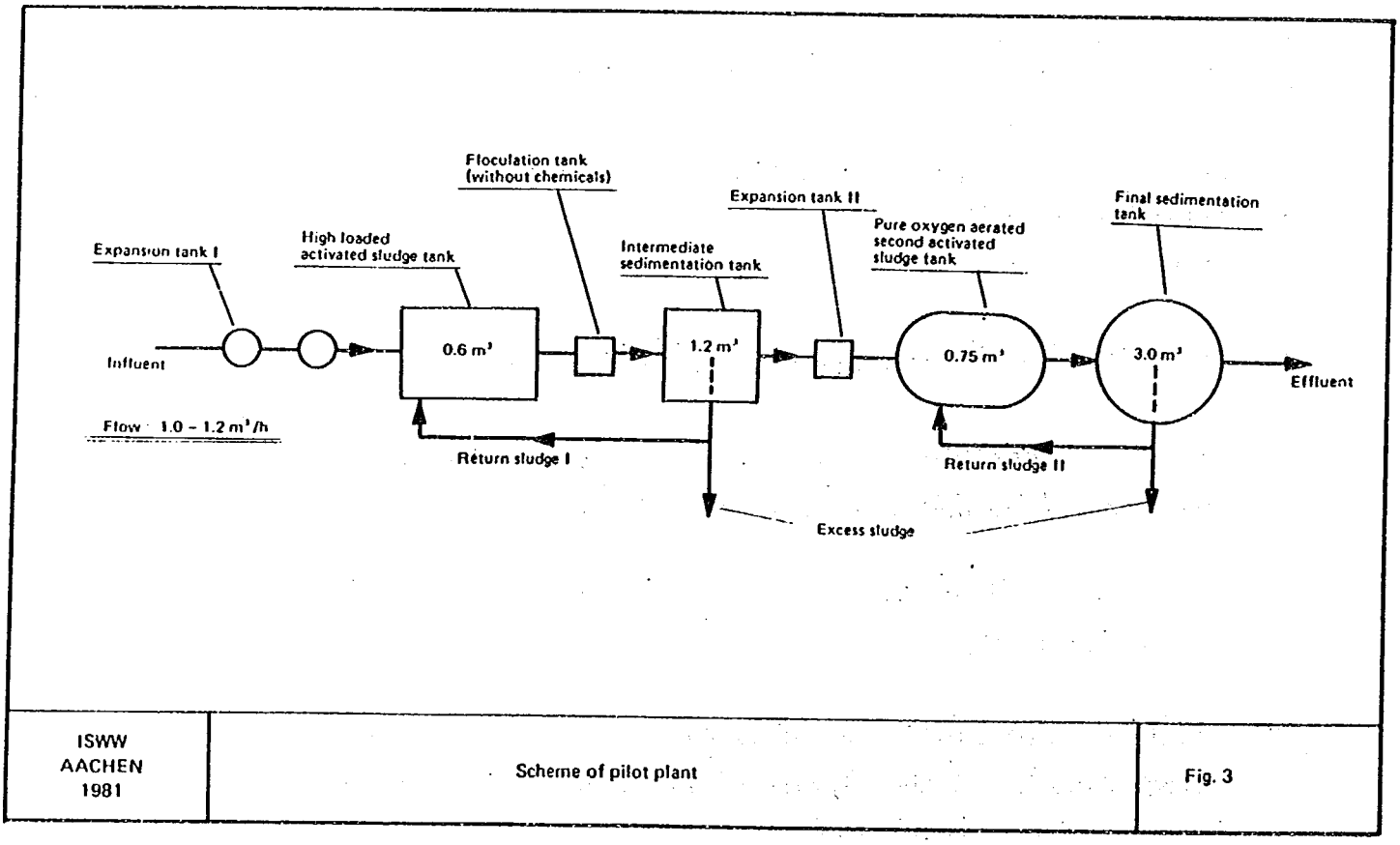
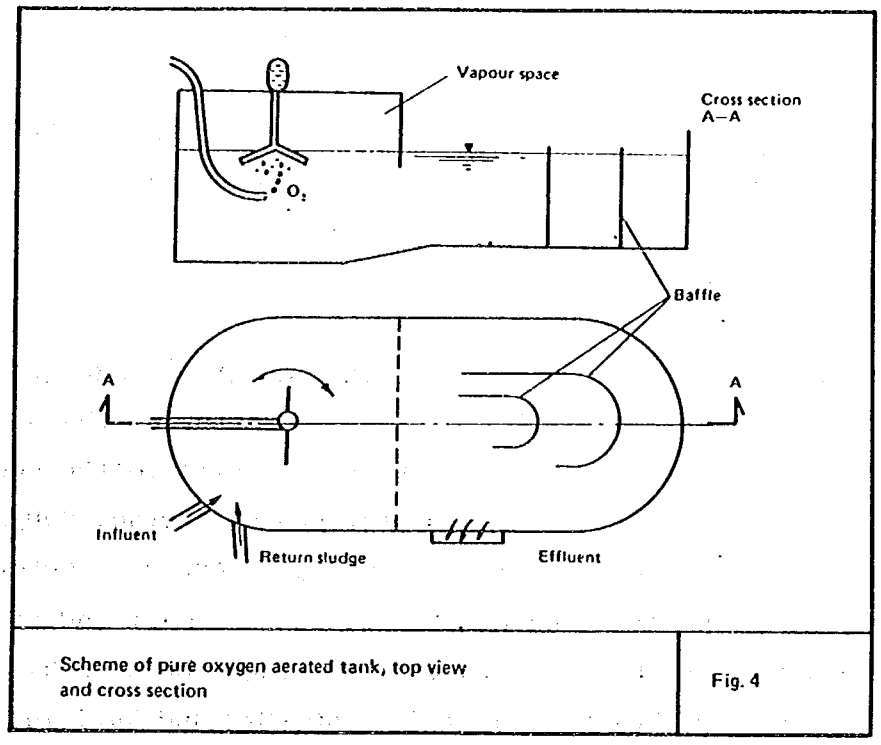


Fig. 3 SCHEME OF PILOT PLANT

During the several experiment-phases flow ranged between 1.0 m^3/h and 1.2 m^3/h. Second stage aeration tank's design is shown by Figure 4. The pure oxygen aerated tank is only partly covered. Aeration is achieved by a combined system of surface aerater and coarse bubble diffuser.

An advantage of the ajar construction is that severe pH-value decrease can be prevented because carbon dioxide is allowed to escape to free atmosphere.



Scheme of pure oxygen aerated tank, top view and cross section

Fig. 4

Fig. 4 Scheme of pure oxygen aerated tank, top view and cross section

The pilot plant has been supervised by continuous on-line-measurement of pH-value, conductivity, temperature and dissolved oxygen concentration (in liquid and in vapor). All values have been registered continuously by operators and also automatically. Also 24-h-time corresponding samples have been taken for determination of BOD_{5T}, COD_{Cr}, TOC and nitrogen compounds like NH_4-N and NO_3-N.

4.2 Experimental programme/conditions of operation

The Experiments have been achieved between 13th May 1979 and 20th November 1980. Several conditions of operation have been adjusted during the 20 months lasting experimental period.

Especially the following modes have been adjusted in first and second step respectively.

High loaded step	pure oxygen aerated step
1. facultative anaerobe	BOD_5-sludge load ranging between 0.15 and 0.90
2. aerobe	O_2-concentrations between 3.0 mg/ℓ and 9.0 mg/ℓ

In final evaluation the several experiment phases will be separatly recognized. All together twelve modes have been tested.

4.3 Influent wastewater quality

Krefeld's sewarage contains a large amount of industrial wastewater. Beside a great lot of textile factories there are also fertilizer-and other chemical factories connected to the sewer.

During the individual experimental phases average BOD_5-influent concentrations between 135 mg/ℓ and 266 mg/ℓ have been determinated. COD_{Cr}-influent concentration ranged between 270 mg/ℓ and 504 mg/ℓ. NH_4-N- and NO_3-N influent concentrations ranged between 81.5 mg NH_4-N/ℓ and 50.3 mg NH_4-N/ℓ and 7.5 mg NO_3-N/ℓ and 30.4 mg NO_3-N/ℓ respectively.

Some days NH_4-N- influent concentration has been determinated to about 100 mg/ℓ and NO_3-N- influent concentration to about 40 mg/ℓ. Serious problems may be caused by the remarkable pH-valve variability. Diurnal variations between 3.0 and 11.0 have been recognized several times. Lowest pH-value measured in the 1st high loaded tank has been 2.6. The highest pH-value inside this tank has been 11.2. With this characterization the wastewate of Krefeld-Municipality surely cannot be expected to be easy treatable.

4.4 Experimental results

With regard to carbon and nitrogen reduction the most essential results of two stage pure oxygen aerated pilot plant will be described by a short review. Also process-stability, pH value behaviour and sludge de-waterbility will be discussed.

4.4.1 BOD_{5T} - and COD_{Cr} - effluent concentrations

Experimental modes' average BOD_{5T} - effluent concentrations corresponding to second stage's BOD-sludge load are shown by Figure 5. The symbols (●, ⊡, ⊙, -┼-) describe the following operating conditions of pilot plant:

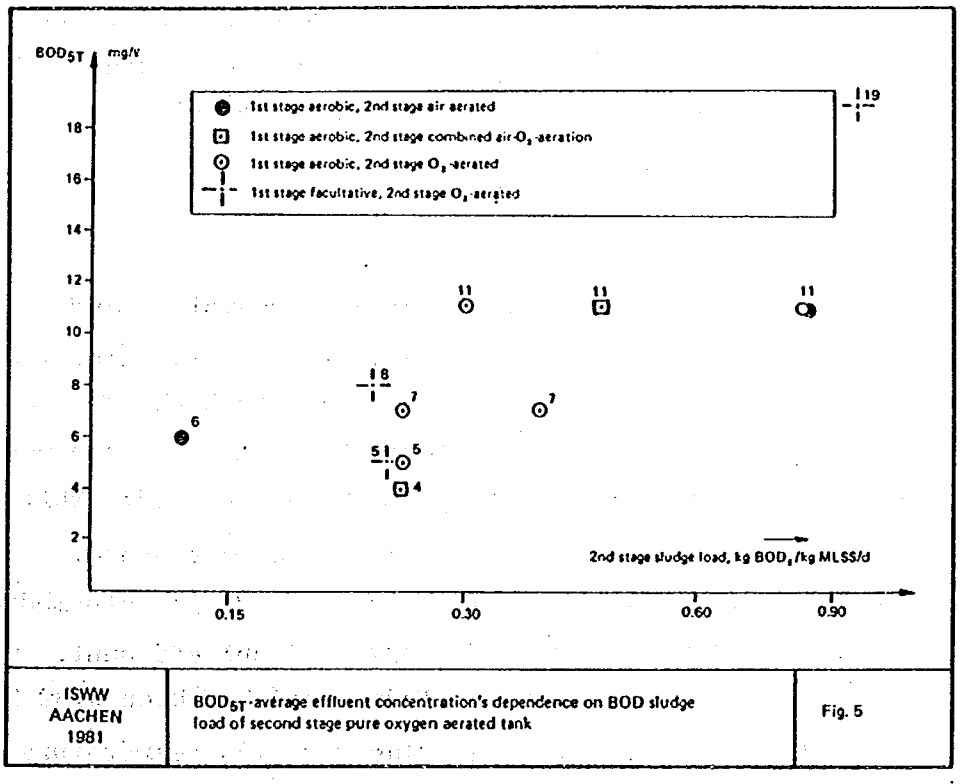


Fig. 5 BOD_{5T}-average effluent concentration's dependence on BOD sludge load of second stage pure oxygen aerated tank

● 1st stage aerobic, 2nd stage air aerated

⊡ 1st stage aerobic, 2nd stage combined air-O^2-aerated

⊙ 1st stage aerobic, 2nd stage O_2-aerated

-┼- 1st stage facultative anaerobic, 2nd stage O_2-aerated

Operation conditions relative to aeration seem to be of low influence on BOD_{5T}-effluent concentration. However correspondence between BOD_{5T}-effluent concentration and second stage's sludge load is obvious. But as long as second stage's sludge laod is less equal 0.60 kg BOD_5/kg MLSS/d BOD_{5T}-effluent concentration is less equal 11.0 mg/ℓ. First stage's sludge load ranged troughout all experimental periods between 2.8 kg BOD_5/kg MLSS/d and 5.8 kg BOD_5/kg MLSS/d. COD_{Cr}* ranged between 83 mg/ℓ and 104 mg/ℓ. It has to be mentioned that the COD_{Cr}-analysis has been done without a correction of the mistake due to oxidation of chloride.

4.4.2 Nitrification

Due to the high variability of influent pH-values and NH_4-N-influent concentrations in the majority of all experimental modes nitrification could not be achieved-although the second stage's sludge load had been low. Nevertheless there has been one period during which nitrification definetly has been achieved. While NH_4-N-effluent concentration of 4.7 mg/ℓ had been reached. NO_3-N-concentration increased from 0.2 mg NO_3-N/ℓ in intermediate clarifayer's effluent to 19.5 mg NO_3-N/ℓ in fianl effluent. During that period second stage's average sludge load had been 0.30 kg BOD_5/kg MLSS/d. Second stage's pH-value was 6.3, temperature 21.9°C and dissolved oxygen concentration in liquid, 3.8 mgO_2/ℓ.

These results show once more that nitrification can be achieved by pure oxygen aerated plants as well as by conventional air aerated plants-presumed the edge conditions like temperature, pH-value and dissolved oxygen concentration are sufficient.

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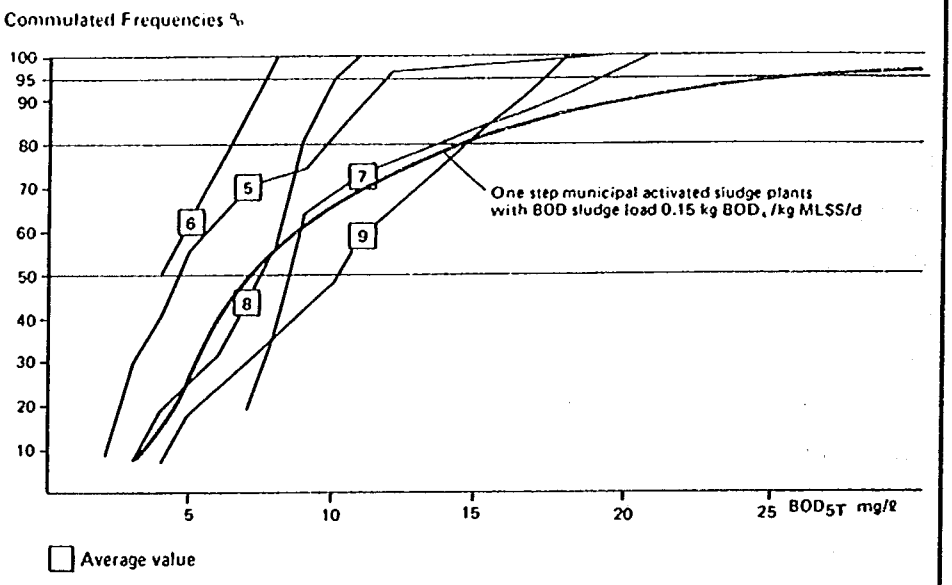
* $K_2Cr_2O_7$ has been used for COD_{Cr}-analysis

4.4.3 Pilot plant's process-stability

To estimate the process-stability of APOP the BOD_{5T}-effluent concentrations of the different periods have been evaluated separately by using statistical methods like frequencies distributions and commulated frequencies.

Figure 6 shows the commulated frequencies lines with regard to BOD_{5T}-effluent concentrations only for those periods during which first stage worked aerobicaly and second stage had been aerated by pure oxygen (AO). The abscissa shows BOD_{5T}-effluent concentrations, the ordinate the commulated frequencies as %-values. The rectangles at the commulated frequencies lines mark the distribution's mean-value position. The figures inside the rectangles describe the experimental period. The small table below the graph explains the operation conditions during the periods.

The thick drawed line shows the commulated frequencies of one-stage municipal wastewater treatment plants which are performed with an average BOD-sludge load of 0.15 kg BOD_5/kg MLSS/d.



Graph No.	Experiment No.	Performance mode	BOD sludge load 2nd stage
5	4.1 + 4.5	AO	0.25
6	4.3	AO	0.25
7	4.9	AO	0.30
8	2.3 + 4.4 + 4.7	AO	0.38
9	4.8	AO	0.82

ISWW AACHEN 1982	Commulated frequencies of BOD_{5T} effluent concentration of two stage 'Adsorption – Pure Oxygen Activated Sludge Process' (APOP)	Fig. 6

Fig. 6 Commulated frequencies of BOD_{5T} effluent concentration of two stage 'Adsorption - Pure Oxygen Activated Sludge Process' (APOP)

By implementing on extensive statistical evaluation recently it has been possible to show that one stage conventional activated sludge plants are able to keep the Western German BOD_{5C}- and COD_{Cr}-effluent standards if they are performed with a BOD-sludge load of less equal 0.15 kg BOD_5/kg MLSS/d.

Comparing the pilot-plant's frequencies lines with the 0.15-curve it become obvious that the pilot plant during all five experimental periods achieved the same or even a better efficiency and process-stability than the one stage conventional 0.15 activated sludge plants. Standard deviation for all five periods is less equal 5.1 mg/ℓ. Standard deviation of conventional 0.15 plant is 10 mg/ℓ. Even if a pilot plant usually is expected to achieve better effluent concentration than a large scale wastewater treatment plant, Figure 6 allows the following statement:

It may be assumed that APOP's process stability with regard to BOD_{5C} is comparable or even higher than process stability of conventional one stage activated sludge plants with BOD sludge load 0.15 kg BOD_5/kg MLSS/d - even if APOP's second stage sludge load is less equal 0.60 kg BOD_5/kg MLSS/d and the influent wastewater is of bad quality.

4.4.4 Pilot plant's pH-value behaviour

Figure 7 gives a good view about pH-value behaviour of the pilot plant.

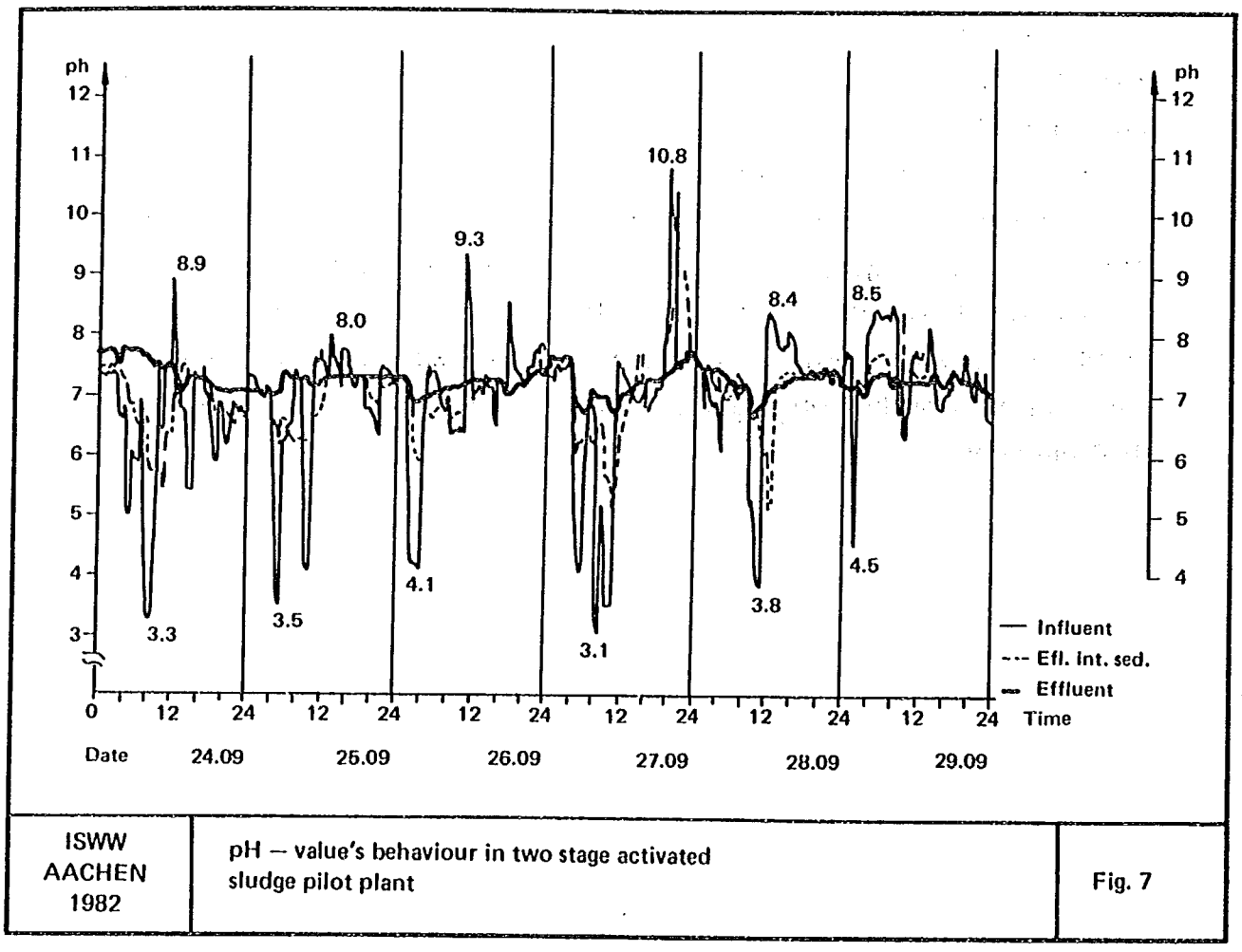


Fig. 7 pH - value's behaviour in two stage activated sludge pilot plant

Effluent pH-values always range between 7.0 and 7.5 even if influent pH-values' variation is remarkable. During the experiments no pH-value control had been performed. pH-value in effluent only decreased when low influent pH values lost for about six hours-so happened twice during the twenty months experimental period. In those cases of severe pH-value decrease effluent quality detereorated distinctly. Nevertheless after these accidents it tooks the pilot plant only two days to reach again low effluent concentrations.

4.4.5 Sludge characteristics

In the first high loaded stage an average sludge volume index of about

45 mℓ/g

could be achieved. Significant difference between sludge volume index of aerobic first stage and facultativ anaerobic working first stage could not be recognized.

Sludge volume index of second stage pure oxygen aerated tank in the majority has been below 100 mℓ/g. Based on the experience of the 20 months lasting experiments sludge volume index of the pure oxygen aerated second stage can be supposed to range between

50 mℓ/g and 90 mℓ/g.

Concluding sludge de-watering experiments with a KHD-* centrifuge proved the sludge mixture of first and second stage exess sludge to have a good de-waterability. Sludge with an inlet water content of 93.5% had been dewatered to 71.6% by dosing only 70 g/m^3 polyelectrolytes as de-watering aid.

4.4.6 Further experiences with pilot plant

With regard to the specific oxygen consumption (kgO_2/kg BOD_5-removed) the design of second stage aeration tank has proved to be not sufficient. Due to remarkable oxygen losses the pilot plants' lowest specific oxygen consumption which could be achieved has been 2.87 kgO_2/kg BOD_5-removed. For this reason the simple design of pilot plant's second stage aeration tank has to be improved for instance by covering a biger part of the ditch and installing a second aerater for using the pure oxygen which remains in vapor space after pasing the first aerater. However, to examine the pure oxygen consumption and also the over all power consumpting of APOP, it will be necessary to implement experiments with a larger scaled treatment plant.

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*Kloeckner Humbold Dewag

5. SUMMARY

After a short review about the nowadays existing Western German pure oxygen wastewater treatment technology, experimental results are introduced which have been achieved at Institute for Sanitary Engineering of Aachen University by running a two stage activated sludge pilot plant about 20 months in 1979 and 1980.

The pilot plant worked with a high loaded first activated sludge stage and a low loaded, partly covered, pure oxygen aerated second activated sludge stage.

The following results have been achieved:

- Even if second stage sludge load has been 0.60 kg BOD_5/kg MLSS/d average effluent concentrations weren't higher than 11.0 mg BOD_{5T}/ℓ (without ATU-dossage) and 104 mg COD_{Cr}/ℓ (without correction of chloride oxidation).

- Process stability of APOP is comparable or higher than process stability of conventional activated sludge plants with sludge load 0.15 kg BOD_5/kg MLSS/d even if APOP's second stage sludge load is 0.60 kg BOD_5/kg MLSS/d.

- Nitrification could be distinctly achieved while second stage tank had been aerated by pure oxygen.

- De-waterability of first and second stage's sludge is rather high. For this reason problems in sludge handling are not to be expected.

These results have proved APOP as a method of wastewater treatment which is well suitable to treat wastewater of high concentrations and changing quality.

Another application possibility of APOP is to be seen in improvement of overloaded wastewater treatment plants.

ANALYSIS OF EXISTING FINAL CLARIFIERS AND DESIGN CONSIDERATION FOR BETTER PERFORMANCE

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1. INTRODUCTION

A final, or secondary, clarifier is supposed to perform two functions; solids separation from supernatant liquor and solids thickening.

"Overflow rate" is a measure to indicate the former function and solids surface loading indicates the latter. The larger number necessary to satisfy either function determines the clarifier's surface area. In almost all publicly owned sewage plants treating mainly domestic wastewater, the surface area required to satisfy solids separation is greater. Thus overflow is usually the controlling factor.

The "overflow rate theory" is sound and proves itself well under ideal conditions. However, the efficiency of settling in a continuous flow tank is far less than that predicted by the theory. This is primarily because flow conditions in a typical continuous flow tank are not equal to those considered by the theory.

There has been no way to predict efficiency loss in continuous tanks, so very high overflow rate values have been used in design to ensure adequate performance. In general, 30 $m^3/m^2/day$ has been considered a satisfactory value for secondary clarifiers.

The efficiency of settling is reduced mainly by short circuit caused by density currents. If short circuit can be controlled or reduced, a larger overflow rate is permitted in the clarifier design, and space-saving results.

The problem is how to control these circuits. There are already 12 sewage treatment plants in Tokyo using different kinds of final clarifiers. We studied the performance of some of these clarifiers. The types surveyed were:

1 Center-feed peripheral-out circular clarifier (CPC)
2 Peripheral-feed peripheral-out circular clarifier (PPC)
3 Double-deck rectangular clarifier (DDR-1)
4 " " " (DDR-2)
5 Perforated buffle plate single-deck rectangular clarifier (PSR)

Outlines of these clarifiers are shown in Fig. 1.1 ∿ 1.5 and specifications are listed in Table 1.

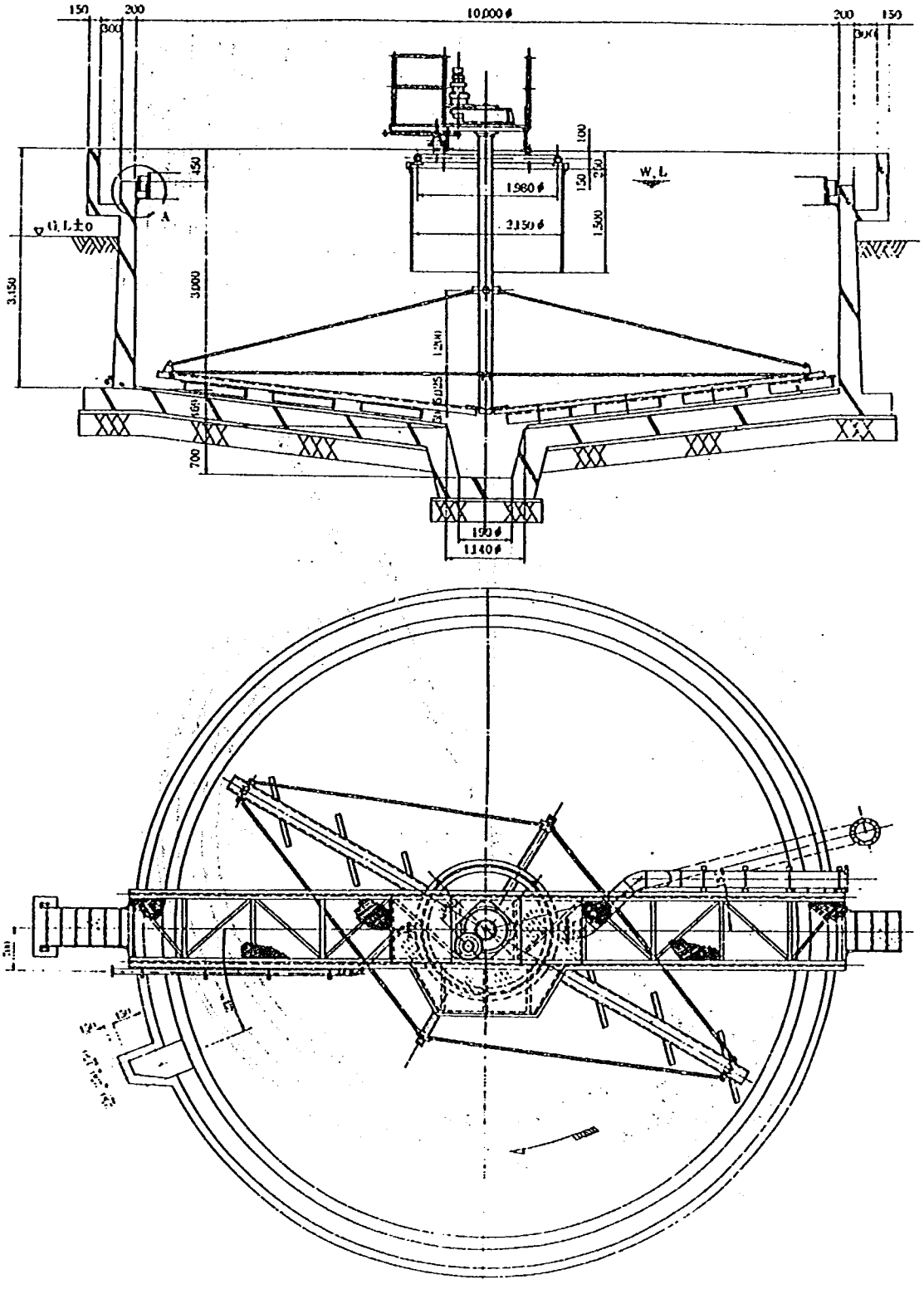


Fig. 1.1 Center-feed Peripheral-out circular clarifier (CPC)

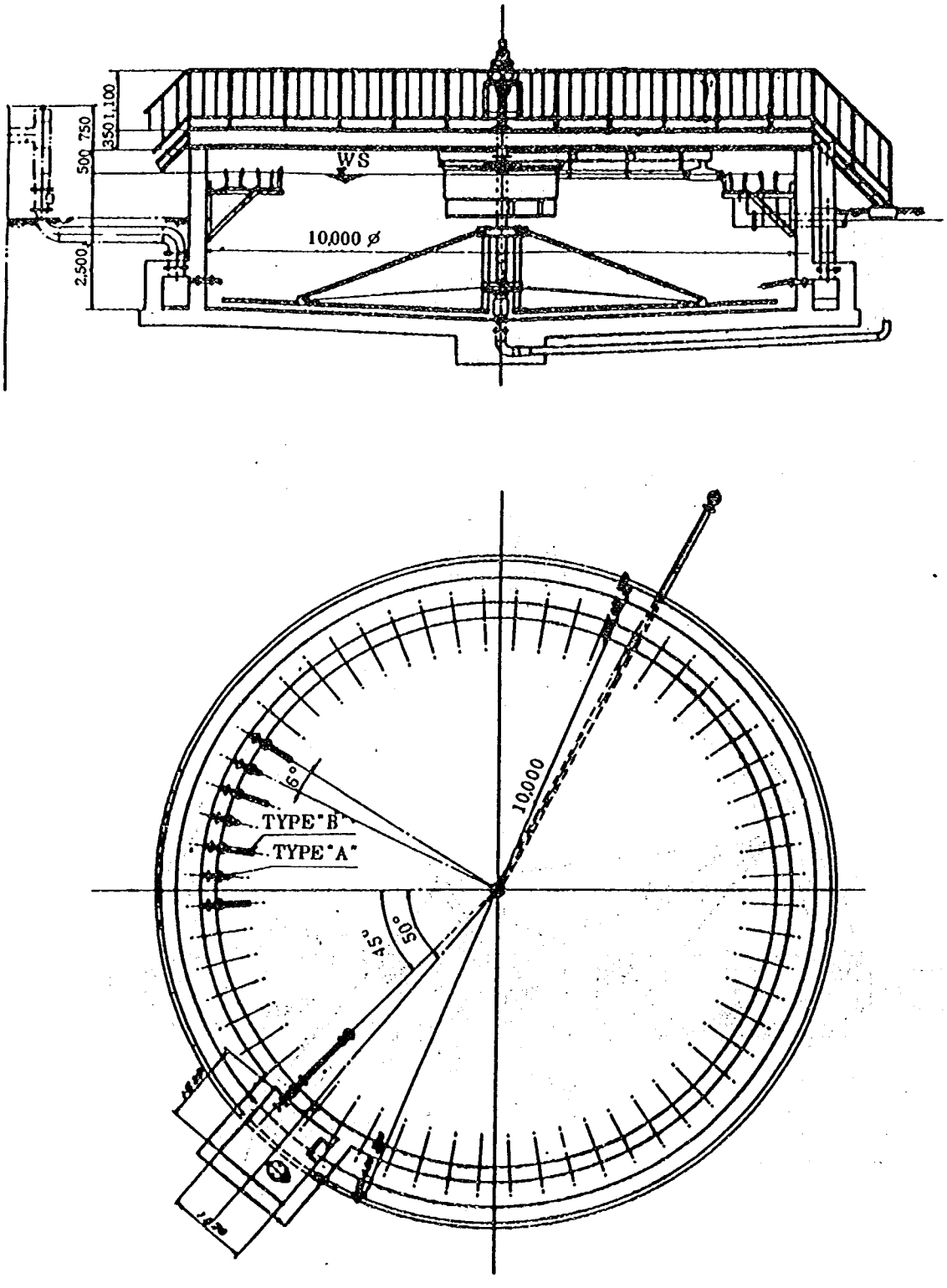


Fig. 1.2 Peripheral-feed Peripheral-out Circular clarifier (PPC)

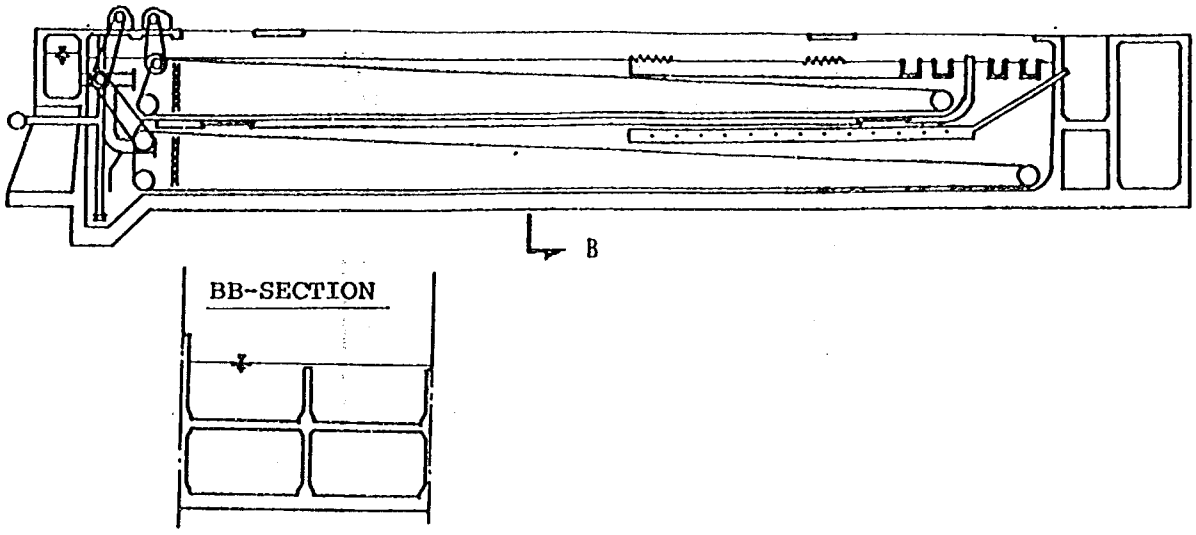


Fig. 1.3 Double-Deck Rectangular clarifier (DDR-1)

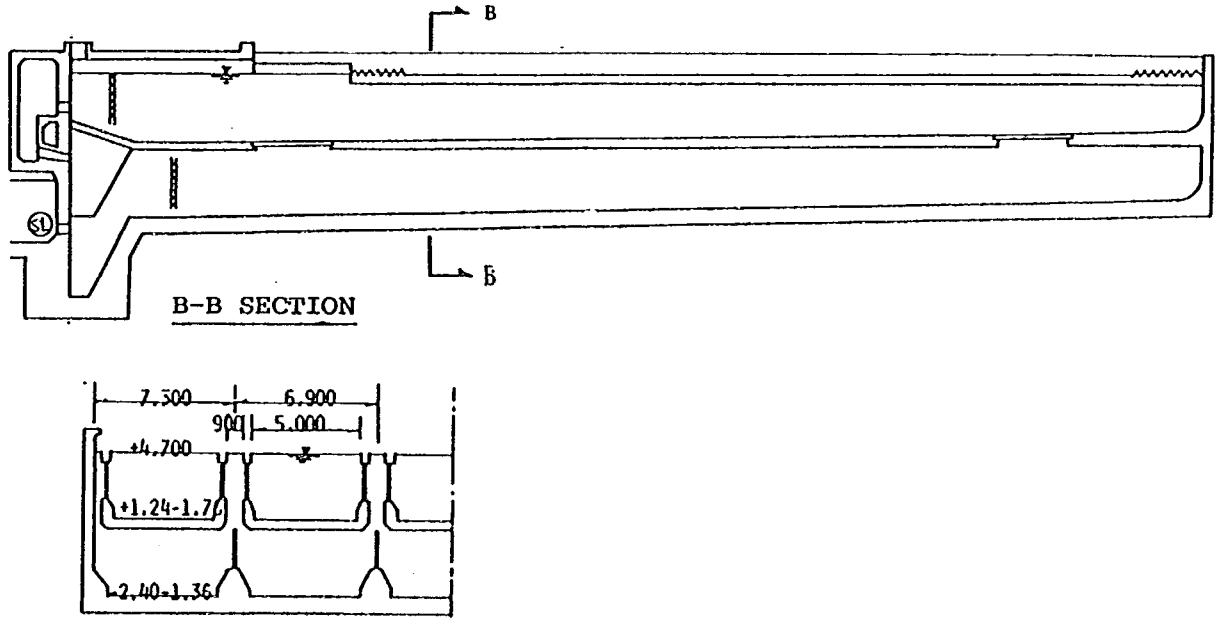


Fig. 1.4 Double-Deck Rectangular clarifier (DDR-2)

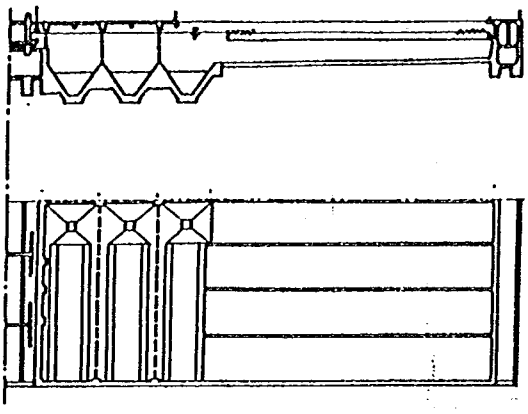


Fig. 1.5 Perforated baffle plate Single-deck Rectangular clarifier (PSR)

Table-1 Specification of clarifiers to be tested and operational condition

Parameter	Unit	Type of Clarifier						
		C.P.C.	P.P.C.	DDR - 1		DDR - 2		PSR
				Upper Sect.	Bottom Sect.	Upper Sect.	Bottom Sect.	
Diameter	m	10	10					
Length	m			39.7	41.4	55	55	47
Width	m			4.6 x 4	4.6 x 4	5 x 5	6.9 x 5	4.6 x 4
Depth	m	3	2.5	2.7	2.7	3.2	3.0	3.3
L/W				8.1	8.8	11.0	8.0	10.2
Surface Area	m^2	78.5	78.5	1,550		3,790		865
Volume	m^3			4,240		11,765		2,730
Flow Distributer		Center Well	60 Pipes	1 Baffle Plate		1 Baffle Plate		3 Baffle Plates
Sludge Coll.		Rakes	Siphon	Longitudinal Link-belt		Longitudinal Link-belt		Lateral Link-belt
Overflow Rate	$m^3/m^2/day$	24.4 ∿ 40.6	24.4 ∿ 40.6	22.5 ∿ 48.2		18.3 ∿ 26.4		9.4 ∿ 59.5
Weir Rate	m^3/m /day	66 - 110	17 - 24			82 - 121		70 - 204
Detention	Hours	1.9 ∿ 3.1	1.6 ∿ 2.6	1.4 ∿ 2.9		2.8 ∿ 4.1		1.3 ∿ 8.5
MLSS	g/ℓ	2.5 ∿ 4.8	1.5 ∿ 6.0	0.7 ∿ 1.6		1.2 ∿ 1.3		0.8 ∿ 2.4
Solid Load	$kg/m^2/day$	88 - 210	60 - 210	22 - 82		27 - 43		21 - 145

2. PARAMETERS TO INDICATE PERFORMANCE

Liquid in the clarifier does not flow as theory demands, and the real flow pattern is far from ideal due to eddy currents and/or density flow. Some flow reaches the clarifier outlet in less than theoretical detention time, and some takes much longer.

The degree of deviation from ideal flow can be evaluated by adding dye at the clarifier inlet and recording the time required for it to reach the outlet. A typical relationship between dye concentration at the outlet vs. elapsed time is shown by "Fair" in Fig. 2.1.

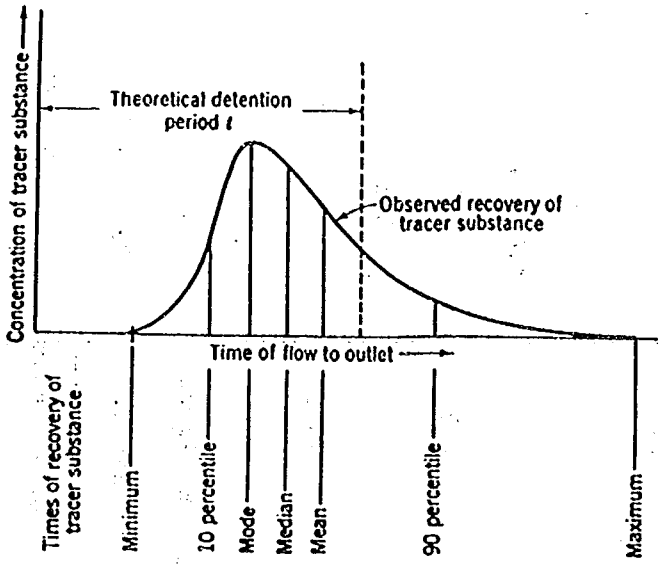


Fig. 2.1 Concentration and time of recovery of tracer substance

From data of dye tests on the different clarifiers (1) Minimum arrival time, (2) Mean retention time, (3) Variance, and (4) Stabilizing coefficient (Fair's n) were determined. These were used to evaluate clarifier performance.

2.1 MINIMUM ARRIVAL TIME: t_i

Minimum retention time, ti, is recorded when the first measurable amount of dye arrives at the outlet. Thus t_i is an index of the shortest retention time, or short circuit.

2.2 MEAN RETENTION TIME: t_m

Dye concentration, c, at the outlet is observed at time t. Mean retention time is defined by Eq. 1:

$$t_m = \frac{\Sigma t \cdot c}{\Sigma c} \qquad \text{Eq. 1}$$

The variance, σ^2, of the dye concentration-elapsed time curve is determined by Eq. 2:

$$\sigma^2 = \frac{\Sigma t^2 c}{\Sigma c} \Big/ \left(\frac{(\Sigma tc)^2}{\Sigma c} - 1 \right) \qquad \text{Eq. 2}$$

2.3 FAIR'S n - STABILITY INDEX

Fair showed that removal rates of solids whose settling velocity is V_o is given by Eq. 3:

$$E = 1 - \left(1 + \frac{1}{n} \frac{V_o}{Q/A} \right)^{-n} \qquad \text{Eq. 3}$$

Where, Q : Influent flow rate

A : Surface area of the clarifier

"n" is the so-called dumping coefficient. The higher n is, the better the suppression of turbulence that retards settling. The n is related to the variance σ^2 by the following equations:

$$n = \frac{1}{2d} \qquad \text{Eq. 4}$$

$$\sigma^2 = 3d^2 + 2d \qquad \text{Eq. 5}$$

2.4 DEAD ZONE INDEX

Spaces in a clarifier where liquid rotates currently up and down are not effective to solids separation. This is related to "dead space" which is defined by Eq. 6:

$$D_r = 1 - \frac{t_m}{T} \qquad \text{Eq. 6}$$

Where, T : Theoretical retention time

The dead space ratio, D_r, indicates volumetric efficiency.

3. DYE EXPERIMENT

Uranine dye was used as the tracer substance. The test was carried out under the different overflow rates of the five clarifiers. Influent flow was kept at a given rate for an hour before the dye test began.

Every three minutes, influent and effluent were sampled. Suspended solids were removed from the sample liquid by a centrifuge at 3000 rpm for 5 min. Then uranine concentrations were determined with a fluorospectro photometer.

4. RESULT

4.1 SHORT CIRCUIT

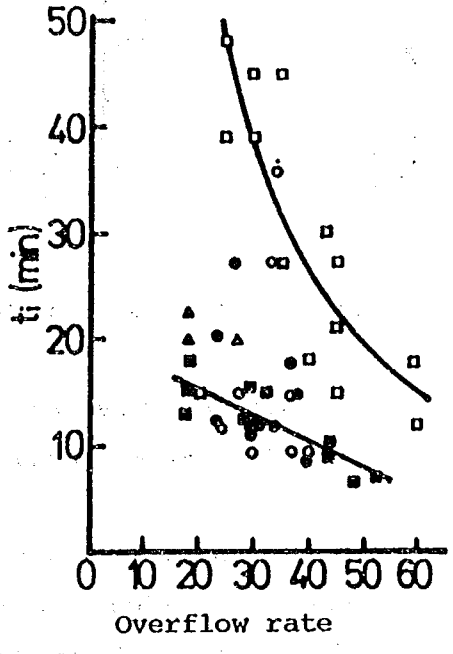


Fig. 4.1 Short circuit

Fig. 4.1 shows the relationship between minimum retention times and overflow rates for the clarifiers. At a given overflow rate, the perforated baffle plate single-deck rectangular clarifier (PSR) apparently has a much larger t_i than the other clarifier. The PSR is least subject to the short cycle.

4.2 MEAN RETENTION TIME/DEAD SPACE

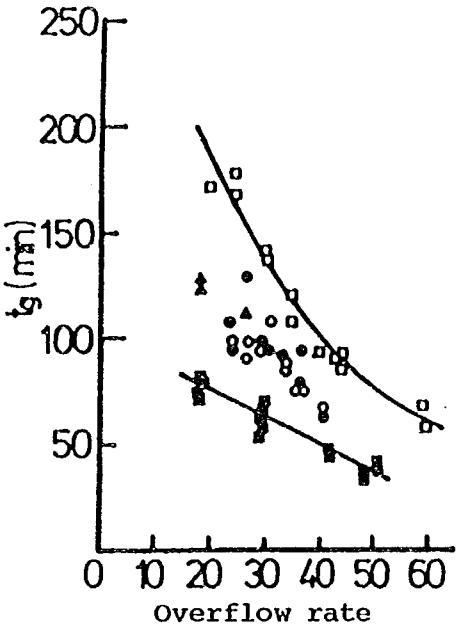


Fig. 4.2 Mean retention time

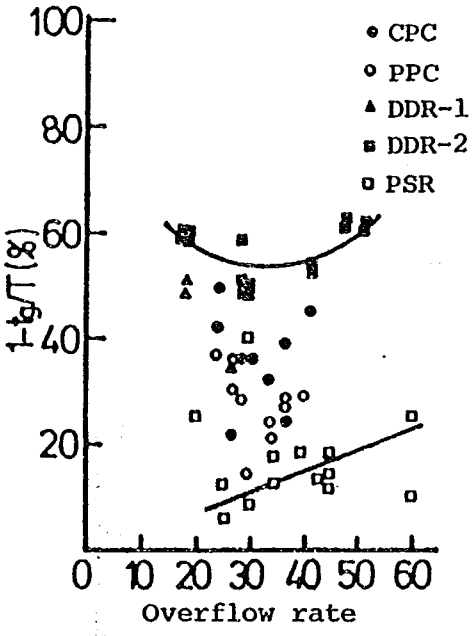


Fig. 4.3 Dead space ratio

At any given overflow rate, the PSR has the longest mean retention time while the double-deck rectangular clarifier No. 1 (DDR-1) has the shortest.

The dead space ratio (Fig. 4.3) indicates that the PSR has the highest volumetric efficiency, while the peripherical-feed peripheral-out circular clarifier (PPC) second, the center-feed peripheral-out circular clarifier (CPC) third, the DDR-1 fourth and the DDR-2 last.

Dead space occupies 45∿60% of total space in the DDR-2, but only 2∿20% in the PSR.

4.3 STABILIZATION

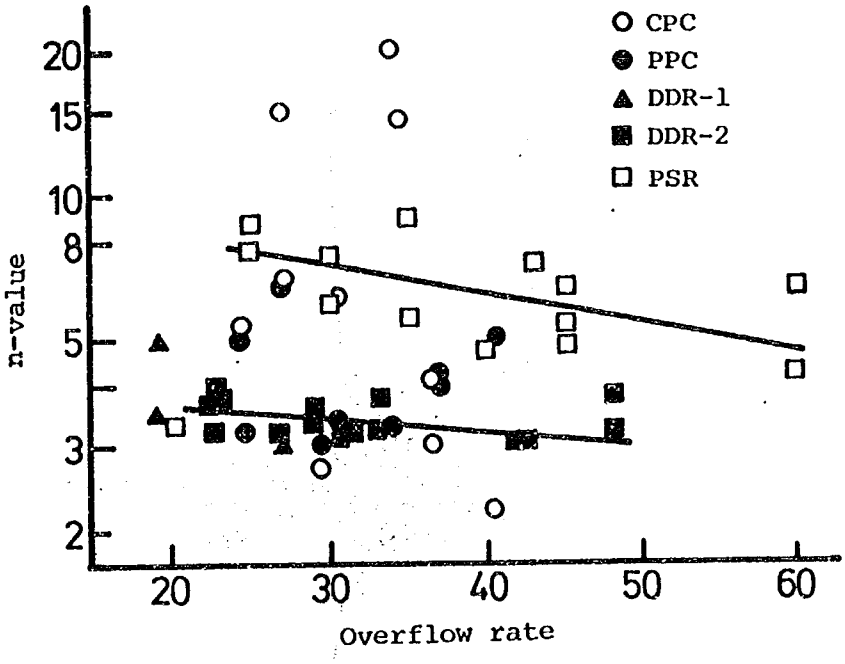


Fig. 4.4 Fair's n (stabilization index)

Fig. 4.4 shows the relationship between Fair's n and overflow rate for the five clarifiers. In general, an n value greater than 4 indicates good performance.

At the overflow rate of 30 m^3/m^2/day, for the PPC is very high, but it drops sharply with increases in the overflow rate. At an overflow rate of 40 m^3/m^2/day, falls below 2.2. Thus this type of clarifier is greatly affected by the influent flow rate.

The CPC remains relatively steady with a wide range of overflow rates, but it is not high. This is also the case with the two DDR units.

The PSR shows the highest n values. It is superior to the others in terms of steady performance.

4.4 SUSPENDED SOLIDS REMOVAL

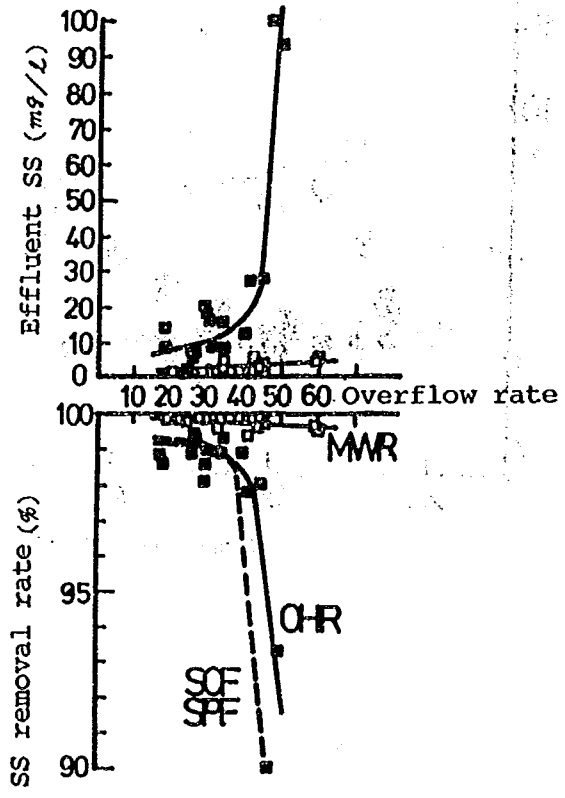


Fig. 4.5 SS Removal

In Fig. 4.5, the suspended solids (SS) removal rate is plotted against overflow rate. The CPC, PPC and DDR-2 clarifiers have very similar SS removal vs. overflow rate curves, showing rapid decreases in the SS removal rate with loading over 40 $m^3/m^2/day$.

However, the PSR's SS removal rate remains virtually constant up to loads of 60 $m^3/m^2/day$.

5. TEST OF PERFORATED BAFFLE PLATE SINGLE-DECK RECTANGULAR CLARIFIERS

All tests indicated that PSR, exhibits flow conditions most similar to those on which the settling theory is based. It appears that perforated baffle plates work effectively. To determine how the baffles reduce eddy and/or density currents, two pilot units were built.

Side panels of the pilot clarifiers were made of transparent plastic, through which the inside of the clarifier was easily observed. The dimensions of the units were as follows: 1 m in height, 1 m in width, 15 m in length.

The No. 1 pilot clarifier was equipped with three perforated baffles, each placed across the tank at right angles to the direction of flow. Each plate had open areas totaling about 5%. These plates divided the tank into four small compartments.

The No. 2 pilot clarifier was equipped with only one plate. The bottom of the plate was cut off so freight-conveyer could pass through. Thus open area totaled 40%.

At the beginning, the tanks were filled with tap water and then mixed liquor was fed through a pipe. The tap water was gradually replaced by the mixed liquor. Effluent flowed over troughs at the ends of the clarifiers. The influent rates were so fixed that overflow rates were 30 $m^3/m^2/day$.

Intersurfaces between tap water and mixed liquor were continuously recorded on videotape and periodically recorded by a still camera. Shapes and locations of intersurfaces vs. elapsed time are illustrated in Fig. 5.1 and 5.2.

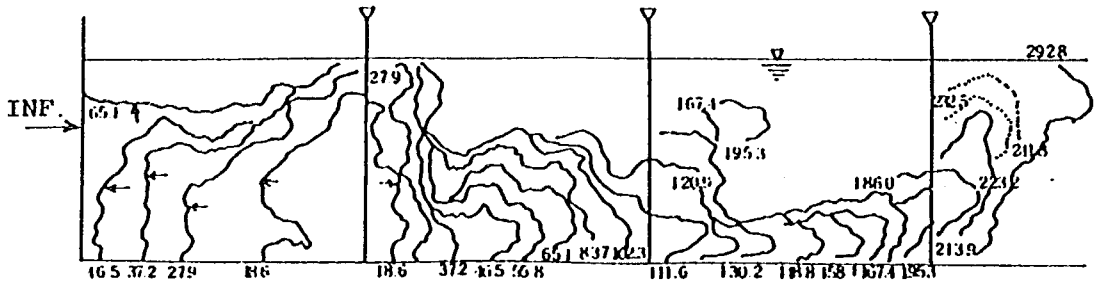


Fig. 5.1 Density current, 3 baffle plates, opening 5%

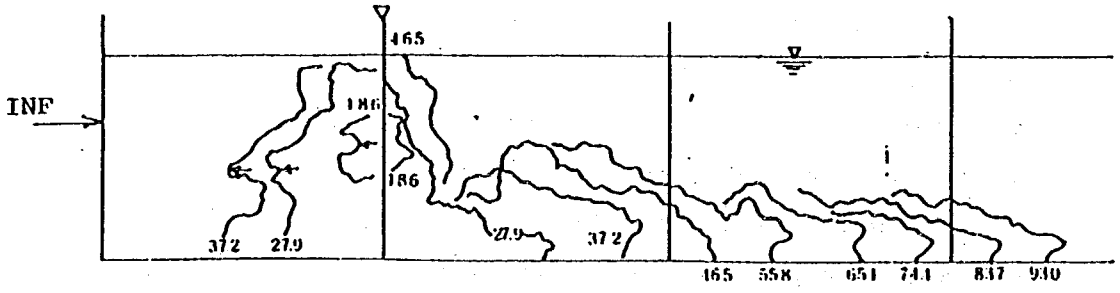


Fig. 5.2 Density current, 1 baffle plate, opening 40%

5.1 NO. 1 CLARIFIER: THREE BAFFLE PLATES

The influent mixed liquor immediately met the No. 1 baffle. Although, small amounts of influent returned to the inlet in the first compartment, the first baffle equalized current just after the plate, with flow velocity uniform from bottom to surface.

Density current appeared in the second compartment and creeped on the bottom of the clarifier to the second baffle. But the second plate retarded the current and helped to equalize flow distribution. Thus the plates apparently increasing Fair's cell number.

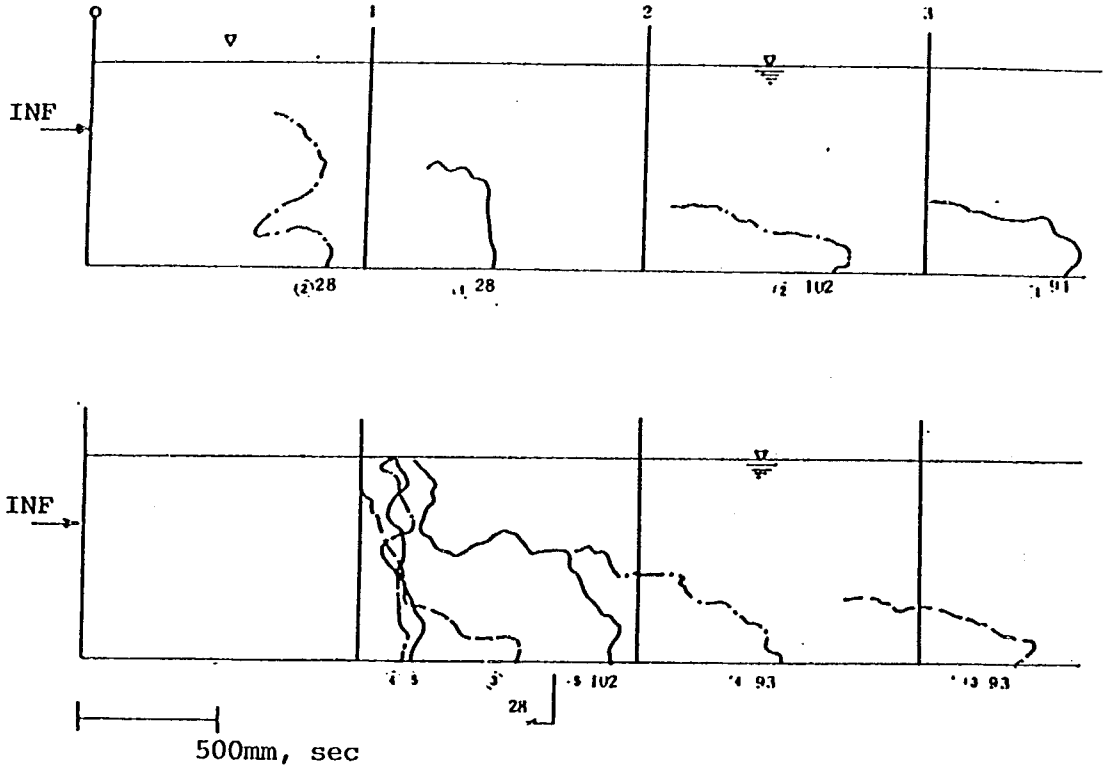
5.2 NO. 2 CLARIFIER: ONE BAFFLE PLATE

Although one baffle plate distributed flow fairly well, density current appeared after the plate and reached the outlet where it turned upward. Apparently the short circuit was dominant.

5.3 OPTIMUM OPEN AREA AND NUMBER OF BAFFLE PLATES

With different numbers of plates (and different open areas), the same experiments were continued. Results are shown in Fig. 5.4.

The greater the number of plates, and the smaller the open area, the better the suppresion of density currents.



	Opening ratio	Number of baffle plates	Location
1		0	0.5 m
2	40%	1	1
3	40%	1	1
4	5%	1	1 ∿ 3

Fig. 5.4 Density current vs. open area and numbers of plates

6. DISCUSSION

In the past, to increase settling efficiency or save space, much effort was placed on decreasing overflow rates and increasing the surface area of clarifiers. Double-deck rectangular clarifiers were typical examples of this effort.

Although this emphasis followed settling theories, DDR performance was not satisfactory. Although surface area was doubled with the twin-deck units, settling efficiency was not.

This study shows clearly that it is more important to provide some means to prevent density currents. The perforated baffle plates are a solution for this problem. The baffles divide a clarifier into small cells, helping bring the clarifier basin close to Hazen's assumption that suspended solids are distributed uniformly throughout the basin.

One of the problems associated with baffles placed across the basin is how to collect sludge. The sludge is usually collected by freight-conveyers moving from the outlet side of a rectangular clarifier to the inlet. But, longitudinal collection can't be used when baffles are installed.

Lateral collection is the logical alternative, but this is a relatively new concept and may not be easily applied. Tokyo has one example of this system in the Morigasaki Sewage Plant, and it works satisfactorily.

7. CONCLUSION

A. According to Fair's n, the following was found:

1 Center-Feed Peripheral-Out Circular Clarifier

Depending on overflow rate, n fluctuates widely. This kind of clarifier is susceptible to density current and thus often exhibits poor settling.

2 Peripheral-Feed and Peripheral-Out Circular Clarifier

At design flow, n is very high. But it drops sharply with an increase in overflow rate. Accordingly, at a constant influent rate, high efficiency is expected.

3 Rectangular Clarifier with Perforated Baffle Plates

This seems to have special promise, since n remains high with a wide range of overflow rates. This capability is confirmed by the SS removal rates of existing large clarifiers.

B. The smaller the open area of baffles, the better is density current retardation. A 5% open area is satisfactory. Baffles with large openings in the bottom portion rather increase density current.

C. Hydraulic loading can be increased up to 60 $m^3/m^2/day$ without reducing the suspended solids removal rate, if three baffles with 5% open area are placed in a rectangular clarifier.

CONTROLLING ODOROUS SUBSTANCES IN WASTEWATER TREATMENT PLANTS

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1. INTRODUCTION

Japan has a population of 117 million (as of 1980) on a relatively small land area of 372,000 km^2 (population density = 315 persons/km^2), and is among the most densely populated countries in the world. About 70% of the land area is comprised of uninhabitable mountains. High economic growth that started in the 1960s has increased the gravitation of people and industries to cities, sending up population density in urban areas. Consequently, various forms of pollution have become serious social problems.

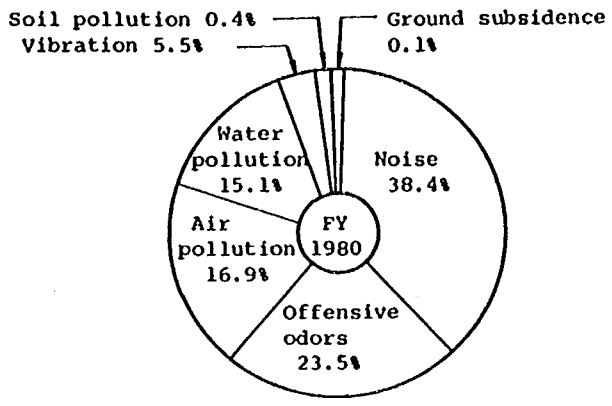
Because of the limited availability of land, wastewater treatment plants are often built in or near residential areas. Neighboring residents always lodge complaints about offensive odors from these plants, but no plant design has eliminated this pollution. In fact, odors have become a serious problem demanding urgent corrective measures. Although this situation is often made light of as a nuisance and not a hazard, the number of complaints about offensive odors is second only to that of complaints about noise. This means people are much interested in the odor problem.

Concerned officials initiated offensive odor control measures. Since in fiscal 1972 the number of complaints has decreased at a slow but steady rate (see Figs. 1 and 2). There is no significant difference in the ratio of complaints about offensive odors to the total number of complaints about public nuisances between various scales of urban areas (see Fig. 3).

According to surveys conducted by the Environment Agency, complaints about offensive odors are classified by types of industry as shown in Fig. 4.

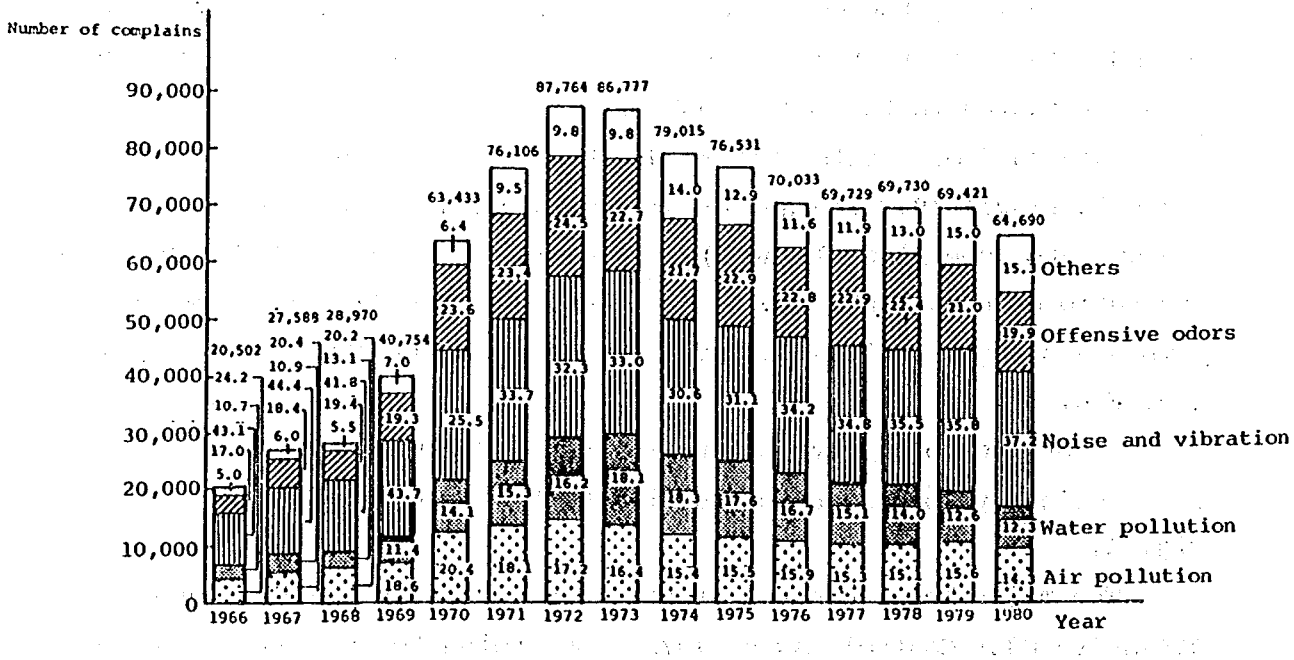
The major legal control of offensive odors is provided by the Offensive Odor Control Law of 1971. According to the law, prefectural governors are obliged to designate "Regulated Areas" where it is necessary to prevent offensive odors for the protection of amenities and to limit the emission concentration of specific odorous substances from the factories and places of business within the areas.

While wastewater treatment plants and pump stations would normally be classed with the factories in the law, sewer lines were made exempt.



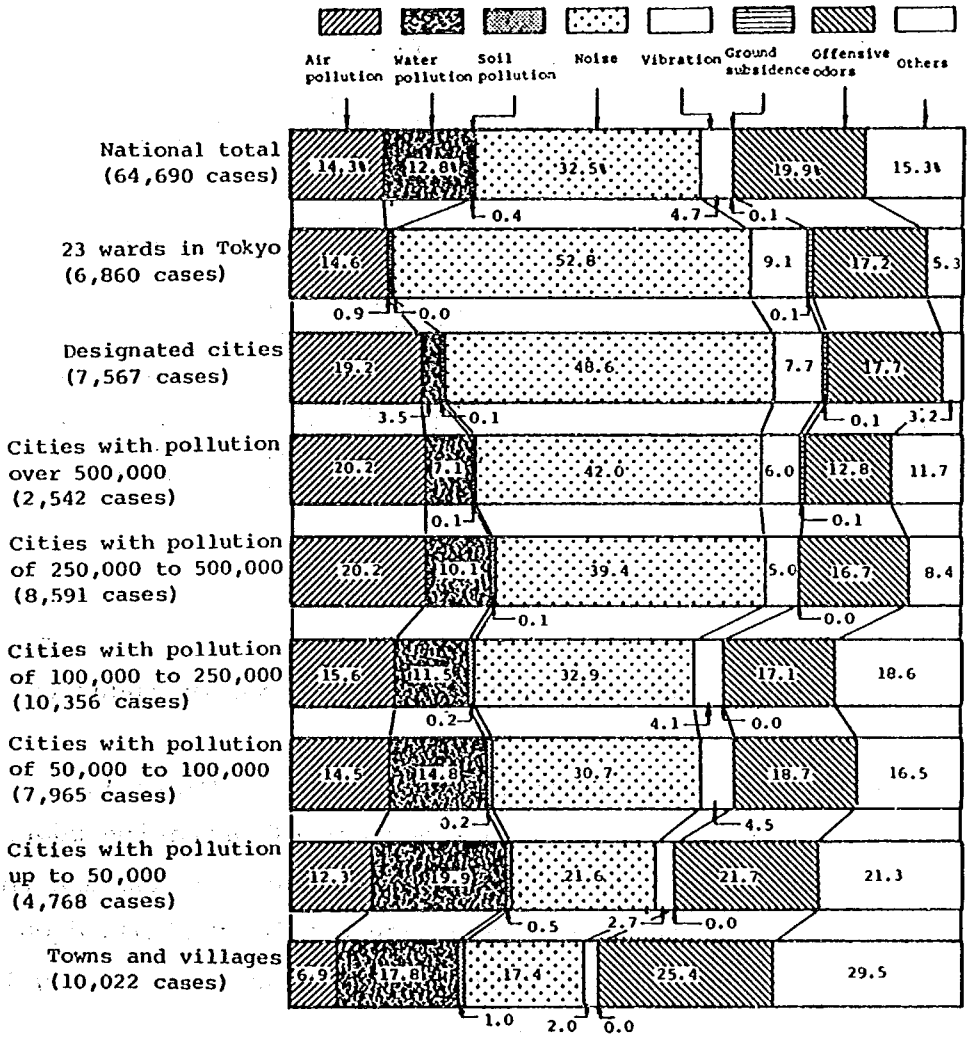
Note: Survey by Pollution Control Council

Fig. 1 Breakdown of complaints about seven typical public nuisances



Note: Survey by Pollution Control Council

Fig. 2 Annual transition and component ratios (%) of complaints about public nuisances



Note: Survey by Pollution Control Council

Fig. 3 Ratios of complaints about public nuisances by urban area

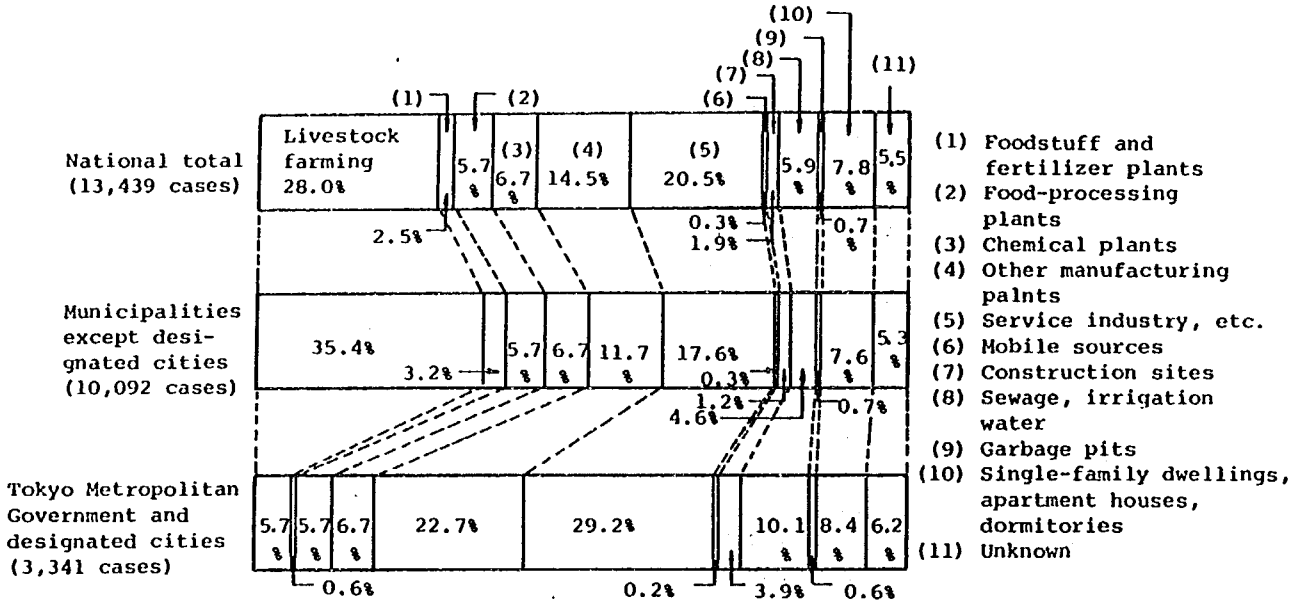


Fig. 4 Complaints about offensive odors by types of industry

Legal limits on odorous substances are given in Table 1.

These substances are the principal sources of offensive odors, and have been listed since methods have been established to measure their concentrations in air. The limits have been established based on those concentrations which the majority of people in regulated areas will accept.

Thus prefectural governors are obliged to set concentration limits on odorous substances within 2.5 to 3.5 points on the six-grade of odor strength (see Tables 1 and 2).

The Environment Agency has so far advised prefectural governments to limit concentrations to 2.5 to 3 points in residential areas and to 3.0 to 3.5 points in industrial areas. (Most governments have set limits at 2.5 points for residential areas and 3.0 or 3.5 points for industrial areas.)

Table 1 Odorous substance limits according to Offensive Odor Control Law

Odorous substances	Limits (air-borne concentration at plant boundary (ppm))
Ammonia	1 to 5
Methyl mercaptan	0.002-0.1
Hydrogen sulfide	0.02-0.2
Dimethyl sulfide	0.01-0.2
Trimethylamine	0.005-0.07
Dimethyl disulfide	0.009-0.1
Acetoaldehyde	0.05-0.5
Styrene	0.03-20

The law says that limits are to be established in terms of:

1. Concentration in the air at the plant boundary.
2. Concentration of gases at the stack outlet, etc.
3. Concentration in industrial effluent.

There is yet no practical method of calculating concentration for No. 3. Concentration limits for No. 2 are set to maintain satisfactory ground level concentrations at the plant boundary.

While the Offensive Odor Control Law has been effective to some extent, there still is much left to be desired. One problem concerns methods of measuring odorous substances. Usually an odor is sensed even when its concentration is very low, so the personal judgement of local inhabitants often comes into conflict with instrument measurements.

Because of this, odor control by organoleptic testing is being considered instead of instrument analysis of the eight listed odorous substances.

Table 2 Odorous substances and their odor strength

Odor strength / Odorous substances	1	2	2.5	3	3.5	4	5
Ammonia	ppm 0.1	ppm 0.6	ppm 1	ppm 2	ppm 5	ppm 1x10	ppm 4x10
Methyl mercaptan	0.0001	0.0007	0.002	0.004	0.01	0.03	0.2
Hydrogen sulfide	0.0005	0.006	0.02	0.06	0.2	0.7	8
Dimethyl sulfide	0.0001	0.002	0.01	0.05	0.2	0.8	2
Dimethyl disulfide	0.0003	0.003	0.009	0.03	0.1	0.3	3
Trimethylamine	0.0001	0.001	0.005	0.02	0.07	0.2	3
Acetoaldehyde	0.002	0.01	0.05	0.1	0.5	1	1x10
Styrene	0.03	0.2	0.4	0.8	2	4	2x10

Table 3 Six-grade of odor strength

Odor strength	Description
0	No odor
1	Barely detectable (detection threshold level)
2	Barely identifiable level of odor (identification threshold level)
3	Easily identifiable
4	Strong odor
5	Very strong odor

Among these methods, the triangle air bag method (a kind of air dillution method) developed by the Tokyo Metropolitan Government is considered quite effective, and some local governments have already specified it in pollution control regulations.

Another problem concerns the control of odorous substances discharged with industrial effluent, since no practical method of measuring odorous substances evaporated from industrial effluent has been established. In addition, the relationship between these measurements, if possible, and the limits at the plant boundary has yet to be clarified.

2. ODOROUS SUBSTANCES DEVELOPED BY WASTEWATER TREATMENT PLANTS

Identification of odorous substances and their sources is very important for the formulation of odor control measures. Table 4 shows the concentrations of odorous substances at three wastewater treatment plants. The features of the three plants are as follows:

A: A medium-size plant processing an average daily flow of about 65,000 m^3 of domestic wastewater (including industrial effluent, which accounts for 4%). Night soil is not charged into the system. It uses the anaerobic sludge digestion process.

B: The average daily flow is about 79,000 m^3/day.
Industrial effluent accounts for 16%.
Sludge from small flow plants is charged into the primary sedimentation tank, and its influence on odor emission is significant. The sludge is heat-treated, causing odor emission. For this reason, various odor control measures are taken.

C: Average daily flow: 22,000 m^3/day
The plant is affected by industrial effluent, etc. Night soil is charged into the digestion tank, and its supernatant is returned to the grit chamber.

As shown in Table 4, the highest odor concentrations are in the sludge thickener (At the B wastewater treatment plant, samples were not taken in the thickener). With the exception of the B plant, where night soil is charged into the primary sedimentation tank, odor concentrations at points in the treatment process other than the thickener are less than 1,000. In ordinary plants, odor concentrations rarely exceed this value.

In those plants where special influents are treated, odor concentrations easily exceed 1,000. In many cases, odors generated occasionally are the worst. For example, odors generated during sludge removal from the grit chamber, transportation of screenings, and shipment of sludge into the sludge thickener.

Table 5 shows the odor threshold values of major sewage-derived odorous substances. The values were determined according to The Triangle Air Bag Method.

Table 4 Odorous substance concentrations at three wastewater treatment plants, unit in ppb except for order concentration

No.	Name		Symbol / Sampling position / Name of wastewater treatment plant / Chemical formula	1 Grit chamber				2 Primary sedimentation tank				3 Aeration tank				4 Sludge thickener				5 Dewatering machine			
				A	B*	C (winter)	C (summer)	A*	B	C (winter)	C (summer)	A*	B*	C (winter)	C (summer)	A*	B**	C (winter)	C (summer)	A	B***	C (winter)	C (summer)
Odoc concentration				160	130	310	730	62	13,000	98	550	170	640	73	73	7,300	1,700	13,000	55,000	Up to 10	160	Up to 10	10
1		Ammonia	NH_3	88	79	95	230	134	290	32	270	56	120	3	110	68	210	69	780	1,890	140	308	860
2	Sulfur compounds	Hydrogen sulfide	H_2S	10	30	92	150	12	3,540	11.5	230	4	300	-	N.D.	500	800	65,400	22,600	7	28	1.7	4
3		Methyl mercaptan	CH_3SH	T.r.	5	18	4	T.r.	176	4.6	3	T.r.	60	-	T.r.	60	T.r.	328	N.D.	N.D.	T.r.	0.3	T.r.
4		Dimethyl sulfide	$(CH_3)_2S$	T.r.	T.r.	4	T.r.	N.D.	61	12.6	2	T.r.	64	-	5	N.D.	N.D.	59.4	N.D.	N.D.	N.D.	1.5	T.r.
5		Dimethyl disulfide	$(CH_3S)_2$	1	N.D.	4	N.D.	1	10	N.D.	T.r.	N.D.	22	-	T.r.	4	N.D.	N.D.	N.D.	N.D.	N.D.	N.D.	N.D.
6	Lower fatty acids	Acetic acid	CH_3COOH	1.7	-	1.2	5.3	7.1	-	T.r.	2	1.4	-	0.7	11.9	1.2	-	N.D.	2.2	2.1	-	T.r.	2.4
7		Propionic acid	CH_3CH_2COOH	0.2	-	0.2	1	0.9	-	N.D.	3	0.1	-	0.1	0.9	0.2	-	0.1	0.3	0.3	-	0.1	0.2
8		i-buthyric acid	$(CH_3)_2CHCOOH$	0.1	-	0.1	0.6	0.8	-	0.3	2.1	2	-	0.3	1	N.D.	-	0.1	0.3	N.D.	-	0.1	0.2
9		n-pentanoic acid	$CH_3CH_2CH_2COOH$	0.3	-	0.6	1.5	1.3	-	0.1	1.1	N.D.	-	0.1	2.6	0.2	-	0.2	0.4	0.3	-	0.2	0.2
10		i-pentanoic acid	$(CH_3)_2CHCH_2COOH$	0.1	-	N.D.	N.D.	N.D.	-	N.D.	0.3	0.2	-	N.D.	T.r.	0.1	-	N.D.	N.D.	T.r.	-	N.D.	0.1
11		n-pentanoic acid	$CH_3(CH_2)_3COOH$	T.r.	-	T.r.	0.3	0.1	-	T.r.	0.7	T.r.	-	T.r.	0.2	T.r.	-	N.D.	N.D.	0.1	-	0.1	0.2
12	Low-boiling point compounds	i-pentane	$(CH_3)_2CHCH_2CH_3$	5.8	9.7	1.8	1.8	6.3	13.1	2.5	2.6	3.7	14.9	1.3	-	4.9	5.0	0.7	6.8	4.7	3.0	11.7	1.2
13		n-pentane	$CH_3(CH_2)_3CH_3$	2.7	5.5	2.5	1.8	3.2	5.8	4.6	1.7	1.8	4.6	2.5	-	3.4	2.9	0.9	3.7	2.1	1.7	6.3	0.7
14		Methyl mercaptan + 2-methyl pentane	$CH_2SH+(CH_3)_2CH(CH_2)_2CH_3$	1.9	2.6	4.8	1.2	2.2	33.7	-	1.7	1.9	45.8	-	-	14.2	3.0	-	5.1	1.7	0.6	6.0	0.6
15		3-methyl pentane	$(CH_3)_3(CH)_2CH$	1.0	1.6	1.3	0.7	1.5	1.5	-	1.1	1.9	2.3	-	-	1.9	1.2	1.4	3.6	0.1	0.4	4.0	19.6
16		n-hexane	$CH_3(CH_2)_4CH_3$	5.0	2.3	11.9	6.8	6.7	2.2	12.3	3.4	8.0	2.6	97.2	-	1.0	2.4	1.8	5.2	2.5	0.7	7.0	-
17		Methyl cyclopentane	$C_5H_8CHCH_3$	N.D.	-	2.1	0.6	N.D.	16.2	6.8	1.0	N.D.	17.2	3.3	2.2	0.8	0.7	-	2.3	N.D.	0.6	2.0	1.8
18		Methyl sulfide + 2-methyl hexane	$(CH_3)_2S+(CH_3)_2CH(CH_2)_2CH_3$	1.1	1.2	0.8	0.3	1.3	7.0	-	0.9	1.5	13.4	-	0.4	3.9	-	-	4.8	1.1	-	1.7	-
19		3-methyl hexane	$(CH_3)_3CH(CH_3)_3$	0.8	1.3	1	0.3	3.5	1.2	-	1.2	0.8	5.2	-	0.4	1.6	0.8	0.3	6.7	0.8	0.5	2.7	1.1
20		n-heptane	$CH_3(CH_2)_5CH_3$	1.5	-	0.8	0.3	1.8	-	-	-	N.D.	-	-	-	2.3	-	0.4	-	1.5	-	2.0	0.1
21		Carbon tetrachloride + 1,1,1, trichloroethane	$CCl_4+CH_3CCl_3$	8.3	4.0	-	0.6	10.4	5.4	-	2.3	15.2	166	-	1.3	8.3	3.0	-	7.2	11.0	46.5	-	-
22		n-octane	$CH_3(CH_2)_6CH_3$	1.2	1.2	-	1.4	1.6	7.9	-	-	1.4	8.5	-	2.6	0.1	14.4	-	7.6	1.2	-	-	1.9
23		Benzene + chloroform	$C_6H_6+CHCl_3$	4.3	3.1	3.6	1.9	7.3	26.2	09.8	2.5	9.4	21.6	-	3.3	7.3	6.4	-	10.4	4.3	3.1	-	1.7
24		Trichloroethylene	Cl_2CCHCl	N.D.	-	12.8	3.8	N.D.	34.4	35.1	7.5	N.D.	48.3	26.1	6.7	23.0	-	15.4	-	N.D.	26.6	-	0.6
25		n-nanone (1)	$CH_3(CH_2)_7CH_3$	4.4	4.1	-	2.6	3.0	6.4	-	2.2	1.7	8.0	-	5.0	3.7	8.7	-	4.3	N.D.	12.3	-	-
26		Toluene (1)	$C_6H_5CH_3$	8.2	16.3	8.1	-	11.6	84.8	129	20.0	17.5	41.0	12	7.0	38.3	31.7	121	65.8	8.2	109	18.6	-
27		Tetrachloroethylene	Cl_2CCCl_2	N.D.	15.0	1.4	-	N.D.	45.6	20	-	N.D.	102	11.5	6.0	18.4	-	-	-	N.D.	-	-	-
28		2-methyl nanone	$CH_3CHCH_3(CH_2)_6CH_3$	3.7	5.5	-	-	1.9	-	-	1.4	1.3	3.3	-	2.5	4.7	1.4	-	1.6	0.7	3.0	-	-
29		3-methyl nanone	$CH_3CH_2CHCH_3(CH_2)_5CH_3$	2.1	1.7	-	-	0.4	-	-	0.9	1.6	1.4	-	2.4	1.7	-	-	1.1	N.D.	1.4	-	-

Table 4 (Continued)

No.	Name	Symbol / Sampling position / Name of waste-water treatment plant / Chemical formula	1 Grit chamber A	1 Grit chamber B*	1 Grit chamber C (winter)	1 Grit chamber C (summer)	2 Primary sedimentation tank A*	2 Primary sedimentation tank B	2 Primary sedimentation tank C (winter)	2 Primary sedimentation tank C (summer)	3 Aeration tank A*	3 Aeration tank B*	3 Aeration tank C (winter)	3 Aeration tank C (summer)	4 Sludge thickener A*	4 Sludge thickener B**	4 Sludge thickener C (winter)	4 Sludge thickener C (summer)	5 Dewatering machine A	5 Dewatering machine B***	5 Dewatering machine C (winter)	5 Dewatering machine C (summer)
Odor concentration			160	130	310	730	62	13,000	98	550	170	640	73	73	7,300	1,700	13,000	55,000	Up to 10	160	Up to 10	10
30		n-decane	$CH_3(CH_2)_7CH_3$	12.8	6.5	-	-	4.9	5.3	-	3.4	2.2	13.4	-	6.2	10.1	-	-	3.8	1.4	8.5	-	1.4
31		Ethyl benzene	$C_6H_5CH_2CH_3$	2.1	4.6	-	-	5.3	6.4	-	20.2	5.3	11.1	-	5.6	20.3	-	-	47.8	4.4	4.8	-	4.4
32		m, p-xylene	$C_6H_4(CH_3)_2$	4.6	5.6	-	-	6.2	8.5	-	28.3	6.7	10.1	-	8.2	19.8	-	-	143	5.0	7.7	-	3.2
33		o-xylene (1)	$C_6H_4(CH_3)_2$	1.9	3.0	-	-	4.8	8.0	-	13.8	3.3	7.0	-	5.2	6.6	-	-	48.4	2.4	5.0	-	1.2
34		n-undecane + Chlorobenzene	$CH_3(CH_2)_9CH_3+C_6H_5Cl$	18.6	9.9	-	-	2.7	4.5	-	2.5	3.9	0.8	-	-	5.4	3.0	-	3.2	1.0	4.8	-	1.3
35		Propyl benzene	$C_6H_5(CH_2)_2CH_3$	T.r.	-	-	-	T.r.	-	-	2.5	T.r.	-	-	-	1.3	6.4	-	8.4	T.r.	-	-	0.3
36		m, p-ethyl toluene	$C_6H_4CH_3CH_2CH_3$	T.r.	-	-	-	T.r.	11.0	-	18.2	T.r.	-	-	9.3	5.9	10.2	-	43.6	T.r.	-	-	1.2
37		o-ethyl toluene	$C_6H_4CH_3CH_2CH_3$	T.r.	-	-	-	T.r.	2.5	-	4.8	T.r.	-	-	2.6	1.9	4.2	-	13.1	T.r.	-	-	0.3
38		1,3,5-trimethyl benzene	$C_6H_3(CH_3)_3$	T.r.	-	-	-	T.r.	2.9	-	3.0	T.r.	-	-	1.6	2.2	2.3	-	9.0	T.r.	-	-	0.3
39		1,2,4-trimethyl benzene (1)	$C_6H_3(CH_3)_3$	T.r.	-	-	-	T.r.	4.9	-	21.1	T.r.	-	-	12.4	5.9	-	-	42.9	T.r.	-	-	2.9
40	High-boiling point compounds	Toluene (2)	$C_6H_5CH_3$	11.1	205	31.3	11.9	13.5	159	84.3	223	11.7	139	48.6	62.2	32.4	563	34.6	12.6	47.2	26.9	16.7	15.9
41		n-nanone (2)	$CH_3(CH_2)_7CH_3$	1.8	2.8	4.7	1.2	4.9	9.7	2.4	205	1.4	30.7	13	30.4	4.9	-	2.6	0.6	12.4	4.3	2.7	0.6
42		Ethyl benzene + m,p-xylene	$C_6H_5CH_2CH_3+C_6H_4(CH_3)_2$	2.7	42.7	27.8	9.3	4.9	35.7	18	2,104	7.6	26.9	35.9	119	40.2	35.5	20.8	8.8	24.2	27.4	31.1	1.6
43		n-decane (2)	$CH_3(CH_2)_8CH_3$	10.5	20.6	4.2	3.3	5.3	19.2	3.1	248	7.2	22.7	14.7	62.4	22.4	12.4	18.8	2.5	20.0	12.3	4.5	4.1
44		o-xylene (2)	$C_6H_4(CH_3)_2$	3.8	6.7	13.1	20.8	N.D.	15.8	6.9	491	N.D.	7.3	11.7	21.4	8.3	5.3	9.3	1.3	N.D.	3.0	8.8	2.1
45		Ethyl toluene + 1,3,5-trimethyl benzene	$C_6H_4CH_3C_2H_5+C_6H_3(CH_3)_3$	12.9	19.3	-	8.5	2.4	30.1	-	3,110	7.0	13.9	-	75.6	32.2	7.6	-	2.7	12.6	19.2	-	6.3
46		n-undecane + 1,2,4-trimethyl benzene	$CH_3(CH_2)_4CH_3+C_6H_3(CH_3)_3$	23.6	19.5	-	2.6	2.6	32.5	5.3	3,300	8.6	28.8	-	62.0	28.0	12.0	-	3.2	4.9	15.0	-	3.6
47		1,2,4-trimethyl benzene (2)	$C_6H_3(CH_3)_3$	3.5	3.6	-	5.6	N.D.	5.1	-	-	2.0	8.0	-	-	4.0	1.7	-	-	N.D.	-	-	5.6
48		n-dodecane	$CH_3(CH_2)_{10}CH_3$	19.4	9.2	5.7	2.6	1.0	10.8	3.4	41.2	5.4	12.2	7.7	14.1	6.5	4.5	9.5	-	1.5	2.4	2.9	2.7
49		n-tridecane	$CH_3(CH_2)_{11}CH_3$	10.1	3.5	2.5	6.8	0.3	4.3	1.2	48.8	3.7	2.8	2.3	11.3	5.4	1.9	1.0	1.6	2.2	0.7	0.9	3.9
50		n-tetradecane	$CH_3(CH_2)_{12}CH_3$	3.5	1.5	2.4	1.6	0.4	2.4	0.8	12.6	1.4	1.0	1.6	4.8	2.4	1.0	1.0	0.7	T.r.	1.2	1.1	0.8
51		n-pentadecane	$CH_3(CH_2)_{13}CH_3$	0.9	0.7	1.2	0.6	0.3	0.6	0.4	-	T.r.	1.3	0.9	1.9	0.4	1.1	0.3	0.4	T.r.	0.5	0.1	0.4
52		n-hexadecane	$CH_3(CH_2)_{14}CH_3$	0.3	-	0.4	1.0	T.r.	0.4	0.2	-	T.r.	0.8	0.4	1.6	0.2	0.8	0.2	-	T.r.	0.2	-	0.8
53		n-heptadecane	$CH_3(CH_2)_{15}CH_3$	T.r.	-	0.3	1.1	T.r.	-	-	-	T.r.	-	-	0.9	T.r.	1.4	0.1	-	T.r.	0.1	-	1.0
54		n-octadecane	$CH_3(CH_2)_{16}CH_3$	T.r.	-	-	2.8	T.r.	-	-	-	T.r.	-	-	-	T.r.	0.7	-	-	T.r.	0.1	-	1.8
55		n-nanodecane	$CH_3(CH_2)_{17}CH_3$	T.r.	-	-	-	T.r.	-	-	-	T.r.	-	-	-	T.r.	-	-	-	T.r.	-	-	-
56		n-eicosane	$CH_3(CH_2)_{18}CH_3$	T.r.	-	-	-	T.r.	-	-	-	T.r.	-	-	-	T.r.	-	-	-	T.r.	-	-	-

Remarks *: Sampling at a point 10 cm above the water surface, **: Sampling within the thickner, ***: Sampling in the sludge heat-treating chamber, No mark: Sampling from duct, T.r.: trace, N.D.: not detected, -: No measurement

Table 5 Odor threshold values of major odorous substances (Triangle Air Bag Method)

Odorous substance	Threshold value (ppb)
Ammonia	1,510
Hydrogen sulfide	0.33
Methyl mercaptan	0.07
Dimethyl sulfide	3.0
Dimethyl disulfide	2.2
n-buthyric acid	0.33
i-buthyric acid	0.23
Trimethyl amine	0.027
Acetoaldehyde	1.54
Acetic n-buthyl	27
Toluene	310
o-xylene	700
m-xylene	30
p-xylene	60
Methanol	33,000
Ethanol	520
Isobuthyl alcohol	11

Values vary widely depending on the measuring methods and groups of organoleptic inspectors.

The Triangle Air Bag Method used is based on the same principles as the measurement of odor concentrations, and is significant in that it integrates organoleptic and instrument test results.

According to Tables 4 and 5, the substances whose concentrations were above the odor threshold values are as follows:

Ammonia (1), hydrogen sulfide (2), methyl mercaptan (3), methyl sulfide (4), methyl disulfide (5), n-buthyric acid (9), i-buthyric acid (10), and toluene (26)

In Table 4, there are substances whose odor threshold values have not yet been determined, and it is now assumed that the concentrations of almost all these substances are below odor threshold values.

Those substances which were not measured and which are likely to cause odors include aldehydes, and where industrial effluent is predominant, styrene, xylene and benzenes.[3] In ordinary treatment plants, however, the eight substances cited previously are the main causes of odors.

Of the eight, six substances were studied to determine the correlation between organoleptic tests and instrument measurements. The six were ammonia, hydrogen sulfide, methyl mercaptan, methyl sulfide, and i- and n-buthyric acids. The concentrations shown in Table 4 for each process at the A and B plants were divided by the odor threshold values specified in Table 5, and the results were compared with odor concentrations (see Table 6).

The organoleptic test is influenced by the human equation, and its results usually contain errors of about one order. Table 6 shows that at the B treatment plant estimated odor concentrations show good agreement with measurements.

This is probably due to the fact that hydrogen sulfide likely to be derived from night soil is the main cause of odor at the B plant.

At the A wastewater treatment plant, on the other hand, there is a somewhat greater difference between estimated values and measurements. Originally, this plant had low odor concentrations. Because domestic wastewater is the main subject of treatment, there are no particularly high-concentration odorous substances to speak of.

The difference may have been caused by the multiplying effects of known substances or by unknown trace substances. At the aeration tank, the error is more than one digit, and may have been caused by odorous substances peculiar to the aeration tank.

At those wastewater treatment plants, such as A, where neither unusual industrial effluent nor night soil is processed, odors are created as a complex mixture of sulfur compounds such as hydrogen sulfide, methyl mercaptan, methyl sulfide, and methyl disulfide, lower fatty acids such as i-buthyric acid, and other undetected substances.

Table 6 Instrument analysis vs. organoleptic tests

Wastewater treatment plant	Sampling position	Component concentration/threshold value						Odor concentration	
		Ammonia	Hydrogen sulfide	Methyl mercaptan	Dimethyl sulfide	i-buthyric acid	n-buthyric acid	Estimate (max.)	Measure-ment
A	Grit chamber	<1	29	<1	<1	4	1	29	160
	Primary sedimentation tank	<1	35	<1	<1	3	4	35	62
	Aeration tank	<1	11	<1	<1	9	<1	11	170
	Sludge thickener	<1	1,514	545	<1	<1	<1	1,514	7,300
	Dewatering machine room	1	23	<1	<1	<1	1	23	Up to 10
B	Grit chamber	<1	92	45	<1	-	-	92	130
	Primary sedimentation tank	<1	18,182	2,727	28	-	-	18,182	13,000
	Aeration tank	<1	1,457	872	21	-	-	1,457	610
	Sludge thickener	<1	2,424	<1	<1	-	-	2,424	1,700
	Heat treatment room	<1	84	<1	<1	-	-	84	160

At treatment plants with night soil charging facilities (such as the primary sedimentation tank of the B wastewater treatment plant and the digestion tank of the C wastewater treatment plant), it was found that odor concentrations become high at, for example, the grit chamber into which supernatant of night soil flows, and the main odorous substances are hydrogen sulfide and methyl mercaptan.

Table 7 compares the odorous substances generated from the aeration tank of the conventional activated sludge process with those of the other processes.

Table 7 Comparison of odorous substances from various biological treatment processes

Process / Sampling position / Chemical formula / Substance		Conventional activated sludge process	Pure oxygen aeration process	Deep shaft	Rating Bio Disk
		Aeration tank inlet	Aeration tank inlet	Aeration tank top	Bio-disk inlet
Odor concentration		1,700	7,300	73	170
Ammonia	NH_4	120	196	74	66
Hydrogen sulfide	H_2S	500	90	3	N.D.
Methyl mercaptan	CH_3SH	80	50	tr.	1
Dimethyl sulfide	$(CH_3)_2S$	8	160	N.D.	1
Dimethyl disulfide	$(CH_3S)_2$	2	120	N.D.	1

In the conventional activated sludge process, the aeration tank inlet shows the highest odor concentration. For other processes, the samples were taken at places most likely to generate odors.

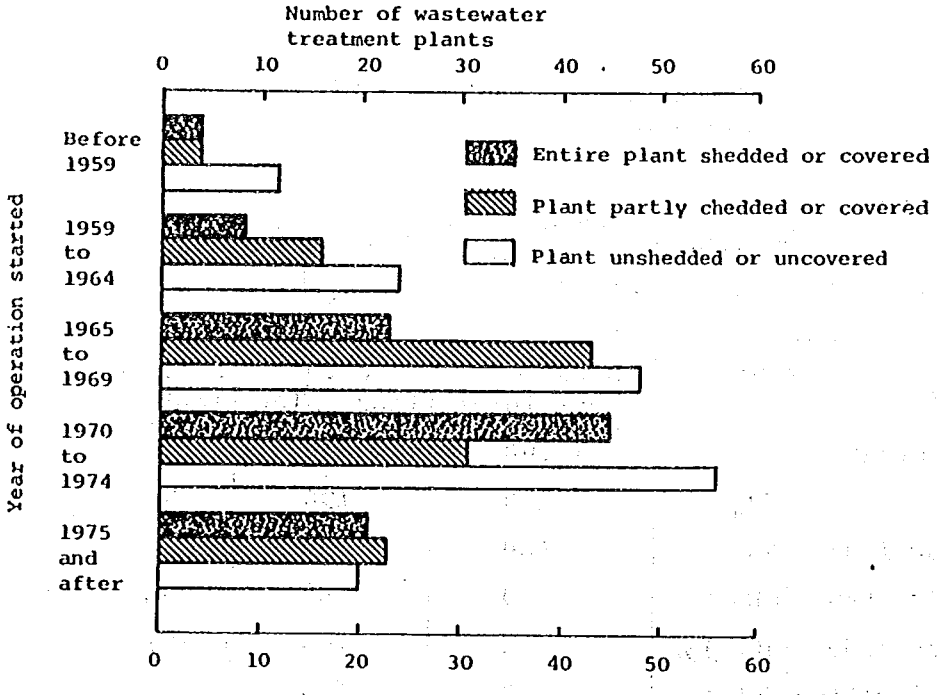
The deep-shaft and rotating bio-disk processes cause odor problems to a lesser degree than the conventional activated sludge process because their odor concentrations are lower and the ventilation rates are low.

The pure oxygen process shows a considerably high odor concentration of 7,300 ppm, but the ventilation rate is one-fortieth to one-fiftieth that of the conventional activated sludge process. Thus its impact on the environment will be little.

3. RUNNING CONDITIONS OF DEODORIZING FACILITIES

In 1978, the Public Works Research Institute conducted a questionnaire survey to take stock of offensive odor control measures implemented by wastewater treatment plants in Japan.

Fig. 5 shows the start of operation year of the plants and the installation of sheds or covers.



Note: There are eight plants whose year of operation started are not known.

Fig. 5 Plant commissioning years and shelter conditions

Nearly one-third of those plants put into operation since 1970 are equipped with some means to totally enclose the plant (including the final sedimentation tank and sludge thickener). The ratio of open treatment plants to the total was 43.1% during 1970-74, but has dropped to 31.3% since 1975.

Sheds at treatment plants, for the most part, are used as sports facilities, parks and public halls, etc., for space economy. In such instances, deodorization of exhaust from the plant is indispensable.

Of the treatment plants in service at the end of fiscal 1977, those having deodorizing facilities numbered 129, or 33.5% of a total 385 plants.

Table 8 lists deodorizing facilities classified by process and size. The water scrubbing process is excluded from the deodorizing facility to simplify the table (for example, the water scrubbing/ion exchange/activated carbon process is classed in the "ion exchange + activated carbon").

The deodorizing systems shown in Table 8 range from water scrubbing (1) to acid/alkali scrubbing + ozonation + activated carbon process (28). The alkali scrubbing facility (2) leads the list, followed by ozonation (11), water scrubbing (1), direct incineration (12), and activated carbon adsorption (6). These five leading deodorizing facilities account for 61.2% of the total (170).

In the first half of Table 8, treatment processes are classified into three major categories:

1. Primary Treatment including pump well, grit chamber, primary sedimentation tank, and preaeration tank.
2. Secondary Treatment including aeration tank and final sedimentation tank.
3. Sludge Treatment including sludge thickener, sludge treatment facilities and dewatering facilities.

For each category, the types of deodorizing facilities used were studied. As expected, treatment plants where emphasis is placed on the deodorization of exhaust from the sludge treatment process account for the overwhelming majority, numbering 81. They are followed by those plants where the primary treatment processes are deodorized, numbering 40. There are 18 plants where all processes are equipped with deodorizing facilities.

Table 8 Deodorizing facilities in service

No.	Classification: Deodorizing system	Classification by wastewater treatment processes								Classification by deodorizing facilities				
		Primary treatment	Secondary treatment	Sludge treatment	Primary and secondary treatment combined	Secondary treatment and sludge treatment combined	Primary treatment and sludge treatment combined	Primary treatment, secondary treatment, and sludge treatment combined	Total	Up to 100 Nm^3/min.	100 to 500 Nm^3/min.	500 to 1000 Nm^3/min.	1000 Nm^3/min. up	Total
1	Water scrubbing alone	10	-	8	-	-	-	1	19	2	12	2	3	19
2	Alkali scrubbing	4	2	17	2	2	2	1	30	7	11	8	4	30
3	Acid/alkali scrubbing	2	-	4	2	-	2	1	11	4	5	2	-	11
4	Acid scrubbing	-	-	-	-	-	1	-	1	-	1	-	-	1
5	Acid/alkali scrubbing + hypochlorous acid scrubbing	-	-	-	-	-	-	1	1	-	1	-	-	1
6	Activated carbon adsorption	3	-	8	2	-	2	-	15	8	6	1	-	15
7	Ion exchange resin adsorption + activated carbon adsorption	3	1	3	1	-	2	2	12	4	3	3	2	12
8	Ion exchange resin adsorption	1	-	-	-	-	-	-	1	-	-	1	-	1
9	Soil deodorization	1	-	2	-	-	-	-	3	1	1	-	1	3
10	Soil deodorization + activated carbon adsorption	-	-	1	-	-	-	-	1	-	1	-	-	1
11	Ozonation	7	-	6	5	-	1	5	24	4	9	3	8	21
12	Direct incineration	1	-	15	-	-	-	-	16	10	6	-	-	16
13	Catalytic combustion	-	-	2	-	-	1	-	3	3	-	-	-	3
14	Blowing into aeration tank	2	-	5	-	-	-	-	7	2	4	1	-	7
15	Masking	2	-	1	-	-	-	-	3	1	2	-	-	3
16	Deodorizing chemicals	1	-	-	-	-	-	-	1	-	-	-	-	-
17	Alkali scrubbing + activated carbon adsorption	-	-	1	-	-	-	-	1	-	1	-	-	1
18	Alkali scrubbing + hypochlorous acid scrubbing + activated carbon adsorption	-	-	1	-	-	-	-	1	1	-	-	-	1
19	Alkali scrubbing + ion exchange resin adsorption + activated carbon adsorption	-	-	1	-	-	-	-	1	1	-	-	-	1

Table 8 Deodorizing facilities in service (cont'd)

No.	Classification / Deodorizing system	Classification by wastewater treatment processes								Classification by deodorizing facilities				
		Primary treatment	Secondary treatment	Sludge treatment	Primary and secondary treatment combined	Secondary treatment and sludge treatment combined	Primary treatment and sludge treatment combined	Primary treatment, secondary treatment, and sludge treatment combined	Total	Up to 100 Nm^3/min.	100 to 500 Nm^3/min.	500 to 1000 Nm^3/min.	1000 Nm^3/min. up	Total
20	Acid/alkali scrubbing + activated carbon adsorption	-	-	1	-	-	-	3	4	-	2	1	1	4
21	Acid/alkali scrubbing + chelating agent	-	-	-	-	-	1	-	1	1	-	-	-	1
22	Acid/alkali scrubbing + hypochlorous acid scrubbing + activated carbon adsorption	-	-	-	-	-	-	1	1	-	-	1	-	1
23	Ozonation + deodorizing agent	-	-	1	-	-	-	-	1	-	-	-	1	1
24	Ozonation + activated carbon adsorption	-	-	2	1	-	-	2	5	-	2	2	1	5
25	Acid/alkali scrubbing + ozonation	1	-	1	-	-	1	-	3	1	-	2	-	3
26	Acid scrubbing + ozonation + activated carbon adsorption	-	-	-	-	-	-	1	1	-	-	-	1	1
27	Alkali scrubbing + ozonation + activated carbon adsorption	-	-	1	-	-	-	-	1	-	-	-	1	1
28	Acid/alkali scrubbing + ozonation + activated carbon adsorption	2	-	-	-	-	-	-	2	-	1	1	-	2
Total		40	3	81	13	2	13	18	170	50	68	28	23	169

Alkali scrubbing (2) is the most popular method of removing hydrogen sulfide, methyl mercaptan and other odorous substances common to all the treatment processes. It is employed for all of the primary, secondary and sludge treatment processes. Direct incineration (12), which is effective for a small flow of concentrated odors, is used for almost every sludge treatment process.

It is worth noting that ozonation (11) is used for deodorization of the entire plant when the flow of odorous gases is large, because it costs less than other deodorizing systems.

The ion exchange resin + activated carbon system (7) is one deodorizing system which has been gaining popularity in recent years. The ion exchange resin used is composed of alkali and acid atmospheres, and those substances which have escaped the resin are trapped in the activated carbon. As the ion exchange resin + activated carbon system is able to cover a wide range of odorous substances, it is used in almost every treatment process.

At the time of survey, there were no more than three treatment plants where the deodorizing system was combined with a chemical scrubbing system using oxidizing agents such as hypochloric acid. Lately, the deodorizing performance of oxidizing agents has been called more attention. It is claimed that this kind of deodorizing system is effective, particularly for removal of methyl sulfide and methyl disulfide.

In the latter half of Table 8, deodorizing facilities are classified by size. The facilities with a capacity of 100 to 500 Nm^3/min lead the list. The facilities of up to 100 Nm^3/min are mostly used to deodorize fumes extracted from nearly enclosed process points such as the sludge thickener, and in many cases use the direct incineration system (12) or activated carbon adsorption system (6).

On the other hand, large facilities with a capacity of 1000 Nm^3/min or more are used mostly to deodorize the bulk of each of the plants where they are installed.

Those plants with large deodorizing facilities which use an ozonator in some way or other number 13 out of 23.

The relationship between the scale of deodorizing facility and the scale of treatment plant is shown in Fig. 6. Those plants with a design dry-weather daily flow of 10,000 to 100,000 m^3 are most in number, and also account for the most deodorizing facilities.

It is noteworthy that the large-capacity deodorizing facilities of 1000 Nm^3/min and up are concentrated in this class of treatment plant.

For a small-scale plant of 10,000 to 100,000 m^3/day, it is comparatively easy to totally enclose its treatment processes and deodorize odors together. This probably justifies installation of a large-scale deodorizing facility.

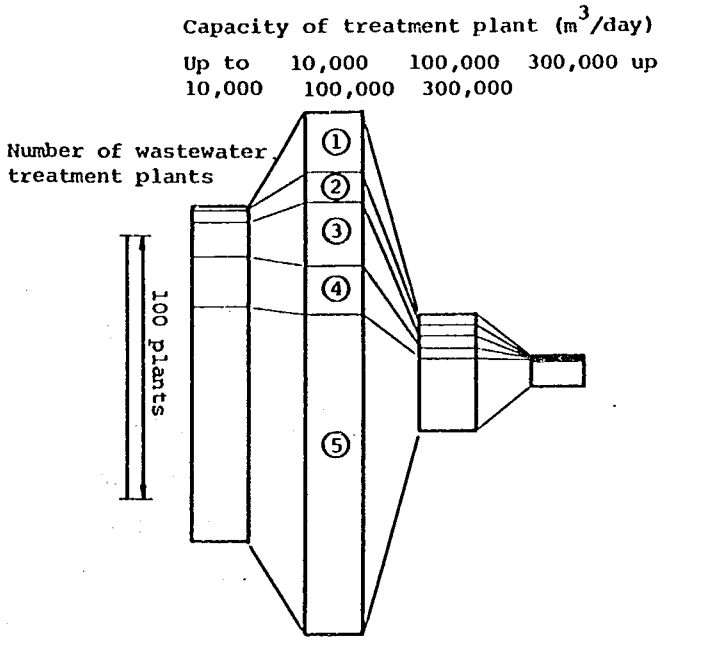


Fig. 6 Classification of deodorizing facilities by size of treatment plants

For the plants with a capacity of up to 10,000 m^3/day, deodorizing facilities installed are, to a significant extent, small. This is probably because this class of plants is low in potential as an odor source, and also because odor control will be achieved if deodorization is carried out only at the sludge treatment process, grit chamber, etc.

There are few large-capacity treatment plants of 300,000 m^3/day and up which are equipped with deodorizing facilities. It is too costly to deodorize a plant with a large processing capacity. Even if deodorization is limited to major local sources, the cost is high. The large-scale plants call for large areas, and usually the major odor-generating processes are isolated from nearby residential areas to minimize odor problems.

4. EFFICIENCY OF DEODORIZING FACILITIES

According to the questionnaire survey referred to earlier, treatment plants where deodorizing efficiency is measured at regular intervals number 20.2%. Most of the plants seem likely to operate and manage their deodorizing facilities empirically.

The Public Works Research Institute investigated deodorizing efficiency at various plants, and Table 9 shows the results for hydrogen sulfide, methyl sulfide, and methyl mercaptan. So far as methyl sulfide and methyl mercaptan are concerned, every deodorizing facility shows high removal efficiency. It is to be noted that the removal efficiency becomes lower as the odor concentration becomes low.

On the other hand, the methyl sulfide removal efficiency is generally low. Where methyl sulfide is a major odor source, the deodorizing system must be selected with care. According to Table 9, the removal efficiency of hydrogen sulfide and methyl mercaptan remains low just after acid/alkali scrubbing or after ion exchange, but becomes high after the activated carbon deodorizing stage.

Table 9 Efficiency of deodorizing facilities

Odorous substance	Wastewater treatment plant	Deodorized process	Deodorizing system	Ventilation rate (Nm^3/min.)	Predeodorization concentration (ppb)	Concentration after acid/alkali scrubbing	Concentration after ion exchange resin adsorption	Concentration after activated carbon adsorption	Concentration after ozonation	Concentration after alkali scrubbing	Concentration after soil deodorization	Ultimate deodorizing efficiency (%)
Hydrogen sulfide	A	Grid chamber	Acid/alkali scrubbing + activated carbon adsorption	335	78	307	-	14	-	-	-	82
		Overall wastewater treatment plant	Acid/alkali washing + activated carbon adsorption	540	11.5	16.6	-	9.6	-	-	-	17
		Sludge scrubbing tank	Acid/alkali scrubbing + activated carbon adsorption	170	42.9	2.5	-	2.3	-	-	-	95
		Sludge thickener	Ion exchange resin adsorption + activated carbon adsorption	100	65,400	-	11,200	41	-	-	-	100
	B	Grit chamber	Acid/alkali scrubbing + ozonation	500	33.5	19.5	-	-	14.2	-	-	58
		Primary sedimentation tank	Acid/alkali scrubbing + ozonation	250	53,7	25.8	-	-	N.D.	-	-	100
	C	Grit chamber	Alkali scrubbing	600	15	-	-	-	-	4	-	73
	D	Grit chamber	Ozonation	1,100	10	-	-	-	1	-	-	90
	E	Overall wastewater treatment plant	Ion exchange resin adsorption + activated carbon adsorption	1,000	10	-	5	N.D.	-	-	-	100
	F	Overall wastewater treatment plant	Soil deodorization	-	49	-	-	-	-	-	5	90
Dimethyl sulfide	A	Grit chamber	Acid/alkali scrubbing + activated carbon adsorption	335	2.5	2.4	-	1.9	-	-	-	24
		Overall wastewater treatment plant	Acid/alkali scrubbing + activated carbon adsorption	540	6.7	5.9	-	6.6	-	-	-	1
		Sludge scrubbing tank	Acid/alkali scrubbing + activated carbon adsorption	170	0.8	0.6	-	0.3	-	-	-	63
		Sludge thickener	Ion exchange resin adsorption + activated carbon adsorption	100	59	-	127	36	-	-	-	39

Table 9 Efficiency of deodorizing facilities (cont'd)

Odorous substance	Wastewater treatment plant	Deodorized process	Deodorizing system	Ventilation rate (Nm^3/min.)	Predeodorization concentration (ppb)	Concentration after acid/alkali scrubbing	Concentration after ion exchange resin adsorption	Concentration after activated carbon adsorption	Concentration after ozonation	Concentration after alkali scrubbing	Concentration after soil deodorization	Ultimate deodorizing efficiency (%)
Dimethyl sulfide	B	Grit chamber	Acid/alkali scrubbing + ozonation	500	2	2	-	-	1.6	-	-	20
		Primary sedimentation tank	Acid/alkali scrubbing + ozonation	250	2.1	1.6	-	-	1.4	-	-	46
	C	Grit chamber	Alkali scrubbing	600	2	-	-	-	-	1	-	50
	D	Grit chamber	Ozonation	1,100	T.r.	-	-	-	N.D.	-	-	-
	E	Overall wastewater treatment plant	Ion exchange resin adsorption + activated carbon adsorption	1,000	6	-	5	3	-	-	-	50
	F	Overall wastewater treatment plant	Soil deodorization	-	14	-	-	-	-	-	8	43
Methyl mercaptan	A	Grit chamber	Acid/alkali scrubbing + activated carbon adsorption	335	26	12.7	-	0.9	-	-	-	97
		Overall wastewater treatment plant	Acid/alkali scrubbing + activated carbon adsorption	540	8.2	8.6	-	N.D.	-	-	-	100
		Sludge scrubbing tank	Acid/alkali scrubbing + activated carbon adsorption	170	6.2	14.9	-	2.7	-	-	-	56
		Sludge thickener	Ion exchange resin adsorption + activated carbon adsorption	100	328	278	-	35	-	-	-	89
	B	Grit chamber	Acid/alkali scrubbing + ozonation	500	4.1	4.5	-	-	1.7	-	-	59
		Primary sedimentation tank	Acid/alkali scrubbing + ozonation	250	9.1	8.9	-	-	2.4	-	-	74
	D	Grit chamber	Ozonation	1,100	T.r.	-	-	-	N.D.	-	-	-
	E	Overall wastewater treatment plant	Ion exchange resin adsorption + activated carbon adsorption	1,000	13	-	8	1	-	-	-	92
	F	Overall wastewater treatment plant	Soil deodorization	-	N.D.	-	-	-	-	-	N.D.	-

The acid/alkali scrubbing system installed at the A and B treatment plants is of the spray type, and its removal efficiency is considered lower compared with the packed media type or wet-net type. Regardless, it is difficult for the acid/alkali scrubbing method to deodorize low-concentration odors, as in the wastewater treatment process, at a high efficiency.

The same holds true with the ion exchange resin method. While these methods exhibit high efficiency when applied to high-concentration odors such as those from the sludge thickener, the activated carbon adsorption method is indispensable for a higher degree of deodorization.

5. FUTURE MEASURES

In planning odor prevention measures for the future, the following considerations are important:

1. Present and Future Odor Problems

At existing treatment plants, it is important to identify major odorous substances by organoleptic tests and instrument analysis.

In addition to the eight substances specified in the Offensive Odor Control Law, there are many odorous substances likely to be generated by treatment plants. They include i-buthyric acid, n-pentanoic acid and other lower fatty acids, benzene, xylene and other aromatic hydrocarbon compounds. Most of them can be determined quantitatively.

It is possible to estimate major odorous substances by comparing the results of instrument tests with the threshold values (Table 5) of various substances determined by the air bag method.

While the odorous substances may produce synergetic or countervailing effects, such effects seem unlikely according to current data.

At present, however, substances causing odors in the aeration tank and special sludge treatment processes (heat treatment process, Zimpo process) have yet to be identified. In order to install a deodorizing facility, it is necessary to install a cover or shed and ventilate the enclosed space. How to seal and ventilate is the key to the design of the deodorizing system.

Both capital and running costs will be cut down sharply if the odor-collecting facility is designed properly. Thus the odor-collecting system (sealing and ventilating system) should be carefully studied in the design stage.

The ventilation rate usually is determined in terms of the number of displacements per hour. Originally, this method would have been intended to improve the working environment at the dewatering machine, etc. But from the viewpoint of deodorization, this method is not only insignificant, it is uneconomical as well.

For economical deodorization, it is highly important to totally seal off odors from working space and nearby residential areas and to minimize the emission of foul gas. In other words, the sources of odor must be totally enclosed while ensuring spaces for operation and maintenance, and ventilation should be carried out to the extent that odor will not leak through unsealed gaps.

It is reported that for the dewatering machine and the like, odor will not leak into the open air if ventilation introduces air into the machine at a rate of 0.6 m^3/sec.

Consider a filter press dewatering system 10 m in length, 3 m in width, and 4 m in height, and assume that it is covered totally except for a 1.2 m wide working space. The uncovered opening will be about 1 m^2. Thus the ventilation rate required is calculated at 36 Nm^3/min (= 1 m^2 x 0.6 m/sec. x 60 sec.).

The space of the room accommodating four units of the filter press is about 20 m long by 25 m wide by 6 m high. If the room is ventilated 10 times an hour, the ventilation rate is calculated at 500 Nm^3/min (= 20 x 25 x 6 x 10 + 60), which is by far larger than the ventilation rate with the machines covered (36 Nm^3/min 4 units = 144 Nm^3/min).

The output of odorous gases is determined by the ventilation rate, concentration of odorous substances at source (sewage or sludge) and the stirring conditions at source. According to experiments, air-borne odorous substances are proportional to the concentration of odorous substances at source. The relationships between air-borne odorous substances and the ventilation rate or stirring conditions of the source are under study.

2. Forecasting Odor Hazards

For those who are concerned with offensive odor control, judging the possibilities of odor hazards and selecting engineering specifications for deodorizing facilities are of crucial importance.

The possibility that an odor source will cause environmental problems are evaluated in terms of the Total Odor Emission Rate (TOER). TOER is expressed as the product of odor concentration and ventilation rate (Nm^3/min). The odor offensiveness vs. TOER relationship is established as shown in Table 10.

The TOER of the treatment plants covered previously is calculated as follows:

Before deodorization

- B wastewater treatment plant: 5.82×10^6 Nm^3/min
- C wastewater treatment plant: 6.3×10^6 Nm^3/min in summer
  1.6×10^3 Nm^3/min in winter

According to Table 10, these values indicate a minor odor problem.

Both plants are fed with night soil. Without night soil, these values would be reduced by nearly one order of digit.

Table 10 TOER and susceptibility to environmental odor problems

Total odor emission rate (TOER)	Susceptibility to environmental odor problem	Typical odor sources	Remarks
Up to 10^4	Little except in special cases	Bakeries, breweries	
10^{5-6}	A minor environmental odor problem already exists, or potentially exists.	Painting shops, paint manufacturing works, print shops, ink manufacturing plants, tanneries, FRP works, foodstuff and fertilizer manufacturing plants, sewage treatment plants	The maximum ultimate distance of offensive odor diffusion is 1 to 2 km. Complaints about offensive odor usually come within a radius of 500 m from source; complaints are rare from residents 1 km or more from source
10^{7-8}	A minor to medium environmental odor problem	Foundries, night soil treatment plants, swineries, poultry farms, petrochemical plants	The maximum ultimate diffusion distance is 2 to 4 km; the affected area is within 1 km from source
10^{9-10}	A major environmental odor problem	KP plants, cellophane plants, rendering houses, rayon plants	The maximum ultimate diffusion distance is within 10 km; the affected area is within 2 to 3 km from source
10^{11-12}	The gravest environmental odor problem (rare)	Large-scale KP plants without deodorizing facilities	The maximum ultimate diffusion distance is several tens of kilometers; the affected area is 4 to 6 km in radius

For a more precise forecast of environmental odor problems, it is necessary to apply meteorological data to an air diffusion formula. At present, there are formulas, including Sutton's, for expressing the diffusion phenomena in air. Where odorous gases are vented from a point source like a ventilator, their diffusion into the open air can be predicted through these formulas.

Estimates by these formulas so metimes differ more than one order of digit from measurements. Since the organoleptic measuring accuracy of human beings is much the same, the estimation errors may be absorbed with a safety factor in the practical design.

In case the odorous gases are directly vented from a water surface without a collecting cover, the degree of odor problem cannot be estimated because the relationships of odor diffusion velocities to wind velocities, ambient temperature, concentrations of odorous substances in the water, etc., are now unknown. In the future, the Public Works Research Institute will study these diffusion problems.

BEHAVIOUR OF ORGANO-CHLORINATED COMPOUNDS PRECURSOR IN BIOLOGICAL TREATMENT PROCESS

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1. INTRODUCTION.

It is well known that trihalomethanes (THM) in tap water are formed during chlorination by the reaction of chlorine and organic substances, such as humic acid and related compounds. 1-3) The toxicity, especially of the carcinogenecity of THM has been reported by many authors. 4-5) Among THM, the mutagenicity and carcinogenecity of chloroform are revealed by the Ames test and the mammal test, respectively. 6) There are also epidemiological studies on the relationship between cancer mortality and THM in drinking water. 7-9) Because of various uncertainties in the epidemiological survey, some reports have criticized the conclusion leading to that relationship.10) When viewed collectively, however, the epidemiological studies provide sufficient evidence for maintaining the hypothesis that a health risk may occur and that the positive relationship may be a reflection of the presence of a relationship between constituents of water and cancer mortality.

There exist various non-volatile organo-halogenated compounds which are also formed during chlorination process in the water purification system.11) Actually, the working group for water pollution in OECD has suggested the importance of controlling these compounds, because these comprise the most part of total organo-halogenated compounds in drinking water,12) whereas only 20 - 30 % of these are known to be THM. The number of natural organic compound which has been identified as a component of chlorides is very small, thus the existence of organo-chlorinated compounds in drinking water is an unnatural phenomenon.

With this background, the control of organo-chlorinated compound should be implemented to promote public health condition as much as possible. The Drinking Water Quality Guideline, WHO, has set the guideline level of chloroform in drinking water to 30

ppb., and the Japanese Government set the tentative guideline level of THM to 100 ppb. in 1981. In order to provide available technology for the organo-chlorinated compound control, the research project has been supported by the Environmental Agency and Ministry of Health and Welfare since 1974. Herein are presented the THM formation properties of selected wastewater and the relationship between THM precursor and biological synthesis.

2. EXPERIMENTAL PROCEDURES

2.1 Sample collection

Surface water, i.e. river and lake water were collected at 228 stations throughout Japan in 1978. Primary settled sewage and its activated sludge treatment effluent were sampled at a sewage treatment plant and a bench scale pilot plant installed at same place. 13) Human excreta and its biological denitrification process effluent and coagulation sedimentaion process effluent were sampled at a collected human excreta treatment plant.

2.2 Measurement of THM

THM was analyzed by the head space method.14) 20 ml of a sample is introduced to a vial, 26.9 ± 0.1ml of capacity, and added a available amount of Na_2SO_3 to quench the residual chroline. The vial is then sealed with a teflon covered silicone plug and aluminum cap. The vial is then set into a waterbath at 25 ± 0.5 C for one hour. After the equilibrium of THM between gas and liquid phase is obtained, 50 ul of gas is taken from the head space in the vial by using a gas-tight micro syringe, and injected into the gas chromatogrph equipped with a ^{63}Ni electron capture detector. The analytical condition of a gaschromatography is as follows:

Column: Chromosorb-W AW-DMCS 25% Silicone DC-550, 3mm·2.1m
carrier gas : N_2 60ml/min.
Temperature: Column 90℃, Injection port and detector 150℃

2.3 Measurement of THM formation potential and other water quality items.

The THM formation potential(THMFP) is defined as the concentration of THM that is formed by the reaction of THM precursor materials in the water and halogen species under the following condition. The sample which had been adjusted to 20℃ was placed into a glass flask. The chlorine solution was added to the sample, and the pH was immediately adjusted to 7. Then the sample was transfered into a glass bottle having an inert silicone rubber sealer with screw cap, and let stand for 24 hours at 20℃. The amount of chlorine to be added was controled so as that the free chlorine of the sample after 24 hours reaction would be left 1.0 ± 0.5 ppm.

Chroline, Total organic carbon(TOC), KMnO4 consumption values(PV) and other water quality items were measured by the Standard Methods for Drinking Water. 15)

2.4 Culitivation of Microcystis aeruginosa.

Microcystis aeruginosa which is dominant alga species in Japanese eutrophic lake was selected to model alga. M.aeruginosa which had been separated from lake Kasumigaura was seeded into Gorham No.11 revised media, and cultured in the shaker flasks at 20℃, 4-Kluxes(10 hours irradiation a day). The content in the cultivation flask was sampled at appropriate intervals and analyzed by the method shown in Figure 1.

2.5 Gel-chromatography and eluent analysis

Gel-chromatography on Sephadex G-15(exclusion limit,MW1500) and G-25(exclusion limit MW5000) were used to fractionate the

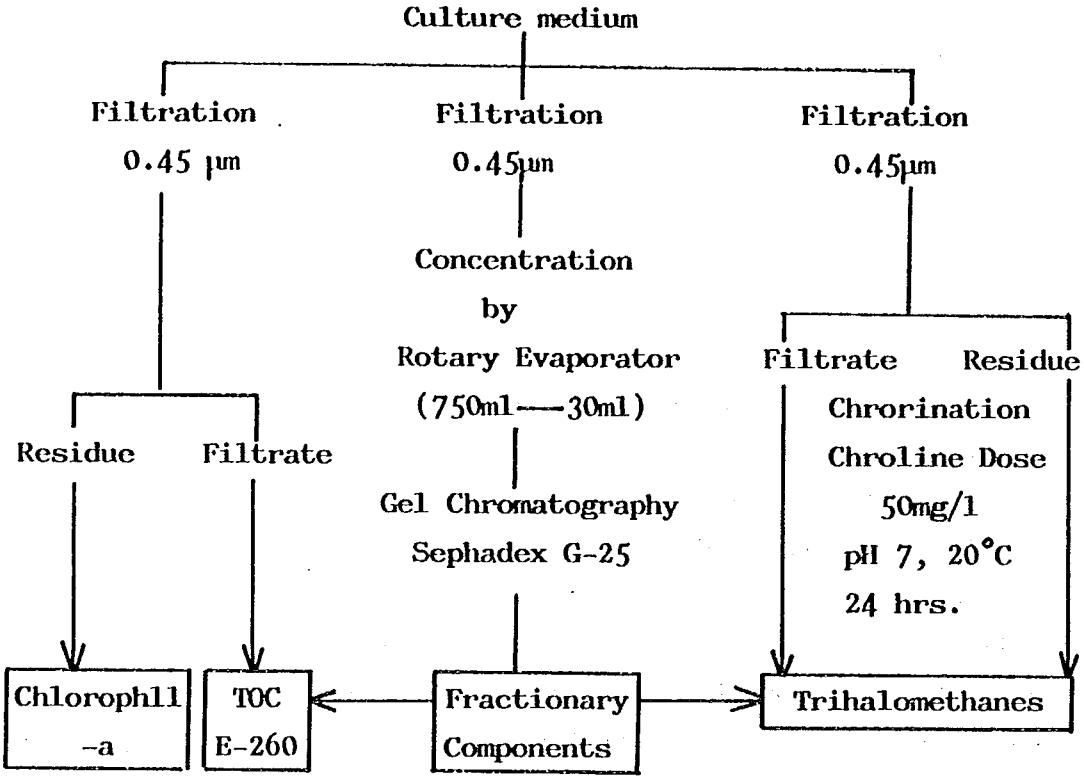


Figure 1. Experimental Procedures.

dissolved organic substances in sample. The concentrated sample of 10 ml(300-500 mgTOC/l) was placed on the column(2.6cm 90cm) and eluted with distilled water with flow rate of approximately 1 ml/min. The elluents from Sephadex column were collected with a fraction collector into equal volumes.

The effluent fractions were analyzed for TOC and UV absorbancy at 220 and 260nm to given an estimate of organic substances.

In each case, three and four well difined peaks were obtained from Sephadex G-25 and G-15 column, respectively. According to the flow rate of elution, the gel elution sequence from gel column were divided into four or three groups. The

molecular weights of first group(G-1) are 5000 or higher for Sephadex G-25, 1500 or higher for G-15, respectively. The molecular weights of finalgroup (G-III) for Sephadex G-25 are 1500 or less, and (G-IV) for G-15 are 500 or less. The soluble organics in sample were fractionated into three groups(MW › 5000, 5000 › MW › 1500, MW › 1500)by G-25, four groups(MW › 1500 two groups of 1500 › MW › 500, MW › 500) by G-15, respectively.

The recovery of organic carbon from column was 90-98%.

3.RESULTS AND DISCUSSION

3.1 THMFP of surface water and selected wastewater.

The frequency distribution of THMFP in Japanese river and lake waters and the relationship between THMFP and PV are shown in Figure 2 and 3, respectively. The THMFP of surface water in Japan are distributed widely and its average is about 50 ppb.. The PV among othrer water quality parameters was found to have the closest correlation with THMFP. It is said that the THM

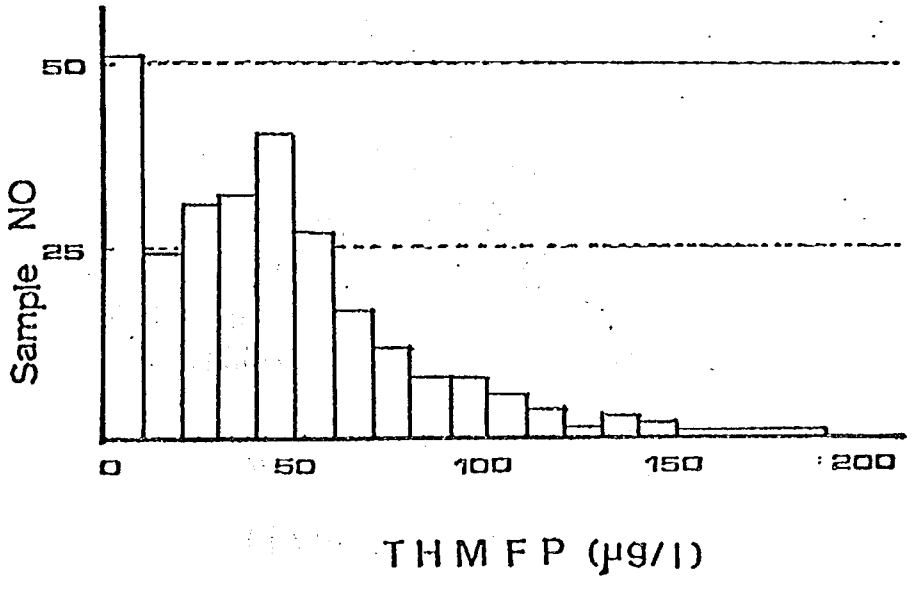


Figure 2. THMFP Distribution of Rivers and Lakes in Japan.

precursor is a humic substance in natural water. The surface water in Japan contains humic substances about 2-3 ppm as PV, except the river water which run through peat land area in Hokkaido. Thus it is considered that the PV of the sample in Figure 3 is mainly contributed by the other organic substance. That is, the main source of the THM precursor in Japanese surface water is domestic wastewater and/or industrial wastewater. The PV is arbitrarily adopted to the index of the organic substance concentration in drinking water. If the PV value is converted into COD and BOD, both COD and BOD come to be about 3-5 times of PV.

The THMFP and its THM formation intensity (THMFI), which is defined as THM μg/mg TOC, of selected wastewater and few chemicals are shown in Table 1. It is easy to understand that

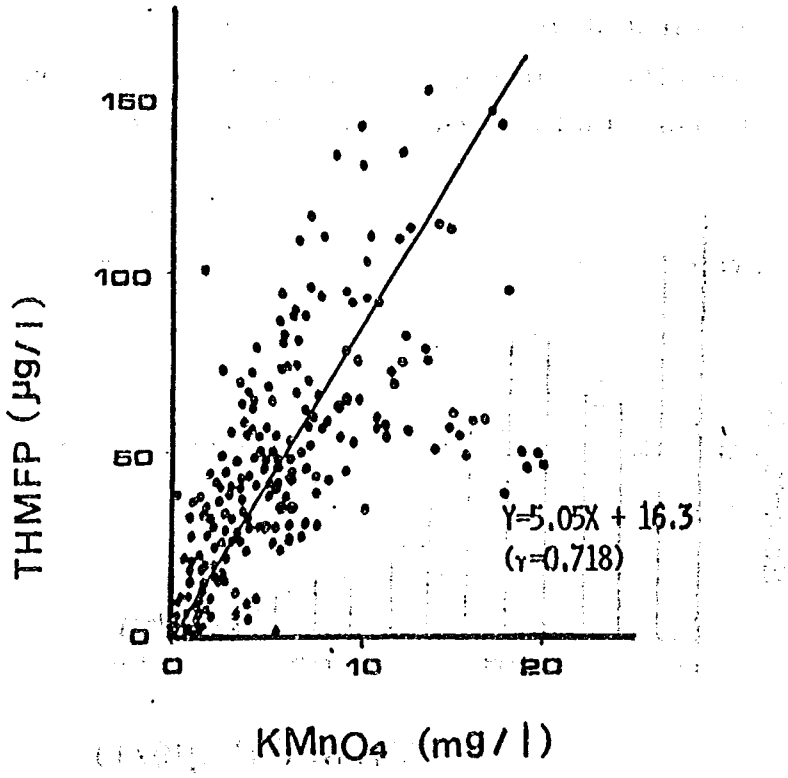


Figure 3. Relationship btween THMFP and $KMnO_4$ Consumption.

pulp factory wastewater has a high THMFI, however, it is interesting that the wastepaper reclamation factory wastewater and human wastewater also contain a high amount of THM precursors. In particular the human wastewater which may not contain a vegetable humic substance has high THMFP.

Table-1. THM Formation Properties of Organic Wastewater

Type of Wastewater	THMFP (μg/l)	THMFI (μgTHM/mgTOC)
SP Wastewater	590	25
KP Wastewater	1125	50
WPP Wastewater	2250-2260	1.5-1.7 (μgTHM/mgPV)
Primary Settled Sewage	480-770	—
Primary Settled Sewage	1271	27
Biologically Treated Sewage	460	33
Biologically Treated Sewage	75	11
Human Excreta	44500	9
Biologically Treated Human Excreta	1500	39
Biologically Treated Human Excreta	1300	28
Natural Humic Colored Water	1440	48
Humic Acids	—	37
Phenol	—	57

3.2 The Precursor and Biological Activity

The THMFP of the culture during the cultivation media are shown in Figure 4. THMFP curves of M. aeruginosa resemble to the growth curve of M. aeruginosa, and THMFP is highest during

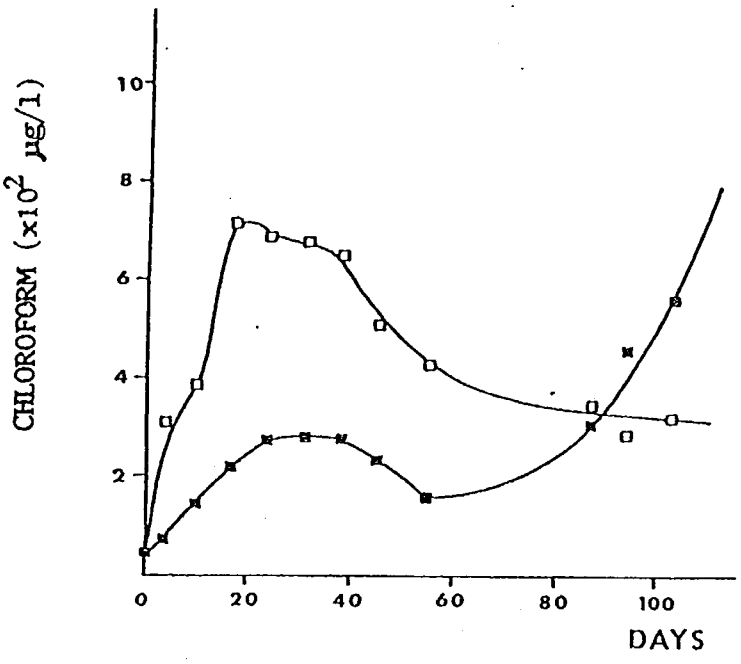


Figure 4. THMFP of M.aeruginosa.

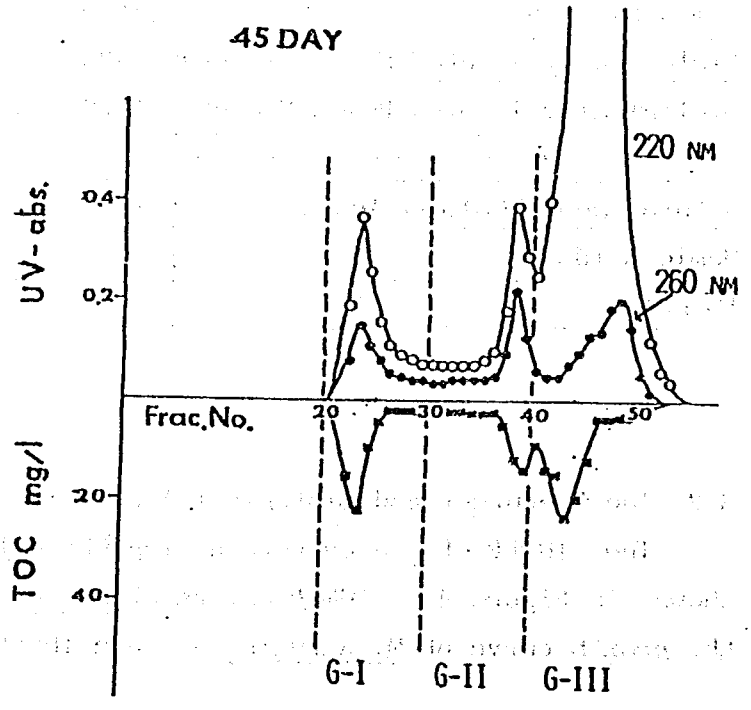


Figure 5. Gel Chromatogram of Soluble Organics.

(Sephadex G-25)

changes from the logarithmic to stationary growth phase. The THMFP of the soluble organic substance excreted by M. aeruginosa increased from the degradation phase and reach to higher THMFP than that of M. aeruginosa cell gradually.

The soluble organic substances excreted by M. aeruginosa were fractionated by gel chromatography, as shown in Figure 5, and fallen into three groups according to their molecular

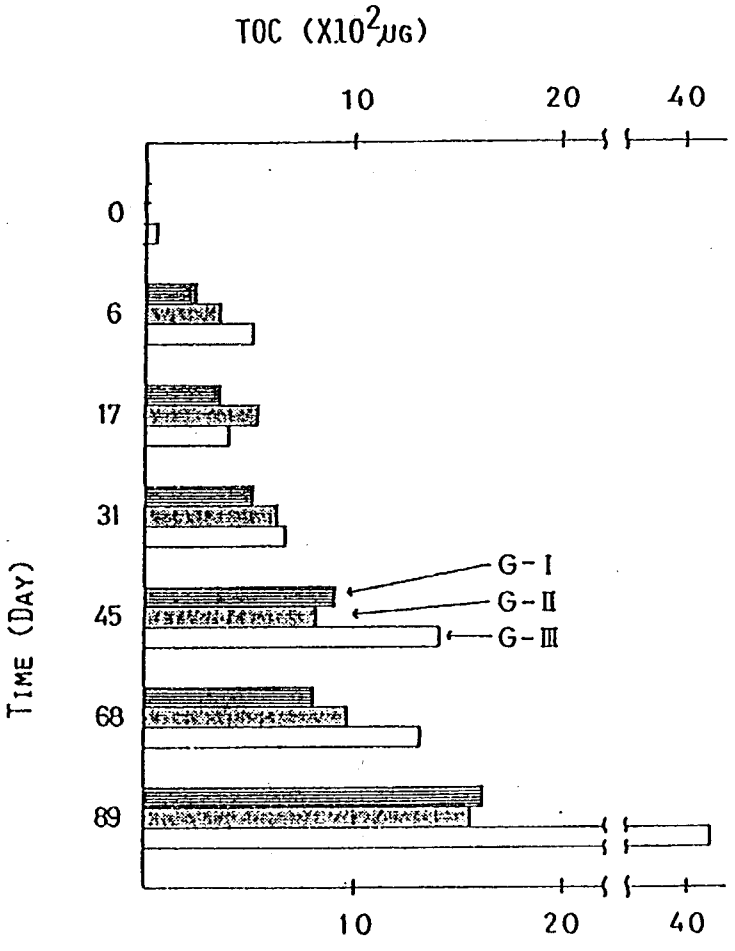


Figure 6 . Variation of Fractionated Organics.
(Sephadex G-25)

weights. The concentration of each group changes remarkably with a culture age, as shown in Figure 6, and the THMFI of each group changes also, as shown in Figure 7. The TOC proportion of group III in total is higher than those of group I and II, however the THMFI of group I and II are higher than that in group III. The maximum THMFI of soluble organic substance excreted by M. aeruginosa is higher than that of humic acid. These results have also been observed in the experiment of Nitzschia palea. From these, it is clear that an eutrophication of lakes and reservoirs should be controlled to reduce the precursors.

The THMFP of primary settled sewage and its activated sludge treatment process effluent were measured with respect to soluble organics and suspended solid and results are shown in Figure 8.

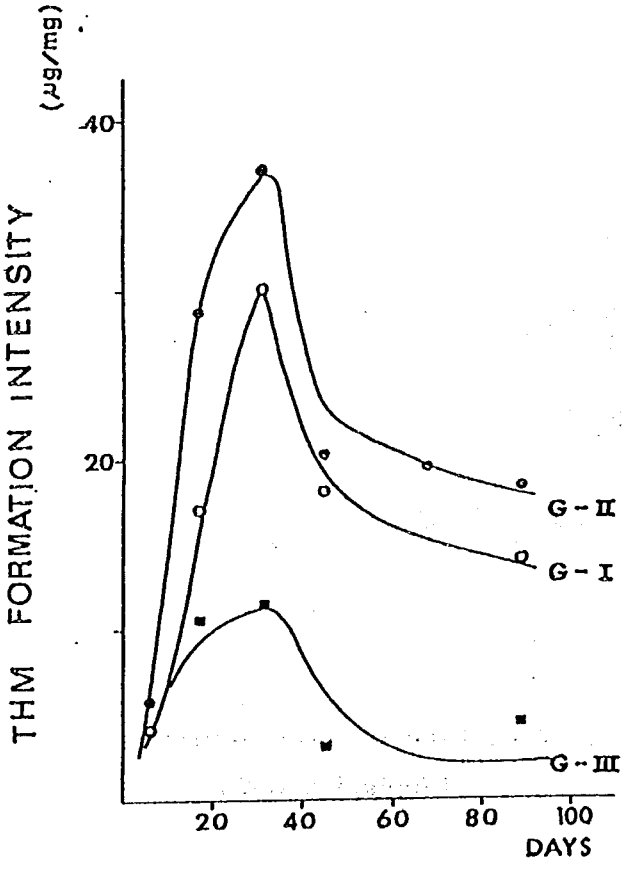


Figure 7. Variation of THM Formation Intensity.

(Sephadex G-25)

The results of the THMFP examination of human excreta, its biological denitrification process effluent, biological advanced treatment process effluent and coagulation sedimentation process effluent are shown in Table 2. Both of soluble and suspended organic substances in the domestic wastewater will produce THM equally. The biological treatment process can reduce the THMFP, however its removal efficiency is barely about 60% for sewage and 65% for human excreta. The effluent of human excreta treatment process in Table 2 was diluted 5 timse by freshwater.

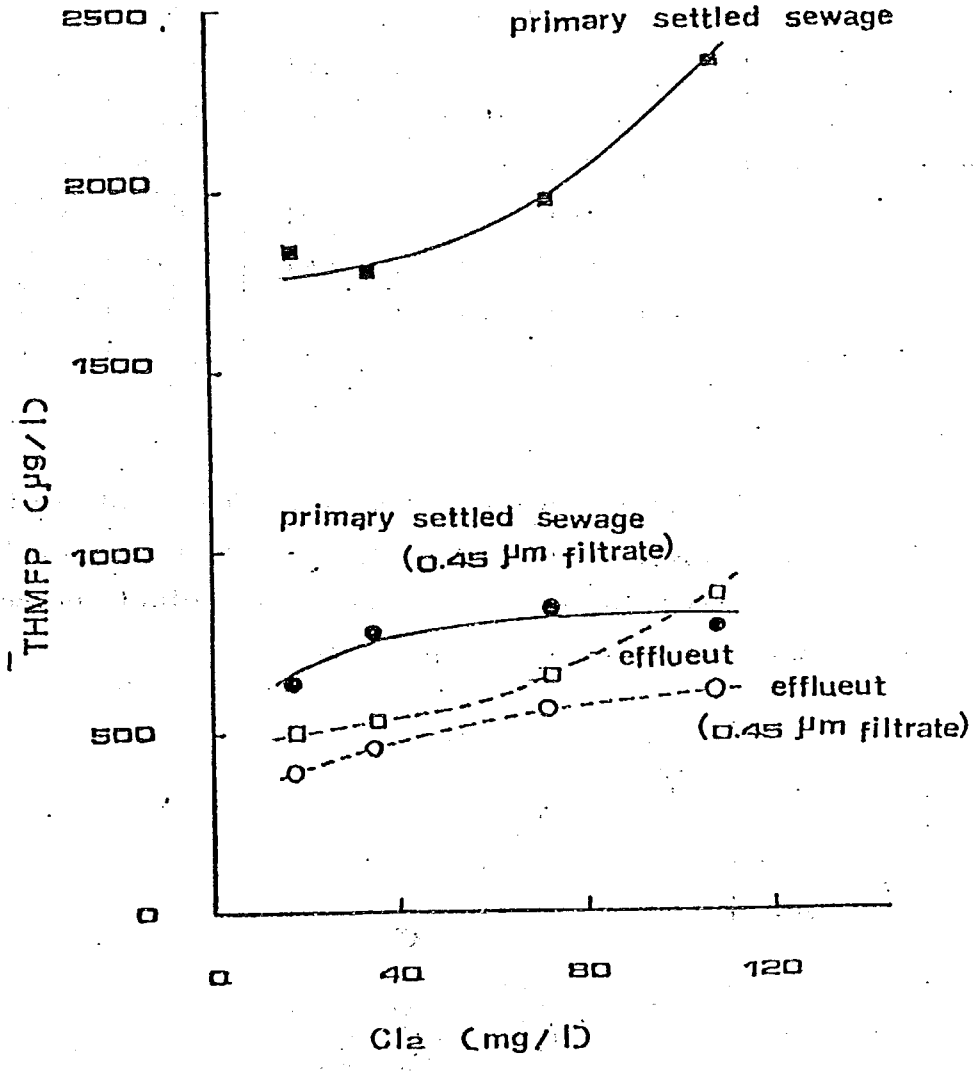


Figure 8. THMFP of Sewage and Biologically Treated Effluent.

Table-2. TOC, THMFP and THMFI of Night Soil

		TOC (mg/l)	THMFP (μM)	THMFI (μM/mg)
Collected Night soil	Dissolved Matter	2100	141	0.067
	Suspended Matter	2900	232	0.080
	Total	5000	372	0.074
Biological Denitrification Process Fffluent	Dissolved Matter	81	20.2	0.250
	Suspended Matter	23	4.1	0.179
	Total	104	24.3	0.234
Honey-comb tube Process Effluent	Dissolved Matter	62	17.6	0.284
	Suspended Matter	2	0.5	0.252
	Total	64	18.1	0.283
Coagulation-Sedimentation Process Effluent	Dissolved matter	44	11.0	0.250
	Suspended Matter	3	0.8	0.280
	Total	47	11.8	0.252

It is peciliar that the THMFI of the suspended solids in the effluent of the sewage is lower than that of primary settled sewage. On other hand the removal efficiency of soluble organic substance is about 20% as shown in Figure 9.

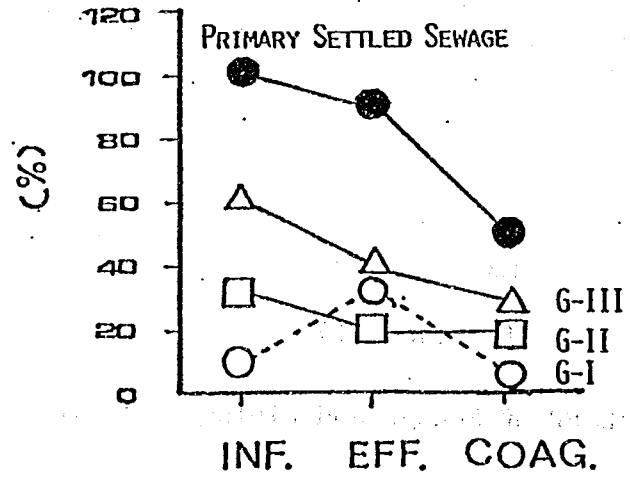


Figure 9. Concentration as a % of total Influent Soluble Organic Substance.

These result show that the activated sludge process can reduce the THMFP of the sewage mainly by removing the precursors in the suspended solid . The THMFP caused by soluble organic substance was not removed uniformly as shown in Figure 9. The concentration of group I substances in the effluent has higher than that of the influent.

The soluble TOC removal efficiency was influenced by the sludge retention time as shown in Figure 10. That is the removal efficiency of higher moleclar weights organics decreased with the sludge retention time. That means the higher molecular organic substances are produced by the activated sludge in the same manner as algae. It is well known that the removal efficiency of an activated sludge system is governed by the biological adsorption in the aeration tank, and that a long sludge retention time provides an activated sludge which has a high adsorption capacity. From these considerations, there may be optimum operational condition to produce the minimum THMFP effluent.

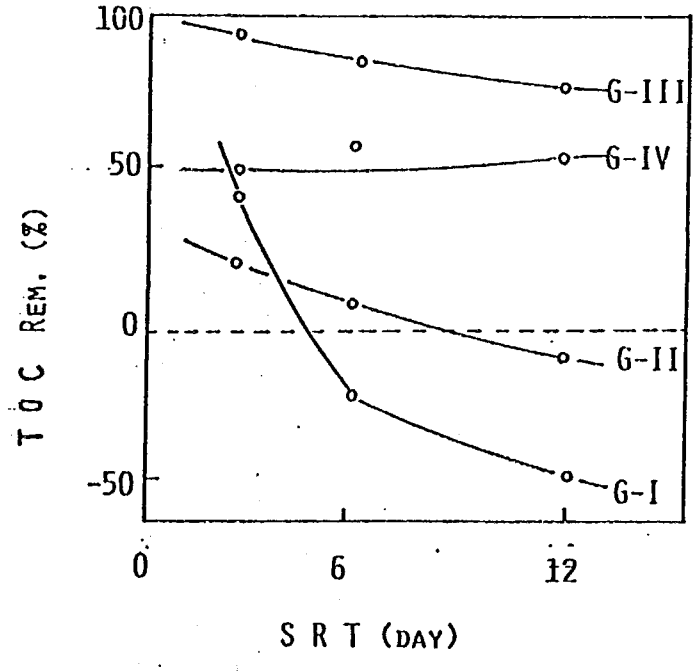


Figure 10. The Effect of SRT to TOC Removal Efficiency.

(Sephadex G-15)

Table 3 Performance of Biological Reacter

	Influent			Honey-comb Tube			Rotary Disk			Fluidized bed		
Run	1	2	3	1	2	3	1	2	3	1	2	3
Retention time	-	-	-	2.7 (hr)	3.8	4.5	22.5 (min)	48.6	92	8.6 (min)	8.6	10.6
Hydraulic Load (l/sqm/d)	-	-	-	86	62	55	405	187	99	42	42	34
Temp	20.3	13.0	5.0	20.0	13.2	5.3	20.2	12.9	4.9	20.5	14.4	5.5
Turb	9	9	11	8	8	9	10	9	8	9	10	9
TOC	4.8	3.9	7.4	4.2	3.6	6.7	4.5	3.8	7.1	4.9	3.9	7.8
PV	9.8	8.4	18.5	5.5	4.6	16.0	12.4	6.3	17.2	11.5	8.5	17.7
NH_3-N	1.4	2.7	8.1	0.5	0.4	3.1	0.4	0.3	0.9	-	0.1	3.2
THM	41.4	47.8	-	44.3	43.2	47.8	-	45.6	41.0	46.8	-	53.6

The results of the experimental work in treating river water which is polluted by sewage treatment effluents were reported as shown in Table 3. 16) The ammonium nitrogen is oxidized to nitrite or nitrate nitrogen, however, PV, TOC and THMFP were not reduced by the biological treatment process. It is considered that the organic substance which have been biologically degraded in the wastewater treatment plant and/or in the river will not be further reduced by additional biological treatment.

It is concluded that biological treatment process have a upper limit of THMFP removal efficiency. Therefore a physico-chemical treatment process should be installed together with the biological treatment process in order to control the precursor in the effluent to the public water bodies.

4. CONCLUSIONS

The THM precursor in Japanese lakes and reservoirs mainly originates from domestic and industrial wastewater. Domestic sewage and its biologically treated effluent has the same THMFP as that of natural humic water.

The Microcystis aeruginosa has THMFP and also excretes soluble organic substances which form THM. The THMFP of M.aeruginosa and the soluble organic substance change with culture age. The higher molecular weight fraction of soluble organic substance has higher THMFI.

The soluble and suspended organic substances in the sewage produce THM equally. Biological treatment process reduce THMFP mainly by removing the precursor in suspended solids. The removal efficiency of THMFP of the soluble organic substance is limited by a biological excretion of higher molecular organic substances which form THM. The biological treatment process has an upper limit of THMFP removal efficiency, therefore, a physico-chemical treatment process should be installed to back up a

biological treatment process.

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MULTISTEP PROCESSESFOR BIOLOGICAL TREATMENT WITH LOW ENERGY CONSUMPTION

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Multistep Processes for Biological Treatment with low Energy Consumption

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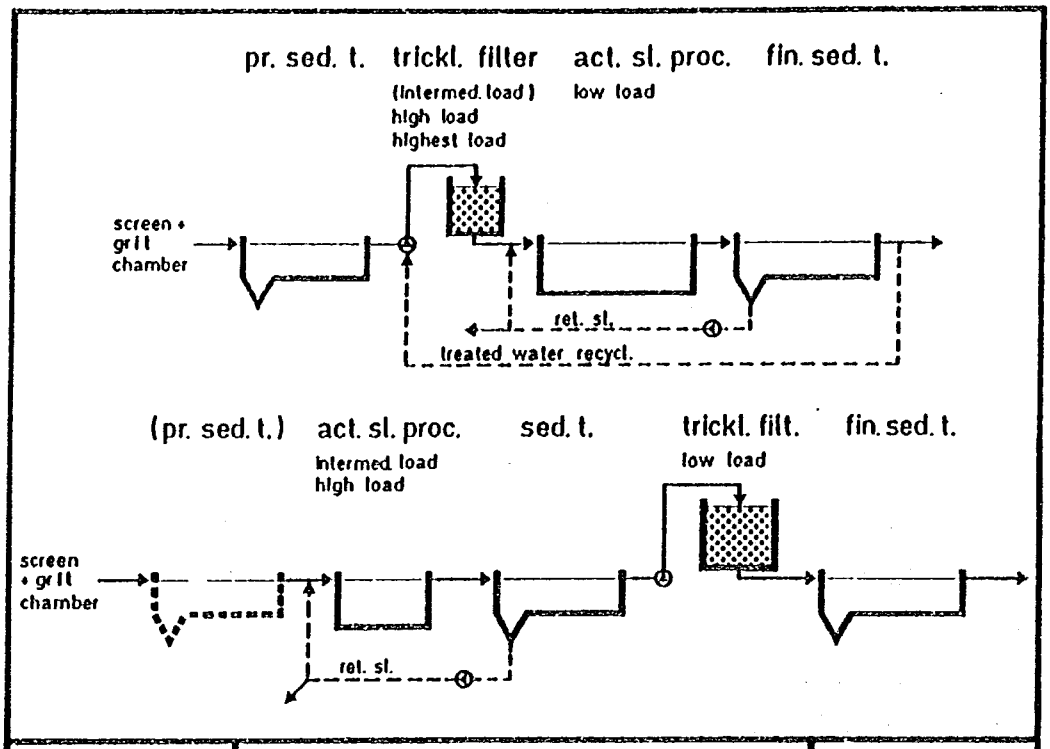
Survey of possible two-stage processes

The scope of waste water treatment and management is quite different in Japan and the Federal Republic of Germany [1]. This naturally has a corresponding effect on plans for waste water treatment technology. In Japan the waste water of about 40 million sewer served population is treated in 450 relatively large plants. A large number of different processes are thereby employed; however surprisingly no two-stage biological treatment process. This is on the one hand surely due to the relatively low japanese waste water concentration and on the other probably due to the conflicting reports in international literature.

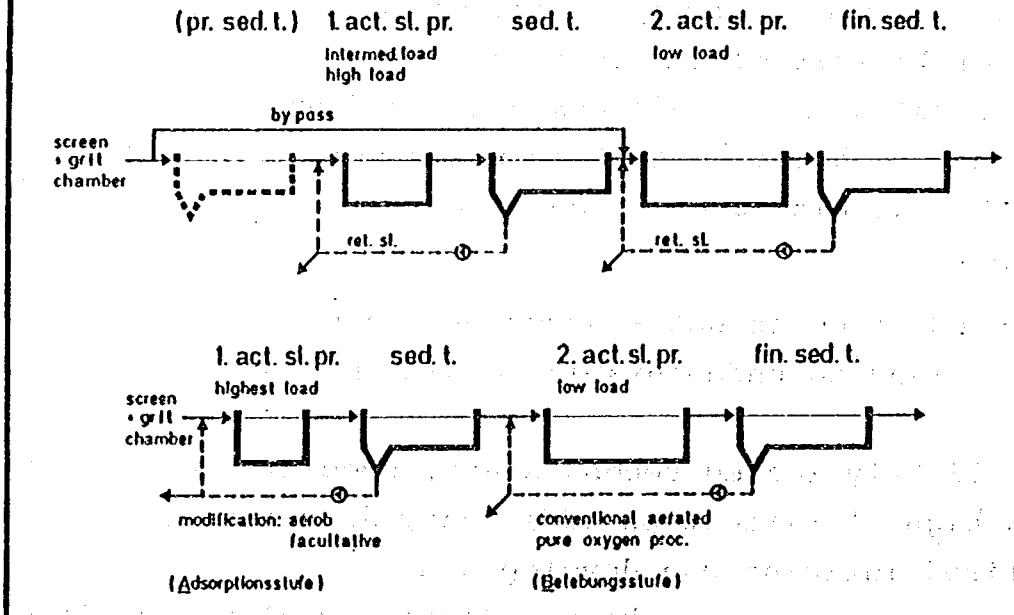
In this paper a short survey of the basic possibilities of two-stage biological treatment is given followed by a presentation of results on the process preferred by us. To start with, it must be noted that it is preferable to operate the first of the two biological stages at a higher load. This assures that

- the second stage is not starved or a non-ideal composition does not lead to unfavourable conditions for the second stage,
- in the lightly loaded second stage, special biocoenosis with a high clarification efficiency as consequence of stabilised influent can develop and
- savings in energy are achieved since a smaller specific degradation require is needed.

Fig. 1 shows the possible combinations of activated sludge and trickling filter processes. Satisfactory effluent results



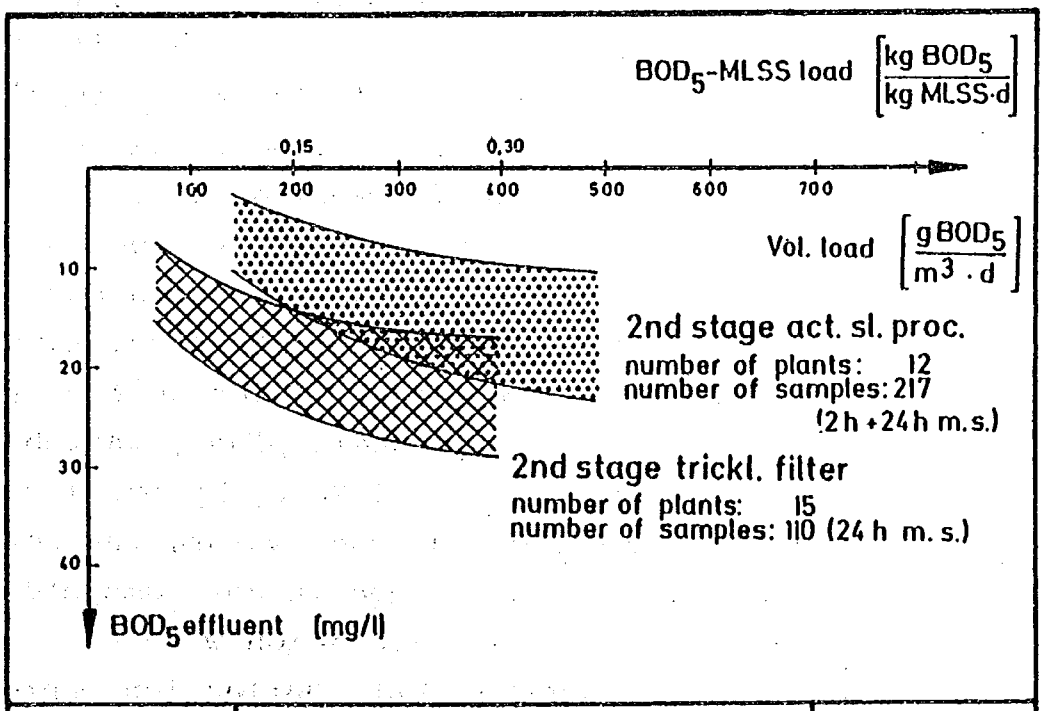
ISWW T.Univ. Aachen 1982	2-Stage Biological Treatment Combinations of Trickl. Filter and Act. Sl. Proc.	Fig. 1



ISWW T.Univ. Aachen 1982	2-Stage Biological Treatment Combinations of Activated Sludge Processes	Fig. 2

are reported [2, 3] from the process combination activated sludge process - trickling filter as well as the reversed arrangement. It is to be noted,however, that with the arrangement trickling filter - activated sludge process for loads exceeding 1000 g $BOD_5/(m^3 \cdot d)$ the trickling filter needs a synthetic filling and for the usual german waste water concentrations recirculation must always be provided in order to prevent clogging of the trickling filter. A sedimentation tank for the trickling filter sludge is not necessary since it has been shown that the trickling sludge does not influence the biological processes in the following aeration tank.

For the arrangement activated sludge process - trickling filter a sedimentation tank in good working condition must always be interposed. In case the first stage is not very highly loaded, a preclarification is also meaningful under certain conditions. A recirculation for the trickling filter is generaly not necessary since the waste water concentration is low enough by prior passage through the biological stage.



ISWW T.Univ. Aachen 1982	Comparison of BOD_5-Effluent of 2nd biol. Stage	Fig. 3

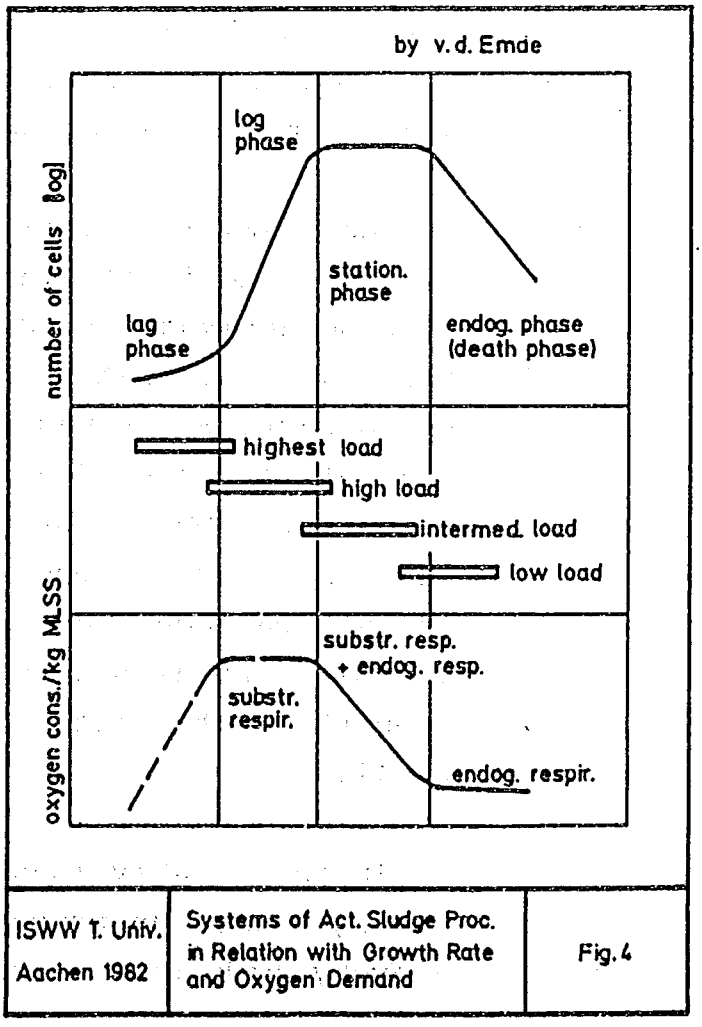
As an assessment of these combinations it can be stated that the lowly loaded trickling filter in the second stage leads to very stable nitrification, since a fixed bed reactor provides the nitrificants with better living conditions. The activated stage as the second stage does not possess for nitrificants the ideal conditions of a fixed bed reactor, however in comparison to the trickling filter better fine clarification is achieved due to the better contact between substrate and activated sludge (fig. 3). However, as will be shown later, a higher sludge age is possible in a subsequent activated stage resulting in improved nitrification as compared to single stage plants.

Fig. 2 shows the possibilities of a combination of two separate activated sludge processes connected in series. With this combination a strict separation of the sludge circulation has to be maintained, so that each stage can develop its own biocoenosis without outside interference. This basic rule has not been observed earlier leading to disturbances in the biology and corresponding effluent deterioation in older plants. Furthermore, with a normally loaded or fairly highly loaded first stage the danger of "starvation" exists as already mentioned earlier, so that a raw waste water bypass around the first stage should always be available. A preceding primary sedimentation tank leads here too to savings in energy. The clarification efficiency of a normally loaded or fairly highly loaded activated plant is relatively high for a simply calculable energy cost, which generally leads to good effluent values for the whole plant, whereby the amount contributed to the total efficiency by the low loaded second stage is rather low. In other words, the normal efficiency of the lowly loaded second stage is not completely utilized due to much too good preclarification with a simultaneously high specific energy cost. Reflection therefore led to the attempt to transfer the biological preclarification to an area where enough nutrients are fed to second stage and at the same time a higher percentage of

the clarification efficiency can be achieved with a much lower specific energy consumption. This is possible with the help of the so-called AB process which is discussed in more detail below.

Introduction to the AB-process

It is very easy to follow the change in the population in the reactor during the course of a batch experiment injected with activated sludge and without continuous nutrient supply (fig. 4). This pattern is generally known, it is only stressed that in the initial phase and at the beginning of the exponential growth phase, the procaryotes with a short generation time and high transformation rates have preferential living conditions due to adequate nutrient supply. In



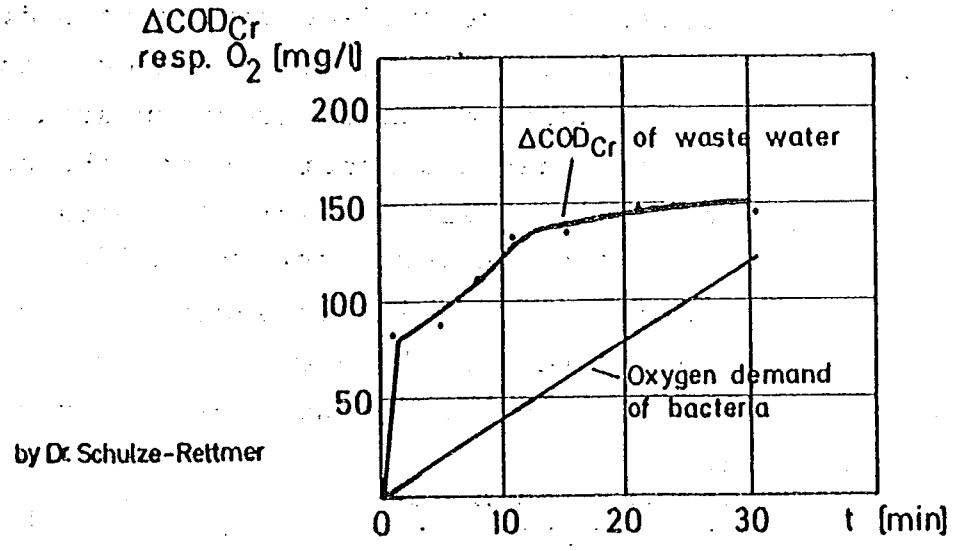
comparison, the Eucaryotes which occur in lesser loaded plants, consume oxygen without a high clarification contribution and use the procaryotes, the actual workers, as nutrient, do not have any chances to live here. This sole field of activity of the procaryotes is utilized in the A-stage of the process. In practice, this field of activity is maintained through

- a volume load of 6 - 10 kg $BOD_5/(m^3 \cdot d)$
- a strict separation of both the activated stages and
- a mixed liquor suspended solid concentration in the A-stage with the help of sufficient excess sludge removal in the 1,5 - kg/m^3 range with BOD_5-MLSS load > 2 (generally $\sim$ 5).

In the high sludge load ratio of $\sim$ 5 kg BOD_5 (kg · MLSS·d) with a retention time of 25 - 30 min. primarily coagulation reactions, flocculation and biogenic adsorption take place. In comparison to conventionally loaded plants less biologic activity takes place through initial assimilation, with a corresponding low oxygen and energy consumption.

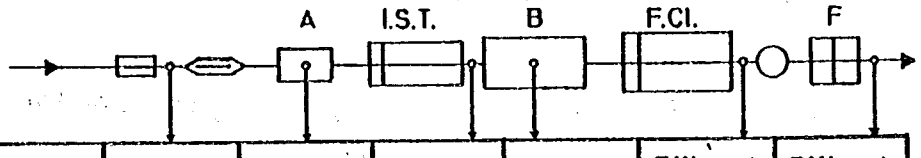
Dr. SCHULZE-RETTMER at the ISWW, Technical University, Aachen showed how fast and distinct the first clarification reaction takes place. Fresh activated sludge was added to unclarified, filtered raw waste water and the COD_{Cr} reduction of the filtered waste water was monitored as a function of time (fig. 5). It is evident that already after one minute about 80 mg/l COD_{Cr}, corresponding to about 38 % of the organic substance were removed. After 10 minutes an elimination rate of 56 % was achieved; after 20 min. 66 % were eliminated. Similar results with the same experimental arrangement were reported with waste water of the austrian city of Graz [4].

It is interesting that in this highly loaded zone the most primitive form of microorganism, the procaryotic bacteria can only exist. As in every lowly loaded activated plant, these



ISWW T. Univ. Aachen 1982	Treatment Effect of Act. Sludge (ΔCOD_{Cr}), Batch Experiment	Fig. 5

bacteria are also present in the sludge of the B-stage, however together with number of higher organisms. An investigation by BAHR [acc. 5] shows clearly the different biocoenosis structures in the A and B stages with the help of germ data in these stages (fig. 6). The larger number of germs per suspended solid unit show that the germs, i.e. the bacteria, must be smaller in the A-stage than in the B-stage, in spite of lower suspended solid content.



Agar 37°C	Influent	A-Stage	Effl. Interm. Sed. Tank	B-Stage	Effluent Fin.Clar.	Effluent Filter
Number of Germs per ml x 1000	6.360	31.500	4.800	1.440	150	64
H_2S-Producer per ml	25.000	110.000	18.600	35.800	340	175

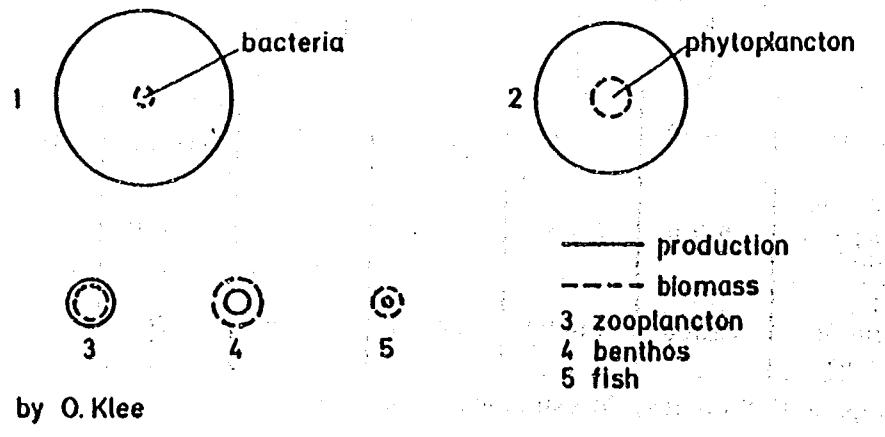
Average of 14 Samples, Investigation of Dr. Bahr

ISWW T.Univ. Aachen1982	Bacteriological Analysis of Pilot Plant Rheinhausen	Fig. 6

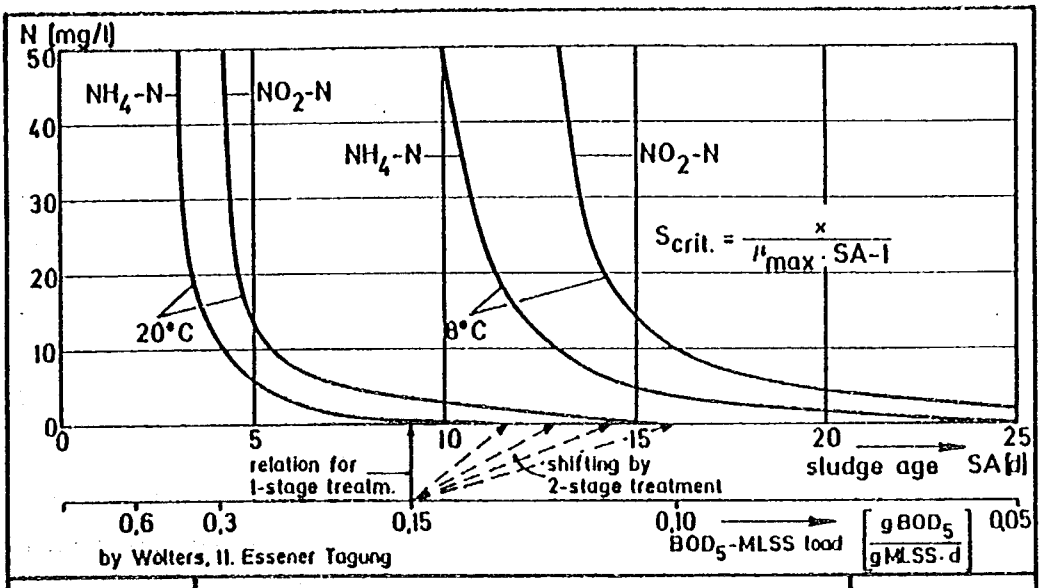
A very clear comparison of the surface ratios of bodies of different sizes is given in fig. 7. Even though the higher organisms have increased their reaction area through spurs, inversion, intestines and inner organs, the great advantage of procaryotes is self-evident. A diagramm by O.KLEE [acc. 6] shows again the very high metabolic intensity of the smallest organisms (fig. 8). In short, in the stage with the highest load a relatively high clarification efficiency is achieved with low energy consumption and extensive assimilation.

species	mass γ^{-1}	surface of bodies	surface of procariot	$O_{SP}:O_P$
dimension		cm^2	cm^2	-
1	2	3	4	5
Procariot	1 μm^3	$6 \cdot 10^{-8}$	$6 \cdot 10^{-8}$	1: 1
Eucariot	1000μm^3	$6 \cdot 10^{-6}$	$6 \cdot 10^{-5}$	1: 10
Mouse	0,1dm^3	130	$6 \cdot 10^5$	1: 46.000
Human	0,1 m^3	$13 \cdot 10^3$	$6 \cdot 10^9$	1: 460.000
Cow	0,5 m^3	$38 \cdot 10^3$	$30 \cdot 10^9$	1: 790.000
Elephant	3,0 m^3	$13 \cdot 10^5$	$18 \cdot 10^{10}$	1:1.400.000

ISWW T. Univ. Aachen 1982	Surface Ratio between Different Bodies as Function of an Equiv. Biomass Procariots	Fig. 7



ISWW T. Univ. Aachen 1982	Intensity of Metabolism of Organisms	Fig. 8



ISWW T. Univ. Aachen 1982	Residual N-Compounds as Function of Sludge Age and BOD_5-MLSS Load	Fig. 9

It has become evident that in the second, lowly loaded activated sludge stage with lower energy consumption a higher sludge age results than in a comparably loaded single stage plant (fig. 9). It is surmised that the causes for this are extensive elimination of the suspended solids in the first stage [7] and generally favourable conditions for microbial activity in the second stage. The higher sludge age, i.e. longer generation time means a better life expectancy for specialists e.g. nitrifiers. However in case of process operation with denitrification, inferior values for nitrogen removal are achieved due to the lower influent concentrations.

Experimental Results

In the meanwhile results from 30 experimental runs with 16 different waste waters are available at the Institute für

Siedlungswasserwirtschaft (ISWW) at the Technical University, Aachen. The results of the first experimental runs with 13 different waste waters are given in fig. 10. 13 experimental runs were carried out in pilot plants and 6 in large scale plants. 7 plants with a total clarification capacity of 1,95 mil. pop. eq. are at present under construction in Germany or have just been completed. Further plants are being planned.

Fig.10 appears to contain some contradictions. For the A-stage with an average clarification efficiency of 45 % a large fluctuation in the BOD_5 from η_A = 73 % to η_A = 14 % is documented. However, it turned out that even in plants with a lower efficiency in the A-stage a satisfactory overall result was achieved.

City	Vol. load (A) $kg BOD_5/m^3 \cdot d$	BOD_5 MLSS (A) $kg BOD_5/kg MLSS\,d$	BOD_5/COD_{Cr} Infl.	BOD_5 (A) Infl. mg/l	BOD_5 (A) Int.S.T. mg/l	η_A %	COD_{Cr} (A) Infl. mg/l	COD_{Cr} (A) Int.S.T. mg/l	η_A %	BOD_5 MLSS (B) $kg BOD_5/kg MLSS\,d$	BOD_5 Effl. mg/l	η_G %	COD_{Cr} Effl. mg/l	η_G %
Te Osterwalde O	8	—	0,38	220	60	73	579	330	43		19	91,4	101	82,6
Krefeld Δ	9,6	6,00	0,39	194	60	69,1	503	196	61	0,30	9	95,4	91	81,9
Rheinhausen I Δ	23	8,90	0,58	446	146	67,3	769	326	57,6	0,32	15	96,6	68	91,2
Schwarzenbeck X	9	5,56	0,95	721	252	65	761	436	42,7	A-stage investigated only				
Aachen III a X	10,9	3,13	0,61	231	103	55,5	380	190	50	A-stage investigated only				
Pulheim X	11,5	4,77	0,54	344	160	53,5	633	295	53,4	0,39	17	95,1	68	89,3
Aachen I/II X	5,1	3,40	0,43	104	52	50,1	244	196	20	0,14/0,08	3/12	97,1	45/43	81,6/82,5
Borken X	16,6	8,68	0,51	301	163	45,8	585	331	43,5	0,30	10	96,7	109	81,4
Aachen IV f X	10	3,66	0,61	231	127	45,2	380	229	39,5	A-stage investigated only				
Terborg O	9,6	—	0,43	214	142	34	497	315	37	—				
Herringer Bach X	7,1	3,20	0,30	139	49	64,5	466	140	70,0	A-stage investigated only				
Rheinhausen II Δ	15,5	4,20	0,49	283	190	33	581	394	32,2	2nd stage with pure oxygen				
Rumeln I X	10	4,90	0,39	393	273	30,4	1013	628	38	0,49	31	92,1	137	86,5
Rumeln III X	8,2	3,20	0,45	387	269	30,5	868	555	36,1	0,52	14	96,4	87	90,0
Lörrach X	8,8	3,60	0,47	169	137	18,6	363	324	10,6	0,22	13	92,3	86	76,3
Kreuztal I X	14,9	6,20	0,54	561	481	14,3	1042	846	18,8	0,87	7	98,8	90	91,4
Kreuztal II X	16,6	5,00	0,61	502	431	14,1	825	623	24,5	1,02	16	96,8	108	86,9
Neuenkirchen I X	7	3,20	0,51	138	84	39,1	242	170	29,8	0,20	6	95,7	68	71,8
Neuenkirchen II X	11,4	5,60	0,48	260	149	42,7	505	343	32,1	0,15	8	96,9	92	81,8

X by GETHKE · O by FAUBERT · Δ by DIERING/OASSEN

ISWW T.Univ. Aachen 1982 | Results of A-B-Treatment Investigations | Fig.10

change of COD_{Cr}/BOD_5 ratio in general (for one stage tr.):

	Inflow	effluent of BOD_5 MLSS-load				
		A-stage 6,0	0,6	0,3	0,15	0,05
$\frac{COD_{Cr}}{BOD_5}$	2,0	aer. 2,2 fac.<2,0	3	4 - 5	6 - 8	6 - 10

two examples for A-B-treatment:

	Inflow	effluent A-stage	η A-stage	effluent B-stage	η B-stage	MLSS-load of B-stage	plant
$\frac{COD_{Cr}}{BOD_5}$	$\frac{503}{194}=2,59$	$\frac{196}{60}=3,20$	$\frac{61\%}{69\%}$ aer. <1	$\frac{91}{9}=10,1$	$\frac{54\%}{85\%}$	0,30	Krefeld
$\frac{COD_{Cr}}{BOD_5}$	$\frac{825}{502}=1,64$	$\frac{623}{431}=1,45$	$\frac{24,5\%}{14,1\%}$ fac. >1	$\frac{108}{16}=6,7$	$\frac{83\%}{96\%}$	1,02	Kreuztal II (with brewery)

ISWW T.Univ. Aachen 1982	Shifting of COD_{Cr}/BOD_5 Ratio	Fig.11

The fluctuating efficiency of the A-stage and its effects on the efficiency of the B-stage are explained with the help of two examples (fig. 11). This can be expressed very well with change in the ratio of hard degradable substances to degradable organic compounds, expressed as the COD_{Cr}/BOD_{5C} ratio. For the right interpretation, knowledge of changes in this ratio for single stage plants as a function of the sludge load is necessary. The upper part of fig. 11 shows the normal displacements for german conditions.

The first example in fig. 11 (treatment plant Krefeld) has a higher content of hard degradable substances (COD_{Cr}/BOD_5 = 2.59 > 2,0), which is caused by the larger industrial amount. However the efficiency of the A-stage under aerobic conditions is for BOD_5 very impressive and particularly for COD_{Cr} very large. The result is a significant change in the COD_{Cr}/BOD_5 ratio from 2.59 to 3,20. With a generally good clarification efficiency, the BOD_5 is as expected more extencively eliminated than COD.

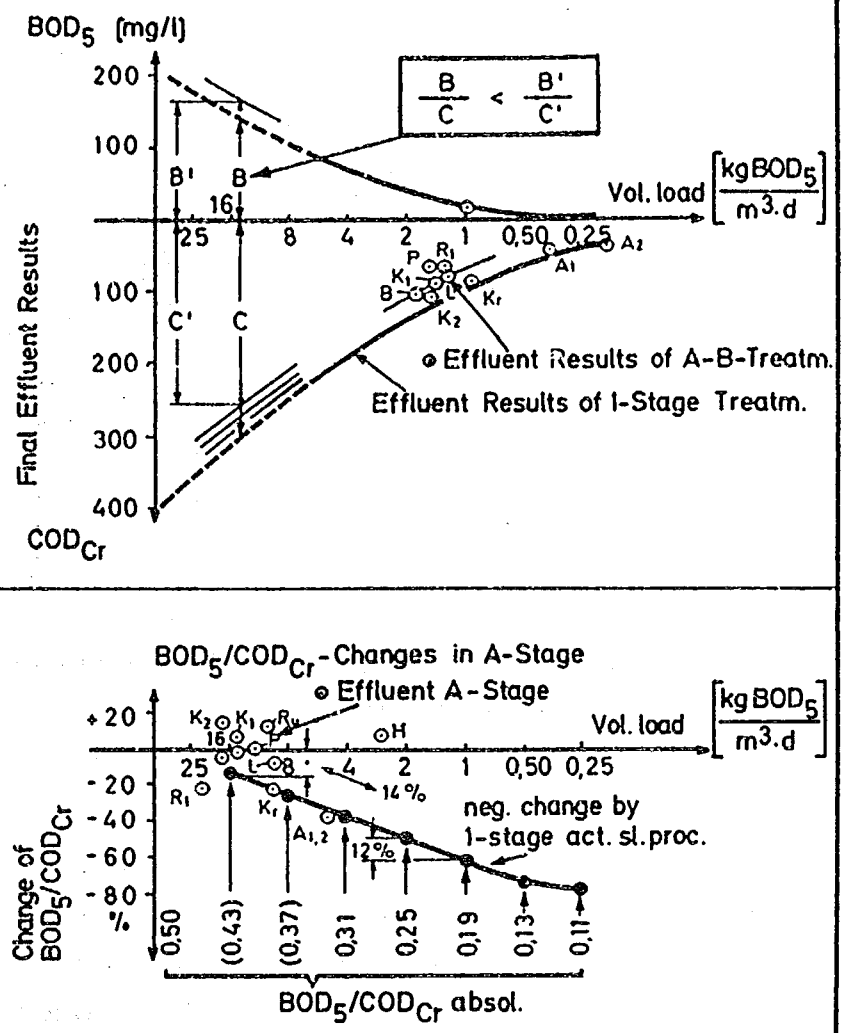
The example STP Kreuztal II shows that easily degradable waste water is present (COD_{Cr}/BOD_5 = 1,64 < 2).

Here it is surprising that for a fair initial clarification efficiency of the A-stage, the hard degradable contents in the A-stage are more extensively eliminated than the easily degradable contents. The COD/BOD_5 ratio decreases from 1,64 to 1,45! This appears at first contradictory. However as shown by tests, the A-stage of the Kreuztal II plant, which was not controlled by the oxygen input, was not sufficiently supplied with oxygen. Therefore anaerobic conditions were present in the A-stage. This obviously led to the decomposition of a part of the hard degradable organic compounds into easily degradable material. This hypothesis is supported by the fact that here in the in fact fairly highly loaded plant excellent percentage results were obtained. The same percentage clarification result of course cannot be expected in the second stage in Krefeld after the high preclarification efficiency of the A-stage. The final effluent results for both plants can be termed as excellent considering the influent concentrations.

The interesting results with anaerobic operation and good overall results have also been observed at other plants in Aachen, Rumeln and Gruiten.

A graphic presentation of the corelations discussed in the examples is shown in fig. 12 using several effluent results obtained with the help of the displacement in the BOD_5/COD_{Cr} ratio. The phenomenon of facultative anaerobic process operation explained earlier is clearly shown through a displacement of the BOD_5/COD_{Cr} ratio from the negative to the positive region.

Fig. 13 shows a comprehensive statistical comparison of the final effluent results of single-stage biological plants with AB-plants. It is clear that the effluent results of the AB-plants are better than those for single



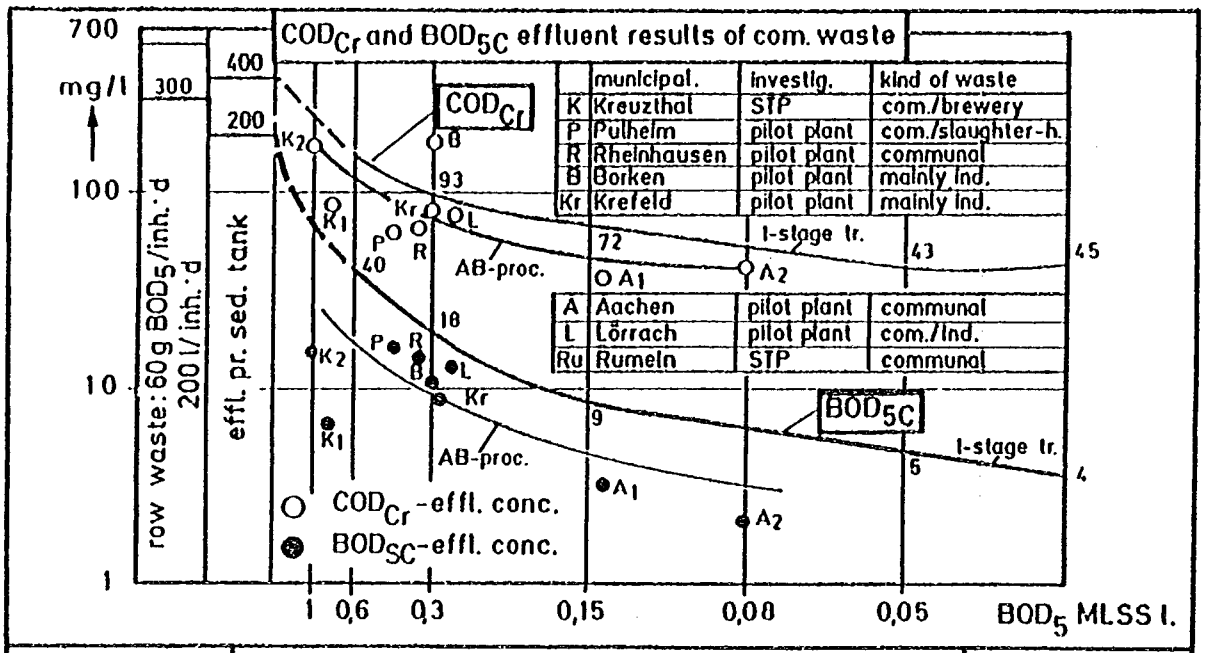
ISWW T.Univ. Aachen 1980	Shifting of BOD_5/COD_{Cr} Ratio as Function of Volume Load	Fig.12

stage plants with a corresponding sludge load in the second stage, with regard to the BOD_5 as well as the COD_{Cr} values.

Particulary from the viewpoint of governmental control, not only are the average effluent results of interest but also the process stability of the system. Here too the investigations show that the two-stage plants are better than comparable plants with respect to sludge load ratio.[1)]

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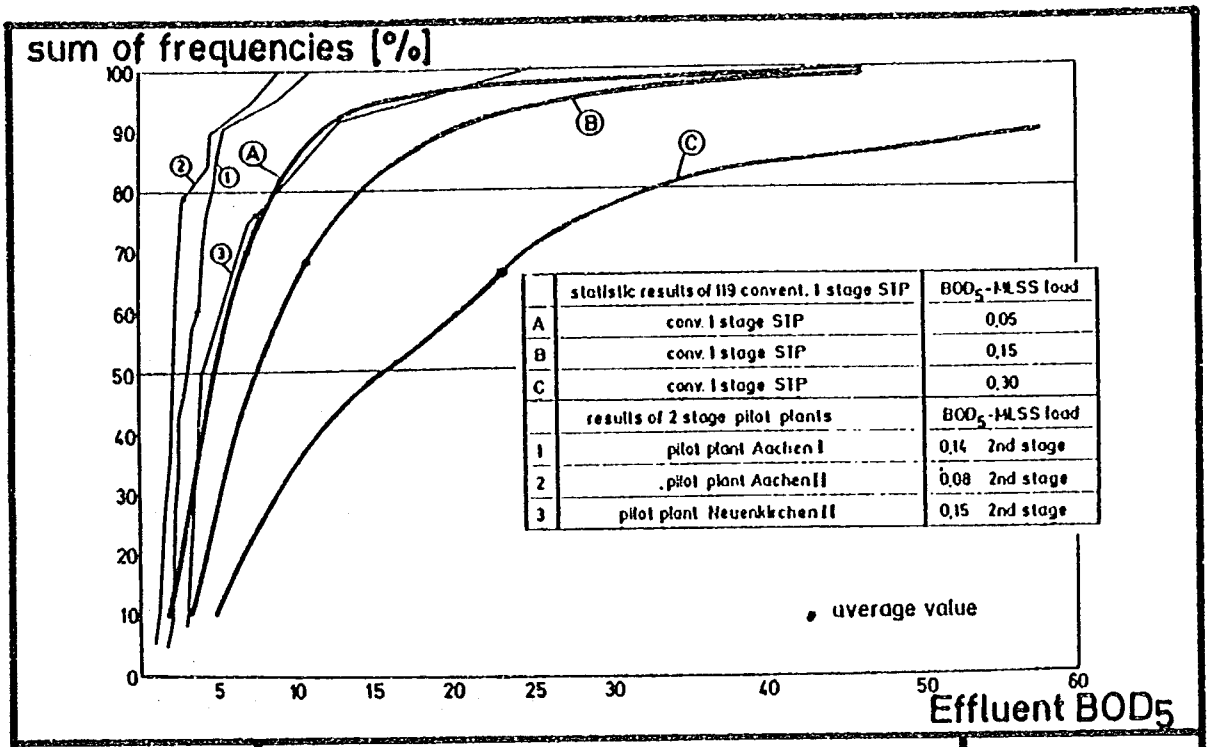
1) compare also paper G 9, fig. 4



ISWW T. Univ. Aachen 1982	Comparison of A-B Treatment Investigations with 1 stage biol. STP	Fig.13

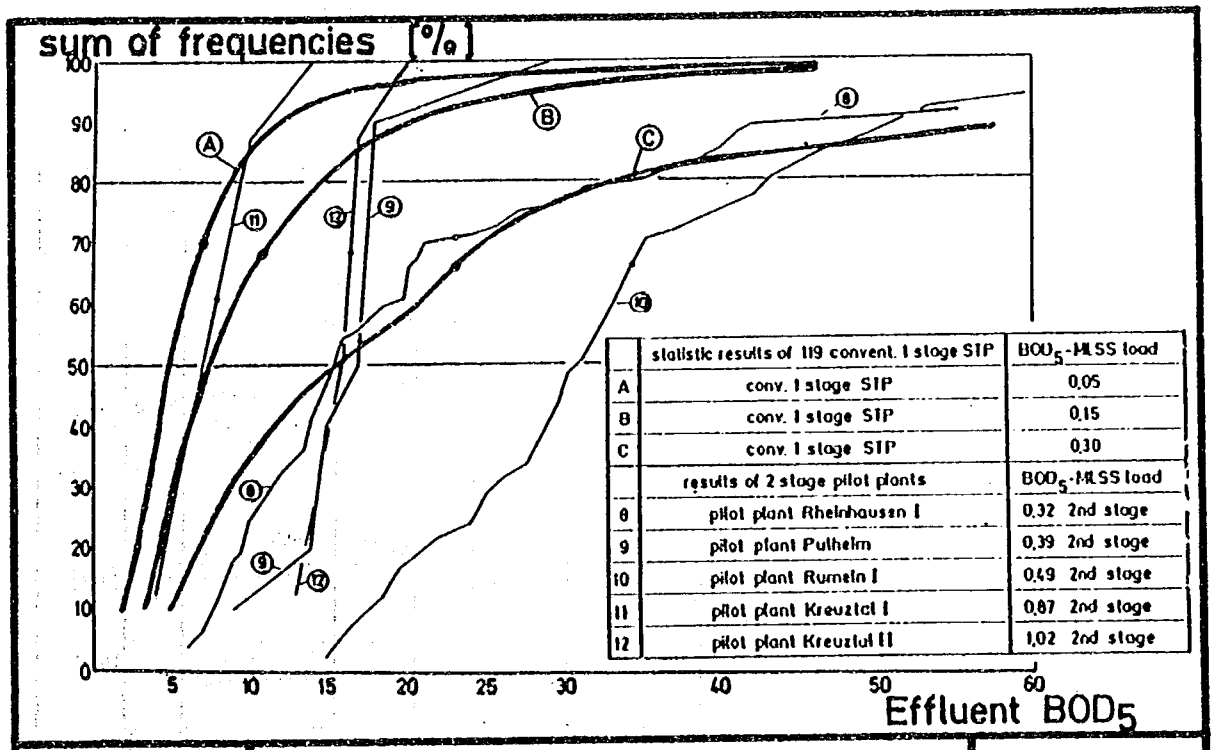
In preliminary experiments in Krefeld, e.g., short term pH-reductions up to pH 2 were measured. The pH value was buffered in the tank of the A-stage and in the intermediate clarification, such that the biocoenosis of the following lowly loaded stage remained unaffected with no loss in efficiency. The disturbed biocoenosis in the A-stage due to the short sludge age in this stage had already recuperated within a few hours and recovered its full efficiency.

Figs. 14 and 15 illustrate the empirical clarification efficiency of the AB-experimental runs as compared to single stage plants with the help of below-level frequencies of the total analyses. The curves are not directly comparable since the curves for single stage plants results from a large number of plants with a correspondingly high number of analyses as compared to the AB-plants presented here. The positions of the average values are however comparable and the curves also the qualitative conclusion of high process stability for AB-plants even after this emperical criteria.



	statistic results of 119 convent. 1 stage SIP	BOD_5-MLSS load
A	conv. 1 stage SIP	0,05
B	conv. 1 stage SIP	0,15
C	conv. 1 stage SIP	0,30
	results of 2 stage pilot plants	BOD_5-MLSS load
1	pilot plant Aachen I	0,14 2nd stage
2	pilot plant Aachen II	0,08 2nd stage
3	pilot plant Neuenkirchen II	0,15 2nd stage

ISWW T.Univ. Aachen 1982	Comparison of 1 Stage Biol. Treatment with AB-Treatm. - low load 2nd stage	Fig. 14



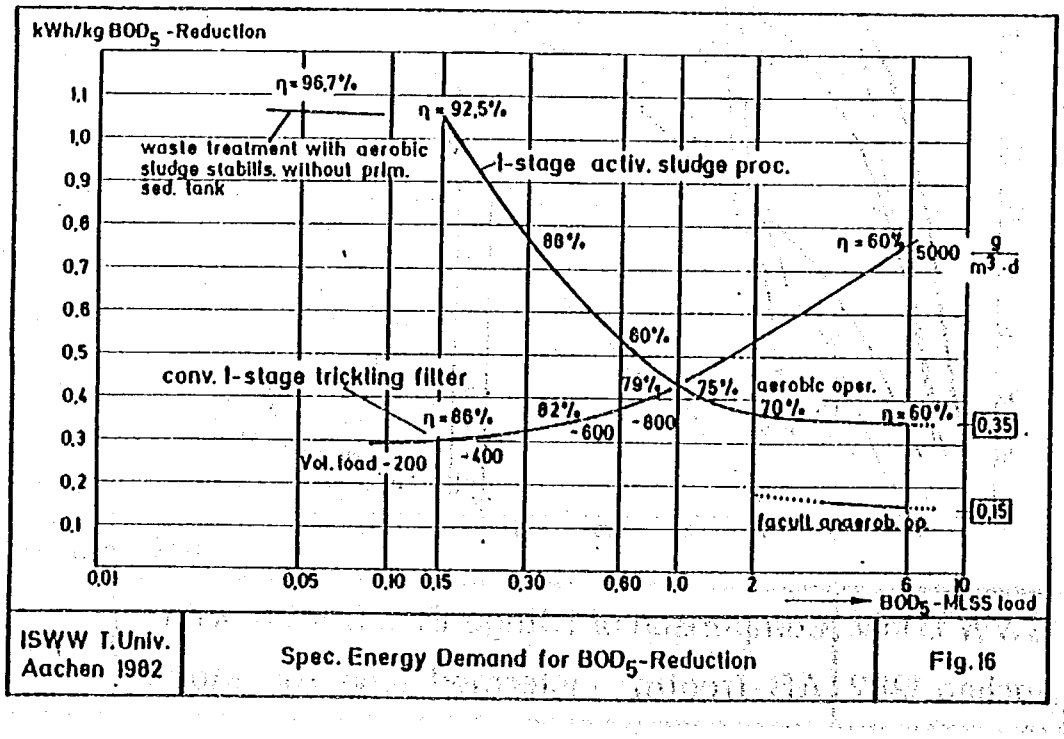
	statistic results of 119 convent. 1 stage SIP	BOD_5-MLSS load
A	conv. 1 stage SIP	0,05
B	conv. 1 stage SIP	0,15
C	conv. 1 stage SIP	0,30
	results of 2 stage pilot plants	BOD_5-MLSS load
8	pilot plant Rheinhausen I	0,32 2nd stage
9	pilot plant Pulheim	0,39 2nd stage
10	pilot plant Rumeln I	0,49 2nd stage
11	pilot plant Kreuztal I	0,87 2nd stage
12	pilot plant Kreuztal II	1,02 2nd stage

ISWW T.Univ. Aachen 1982	Comparison of 1 Stage Biol. Treatm. with AB-Treatm. - intermed. load 2nd stage	Fig. 15

Volume and energy needs of the AB-process

The specific energy requirements for the removed contamination is shown in fig. 16 as a function of the load of the biological stage [9]. It is discernible that the activated sludge process and the trickling filter process show opposite tendencies. The specific energy needs of the trickling filter increase with volume load due to higher recirculation pump requiry and higher construction resulting from the use of synthetic filling even though it does not require energy for oxygen input. The activated process, on the other hand requires energy for oxygen supply. This process shows decreasing specific energy demand for increasing load since nitrogen is not oxidised any more and the adsorbed suspended solids are not descomposed.

The area of lowest energy requirements is deliberately utilized in the A-stage of the AB-process.



	position	treatment volume (m^3) HL	PS/IS	LL/AST	FS	PTH	DI	STH	Sum	Energy biol. kWh/(inh. eq·a)	Energy total *)
BOD_5 - effluent <10 mg/l	A-B-process BOD_5 MLSS load: 1st stage: 5,0 2nd stage: 0,15	550	1.650	5.500	5.500	750	3.750	450	18.150	ED: 13,3	19,0
	1 stage act. sl. proc. BOD_5 MLSS l.: 0,05	-	-	26.700	9.900	780	-	-	37.380	ED: 24,7	28,8
BOD_5 - effluent 13 mg/l	A-B-process BOD_5 MLSS load: 1st stage: 5,0 2nd stage: 0,30	550	1.650	2.800	4.400	850	4.200	550	15.000	PD: 12,6 ED: 11,9	17,9 17,0
	1 stage act. sl. proc. BOD_5 MLSS l.: 0,15	-	1.650	8.900	6.150	720	3.270	430	21.120	PD: 15,9 ED: 13,1	21,4 17,9

*) Energy total: incl. biol., RS-pumping, mechanical part, sludge treatment and dewatering, other

SC	screen	IS	interm. sedim. tank	DI	digester
GCH	grit chamber	FS	final sedim. tank	MSD	mech. sl. dewatering
HL	highest load act. sl.	RS	return sludge		
LL	low load act. sl.	ES	excess sludge	ED	extensive denitrification
AST	act. sludge tank	PTH	pr. thickener	PD	partly denitrification
PS	prim. sedim. tank	STH	sec. thickener		

ISWW T. Univ. Aachen 1982 | Treatment Volume and Energy Consumption of A-B-Proc. and Comparable 1-Stage Treatm. (Expl. for 100.000 inh.) | Fig. 17

The first example worked out for german conditions (fig. 17) with an average demand of < 10 mg BOD_{5C}/l in the final effluent has as an alternative to the AB-process with a BOD_5-MLSS load = 0,15 loaded second stage only an aerobic stabilising process with BOD_5-MLSS load = 0,05. The comparison shows that the single stage process requires about double the volume needs and about 50 % higher electrical energy as against the AB-process.

The second example shows alternatives for an average effluent value of 13 mg BOD_{5C}. The single stage treatment volume here is about 40 % higher than for the AB-process. Hereby it is interesting that when operated with extensive denitrification, only a small energy gain is possible, which increases considerably with conventional operation.

An exact comparison must also consider that the AB-process produces 15 - 25 % more sludge, which however has more energy than the activated sludge of conventionaly operated plants. Exact gas production values are not available at present, however, a preliminary investigation has already started in Aachen.

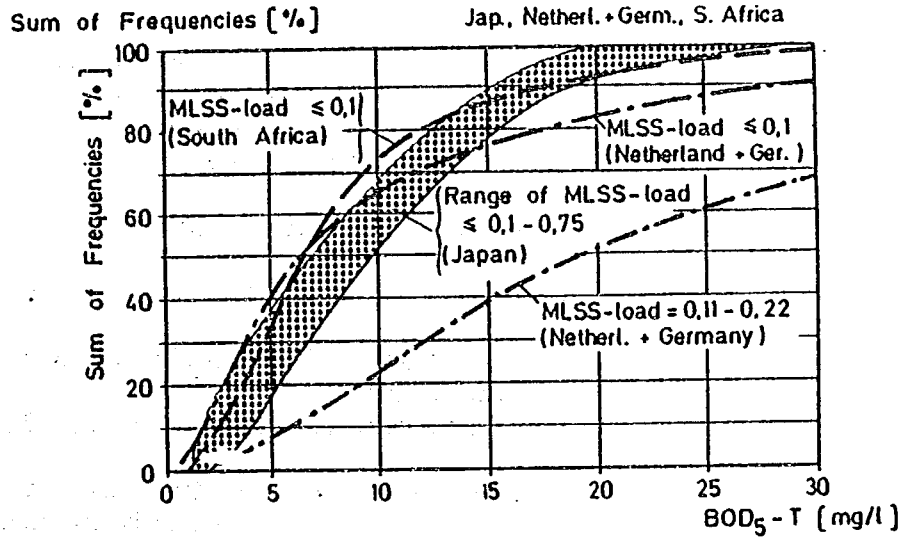
Concluding Remarks

The two-stage biological treatment process according to the AB-principle is an interesting possibility for mideuropean conditions with influent concentrations of > 150 mg BOD_5/l, specially for average and larger plants. In addition to the good effluent results and high process stability, the process is an interesting alternative to conventional treatment plants also with respect to needed construction volume and energy consumption.

Transfer of the results shown to average japanese conditions is however problematic. The curves which show the sum of frequencies of the japanese activated single stage plants are different from european curves due to the generaly lower waste water concentrations in Japan (comp. fig. 14 - 15). The japanese effluent values for BOD_{5T} - based however on a small amount of data - are almost independent of the sludge load ratio (fig. 18). It is to be concluded that the results shown here for AB-plants can be compared to only a few japanese experiences with waste water concentrations > 150 mg BOD_5/l, so that it would be meaningful to conduct separate experiments.

	MLSS - load	Number of Samples	Number of Plants	Average Infl. Conc. of Aeration Tank BOD_5 [mg/l]
Japan	≤ 0,10	140	4	72
	0,10 - 0,22	206	8	82
	0,23 - 0,40	63	3	77
	0,41 - 0,75	79	4	82
Netherland [1] + Germany	≤ 0,1	775	48	309
	0,11 - 0,22	185	12	270
South Africa [1]	≤ 0,1	246	8	~200 - 250

1) by Damiecki



ISWW T.Univ. Aachen 1982	Final Results of 1-Stage Treatment International Comparison	Fig.18

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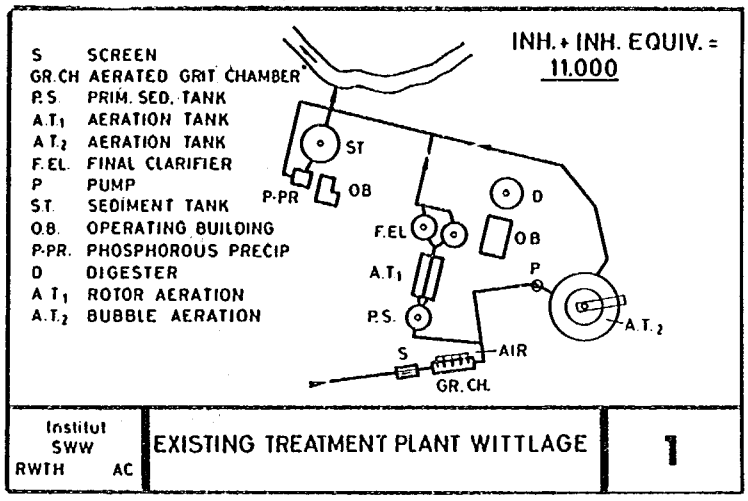
Appendix 1

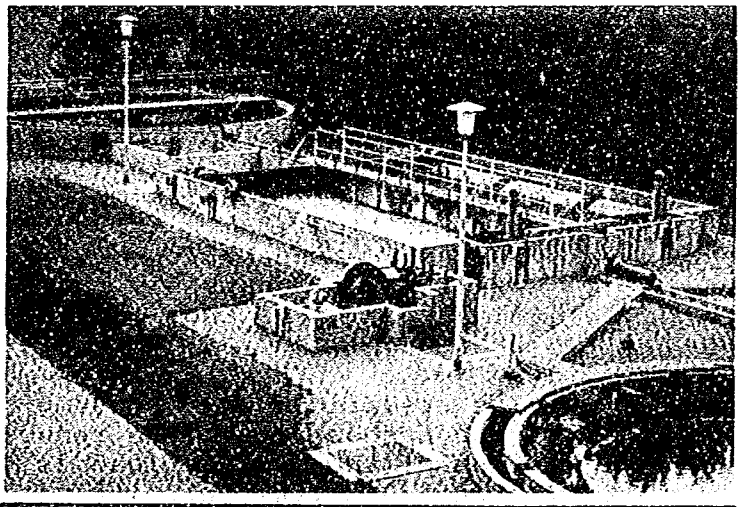
Proposal for the expansion of an overloaded conventional treatment plant from 11 000 inh./inh. equiv. to 30 000 inh./inh. equiv.

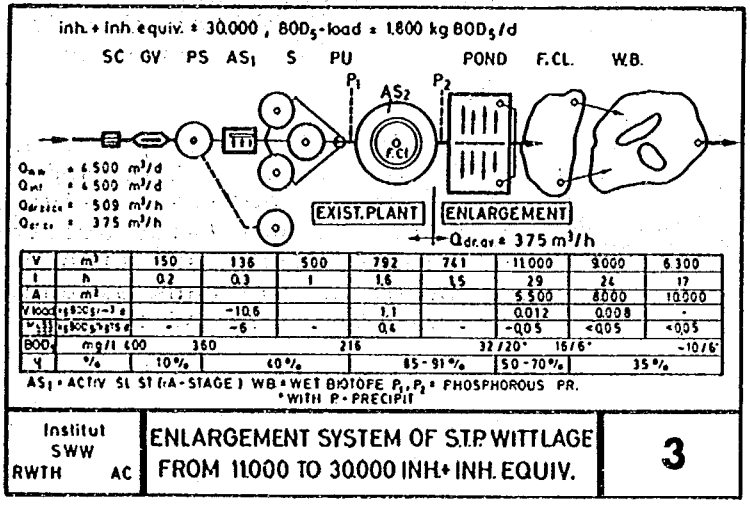
Worked out by Prof. Dr. Ing. B. Böhnke

Present:

A two-stage activated plant for 11000 inh./inh. eqiuv. is present.







The first activated slugde is aerated with Kessener brushes (rolls), the second activated sludge line is a Schreiber counter-current system with bubble aeration (fig. 4). A digestion tank with I= 600 m^3 is available. The clarified water is added to the Dümmer lake and must be extensively clarified. Lewer infiltration water influent is about the same as the waste water, at times it can sink to "Zero". At present the clarified waste water is subsequently treated in the phosphate precipitation plant (fig.1). The individual stages of the plant are in good order.

Requirement:

Expansion to 30 000 inh./inh. equiv., BOD_5 effluent = 7 mg/l
PO_4-P in the effluent = 0,5 mg/l

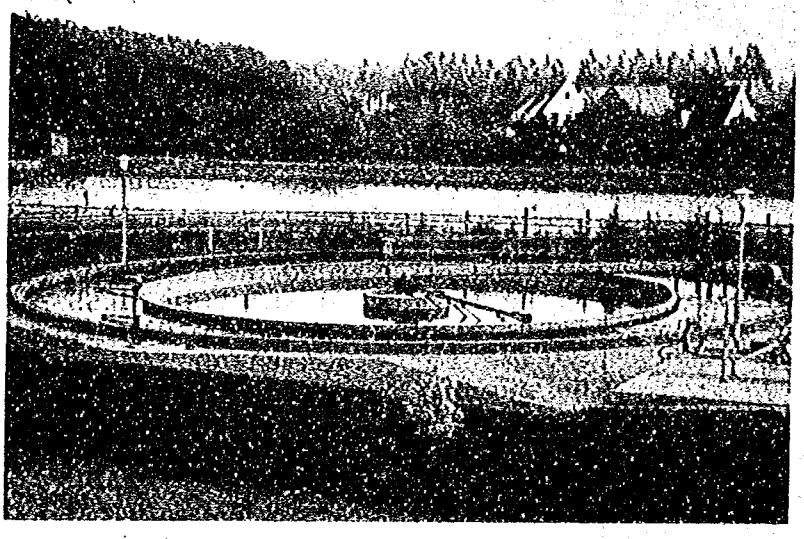
Solution:

The process chosen is shown in fig. 3. The 22 500 m^2 required for expansion are available. The smaller act. sl. stage (brush aeration, see fig.2) is operated as a highly loaded A-stage with a volume load of 12 kg $BOD_5/m^3 \cdot d$ and BOD_5-MLSS load 6. The secondary clarification for the A-stage is to be expanded. The second activated sludge line is to be connected in series and operated a sludge load ratio of 0,40 kg BOD_5/kg MLSS·d. After this stage an average BOD_5 effluent of about 32 mg/l is achieved with phosphate precipitation (P_1). With

phosphate precipitation P_1 an average effluent of about 20 mg BOD_5/l is attained.
Advanced treatment takes place in the following aerated pond.
A P-precipitant (Fe-II sulfate) is added to the pond influent. The double precipitant addition at P_1 and P_2 taken as a whole requires less precipitant and attain a better precipitation (PO_4-P in the effluent = 0.5 mg/l).
Cost estimates for the conventional expansion of this treatment plant load to an investment of 4950 000,- DM at a specific cost of 260.-DM/inh.

Calculations carried out by a consultant led to an investment amount of 4.500 000.- DM including P-precipitation.
The expansion according to the proposed solution re uires:

1.	Primary sedimentation tank for the A-stage	300 m^3	á	600.- DM/m^3	=	180 000.- DM
2.	Aerated ponds	11 000 m^3	á	70.- DM/m^3	=	770 000.- DM
3.	Secondary sedimentation tank	9 000 m^3	á	60.- DM/m^3	=	540 000.- DM
4.	Wet biotope	6 300 m^3	á	45.- DM/m^3	=	284 000.- DM
5.	Sludge thickener	300 m^3	á	330.- DM/m^3	=	100 000.- DM
6.	Precipitation station				=	50 000.- DM
7.	Channel and pipe laying				=	250 000.- DM
8.	Secondary works				=	300 000.- DM
9.	Safety etc.				=	526 000.- DM
	Total				=3	000 000.- DM



The expansion according to the pond aeration method is significantly cheaper (=55%). The addition of a highly loaded A-stage Followed by ponds reduce markedly the difficult to degrade organic substances.

The specific energy consumption necessary for the reduction of the organic load is about 30 % less in spite of the very much better clarification at a higher process stability.

The activated sludge produced in the preclarification and a part of the activated sludge from the A-stage (=1st activated sludge stage) are treated in the digestion tank already present. The rest of the



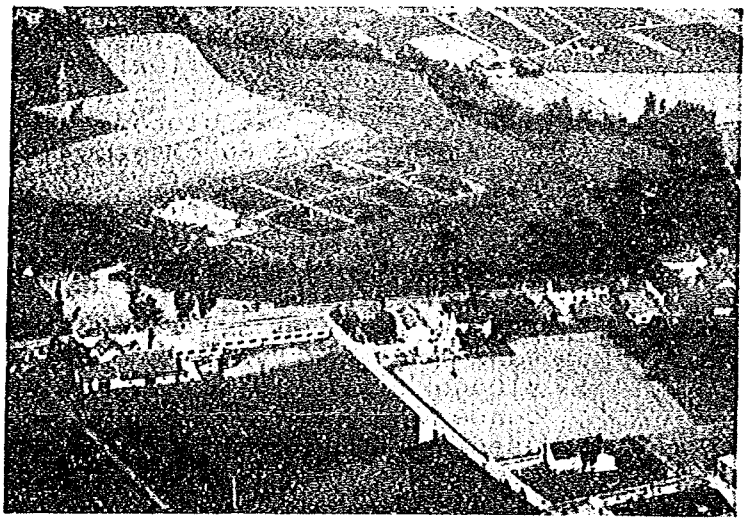
excess sludge from the A-stage and the excess sludge from the 2 nd activated sludge stage are stored in the aerated ponds. The sludge as aerobically stabilized as it sinks to the bottom and removed once a year. The maintainance requirements for this multiple stage treatment plant is insignificantly more than the older plant. The following ponds require little maintainance. The wet biotope with a depht of 60 cm grows over and one-third of it must be removed yearly, whereby phosphates, nitrates and algae are retained.

The existing sludge thickining tank for the stabilized sludge have very good conserved (fig.5) and will be used further on.

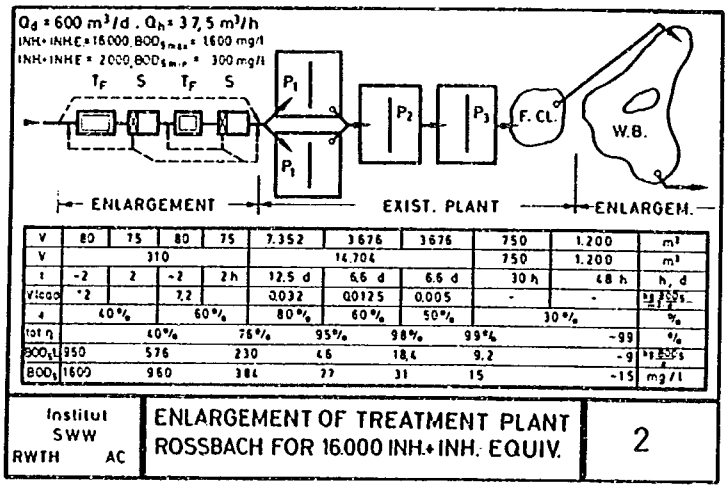
Appendix 2

Proposal for the expansion of an oxidation pond plant to an adsorption-oxidation pond plant for seasonal operation (inh./inh. equiv$_{min}$ = 2000, inh./inh.equiv.$_{max}$ = 16000)

Worked out by Prof. Dr. Ing. B. Böhnke

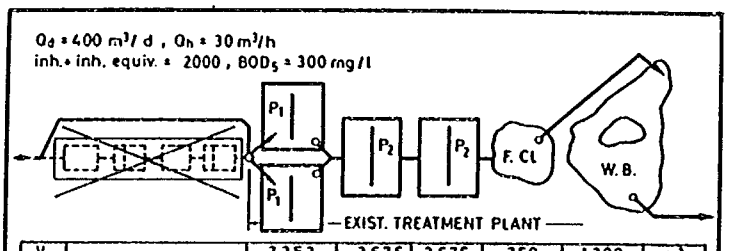


The community Rossbach in Bavaria has about 2000 inh. Some of the small scale industries in the vegetable processing sector cause considerable loads from June to January. The treatment plant was planned for 6 500 inh./inh.equiv. and is operated as an aerated pond plant (fig. 1). During the season loads of up to 30 000 inh. inh. equiv. were monitored in the treatment plant. The internal measures (sieves, screens) the loads were reduced to 16 000 inh./inh. equiv. The treatment plant must be expanded.



V	80	75	80	75	7.352	3.676	3.676	750	1.200	m³
V	310				14.704			750	1.200	m³
t	-2	2	-2	2h	12.5 d	6.6 d	6.6 d	30 h	48 h	h, d
Vt load	*2		7.2		0.032	0.0125	0.005	-	-	kg BOD5/ kg TS d
η	40%		60%		80%	60%	50%	30%		%
tot η	40%		76%		95%	98%	99%	-99		%
BOD_{5t}	950	576	230	46	18,4	9.2		-9		kg BOD5/ d
BOD_5	1600	960	384	77	31	15		-15		mg/l

Institut SWW RWTH AC	ENLARGEMENT OF TREATMENT PLANT ROSSBACH FOR 16.000 INH.+INH. EQUIV.	2

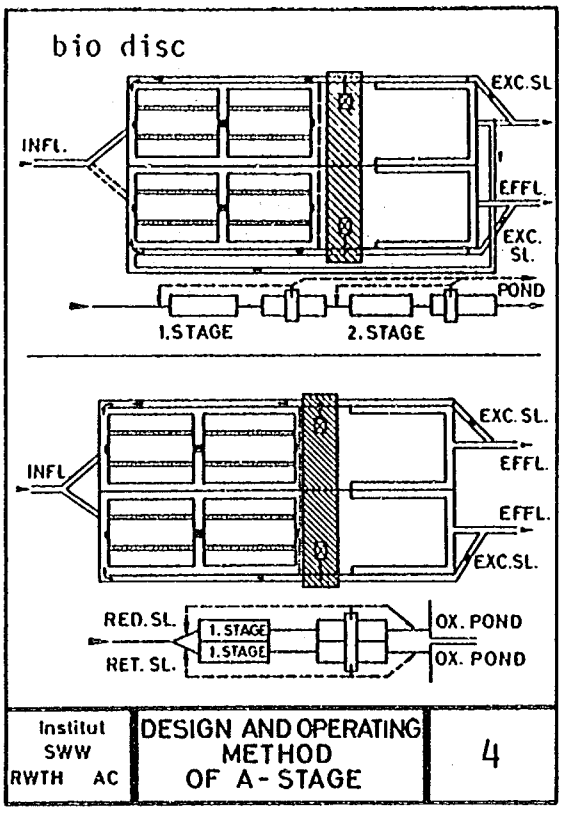


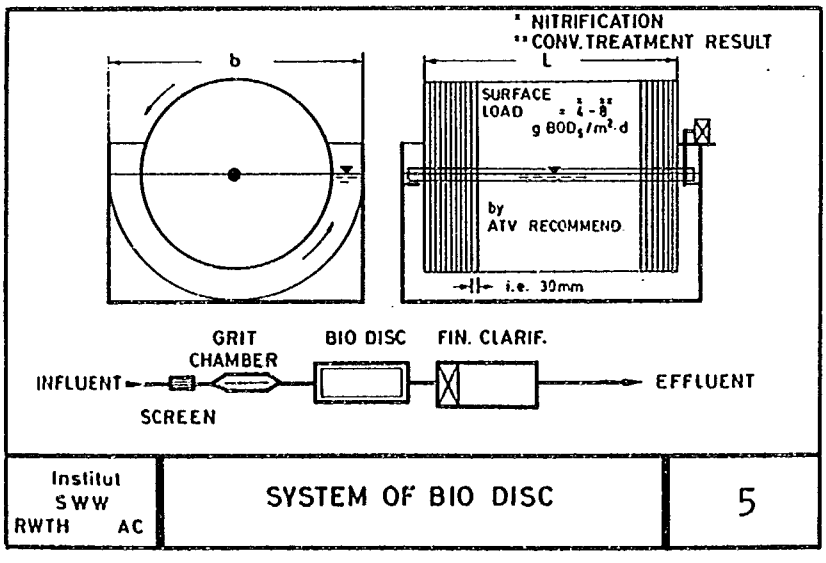
V		7.352	3.676	3.676	750	1200	m³
V		14.704			750	1.200	m³
t		18.4 d	9.2 d	9.2 d	1.8 d	3 d	h, d
Vol load		0.016	-0.007	-0.003			kg BOD5/ kg TS d
η		-80%	-60%	-50%		-30%	%
tot η		80%		92%	96%	97%	%
BOD_{5tot}	120	24	9.6	-5		3.5	kg BOD5/ d
BOD_5	300	60	24	12		≤ 10	mg/l

Institut SWW RWTH AC	ENLARGEMENT OF TREATMENT PLANT ROSSBACH FOR 2.000 INH.+ INH. EQUIV.	3

An expansion with oxidation ponds requires too much area (Δ F=22 000 m^2). A single or 2-stage highly loaded activated stage preceding the old plant requires only 2 500 m^2 area including the following wet biotope. Continuous sludge treatment is not necessary since the sludge is stored in the aerated ponds. The aerobicaly stabilised sludge has to be removed once a year. The operating data at full load (16000 inh./inh. equiv.) during season are presented in fig. 2. During the off-season the preceding highly loaded stage is closed down and only the oxidation ponds are operated. The operating and load data is presented in fig.3. For both operating conditions, the minimum requirements of 20 mg BOD_5/l are observed.

The costs of expansion to 16 000 inh./inh. equiv. - i.e.an additional 9500 inh./inh.equiv. - are 900 000,- DM. The specific construction costs at 95,-DM/inh. are extremely low. A conventional expansion by means of an activated sludge system through





heated digestion tanks would cost 300,- DM/inh. or 2850000,- DM. Expansion by mean of a conventional activated plant preceding the ponds whereby the sludge is also stored in the aerated ponds costs 1300 000,- DM. The specific energy costs for 1 kg BOD_5 are 1.2 kWh according to this process. For a preceding highly load activated stage (=adsorption stage) the specific energy requirement is 0.5 - 0.60 kWh/BOD_5-reduction. In other words an energy saving of 50 %. The preceding adsorption stage (A-stage) removes about 76 % of the total load (fig.2). Both the activated sludge stages are e.g. equipped with bio discs, that operate according to the method shown in fig. 5. The forced aeration achieved hereby allows for aerobic and facultative anaerobic operation sludge loads of about 7 - 10 kg $BOD_5/m^3.d$ without additional artificial aeration.

The development of the preceding highly loaded stage (A-stage) as shown in fig.5 can operate as a single or double stage depending on the load. The available retention time of 2 - 4 h is quite enough considering the sludge index of 100 ml/g. The secondary sedimentation tank should be operated with suction removers.

Conclusion

1. The existing ponds can be retained
2. Due to the multiple stages, operation can be adjusted to the load. In season the A-stage is turned on. The adaptability is exellent.
3. The plant has very high process stability due to the biological multiple stages and achieves a high clarification efficiency (η = 99 %).

4. The construction costs are very low (95.- DM/inh.).
5. The extra area required is very little. The preceding A-stage has an volume requirement of only about 20 l/inh. and achieves a clarification effect of 76 %. Conventional treatment processes (nitrification plants with BOD_5-MLSS = 0.15) need a specific volume of 240 l/inh. for a clarification effect of 95 %.
6. The specific electric energy requirement under full load (16000 inh./inh. equiv.) is very low at 0.50 - 0.60 kWh/inh.·a
7. Even with an expansion to 1600 inh./inh. equiv. a continuously run sludge treatment plant is not necessary, since the sludge is stored in aerated ponds.

THE INSTRUMENTATION AND THE EFFECTIVE OPERATION OF KANAGAWA SEWAGE TREATMENT PLANT

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Construction Department

Sewage Works Bureau

The City of Yokohama

1. Introduction

The increased quantities of sewage to be treated, the enlarged scale of sewage treatment plant, and the improvements in sewage treatment have necessitated larger scale and more complicated treatment systems. As a countermeasure, quantity control has been conventionally carried out and recently quality control has also been emphasized with the establishment of severer effluent quality standards into receiving streams, lakes and oceans.

Yokohama City has taken countermeasures to meet the situation in terms of saving manpower, resources and energy in the construction and maintenance of sewage treatment installation. The city introduced electronic equipment for the purpose of saving manpower, and to obtain stable and efficient sewage treatment.

The instrumentation and operation of Kanagawa Sewage Treatment Plant, operated in March in 1978, which is the seventh and the largest plant in Yokohama is described below as an example.

2. Outline of Kanagawa Sewage Treatment Plant

2.1 Outline of the Plant

The plant covers Kanagawa-ku, Nishi-ku, and Hodogaya-ku, which are the urbanized regions of Yokohama City, as the treatment district. Most of the area employs a combined sewer system. The sewage is collected into the plant either directly or via relay-pumping stations located in Hodogaya and Sakuragi, treated by an activated sludge process, disinfected, and discharged into Tokyo Bay. Because the plant is situated near the Yokohama Station in the metroplitan area and the site area is so limited, the management house, pumping room, and grit chamber are collected into one building of a solid metrical structure to use the limited site effectively.

Site Area	103,000 m^2
Designed Treatment Area	5,030 ha
Designed Treatment Population	994,000
Designed Treatment Sewage Flow	
Dry Weather Flow	517,000 m^3/D (First stage, 172,000 m^3/D)
Wet Weather Flow	1,748,000 m^3/D (First stage, 582,000 m^3/D)
Treatment Process	
Activated Sludge Process	

The excess sludge produced is dewatered after concentration and incinerated in other treatment plant.

Fig. 1 shows a map of the treatment district and Fig. 2 a plan of the plant.

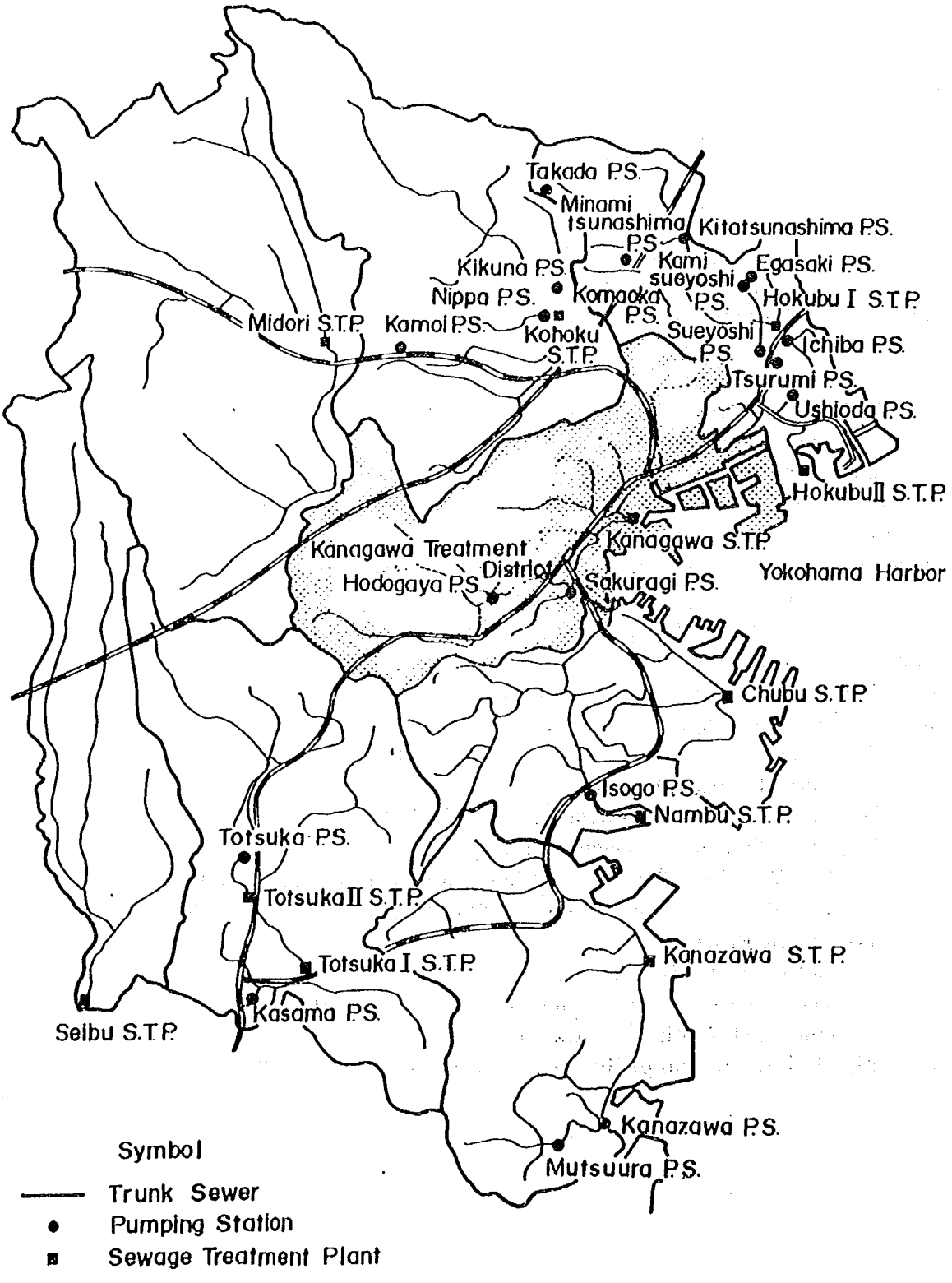


Fig. 1 The Sewerage Plan of Yokohama

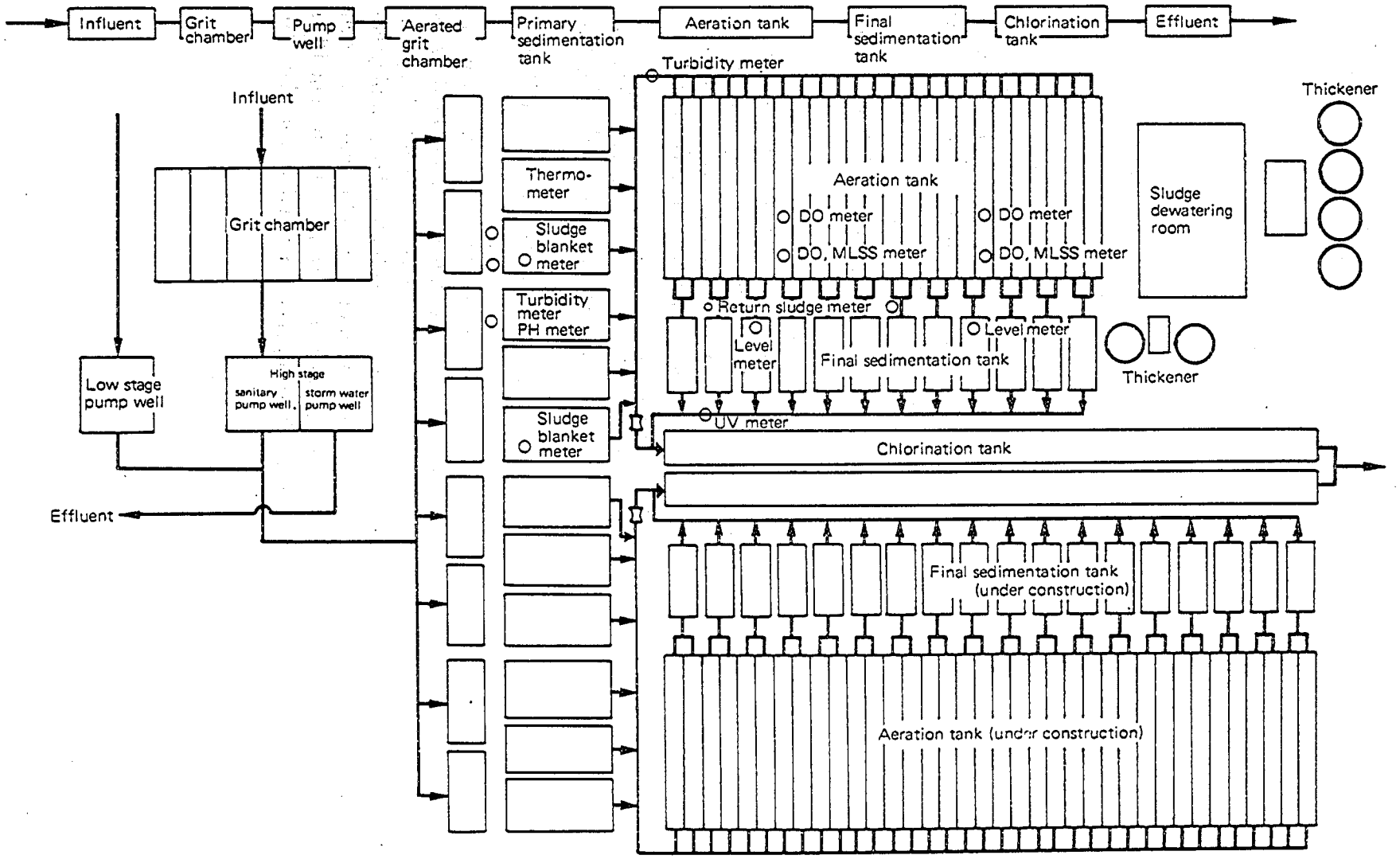


Fig. 2 Plan of Kanagawa Sewage Treatment Plant

2.2 Characteristics of the Plant

The newest type of centralized supervision and distributed control system is employed for the purpose of improving quality control and maintenance. In order to achieve those purposes, the plant was designed taking the following into consideration.

1) Automatic is scaled-up and the quality of control is improved to minimize the personnel required for operation and control of the plant and the relay-pumping stations (one or two employees are used under normal conditions).
2) Central or local troubles in computer system have no overall and detrimental effects.
3) The control system is intended for the selection of the optimal control mode according to the conditions and for readily taking measures to meet technical progress in future.
4) Quality control of a high degree will be introduced step by step.
5) Centralized supervision of the processes for the sewage treatment plant and the operation of neighboring pumping stations are computerized. Control is performed by distributed control system in which a microcomputer is provided with a control function per each electrical control room.

3. Supervision and Control System and Instrumentation of Other Sewage Treatment Plants in Yokohama City

3.1 Transition of Supervision and Control Sytems

Since the start of operation of the first plant in Yokohama, Chubu Sewage Treatment Plant in 1962, Nanbu, Hokubu I, Totsuka II, Kohoku, Midori, Kanagawa, and Kanazawa Sewage Treatment Plants in this order were constructed and the eight plants are at present under opertion. Seibu, Totsuka I, and Hokubu II plants are at present under construction.

The methods of operation of the plants are subjected to centralized supervision and automatic control, and the details vary with time in relation to the development and innovation of technology.

The course until the construction of kanagawa Sewage Treatment Plant is described below.

The initial plants, Chubu and Nanbu, employed a centralized supervision and control system for each local block, blocks which were determined by dividing the treatment processes. Afterwards, a centralized supervision control system including the remote control of the plants and their subordinate relay-pumping stations was gradually introduced until the completion of Midori Sewage Treatment Plant which was the sixth plant in the city. Kanagawa Sewage Treatment Plant employed a more advanced system, centralized supervision and distributed control system.

3.2 Centralized Supervision and Control System for Each Local Block

The Chubu Sewage Treatment Plant (dry weather flow 96,300 m³/day) introduced the newest control system at that point.

The main points are as follows:

1) Three-block centralized supervision control consisting of sewage treatment block, sludge transfer block, and sludge treatment block. The last process was managed by Public Cleaning Bureau for treating the sludge by mixing with collected night soil because of the special circumstances in Japan.
2) In the sewage treatment block, the grit chamber apparatus, the main pumps and the blowers are subjected to centralized supervision, handling, and automatic operation in the control center. Other control means include a constant air flow control by means of an analog controller; intermittent withdrawal control of sludge from primary sedimentation tanks, excess sludge, and thickened sludge by means of motor timers; and automatic control of main pumps with mechanical screens by means of a wired logic sequence.
3) The instrumental devices measured only the quantity of sewage. The water levels were measured by Selsyn more-typed floats, and the flow rates by electromagnetic, Parshall flume, and venturitube systems.
4) In the centralized supervision and control system for each local block, there are the problems that each block should be provided with a control desk and the conditions of the neighboring blocks are incomprehensible. Countermeasures taken since 1973 or so were the indication of data on a graphical panel and tabulation of data required for control on a definite format of a control disk at regular intervals by menas of data logging equipment. The countermeasures lead to supervision by one-man control with fewer operators required than in the case of the conventional control system.

Nanbu Sewage Treatment Palnt (dry weather flow, 234,000 m³/D), the second plant in the city, also employed a three-block centralized supervision and control system consisting of sewage treatment, air delivery, and sludge treatment blocks. The plant was remodeled to a centralized supervision and control system in 1974 as was the case with the above Chubu Sewage Treatment Plant.

3.3 Centralized Supervision and Control System

Hokubu Sewage Treatment Plant I (dry weather flow, 196,000 m³/D), began operating in 1968, and employed a centralized supervision and control system with reference to the course of conventional plants.

The main points are as follows:

1) Supervision was conducted with a graphic control panel on which the treatment lines were graphically indicated on the front and on which lamps indications of the statuses of equipment and meters such as flow meters, angle meters for open-and-close of valves, voltmeters, and ammeters were functionally arranged in important positions of the lines, Industrial televisions were located for the supervision of important in-site positions such as grit chamber, generator room, and sludge dewatering room.

2) Control is in principle carried out automatically. In the absence of automatic operation, equipment out of service will be switched into a spare equipment automatically. Because of the manual operation the ON-OFF operation of the main equipment such as inlet gates, mechanical screens, main pumps, and blowers can be performed on a control desk different from the control panel.
3) In addition to the measured values indicated on the graphic panel, data necessary for control are automatically tabulated on a definite format of a daily report at regular intervals by means of data logging equipment.

The above allows supervision by a one-man control system with fewer personnel than with the previous supervision and control system and enables consistent control of the treatment processes.

4. Supervision and Control System

4.1 Centralized Supervision and Distributed Control System

As the scale of the treatment plant increses, the number of instruments and facilities will also increase, resulting in ever larger amounts of control and information, accompanied by larger sites and increases communication, power cables and manpower. The data transmission system was employed for reducing the amound of wiring for control and instrumentation as was the case with a substation per process control block for reducing the wiring needed for power cables. Automated distributed control and digitalized data transmission were performed by a microcomputer for the reliability of control if data-way troubles, and centralized system failures occurred. These systems reformed the centralized supervision and operation system and made the role of the man-machine interface more important.

4.2 Data Transmission System

The system enables the high-transmission of large quantities of data, voluntary interstational data exchange and N:M construction as an insite transmission system. The system made the central master station duplex by means of data-way equipment with a transmission speed of 1.5 MB/sec. Eight remote stations are presently installed and will be increased by two when the lines are increased and additionally by three for the sludge transportation and storm water storage tanks (Fig. 3). Table 1 gives the amount of input and output in each station. Fig. 4 gives the locations of the distributed controlled electric rooms.

4.3 Centralized Computer System

The central computer system was duplexed and took into consideration future expansion for the general control and automated distributed control of the complex and scale sewage treatment plant with high reliability.

4.4 Centralized Supervision and Operation System

Conventional data supervision collected in the center and the number of process operations will make the instruments larger, more complex and of higher performance, resulting in over-load of the operator control. However, the instruments will not be able to change immediately.

The steps of the innovation are as follows:

1) A Mosaic-type graphic panel was installed for collectively grasping the line operation with highly centralized indicating lamps.
2) A trouble-indicating panel according to the calssifications of "EMERGENCY" or "TROUBLE" was installed in the operation desk for providing rapid supervision for each process block.
3) All of the above details are indicated on the CRT display and the contents of the troubles are automatically recorded.
4) The CRT-equipped operator console plays an important role of the man-machine interface in the central supervision and control devices. The CRT display will indicate high-degree contents such as the condition of the equipment, process data, selective operation, and operation control modes even during the control desk operation. Selection of the control modes fed into the local microcomputer, modification of setting values and parameters, and establishment of the sequences of the equipment operation are available by the CRT display with a combining key board or a light pen.
5) Out-site instruments coupled to the TM/TC system are operated in the same manner as the above system.

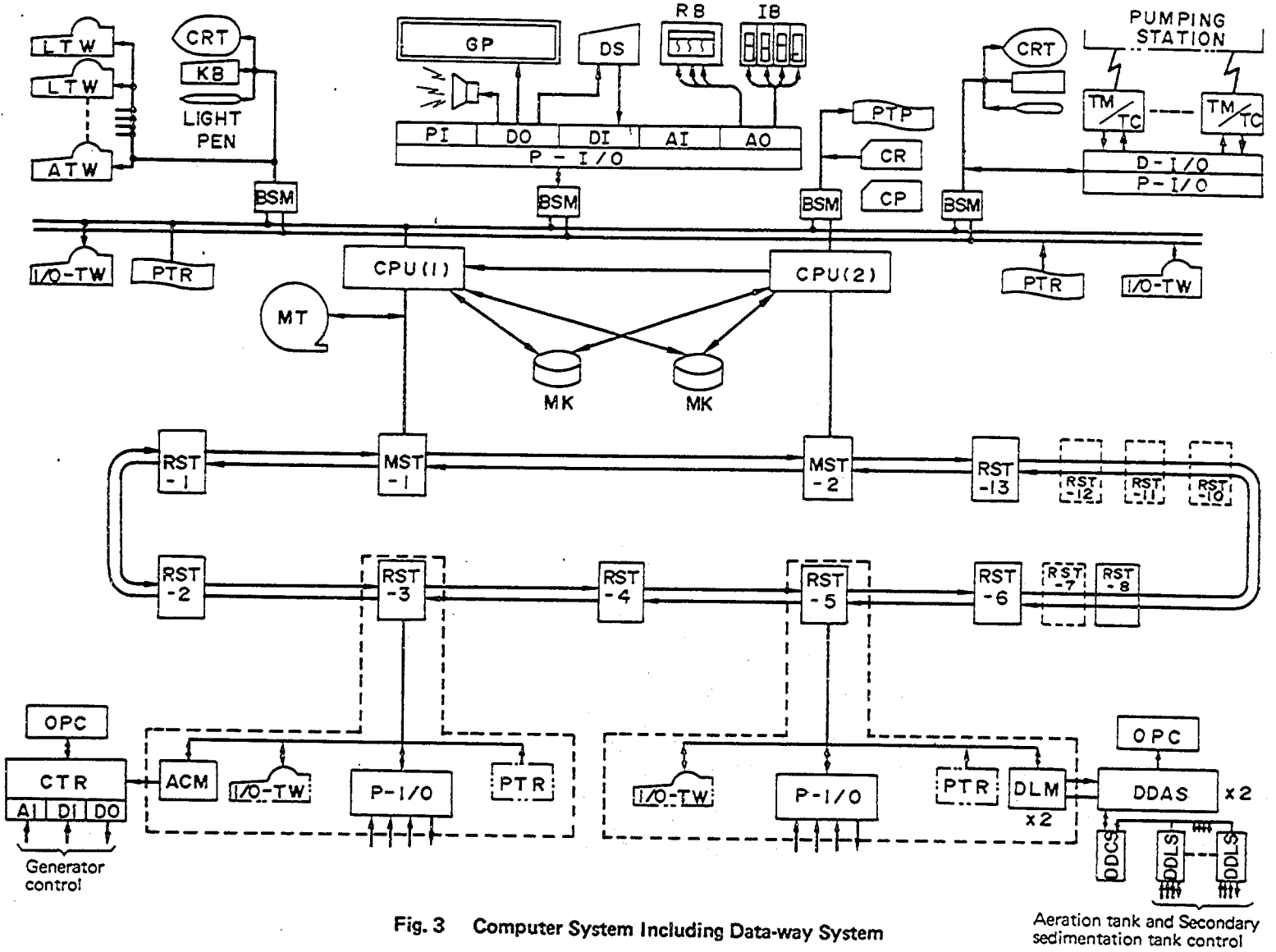


Fig. 3 Computer System Including Data-way System

Table 1 Data-way Remote Station P-I/O and Constitution of Control Equipment

Remote station		Number of process inputs and outputs			Microcontroller	Sequence controller	Main installations for supervision and control in each station
No.	Name of subdivision	Digital	Analogue	Pulse		PROSEC-2000	
RS-1	Receiving and distribution	272	64	32	1 unit	1 unit	66kV-12.5MVA x 2 — Common, spare receiving-miniclad and 6.6kV distributing boards
RS-2	Pumping room and grit chamber	1.600	192	48	2 units	7 units	High-stage sanitary pumps, high-stage storm pumps, dust collectors, and gates of mechanical scrapers of settled grit
RS-3	Generator	456	160	15	1 unit	3 units	5.625kVA x 3 diesel generators, dummy load
RS-4	A-series primary sedimentation tank	648	96	32	1 unit	4 units	Two series of pre-aeration, scum skimmer, gates, primary sludge collectors, and sludge pumps
RS-5	A-series secondary sedimentation tank	1,088	304	64	2 units	2 units (3 units)	Three series of air valves for controlling aeration, gates, secondary sludge collectors, valves for sludge withdrawal, return sludge pumps and excess sludge pums
RS-6	Sludge III	1,604	80	32	2 units	4 units	Filter presses, and centrifuges (centrifugal dewatering equipment)
(RS-7)	B-series secondary sedimentation tank	—	—	—	—	—	Installed in future
(RS-8)	B-series primary sedimentation tank	—	—	—	—	—	Installed in future
RS-9	Disinfection and filtration	608	144	16	2 units	1 unit (2 units)	Injectors of sodium hypochlorite, defoaming pumps, and sand filters
(RS-10)	Sludge I	—	—	—	—	—	Installed in future
(RS-11)	Sludge II	—	—	—	—	—	Installed in future
(RS-12)	Storm water storage tank	—	—	—	—	—	Installed in future
RS-13	Blower	1,088	144	32	1 unit	2 units (4 units)	A-series blowers (B-series blowers)

* The number of process inputs and outputs will be increased in future.

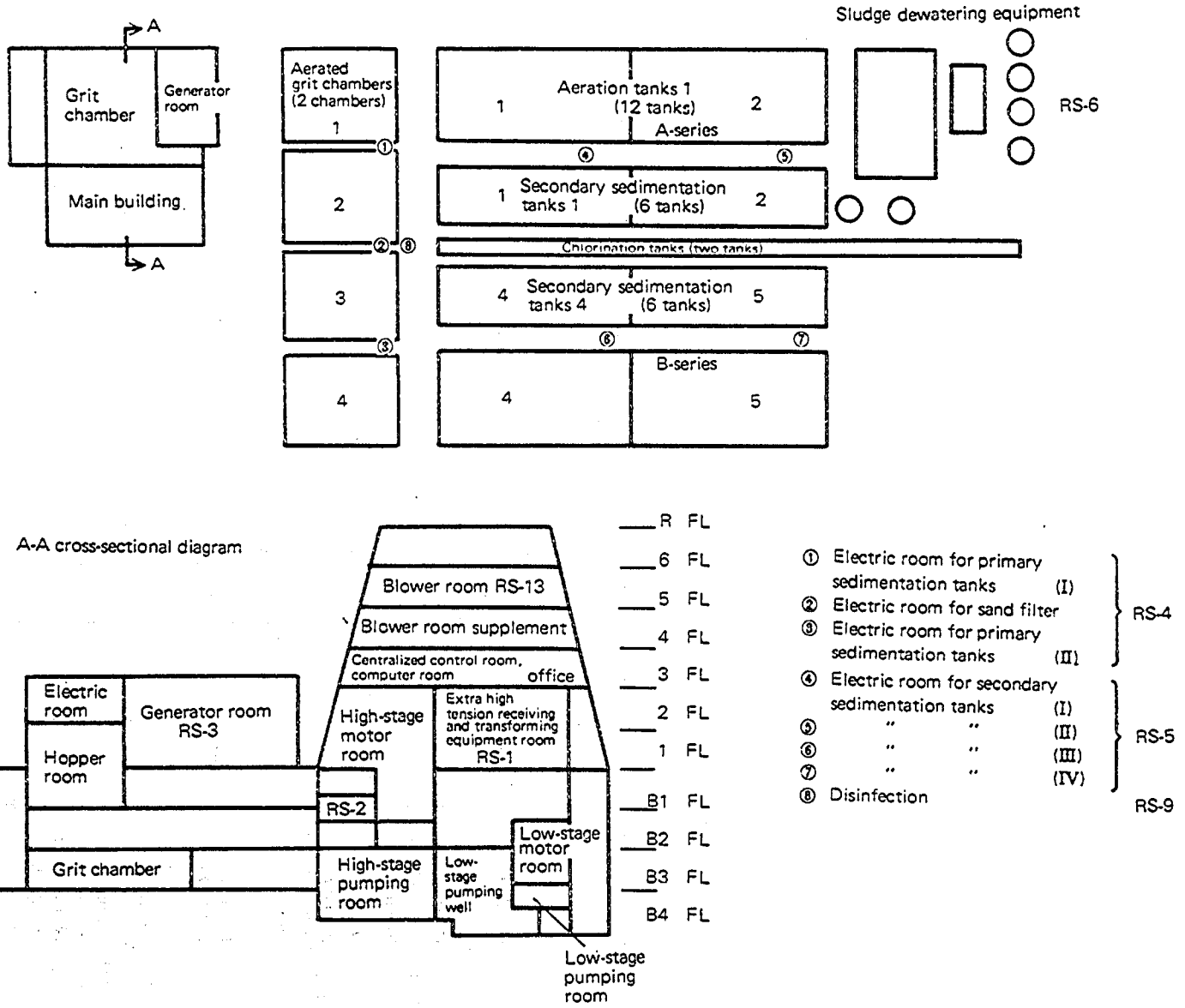


Fig. 4 Diagram of Distributed Controlled Electric Rooms

5. Instrumentated Control System

5.1 Digital Control

The instrumentation and control system of the automated distributed control system using digital controllers instead of conventional analogue controllers had the advantages of reliability resulting from hierarchical control and flexibility to soft-wired installation of additional equipment and to soft-wire remodelling of the equipment. The control system of each of the blocks was improved based on the conditions of the present equipment. The new control system was so constructed that supplementation and alternation of the control program could be made only by replacing or rewriting the memory module, and the hardware only by the replacement of the name plates of the control mode switches, when a model of good controllability would be developed. The expansion of sensors has been carried out both in quality and quantity to fulfill the contents of the control, and excess space was obtained for the supplement of sensors installed in future.

The respective control blocks have respective plural control systems as necessary and programs requiring complex calculation.

Each control system from the sequence from the lower to upper controller level enables arbitrary selection at the center and field positions. A selective circuit sequence has been prepared such that, if the upper system fails, the system is automatically switched to the lower system. The above system is an important feature in the stable operation of the sewage treatment plant and effective in management of operators, back-up operational span limit, and prevention of malfunction in the case of system failure. The system enabled analysis of treatment effectivencess by operating a series unit based on a different control model when the investigation and development of a quantity-controlling model is to be considered.

When the distributed control system is employed, the process data should be transferred between remote stations.

In the air flow control by a blower, the process data involve aerated grit chamber flow rates from the control block of the primary sedimentation tanks, and DO in each line, activated sludge process effluent, air flows of channel, pressure data of aeration tank inlets in each series from the control block of the sencodary sedimentation tank, and in addition, operation status and control modes of operation.

The process data in the sludge treatment control involve the reception of the respective amounts of sludge withdrawal and sludge blanket and transmissin of the predetermined amounts of sludge withdrawal. The disinfection control data involve the effluent from the secondary sedimentation tank and the effluents of storm water from the main pumps. The power line control requires the communication of the operation conditions in a large capacity load of the main pumps and blowers and load control interlock data. These data-way systems transfer these data when the control system requires them.

Table 2 gives the contents of the instrumentation control by the digital controllers.

5.2 Instrumentation

The characteristic of the instrumentation of Kanagawa Sewage Treatment Plant is the extensive introduction of instruments (sensors) for control and supervision.

Although quantitative sensors have been used for the determination of water levels, flow rates, air flow rates, pressures and temperatures, qualitative sensors could not be introduced extensively because of difficulties of maintenance, reliability and cost.

Meters could be improved to a degree of usability for supervision on the basis of experiments using DO meters, MLSS meters, sludge concentration meters, turbidity meters and sludge blanket meters.

The meters were adopted entirely and a part of them were used for control, Fig. 5 shows the locations for metering. A list of meters used for the sewage treatment plant is shown as accompanying information.

6. Evaluation of Automatic Water Quality Measuring Equipment

Various kinds of automatic water quality measuring equipment, which will be called water quality meters later, were introduced into Kanagawa Sewage Treatment Plant for the control or supervision of water qualities. However, the maintenance and performance of this equipment remain largely unknown, and therefore field tests were conducted in five sewage treatment plants including the plant to evaluate these meters during a period from August, 1979 to December, 1980.

6.1 Outline of Water Qaulity Measuring Equipment as the Objects

The Plant is equipped with 7 kinds, a total of 22 water quality meters: pH meters, DO meters, turbidity meters, MLSS meters, sludge blanket meters, sludge concentration meters, and U.V. spectrophotometers. Evaluation was made on DO meters, MLSS meters, turbidity meters, and sludge concentration meters (for return sludge) for use in control, and U.V. spectrophotometers for use in the supervision of effluent qualities.

6.1.1 Types and Principles of the Equipment

a) DO Meters

A DO meter is composed of a teflon diaphragm, a gold cathode, a copper anode, and internal electrolyte, and a temperature-compensating thermister. Although materials used in the anodes and cathodes vary with the manufacturer, the measuring principle is in any case dependent upon the diaphragm galvanic cell system. Fig. 6 shows the structure diagram.

b) MLSS Meters

The MLSS meter relates to the application of the measurement of scattered lgiht. Beams from a light source will cause primary scattering and multiple scattering due to suspended solids in water, and any scattered light is received into two receiving elements. The signals treated by the calculation give MLSS concentrations. The principle and the structure are shown in Fig. 7.

c) Turbidity Meters

Turbidity meters are dependent upon two kinds of principles: measurement of scattered light on the surface, and determination of the ratio of the quantities of transmitted light to scattered light. In the former system, beams are directed to the static water surface onto which a sample is allowed to overflow to produce scattered light.

Turbidity is obtained by determining the scattered light dependent upon the scattered light quantity correlated to the number, shape, and surface areas of the turbidity components. In the latter system, beams are directed to a sample water passing through a cell to give transmittent light and scattering light scattered by particles in the sample water. Turbidity is obtained by determining the ratio of the transmittent light and the scattered light received by the respective receiving elements. The respective principles and structures are given in Fig. 8.

Table 2 List of the Contents of Digital Control (1/2)

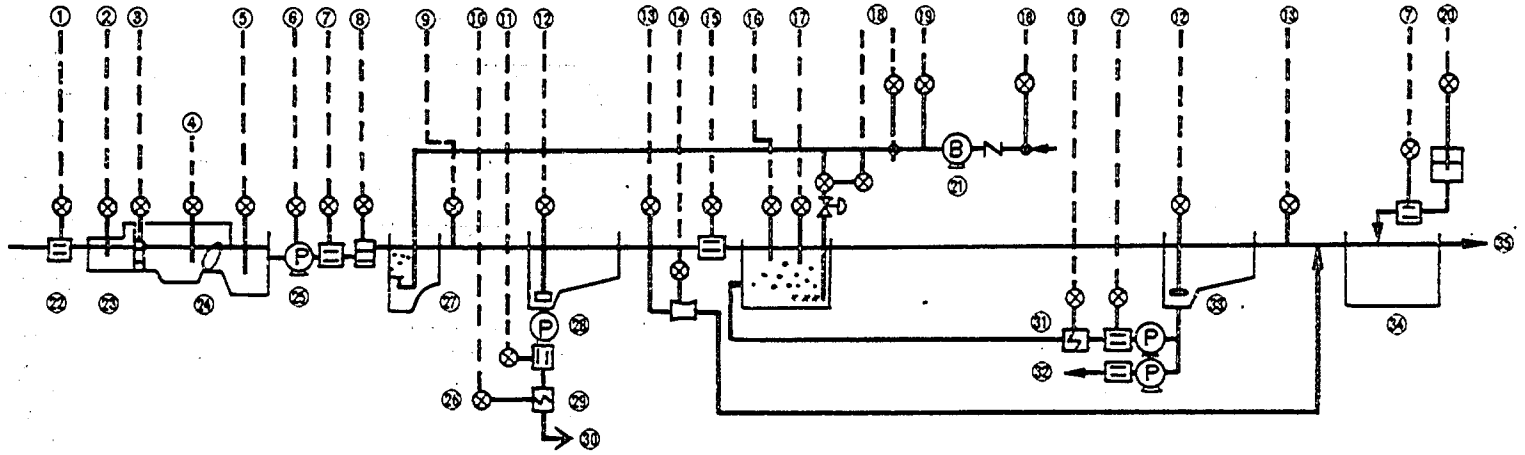
Remote station (Control block)	Control mode	Division of control functions			Objects of control	Note
		Upper controllers	Lower controllers	Sequence controllers		
Main pump Grit chamber Gates	(1) Water level control	◎	○		Five high-stage pums (10) (1) One set of cerbius equipment (650kW — 1450kW switching system) (2) Liquid resistors 1500kW for storm water 1,250KW for storm water 2	Number under operation control, speed control
	(2) Flow rate control 1,2	◎	○			
	(3) Water level control		◎			
	(4) Flow rate control		◎			
	(5) Alarm-setting equipment			◎		
	(1) Water level control	◎	○		Four low-stage pums (9) (1) One set of cerbius equipment (two 500kW switching system) (2) Two liquid resistors (for 500kW)	The same as the above
	(2) Flow rate control 1,2	◎	○			
	(3) Water level control		◎			
	(4) Flow rate control		◎			
	(5) Alarm-setting equipment			◎		
					Eighteen grit chambers, two communicating gates	
					Ten gates for discharge channels	
Blowers A-series (B series blowers are now under construction)	(1) DO/inflow ratio control	◎	○		Flour A-series blowers 4 (6) (1) Two 450kW blowers 2 (275m³/M-6800mmAq) (2) Two 600kW blowers 2 (4) 370m³/M-6800mmAq) (3) Four suction air rate-controlling valves 4 (6)	Number under operation control, suction valve control
	(2) Constant DO control		◎			
	(3) Inflow ratio control		◎			
	(4) Constant pressure control		◎			
	(5) Constant rate of air flow control		◎			

◎ : Main instruments ○ : Supplement instrument for ◎-marked

Table 2 List of the Contents of Digital Control (2/2)

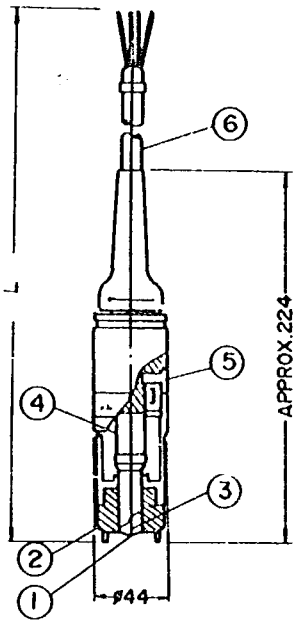
Remote station (Control block)	Control mode	Division of control functions			Objects of control	Note
		Upper controllers	Lower controllers	Sequence controllers		
Primary sedimentation tank A-series (B-series are under construction)	(1) Constant rate of inflow control		◎		Aerated grit chamber air flow rate-control valve x 2/series	
	(1)		◎		Two mechanical scrapers of settled grit, one set of conveyors	Speed control
	(1) Preset withdrawal control	◎	○		Nine primary sludge withdrawal valves/series (three/tank x three tanks/series) Two primary sedimentation tank sludge pumps/series accompanied by speed control	
	(2) Continuous withdrawal control	○	◎	○		Withdrawal on the basis of fixed preset valves. According to the instruction "Start withdrawal"
	(3) Intermittent withdrawal control	○	◎	○		
	(4) Timer/sludge blanket control			◎		
Secondary sedimentation tank A-series (B-series)	(1) DO control		◎		Aeration tank Eight air flow-rate control valves/series	
	(2) Constant rate of air flow control		◎			
	(3) Constant air flow control		◎			
	(4) Balancing air flow control	○	◎			Co-acting with blowers
					Three return sludge pumps/series accompanied by speed control	
	(1) Constant rate of return sludge control		◎	○		
	(2) Constant return rate control		◎	○		
	(3) Constant flow rate control		◎	○		
	(1) Total sludge control	◎	○	○	Two excess sludge pumps/series accompanied by speed control	Two modes: Continuous/ intermittent
	(2) Constant rate of withdrawal control		◎	○		
	(3) Constant volume/fixed preset control		◎	○		
	Balancing withdrawal flow rate control		◎		Six secondary sedimentation tank sludge withdrawal flow rate control valves/series	
Disinfection					Three sodium hypochlorite injectors 3 (5) (1) 360ℓHx2 (2) 1200ℓHx1(3)	
	(1) Constant rate of injection control		◎			
	(2) Constant total volume of injection control		◎			
					Two sodium hypochlorite injection pumps (2)	
Treated water filtration	(1) Changing rate of water level control	◎				
	(2) Constant water level control			◎	Filters 2 (4)	

◎ : Main instruments ○ : Supplement instrument for ◎-marked



1 Ultrasonic flow meter
2 Pressure-type water level meter
3 Open-degree meter
4 Screen-related differential pressure meter
5 Pressure-type water level meter
6 Rotation meter
7 Electromagnetic flow rate meter
8 Pressure-type water level meter
9 PH · thermometer · turbidity meter
10 Sludge concentration meter
11 Electromagnetic flow rate meter
12 Sludge blanket meter
13 Turbidity meter
14 Parshall flume
15 Electromagnetic flow rate meter
16 DO meter
17 PH · thermometer · DO meter
18 Orifice flow rate meter
19 Thermometer, pressure gauge
20 Ultrasonic level meter
21 Blower
22 Inflow channel
23 Grit chamber
24 Pump well
25 Discharge channel
26 Grit chamber
27 Aerated grit chamber
28 Primary sedimentation tank
29 Primary sedimentation tank Widthdrawn sludge
30 To thickener tank
31 Return sludge
32 Excess sludge
33 Secondary sedimentation tank
34 Chlorination tank
35 Effluent

Fig. 5 Main Location for Metering



①	ELECTRODE (Au)
②	DIAPHRAGM (CARTRIDGE TYPE)
③	THERMISTOR FOR TEMPERATURE COMPENSATION
④	ELECTRODE (Cu)
⑤	INJECTING HOLE OF ELECTROLYTE
⑥	3-CORE CABLE ø 11.5

Fig. 6 Diagraph of DO Meter Structure

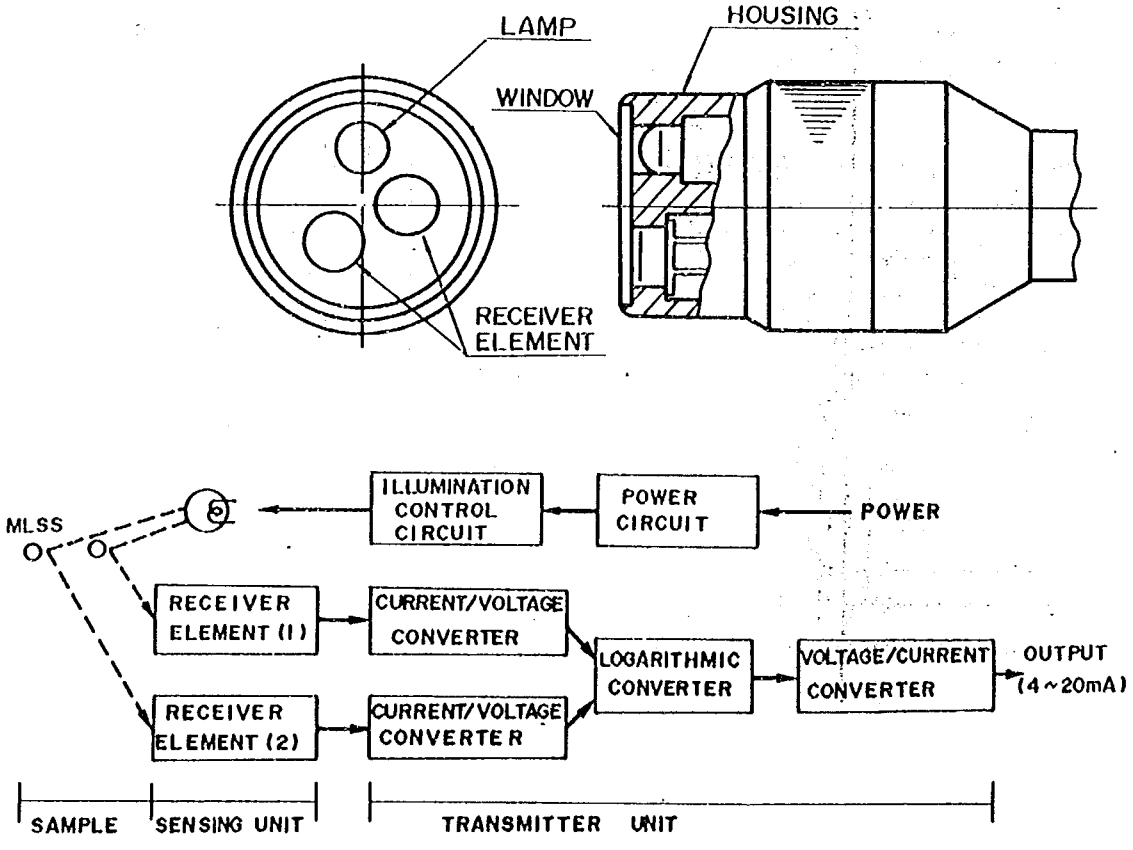
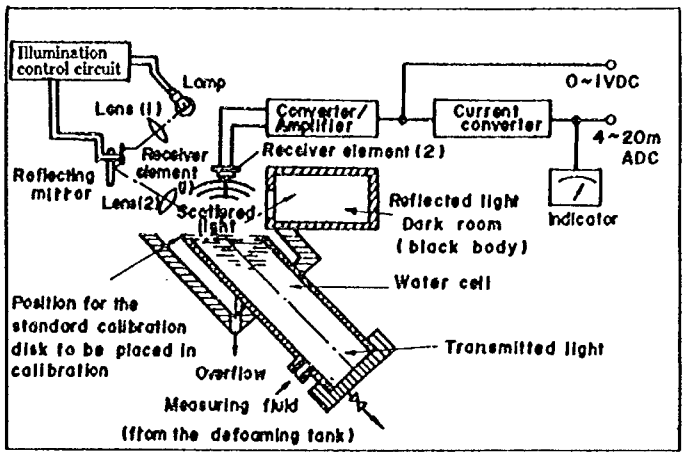


Fig. 7 Acting Principle and Structure of MLSS Meter

(1) SCATTERED LIGHT METHOD



(2) TRANSMITTED LIGHT TO SCATTERED LIGHT RATIO COMPUTING METHOD

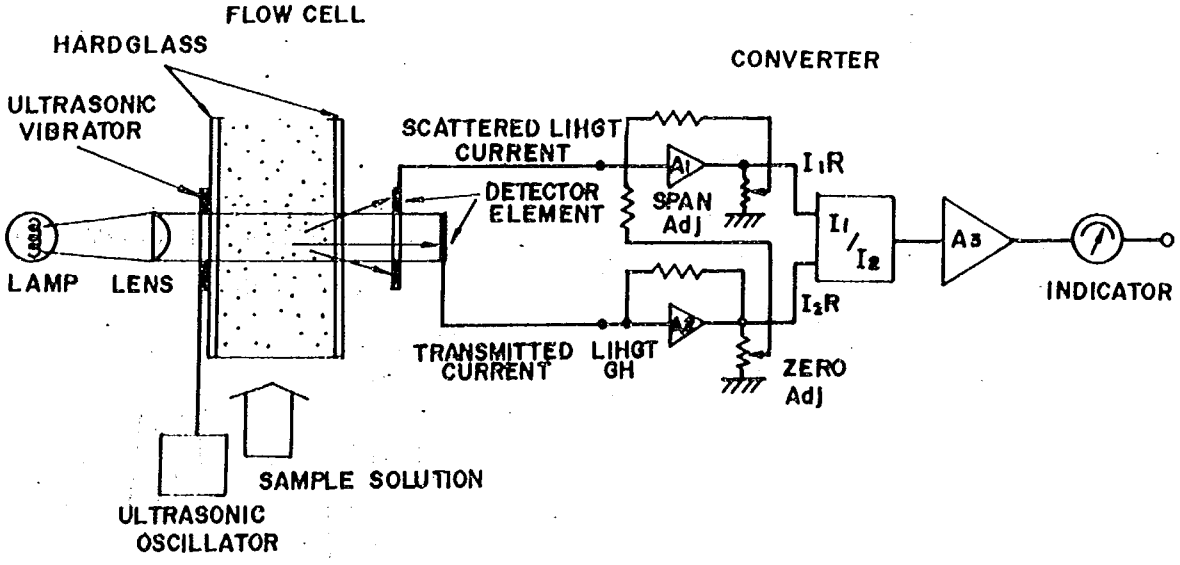


Fig. 8 Principle and Structure of Turbidity Meter

d) Sludge Concentration Meters (For Return Sludge)
There are two measurement principles concerning sludge concertrations: measurement of scattering light as with turbidity meters and MLSS meters, and the principle based on ultrasonic damping. The former is based upon the same principle as described in b) and c). In the latter, bubbles in a sample are pressureized to remove them, and a definite level of continuous ultransonic is transmitted to the sample at regular intervals and the meter receives the ultransonic from the reflector. The sludge concentration is obtained by the damped amount. the structure is given in Fig. 9. The object meters in the plant are of the ultrasonic damping system type.

e) UV Spectrophotometers (UV meters)
The equipment was installed for the supervision of effluent qualities because of the good relationship between ultraviolet absorbancies and organic contents (COD) of the effluent from the plant. When beams from the low-voltage mercury lamp are directed to sample water passing through the cell, the meter will receive ultraviolet light ($\lambda = 254$ nm) and visible light. The ultraviolet light is damped by organic-derived absorption and supended solids-derived scattering, and the visible light is damped by the scattering. The above are calculation-treated to obtain the ultraviolet absorvance, which is substituted into organic amounts (COD) -ultraviolet absorbancies conversion table to obtain the organic content. The structure is given in Fig. 10.

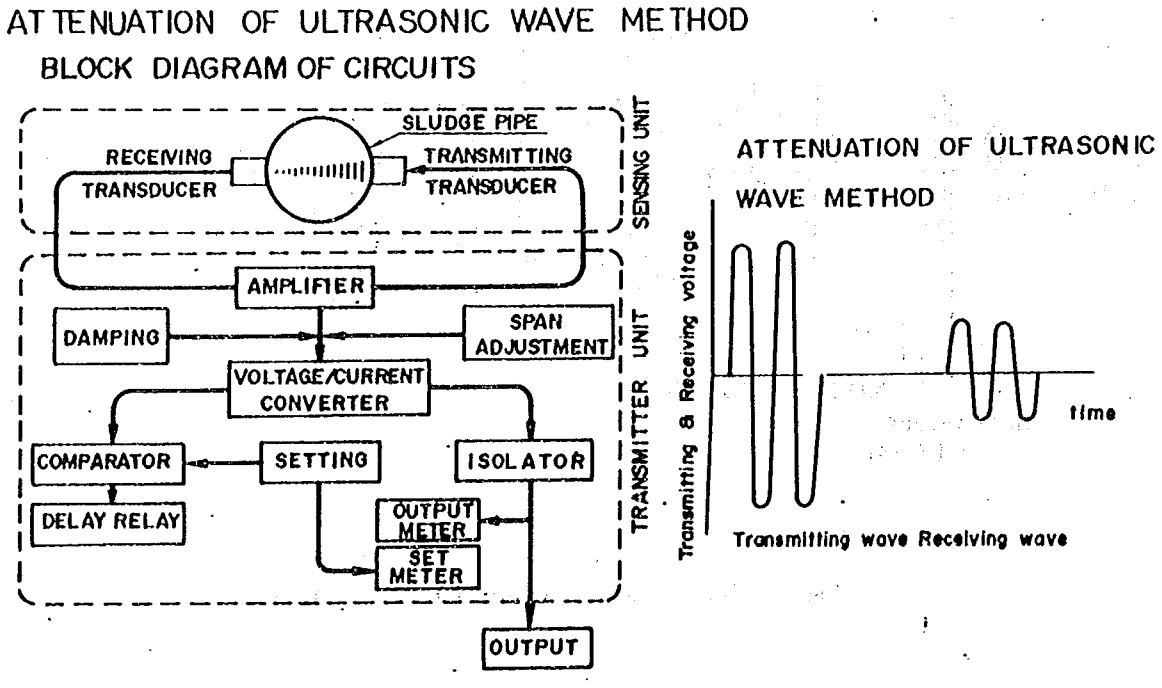


Fig. 9 Sludge Concentration Meter

6.1.2 Locations and Conditions of the Equipment

The Table 3 gives an outline of five sewage treatment plants in terms of an investigation of water quality meters. All of the plants employ the conventional activated sludge process.

a) DO Meters

DO meters were installed in aeration tank sites in all the plants. Fig. 11 shows the locations. They were immersed at a water depth of 0.5 m on the opposite side of the diffuser so as to prevent the effect of bubbles.

b) MLSS Meters

They were also installed in the aeration tank sites.

c) Turbidity Meters

Because turbidity meters could not be immersed into the water directly, water was pumped up at every sampling point, then fed into a debubbling tank to remove bubbles and finally introduced into the measuring tank of the turbidity meter. Fig. 12 shows the turbidity meter flow sheet. They were installed at the primary sedimentation tank inlets and outlets, and secondary sedimentation tank outlets.

d) Sludge Concentration Meters

Sludge concentration meters were fitted to bypass pipes connected to return sludge pipes. The structure is shown in Fig. 13.

e) UV Spectrophotometers

There are two types of U.V. spectrophotometers: the immersion type as with DO meters and MLSS meters, and the other type measured by a method similar to that described under c). They were installed in channels between the secondary sedimentation tanks outlets and the chlorination tank.

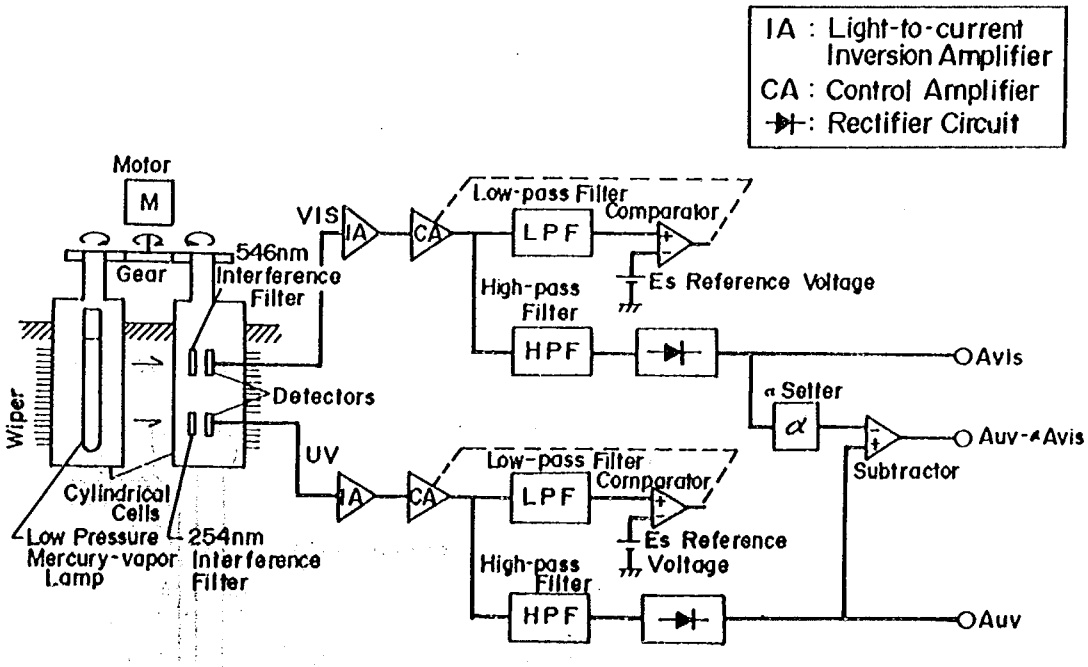


Fig. 10 UV (Ultraviolet) Absorptiometer Flow Sheet

Table 3 Sewage Treatment Plant in Yokohama

(Mar. '82 at present)

Plant	Location	Site area	Treatment capacity					Treatment process	Receiving water	Operation started
			Designed			Existing				
			Area	Population	Dry weather flow	Population	Dry weather flow			
Chubu	Nishiki, Naka-ku	m2 46,423	ha 880	185,000	m³/day 96,300	90,000	m³/day 47,000	Activated sludge process	Tokyo bay	Jul. '62
Nambu	Shin-isogo, Isogo-ku	70,615	2,245	450,000	234,000	450,000	234,000	"	"	Jul. '65
Hokubu No.1	Motomiya, Tsurumi-ku	62,893	2,150	377,000	196,000	377,000	196,000	"	Tsurumi River	Jul. '68
Totsuka No.2	Naganuma, Totsuka-ku	88,446	4,232	396,000	206,000	108,000	56,000	"	Kashio River	Oct. '72
Kohoku	Hutoo, Kohoku-ku	125,360	6,270	823,000	428,000	130,000	67,800	"	Tsurumi River	Dec. '72
Midori	Saedo, Midori-ku	87,000	8,096	852,000	443,000	142,000	73,800	"	"	May, '77
Kanagawa	Chiwaka, Kanagawa-ku	103,000	4,603	994,000	517,000	331,000	172,300	"	Tokyo bay	Mar. '78
Kanazawa	Sachiura, Kanazawa-ku	236,000	4,762	646,000	336,000	108,000	56,000	"	Tomioka River	Oct. '79
Totsuka No.1	Kosugaya, Totsuka-ku	31,255	2,004	238,000	124,000	—	—	"	Itachi River	Under construction
Seibu	Higashimatano, Tosuka-ku	83,470	3,813	367,000	191,000	—	—	"	Sakai River	"
Hokubu No.2	Suehiro, Tsurumi-ku	369,847	637	166,000	86,400	—	—	"	Tokyo bay	"

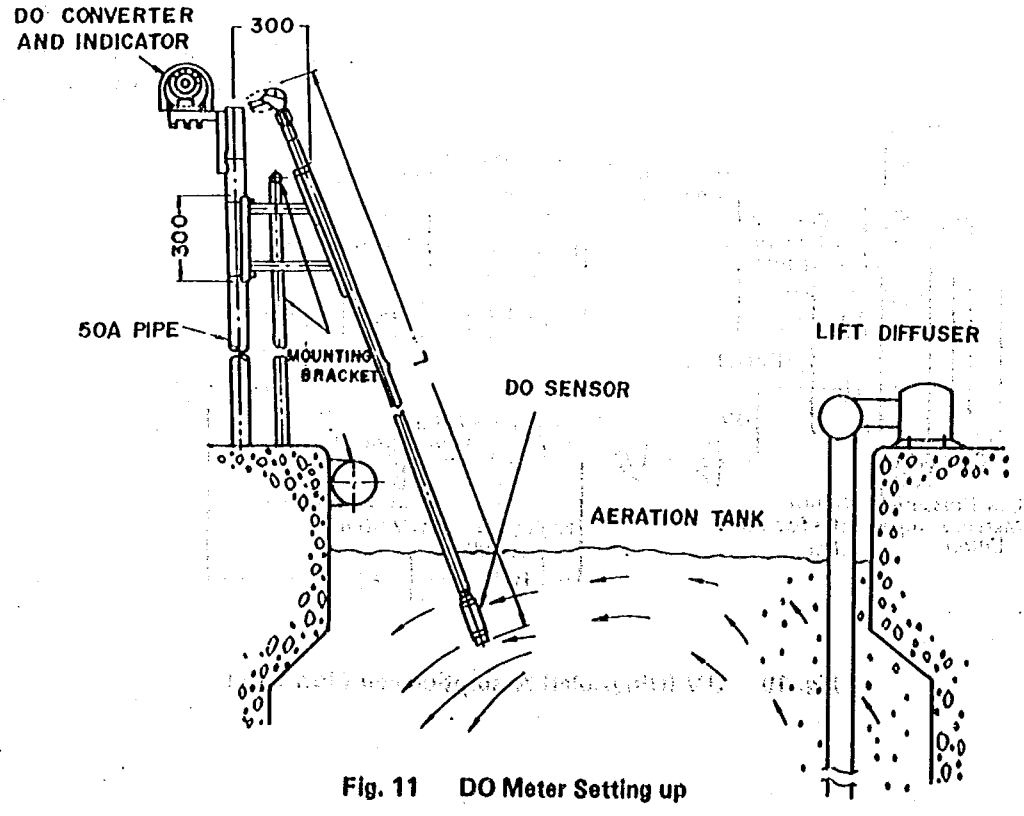
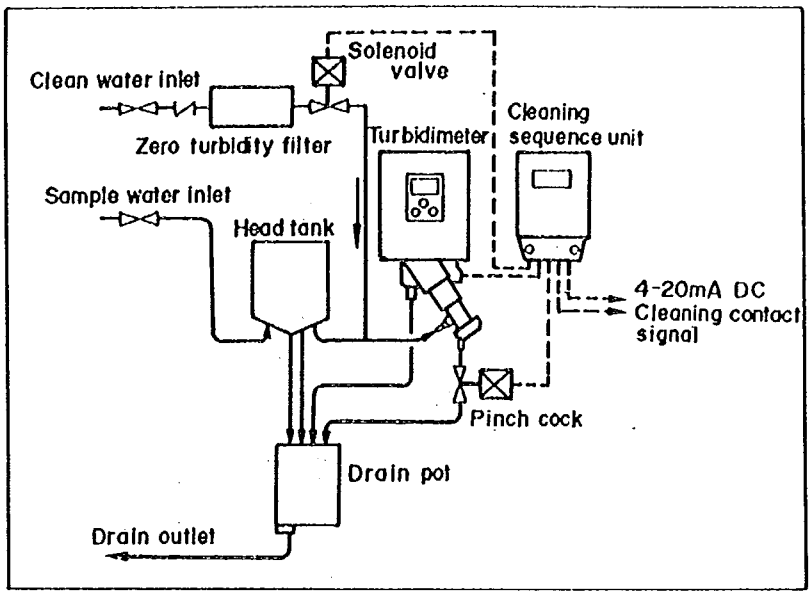


Fig. 11 DO Meter Setting up

(1)



(2)

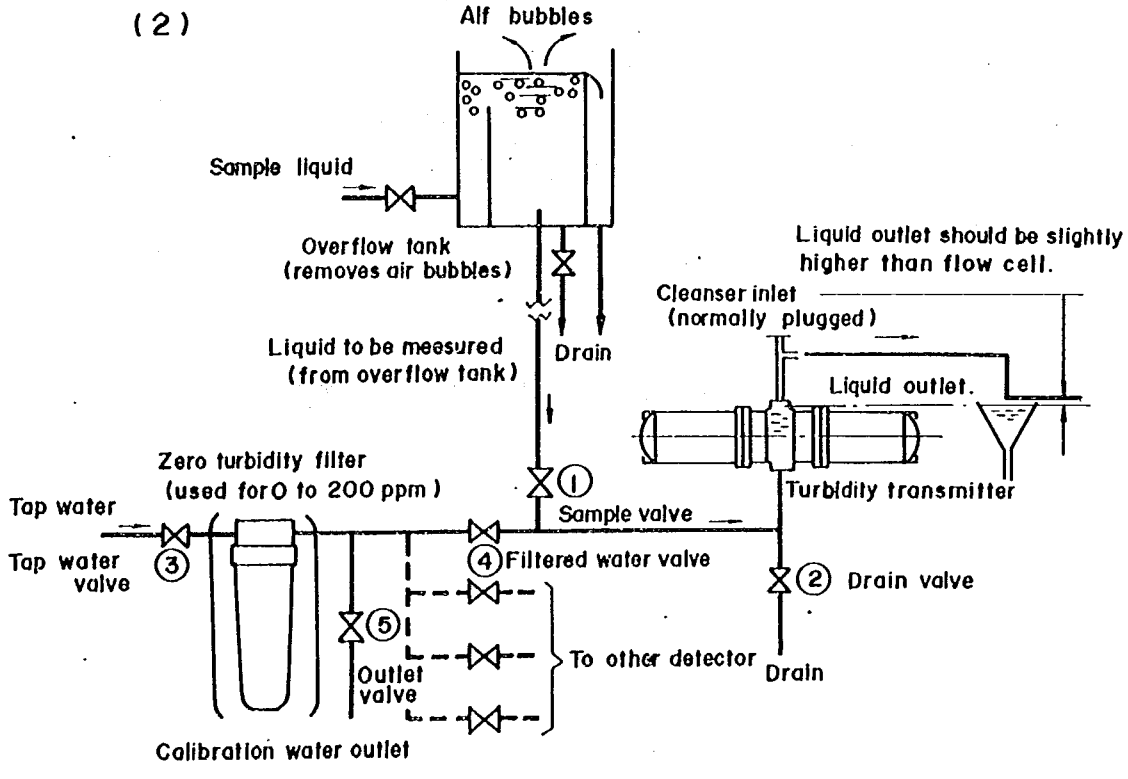


Fig. 12 Turbidity Meter Flow Sheet

BLOCK DIAGRAM OF CIRCUITS

In the description of the detector unit, the broken-lines indicate necessary preparation by the customer.

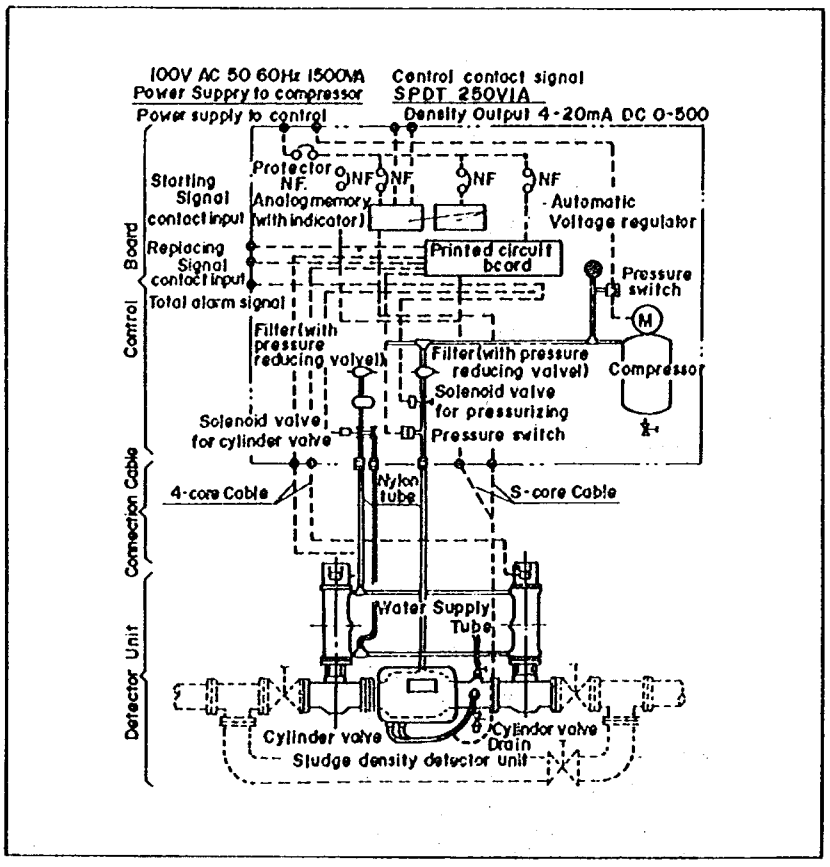


Fig. 13 Sludge Concentration Meter Block Diagram

6.2 Investigation of the Equipment

6.2.1 DO Meters

An investigation was carried out concerning 20 DO meters manufactured by 3 companies in 5 sewage treatment plants. Prior to the investigation, a control target value, namely within 5% of deviation, was determined for the consistent maintenance of the plants. The meters were cleaned if the observed value was exceeded three times successively; the zero and span point were calibrated if the values are not attained even by cleaning the meters; diaphragm replacement and electrode polishing were carried out if even calibration was impossible. The deviations stated here refer to (meter-indicated values—values obtained by manual analysis)/meter full-scale values, and the values obtained by manual analysis refer to measurements by the modified Winkler-sodium azide method.

The investigated items were as follows:

Relationships between meter-indicated values and the values obtained by manual Analysis.

Precision and stability of the meters.

Meter reliability and maintainability for each type.

Meter degradation patterns

6.2.2 MLSS Meters

Four MLSS meters manufactured by two compaines were investigated in three sewage treatment plants. Although the control target value was determined initially, the calibration was too troublesome that cleaning and calibration were carried out at one-month intervals and once to four times a year respectively.

The investigated items were as follows:

The relationships between the meter-indicated values and the values obtained by manual analysis.

Meter stability and precision for every sewage treatment plant.

6.2.3 Turbidity Meters

Investigation was made for nine turbidity meters manufactured by two companies in four sewage treatment plants. The meters were cleaned and calibrated at one-month intervals for maintenance. The relationships between the manual analytical values for SS, COD, and BOD and the meter-indicating values were investigated.

6.2.4 Return Sludge Concentration Meters

An investigation was carried out relating to two return sludge concentration meters in our plants. Cleaning and calibration were carried out, if necessary.

The investigated items were as follows:

The relationships between the meter-indicating values and the manual analytical values.

The stability and precision of the meters.

6.2.5 UV Spectrophotometers

An investigation was made for three kinds, a total of six UV spectrophotometers in four sewage treatment plants. Maintenance was carried out by cleaning and calibrating the meters initially at one-week intervals.

The investigated items are as follows:

Performance of the meters.

The relationship between the meter-indicating values and the manual analytical COD values.

Evaluation of the COD convertion equation.

6.3 Results and Discussion

6.3.1 DO Meters

Table 4 shows the relationship between the meter-indicating values and the analytical values per each item of equipment. Except for the four meters (No. 5, 11, 12 and 17) for which maintenance was not made thoroughly the correlation coefficients were more than 0.88 with high statistical significance. The standard deviation ($\sqrt{Vyx}$) of the deviation was less than 0.6 mg/ℓ showing a high relationship between the meter-indicating values and the manual analytical values. During the investigation period, three (No. 1, 15 and 18) could not be operated due to amplifier failure, and the investigation of the three had to be discontinued.

The precision and stability of the meters was checked on the basis of the $\bar{x}$-R control chart. A typical example (No. 2) is shown in Fig. 14. Referring to the chart, both $\bar{x}$ (deviation average) and R (range of deviation) were within the control limits for more 95% of the total and both the precision and stability were found to be good.

Reliability and maintainability for each type were obtained for 13 items of equipment which were installed at the same time and for which careful maintenance was carried out. The results are shown in Table 5.

Table 4 Relationships Between DO Manual Analytical Values and DO Meter-indicating Values

A Company	No. 1	No. 2	No. 3	No. 4
	N sewage treatment plant	K sewage treatment plant	K sewage treatment plant	K sewage treatment plant
Sample size	143	186	140	140
Correlation coefficient	0.953	0.987	0.987	0.984
Regression equation y	0.94x+0.57	0.94x+0.59	0.93x+0.40	0.97x+0.48
Standard deviation ($\sqrt{Vy.x}$)	0.53	0.28	0.33	0.28

B Company	No. 5	No. 6	No. 7	No. 8	No. 9	No. 10
	N sewage treatment plant	K sewage treatment plant	K sewage treatment plant	K sewage treatment plant	M sewage treatment plant	M sewage treatment plant
Sample size	124	207	136	186	184	211
Correlation coefficient	0.852	0.877	0.926	0.940	0.959	0.941
Regression equation y	0.87x+0.04	0.83x+1.22	0.98x+0.63	0.85x+1.03	1.13x−0.54	1.18x−0.56
Standard deviation ($\sqrt{Vy.x}$)	0.68	0.75	0.58	0.59	0.42	0.62

C Company	No. 11	No. 12	No. 13	No. 14	No. 15	No. 16
	N sewage treatment plant	N sewage treatment plant	M sewage treatment plant	M sewage treatment plant	T sewage treatment plant	T sewage treatment plant
Sample size	171	173	212	205	104	218
Correlation coefficient	0.769	0.895	0.923	0.949	0.959	0.909
Regression equation y	1.02x−0.29	1.00x−0.06	0.99x−0.30	1.07x+0.23	1.04x−0.42	0.75x+1.03
Standard deviation ($\sqrt{Vy.x}$)	1.30	0.77	0.52	0.52	0.52	0.63

C Company	No. 17	No. 18	No. 19	No. 20
	T sewage treatment plant	H sewage treatment plant	H sewage treatment plant	H sewage treatment plant
Sample size	219	171	176	215
Correlation coefficient	0.863	0.882	0.947	0.931
Regression equation y	0.78x+1.10	0.95x−0.22	1.18x−0.66	0.98x−0.32
Standard deviation ($\sqrt{Vy.x}$)	0.90	0.61	0.57	0.59

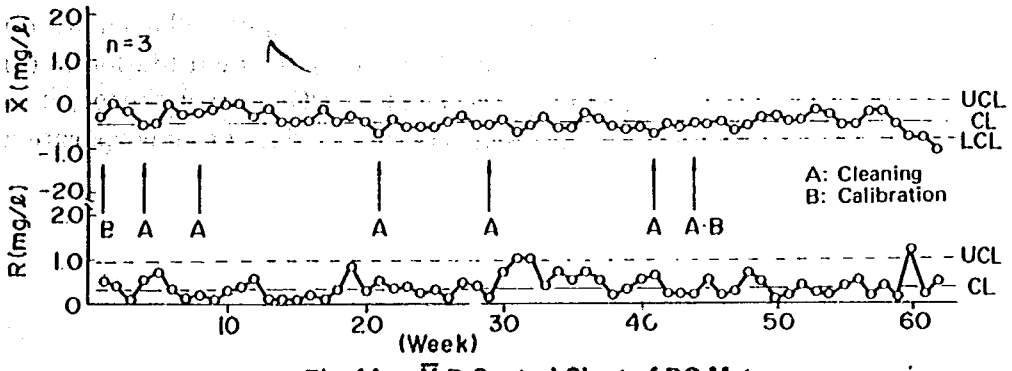


Fig. 14 X̄-R Control Chart of DO Meter

Table 5 Reliability and Maintainability of DO Meter for Each Type

	Availability		Maintenance time		Mean time between failure (Day)	
	Working time / working time+non-working time	Day/yr	(Man·hr/yr)	Maintenance time/30D.action	MTBF	90% confidence
A	0.92	337	5.3	0.5	92	22
B	0.77	280	11.2	1.2	48	12
C	0.66	241	14.0	1.7	45	10

Table 6 MLSS Meters Relationships Between Manual Analytical Values and Meter-Indicating Values

	D Company		E Company	
	N sewage treatment plant	K sewage treatment plant	K sewage treatment plant	M sewage treatment plant
Sample size	176	147	77	212
Correlation coefficient	0.657	0.871	0.908	0.964
Regression equation y	0.61x+580	0.97x+30	0.72x+580	1.12x−120
Standard deviation ($\sqrt{Vy.x}$)	280	190	120	140

Reliability for each meter was obtained by plotting the meter trouble times on the Weibull probability paper to calculate the mean time between failure (MTBF) and the 90% confidence limit. The meter trouble time was defined as the time when the deviation [(meter-indicating value — manual analytical value)/full scale value] exceeded the control target value (±5%) three times successively from the point of the meter calibration or time (days) until the point of cleaning and calibration due to the possibility of the value exceeding the control target value. The maintainability was obtained in terms of the meter availability (working time/(working time + non-working time)), average maintenance time (man·hour/year) and average maintenance time for a month(man·hour). The results in Table 5 show DO meters utilizing the same principle and which have different performance.

Fig. 15 gives DO meter degradation patterns, in which (1) shows rapidly decreased output followed by no response, (2) gradually decreased output followed by no response, and (3) gradually decreased response. Because meter degradation in (1) and (2) may cause "running away", if they are used as DO control sensor, it is necessary to understand the meter characteristics beforehand and to incorporate a fail-safe device.

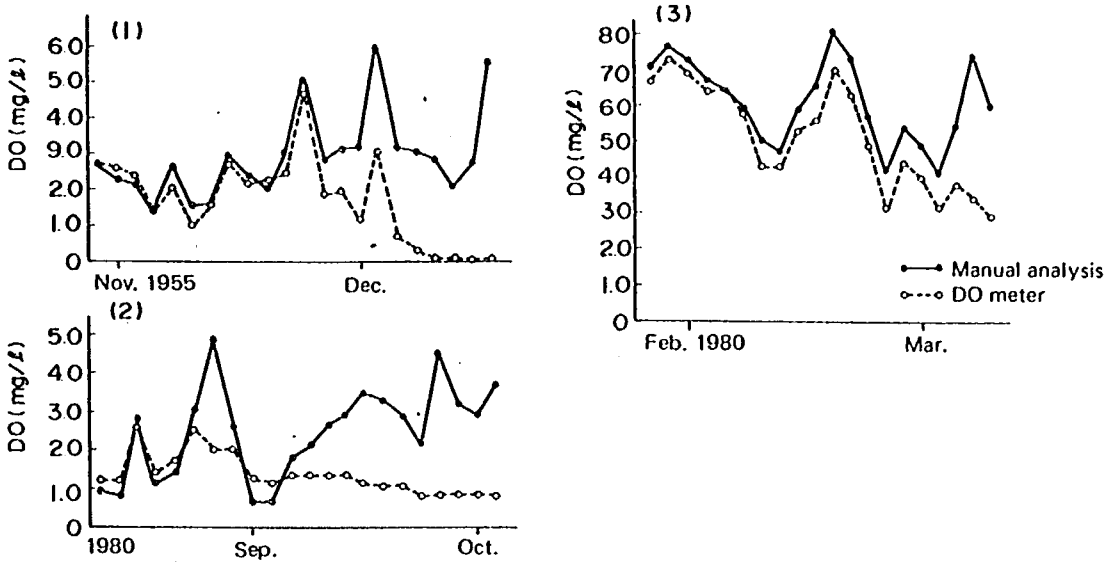


Fig. 15 Degradation Pattern of DO Meter

6.3.2 MLSS Meters

Table 6 shows the relationship between meter-indicating values and manual analytical values. The correlation coefficients were 0.66 to 0.96, and the standard deviations ($\sqrt{Vyx}$) of deviations were 120 to 280 mg/ℓ with differences between the sewage treatment plants installed with meters observed.

The meter precision and stability for each plant were investigated referring to the $\bar{x}$-R control chart of deviations and were given in Fig. 16. Plant (1) is a separate sewer system having no sludge treatment process. Plant (2) is a combined sewer system having non-digestion sludge dewatering process. Plant (3) is a combined sewer system having anaerobic sludge digestion process. As a result, the deviation variations became large in the order of (1) (2) (3) and frequencies exceeding the control limits were also in the same order. The reason may probably be the problems of the measuring principles of the meters. The scattering light measuring method is utilized only the premise that the properties of the activated sludge to be measured such as the sizes and densities of the suspended solids are constant.

The relationships between storm water inflows and deviations likely responsible for the properties of the sludge were analyzed for the sewage treatment plants of (2) and (3).

The difference between the meter-indicating value (M_n) and the manual analytical value (A_n) on the n-th day is replaced by X_n. The difference between the meter-indicating value (M_{n-1}) and the manual analytical value (A_{n-1}) on the (n-1)th day is replaced by X_{n-1}.

A daily increment is replaced by $\Delta X = X_n - X_{n-1}$.

The volume of the inflow into the aeration tank on the n-th day is replaced by $F°_n$.

The volume of the inflow into the aeration tank on the (n-1)th day is replaced by $F°_{n-1}$.

The storm water inflow parameter on the n-th day is replaced by
$Fn = 1/2F°n + F°n\text{-}1$.
The storm water inflow parameter on the (n-1)th day is replaced by
$Fn\text{-}1 = 1/2F°n\text{-}1 + F°n\text{-}2$.
The daily increment is replaced by $\triangle F = Fn - Fn\text{-}1$

In Fig. 17, $\triangle F$ and $\triangle X$ were plotted against the coordinate and the abscissa respectively. The relationship based on the data was positive ($r = 0.48$, $n = 66$). If there is an effect by strom water at a given point, the parameter ($\triangle F$) will shift in the plus or minus direction from the origin accompanied by a shift of the meter error parameter ($\triangle X$) in the plus or minus direction.

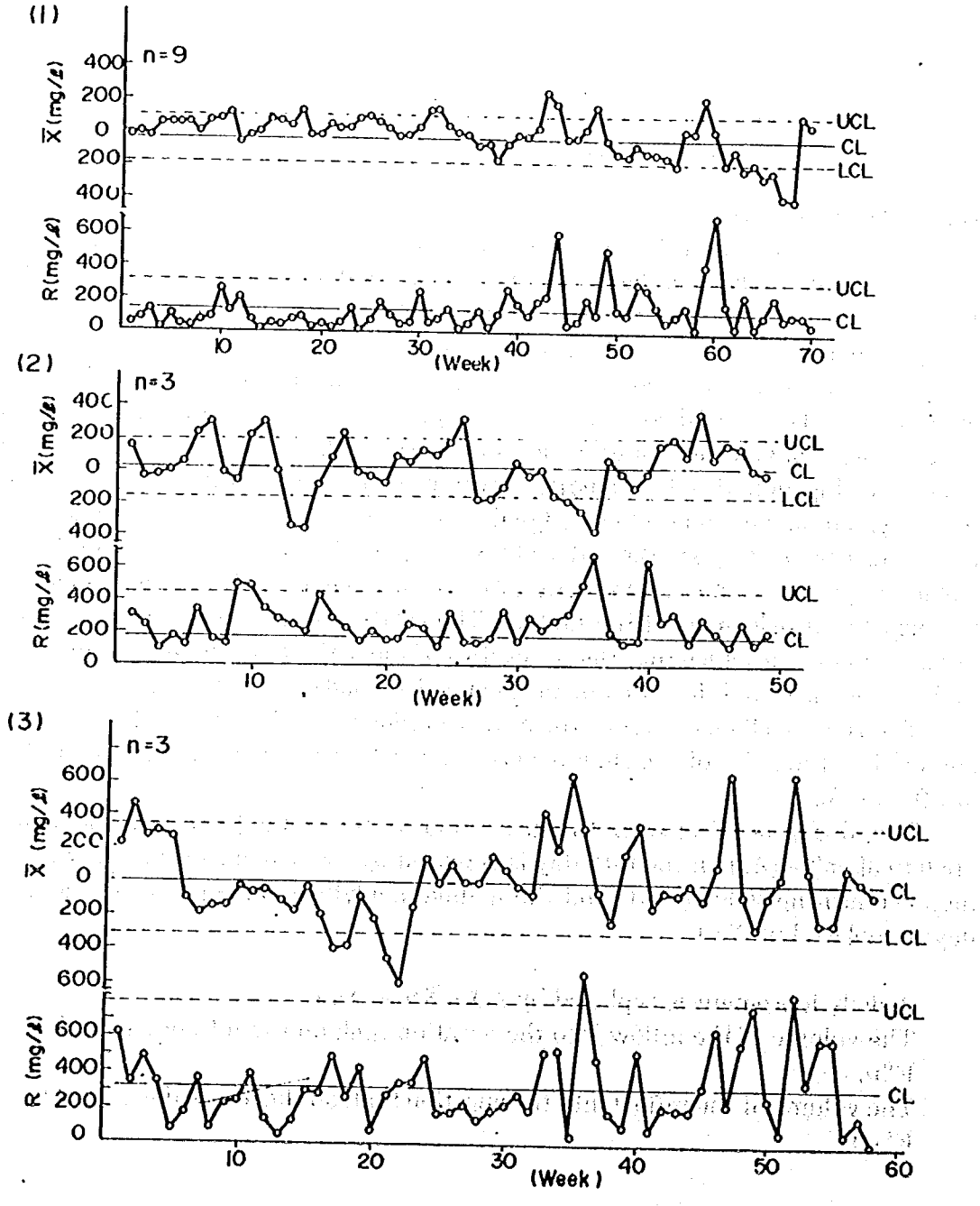


Fig. 16 $\bar{X}$-R Control Chart of MLSS Meter

From the above, storm water inflow was found to be responsible for the variations of deviations. Because the presence or absence of return water from the sludge treatment process and operation change in the sewage treatment process may affect the properties of the activated sludge, they probably amplify the variations of deviations. When storm water inflow, return water, and operation change in the treatment process are taken into consideration, MLSS meters can be used for supervision of the abnormal process operation and short period monitors but are difficult to use as control sensors.

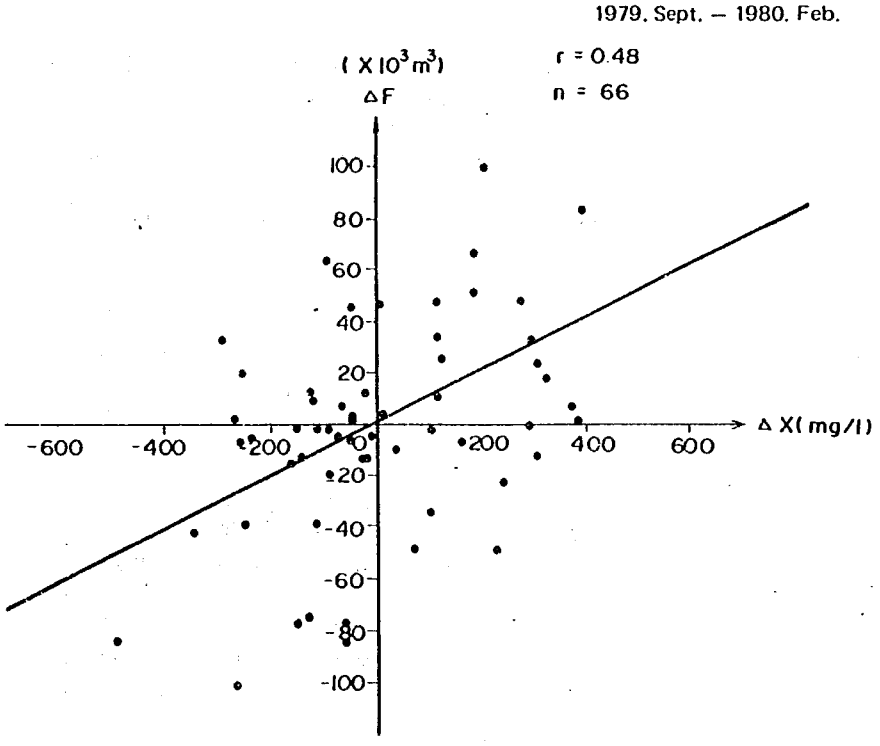


Fig. 17 Relationship Between Storm Water Inflow and Meter Error

6.3.3. Turbidity Meters

Table 7 shows the relationships between meter-indicating values and manual analytical values (SS, COD, and BOD). The correlation coefficients greatly vary depending on the treatment plants and meter locations: SS, 0.50-0.84; COD, 0.05-0.91; BOD, 0.06-0.88. The cause of scatters of the correlation coefficient with SS may be the measuring principle of turbidity meters similar to that of MLSS meters. The relationship will be very strong when the shape and distribution of the suspended solid particles are constant; while the relationship will be poor when the suspended solid particle patterns change due to storm water inflow or return water from the sludge process. Estimation of COD and BOD from turbidity meters-indicating values was found to be difficult because of fluctuations in the water qualities of the influent into the sewage treatment plant. However turbidity meters are useful for monitoring abnormal process operations because they sufficiently respond to wide range variations in water qualities resulting from return water.

6.3.4 Return Sludge Concentration Meters

Table 8 shows the relationships between meter-indicating values and the manual analytical values. The correlation coefficient was more than 0.88 and highly significant. The standard deviations ($\sqrt{Vy \cdot x}$) of deviations was less than 1,000 mg/ℓ showing a high relationship. The precision and stability of meters were checked using the $\bar{x}$-R control chart shown in Fig. 18. More than 90% of both of the $\bar{x}$ (deviation means) and R (deviation ranges) were within the control limits showing good precision and stability.

Table 7 Turbidity Meter Relationship Between Manual Analytical Values (SS, BOD, and COD) and Meter-indicating Values

	N plant (Primary Treated Water) F Company			N plant (Secondary Treated Water) F Company		
	SS	COD	BOD	SS	COD	BOD
Sample size	35	35	35	36	36	36
Correlation Coefficient	0.655	0.241	0.262	0.749	0.571	0.181
Regression Equation y	1.35x+17	0.27x+33	0.63x+55	0.94x+2.6	0.91x+8.3	0.50x+9.9

	M plant (Primary Treated Water) F Company			M plant (Secondary Treated Water) G Company		
	SS	COD	BOD	SS	COD	BOD
Sample size	41	41	41	41	41	41
Correlation Coefficient	0.679	0.900	0.886	0.685	0.910	0.882
Regression Equation y	0.68x−2.6	0.58x+6.7	1.90x−15.4	0.61x+5.3	0.52x+13.4	1.69x+8.2

	K plant (Primary Sedimentation tank Influenty) F Company			K plant (Primary Treated Water) F Company			K plant (Secondary Treated Water) F Company		
	SS	COD	BOD	SS	COD	BOD	SS	COD	BOD
Sample size	33	33	33	46	46	46	41	41	41
Correlation Coefficient	0.840	0.814	0.885	0.714	0.759	0.686	0.589	0.555	0.499
Regression Equation y	1.93x−4.8	0.92x+3.6	2.14x−0.8	1.12x+1.3	0.36x+15.8	1.05x+21.5	0.69x+1.8	1.15x+4.7	1.77x+1.4

	Ka plant (Primary Treated Water) F Company			Ka plant (Secondary Treated Water) F Company		
	SS	COD	BOD	SS	COD	BOD
Sample size	39	39	39	33	33	33
Correlation Coefficient	0.500	0.436	0.392	0.730	0.050	0.063
Regression Equation y	0.60x+29.8	0.39x+47.6	0.53x+65.2	0.99x+0.9	0.16x+12.0	0.10x+3.0

Table 8 Sludge Concentration Meters Relationships Between Manual Analytical Values and Meter-indicating Values

	K sewage treatment plant	K sewage treatment plant
Sample size	236	233
Correlation coefficient	0.948	0.885
Regression equation y	1.07x−700	0.85x+810
Standard deviation $\sqrt{Vy.x}$	660	1000

Table 9 UV Spectrophotometer Relationship Between COD Manual Analytical Values and Meter-indicating Values

	A sewage treatment plant	B sewage treatment plant	C sewage treatment plant	D sewage treatment plant
Period	Aug.—Oct.	Feb.—Nov.	Jun.—Oct.	Aug.—Nov.
Correlation coefficient	0.923**	0.790**	0.790**	0.826**
Sample size	103	117	69	61
Regression equation y	98.2A−0.43	45.8A+2.28	73.9A−0.94	65.6A+0.35
	Monthly data in A plant (Aug.—Oct.)			
Period	Aug.	Sept.	Oct.	
Correlation coefficient	0.964	0.810	0.931	
Sample size	19	39	45	
Regression equation y	101.1A−1.32	90.1A+0.58	105.5A−1.01	

** : significant at 1%
A : UV absorptivity converted to 10mm cell

Table 10 UV Spectrophotometer Relationship Between COD Manual Analytical Values and Meter-indicating Values by Equipment Types

	UV-1	UV-2		UV-3	
Visible correction	done	none	done	none	done
Correlation coefficient	0.931**	0.884**	0.856**	0.925**	0.732**
Sample size	45	45	43	44	43
Regression equation y	105.5A−1.01	102.0A−1.63	107.5A−1.70	92.2A−0.92	114.5A−0.52

** : significant at 1%
A : UV absorptivity converted to 10mm cell

6.3.5 UV Spectrophotometers

Zero and span drifts and linearity with the standard solution were investigated for three types of meters to confirm their performance with the results shown in Fig. 19 (1) and (2). The zero and span drifts for a period of one month within ±3% and ±5% of each of the full scales. The reproducibility using a standard solution, determined three weeks later, of potassium biphthalate, was also satisfactory.

Table 9 shows the relationships between COD and meter-indicating values in each sewage treatment plant. The correlation coefficients were 0.79 to 0.92 and highly significant.

No significant difference was found for data and regression equations from August to October in A Sewage Treatment Plant, and therefore the same regression equation was proved to be usable during the above period.

Table 10 shows the relationships between the meter-indicating values and COD, both of which were measured simultaneously, using three kinds of the meters in A Sewage Treatment Plant. The correlation coefficients were 0.85 to 0.93, which were not significant in spite of scattering showing that there were no differences between the meter-derived values. This is probably due to the same measurement principle used.

It is evident that the UV spectrophotometer can be effectively used for effluent quality monitoring. The UV spectrophotometers are at present monitoring effluent qualities in seven sewage treatment plants in the city.

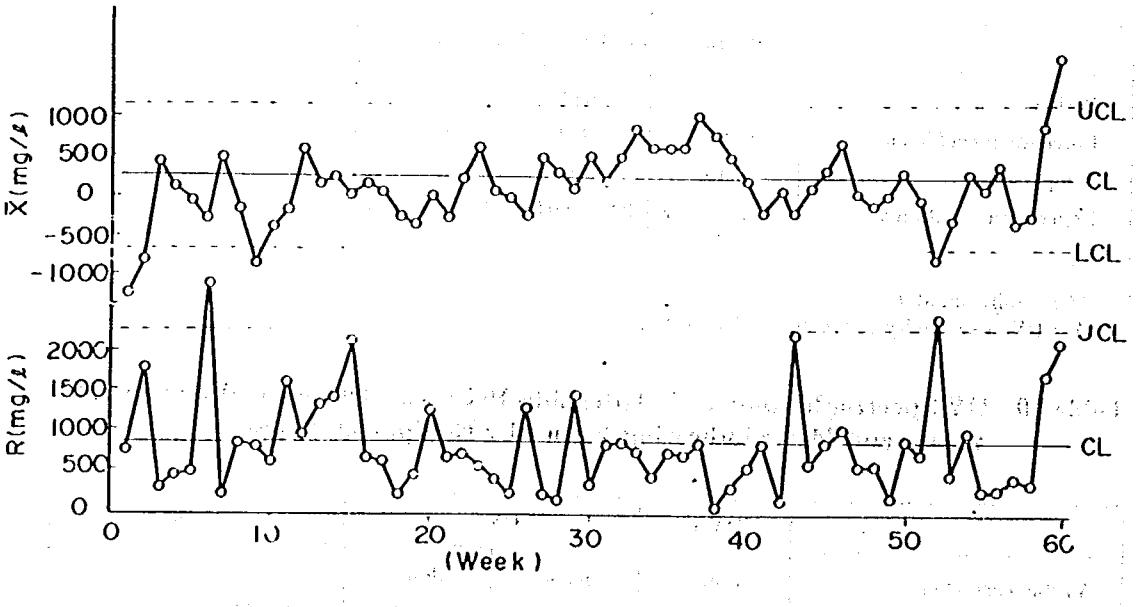


Fig. 18 X̄-R Control Chart of Return Sludge Concentration Meter

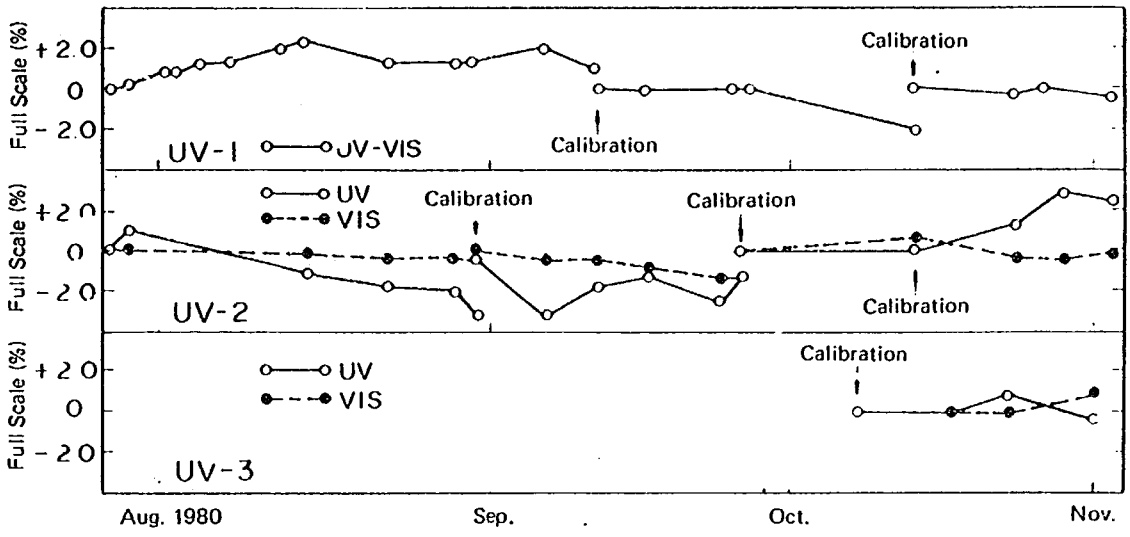


Fig. 19-(1) Zero Drift Stability of UV Spectrophotometer

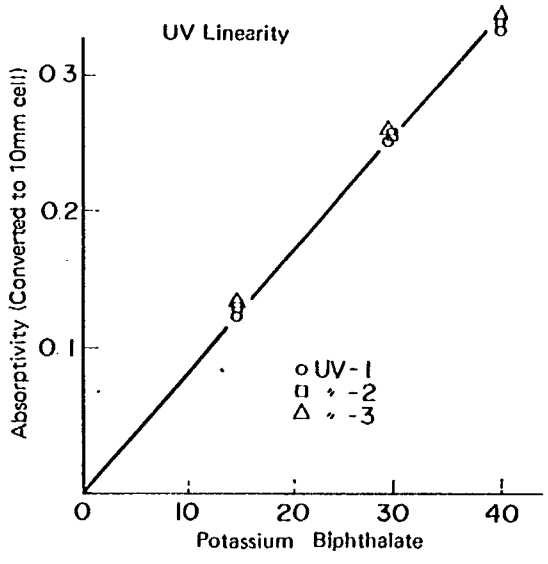


Fig. 19-(2) Calibration Curve Linearity of UV Spectrophotometer

7. Examples of Automatically Controlled Operation

7.1 DO Control

Conventional aeration tank operation has mainly relied upon constant air flow control; whereas air flow control in which DO concentrations in an aeration tank were kept constant has recently been studied from the standpoints of stability of sewege treatment and energy-saving.

The location of a DO sensor and control responsiveness which are the fundamental problems of constant DO control were also investigated in Kanagawa Sewage Treatment Plant. Dependent upon the results, automatically operated constant DO control was carried out to investigate the treatment characteristics due to differences in DO levels.

7.1.1 Optimal Location of DO Sensor

Because one sensor for constant DO control is as a rule to be used, the location of the sensor was determined by taking the following points into consideration.

a) The diffusers and the structure of the aeration tank.
b) The mixing characteristics of the tank (the mixing conditions of MLSS).
c) The characteristics of the volume and qualities of influent.
d) The DO profile in the tank.

a) The diffusers and the structure of the aeration tank

As shown in Fig. 20, the structure of the aeration tank in our plant is of a 5-tank serial type provided with division walls for preventing short circuits. The tank capacities are each 190 m^3 for the first and fifth tanks, and 380 m^3

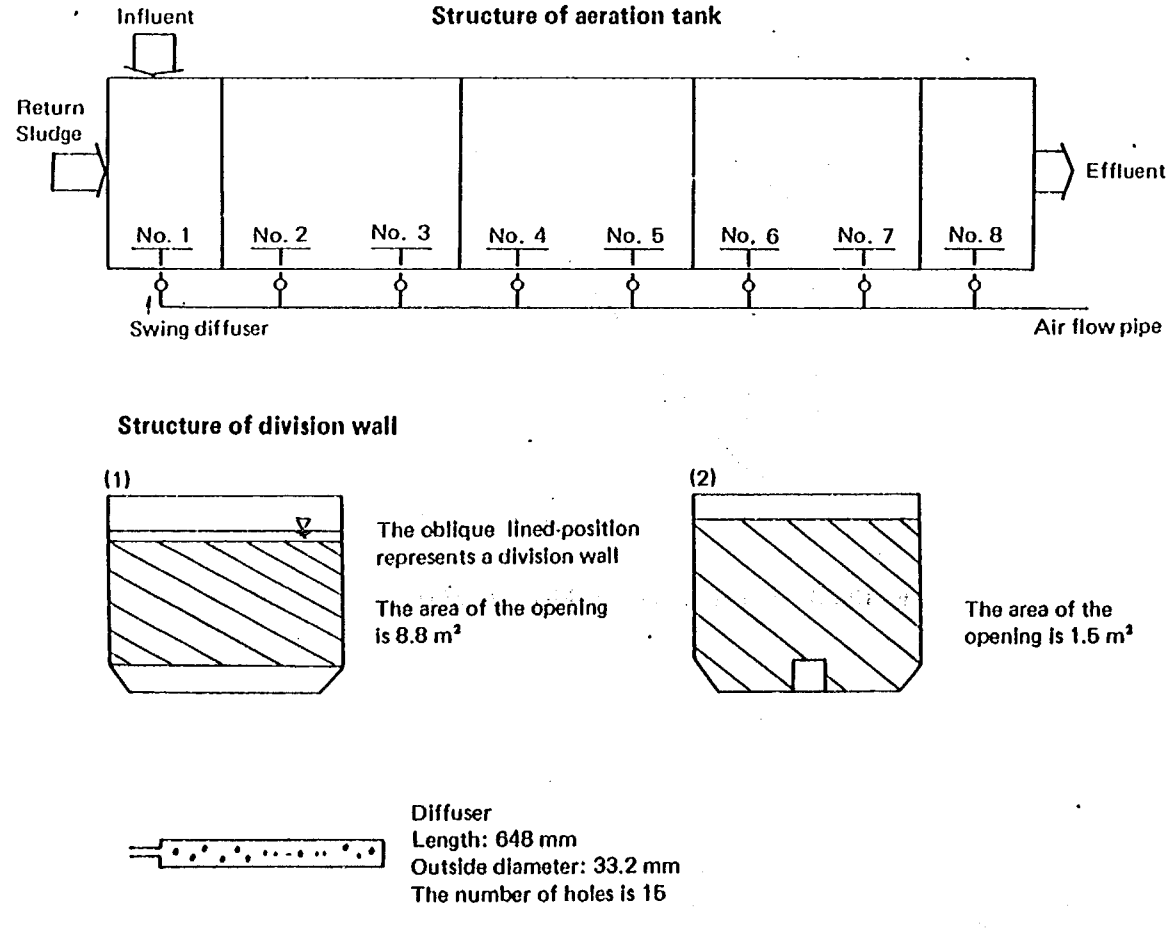


Fig. 20 Structure of Aeration Tank and Division Walls

for from the second to the fourth tanks, the total being 1,520 m^3.
(1) in Fig. 20 shows the structure of short circuit-preventing division walls. The top and bottom are provided with openings. Both line I and II are provided with respective 12 aeration tanks, the structures of all of which are the same as the above. The tanks are operated by the conventional activated sludge process with a designed retention time of six hours.
Aeration is performed by the swing diffuser and the diffusing pipes are stainless steel pipes with small holes. They are 648 mm in length, 33.2 mm in outside diameter and they have 15 holes with a diameter of 4 mm arranged alternately (the deviation is $\pm 15°$ from the midline). Each Swing diffuser is provided with 17 diffusing pipes, the total of which is 136 per aeration tank.

b) The mixing chracteristics of aeration tanks
Five aeration tanks are arranged in series and a spiral plug flow is designed in them. The mixing characteristics in the actual tank were compared with those of an aeration tank, as a reference tank, which was remodeled such that it was provided with division walls as shown in (2) in Fig. 20.
Both in the actual and reference tanks, the inflow volume and the amount and concentration of return sludge were so adjusted to maintain the MLSS constant in each tank, a definite volume of inflow was introduced, the return sludge was stopped for 2 hours, and the MLSS in each tank was analyzed at 30 minute intervals.
The results are shown in Figs. 21 (1) - (5). When MLSS time course variations in the first tank were observed, the MLSS decreased only by 400 mg/ℓ in the actual tank but more than 1,400 mg/ℓ in the reference tank. Differences in decreasing MLSS became smaller between the actual and reference tanks in the other tanks and the difference was as small as 100 mg/ℓ in the fifth tank. The results show that the actual tank has a larger proportion of short circuit flow than in the reference tank.

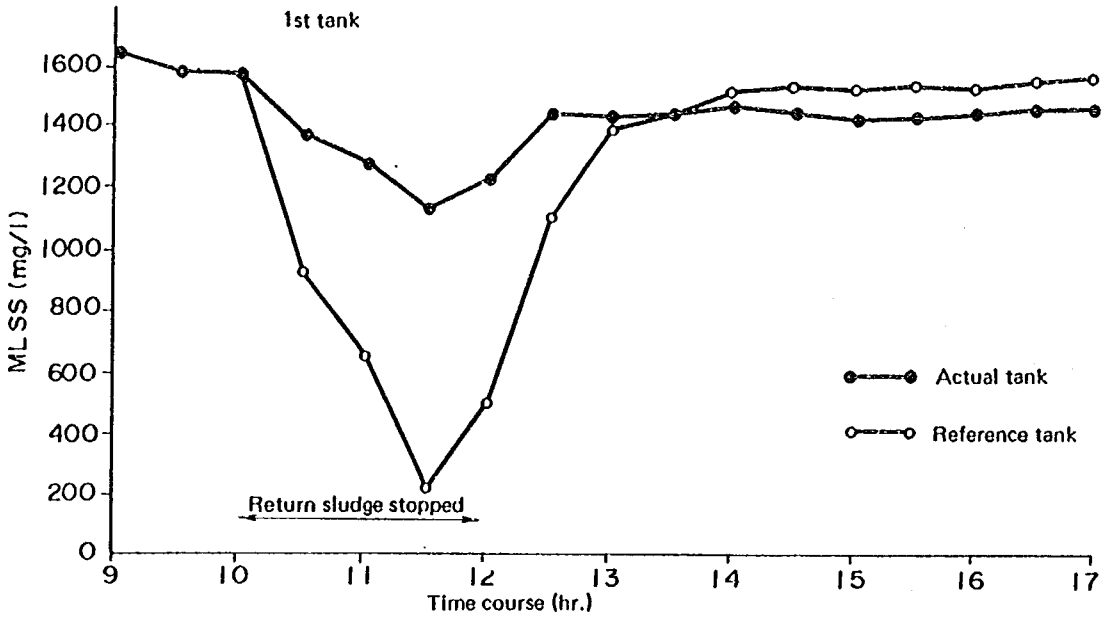


Fig. 21 (1) MLSS Time-course Variations (1st tank)

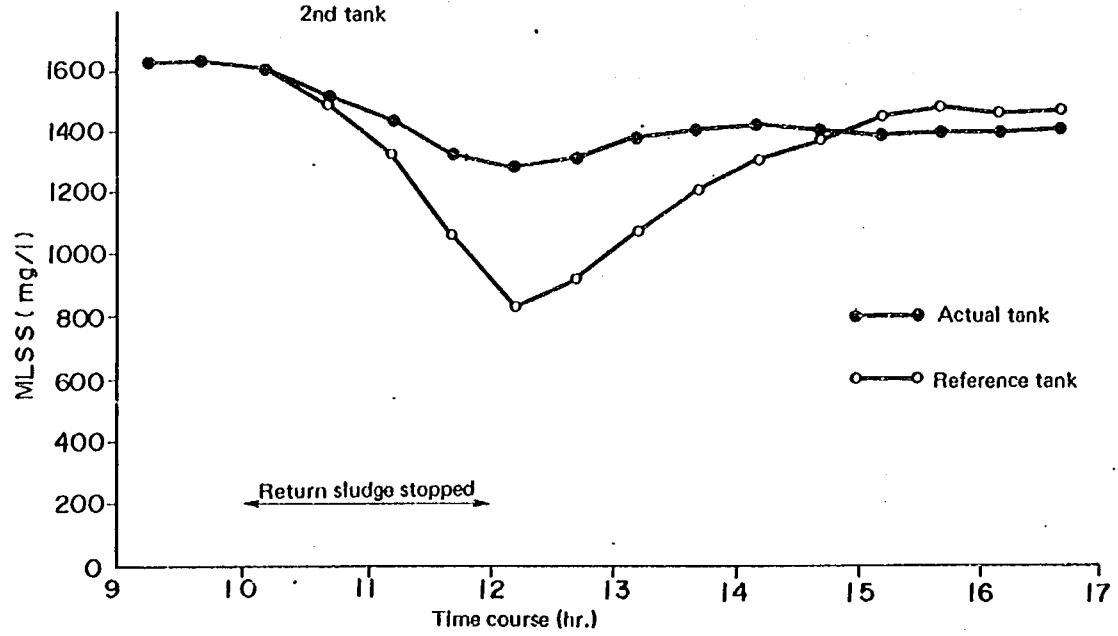


Fig. 21 (2) MLSS Time-course Variations (2nd tank)

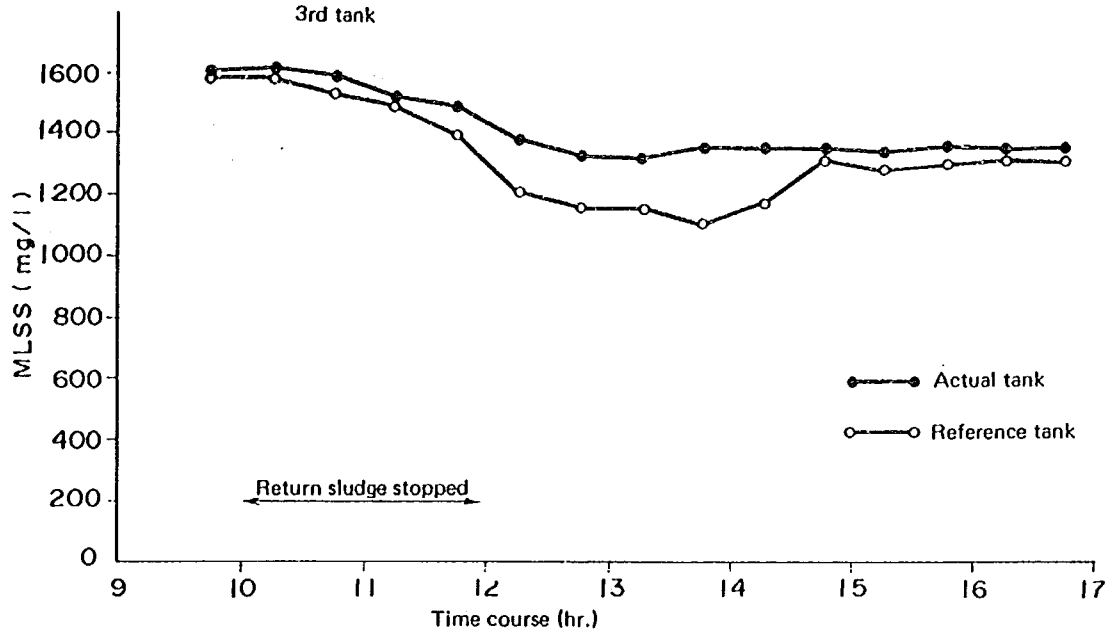


Fig. 21 (3) MLSS Time-course Variations (3rd tank)

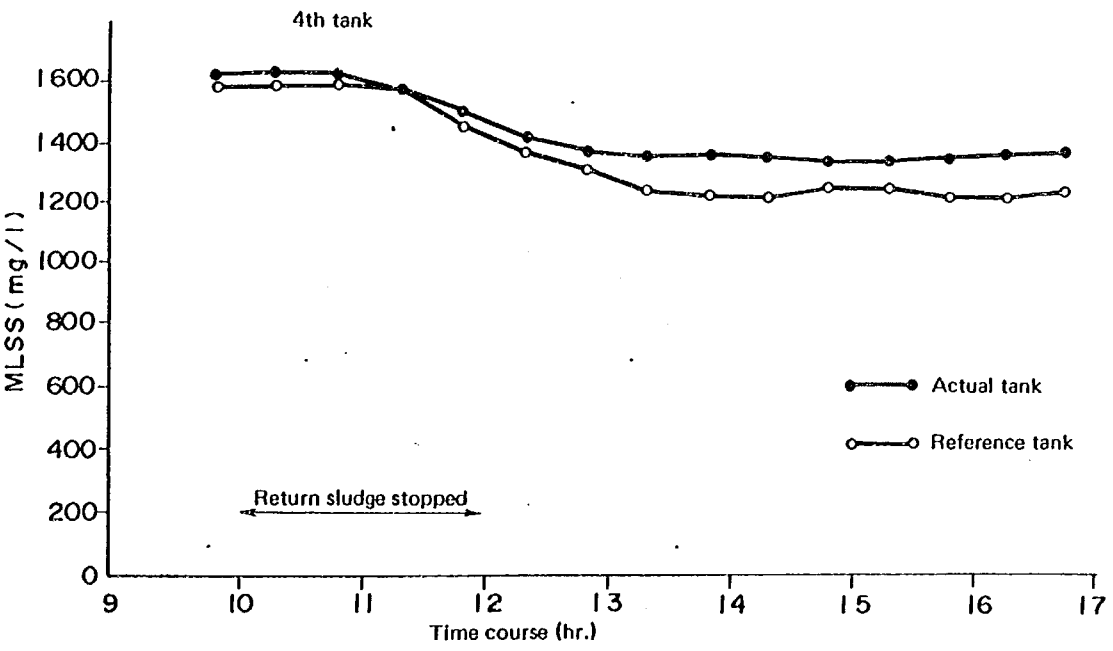


Fig. 21 (4) MLSS Time-course Variations (4th tank)

c) The characteristics of volumes and qualities of inflow
Kanagawa Sewage Treatment Plant began to operate in March, 1978. The inflow volume was 25,000 m^3/D at that time and is 150,000 m^3/D at present, and the patterns of the time course volumes into the Plant and those of the time course qualities into aeration tanks remain nearly unchanged. The results are shown in Fig. 22. The pattern of the inflow volume is of a typical urban type reaching a mixmum at about A.M. 11:00 and about P.M. 7:00 and a minimum at about A.M. 5:00. Primary sedimentation tank effluents into the aeration tanks are highly-polluted in the day time and have a low pollution factor in the morning with BOD 70-120 mg/ℓ, COD 40-70 mg/ℓ, and SS 20-50 mg/ℓ.

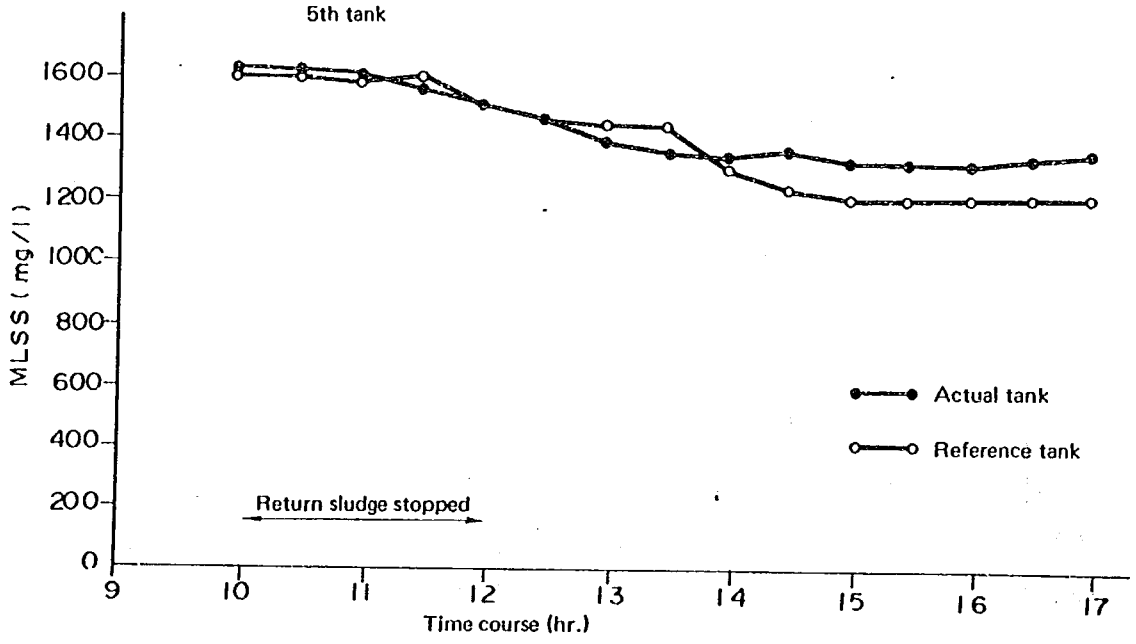


Fig. 21 (5) MLSS Time-course Variations (5th tank)

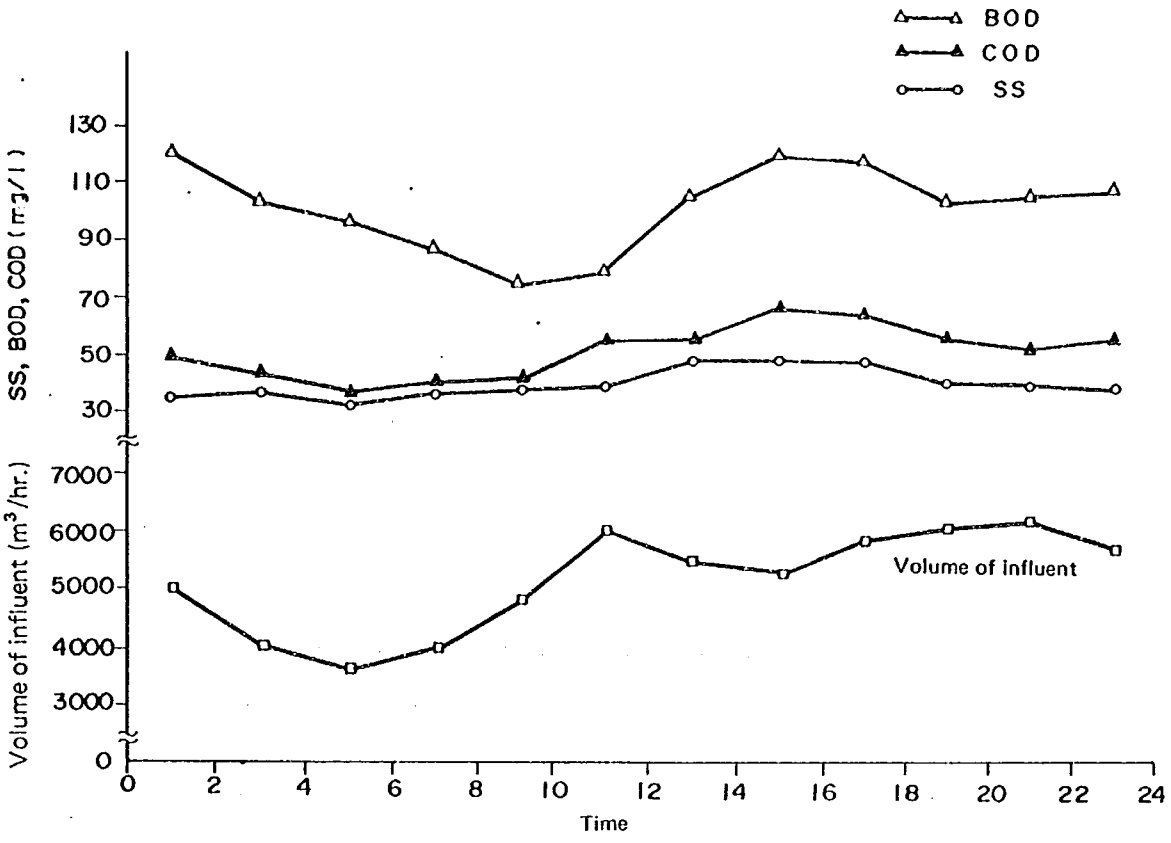


Fig. 22 Time-course Variations of the Volume and Qualities of Inflow

d) The DO profile in the tank

The distribution of DO and the removal of soluble BOD and COD from the inlet to the outlet of the aeration tanks were investigated by equally distributing and adjusting the air flow rate into the each tank. As shown in Fig. 23, the DO distribution assumed a typical curve in which the concentration was lower at the inlet and higher at the outlet and even higher in the fourth and fifth tanks due to completion of the treatment.

Both the inflow qualities and DO time course variations in the respective tanks were investigated.

DO meters were installed in the second and fifth tanks. As shown in Fig. 24, the DO levels in the second tank were low, 0.2 ∿ 0.7 mg/ℓ, and unresponsive to load variations observed.

The DO values in the fifth tank varied from 1.0 mg/ℓ to 4.5 mg/ℓ and were responsive to load variations accompanied by a time lag. If we had used a DO sensor in the second tank in constant DO control, the variations of DO values in the fifth tank would be wide.

In view of the results of a) ∿ d), it was decided that the DO sensor was to be located in the fourth tank.

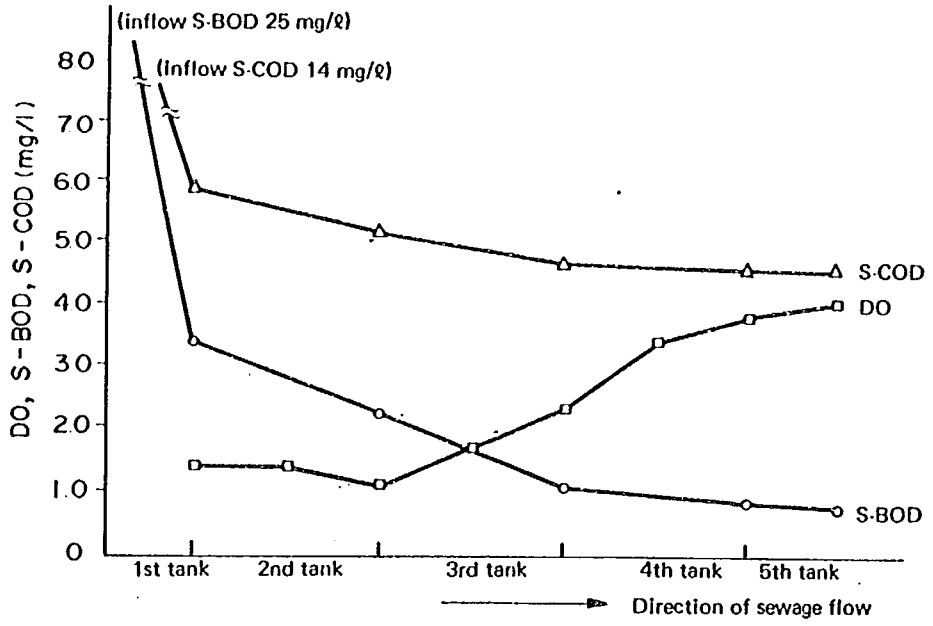


Fig. 23 DO Distribution and Behaviors of soluble BODand soluble COD

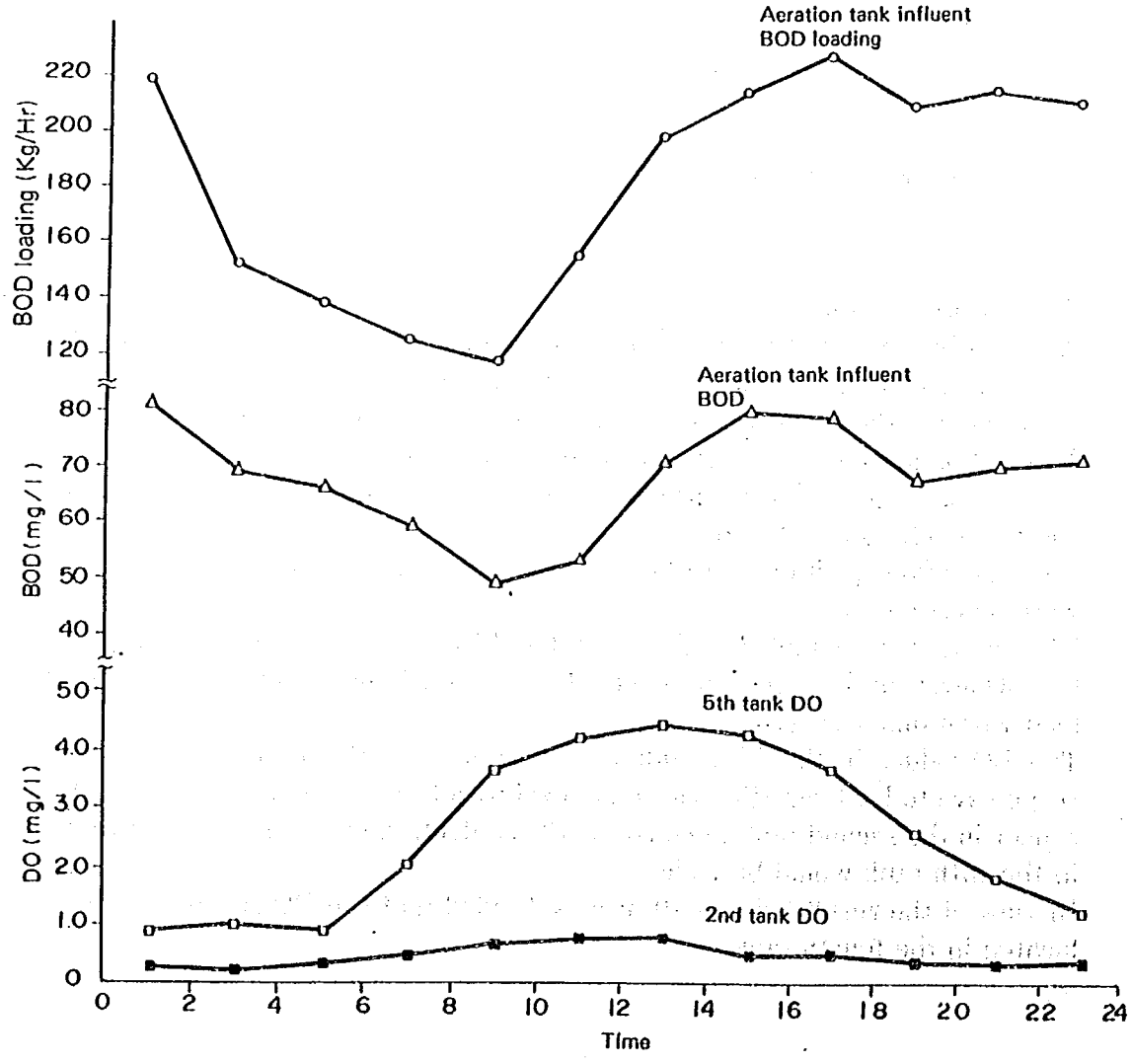


Fig. 24 Time-course Variations of BOD of Aeration Tank Inflow and DO in the 2nd tank and 5th tank

7.1.2 Control Characteristics

There are three types of constant DO control methods: the feed-forward control system provided with a constant ratio of Rr (oxygen ultilization rate) to KLa (overall oxygen transfer coefficient), the rate which is an external disturbance factor, the feedback control system in which the KLa is so operated that the DO value measured in the aeration tank may be kept constant; the feed forward-feedback control system which is a combination of the above two systems.

The plant has introduced the feedback control system and the feedback-feed forward control system, but the present investigation relates totally to the operation examples of the feedback control system. Fig. 25 shows the control algorithm and Fig. 26 the control diagram.

In the control system, PID calculation is so performed that the deviation of the Set point DO value (SV) with the Process DO value (PV) may be approximately equal to zero. The output is converted into the manipulated value (MV) which is fed to the air flow control system of the aeration tank.

Fig. 27 shows the DO time course variation in the fourth aeration tank in the case of constant DO control and constant air flow control. When the air flow is constant, the DO is high in the daytime or about 4.0 mg/ℓ, while it is very low at night or less than 1.0 mg/ℓ. In the case of constant DO control, all of the set point DO values, 1.0 mg/ℓ and 2.0 mg/ℓ were responsive, being constant within a range of ±0.1 mg/ℓ.

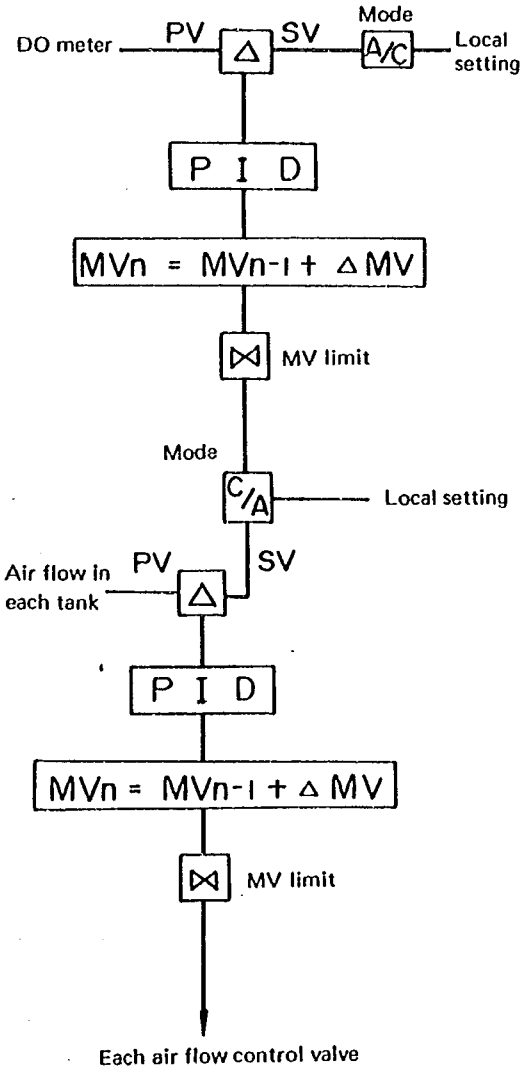


Fig. 25 The Algorithm of Constant DO Control

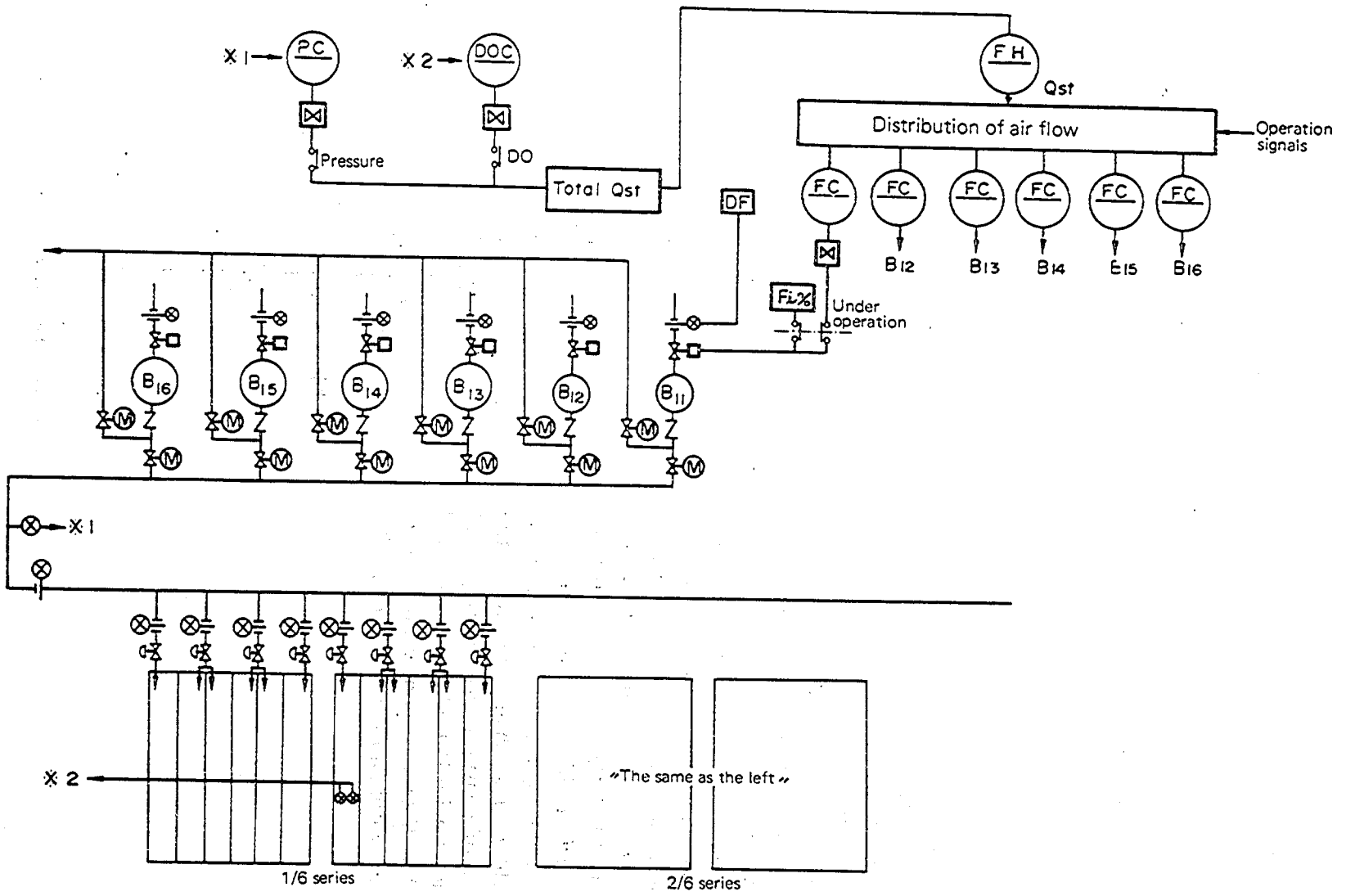


Fig. 26 Blower Air Flow Control Diagram

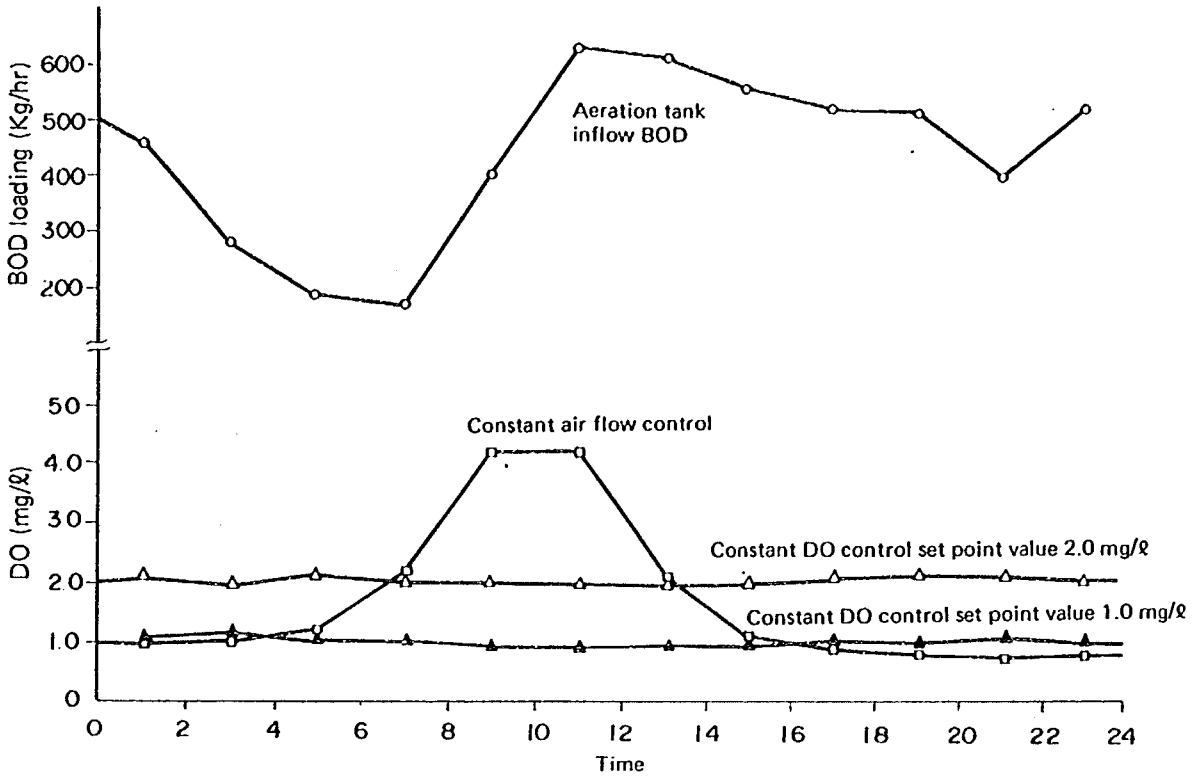
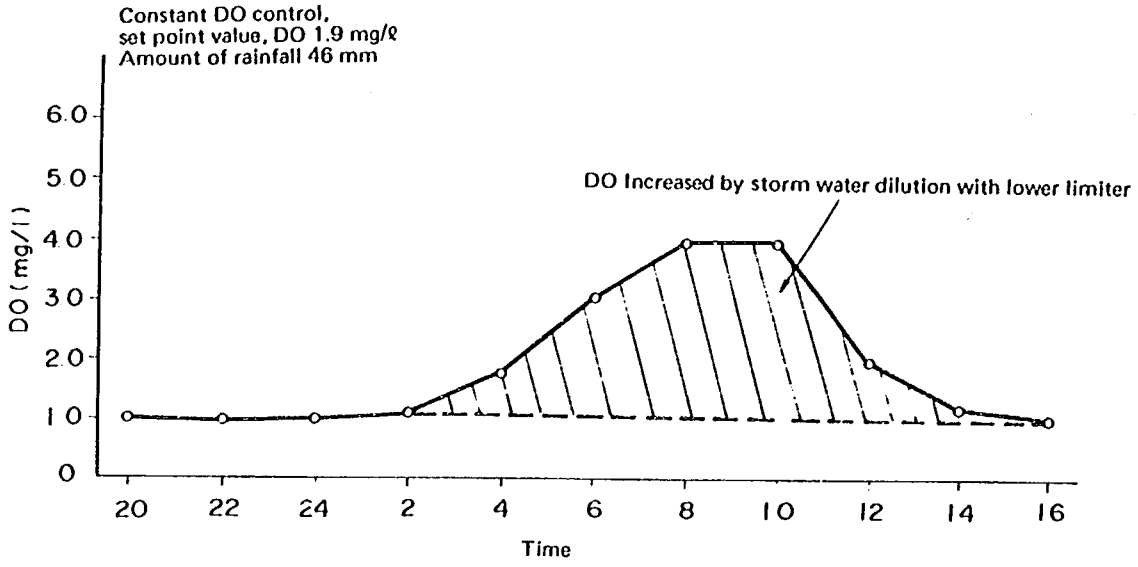


Fig. 27 Time-course Variations of DO of the 4th Aeration Tank and Inflow BOD Loading (Dry weather)

The DO time course variations in the cases of constant DO control and constant air flow control were also investigated under wet weather conditions, because the plant experiences relatively more rainy days, and the number of rainy days numbers 50 corresponding to 13% of the annual total number in 1981. The results are shown in Fig. 28. In the case of constant air flow control, the DO was low, 0.5 mg/ℓ due to the increased inflow loads at the initial flush and then gradually increased due to storm water inflow, reaching more than 6.0 mg/ℓ.

In the case of constant DO control (set point value, 1.0 mg/ℓ) in relation to a nearly identical amount of precipitation, the DO was kept constant despite being affected by the initial flush.

Although storm water inflow decreased the DO requirement to a level less than the minimum air flow in the maintenance of spiral flow, the actual air flow rate was limited by a lower limiter and the DO temporarily increased to 4.0 mg/ℓ. The DO requirement increased afterwards and the DO was again kept constant.



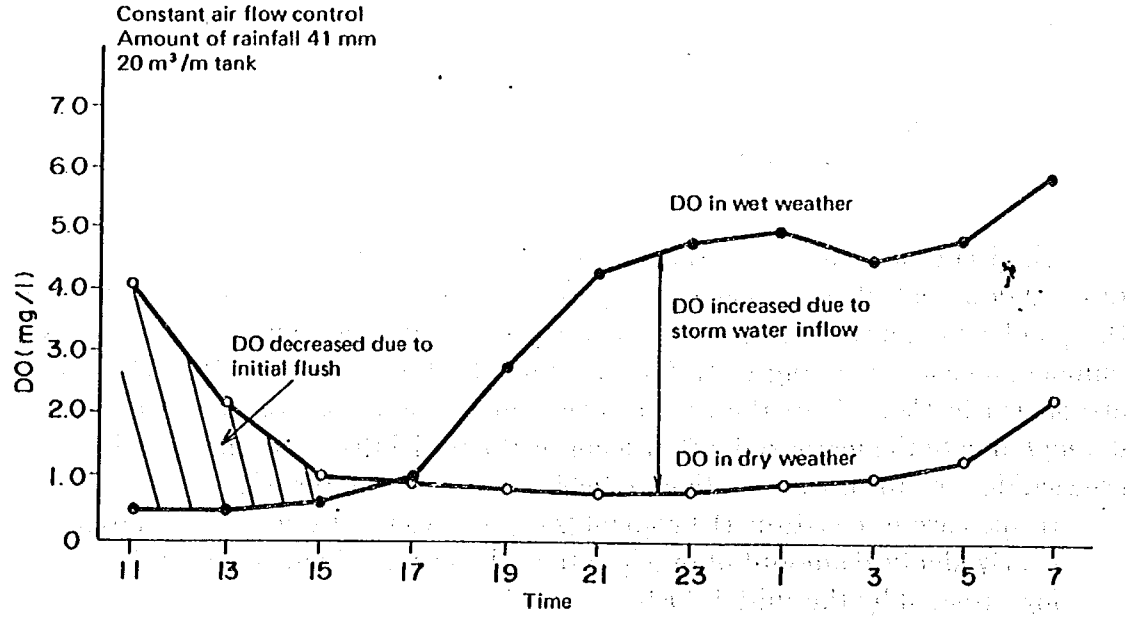


Fig. 28 DO Time-course Variation in the Fourth Aeration Tank (Wet Weather)

7.1.3 Treatability of Operation under Constant Air Flow Control and constant DO Control

The DO sensor has been proved to be adequately located in the fourth aeration tank under constant DO control.

The DO set point value which is one of the important control factors cannot be simply determined because it is dependent upon various other factors. Treatment conditions were investigated for a constant flow rate control mode so that the DO values of the aeration tank effluent might be 1.0 mg/ℓ to 3.0 mg/ℓ and for a constant DO control in which the DO set point values were 1.0 mg/ℓ and 2.0 mg/ℓ.

The contents investigated were DO, Rr, soluble BOD and COD, and nitrogen in the influent, and each tank which was a section of 5 divisions from the inlet to the outlet and BOD, COD, SS, and nitrogen in the secondary treated water.

a) Example of Constant Air Flow Rate Control Operation

Fig. 29 shows the behavior of each component from the inlet to the outlet of the aeration tank. The DO distribution was 2.1 mg/ℓ in the first tank, 5.2 mg/ℓ in the fourth tank, and 3.4 mg/ℓ in the fifth tank. The cause of low DO in the fifth tank is the reduced air flow rate of the outlet diffuser (No. 8 in Fig. 20). Removal of soluble COD was lower in the fourth tank, 70%, but removal of soluble BOD was 80% in the first tank and more than 95% in the third tank showing the completion of carbonaceous BOD removal.

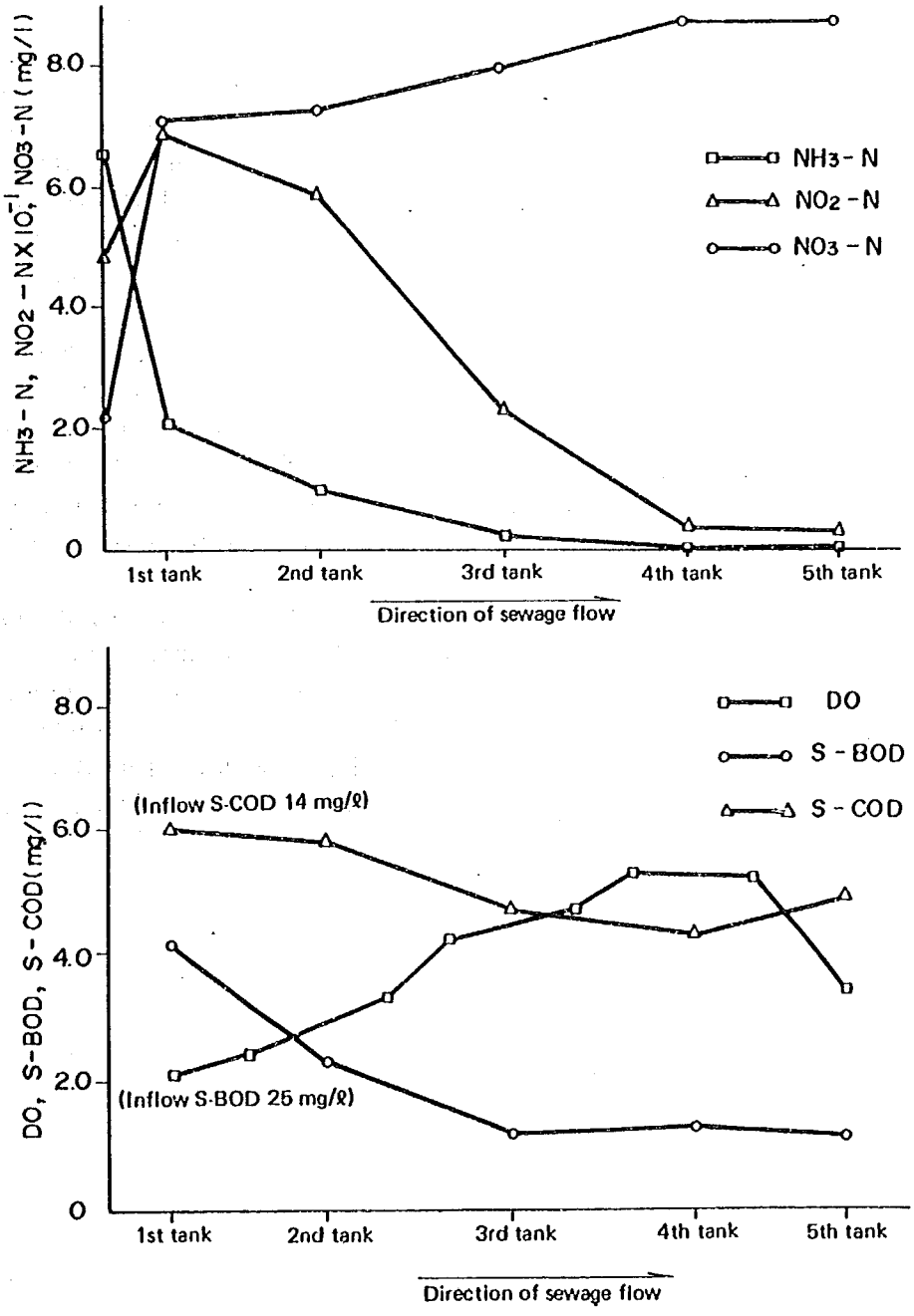


Fig. 29 Behaviors of DO, soluble BOD, soluble COD, and N under Constant Flow Rate Control (Nov. 1981)

The behaviour of nitrogen compounds was as follows: Ammonia nitrogen was removed at a rate of 74% in the first tank and 100% in the fourth tank in which it reached by 0 mg/ℓ.

Nitrate nitrogen showed a tendency converse to ammonia nitrogen, the detected values were 2.2 mg/ℓ in the influent, 7.1 mg/ℓ in the first tank, and 8.8 mg/ℓ in the fifth tank. Nitrite nitrogen somewhat increased in the first tank but decreased in the second tank and the subsequent tanks and 0 mg/ℓ in the fourth tank showing completion of nitrogen compounds treatment (nitrification).

Rr's were 25 mg/ℓ·Hr in the first tank, 15 mg/ℓ·Hr in the third tank, and 9.0 mg/ℓ·Hr in the fifth tank respectively. In view of the Rr's also, carbonaceous and nitrogenous BOD removal were completed at the aeration tank outlet.

BOD, COD, and SS in the effluent were low, therefore good performance was obtained.

b) Example of Constant DO Control Operation

Figs. 30 and 31 show the behaviour of each component in the aeration tank in summer (water temperature 24.2°C, DO set point value 1.0 mg/ℓ) and in winter (water temperature 12.8°C, DO set point value, 2.0 mg/ℓ) respectively. In both cases, the DO levels were lower than the constant air flow rate control, but the removal of soluble BOD and COD, and the pattern of decreased ammonia nitrogen and increased nitrate nitrogen were similar to the constant air flow rate control.

Fig. 32 shows the behaviour of each component under the same operation conditions (water temperature 12.8°C, DO set point value 2.0 mg/ℓ) as those in Fig. 31. As compared with Fig. 31, the DO patterns, the removal of soluble BOD and COD, and the decrease in Rr were similar but the behaviour of nitrogen compounds was quite different. In Fig. 32, a smaller degree of decrease in ammonia nitrogen and a small degree of increase in nitrate nitrogen were obseved.

It is considered that low water temperatures and DO concentrations do not permit nitrification (oxidation from ammonia nitrogen to nitrate nitrogen) in aeration tanks. During actual operation in the plant, nitrification sometimes proceeded when the DO concentration was low (Fig. 30) and even when both the water temperature and the DO concentration were low. (Fig. 31).

Table 11 shows the data relating to the operation conditions and treated water eqalities in Figs. 29 ∿ 32.

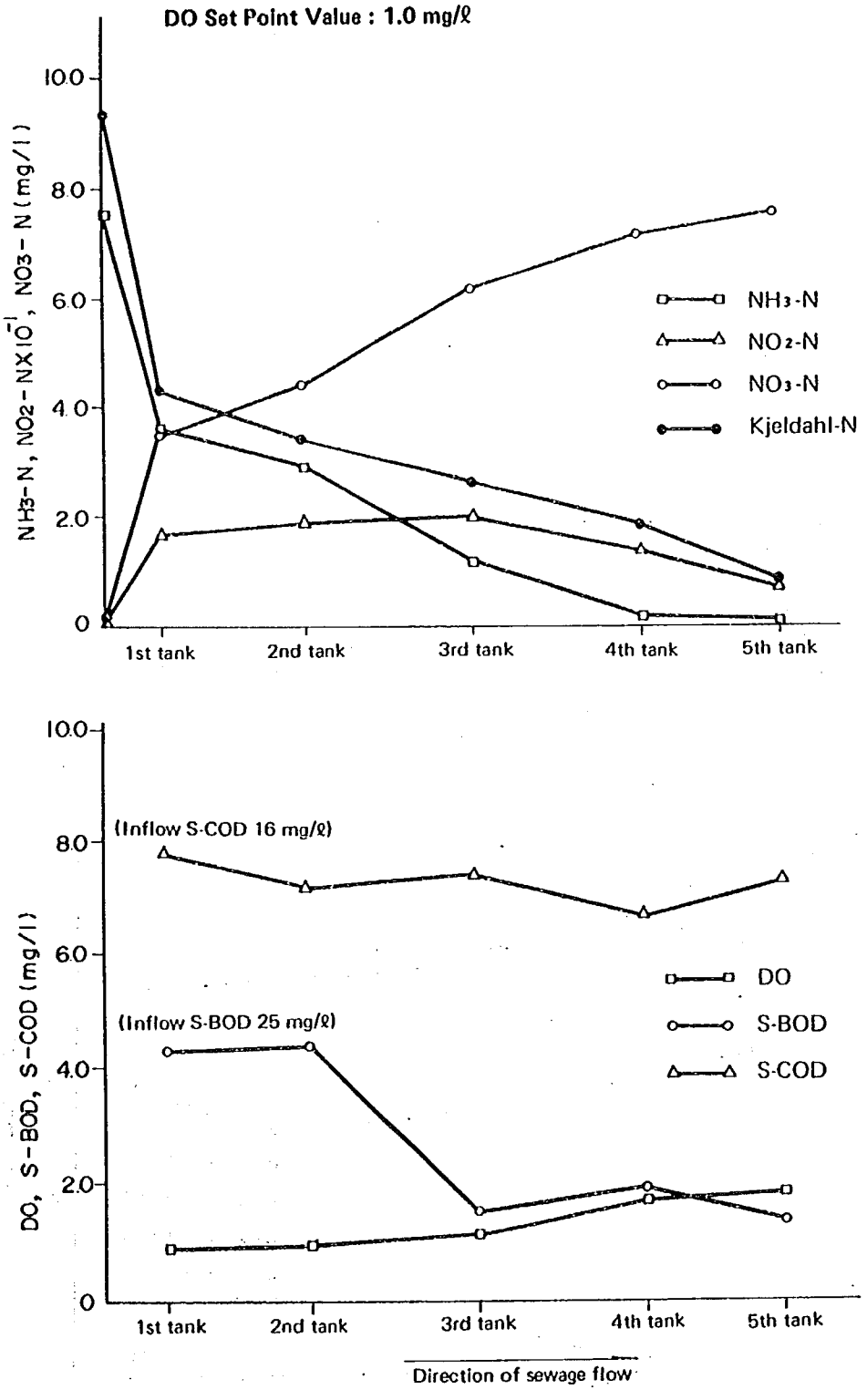


Fig. 30 Behavior of DO, soluble BOD, soluble COD, and N under Constant DO Control (Aug. 1981)

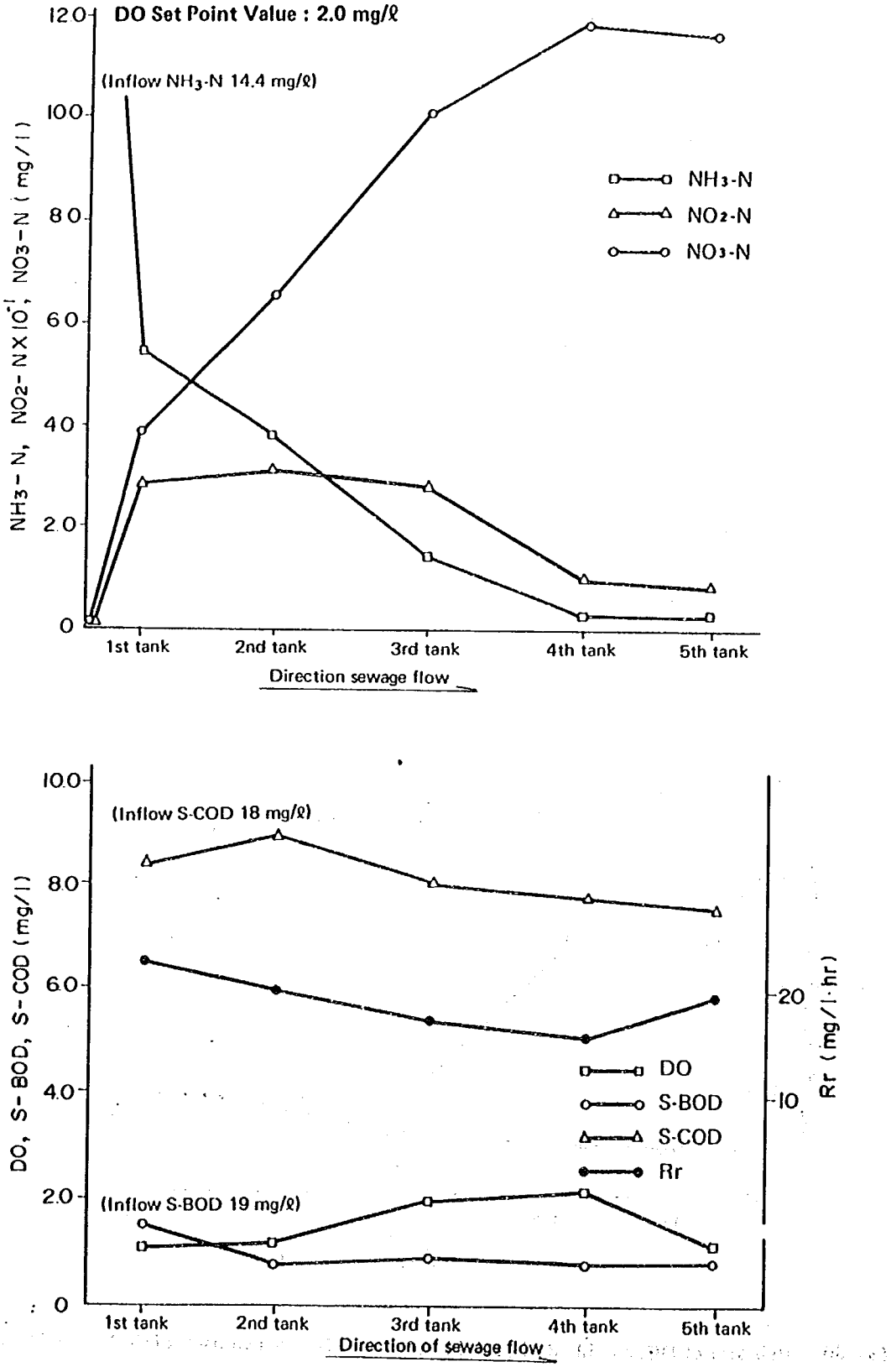


Fig. 31 Behavior of DO, soluble BOD, soluble COD, N and Rr's under Constant DO Control (Jan. 1982)

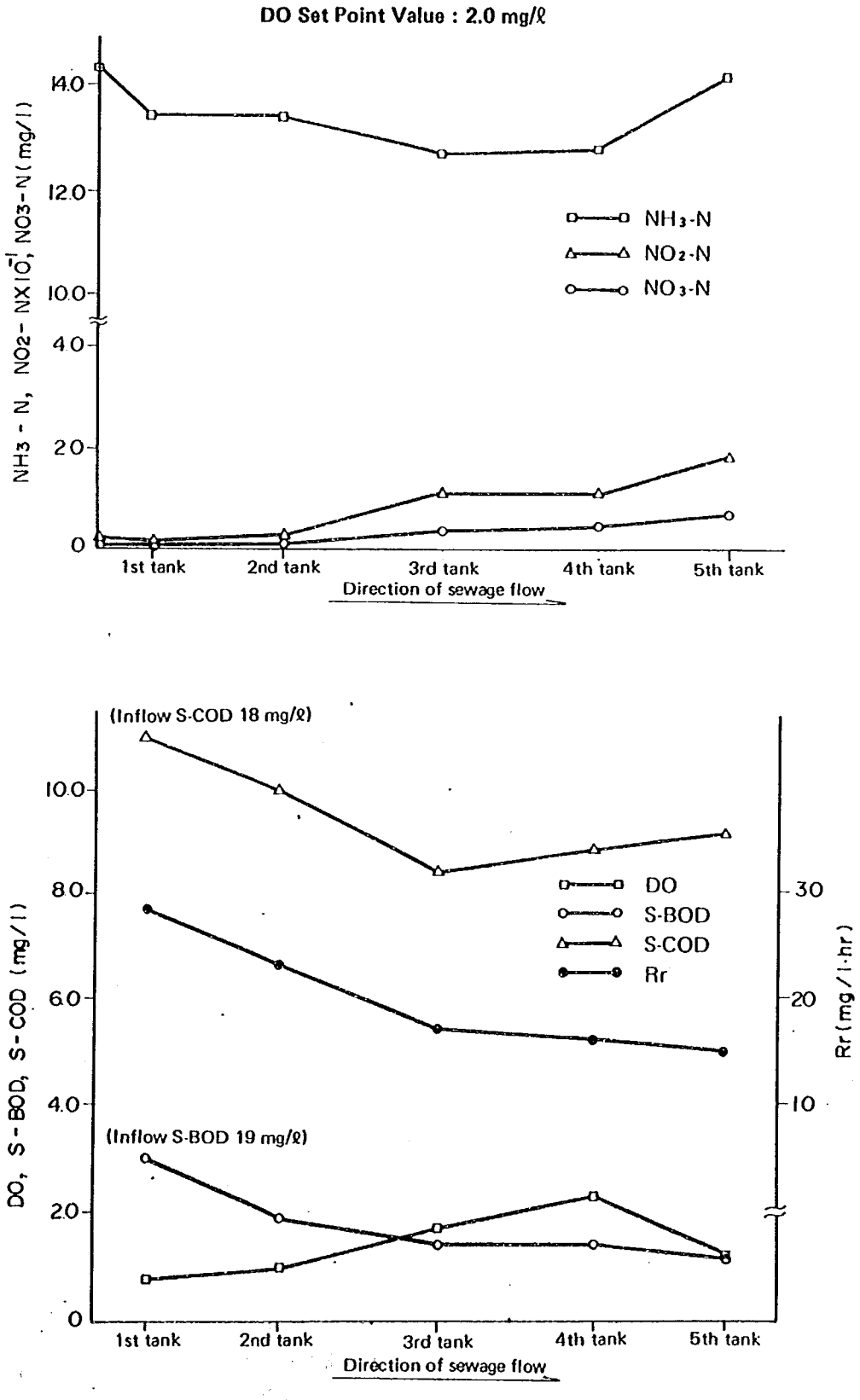


Fig. 32 Behavior of DO, soluble BOD, soluble COD, N and Rr's under Constant DO Control (Jan. 1982)

Table 11 Operational Conditions and Treated Water Quantities in each Operation Mode

Month	Control mode	Water temperature (C°)	MLSS (mg/ℓ)	BOD-SS loading (BODkg/100kg SS • day)	Sludge age (day)	Blowing ratio (m^3/m^3)	Air flow quantity per BOD removal (m^3/BOD•kg•day)	BOD (mg/ℓ)	COD (mg/ℓ)	SS (mg/ℓ)	Kjeldahl-N (mg/ℓ)	NH_3-N (mg/ℓ)	NO_2-N (mg/ℓ)	NO_3-N (mg/ℓ)	Referring diagraph
Nov. 1981	Constant air flow	17.7	1,300	18	11	3.3	82	3.0	5.0	2.6	2.4	0.6	0.13	8.3	Fig. 29
Aug. 1981	Constant D O (1.0mg/ℓ)	24.2	1,100	26	8.6	4.6	84	4.9	8.2	1.5	2.4	1.8	0.12	6.9	Fig. 30
Jan. 1982	Constant D O (2.0mg/ℓ)	12.8	1,640	15	23	7.3	109	3.7	8.7	2.4	4.1	0.3	0.24	11.6	Fig. 31
Jan. 1982	Constant D O (2.0mg/ℓ)	12.8	1,580	16	22	4.0	68	14	11	4.8	16.9	15.2	0.09	0.3	Fig. 32

c) DO Set Point Value

DO levels maintained at somewhat higher than a proper value may cause over-aeration, resulting in activated sludge deflocculation and sludge floc overflowing; whereas, DO levels maintained at a somewhat lower level may cause oxygen deficiency, resulting in decreased removal of organic matter. In addition, the proper DO value may vary with water temperature fluctuations, the properties of the activated sludge (whether bulking occurred or not), and the influent qualities.

Although an evaluation of constant DO control should correspond to the above factors, it is at present focused on an evaluation as energy-saving, namely set point DO values. In wet weather, constant DO control under the conditions shown in Table 12 resulted in the values of blowing ratio 2.2, when the set point value was 1.0 mg/ℓ and the blowing ratio 2.7 when the set point value was 2.0 mg/ℓ and the ratios were smaller than 3.0 under constant air flow control operations. For this reason, a decrease in the air flow quantity by decreasing the set point DO value while supervising the DO levels in wet weather will be sufficiently possible and practical.

As is obvious referring to the results in b), constant DO control operation in the plant (water temperature 12.4°C, set point DO value 2.0 mg/ℓ) allowed nitrification in some cases but not in others. Under the operating conditions, the blowing ratio and the air flow rate per unit BOD removal when nitrification occurred were 1.8 times and 1.6 times greater than the case when nitification did not occur.

Table 12 Control Modes and Air Flow Ratios in the Wet Weather

Control mode / Item	Constant DO Control		Constant air flow control
	Set point value 1.0mg/ℓ	Set point value 2.0mg/ℓ	
Treated volume (m^3/D)	201,000	190,000	184,000
Air flow quantity (m^3/D)	507,000	514,000	571,000
Blowing ratio (m^3/m^3)	2.5	2.7	3.4
Treated volume (m^3/D)	198,000	214,000	220,000
Air flow quantity (m^3/D)	526,000	642,000	576,000
Blowing ratio (m^3/m^3)	2.2	3.0	2.6
Treated volume (m^3/D)	217,000	219,000	199,000
Air flow quantity (m^3/D)	441,000	544,000	571,000
Blowing ratio (m^3/m^3)	2.0	2.4	2.9
Mean blowing ratio (m^3/m^3)	2.2	2.7	3.0

* Data were taken on days which had more than 20mm precipitation during Sept. 1981 and Nov. 1981.

Under such constant DO control operation, the properties of the activated sludge (mainly the degree of proliferation of nitrifying bacteria) may cause occurrence and cessation of nitrification, resulting in large range variations in the air requirements. If the operation and control are aimed at energy-saving, the set point DO value will be determined to be somewhat lower so as to minimize nitrification. If the operation and control are made by taking the effects on the environment of effluent qualities and the subsequent nitrogen removal into consideration, the set point DO value will be increased so that the effluent is nitrificated as thoroughly as possible.

Accordingly it will be meaningless to discuss energy-saving (decreased rate of air flow) disregarding the above fact. Further investigations will be necessary concerning the relationships between set point DO values and Rr's, and between nitrification and organic matter removal or the properties of the activated sludge (proliferation of nitrifying bacteria).

7.2 Return sludge Quantity Control

In parallel with DO control, return sludge amount control so as to provide an appropriate quantity of microorganisms corresponding to the amount of inflow organic matter should be taken into consideration for effective and stable sewage treatment.

Methods for controlling the amount of return sludge under investigation involve the return sludge amount/inflow quantity ratio control by feed-forward control (constant rate of return sludge control), constant MLSS control and constant F/M control by feed-back control.

As Kanagawa Sewage Treatment Plant is provided with two sludge control systems: a system of constant rate of return sludge amount and another mode of constant rate of return sludge solid considering the concentration of return sludge, investigations were carried out in relation to both modes.

7.2.1 Outline of Control

Fig. 33 gives the layout of the aeration tanks and secondary sedimentation tanks. The capacity of the latter is 1,660 m^3 per tank with a designed retention time of two hours.

The sedimentation tanks consist of one tank per two aeration tanks, a total of 6 tanks both in the I and II lines.

The sludge withdrawal pipe of each of the secondary sedimentation tanks is equipped with an electromagnetic flowmeter and an air valve and is connected to a return sludge pump and an excess sludge pump directly. The end of each of the pumps is equipped with an electromagnetic flowmeter and a sludge concentration meter.

Fig. 34 shows the algorithm of the return sludge amount control in the plant. The constant amount of return sludge control process consists of feeding the return amount to a set point value (SV), obtaining the deviation from the process value (PV), performing the P/I calculation so that the deviation approaches 0, and feeding the output to the control system of the return sludge pump as an manipulated value (MV).

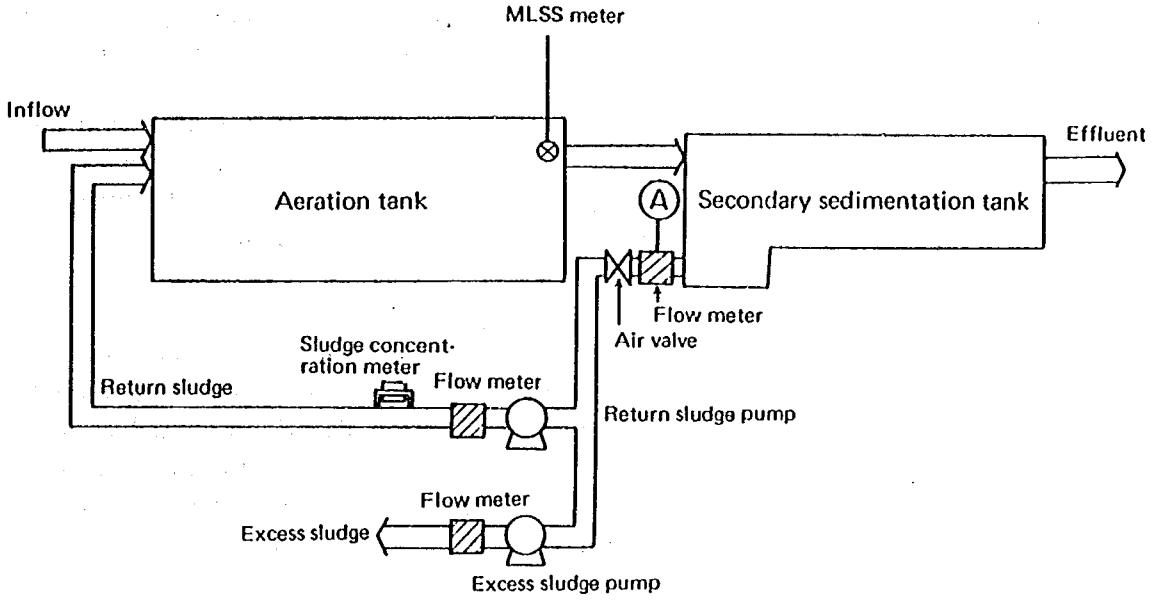


Fig. 33 Flow Diagram of Aeration Tank and Secondary Sedimentation Tank

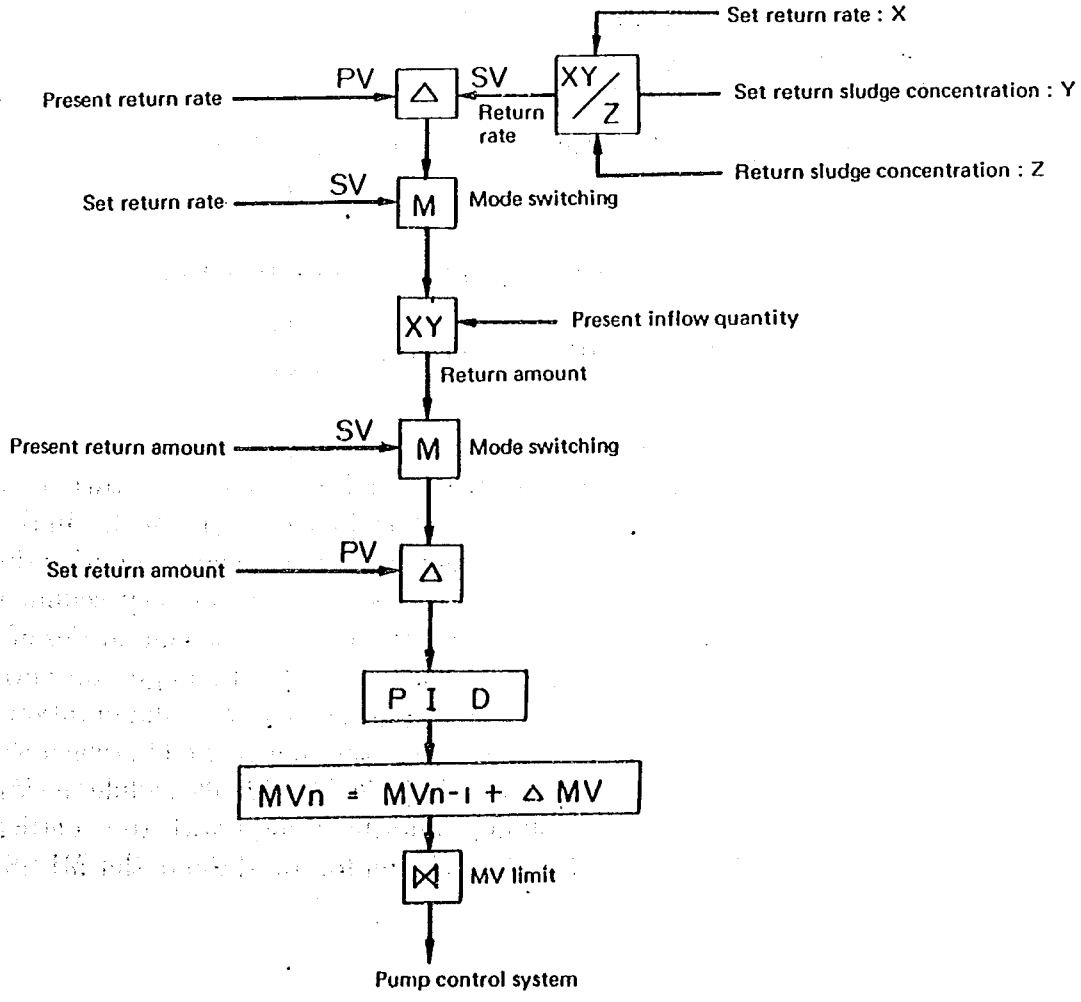


Fig. 34 The Algorithm of Constant Ratio of Return Sludge Control

The constant rate of return sludge control is a process in which the return rate is fed as an input into the set point value (SV) which is the return amount multiplied by the present inflow quantity.

In the constant rate of return solid control, the sludge concentration and the return rate are applied as input to the set point value to modify the return rate from the process value of the sludge concentration meter. The return amount multiplied by the process value of the inflow quantity is given as the set point value.

7.2.2 Control Characteristics

Control of the return sludge amount is performed in order to make MLSS in an aeration tank constant. A discussion was carried concerning relationships between inflow quantities, return sludge concentrations and MLSS under each control mode (constant return amount, constant return rate, and constant rate of return solid). MLSS was calculated by the following equation in the inlet (the first tank) and measured by the MLSS meter in the outlet (the fifth tank).

Inflow quantity into an aeration tank : Q (m^3/day)
Suspended solid concentration of aeration tank inflow : C (mg/ℓ)
Return sludge flow rate : Q_R (m^3/day)
Return sludge concentration : C_R (mg/ℓ)
Return rate : $r = Q_R/Q$
MLSS in the inlet (the first tank)

$$= \frac{Q \times C + Q_R \times C_R}{Q + Q_R} = \frac{Q \times (C + r \times C_R)}{Q + rQ}$$

Where C is negligible because it is much smaller than C_R.

$$\text{Hence MLSS in the inlet (the first tank)} = \frac{r \times C_R}{1 + r}$$

a) Constant Return Amount Control (Conventional Operation)
Fig. 35 shows a typical example operated under a constant return amount in dry weather. Reduced quantity of the inflow increases the MLSS in the inlet, resulting in increased MLSS in the outlet accompanied by a retention time lag. Following this, decreased inflow quantities into a secondary sedimentation tank decrease the return sludge concentrations. Increased quantities of inflow decrease MLSS in the inlet but increase the return sludge concentrations because of incrased inflow quantities into the secondary sedimentation tanks. As the time course variations, the MLSS in the inlet varied between 820 mg/ℓ and 1,690 mg/ℓ with a coefficient of variation (CV) of 20.0%, while the MLSS in the outlet varied between 1,660 mg/ℓ and 2,010 mg/ℓ with the coefficient of variation of 6.7%. The inlet MLSS variation followed the outlet MLSS variations with a time lag.

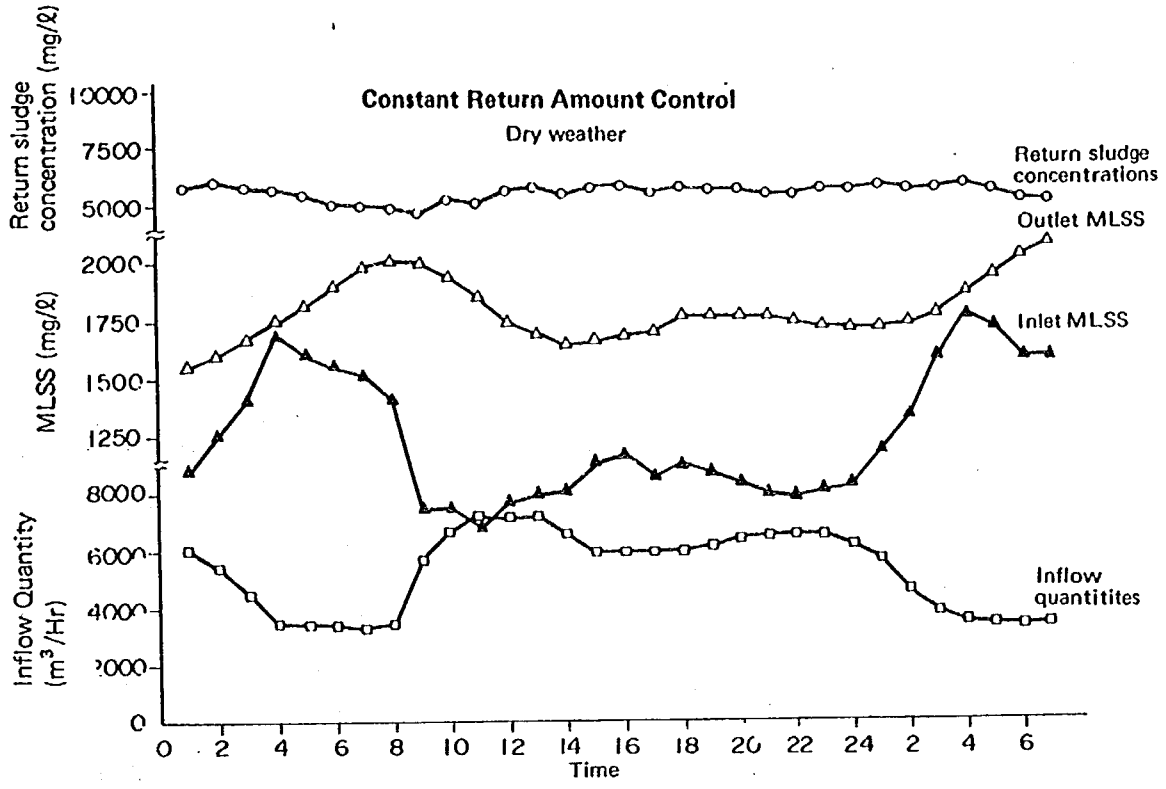


Fig. 35 Time-course Variations in Aeration Tank Inflow Quantities, MLSS and Return Sludge Concentrations

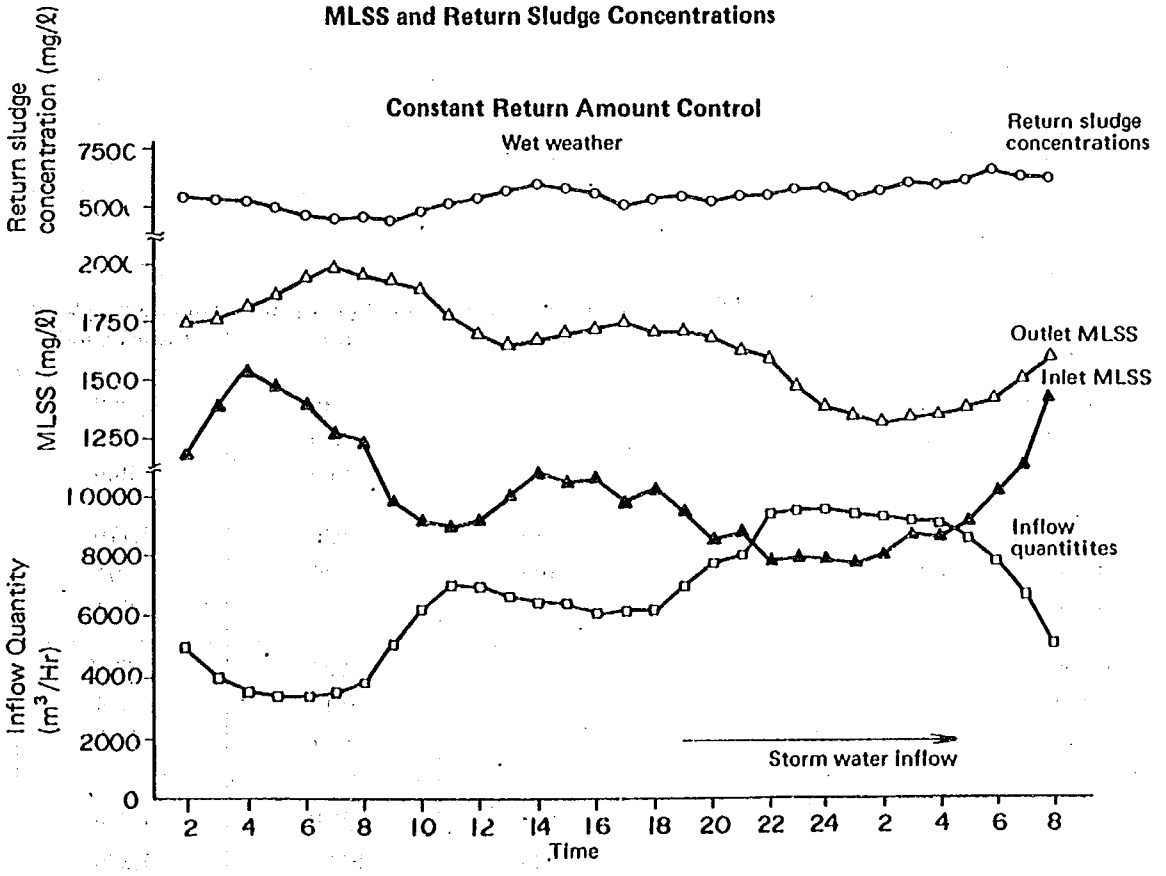


Fig. 36 Time-course Variations in Aeration Tank Inflow Quantities, MLSS and Return Sludge Concentrations

Fig. 36 shows variations in the wet weather. Increased quantities of inflow temporarily decrease the MLSS with the same as that observed under dry weather conditions. The outlet MLSS varied between 1,320 mg/ℓ and 1,990 mg/ℓ with the coefficient of variation of 12.5%, and the latter parameter was more than twice that of the dry weather. (Refer to Table 13).

Table 13 Variations in MLSS, Return Rate, and Return Sludge Concentrations in each Control Mode

Dry Weather

Parameter \ Control mode		Constant return amount	Constant return rate	Constant rate of return solid
Inlet (1st tank) MLSS (mg/ℓ)	$\bar{x}$	1,160	1,680	1,150
Standard deviation (mg/ℓ)	σ	230	170	120
Coefficient of deviation (%)	CV	20.0	10.3	10.3
Outlet (5th tank) MLSS (mg/ℓ)	$\bar{x}$	1,760	1,850	1,780
Standard deviation (mg/ℓ)	σ	118	22.4	21.3
Coefficient of deviation (%)	CV	6.7	1.2	1.2
Return rate (%)	$\bar{x}$	26.9	22.7	20.5
Standard deviation (%)	σ	8.0	0.72	1.6
Coefficient of deviation (%)	CV	29.7	3.2	7.8
Return sludge concentration (mg/ℓ)	$\bar{x}$	5,610	9,000	6,800
Standard deviation (mg/ℓ)	σ	380	890	380
Coefficient of deviation (%)	CV	6.7	9.9	5.5

Wet Weather

Parameter \ Control mode		Constant return amount	Constant return rate	Constant rate of return solid
Inlet (1st tank) MLSS (mg/ℓ)	$\bar{x}$	1,040	1,830	1,400
Standard deviation (mg/ℓ)	σ	235	245	148
Coefficient of deviation (%)	CV	22.6	13.5	10.6
Outlet (5th tank) MLSS (mg/ℓ)	$\bar{x}$	1,630	1,940	1,770
Standard deviation (mg/ℓ)	σ	205	124	48
Coefficient of deviation (%)	CV	12.5	6.4	2.7
Return rate (%)	$\bar{x}$	21.4	22.5	22.4
Standard deviation (%)	σ	6.5	0.17	2.4
Coefficient of deviation (%)	CV	30.4	0.7	10.7
Return sludge concentration (mg/ℓ)	$\bar{x}$	5,480	9,920	7,880
Standard deviation (mg/ℓ)	σ	520	1,320	1,260
Coefficient of deviation (%)	CV	9.5	13.3	16.0

$$CV = \frac{100\sigma}{\bar{x}}$$

b) Constant Return Rate Control

Figs. 37 and 38 show the time course variations in dry weather and wet weather respectively. The return rate in dry weather was 21.5 ∿ 24.5% as the variation range and 3.2% as the coefficient of variation in contrast to 22.5% of the set point value. The control system had a good performance even if inflow quantities changed.

The outlet MLSS varied between 1,800 mg/ℓ and 1,900 mg/ℓ and the coefficient of variation was 1.2%. This means that MLSS in nearly constant. It is attributable to small variations in the return sludge concentrations and the relatively stable MLSS in the inlet.

The return rate in wet weather was more stable than that in dry weather with the variation range between 21.8% and 23.2% and the coefficient of variation of 0.7%. However, larger variation in return sludge concentrations was produced by variations in the inflow quantities followed by variations in the inlet MLSS. As a result, the outlet MLSS variation ranged between 1,770 mg/ℓ and 2,140 mg/ℓ with the coefficient of variation of 6.4%.

c) Constant Rate of Return Solid Control

Figs. 39 and 40 show the time course variations in dry weather and in wet weather respectively. The outlet MLSS in dry weather was constant as with the constant return rate control with the variation range between 1,720 mg/ℓ and 1,790 mg/ℓ and the coefficient of variation of 1.2%.

Although it is caused by stable return sludge concentrations, even when a large concentration variation, 5,800 mg/ℓ ∿ 9,900 mg/ℓ, was observed such as in wet weather, the coefficient of variation of the inlet MLSS was 10.6%, the variation range of the outlet MLSS was between 1,680 mg/ℓ and 1,860 mg/ℓ and the coefficient of variation of the outlet MLSS was 2.7%, with stable values observed.

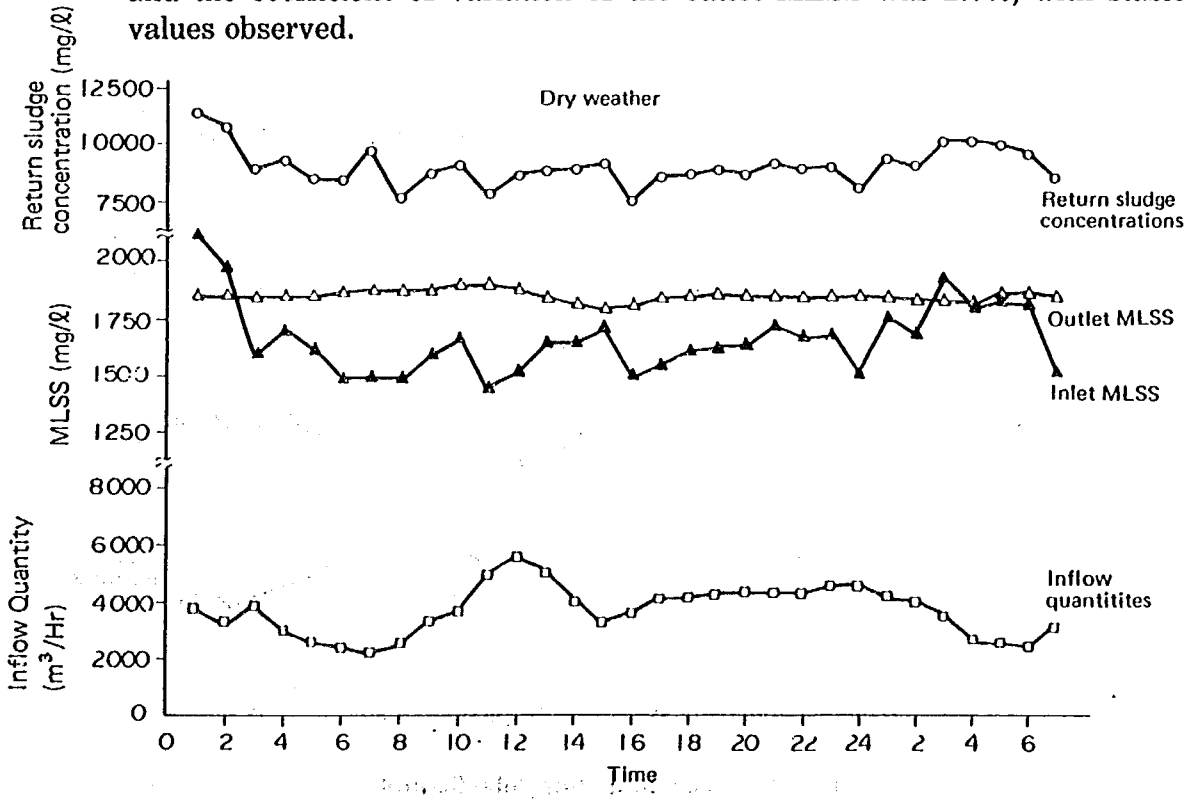


Fig. 37 Constant Return Rate Control

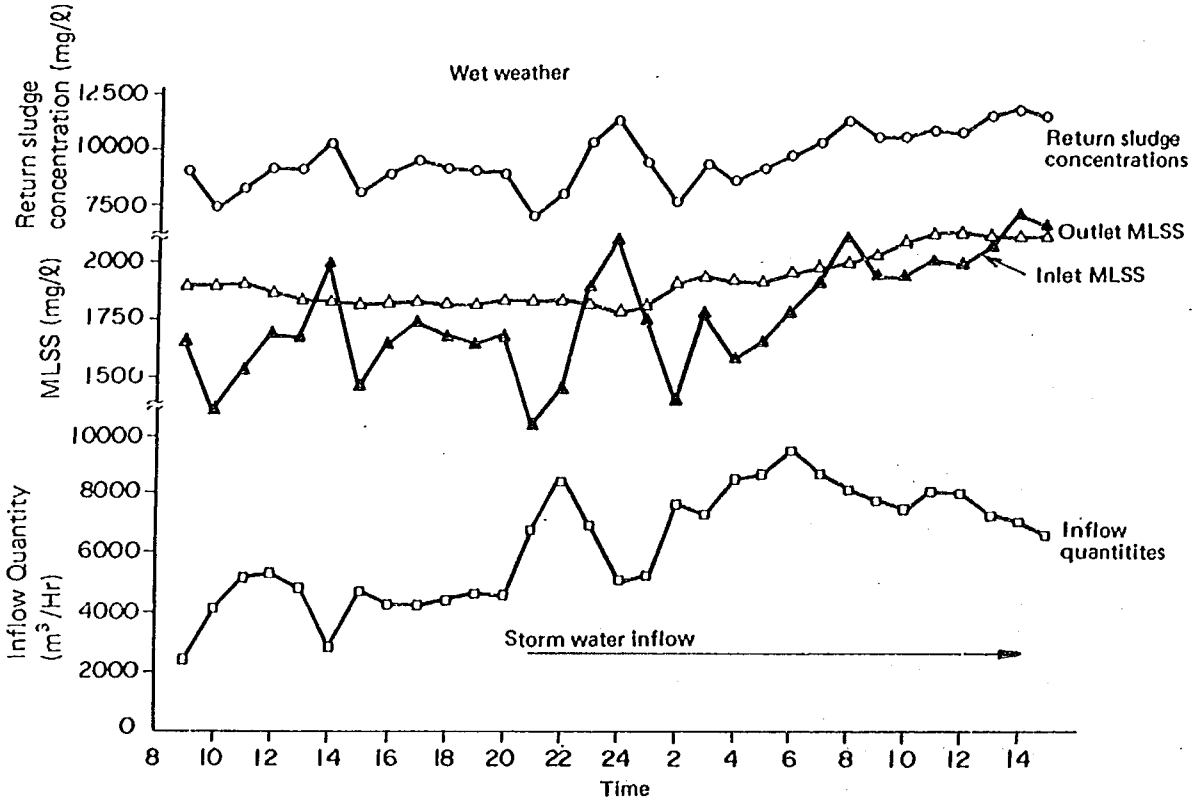


Fig. 38 Constant Return Rate Control

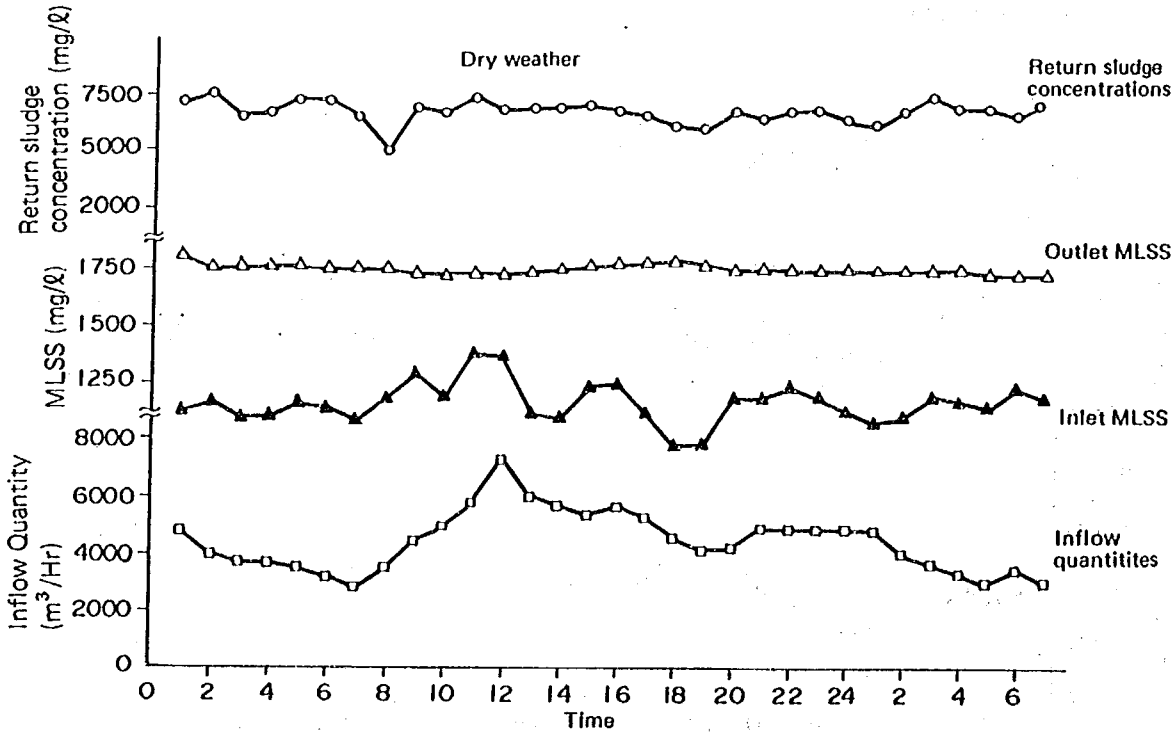


Fig. 39 Constant Return Solid Control

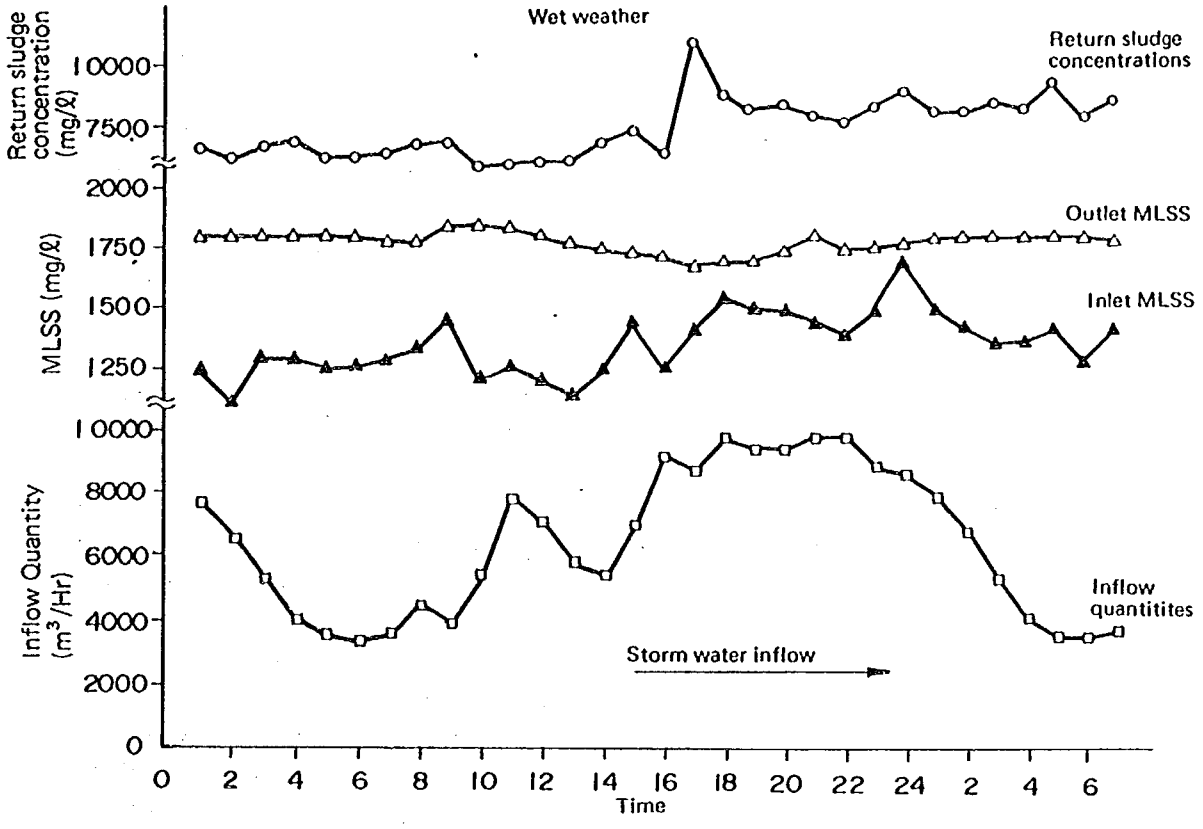


Fig. 40 Constant Return Solid Control

From the above, the following results were obtained.

When the variation range of the return sludge concentration in dry weather is small, MLSS can be made constant both in the constant return rate control and in the constant rate of return solid control. By using these modes, the coefficients of variation of the outlet MLSS were about 1/5 that of the conventional constant return amount control mode. Larger fluctuations in inflow quantities and variation in return sludge concentrations such as in wet weather cannot make the outlet MLSS constant by the constant return rate control. As the constant rate of return solid control will have the tendency to decrease the variation of the return sludge concentration, variation range of the outlet MLSS in the aeration tank becomes smaller.

Although the return sludge control will operate and stabilize the treatment process dependent upon the microorganisms quantities (MLSS as indicator), it will be necessary in future to analyze the correlation between the control system and the treatability concerning the properties of the activated sludge (activity, SVI and biomass), sludge settlability in secondary sedimentation tanks, and the method of withdrawal of excess sludge.

8. Conclusions

We have described the development process of instrumentation for the sewage treatment plants in Yokohama City. Chubu Sewage Treatment Plant is the first plant in the city introducing the centralized supervision and control system for each local block in the sewage treatment lines and sludge treatment lines. The plant next introduced the centralized supervision and control system incorporating a remote control system of the sewage treatment plant and coordinate relay pumping stations. Afterwards, the centralized supervision and distributed control system was employed for the purposes of energy-saving in maintenance and improvement in the effectiveness of treatment functions in accordance with the extended and complicated sewage treatment plant.

The last system was first employed in Kanagawa Sewage Treatment Plant, into which qualitative instrumentation was introduced in addition to conventional quantitative instrumentation.

When the automatic water quality meters used in the plant were evaluated, DO meters were usable for control and supervision, MLSS meters and turbidity meters were useful for supervision of treatment processes, and return sludge concentration meters were usable for control sensors.

DO-controlled and return sludge amount-controlled operations were exemplified including the above sensors and the control systems.

The DO meters were installed in the fourth aeration tank in view of the mixing characteristics, inflow quantities, and water quality characteristics and the DO set point values were fixed at 1.0 mg/ℓ and 2.0 mg/ℓ. The degree of the treatability of the DO control and the degree of the conventional constant air flow rate control were nearly equal. It was found that only constant DO-controlled saving-energy (reduced air flow) could not be discussed by disregarding the considerable influences of the oxidation of the nitrogen compounds in the sewage in the determination of the set point values for DO control.

The constant return rate control and the constant rate of return solid control were investigated in the return sludge control.

There was no difference between both the above modes, and therefore the constant return rate control was found to be able to be performed by constant MLSS without the more sophisticated control mode such as return solid control.

The purposes of automated sewage treatment control are to obtain well treated water steadily and to reduce the energy consumed by blowers using DO control. The results could not be clarified this time and a further investigation will be necessary in future.

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Accompanied Date List of Meters

Low-stage Pumps and High-stage Pumps

	Name	Detector	Measurement range	Note
1	Low-stage inflow sewage quantity	Ultrasonic flow meter	0~7.5/0~15 m³/S	UVH-100, Double range
2	Low-stage inflow trunk sewer water level 1	Throw-in pressure-type water level meter	−20.4~−10.4 m	SL-10 P
3	〃 2	〃	−20.4~−16.4 m	〃
4	Low-stage pump well water level 1	〃	−20.4~−10.4 m	〃
5	〃 2	〃	−20.4~−16.4 m	〃
6	Low-stage pump delivery well water level	〃	−4.5~+5.5 m	〃
7	Pump discharge flow rate ×2	Electromagnetic flow meter ×2	0~2 m³/S	EMS101/D508
8	High-stage inflow sewage quantity	Ultrasonic flow meter	0~10/0~60 m³/S	UVH-100, Double range
9	High-stage inflow trunk sewer water level 1	Throw-in pressure-type water level meter	−13.7~−9.7 m	SL-10 P
10	〃 2	〃	−13.7~−3.7 m	〃
11	High-stage sanitary pump well watger level 1	〃	−15~−8 m	〃
12	〃 2	〃	−18~−8 m	〃
13	High-stage storm pump well water level 1	〃	−15~−8 m	〃
14	〃 2	〃	−18~−8 m	〃
15	Storm water delivery well water level	〃	−1.5~+5.5 m	〃
16	Pump discharge flow rate	Electromagnetic flow meter	0~3 m³/S	EMS101A508
17	〃 ×2	〃 ×2	0~8 m³/S	〃
18	〃 ×2	〃 ×2	0~15 m³/S	〃

Grit Chamber

	Name	Detector	Measurement range	Note
1	Dregs storage hopper level	Ultrasonic level meter	0~4.5 m	F712/D512
2	Sand storage hopper level	Load cell gravimeter	t	
3	Water level differences before and after mechanical screen	Air purge level meter	0~300 mm	DPF100

Blower

	Name	Detector	Measurement range	Note
1	Blower total discharge flow rate	Orifice (ϕ1350)	0~1800 Sm^3/min	
2	Blower discharge temperature	Temperature-measuring resister	0~100°C	
3	Blower discharge pressure	Pressure (differential pressure) transmitter	0~10,000 mmH_2O	DPF100
4	Blower suction air flow ×2	Orifice (ϕ500) ×2	0~400 Sm^3/min	
5	"	Orifice (ϕ600)	0~500 Sm^3/min	

Primary Sedimentation Tank

	Name	Detector	Measurement range	Note
1	Aerated grit chamber temperature	Temperature-measuring resister	0~30°C	
2	Aerated grit chamber pH	Immersion type pH meter	4~10 pH	PH100-G
3	Aerated grit chamber temperature	Turbidity meter	0~300 ppm	WLS301
4	Aerated grit chamber inlet air flow	Two orifices	0~6 Sm^3/min	
5	Aerated grit chamber air flow ×2	Two orifices	0~10 Sm^3/min	
6	Air flow of aerated grit chamber and conduit to primary sedimentation	Orifice	0~30 Sm^3/min	
7	Primary sedimentation tank sludge branket ×2	Two ultrasonic sludge blanket meters	0~5 m	NU-AS
8	Primary sedimentation tank turbidity	Turbidity meter	0~300 ppm	WLS301
9	Primary sedimentation tank sludge withdraw amount	Electromagnetic flow meter (ϕ200)	0~3 m^3/min	F511/D501
10	Primary sedimentation tank sludge concentration	Ultrasonic sludge concentration meter	0~6%	NU-S

Secondary Sedimentation Tank

No.1 ~ No.17 : per one line (Two lines have been constructed)

	Name	Detector	Measurement range	Note
1	Air flow of conduit to aeration tank	Orifice	0 ~ 20 Sm^3/min	
2	Air flow of return sludge condudit	〃	0 ~ 10 Sm^3/min	
3	Channel total air flow quantity	〃	0 ~ 100 Sm^3/min	
4	Aeration tank air flow quantity ×4	〃 (φ350)×4	0 ~ 50 Sm^3/min	
5	Aeration tank air flow quantity ×4	〃 (φ450)×4	0 ~ 100 Sm^3/min	
6	Total air flow quantity	〃	0 ~ 1800 Sm^3/min	
7	Aeration tank inlet pressure	Pressure transmitter	0 ~ 8000 mmAq	
8	Aeration tank DO ×2	DO meter ×2	0 ~ 10 ppm	DO6S/DO6C
9	Aeration tank outlet DO ×2	DO meter ×2	〃	DO6/DO6C
10	Aeration tank temperature	Temperature-measuring resister	0 ~ 30°C	
11	Aeration tank pH	pH meter	4 ~ 10 pH	PH100-G
12	Aeration tank MLSS	MLSS meter	4 ~ 4000 ppm	
13	Return sludge flow rate	Electromagnetic flow meter (φ400)	0 ~ 40 m^3/min	F521/D501 F521-0-400 UP-2W-AS-NS D501-0-AW -1500-0
14	Excess sludge flow rate	〃 (φ150)×6	0 ~ 3/0 ~ 6 m^3/min	F511/D501 F511-0-150 FP-2F-1A20VY D501-0-CW -1800-0
15	Secondary sedimentation tank-withdrawn sludge flow rate ×6	〃 (φ200)×6	0 ~ 6 m^3/min	F511/D501 F511-0-200 FP-2F-1A20-VY D501-0-AW -1600-0
16	Return sludge concentration	Ultrasonic sludge concentration meter	0 ~ 3%	NU-S102
17	Secondary sedimentation tank sludge blanket ×2	Ultrasonic sludge blanket meter ×2	0 ~ 5 m	NU-AS
18	Concentrated sludge withdrawal flow rate (φ200) ×2	Electromagnetic flow meter (φ200) ×2	0 ~ 6 m^3/min	F511/D501
19	Aeration tank inflow rate ×2	〃 (φ900) ×2	0 ~ 3.5 m^3/sec	

Disinfection

	Name	Detector	Measurement range	Note
1	Water quantity after primary sedimentation tank	Ultrasonic level meters ×2	0～3 m³/S	L701/D512 Partial flume L701-0-A2 D512-1-177251-1A-2
2	Secondary sedimentation tank outlet temperature	Turbidity meter	0～20 ppm	WLS301
3	Secondary sedimentation tank outlet COD	COD meter	0～20 ppm	COD-3
4	Sodium hypochlorite does ×2	Electromagnetic flow meters ×2	0～360 ℓ/h	
5	〃	〃	0～1200 ℓ/h	SL2-400Z
6	Sodium hypochlorite tank level ×2	Ultrasonic level meters ×2	0～2.25 m	Converted to capacity (SL1-400)
7	Effuent channel water level	Throw-in pressure level meter	−2.5～+1.5 m	SL-10P
8	Effuent outlet water level	〃	〃	〃

SESSION 4

TREATMENT OF SLUDGE

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GERMAN REGULATIONS FOR DISPOSAL OF SEWAGE SLUDGE AND GENERAL TRENDS IN TREATMENT TECHNOLOGIES

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KEYWORDS

Sewage sludge; land disposal; agricultural, forestry and horticultural utilization; health aspects; legislation; heavy metals and the limitation of their concentration in sewage sludges and in soils at the application of sludge to agriculturally, forestry and horticulturally used soils; limiting conditions with regard to hygienicly hazardous sludges; limitation of the quantity for agriculturally and horticulturally used sludges; controlled tipping; treatment of sludges as a pre-condition for their disposal; basic operations of sludge treatment; general trends in treatment technologies of sewage sludges; aerobic or anaerobic stabilization; reduction of the water content; disinfection; FR Germany;

ABSTRACT

This paper describes the German practice in land disposal of sludge including legislation and health aspects up to now and in the future. Up to now, the use of sewage sludge in agriculture has been regulated by way of standards and information sheets, taking into account hygienic aspects. The present ordinance pursuant to the Waste Disposal Act sets out binding regulations with regard to pollutants (7 heavy metals) and hygienic requirements, which are explained and the effects of which on the use of sewage sludge are analyzed.

It becomes clear, that the operator of the sewage works alone is not in a position to guarantee that the use of the work's sewage sludge in agriculture is safe. The operators of sewage works have to succeed in preventing or cutting down on the discharge of waste waters with a high heavy metals load and which then enter the sewage works. It is necessary that the day-to-day procedures procedures of granting permits

and of supervision are transparent and calculable. Furthermore it is necessary to recognize which measures are to be taken in the future to comply with modifications of the values which have been laid down up to now. A certain decrease of the use of sewage sludge in agriculture is to be expected.

As far as a further land disposal of sewage sludges will be dropped because of general prohibitions or limitations, either because of their pollution load or because of the lack of application areas or other reasons, it will be absolutely necessary to find another way to dispose sewage sludges by dumping, either with or without preceding incineration. It is a condition for controlled tipping of sewage sludges that no odor nuisances arise, and that the sludges are sufficiently thickened and dewatered.

For this purpose the sludges are sufficiently stabilized - aerobicly or anaerobicly -, thickened and in general conditioned and mechanicly dewatered by centrifuges, belt-presses or filter-presses.

1. Superior Premise for the Disposal of Sewage Sludge

One premise will stand above all further discussions, which Dr. K. IMHOFF [1] already postulated in 1951, that is more than 30 years ago, and which is of high topicality up to now:" All statements on treatment and disposal of sludge must not neglect the fact, that one has to get rid of the sludge of sewage treatment definitely and by all means" - let me add to this out of my own experience and responsibility in the management of sewage treatment works: you must get rid of the sludge at once, at any time and above all: reliable! - "A clarification plant in which this is not possible is of no value. Even by sludge-incineration or -gasifying there will remain ash which has to be removed. A final disposal is only possible in agriculture, in the ocean or in sludge sanitary landfill...."

1.1 Tasks and Aims at Disposal of Sewage Sludge

The preventive steps by environmental protection to dispose residues of waste products without hazard also apply to the residues resulting of sewage treatment, for instance sewage sludges;they also have to be disposed hazardlessly. Hazardless, that means: without hazard for human beings and their environment.

According to the law of conservation of energy and substance, however, sewage sludge as well as other waste products cannot be annihilated really, but only be transformed into other solid, liquid or gaseous substances. They must remain somewhere.

That is why the hazardless disposal of the sewage sludges is only possible firstly

- by their re-use, mainly by their integration into a recycling of substances, in practice for sewage sludges is only the possibility of their integration into the natural recycling of substances, and secondly

- by towing them out of the recycling with the help of sanitary landfill, that means = controlled tipping.

The hazardless disposal of sewage sludge consequently means

- as far as possible and significant make use of their content of valuable substances,
- but also cause no undesirable consequences, which might be detrimental or even pernicious for human beings and the environment.

1.2 Sewage Sludges as a Resource for Raw-Materials

Sewage sludges contain valuable raw-materials, for instance nitrogen, phosphate, calcium, potassium, and magnesium. Consequently, sewage-sludges should be used as a resource for raw-material in land disposal by recycling.

Only the agricultural, horticultural or forestry utilization leads to a real solution in disposal of sewage sludges. Therefore - whenever justifiable - the principle should be kept

"Recycling first !"

With other words: Sewage sludges, the composition of which is suitable, should be integrated into the natural recycling.. This way - as the optimal solution - is of great importance for small and mediumsized purification plants, all the more, if the technological problems and the financially higher costs for other solutions are considered; especially for small purification plants.

The specific demand of energy at different methods for sewage sludge disposal (land disposal, disposal on sea / in the ocean, disposal by land dumping grounds (with dewatering to 30 % DS), sludge incineration (with dewatering to 35 % DS)) was determined by REED [2] (look at table 1). Such as it is the principle for the procedures of the sewage treatment [3], it is valid here too: procedures/methods with a little specific consumption of energy require large areas for the disposal of sewage sludges, methods with a small demand of area presuppose a high demand of energy.

1.3 Critical Substances and Critical Effects of Sewage Sludges, which prevent or limit their Use or their Utilization

This utilization must not be prevented by too many critical substances in the sludges or by other possible critical effects. Such negative effects may be a result of

- the transfer of noxious substances with critical concentrations in to sludge-amended soils and into the food-chain.

  or

  epidemicly-hygienicly aspects.

It should be taken notice of the fact, that soils cannot be purified like water or air, but that the output of potential noxious substances out of soils is only very little. Therefore it is to be taken care, that also longsighted there will be no noxious concentration of those substances in the soils. At present in the FRG the following 7 heavy metals are considered to be potential noxious substances, although a great part of them is indispensable as trace-elements: they are lead, cadmium, chromium, copper, mercury, nickel and zinc.

Quite recently also cadmium has got the quality of a trace-element. The potential noxious effect of those non-iron metals is caused by their higher concentration, above all

a) by the phytotoxicity of metals as zinc, copper and nickel; in the UK the phytotoxicity of the non-iron metals is expressed by the values of zinc equivalents [4];

b) by the bioaccumulation of metals in the food-chain, which could mean a risk for health of human beings, for instance cadmium.

2. New Development in the FRG

In fact land disposal of sewage sludges is basically fixed in the so-called "Abfallbeseitigungsgesetz" (Act for waste disposal) in FRG since 1972 [5]. On January, 15th, 1982, "Bundestag" (the first German Parliament) decided unanimously the modification of this law [6]. The ordinance provided for by this law in § 15 (2) will come into force on April, 1st, 1983 [7]. Judical questions on the one hand and technical problems on the other hand required an intensive and therefore longtime preparation of this ordinance.

This ordinance lays down the generally allowed concentrations of the 7 heavy metals (lead, cadmium, chromium, copper, mercury, nickel and zinc) in the sewage sludges and in the soils for land disposal of sewage sludges (look at table 2). This ordinance, moreover, assumes the regulations of the Mandatory Code of Practice M 7 and tightens them up in some points. 1972 the former "Zentralstelle für Abfallbeseitigung beim Bundesgesundheitsamt" (Central Agency for Waste Disposal at the German Minister of Health) published together with LAWA (Board of the German Federal States for Water Affairs) this Mandatory Code of Practice M 7 : "Treatment and disposal of sewage sludges with special regard to their epidemic and hygienic unnoxiousness in agricultural use" [8]. But this Mandatory Code of Practice only considered hygienic and sanitary aspects and not the toxicity of the contents of the sludges, because these problems were still unknown at that time. This Mandatory Code of Practice M 7 contains already relatively exact recommendations for land disposal of sewage sludges with regard to their epidemic and hygienic hazardlessness in agricultural use and in agriculture in general (look at Fig. 1a).

Basing on the regulations in the Mandatory Code of Practice M 7 [8] and on other instruction sheets and information-papers [9, 10, 11] it was possible in the Federal Republic of Germany up to now to dispose 35 - 40% of sewage sludge quantities to agricultural soil (with a quantity of about 38 Mio. m^3 raw sludge p.a. After their aerobic and anaerobic stabilization remain 26 to 30 Mio. m^3 p.a.). The part of sewage sludges used in land disposal is very different within the several regions of the FRG. In the

area of the Niers-River-Association still 100 %, in the area of the Great-Erft-River-Association more than 95 %, in Bavaria about 70 %. in Hesse about 50 - 55 % and in North-Rhine-Westfalia all together 30 - 35 % of the quantities of sewagesludges are used for agricultural purposes.

It is especially difficult to find a suitable parameter to judge the endangering of human beings' and animals' health by transfer of potential noxious substances in the chain waste-water ⇒ sewage-sludge ⇒ soil ⇒ plant ⇒ animal ⇒ humans because of technical problems. To this time in Germany the following heavy metals are considered to be potential critical noxious substances: lead, cadmium, chromium, copper, mercury, nickel and zinc.

Principally the toxical effect of heavy metals is already known for a long time. Up to the last few years, however, the critical parameter was mainly the negative influence on biological processes in the field of the clarification plants, the sewage purification and the sludge treatment. The toxical influence of these heavy metals is of great importance for the methan-fermentation in the anaerobic stabilization process by digestion. In the clarification plants the critical parameters of the pollution load of heavy metals were at first determined by this influence. It is only known recently, that a far more severe criterion for the evaluation of the content of heavy metals in sewage sludges is deduced by their possible transfer into the food-chain soil ⇒ plant ⇒ animal ⇒ humans [9, 10, 11, 12].

This criterion helped to deduce the generally allowable concentration of several heavy metals in sewage sludges for land disposal [7].

It is of high importance - even if the knowledge is still limited nowadays - to observe the mechanism and the properties of heavy metals for a longer period of time. A relevant parameter for this longtime observation can only be the fact, that the pollution load of the sewage sludges which are applied [13] is decisive for the addition of noxious substances in the soil.

The result of those new considerations is compiled in the so-called "Klärschlamm-Verordnung" (ordinance for the application of sewage sludges on agricultural soils).

5 years after the "Klärschlamm-Verordnung" has come into force there will be a revision of the defined generally maximum allowable concentration of noxious substances in sewage sludge and in the soil. The latest scientific findings will be checked before the experts will decide whether and to which extent it will be possible to fix the mentioned values of concentration or to which content it will be necessary to change these values.

3. Future Practice in Land Disposal of Sewage Sludges in the FRG

The new "Klärschlamm-Verordnung" settles on the fact that sewage sludges are used as far as possible on agricultural areas but also makes safe at the same time that any harm to either human beings or animals, which is caused by food produced on those areas should be avoided.

In this connection it is very important, that the problem of heavy metals in sewage sludges is no more to be solved inside the clarification plants but only before them. In other words: the discharging of wastewater, which is loaded with heavy metals into the sewers should be already stopped or diminished as far as possible [14, 15, 16]. This is especially the possibility for waste water, which is loaded with heavy metals and which is of industrial origin. A special research which was made for several years in the Ruhr-area [17], demonstrated - and not only here -, that on the whole chromium, nickel and cadmium had mainly industrial origin, and to a large extent copper and zinc as well. Also surface-water running off, storm-water and sewage out of diffuse origin which is immitted into the clarification plants, contain essential amounts of heavy metals. Especially that is the fact for lead [18, 19].

3.1 Regulations and Appointments in the new Ordinance for the Application of Sewage Sludge on agricultural Soils "Klärschlamm-Verordnung"

In order to realize a further disposal of sewage sludge with respect to the practice as far as possible the new ordinance for the application of sewage sludges on agricultural soils lays down:

A. The ordinance contains definitions for raw sludge, epidemicly and hygienicly hazardless sewage sludge and for epidemicly hygienicly hazardous sludge:

Raw sludge:	Sewage sludge taken out of clarification plants without preceding treatment. Sludge out of small clarification plants for waste water of domestic origin only are not concerned by the definition of raw sludge of the ordinance.
Epidemicly-hygienicly hazardless sewage sludge:	These sludges must have been treated by chemical or thermical conditioning, heating, composting, chemical stabilization or by another process in order to eliminate pathogenic germs.
Epidemicly-hygienicly hazardous sewage sludge:	Sewage sludges which are not treated by any of these processes are not epidemicly-hygienicly hazardless.
Potatoes:	Potatoes are not vegetable but root-crops

B. It is forbidden to apply raw sludge on agricultural, forestry or horticultural areas.

C. It is also forbidden without making any exception to apply sewage sludge to areas of vegetable and orchards.

D. Treated sewage sludges (with exception of raw sludges) can only be applied to agricultural, forestry, or horticultural areas in a limited quantity and under control.

E. Sewage sludges can only be applied to agricultural or horticultural soil, if the concentration of potential noxious substances of those sludges does not exceed the defined generally allowable and controlled limit values.

In the normal case the content of the following heavy metals in the average sample should not be more than for

lead	1 200 mg/kg DS
cadmium	20 " " "
chromium	1 200 " " "
copper	1 200 " " "
nickel	200 " " "
mercury	25 " " "
zinc	3 000 " " "

Even in case only one value is exceeded, it will be necessary to get a permission for the land disposal of this sludge by the authorities.

F. Moreover the sewage sludges have to be disinfected or disposed in agriculture in that way - but only to that extent - that no epidemic an hygienic danger will occur to either humans or animals (look at Fig. 1 b).

After December, 31st, 1986,it is only allowed to dispose epidemicly-hygienicly hazardless sewage sludge to grassland and forage areas. For these areas a temporary arrangement until 31 December, 1986, becomes effective: Up to this date it is allowed to dispose also epidemicly-hygienicly hazardous sludge to grassland an forage areas, but not during the period of vegetation. Up to the 31 December, 1986, it is therefore allowed to dispose those epidemicly-hygienicly hazardous sludges to grassland and forage areas still in the period of time from the end of the utilization or crop until to the end of the year.

These new regulations modify the instructions of the Mandatory Code of Practice M7; but moreover the regulations of this sheet are still authentic (look at Fig. 1 b).

G. Furthermore only those sewage sludges may be applied to agricultural soil which have not more than the generally allowable controlled limit values of noxious substances.

The application of sewage sludges (with an sufficient quality) to soils used for agricultural or horticultural purposes is not permitted, if the analysis of the soil shows, that the content of even one of the following metals is exceeding the special value in the average sample

lead	100	mg/kg	air	dried	soil	
cadmium	3	"	"	"	"	"
chromium	100	"	"	"	"	"
copper	100	"	"	"	"	"
nickel	50	"	"	"	"	"
mercury	2	"	"	"	"	"
zinc	300	"	"	"	"	"

In particular cases the administration can make an exception for the heavy metals lead, chromium, copper, nickel and zinc, if there will be no injury of the general public and no danger for the health of humans and animals.

H. Before applying sewage sludge to agricultural or horticultural soil an examination concerning the concentration of potential noxious substances in the sewage sludges and in the soils as well has to be made, moreover a further examination of those sewage sludges with respect to their contents of other substances as nitrogen, phosphate, potassium, calcium, magnesium has to be made. Only clarification plants to an extent of a population equivalent less than 5 000 inhabitants are excepted of this duty of examination, as far as in these clarification plants only waste water of domestic origin or another waste water with a low load of potential noxious substances is treated.

I. The sewage sludges researched in this way have to be declared by a bill of delivery with the actual result of the research. As an information for the applying farmer he shall be handed over a copy of this bill of delivery, when the sewage sludge is delivered. A duplicate of these bill of delivery has to be preserved for 5 years and has to be presented to the administration if wanted.

K. After the 31 December, 1986, it is only allowed to dispose epidemicly and hygienicly hazardless sewage sludge to grassland and forage-areas. For these areas a temporary arrangement until December, 31st, 1986, becomes effective. Up to this date it is allowed, to dispose also epidemicly and hygienicly hazardous sludge to grassland and forage areas, but not during the period of vegetation. Up to the 31 December, 1986, it is therefore allowed to dispose those epidemicly-hygienicly hazardous sludges to grassland and forage areas still in the period of time from the end of the utilization or crop thru the end of the year.

L. The quantity of applied sewage sludges is limited each time to 5 to dried solids /ha · 3 a or 10 to dried solids /ha · 6 a (a = year).

M. Before the application of sewage sludge a permission of the local authorities is always needed if sewage sludge shall be applied to

- soils used for forestry or in nature reserves and national parks,
- if the concentration of lead, chromium, copper, nickel or zinc in soils used for agricultural or horticultural purposes exceeds the generally allowed values; this permission must be given under premises, if only one value exceeds the critical concentration of noxious substances,
- if the concentration of one of the heavy metals in sludges which shall be applied to agricultural soils exceeds the generally allowed values.

4. Alternatives in Case of Limitation of the Land Disposal of Sewage Sludges: Controlled Tipping with or without preceding Incineration of the Sewage Sludges

As far as a further land disposal of sewage sludges will be dropped because of general prohibitions or limitations, either because of their pollution load or because of the lack of application areas or other reasons, it will be absolutely necessary to find another way to dispose sewage sludges by dumping, either with or without preceding incineration. The consequences for the operators of clarification plants will be: costs for investments , for working expenses and maintenance, because then an equipment will be needed for sludge conditioning, sludge dewatering, and possibly sludge drying and incineration. Moreover costs will arise by tipping the sludge or the ashes, presupposed, that areas for tipping grounds have been provided. In general for larger clarification plants it will be only possible to dispose the accruing sewage sludges by controlled tipping because of the lack of suitable and sufficient agricultural areas.

4.1 Critical Properties of Sewage Sludges for their Controlled Tipping

The controlled tipping of sewage sludges may cause detriments

by the emission of odor nuisances by the decomposition of putrescible organic substances, which form a high percentage in unconditioned sewage sludges

or

by inhibiting effects in controlled tipping because of a too high water content.

In general in the FRG often a minimum content of dry solids of 35 % is wanted for the disposal of sewage sludges in combined dumping grounds.

4.2 The Treatment of Sewage Sludges as a Pre-condition for their Disposal

Because of their critical properties it is not possible to dispose sewage sludges directly as raw sludges, but only after a special treatment. - Therefore, sewage sludge treatment first has to make disposable at all the accruing sewage sludges, and as to the procedure, the extent and the result, sludge treatment has to act in conformity to the requests of the disposal just put to use.

4.3 Tasks and Aims of Sludge Treatment

The preceding treatment of sewage sludges before their disposal aims at an alteration of their critical properties, in order to avoid or to prevent the above mentioned undesirable effects. Therefore, corresponding with the requirements of the sludge disposal the following technical aims are the result of sludge treatment in clarification plants

a) the aerobic or anaerobic stabilization
(by extended aeration or digesting,
also by composting or incineration):
By reduction of the content of organic substances by biological or thermical processes the susceptibility of these sludges to putrefactive bacterias decreases and consequently also the influence of molestance/load for the environments, that is air, soil, ground- or surface water.
Hereby occurs a decomposition of substances, which possibly leads to loads inside the clarification plants. The thermical aerobic stabilization by incineration mineralizes entirely all organic substances of the sludge content = maximum stabilization.

b) reduction of the water content
(by thickening, dewatering and/or drying including conditioning (to improve the capability of the sewage sludges in the thickening and watering process)):
reduces the sludge capacity and makes the further treatment easier and cheaper.
The seperated sludge water has to be passed back into the clarification plants because of the - though very different - substances of its content.

c) disinfection if necessary, but then only to the really required extent.
(either thermical, that is by influence of heat, by irradiating or by a sufficient change of the pH-value; thermical by heating with temperatures of at least 65° C and an influencing time of at least 30 minutes by pasteurisation, heat drying or thermical conditioning; thermical by sufficient self heating with the help of correct composting or by addition of quick lime):
In case hygienic reasons are important for the further disposal or utilization of the sludge, a preventive disinfection is necessary in order to eliminate pathogenic germs or to diminish them.

4.4 Basic Operations of Sludge Treatment

Those aims at the same time correspond with the basic operations of sludge treatment. Fig. 2 gives a general view about the procedures resultant from the fundamental tasks above mentioned.

All procedures and methods for treatment and disposal of sludge have to take in consideration these basic operations. The order of succession and the gradation allow most varied solutions and possibilities as it is shown in the scheme of Fig. 2 .

There are still additional combinations with different gradations in the single divisions exceeding the scheme above shown. Some procedures at the same time lead to a stabilization and a decrease of volume, for instance in case of incineration and composting. Before deciding which

dewatering system provided for is to be chosen, the succeeding procedures must be known, that means, all further steps to final disposal have to be fixed. It depends on this fact, which final water content respectively which final content can be reached in the dewatering step. This question is important for the predecision.

5. General Trends in Treatment Technologies of Sewage Sludges

It is the task of sludge treatment to make disposable the accruing sewage sludges.

The land disposal of sewage sludges requires a low content of heavy metals and also epidemic-hygienic aspects have to be considered. Furthermore there is a prohibition for land disposal of raw sludges and moreover the limitation of the application quantities favours the distribution of sludges as wet sludges. In case sewage sludges are to be used in land disposal a aerobic or anaerobic stabilization has to be made, and an additional disinfection by pasteurization is necessary, if the sludge is to be applied to pasture land or forage areas. This can be done by pre-pasteurization, that means by pasteurization before the stabilization-process. The heavy metals have to be eliminated before the waste water is discharged into the sewers.

It is a condition for controlled tipping of sewage sludges, that no odor-nuisances arise, and that the sludges are sufficiently thickenend and dewatered. For this purpose it is necessary that the sludges are sufficiently stabilized - aerobicly or anaerobicly - thickened, and in general conditioned and mechanicly dewatered by centrifuges, belt presses or filter presses.

The dewatering by centrifuges and belt-presses leads to an average content of solids of 25 - 30 %, by filter-presses to an average content of solids of 40 - 45 % including the conditioning agents. Considering the higher expenses a drying or incineration of the sludges is only used in special cases.

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Table 1: Energy Demand at different Methods of Disposal (according to REED)

Method of Disposal	Operational Power/Fuel [kWh/ton] *)	Energy Equival. for Structures, Machines etc. [kWh/ton] *)	Total Energy Required [kWh/ton] *)
Sea	2.98	1.34	4.32
Liquid to Land (7.5 % d.s.)	0.81	1.22	2.03
Incineration (35 % d.s.)	8.00	11.00	19.00
Cake to Land (30 % d.s.)	3.21	2.65	5.86

*) Per Ton of Liquid Sludge at 3 % Dry Solids

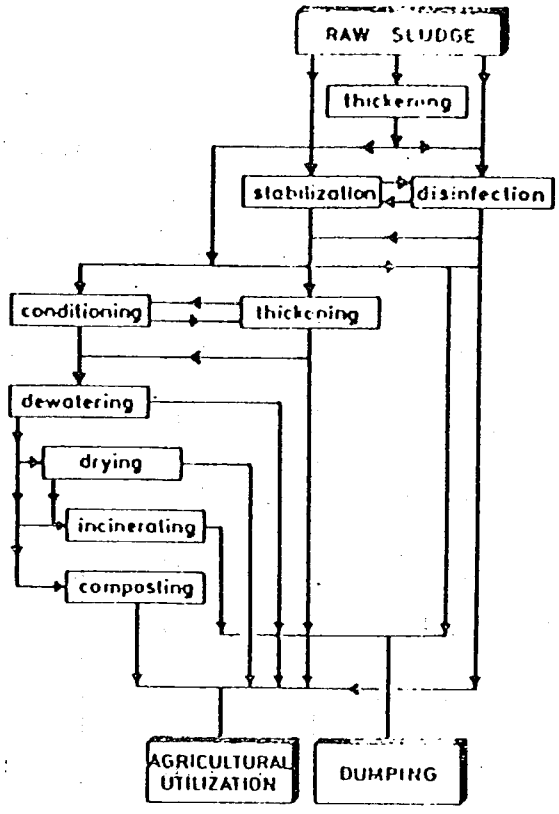


Fig. 1: Possibilities of Sewage Sludge Treatment

Table 2: Standards for Agricultural Use of Sewage Sludge in the FRG

(effective from 1 April, 1983)

[mg Heavy Metals per kg Dry Solids]

	Sewage Sludge	Soil
Lead	1,200	100
Cadmium	20	3
Chromium	1,200	100
Copper	1,200	100
Nickel	200	50
Mercury	25	2
Zinc	3,000	300

Epdemicly and hygienicly hazardless land disposal of sewage sludge

		I	II		III	
		Alternative Areas	Arable Land		Pasture and Forage Area	
			VR	V	VR	V
5.1.1. Raw Sludge	hz	+ (limited)	–	–	–	–
5.1.2. Sludge of small plants and earth pits	hz	+	+	–	–	–
5.2.1. liquid) anaerobically or aerobically stabilized sludge	hz	+	+	–	+	–
5.2.2. dewatered)	hz					
5.2.3. naturally dried)	hz					
5.3.1. chemically) conditioned sludge	hz					
5.3.2. thermically)						
5.4. thermically dried sludge		+	+		+	
5.5. heated wet-sludge (raw, stabilized)						
5.6. composted sludge						

hz = hygienicly hazardous
VR = period of vegetation rest
V = period of vegetation

+ = application allowed
– = application not allowed

Fig. 1a: Recommendations for epidemicly and hygienicly hazardless land disposal of sewage sludges by the instructions of the Mandatory Code of Practice M 7 [4]

Epidemicly and hygienicly hazardless land disposal of sewage sludge

		IV Viticulture Areas VR	V Orchards without subculture VR	VI Tree Nurseries	VII Gardening and Vegetable Gardening
5.1.1. Raw Sludge	hz	–	–	–	–
5.1.2. Sludge of small plants and earth pits	hz	–	–	+	–
5.2.1. liquid) anaerobically or aerobically stabilized sludge	hz	+	+	+	–
5.2.2. dewatered)	hz				
5.2.3. naturally dried)	hz				
5.3.1. chemically) conditioned sludge	hz				
5.3.2. thermically)					
5.4. thermically dried sludge		+	+	+	+
5.5. heated wet-sludge (raw, stabilized)					
5.6. composted sludge					

hz = hygienicly hazardous
VR = period of vegetation rest
V = period of vegetation

\+ = application allowed
– = application not allowed

Fig. 1a: Recommendations for epidemicly and hygienicly hazardless land disposal of sewage sludges by the instructions of the Mandatory Code of Practice M 7 [4]

Epidemicly and hygienicly hazardless land disposal of sewage sludge
by the ordinance for disposal of sewage sludge

		I Alternative Areas	II Arable Land		III Grassland, Pasture and Forage Areas	
			VR	V	VR	V
5.1.1. Raw sludge	hz	+ (limited)	–	–	–	–
5.1.2. Sludge of small plants and earth pits	hz	+	+	–	–	–
5.2.1. liquid) anaerobic-	hz					
5.2.2. dewatered) ly or aerobicly	hz	+	+	–	–[1]	–
5.2.3. naturally dried) stabilized sludge	hz				+[2]	
5.3.1. chemically) conditioned						
5.3.2. thermically) sludge						
5.4. thermically dried sludge		+	+		+	
5.5. heated sludge (raw, stabilized)						
5.6. composted sludge						
5.7. chemically stabilized sludge						

hz = hygienicly hazardous
VR = period of vegetation rest
V = period of vegetation

\+ = application allowed
– = application not allowed

1) After December, 31st, 1986
2) only until December, 31st, 1986

Fig. 1b: Regulations for epidemicly and hygienicly hazardless land disposal of sewage sludge by the ordinance for disposal of sewage sludge ("Klärschlamm-Verordnung'

Epidemicly and hygienicly hazardless land disposal of sewage sludge
by the ordinance for disposal of sewage sludge

		IV Viticulture Areas VR	V Orchards VR	VI Forestry Areas	VII Gardening and Vegetable Areas
5.1.1. Raw sludge	hz	–	–	–	–
5.1.2. Sludge of small plants and earth pits	hz	–	–	+[3]	–
5.2.1. liquid) anaerobic-	hz				
5.2.2. dewatered) ly or aerobicly	hz	+	–	+[3]	–
5.2.3. naturally dried) stabilized sludge	hz				
5.3.1. chemically) conditioned					
5.3.2. thermically) sludge					
5.4. thermically dried sludge		+	–	+[3]	–
5.5. heated sludge (raw, stabilized)					
5.6. composted sludge					
5.7. chemically stabilized sludge					

hz = hygienicly hazardous
VR = period of vegetation rest
V = period of vegetation

\+ = application allowed
– = application not allowed

[3] only with special permission

Fig. 1b: Regulations for epidemicly and hygienicly hazardless land disposal of sewage sludge by the ordinance for disposal of sewage sludge ("Klärschlamm-Verordnung"

PRESENT STATUS OF SEWAGE SLUDGE TREATMENT TECHNOLOGY AND DISPOSAL IN JAPAN

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1. INTRODUCTION

Sludge generated in sewage treatment is characterized by high water content, high organic matter content, and offensive odour. In Japan where scrupulous care of impact on the environment is required owing to the small area of land, it is becoming more and more difficult to ensure disposal sites for sludge. The spread of sewer systems in the future inevitably will lead to an increasing amount of sludge, resulting in serious problems in disposal.

An important problem which sewerage works in Japan confront is the provision of appropriate treatment of sludge and ensuring pertinent disposal methods. Sludge has to be treated properly according to its disposal conditions. It can be said that the extent of sludge treatment has advanced, keeping pace with the change in the conditions of disposal sites, just like the degree of sewage treatment has been improved remarkably according to the conditions of the place where waste water is discharged.

In this paper, the present status and problems of sludge disposal in Japan and the present situation of sludge treatment technology will be discussed.

2. SLUDGE DISPOSAL

2-1 Present Status of Sludge Disposal

Rising percentage of sewered population has resulted in an increase in the amount of sludge generated in wastewater treatment processes. In terms of thickened sludge with 97% water content, the total amount grew from 29 x $10^6 m^3$/year in 1978 (26% sewered population) to 34 x $10^6 m^3$/year in 1981 (30% sewered population). Since this sludge is treated in a variety of ways to reduce its volume and to dispose of it, the actual volume disposed is smaller than these values. The final forms of sludge disposed include thickened sludge, digested sludge, dewatered sludge, incinerated ash, dried sludge and composted sludge. Table 1 illustrates the statistical values of disposed sludge volume by form and disposal site in 1980. The annual trend of disposed sludge volume and the forms of sludge are shown in Figure 1.

As shown in the figure, the total amount of disposed sludge has remained the same, at around 2.4 x $10^6 m^3$, over the last several years. This is because of the development of sludge treatment, for instance, increasing percentage of incineration. The amount of incinerated sludge among thickened sludge reached 49% in 1981.

Figure 2 depicts the annual change of sludge by disposal site. Although utilization (mainly for farm land and green area) has increased gradually, about 80% of the total has been disposed of by land-fill or land reclamation.

Table 1 Statistic of Sludge Disposal[1] (Nov. 1980 ~ Oct. 1981)

(Thousand m^3)

Sludge condition \ Form	Land-fill	Land Reclamation	Utilization	Ocean	Total
Dewatered Sludge	850	765	296	13	1,924
Incinerated Ash	131	103	33	0	267
Dried Sludge	13	0	8	0	21
Digested Sludge	5	0	16	164	185
Total (%)	999 (46)	868 (36)	353 (15)	177 (7)	2,397 (100)

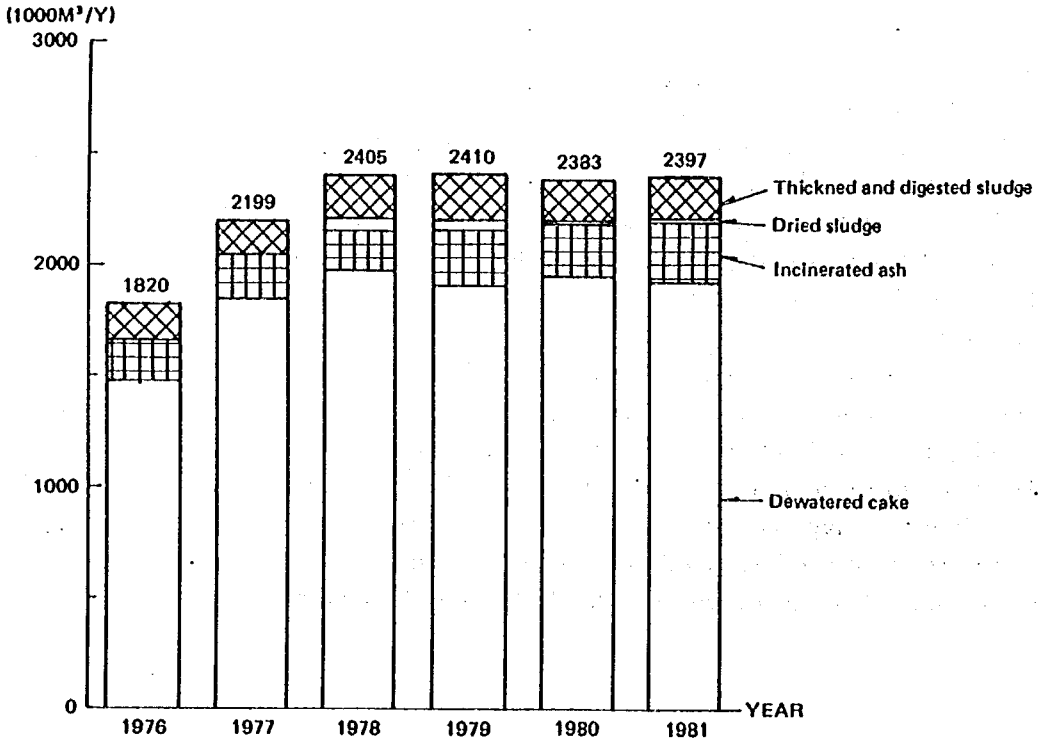


Fig. 1 Form of Disposed Sludge[1]

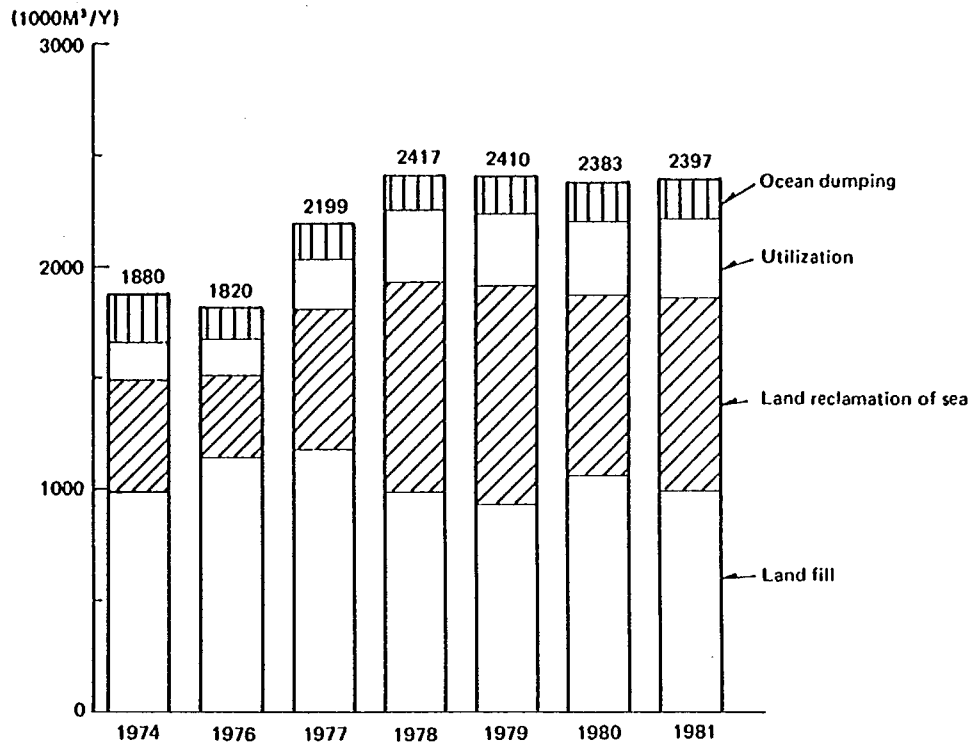


Fig. 2 Sludge Disposal Site[1)]

2-2 Problems of Sludge Disposal

As already mentioned, some 80% of sludge is disposed of by land-fill or land reclamation, but the disposal capacity of these sites is limited. For example, in Tokyo and many of the surrounding municipalities, available disposal sites will be full within 5 years as shown in Table 2. Thus, the prospect of filling up existing disposal sites suggests the necessity of new sites. With respect to land-fill, ensuring disposal sites is becoming more difficult due to the following reasons;

Table 2 Capacity (years and volume) of Existing Disposal Site[2)]

Item \ Years	0–5	5–10	10–15	15–20	>20	uncertain	Total
Number of municipalities	22	5	1	0	0	4	32
Capacity (Volume) (1,000 m³)	7,040	300	20	0	0	–	7,360

(a) Residents living near proposed disposal sites might oppose them for fear of possible environmental and public health problems (for instance, offensive odour, rank weeds, flies and flying incinerated ash).

(b) The spread of urbanized areas has made it more difficult to obtain disposal sites.

As a result, several cities which have great difficulties with land-fill within their own administrative districts transport sludge to sites more than 200 km away (Figure 3).

Therefore, from a long-term viewpoint, recycling utilization of perpetual disposal type, namely for use in agriculture, as construction materials, etc. must be facilitated. However, it may take a long time to practice these measures and also technical and economic problems have to be solved.

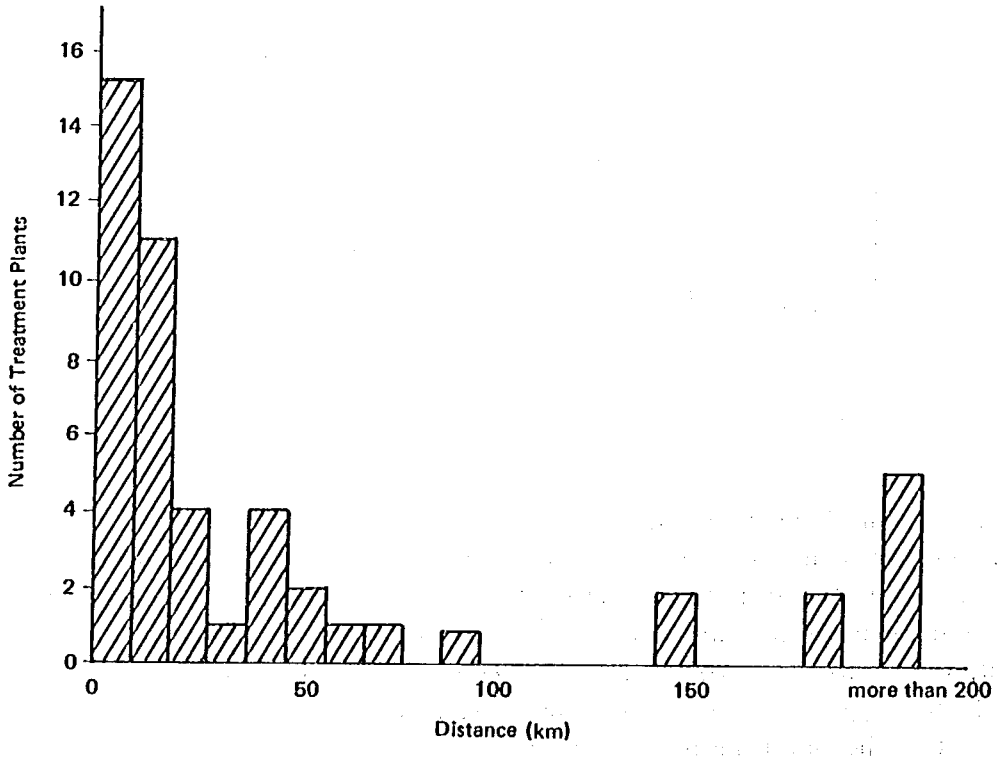


Fig. 3 Distances from Treatment Plants to Disposal Sites[2)]

Consequently, sludge disposal may have to depend on large scale land reclamation of the sea just as other urban refuse. Currently, sludge disposal generated in Greater Tokyo Area and Greater Osaka Area is under examination in the framework of the Phoenix Project.

The Phoenix Project aims at large scale land reclamation of the sea by municipal refuse generated in the great urban areas and industrial waste.

Since this land reclamation might cause water pollution problems by leachate water, an investigation is now being conducted on the characteristics of the leachate from various sludge.

3. SLUDGE TREATMENT TECHNOLOGY

3-1 Present Status of Sludge Treatment Processes

According to "Statistic Data of Sewage Works in Japan 1979"[3], the number of sewage treatment plants in Japan now under operation is 437. Sludge treatment methods applied in these plants are categorized in Table 3.

Since one plant with two sludge treatment methods is counted double, the number of treatment plants in this table exceeds the actual plant number.

This table indicates that "thickening - anaerobic digestion - dewatering" and "thickening-dewatering" are the methods applied most. However, as to capacity (based on thickened sludge), incineration has been applied more and more. In particular, many large scale treatment plants in big cities have used incineration out of necessity to reduce sludge volume.

Table 3 Applied Sludge Treatment Processes[3]

(1979)

	Sludge Treatment Process	Number of Treatment Plants	Capacity (1000 m^3/d)
1	Thickening — Anaerobic Digestion — Dewatering	116	109
2	Thickening ——— Dewatering	99	62
3	Thickening ——— Dewatering — Incineration	53	86
4	Thickening — Aerobic Digestion — Dewatering	16	2
5	Thickening — Anaerobic Digestion — Dewatering — Incineration	15	81
6	Thickening	14	2
7	——— Anaerobic Digestion — Dewatering	14	2
8	Transport to other Treatment Plant	39	–
9	No. Sludge Production	6	–
10	Others	78	57
	Total	447	401

3-2 Thickening

After the sludge is taken from waste water treatment processes, first it is thickened. Thickening is carried out by the gravity thickener method in many cases. In general, the thickener is designed to decrease solid concentration of feed sludge from 1% to approximately 4%. This was considered reasonable in view of past experiences, but presently it has become very difficult to thicken sludge to 4%.

For example, according to a survey conducted by the Tokyo Metropolitan Government shown in Figure 4, the concentration has gone down from 4 to 5% in the past to less than 3% at present. Because the concentration remarkably affects heat added to the digester and energy consumption in the dewatering process, improvement of thickening is required. A report released by the Sewage Works Bureau of the Tokyo Metropolitan Government indicates that prior dilution of sludge can improve thickenability of sludge.

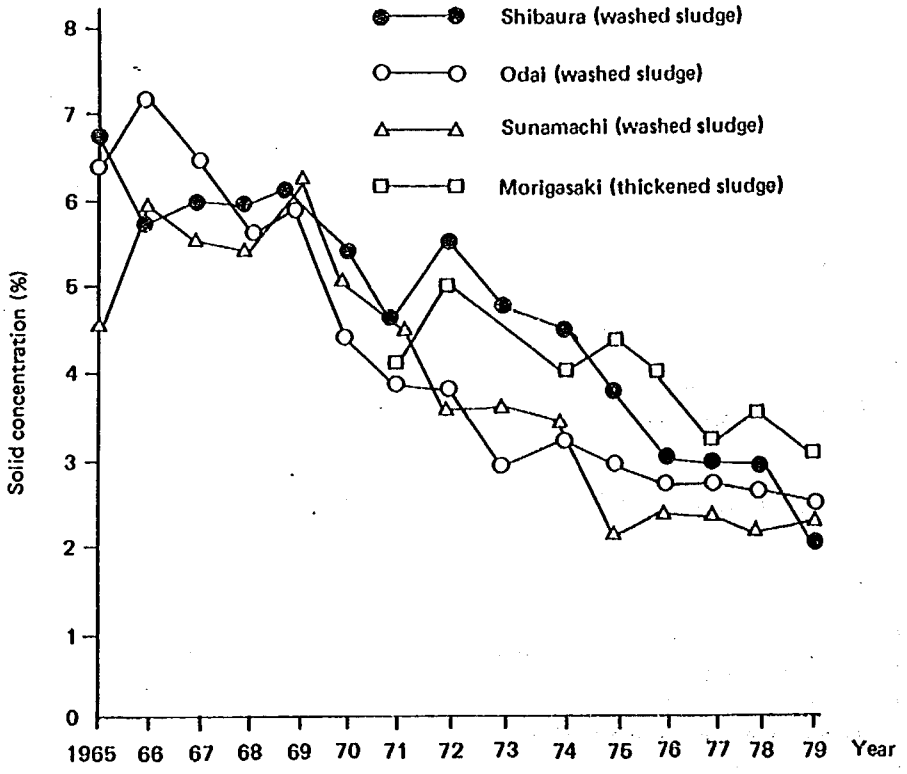


Fig. 4 Annual Change of Solid Concentration in Sludge[4]

However, the thickening of sludge is arrived at through a complex linkage of physical, chemical and biological factors such as specific gravity, particle size, dissolved oxygen and microorganisms, and so its mechanism can not be explained by a simple model. Accordingly, PWRI of the Ministry of Construction has launched a study to clarify these factors for improvement of the gravity thickening.

Because of degradation of the thickenability, currently, flotation thickening and centrifugal thickening are being put forward vigorously. The statistics of 1979 shows that 360 treatment plants with only gravity thickening were under operation, while 12 plants with flotation thickening and 1 plant with centrifugal thickening operated.

The pressurized flotation method has two advantages; ① high and steady concentration of thickened sludge, and ② high solid recovery rate. On the country, its disadvantages are; ① high operation cost, ② limitation of the concentration of thickened sludge, and ③ difficulty in operating intermittently. Although ② and ③, which concern the principle, can not be solved, ① leaves room for improvement. Namely, a higher efficiency of dissolution of air and a resultant reduction of cycling water can reduce motive power of a circulation water pump.

Since mechanical thickening is more expensive compared with gravity thickening, its cost/benefit has to be taken into consideration.

3-3 Digestion

According to the statistics of 1979, the number of plants with a digestion process amounted to 192, or 1/2 of the total. Among these 163 plants used anaerobic digestion and 29 used aerobic digestion.

Digestion can stabilize sludge and reduce its volume, but in Japan no particular attention has been paid to it as a subject of technological development. However, recently it has been attracting more attention as regards utilization of digester gas, especially practical use of electric power generation by digester gas.

The amount of digester gas in Japan is 5 to 10 times of feed sludge, or 0.3 to 0.5 m^3/kg VS in feed sludge. In plants in West Germany it is generally as much as 10 to 25 times the feed sludge. For this reason, the egg shape digestion tank in West Germany attracts more attention.

PWRI of the Ministry of Construction has been researching how to step up the amount of digestion gas generated. We have found that thermal treatment at 150°C to 180°C prior to digestion can increase the amount of gas, which was reported in West Germany.

3-4 Dewatering

The majority of treatment plants use mechanical dewatering while drying beds have been applied less.

Table 4 illustrates the number of plants which use dewatering machines by type of machine. Vacuum filters have been adopted the most frequently, but recently more plants tend to use belt filter presses using polymer. This method is advantageous because of the higher calorific value of cakes in incineration and less consumption of auxiliary fuels.

The number of dewatering machines currently ordered is shown in Table 5.

The Japan Sewage Works Agency has conducted research on the evaluation of dewatering machines.

Table 4 Installed Type of Dewatering Machines[3)]

(1979)

Type	Number of Treatment Plants
Vacuum Filtration	177
Centrifugal Dewatering	76
Recessed Plate Pressure Filters	54
Belt Filter Press	47
Screw Press	2
Others	2

Table 5 Trend of Installed Dewatering Machines[6)]

Type	1977	1978	1979	1980
Belt Filter Press	10	52	31	43
Centrifugal Dewatering	36	38	43	32
Recessed Plate Pressure Filters	23	21	26	22
Vacuum Filtration	38	17	22	9
Others	5	4	13	7

* Number of Machines Installed

3-5 Composing

Dewatered sludge, which still has water content of 60 to 80%, can not be used easily for agricultural utilization. The current trend in utilization of sewage sludge is shown in Table 6. Recently, agricultural utilization after composting has become more popular. Bagged composted sludge is on the market in some cities, but at present stage the cost is not yet recovered. Table 7 shows composting plants in Japan.

Table 6 Present Status of Sewage Sludge Utilization (Nov. 1, 1980 to Oct. 31, 1981)

(Thousand m^3)

Classification \ Sludge condition		Dewatered cake	Ash	Dried sludge	Compost	Digested sludge	Total
Utilization in agriculture and parks	By municipalities	199	0	3	5	15	222
	Selling to fertilizer companies	97	9	0	0	1	107
	Sub-total	296	9	3	5	16	329
Utilizaiton as a construction material		0	24	0	0	0	24
Total		296	33	3	5	16	353

Table 7 Composting Plants

City	Capacity	Type	Dewatering Machines	Coagulant
Tendo	5 t/d	Vertical Silo	Centrifuge	polymer
Kamaishi	2 t/d	Static Pile	Centrifuge	polymer
Yamagata	15 t/d	Horizontal Shovel	Recessed Plate Pressure Filter /	slaked lime ferric chrolide
Tokyo	10 m^3/d	Horizontal Scoop	Recessed Plate Pressure Filter	slaked lime ferric chrolide
Kagoshima	54 t/d	Vertical Silo	Recessed Plate Pressure Filter	slaked lime ferric chrolide
Okayama	0.62 m^3/d	Vertical Paddle	Belt Filter Press	polymer
Tottori	1.58 t/d	Horizontal Scoop	Vacuum Filtration	slaked lime ferric chliride
Fukuoka	?	Horizontal Shovel	Recessed Plate Pressure Filter Vacuum Filtration	slaked lime ferric chloride
Hodaka	1.5 t/d	Horizontal Auger	Vacuum Filtration	polymer

3-6 Incineration

In Japan, incineration is very frequently applied owing to the limited number of sludge disposal sites and difficulties in handling of dewatered sludge.

The statistics of 1979 show that 91 treatment plants used incineration. In Table 8 the number of plants applying incinerators by type is shown.

Table 8 Type of Incinerator

Type	Number of Treatment Plant
Multiple-Hearth Furnace	57
Fluid Bed Furnace	11
Rotary Kiln	9
Inclined Grate Furnace	7
Other	7
Total	91

Incineration requires auxiliary fuel. The main subject of technological development is how to reduce the consumption of the fuel. Though self-sustainable combustion is possible for dewatered cake with low water content and high calorific value, technology which enables self-sustainable combustion of cakes with a high water content has been developed recently.

This method is based on the control of the primary air volume and secondary air volume in a multiple-hearth furhace. In Takaoka City, sludge dewatered by a belt filter press (75% water content) is incinerated successfully without auxiliary fuel in operator free operation.

In addition, co-combustion of sludge with domestic refuse is a promising technology, because it does not require any fuel. Kanazawa City has adopted this method. Its capacity is 175 t/day, which is broken down to 150 t/day domestic refuse and 25 t/day sludge. The type of furnace is a multiple-hearth furnace. Its flowchart is shown in Figure 5.

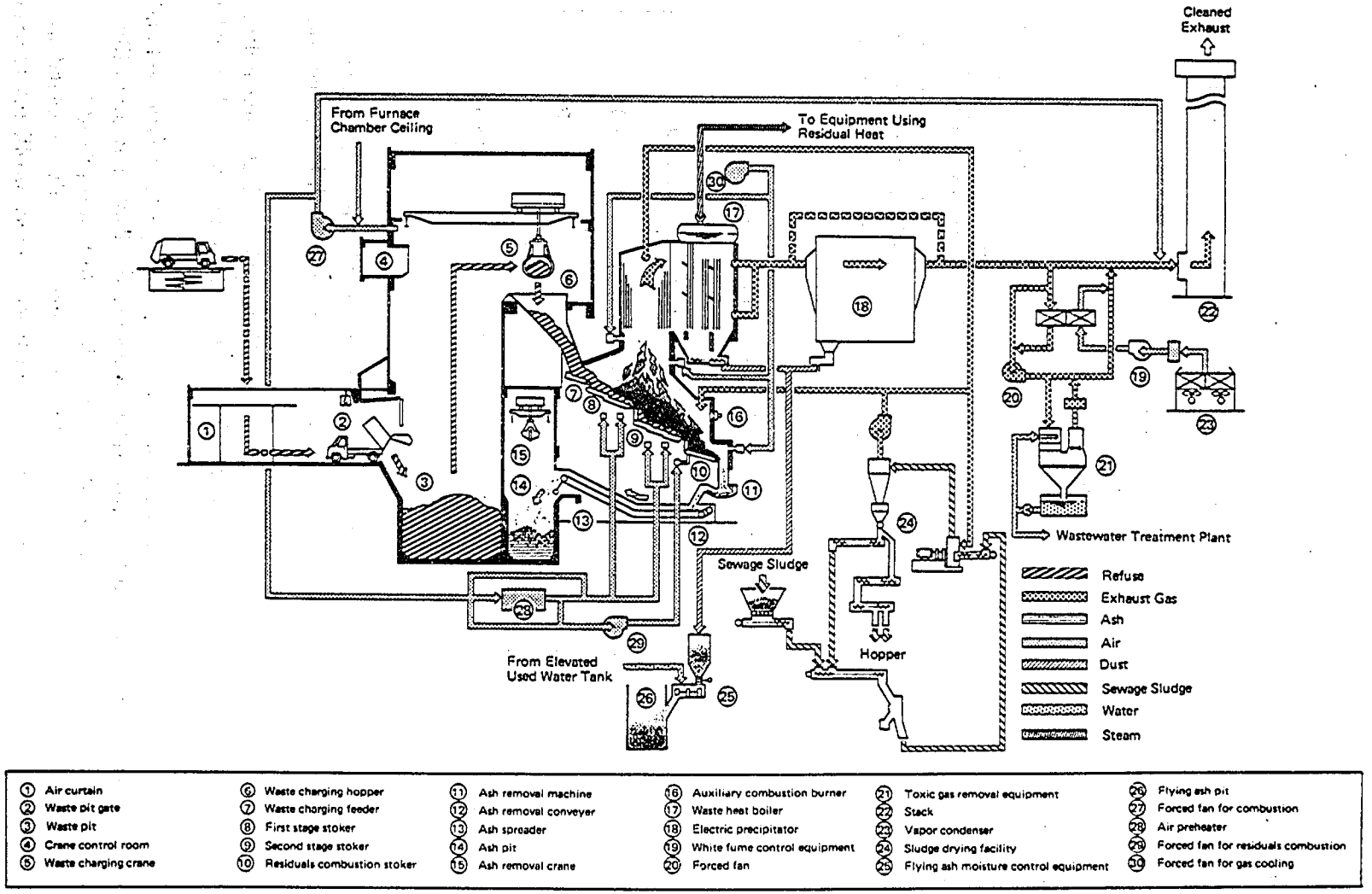


Fig. 5 Flow Chart of Co-Combustion System (Kanazawa City)

3-7 Utilization of Sludge in Construction Materials

One use of sludge is utilization in construction materials. Dewatered sludge and ash has been used for land-fill for a long time, but they have never been sold at a profit. On the contrary, they have to be disposed of at some expense in many cases.

Because of the limited space for disposal sites there is a strong trend toward using processed sludge as construction materials.

Some examples are tile, brick, and artificial light weight aggregate through baking, and fine and coarse aggregate through melting.

Among these, the manufacture of tile and brick has been successful, but these methods can not handle a large quantity of sludge.

The Tokyo Metropolitan Government has succeeded in manufacturing artificial light weight aggregate by granulizing ash from sludge incineration ash and then baking it in a multi-hearth jet furnace. The physical properties of the artificial light weight aggregate are described in Table 9. It has been reported that it is equivalent or better in quality than artificial light weight aggregate currently used with respect to strength, etc. Manufacturing at a demonstration plant (500 kg/h) from 1983 is planned.

Table 9 Characteristic of Artificial Light Weight Aggregate

Item	Test method	Measured value
Specific gravity under oven dry condition	JIS A-1134	1.44
Specific gravity of saturated subface dry aggregate	"	1.64
Absorption rate of water	"	13.9 %

Dewatered sludge or ash is smelted at 1,300°C to 1,500°C and then cooled off to yield aggregate. Rapid cooling produces fine grain aggregate, while gradual cooling produces gravelly aggregate.

The types of smelters are; electric arc furnaces (Figure 6), vertical film smelting furnaces (Figure 7), coke bed type smelters (Figure 8), reflection smelters (Figure 9) and microwave smelting furnaces (Figure 10).

By smelting and solidification, heavy metals in sludge are perfectly trapped and they are not leacheated. It is sufficiently strong as aggregate (Table 10), and utilization as a concrete aggregate or a basement material for paved road is possible.

Utilization of sludge in these construction materials is greatly expected as a measure to solve sludge disposal problems. Nevertheless, many problems remain unresolved such as the higher cost, the lack of designation of standards as aggregate, and a poor the distribution system.

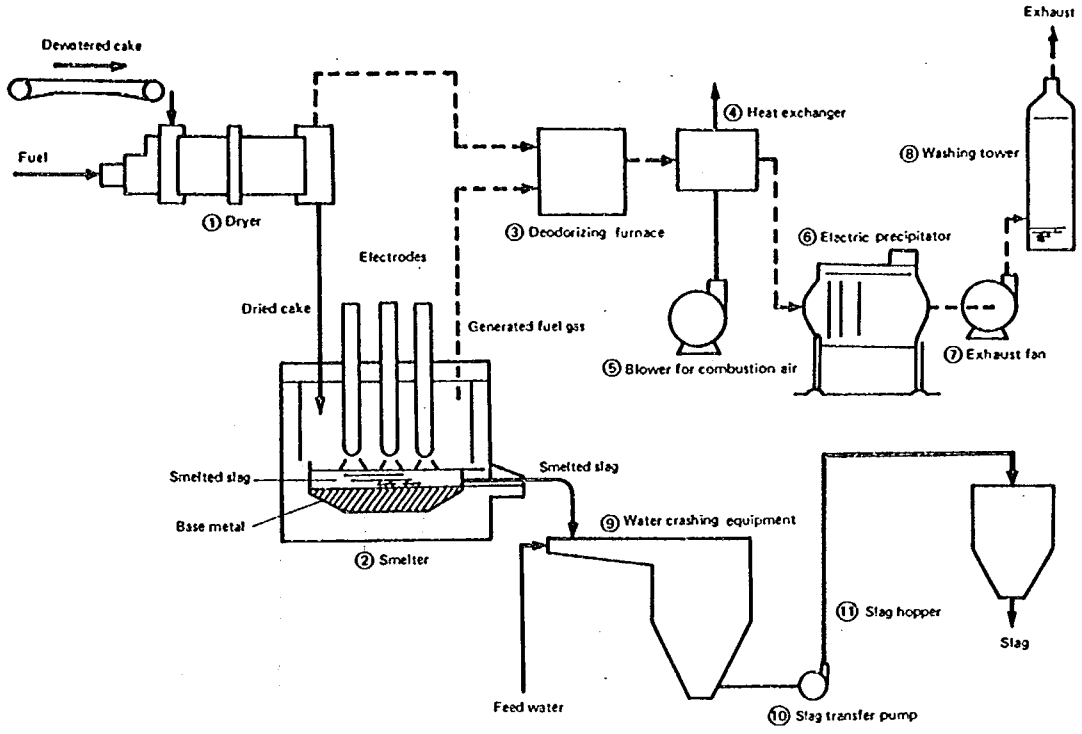


Fig. 6 Electric Arc Furnace for Melting

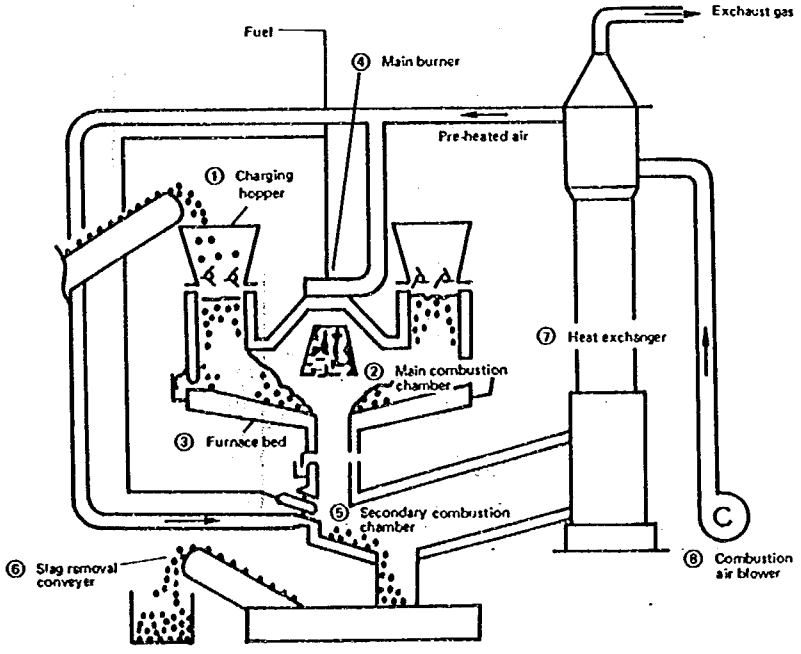


Fig. 7 Vertical Film Smelting Furnace

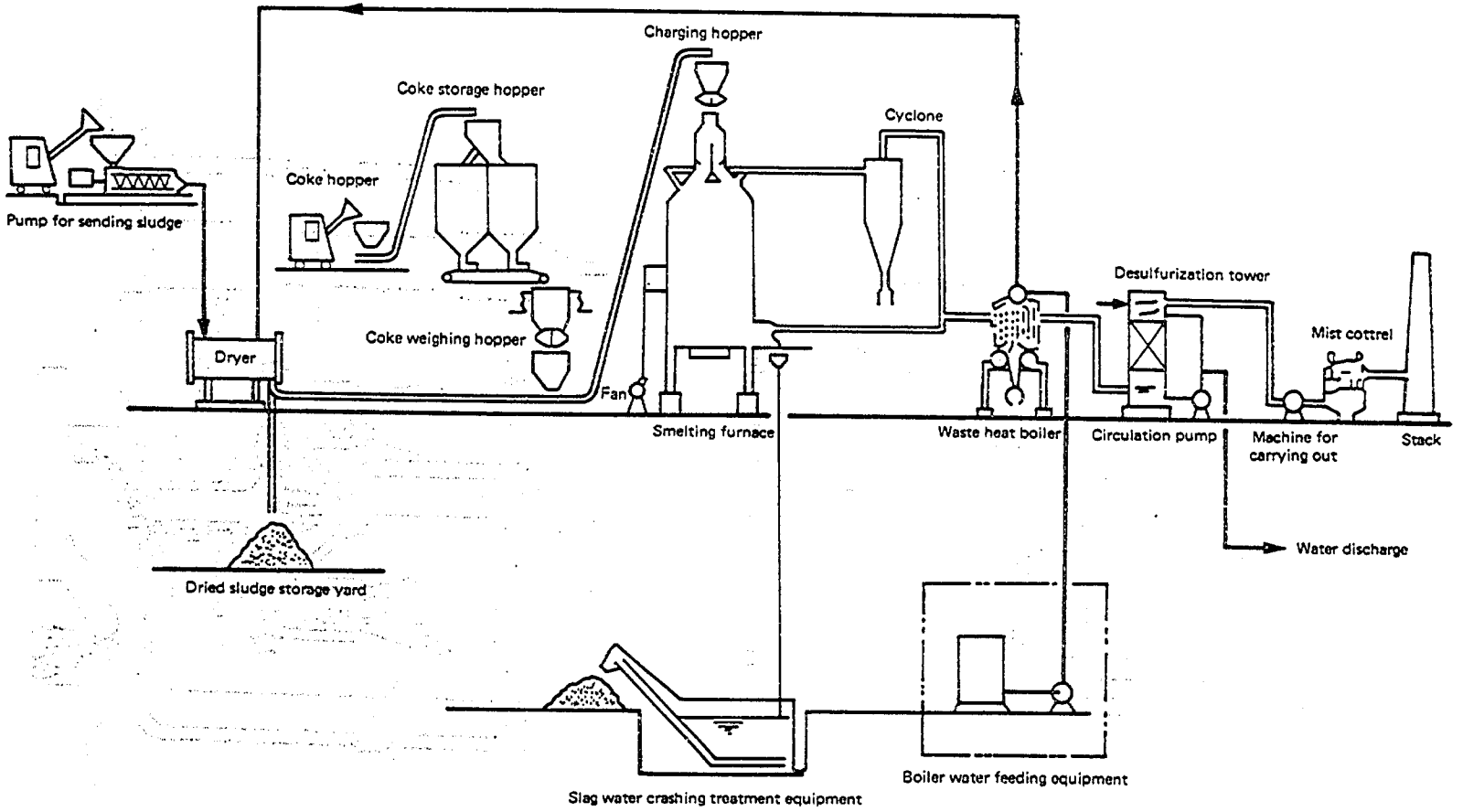


Fig. 8 Coke Bed Type Smelter

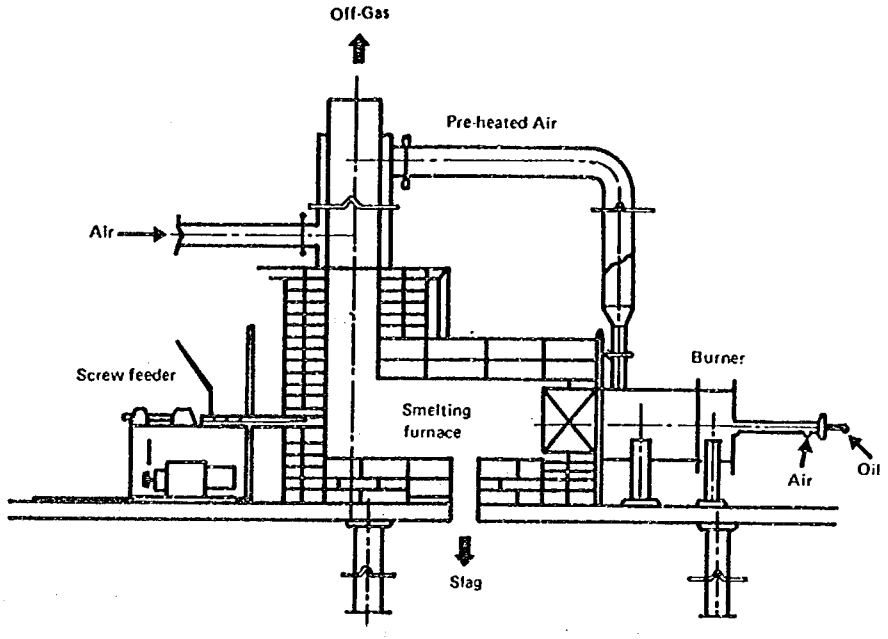


Fig. 9 Reflection Smelter

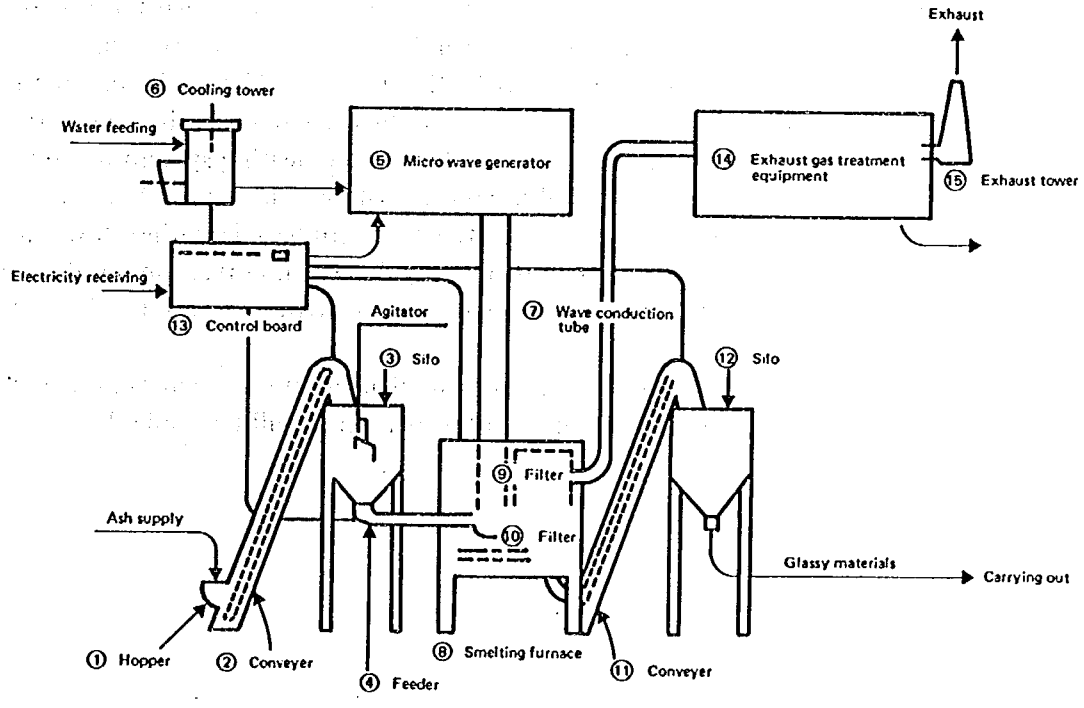


Fig. 10 Micro Wave Type Smelter

Table 10 Strength of Smelted Slag

Items	Rapid cooling	Gradual cooling			Natural rocks	
		Quarter	Hour	Day	Andesite	Tuff
Compression strength kg/cm^2	350	1200	3800	7000	1340	196
Tensile strength "	0 – 25	90	290	620	–	–
Flexural strength "	0 – 35	110	310	670	–	–

3-8 Electric Power Generation with Digester Gas

In the 1960s electric power generation with digester gas was tried, but it was never put into practical use.

However, the recently rising cost of electricity has brought more attention to electric power generation with digester gas again.

PWRI of the Ministry of Construction and the Science and Technology Agency conducted jointly a demonstration test of electric power generation with digester gas with a capacity of 200 kW at Hiakari treatment plant in Kitakyushu City.

The Japan Sewage Works Agency made a survey on electric power generation with digester gas in Okinawa and has launched design of a plant in 1982.

Tomakomai City adopted electric power generation with degester gas for emergency power generation. In addition, several cities are now examining the introduction of electric power generation with digester gas. The problems of the electric power generation system with digester gas are how to increase the concentration of feed sludge to digesters, how to apply a hot water heating system to utilize waste heat fully and how to ensure a stable amount of gas. In Japan, because gas generation efficiency (amount of gas generated/feed sludge) is rather low and many treatment plants have difficulties with heating digestion tanks, the number of plants which can immediately produce electric power with digester gas is quite limited.

West Germany has had long experience in electric power generation with digestion gas, so we are hoping that much useful information will be available.

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PRODUCTION AND USE OF DIGESTER GAS

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Production and use of digester gas

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1. Introduction

The primary purpose of the anaerobic sludge digestion is

- to stabilize the sludge in order to reduce problems in further sludge-handling e.g. unpleasant odours from storage
- to reduce the amount of solids
- to improve the dewaterability
- to reduce pathogenic bacteria

As a by-product of anaerobic sludge stabilization digester gas (in Germany about $250 \cdot 10^6$ m³ per year) with a high caloric value is produced. A large fraction of the gross energy of the gas has to used for heating-up of the sludge and to compensate for heatlosses by radiation. On larger plants since some decades the gas is used to produce mechanical power to drive electrical generators, pumps or blowers. The thermal energy from the gas engines is used for digester heating. Other applications of digester gas are pasteurization and sludge drying. In some cases the excess of digester gas is sold for heating; in earlier days it was used as fuel for cars and trucks.

2. Gas production and energy requirements

For design of sludge handling systems in Germany the following values are used:

94 g	dry solid per capita per day
62%	volatile solids
4%	solids concentration
2.35 l	raw sludge per capita per day

For energy calculations it has to be considered that the design data for sludge handling include some safety factors. The annual average may only be 75% of the design data which leads to

16 kg volatile solids per capita per year

480 l digester gas per kg volatile solids with a caloric value of about 5000 kcal/m³ will be produced:

7.65 m³ gas per capita per year
≙ 21 l gas per capita per day
≙ 38,000 kcal per capita per year
≙ 44.2 kWh per capita per year

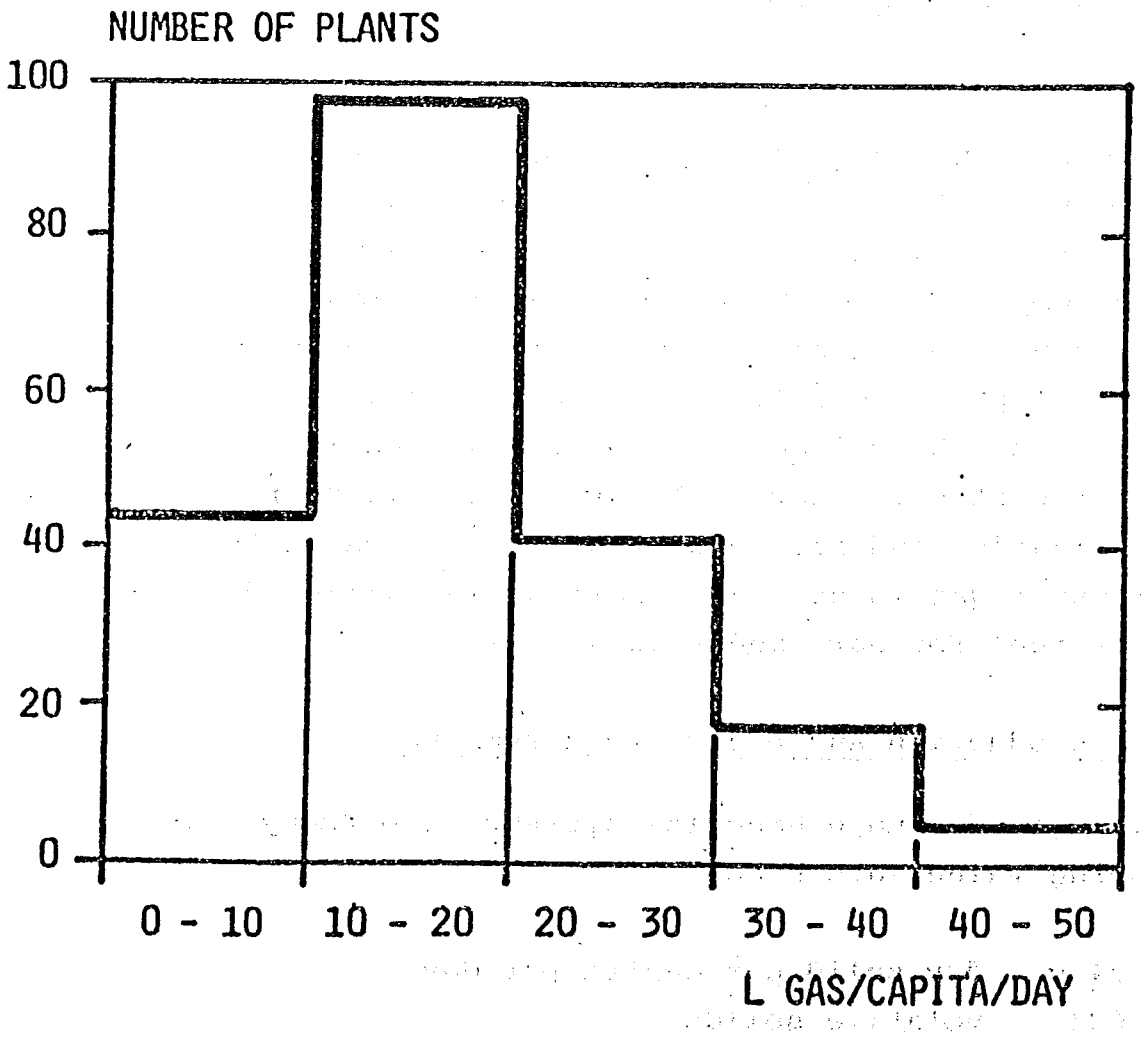


Fig. 1: Specific gas production on plants in Germany (Investigation from LOLL [1])

That in practice in numerous plants less gas than the theoretical amount of 21 l/cap.·d is produced is indicated by the results of a survey from Loll et al. 1 (1980), which are shown in Fig. 1. The average of 17 l/cap.·d is 20% below the theoretical value.

The energy requirement for digester heating is mainly depending on the raw sludge temperature, the ambient air temperature, the size of the digester and the detention time. Fig. 2 shows the energy requirements for digestion for summer (sludge temperature 17°C, air temperature 20°C) and winter conditions (7°C and -10°C).The heatlosses for radiation were asumed to be 1.2 Wh/m²·h·°C, the heatlosses are strongly depending on the size of the digesters because the surface area is decreasing with increasing digester volume (300 m³ ≙ 0.75²/m³; 1000 m³ ≙ 0.5 m²/m³; 6000 m³ ≙ 0.28 m²/m³).

The annual average energy requirement may be calculated as follows:

detention time (design)	15 days
average sludge temperature	10°C
average air temperature	10°C
digester temperature	33°C
digester volume	32 l/capita
average detention time (75%)	20 days
energy for heating up	40 kcal/cap·d
energy for radiation (0.6 m²/m³)	13 kcal/cap·d
(0.3 m²/m³)	6 kcal/cap·d

The average energy requirement should be in the range of 45 to 55 kcal/cap·d (50 to 65 Wh/cap·d). From Fig. 2 it can be seen that the peak requirement in winter may be as high as 70 to 90 Wh/cap·d whilst during the summer 40 to 50 Wh/cap·d may be sufficient.

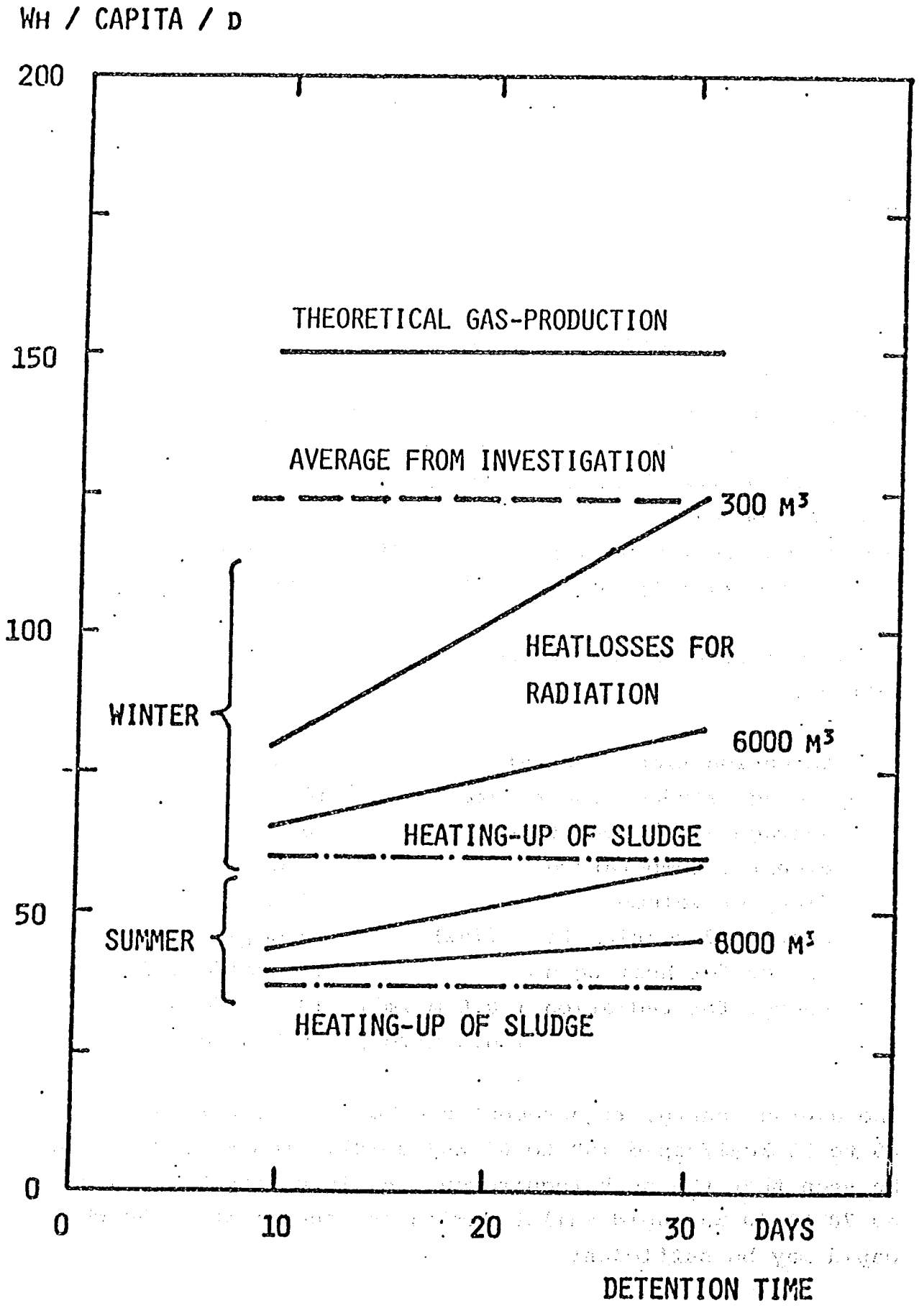


Fig. 2: Energy production and energy requirements of digestion

For further calculations an annual requirement for digester and sludge heating of about

20 kWh per capita per year

will be considered.

The main energy on a treatment plant is needed for aeration. After primary sedimentation a BOD of 40 g/cap·d and a nitrogen load of 12 g/cap·d is asumed. The oxygen requirements without nitrification are about 50 g O_2/cap·d, with nitrification 90 g O_2/cap·d and with nitrification-denitrification 65 g O_2/cap·d. Considering an additional 30% for pumping, other drives and as safety factor and taking into account an aeration efficiency of 1.5 kWh/kg O_2 we obtain

without nitrification:
16 kWh per capita per year

with nitrification-denitrification:
20.5 kWh per capity per year.

The average total energy requirement for a treatment plant is in the range of 35 to 40 kWh/capita·d, about half of which is caloric and electrical, respectively. The total requirement is a little below the gross energy of the digester gas, losses of energy for the conversion from gas to power, however, have to be considered.

3. Use of digester gas

3.1 Gas treatment

The only undessirable impurity of digester gas is hydrogen sulfide, which is easily oxidized to corrosion causing sulphuric acid. By filtration through ferric ore, hydrogen sulfide can be removed. If the gas shall be used, some storage is necessary. In general the storage capacity is about half of the daily gas production.

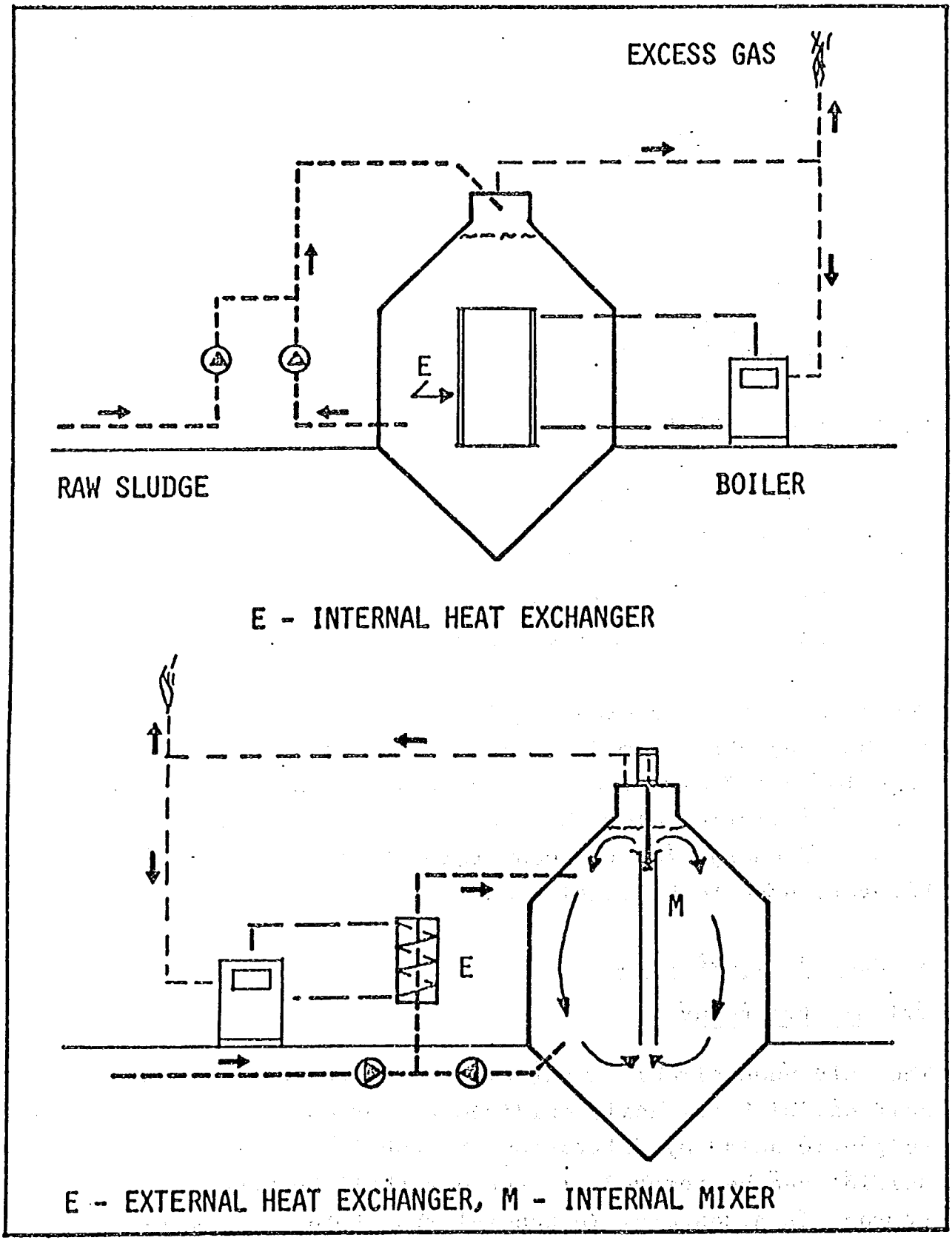


Fig. 3: Schemes of digester heating

3.2 Digester heating

In Germany digesters are heated by internal or external heat exchangers (Fig. 3). The heat-carrier is hot water, which in general is heated in a boiler by digester gas. Instead of using a boiler the hot water can be cooling water from gas engines or other processes, which will be discussed later.

The internal heat exchanger is a double pipe, between which the hot water is flowing. This type of heat exchanger is used for smaller digesters. Mixing of the digester is performed by external pumps. The raw sludge is always mixed with recirculated sludge from the digester.

External heat exchangers are of the pipe-in-pipe system. The sludge flows in the center pipe and the heating water between the pipes. The raw sludge and as much recirculated sludge as necessary to keep the digester temperature are pumped through the heat exchanger.

In order to obtain in large digesters an even temperature the digester content may be mixed with an internal mixer or by gas injection. For boiler heating the produced digester gas in general is sufficient all year round (Fig. 2).

3.3 Sludge pasteurisation

If liquid sludge shall be used as fertilizer on pasture a pasteurisation of the sludge is required in order to reduce pathogenic organisms. It is assumed that a temperature of 65 to 75°C and a holding time of 20 to 30 minutes are sufficient. It is usual to pasteurize the digested sludge (post-pasteurisation), experiments and full scale operations, however, indicate that pre-pasteurisation has advantages over post-pasteurisation.

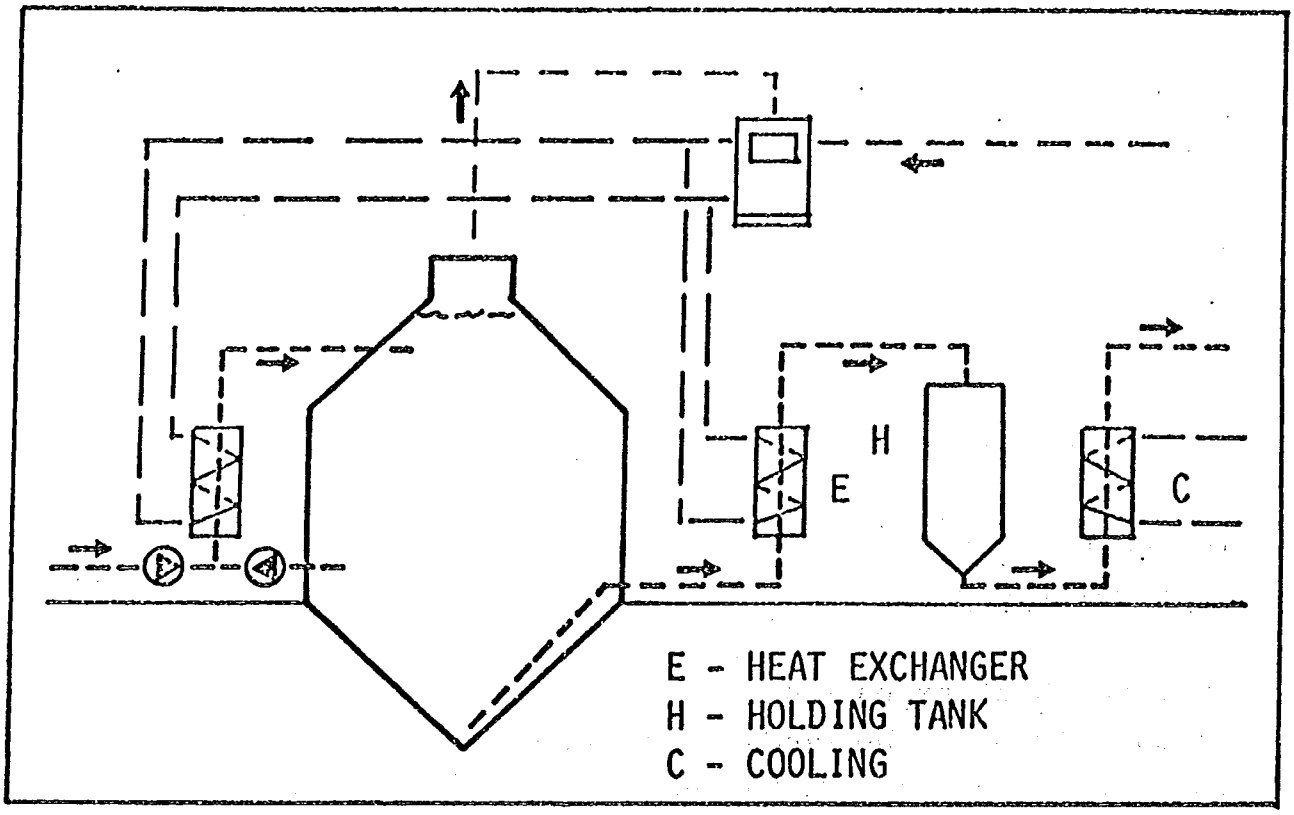


Fig. 4: Scheme of post-pasteurisation

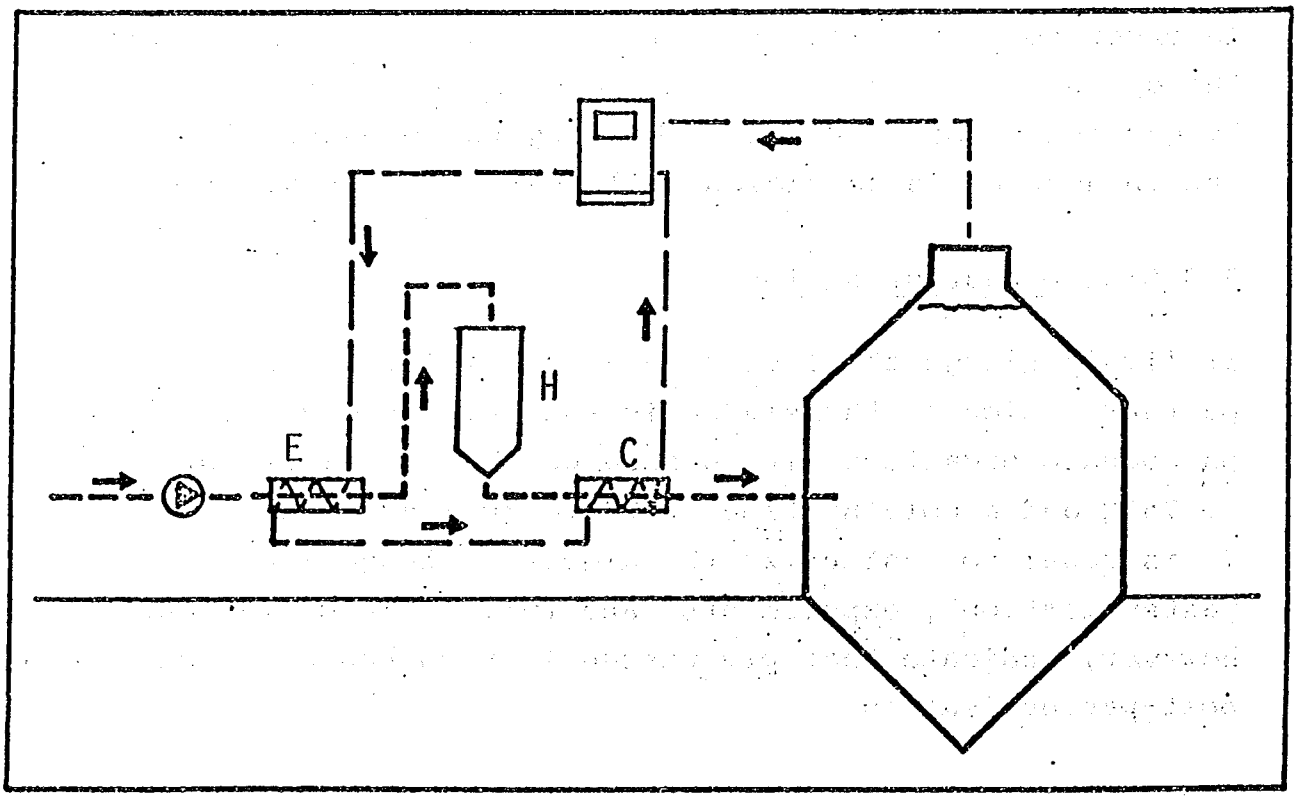


Fig. 5: Scheme of pre-pasteurisation (Görlich, 1978)

The principle of pasteurisation is shown on Fig. 4. The sludge may be heated by steam injection, by a sumerged burner or by a pipe-in-pipe heat exchanger. After leaving the holding tank the sludge has to be cooled.

Post-pasteurisation requires first to heat the sludge for digestion and than to heat the sludge for pasteurisation again and finally to cool it.

Pre-pasteurisation requires only one heating system ahead of the digester. In order not to over-heat the digester a light cooling system follows the holding tank [2]. The cooling water is used to pre-heat the raw sludge (Fig. 5). All the heat for digestion is carried into the digester by the raw sludge. In order to obtain an even temperature and raw sludge distribution, respectively, within the digester, internal or external pumps or other devices for mixing are needed.

Pre-pasteurisation requires only a little more energy than for heating of the digester. The higher losses are due to radiation of the heat exchangers and the holding tank. The heating water may be generated in a boiler or it may be cooling water from gas-engines.

3.4 Sludge drying

The advantage of thermically dried sludge is that due to the heat treatment pathogenic bacteria are killed and that the dried sludge can easily be stored and transported. The disadvantage in general is that after mechanical dewatering the remaining water has to be evaporated by thermal energy. A breakthrough was the invention of the Alfelder Eisenwerke to recover the heat from the vapor and to use the recovered heat for digester heating [3]. A full-scale plant was built for the City of Alfeld. The drying plant is designed for a population of 120.000, presently only 60.000 inhabitants are connected to the plant, which consists of primary treatment and an activated sludge plant. The sludge is digested, thicked and dewatered by a belt filter press (Fig. 6).

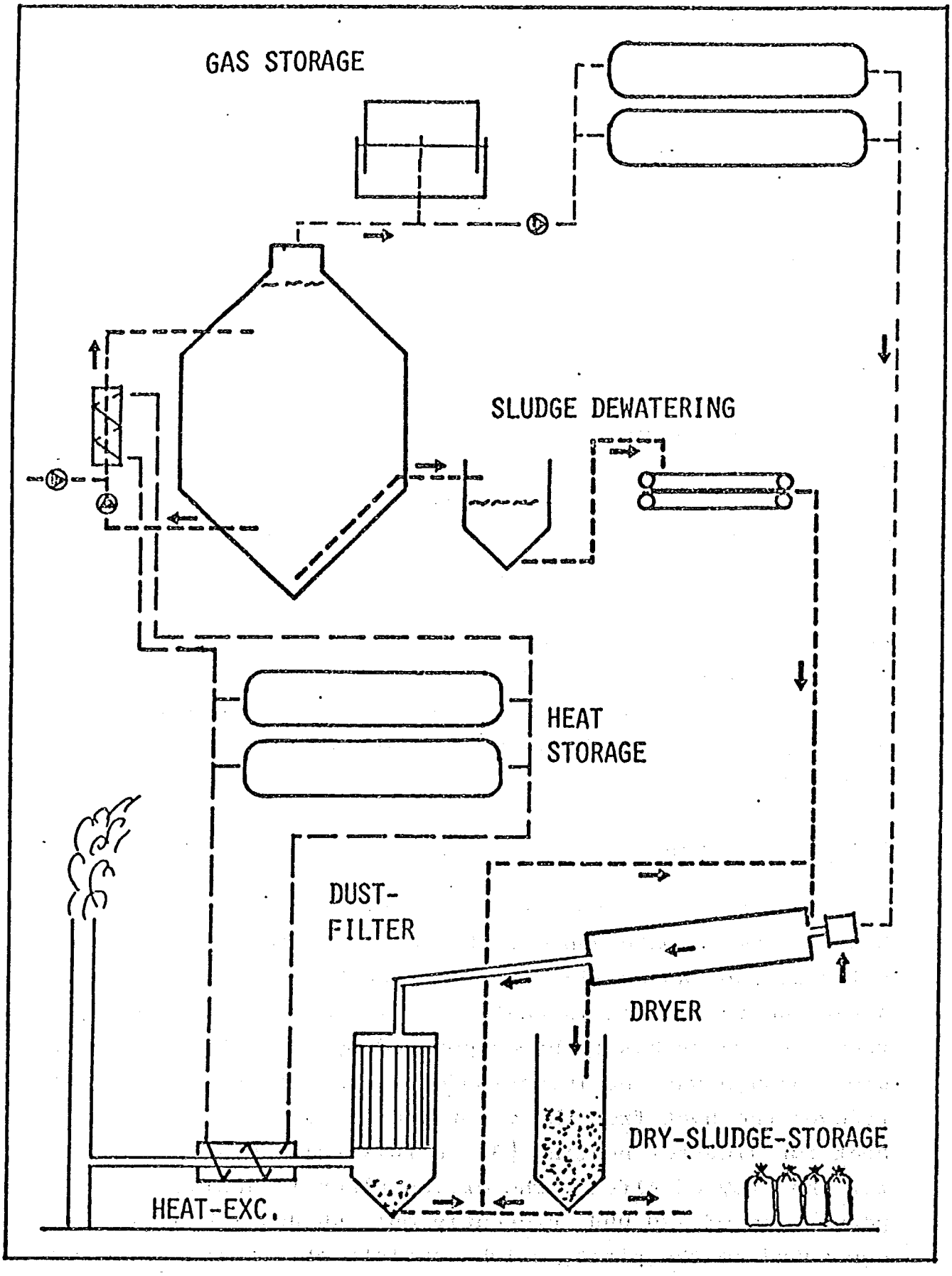


Fig. 6: Sludge drying with heat recovery (ALFELD-process)

The drying plant was planned to be operated only during two shifts on 5 days a week, today it is operated only 4 to 6 hours on 5 days a week. It was calculated that if the solids content of the dewatered sludge would be higher than 27% the digester gas would be sufficient for drying and by heat recovery sufficient for digester heating. Problems were that the weekly produced gas would have to be needed on 5 days during 5 to 10 hours. Therefore, larger amounts of gas had to be stored as usual. On the other hand, thermal energy was released only during the periods of plant operation (five days during 5 to 10 hours) but it was needed 7 days a week. The solution of this problem was to store heat in isolated hot water tanks.

Operational results indicate that because the sludge dewaters very good on the belt filter press the digester gas is sufficient although the plant today is only partly loaded.

3.5 Production of mechanical and electrical energy

Continuously increasing costs for electricity during the last years accelerated the use of digester gas for power production even in smaller wastewater treatment plants. The development of modular "cogeneration units" which produce power and heat simultaneously, simplify the installation. A module consists of an internal combustion engine and a system for heat recovery, from the cooling media and the exhaust gases. Each module is connected to a control cabinet.

The engines for gas are preferably spark fired, they may be operated with digester gas and natural gas. The diesel-gas engine may be operated with digester gas and diesel as supplemental fuel, but even if sufficient gas is available about 10% of the primary energy has to be diesel-fuel.

German manufacturers claim that the primary energy is converted to 33%, electrical power and 50% to 54% utilizable heat. Experienced operators asume 27% electrical power and 45% heat [4]. A survey in Bavaria indicates that in practice

due to operation below the design capacity only 25% power and 30% heat are generated [5].

For further calculations it is assumed that 30% of the gas energy is converted to mechanical power or 27% of electrical power and 45% heat of which 10% may be lost on the way from the engines to the digester thus leaving 40% for digester heating. From 44.2 kWh/cap·a of gross gas energy we obtain

12.0 kWh (electrical) per capita per year
17.7 kWh (heat) per capita per year
≙15.200 kcal per capity per year

The average 16 to 20.5 kWh/cap·a electrical power demand could be covered from in-plant generation by 58 to 75%. Operational results from larger plants confirmed a 70% in-plant generation. The heat of 17.7 kWh/cap·a or 48 Wh/cap·d in summer may be sufficient for the heating of larger digesters. In winter, however, when the heat demand is higher there remains a gap of about 25 to 40 Wh/cap·d. The annual gap will be in the range of 3 kWh/cap·a, which is 17%.

In Germany cogeneration units including the building cost about 2500,- DM per kW electrical power output. If engines for the average gas production would be installed the specific costs would be about 80,- DM pe- capita. The manufacturers of cogeneration units offer a maintenance service including spare parts and repairs for 0.03 to 0.04 DM/kWh. In addition about 0.01 DM/kWh operator wages and 0.01 DM/kWh for lubrication are required. The total operating costs would be about 0.05 to 0.06 DM/kWh. The same results were confirmed by the Emschergenossenschaft [5]. The capital costs have to be based on the working life of the engines which is asumed to be 100.000 hours (11 years of continuous operation). Based on 15% for interest and pay-back the capital costs are about 0.04 DM/kWh. The total costs for one kWh, therefore, theoretically will be about 0.10 DM. Gas power stations for larger plants cannot use modular units, furthermore the buildings

have to be more stable which all makes power production more costly. The City of Hamburg (2 Mio. p.e.) calculated 0.15 DM/kWh.

In Germany the costs for electricity are composed of the peak charge of about 260,- DM/kW (the peak is measured each month as the highest kW-uptake during 15 minutes) and the kWh consumed (0.14 DM/kWh at day time 6 a.m.-9 p.m. and 0.09 DM/kWh at night time 9 p.m.-6 a.m.). As a function of the fluctuation of wastewater flow and load the effective costs are in the range of 0.18 to 0.21 DM/kWh, of which in general are about 30 to 35% for the peak charge and 50 to 60% for day-time consumption.

For further calculations besides the costs for electricity the costs for alternative fuel such as natural gas, propane etc. and diesel have to be considered.

Due to the high costs for electricity at day time it seems very attractive to produce power only at this period. In this subject a research project is carried out on a full scale plant. A computer control system was installed in order to optimize the use of the gas for power and heat generation.

Most calculations and practical experiences indicate that it is favourable to install just as much engines for continuous operation as digester gas safely is available. If the average gas production after deduction of some 10 to 20% as a safety factor is assumed to be the available amount of gas, in winter additional fuel will be needed for digester heating or some of the engines have to be switched off. In practice both concepts are employed. A general scheme is shown on Fig. 7.

In smaller plants the generation of electricity may become to costly. In those cases the engines directly may drive blowers, pumps or even surface aerators.

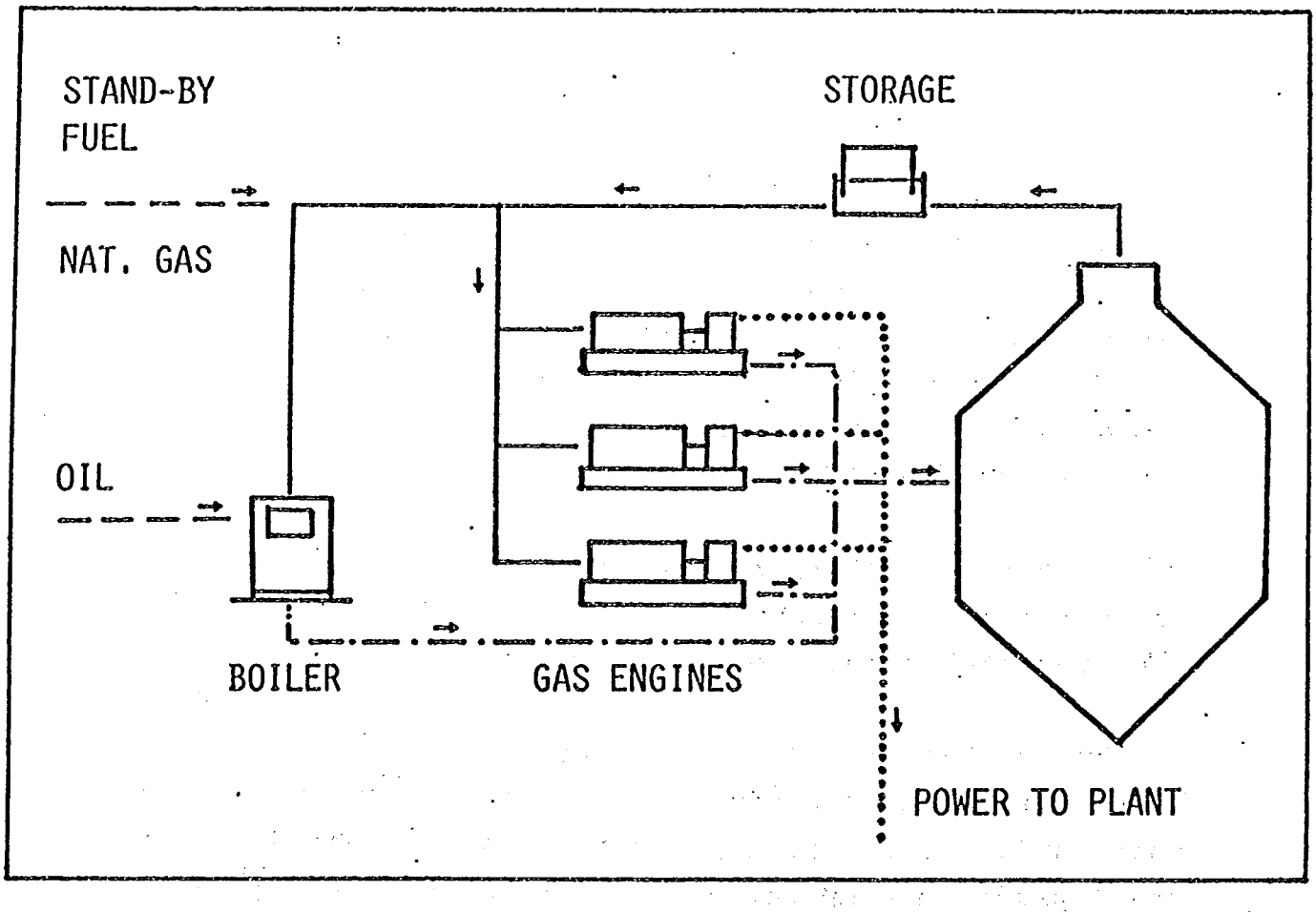


Fig. 7: Scheme for power production from digester gas

4. Use of digester for other purposes

During and after the last war, digester gas in Germany was used as fuel for municipal trucks as for garbage-collection etc. The gas was highly pressurized. From gas-filling-stations the gas was transfered into the gas tanks of the trucks. Because gasoline and diesel became cheap and because all heavier trucks more and more were equiped with diesel-engines the use of digester gas was no longer economical.

The excess gas of the treatment plant of Salzgitter is piped to the City-hospital where it is used as fuel for boilers in winter and in summer as well. The Niesverband upgrades the digester gas to natural gas quality and feeds it into a

nearby gas pipeline. The gas is treated by a high pressure washing process in which the CO_2 is dissolved in final effluent.

5. Final remarks

There are many ways to use digester gas. The production of mechanical/electrical energy and heat will be most favourable in future, because the costs for electricity will increase at a higher rate than the costs for power from digester gas due to its high but constant rate of the capital costs. Considerable savings, however, will only be obtained if waste-water treatment processes with low energy consumption (e.g. high aeration efficiency, two stage processes etc.) are applied. The fluctuations of the power uptake by means of equalization and appropriate control should be minimized as far as possible.

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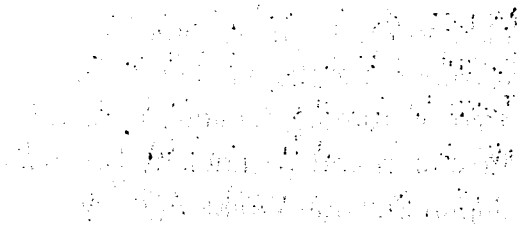
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STUDY OF DIGESTER GAS POWER GENERATION SYSTEM AT NAHA SEWAGE TREATMENT PLANT, OKINAWA

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1. INTRODUCTION

Okinawa is the southernmost part of the Japanese archipelago, and consists of some 70 small and large islands.

Naha City is the largest city in Okinawa with a population of about 300,000 and is located on the south end of Mainland Okinawa.

Since Okinawa is in the subtropical zone, its temperatures are relatively high year-round. In Naha, the annual average temperature is about 22°C, and even in January, when it is coldest, the average temperature rarely falls below 15°C. From summer to early fall, typhoons often visit on Okinawa.

The Naha Sewage Treatment Plant, began operation in 1969, adopted a separate sewer system draining wastewater from Naha City and Urszoe City and has anaerobic digesters for the sludge treatment process.

Many sewage treatment plants in Japan are provided with anaerobic digesters, and the gas emanating from the digester tanks has so far been used mainly for heating the digesters. The Naha plant has been no exception. But because of high ambient temperatures, the Naha digester requires less gas for heating, and significant amount of excess digester gas has been wastefully burnt away.

Because Japan's energy situation is precarious, every effort is being made to save energy and develop alternative energy sources. Even in wastewater treatment plants, energy conservation is required.

With this in mind, the Okinawa Prefectural Govenment decided to introduce a digester gas power generation system to harness gas available at the Naha plant, then in Japan digester gas generation systems was quite unpopular and there were little technical data on the performance of internal combustion engines using digester gas.

Thus the digester gas power project had to begin with identification of design parameters for the system and verification of the performance of an applicable gas engine. At the same time an economic feasibility study was required. These studies were conducted by the Japan Sewage Works Agency for two years (FY1980 and 1981) at the request of the Okinawa Prefectural Government. The studies were conducted with respect to the following subjects according to the flowchart specified in Fig. 1.

(1) Performance study of the functions of the sludge treatment process

(2) Survey of in-plant electrical consumption

(3) Performance study of gas engines

(4) Preparation of a proposal for a digester gas power generation system

(5) Cost-effectiveness analysis of the proposed power generation system

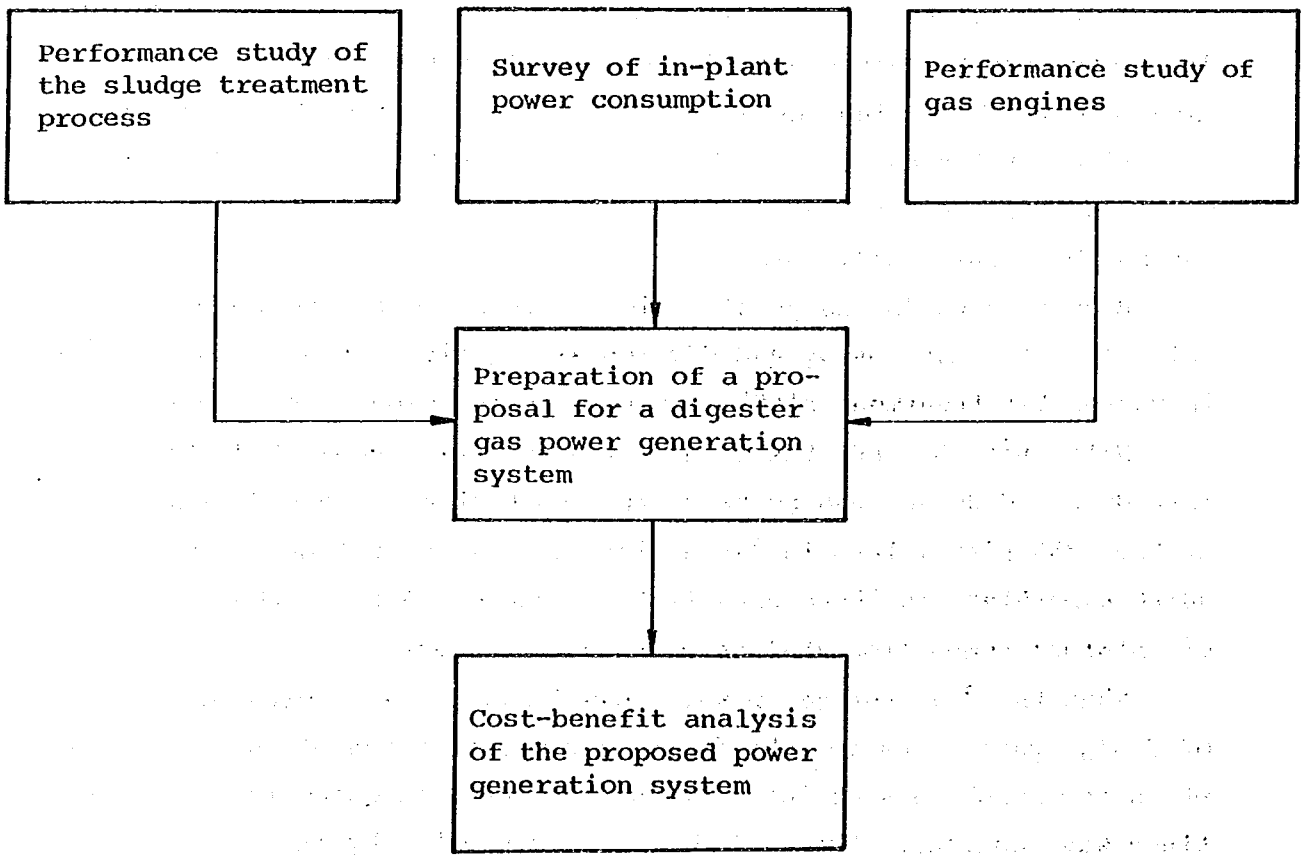


Fig. 1 Flowchart of feasibility study project

2. AN OUTLINE OF NAHA SEWAGE TREATMENT PLANT

The principal particulars of the Naha plant are shown in Table 1, and its flow diagram is illustrated in Fig. 2. Wastewater treatment is carried out by the step-aeration activated sludge process, while sludge treatment is carried out through thickening, anaerobic digestion and dewatering. In the sludge treatment process, the sludge from the primary sedimentation tank is gravitationally thickened, and the excess activated sludge from the secondary sedimentation tank is thickened by flotation. For digestion, a two-stage anaerobic system is used.

Table 1. Principal particulars of Naha Sewage Treatment Plant

Item	Present	Planned
Served area (ha)	1,084	4,151
Served population	193,500	269,500
Treatment capacity (m^3/day)	81,000	126,000

3. PERFORMANCE STUDY OF THE SLUDGE TREATMENT PROCESS

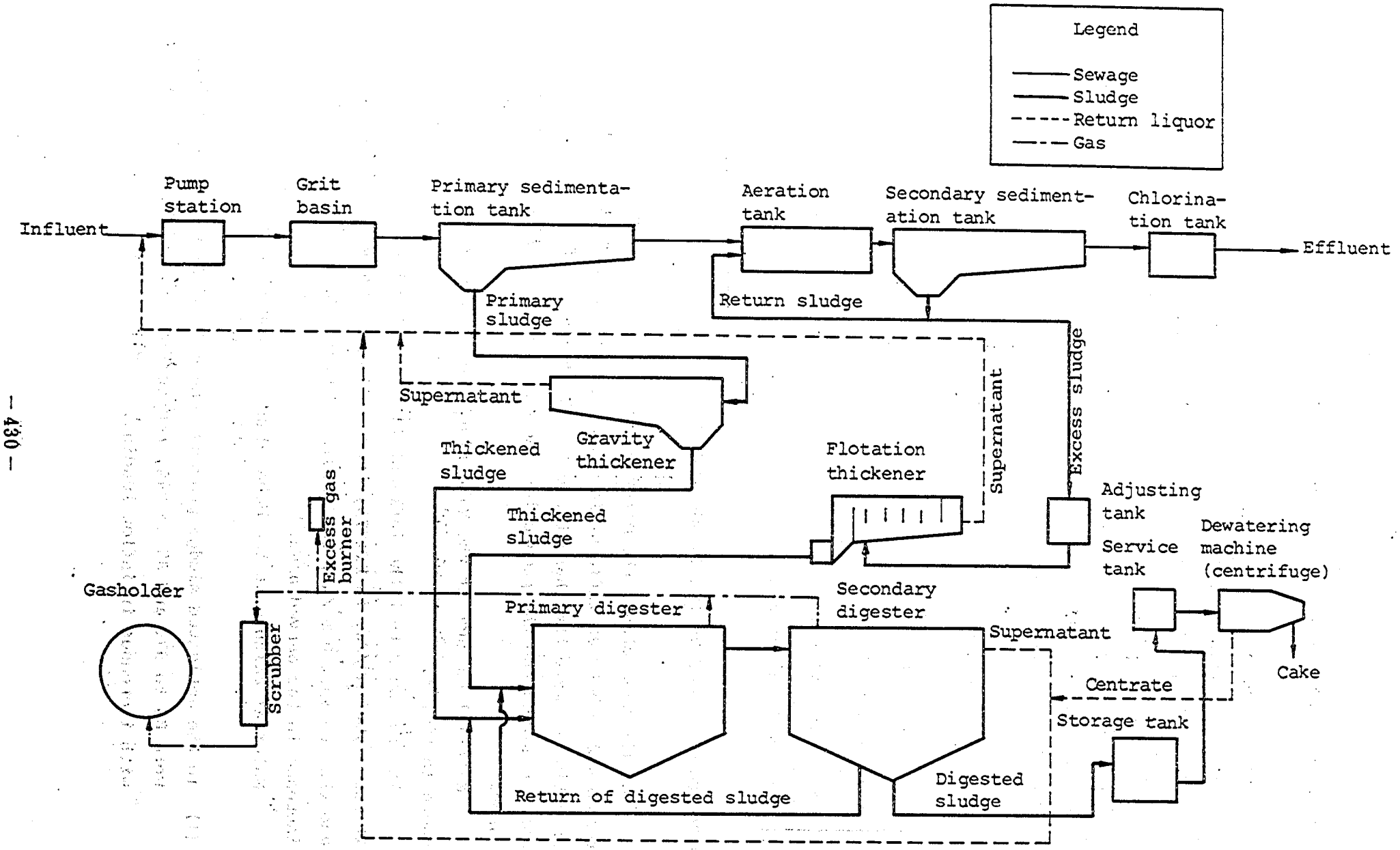
At the Naha plant, the material balance was studied to identify the relationship between influent loadings and sludge production. Also investigated were the characteristics of thickened sludge, variations in solid loadings and the production and chemical composition of digester gases.

Material balance at the Naha plant during the FY1979-81 period is shown in Fig. 3.

It was found that the influent flow rate has been rising steadily: It was 52,500 m^3/day in FY1979, increased to 57,170 m^3/day in FY1980 and then to 59,250 m^3/day in FY1981. This was due primary to annual increase in the served area.

Material balance at the Naha plant is summarized below.

(1) In the activated sludge process, the BOD and SS removal efficiencies were 89 to 97% and 85 to 94%, respectively, and tended to decline with increases in inflow loading.



Note: Seed sludge return in the sludge digestion process since FY1981.

Fig. 2 Flow diagram of Naha Sewage Treatment Plant

(2) SS loadings returned from the sludge treatment process were 16 to 55% of influent SS loadings, and tended to decrease each year with improvement in the recovery efficiency of total solids in the sludge thickener.

(3) The total weight of the primary and the excess sludge was 1.1 to 1.6 times the influent SS loadings, and this ratio tended to drop each year. This was because sludge recirculated in the process got reduced with the decrease in loadings returned from the sludge treatment process.

(4) The gravity thickener and flotation thickener improved in total solids recovery efficiency each year, 71 to 100% and 89 to 100%, respectively.

(5) Total solids fed into the digester remained nearly 1.2 times the influent SS loadings during the three-year period.

(6) Volatile solids destroyed in the digester were 54 to 59% of feed volatile solids.

(7) Total solids recovery efficiency of the dewatering process was 72 to 74%, and loadings returned from the dewater process remained in the range of 16 to 19% of influent SS loadings during the three-year period.

According to the material balance, the relationship between influent SS loadings and total solids fed into the digester is given by the following formula:

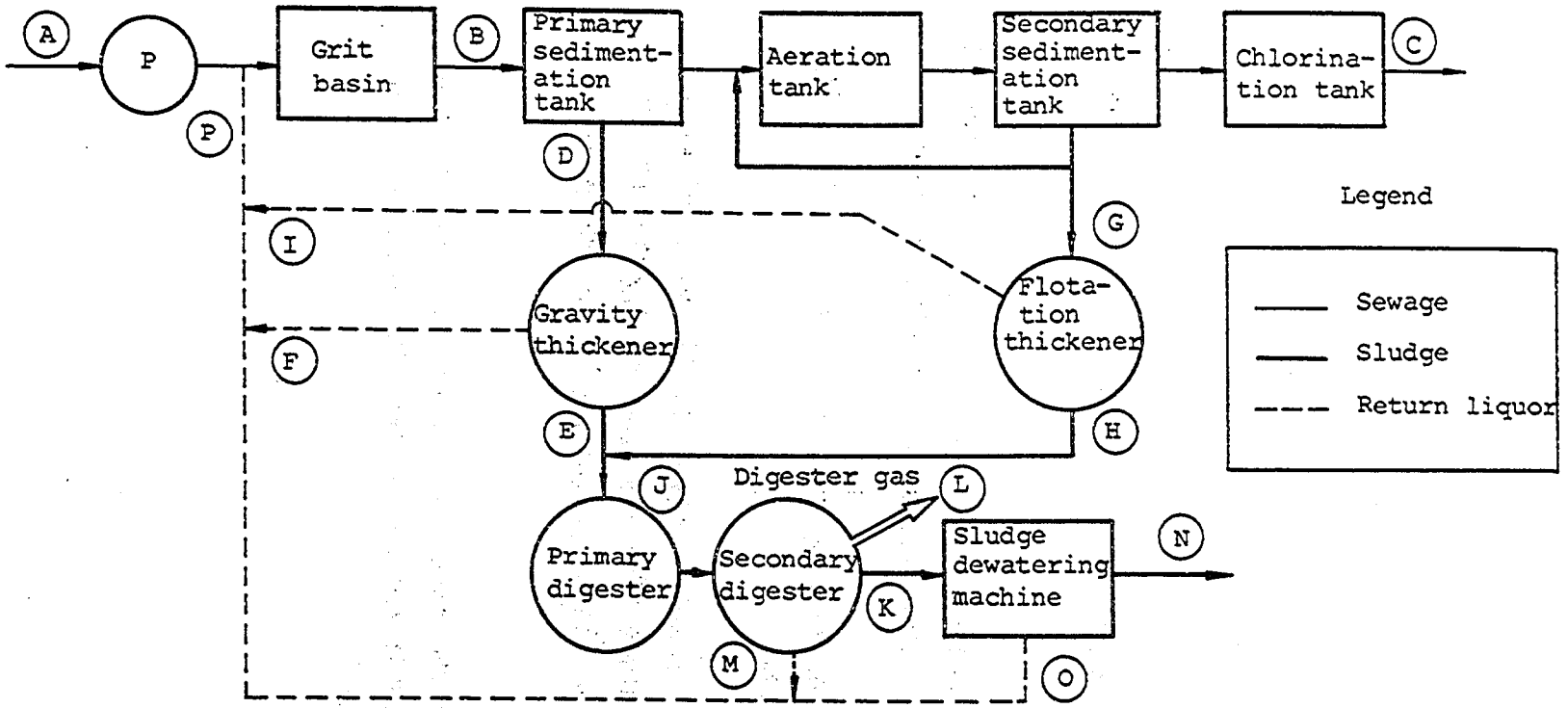


Fig. 3 Material b alance in Naha Sewage Treatment Plant (1/2)

Item			FY 1978	1979	1980
Activated sludge process	Influent A	Flow (m^3/day)	52,500	57,170	59,250
		SS loading (t/day)	5.712	7.710	7.809
		BOD loading (t/day)	6.812	8.421	11.068
	Grit basin effluent (B)	Flow (m^3/day)	54,100	59,500	61,300
		SS loading (t/day)	8.425	8.800	8.278
		BOD loading (t/day)	9.236	10.894	13.400
	Effluent to public waters (C)	Flow (m^3/day)	52,500	57,170	59,250
		SS loading (t/day)	0.326	0.680	1.185
		BOD loading (t/day)	0.200	0.578	0.393
Gravitational thickener	Feed sludge (D)	Total solids (t/day)	5.992	5.281	4.569
	Thickened sludge (E)	Total solids (t/day)	4.261	4.500	5.167
		Volatile solids (t/day)	3.274	3.523	4.081
	Supernatant liquor (F)	Flow (m_3/day)	919	915	857
		SS loading (t/day)	1.731	0.781	0.0
Flotation thickener	Feed sludge (G)	SS loading (t/day)	3.148	4.296	3.872
	Thickened sludge (H)	Total solids (t/day)	2.811	4.298	3.926
		Volatile solids (t/day)	2.257	3.447	3.220
	Supernatant liquor (I)	Flow (m^3/day)	529	1,230	992
		SS loading (t/day)	0.337	0.0	0.0
Sludge digester	Feed sludge (J)	Total solids (t/day)	7.072	5.748	9.093
		Volatile solids (t/day)	5.531	6.970	7.301
	Digested sludge (K)	Total solids (t/day)	3.927	4.873	4.618
		Volatile solids (t/day)	2.402	3.195	2.981
	Digester gas (L)	Volatile solids reduction (t/day)	3.129	3.775	4.320
	Digester supernatant liquor		Nearly zero	Zero	Zero
Dewatering machine	Sludge cake (N)	Total solids (t/day)	2.835	3.615	3.362
	Separated liquor (O)	Flow (m^3/day)	154	188	206
		SS loading (t/day)	1.092	1.258	1.256
Return liquor, total (P)		Flow (m^3/day)	1,600	2,330	2,050
		SS loading (t/day)	3.160	2.039	1.256

Fig. 3 Material balance in Naha Sewage Treatment Plant (2/2)

$$S = Q \times Cs \times (1 + 0.15) \times 0.95 \times \frac{1}{0.9} \times 10^{-6} \quad \text{..... Eq.1}$$

Where S: Total solids fed into digester (TS-t/day)

Q: Daily average flow rate (m^3/day)

Cs: Influent SS concentration (mg/ℓ)

0.15: Loadings returned from sludge treatment process assumed as 15% of influent SS loadings

0.95: SS recovery efficiency of activated sludge process assumed as 95%

0.9: SS to TS ratio of thickened sludge (0.88 to 0.93 rounded to 0.9)

The relationship between influent SS loadings and total solids fed into the digester is given in Fig. 4.

Eq. 1 shows good agreement with the measurements. Total solids fed into the digester can be expressed by Eq. 1 as a function of influent SS loadings.

In order to make clear the seasonal variations in digester gas production, it is necessary to clarify the seasonal variations in influent SS loadings.

The seasonal variations of influent flow rate were as shown in Fig. 5. Though the Naha plant terminates a separate sewer system, its influent flow increases in wet weather periods. Therefore, seasonal variations of influent flow were analyzed based on dry-weather flow. It was found that influent flow becomes high in summer and low in winter.

Seasonal variations in the influent SS concentration were just the opposite of the influent flow, as shown in Fig. 6. The ratio of SS concentration variations from the average was 0.8 to 0.9 in summer and 1.1 to 1.3 in winter, and was larger than the ratio of influent flow variations from the average.

Seasonal variations of influent SS loadings were as shown in Fig. 7. While the variations of influent flow and influent SS concentration were inversely related, influent SS loadings followed the same variations as influent SS concentration because the width of the variations of influent SS concentration was larger than that of influent flow. But the width of the variations of influent SS loadings was smaller than that of influent SS concentration.

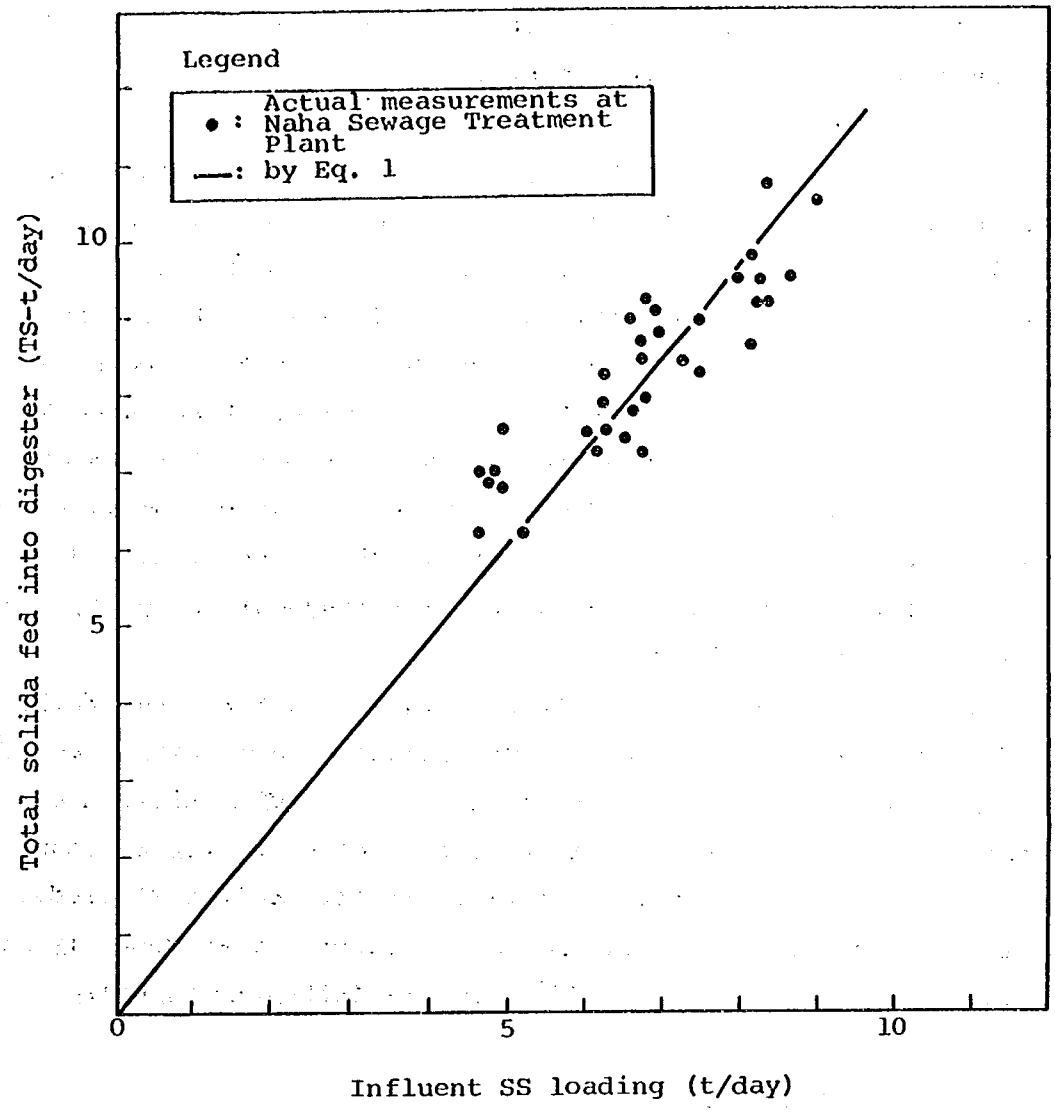


Fig. 4 Influent SS loading vs. total solids fed into digester

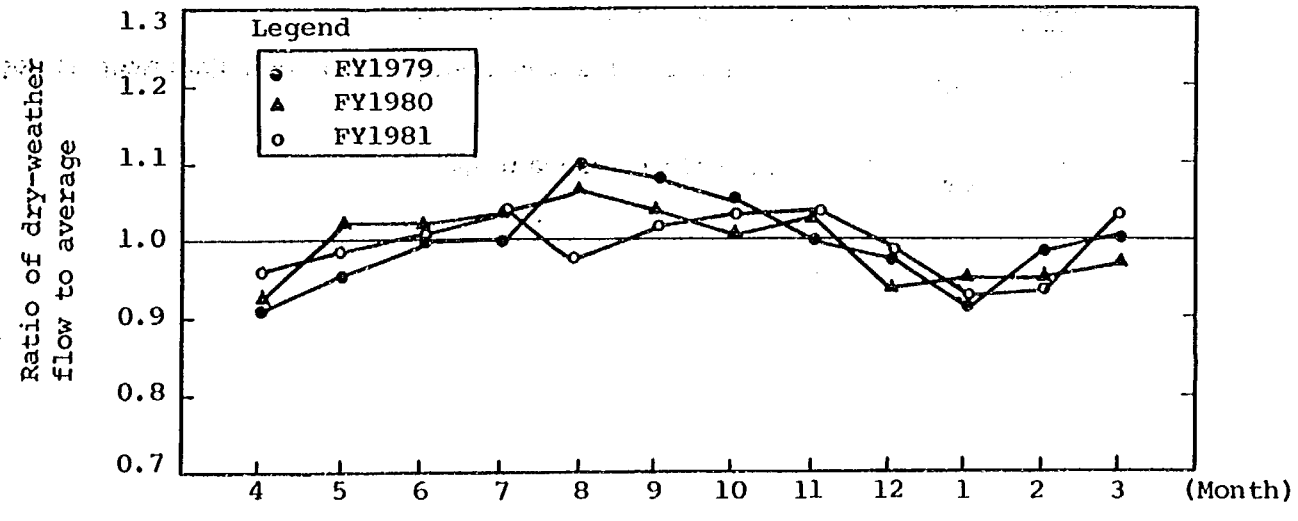


Fig. 5 Seasonal changes in dry-weather sewage flow

Seasonal variations of influent SS loadings and total solids fed into the digester are compared in Fig. 8.

From the above, it was found that seasonal changes in total solids fed into the digester are caused by seasonal changes of influent SS concentration, and that their width is smaller than that of the changes in influent SS loadings. This was probably because the activated sludge process and sludge thickener may have served to moderate variations.

At the Naha plant, digester gas productions had not been measured since 1980 because of gas meter failure. However, it was necessary to estimate the variations of digester gas productions. For the study, variations of digester gas productions were estimated from variations of total solids to be fed into the digester. The concentration of total solids fed into the digester was about 4% throughout the year, and showed little or no seasonal change.

The running data of the anaerobic digester during the FY1979-81 period were as shown in Table 2. The operating temperature was 35°C to 37°C, and the detention time was 24 to 47 days. The relationship between detention time and volatile solids reduction was as shown in Fig. 9, and the relationship between volatile solids content of sludge fed into the digester and volatile solids reduction was as shown in Fig. 10. Volatile solids reduction is defined by the following formula:

$$r_d = \left(1 - \frac{A_{in} \times V_{out}}{V_{in} \times A_{out}}\right) \times 100 \qquad \text{.....Eq. 2}$$

Where r_d: Volatile solids reduction (%)

A_{in}, A_{out}: Ash content in charged sludge (in) and digested sludge (out) (%)

V_{in}, V_{out}: Volatile solids content (%)

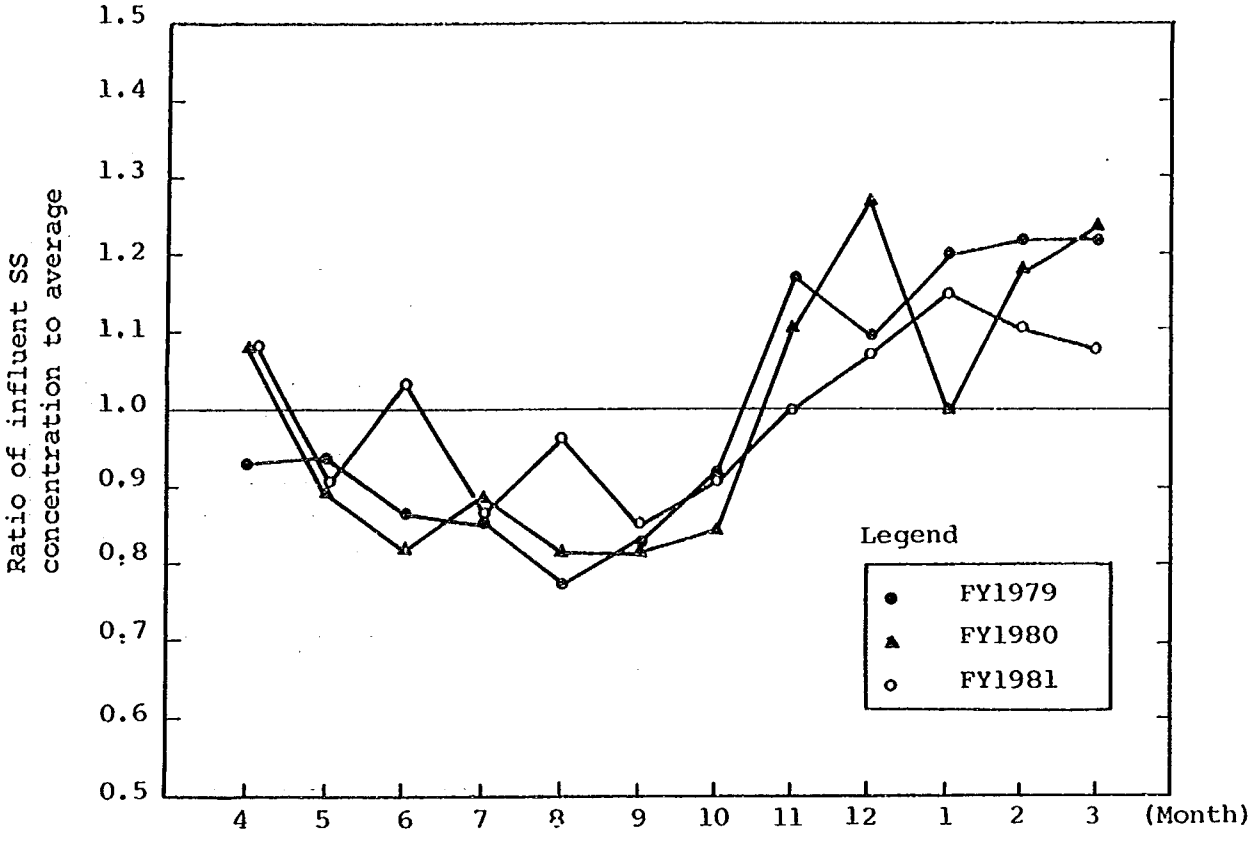


Fig. 6 Seasonal changes in influent SS concentration

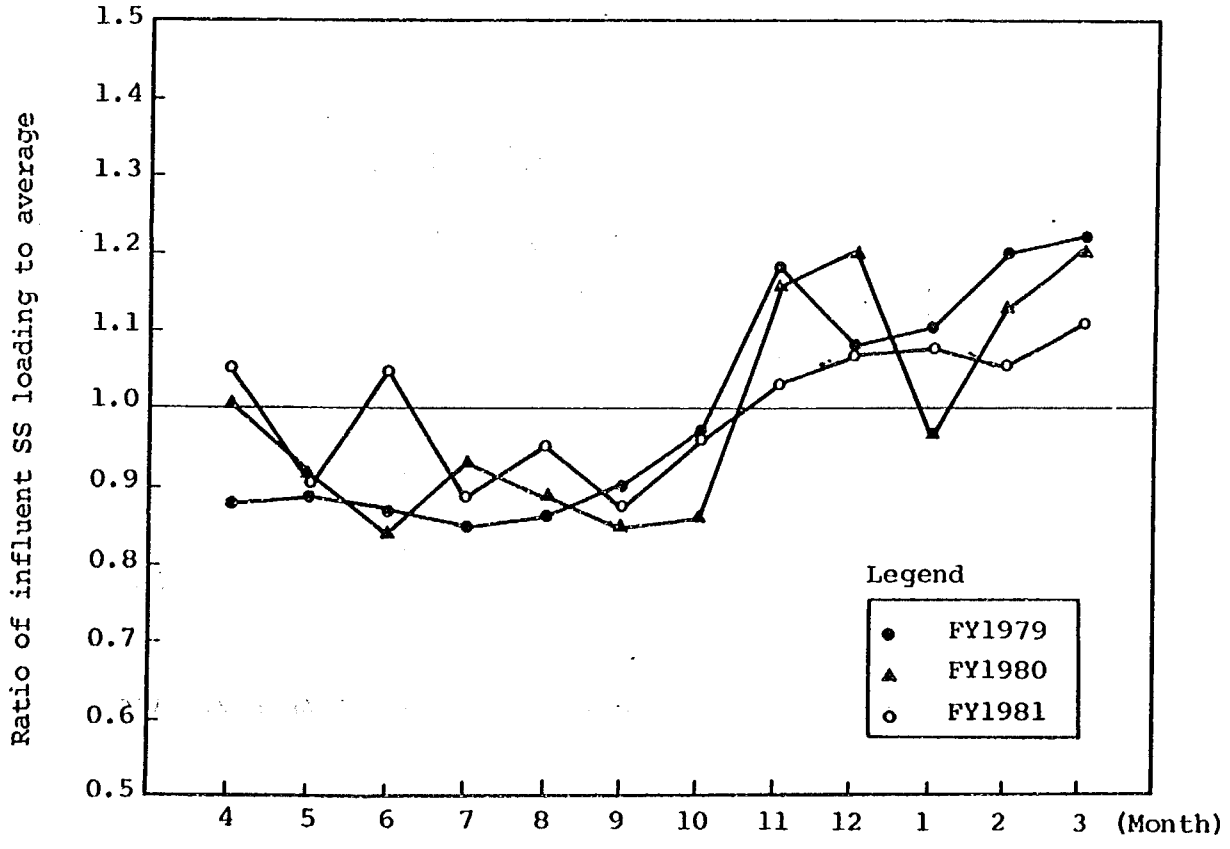


Fig. 7 Seasonal changes in influent SS loading

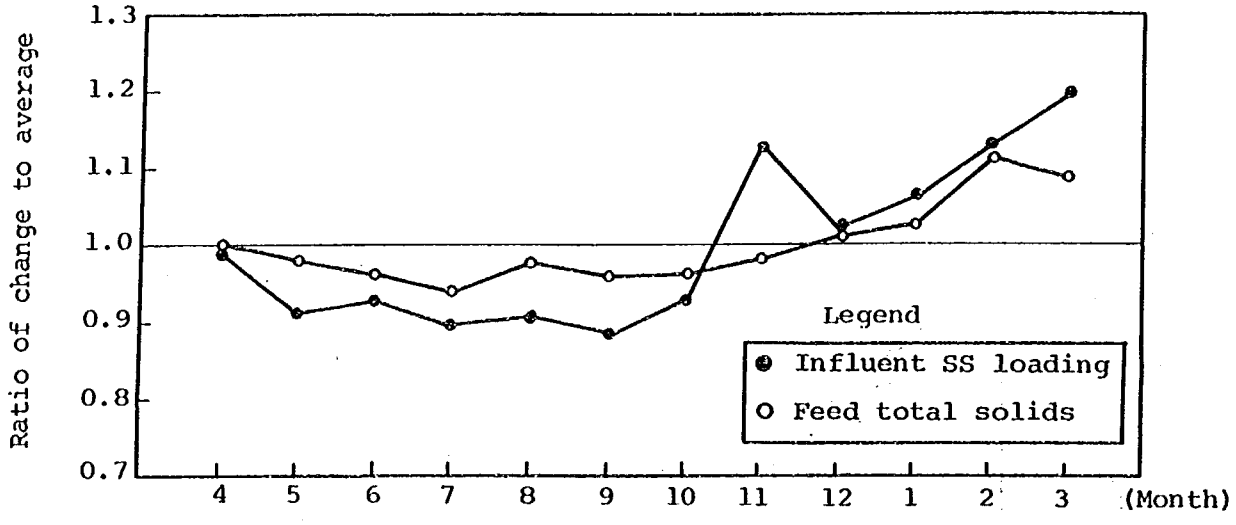


Fig. 8 Seasonal changes in influent SS loading and total solids fed into digester (mean value during FY1979-81)

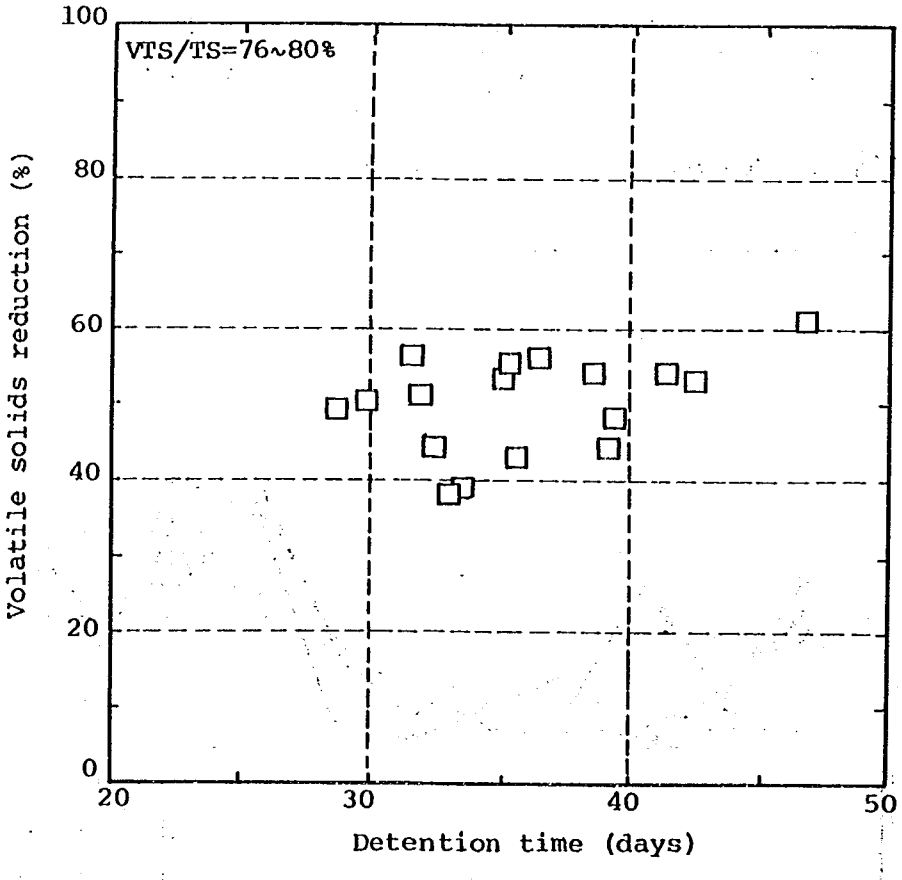


Fig. 9 (a) Detention time vs. volatile solids reduction

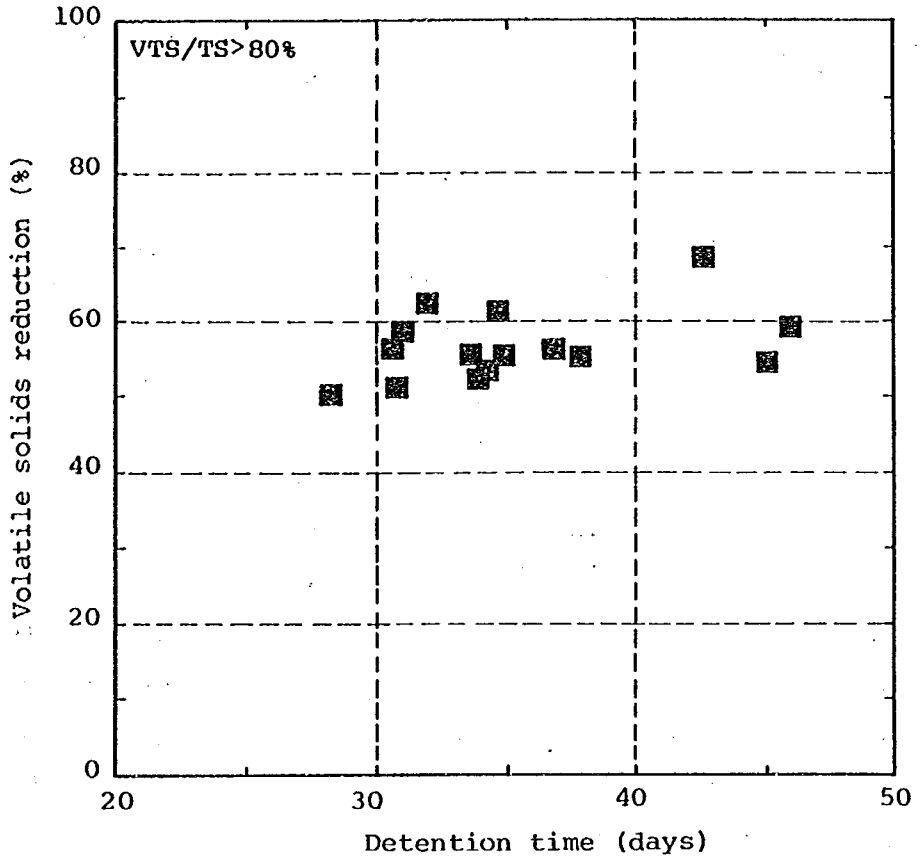


Fig. 9 (b) Detention time vs. volatile solids reduction

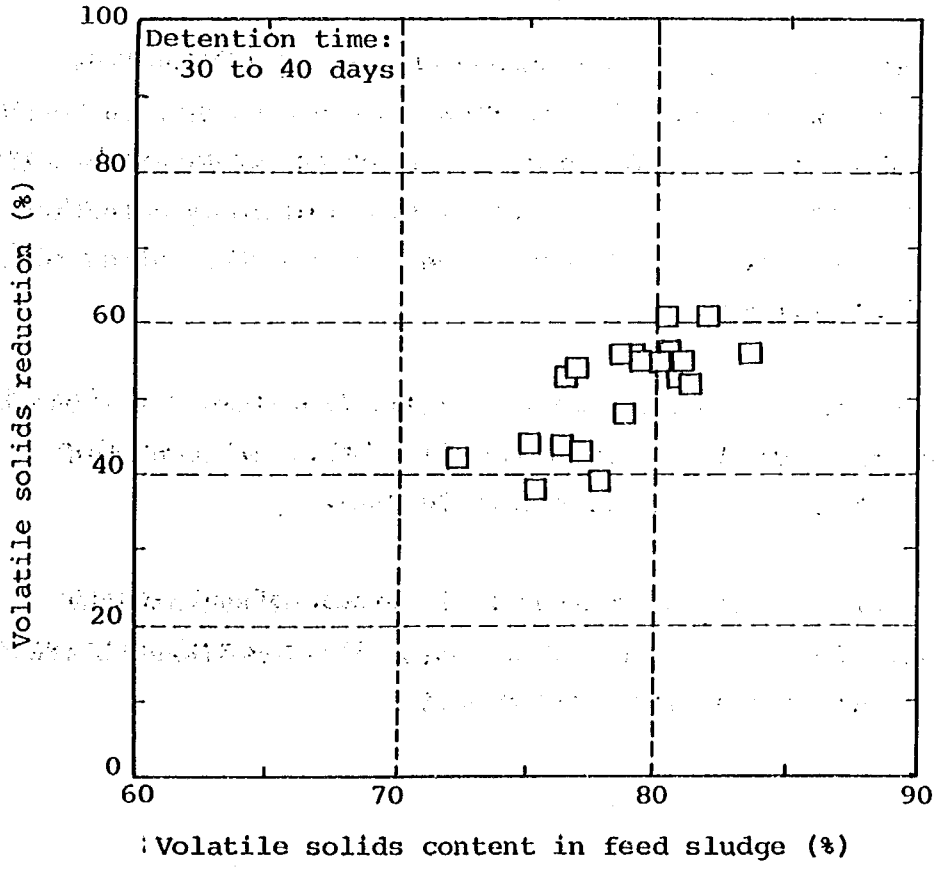


Fig. 10 Volatile solids content vs. volatile solids reduction

Table 2 Detention time, volatile solids reduction, and VTS-to-TS ratio of charged sludge

FY / Month	1979			1980			1981		
	Detention time in days	Volatile solids reduction	Charged VTS	Detention time in days	Volatile solids reduction	Charged VTS	Detention time in days	Volatile solids reduction	Charged VTS
Apr.	24.1	47%	73.5%	39.2	44%	76.3%	32.4	44%	75.0%
May	24.7	45	71.2	36.5	56	79.1	34.5	61	80.3
Jun.	35.1	53	76.4	35.3	55	79.4	34.5	61	81.9
Jul.	41.5	54	76.6	34.8	55	80.5	30.6	51	80.9
Aug.	39.0	42	72.7	35.6	43	77.0	33.5	55	81.0
Sep.	38.6	54	76.8	36.8	56	80.5	31.9	51	78.2
Oct.	46.8	61	78.1	33.5	39	77.7	31.6	56	78.6
Nov.	42.5	68	81.9	37.7	55	80.1	30.9	58	80.7
Dec.	45.9	59	81.1	34.2	53	80.9	31.8	62	83.0
Jan.	44.9	54	81.2	33.8	52	81.3	28.1	50	83.9
Feb.	39.4	48	78.7	28.7	49	79.6	30.5	56	83.5
Mar.	42.5	53	79.3	29.9	50	79.4	33.0	38	75.3
Average	38.8	53.2	77.3	34.7	50.5	79.3	31.9	53.6	80.2

As is clear from Fig. 9, the improvement in volatile solids reduction due to the increase in detention time was slight, but changes in volatile solids content had a significant effect on volatile solids reduction. From the running data, the relationship among detention time, volatile solids contents in the feed sludge and volatile solids reduction is summarized as follows:

(1) Where the volatile solids content in the feed sludge is less than 80%, volatile solids reduction may fall below 50% even when detention time is set at more than 30 days.

(2) In case volatile solids content is in excess of 80% and the detention time is in excess of 30 days, it is possible to attain a volatile solids reduction rate of 50%.

Because digester gas measurement was not carried out in FY1980 and FY1981, available gas data are for FY1979 only. In FY1982, digester gas measurement has been again. But the drawing-out of excess sludge has been hindered because of problems involved in the disposal of sludge cake. So the digester has not been operated normally except in July. For this reason, operating data in FY1979 and July 1982 and laboratory test results were used to determine the specific unit of digester gas production. Gas production per volatile solids destroyed was 0.82 to 0.95 Nm^3/kg-VTS as shown in Table 3, a little lower than the generally reported value of 1.0 Nm^3/kg-VTS. The laboratory tests indicated a smaller specific unit of gas production than the plant runing data.

Table 3. Gas production per volatile solids destroyed

Item \ Data		July 1982	April 1979 to March 1980	Laboratory test	
				RUN1	RUN2
Detention time (in days)		41	24 to 47 (39 on the average)	10	30
Volatile solids reduction (%)		59	45 to 68 (53 on the average)	56	63
Gas production per volatile solids destroyed	Measurement (m^3/kg)	0.96	0.78 to 1.61 (1.04 on the average)	0.89	0.89
	Componsated value (Nm^3/kg)	0.88	0.75 to 1.45 (0.95 on the average)	0.82	0.82

In the study, it was necessary to set the specific unit of gas production for the purpose of forecasting future digester gas production. The mean value of recent laboratory test results (obtained from January to March 1982) and the plant running data (obtained in July 1982) was taken as the specific unit of gas production (0.85 Nm^3/kg-VTS).

Both daily and hourly variations of digester gas production were small, suggesting that digestion was stable. Digester gas was mainly composed of 64 to 66% CH_4 and 33 to 35% CO_2, and its chemical composition was almost free from seasonal change.

4. HEAT BALANCE OF DIGESTER

To clarity the relationship between gas engine waste heat and the energy needed for heating the digester, the heat balance of the existing digester was analyzed. The data used for analysis were obtained from April 1979 to March 1982.

At the Naha plant, the digester is heated by direct injection of steam. The construction of the digester and the type and capacity of the heating boiler are as given in Fig. 11 and Table 4 respectively. At present, one primary digester and one secondary digester are in use.

The flow of heat energy from the heating boiler to the digester is as shown in Fig. 12.

Annual changes in ambient temperature and sludge temperature were as shown in Fig. 13. The mean heat balance in summer (july to September) and winter (January to March) was as shown in Fig. 14. Heat input to the boilder was 15 x 10^5 kcal/day to 25 x 10^5 kcal/day in summer and 40 x 10^5 kcal/day to 55 x 10^5 kcal/day in winter, and 74 to 90% of it worked effectively in heating the primary digester. However, heat carried into the primary digester by thickened sludge was 50 x 10^5 kcal/day to 70 x 10^5 kcal/day because the sludge temperature was as high as 20 to 30°C throughout the year, surpassing the heat delivered from the boiler. Heat loss of the boiler was 10 to 26% of heat input to the boiler, and heat loss of the piping was about 1% because its length is extremely short. Radiation heat loss from the primary digester was 0 to 56%.

Table 4. Type and capacity of heating boiler

Type	Flue smoke tube boiler
Max. service pressure	7 kg/cm2
Equivalent evaporation	3,600 kg/hr.
Feedwater	Tap water
Fuel	Digester gas and heavy oil
Normal steam pressure	3.5 kg/cm2

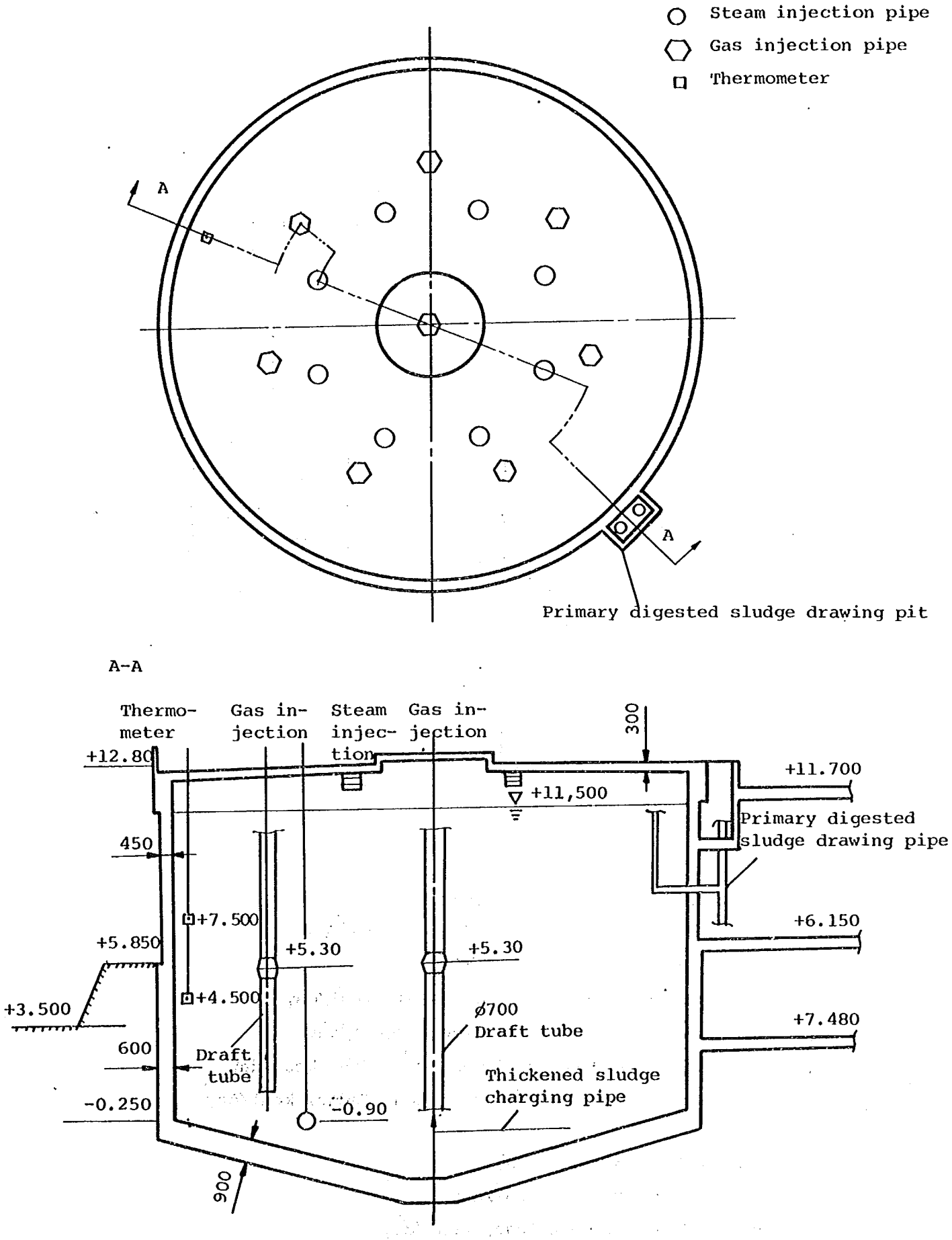
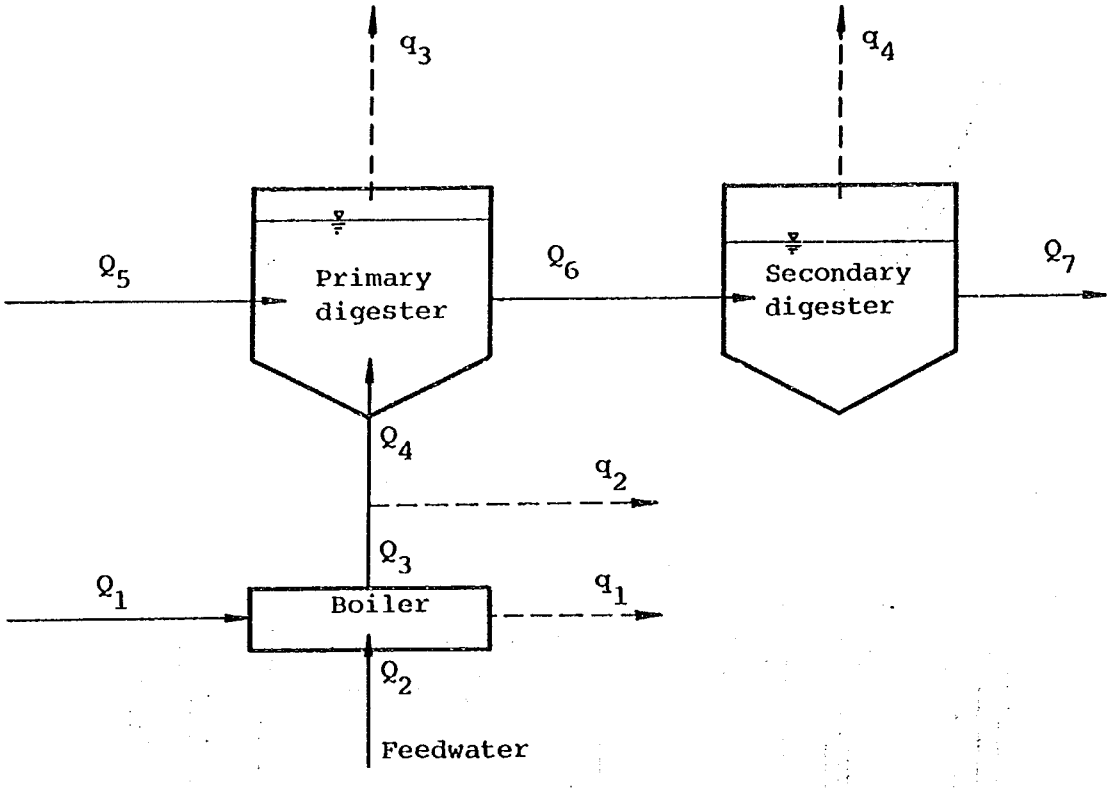


Fig. 11 Construction of digester

As for the heat loss from the primary digester, the value based upon the heat balance and the value estimated by using the coefficient of heat transfer were compared. The average of the former during the two-year period was 7.8×10^5 kcal/day, and the latter was 9.7×10^5 kcal/day, a little over the former.

The estimated value was used to calculate the heat input necessary for heating the digester because there was no significant difference between two values when compared on an average value basis and because the estimated value was a little larger than the other.



Q_1: Heat input to boiler, kcal/day
Q_2: Heat retained in boilder feedwater, kcal/day
Q_3: Heat input to digester, kcal/day
Q_4: Effective heat for heating digester, kcal/day
Q_5: Heat retained in thickened sludge, kcal/day
Q_6: Heat retained in primary digested sludge, kcal/day
Q_7: Heat retained in secondary digested sludge, kcal/day

q_1: Boiler heat loss, kcal/day
q_2: Steam pipe heat loss, kcal/day
q_3: Primary digester heat loss, kcal/day
q_4: Secondary digester heat loss, kcal/day

Fig. 12 Heat energy flow

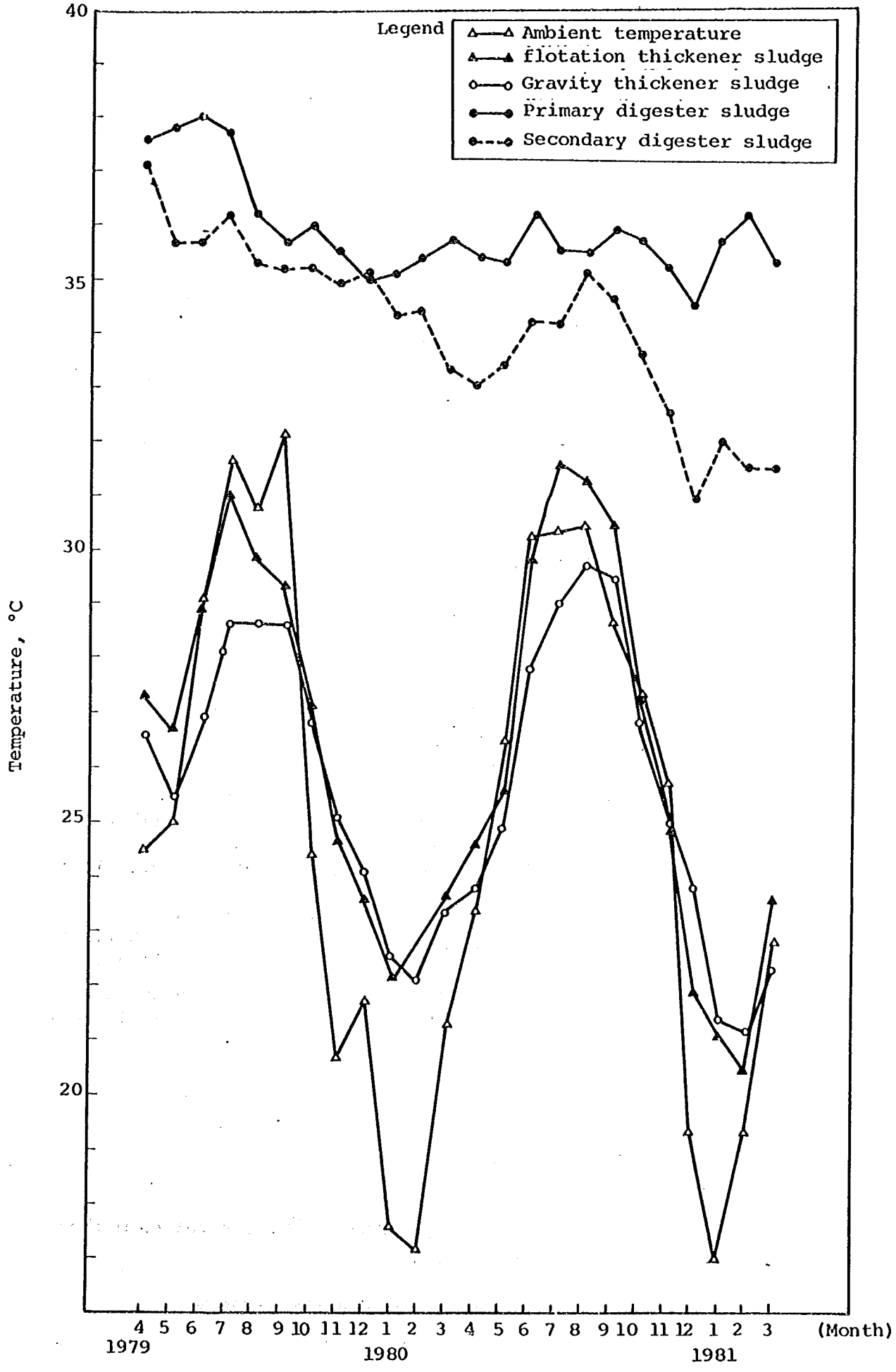
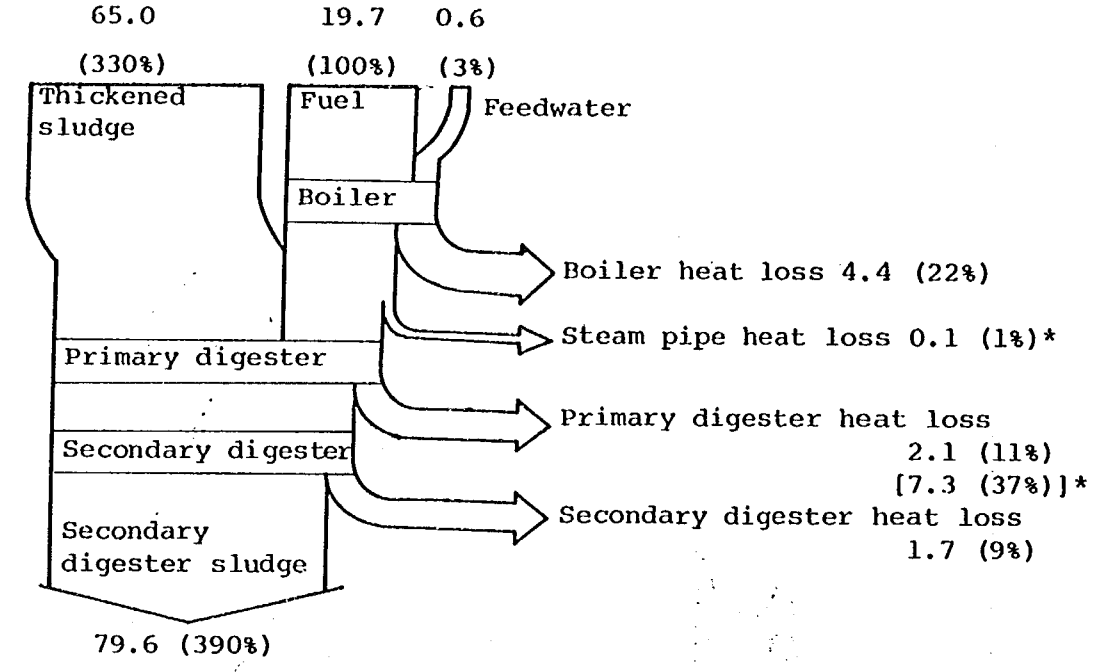
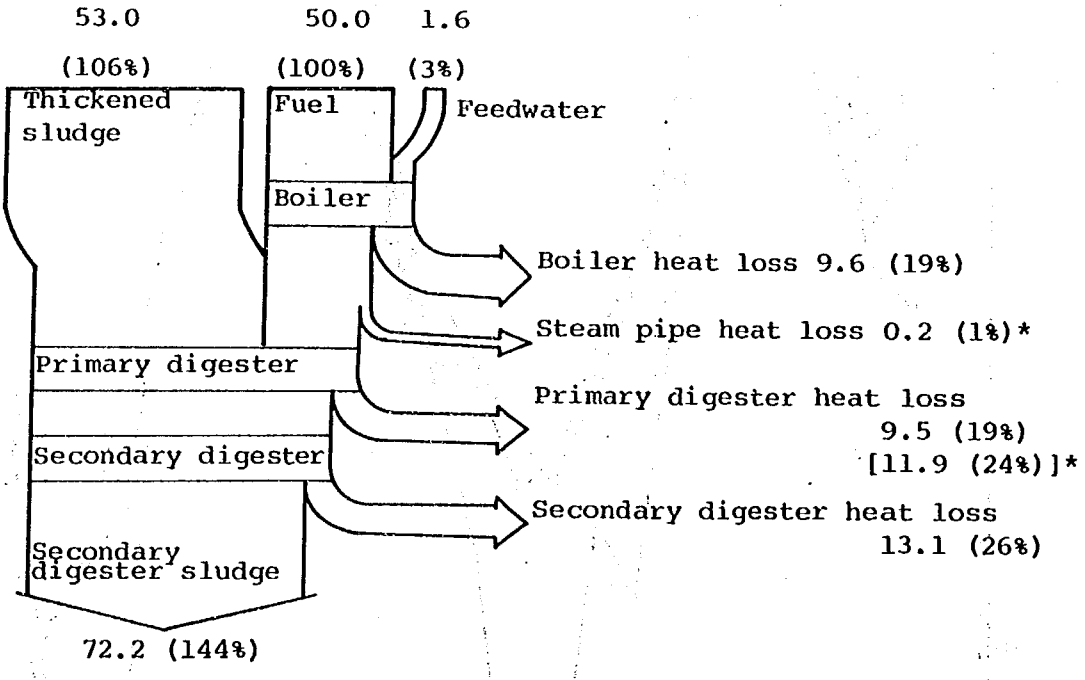


Fig. 13 Seasonal changes in ambient temperature and sludge temperature



(a) Summer (July-September)



(b) Winter (January-March)

Note: Unit: 10^5 kcal; standard temperature, $0^{\circ}C$; the values marked with an asterisk denote estimates.

Fig. 14 Heat balance of digester system

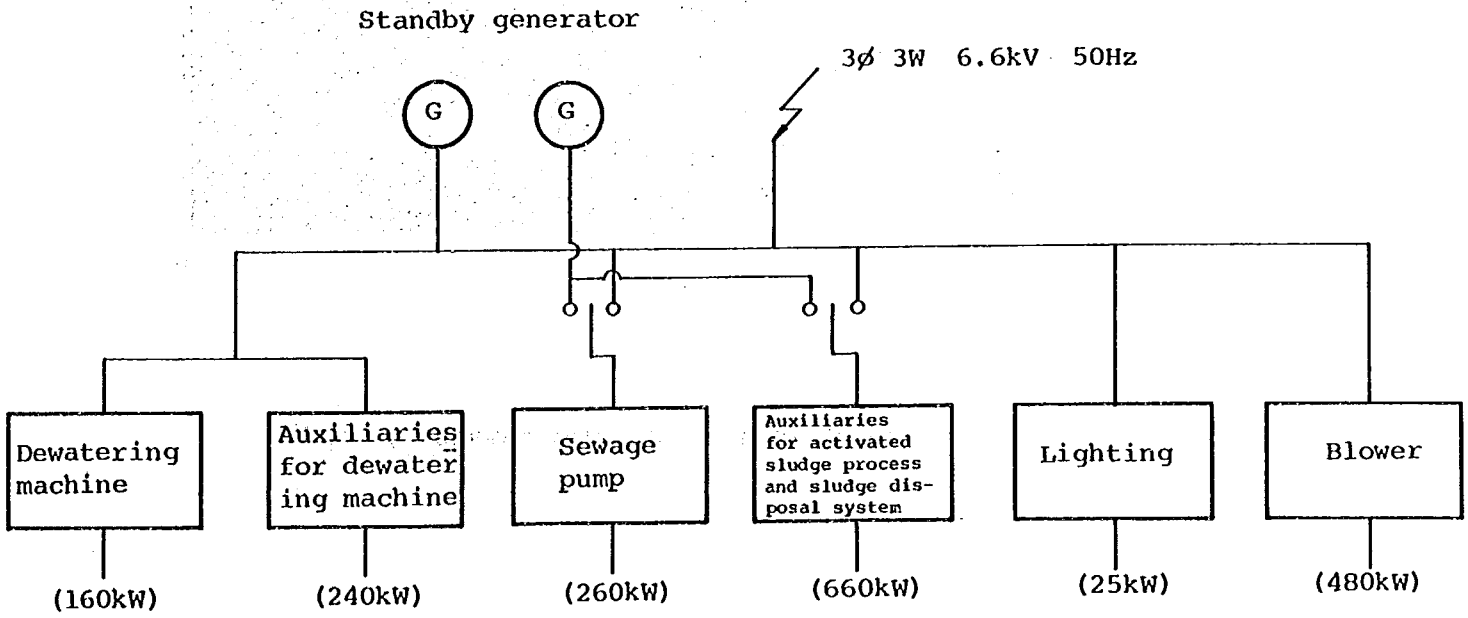
5. SURVEY OF IN-PLANT ELECTRIC POWER CONSUMPTION

Electric power consumptions at the Naha plant was measured to determine the capacity requirement for the digester gas power generation system. The in-plant power distribution system is as shown in Fig. 15. At present, the plant has an installed capacity of about 2,000 kW, and is supplied from a commercial line at a voltage of 6.6 kV for a contract demand capacity of 1,450 kW.

It was found that total power consumption varied from 15,000 to 22,000 kWh/day. Total power consumption was closely related to use of blowers, but was little affected by the change in influent flow. And the blower's power consumption accounts for 50% of plant demand. Total power consumption per unit volume of influent flow was 0.32 to 0.33 kWh/m^3.

Power consumptions of each facility were measured for 24 hours. Some of the data obtained are as shown in Fig. 16. Total hourly power consumption, ranged from 800 to 1,100 kWh/Hr, was high during the day and low at night. Because dewatering machines are operated during the day and sewage pumps work more frequently during the day than at night. Total power consumption did not go down conspicuously at night because the blower was running around the clock.

From the one-day survey of load-wise, in-plant power demand, it was found that the loads to be operated on the digester gas generator independent of commercial power supply should be chosen the blower and the auxiliaries for activated sludge treatment process and sludge disposal process, because their power demands are relatively constant.



Note: Parenthesized values are equipment capacities by system.

Fig. 15 Power distribution system

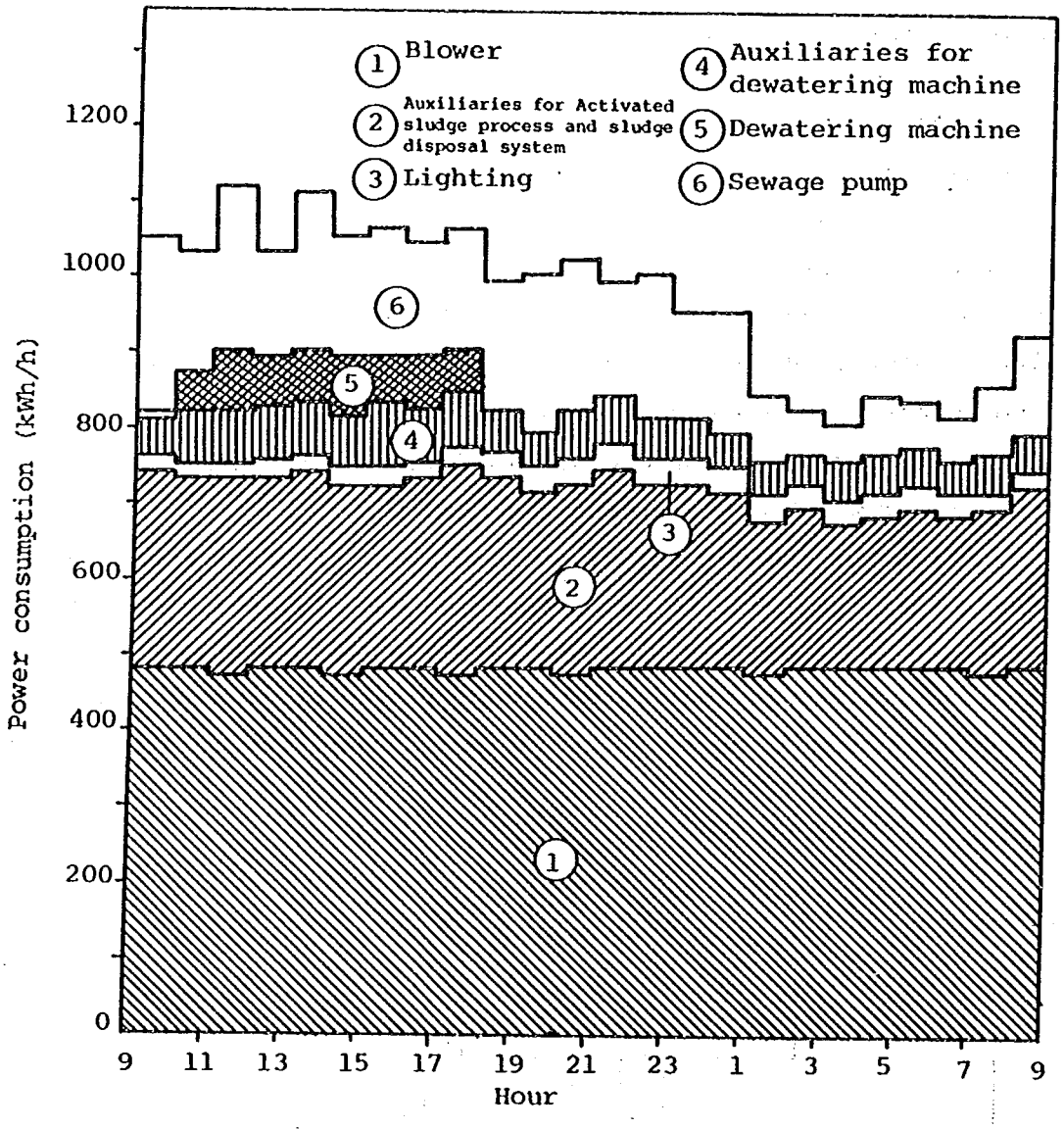


Fig. 16 Hourly change in power demand by load

6. PERFORMANCE STUDY OF GAS ENGINE

Several gas engines were taken to Okinawa for performance tests. Specifications of the engines tested are as shown in Table 5. The low calorific value of the digester gas at the time of test was 5,900 to 6,100 kcal/Nm^3 after scrubbing.

The basic performance characteristics of the gas engines at rated speeds were as shown in Fig. 17. The thermal efficiency under full-load condition was 31 to 36%, and that under fractional loads was lower in the naturally aspirated engines than in the turbocharged ones.

Exhaust temperatures were 400 to 600°C, and changes in exhaust temperature due to changes in load factor were small with the exception of engine (G). There was no significant difference in performance between the engines due to differences in ignition systems, but the dual-fuel engines required heavy oil as pilot fuel equivalent to 10 to 15% of heat consumption.

Table 5. Principal particulars of gas engines tested

Item \ Engine	A	B	C	D	E	F	G
Number and arrangement of cylinders	12, Vee	6, in-line	6, in-line	6, in-line	6, in-line	6, in-line	6, in-line
Rated Output/speed (ps/rpm)	200/1500, 240/1800	200/1200, 300/1800	200/2200	200/1200	210/1200	300/1200	120/1000, 145/1200
Net mean effective pressure (kg/cm^2)	5.74	8.58	11.48	7.35	5.02	9.32	5.65, 5.69
Dry weight (kg)	1150	1990	590	2500	2950	3120	1900
Weight-to-output ratio (kg/ps)	5.75, 4.79	9.05, 6.03	2.95	12.05	14.05	10.4	15.83, 13.10
Aspiration	Natural	Turbocharged	Turbocharged	Natural	Natural	Turbocharged	Natural
Ignition	Spark-ignited	Spark-ignited	Spark-ignited	Spark-ignited	Spark-ignited	Compression-ignited	Compression-ignited
Cooling	Forced-circulated water-cooled	Forced-circulated water-cooled	Forced-circulated water-cooled	Forced-circulated water-cooled	Forced-circulated water-cooled	Forced-circulated water-cooled	Forced-circulated water-cooled

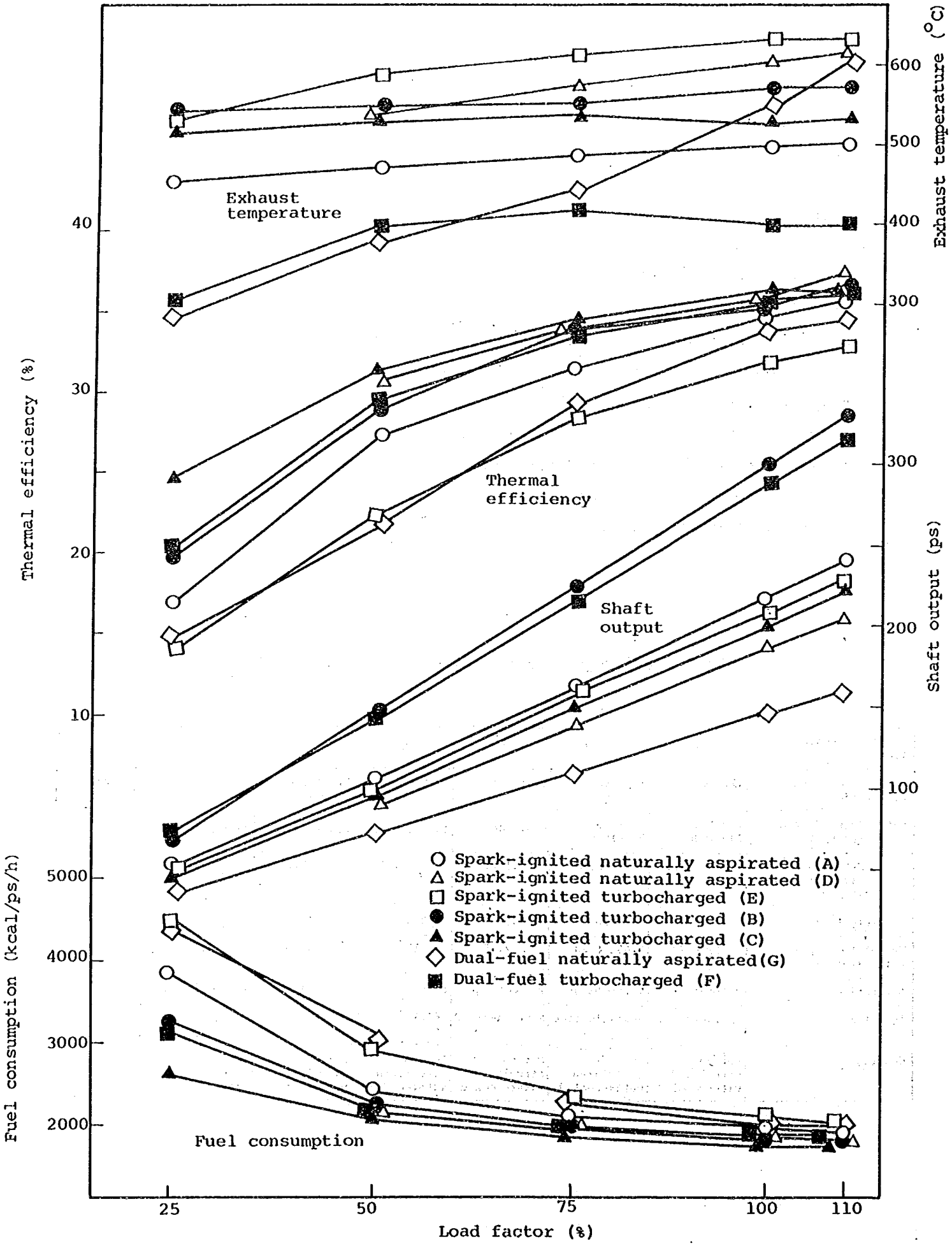


Fig. 17 Engine performance at rated speed

In the dual-fuel gas engines, both the gas running mode and diesel running mode showed almost the same thermal efficiency and the same exhaust temperature under 100% load. But at a low load, the diesel running mode exhibited a higher thermal efficiency and a lower exhaust temperature than the gas running mode.

The heat balance in the gas engine was as follows: Heat input to engine = 100%; shaft output, 31 to 36%; exhaust gas, 25 to 32%; engine cooling water, 27 to 34%; vibration, noise, and radiation, 2 to 13%.

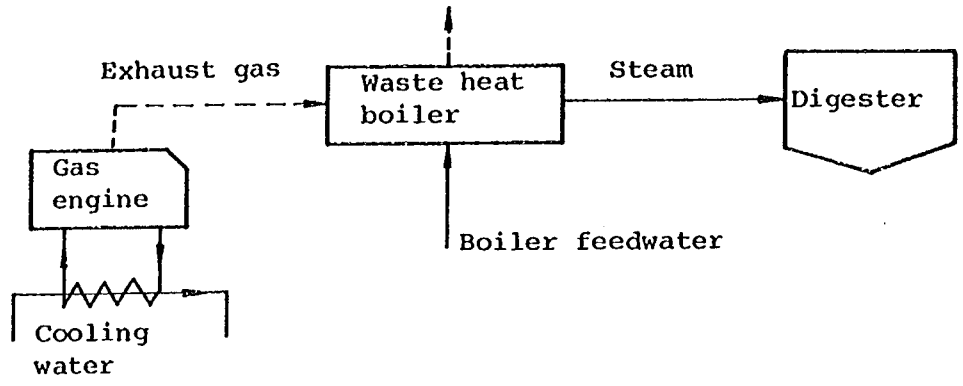
Dynamic characteristics and emission tests were also done on the engines. It was found that the engines show satisfactory dynamic characteristics for driving a generator, with the exception that the turbocharged engines cannot suddently be loaded 100%. NOx concentration in the exhaust gas was as high as 1,000 to 2.000 ppm. In Japan, there are no regulations of exhaust gas for power facilities using internal combustion engines, so this high NOx concentration is not now significant.

7. STUDY OF DIGESTER GAS POWER GENERATION SYSTEM

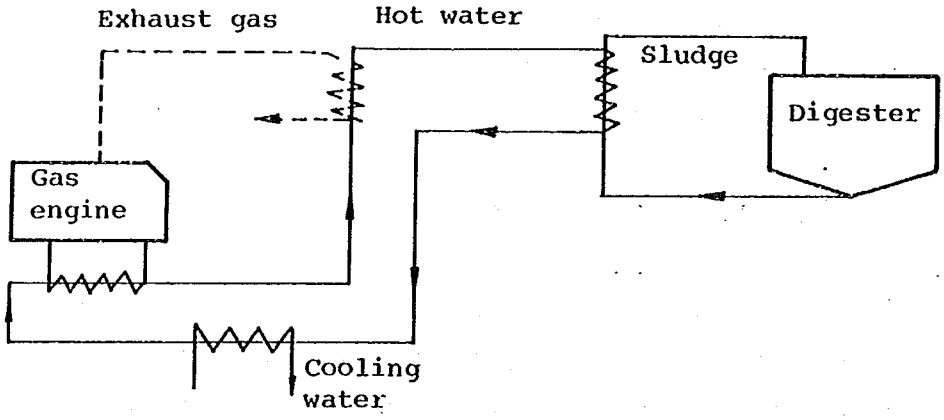
The present and future production of digester gas, the heat required for heating the digester and in-plant power consumption were calculated based upon the studies of sludge treatment process and in-plant power demands. Future plans call for a second train of digesters at Naha, so the design load was set to accommodate extra demand.

Digester gas production and the energy necessary for heating the digester are as shown in Fig. 18. From these, the heat balance was studied for each of the methods of recovering waste heat from the gas engine. Two methods were studied: steam recovery and hot water recovery. Potential applications of waste heat other than heating the digester were excluded. A flow sheet of the waste heat recovery system is shown in Fig. 19.

According to the results of the gas engine performance study, the waste heat recovery rate was set. With a spark-ignited gas engine, it is calculated that recovered waste is about 16% of energy input to the gas engine with a steam recovery system, and about 40% with a hot water recovery system.



(a) Steam recovery system



(b) Hot water recovery system

Fig. 19 Waste heat recovery systems

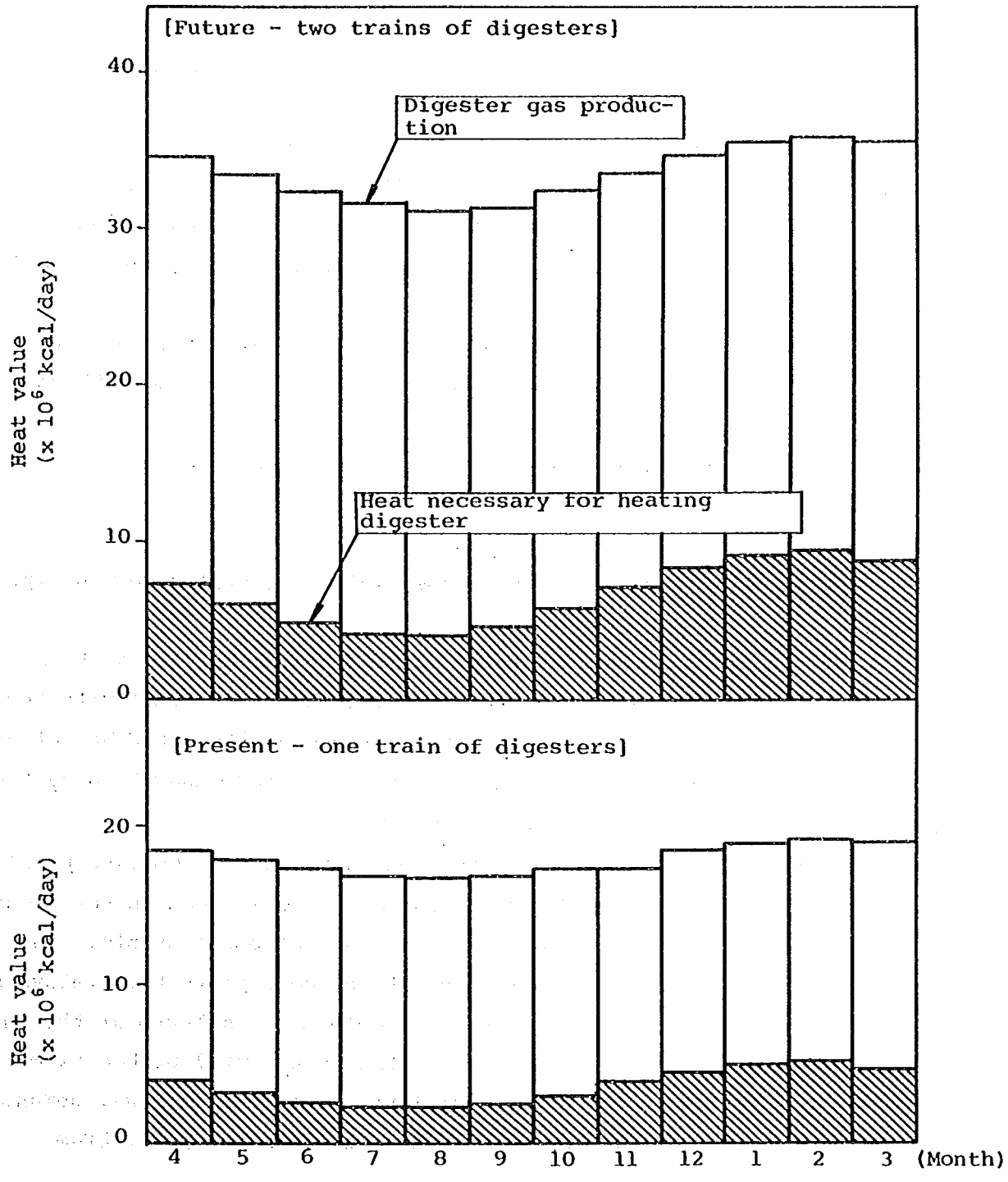
A spark-ignited gas engine system was implemented for these reasons:

(1) There is no significant difference in thermal efficiency between the spark-ignited engine and dual-fuel engine.

(2) The dual-fuel engine always consumes heavy oil, making power costs a little higher than the spark-ignited engine.

(3) The commercial power supply is consistent.

(4) The Naha plant has already been equipped with a standby power plant, and either the commercial power supply or standby power plant can back up the digester gas power generation system when digester gas runs short.



Note: Digester gas production is calculated with the lower calorific value set at 5,500 kcal/Nm^3.

Fig. 18 Digester gas production and heat necessary for heating digester

The relationship between the energy necessary for heating the digester and waste heat recovery is shown in Fig. 20. In the steam recovery system, the existing heating boiler must be run in all seasons but summer.

In the hot water recovery system, however, the digester can be heated with waste heat alone throughout the year. With the steam recovery system, digester gas available for the power generation system was calculated on the condition that the existing heating boiler would be fueled with digester gas.

The use of electric power available from the digester gas power generation system was studied with respect to the following methods:

(1) Running the digester gas power generation system in parallel with commercial power supply.

(2) Running the digester gas independent of commercial power supply.

With the first method, there is no need to identify loads to be borne by the digester gas generator, because it shares loads with the commercial power supply. Thus the gas engine capacity could be set at any size. The only problem is that the digester gas generation system may deliver power to the commercial system.

In the past, power companies in Japan have rarely approved parallel running of a privately owned power plant with their lines on the grounds that they must ensure the stability of commercial power supply. In recent years, however, the development of advanced protective relays has helped solve this problem. If the first method is employed at the Naha plant, there will be little possibility that power will be fed from the digester gas system to the commercial line, because firm power demand at the Naha plant (about 800 kW) is more greater than the power from digester gas system.

With the second method, loads are limited by the power capacity of the digester gas generation system. At the Naha plant, the power demands of the blower and the auxiliaries of the activated sludge process and sludge disposal system are stable, which are proper for an independent digester gas generation system.

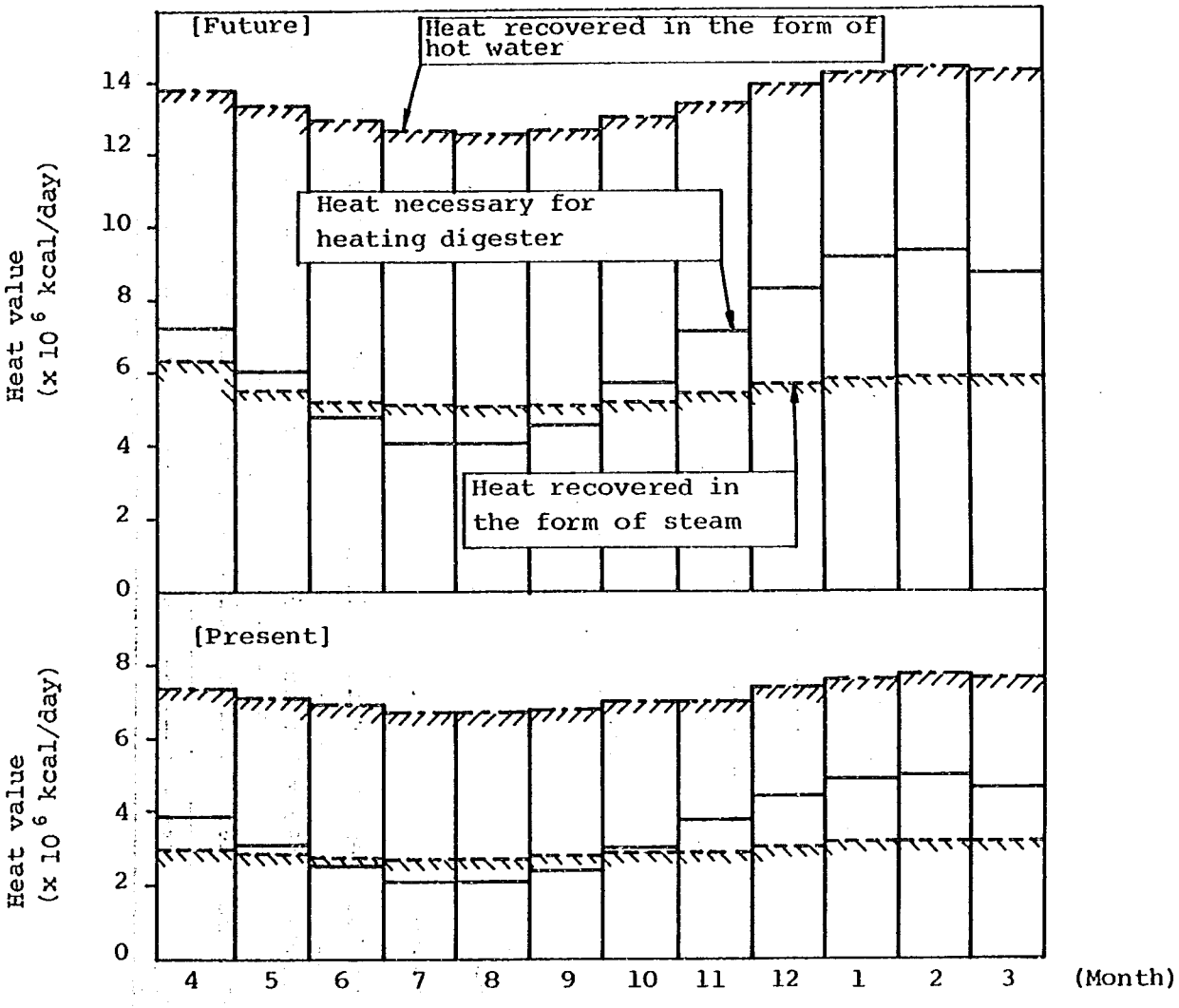


Fig. 20 Waste heat recovery vs. heat necessary for heating digester

Table 6. Principal particulars of digester gas power generation systems for which cost analysis was made

Case No.		1	2	3	4
Waste heat recovery method		Steam recovery	Steam recovery	Hot water recovery	Hot water recovery
Electric power use		Independent	Parallel running with commercial line	Independent	Parallel running with commercial line
Potential gas production (Nm^3/day)	Present	2,830 - 3,140		3,030 - 3,450	
	Future	5,270 - 5,880		5,670 - 6,460	
Gas engine capacity (ps)	Present	490 x 1 unit	370 x 1 unit	490 x 1 unit	410 x 1 unit
	Future	490 x 2 units	370 x 2 units	490 x 2 units	410 x 2 units
Generator capacity (kW)	Present	260 x 1 unit	250 x 1 unit	260 x 1 unit	270 x 1 unit
	Future	260 x 2 units	250 x 2 units	260 x 2 units	270 x 2 units

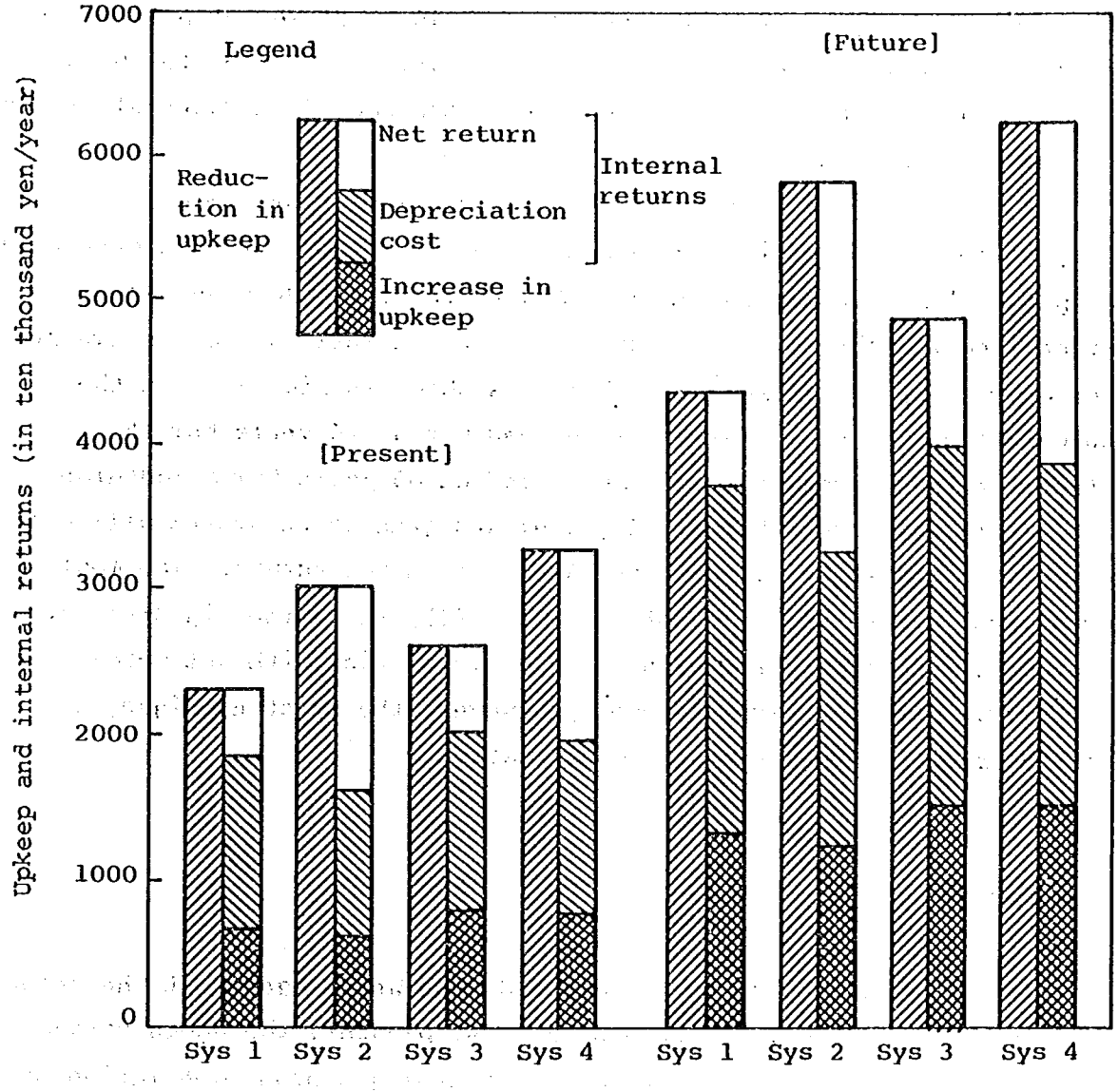


Fig. 21 Upkeep and internal returns as classified by systems (cases)

Each blower is rated at 240 kW, which is almost equal to potential output of the digester gas system. The auxiliaries have an aggregate installed capacity of some 660 kW.

In the study project, therefore, the 240 kW Blower alone was taken as the load for a digester gas generation system. Driving the blower or other loads directly with a gas engine is considered impractical because of the modification work that would be required.

A cost analysis of the digester gas generation system was made with respect to two gas engine waste heat recovery methods and two generator output utilization methods (four cases in all). These cases are summarized in Table 6. For each case, the internal rate of return and the depreciation cost were calculated as shown in Fig. 21. In the calculation, the depreciation period was set at 15 years for the electrical and mechanical equipment, and at 45 years for structures.

It is estimated that any digester gas generation system will cut electric bills 14 to 20% now and 15 to 24% in the future. The most economical system is represented by Case (4), where the digester gas system is run in parallel with the commercial line with hot water recovery. With hot water recovery, however, the existing digester heating facility requires some modification.

8. POSTSCRIPT

Study was conducted by the Japan Sewage Works Agency at the request of the Okinawa Prefectural Government. At present, the Japan Sewage Works Agency has been blueprinting a digester gas power generation system for the Naha Sewage Treatment Plant, and its construction is scheduled to start in FY1982. As negotiations with the power company are well under way, the parallel-running scheme is likely to be permitted.

To close this paper, we would like to express many thanks to the officers and engineering officials of the Sewage Works Division of the Okinawa government and of the Naha plant for their unlimited cooperation extended to us during our stay at Naha, and also to the officers of the Public Works Institute of the Ministry of Construction, with whom we have discussed many problems.

SESSION 5

NITROGEN AND PHOSPHORUS REMOVAL

Study of Nitrified Liquor Recycled Biological Nitrification-Denitrification Process
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STUDY OF NITRIFIED LIQUOR RECYCLED BIOLOGICAL NITRIFICATION-DENITRIFICATION PROCESS

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Research and Technology Development Division,
Japan Sewage Works Agency

1. INTRODUCTION

Marked economic progress from the latter half of the 1950s and the simultaneous mushrooming growthes of large cities have brought the increase of water consumption and wastewater discharge in Japan. Pollutants that run into public waters have diversified and increased in volume. The pollution of public waters affect public health and create significant problems for the manufacturing, agriculture, fishing and industries.

"Red tides" in the Seto Inland Sea and Osaka Bay, and eutrophication in landlocked waters such as Kasumigaura, Lake Biwa, Lake Suwa and reservoirs in various parts of the country prove the problems of water pollution are widespread and serious.

Many researchers have reported on the causes of eutrophication, and it is generally accepted that there are many factors which cause eutrophication. The most typical phenomenon of eutrophication, the proliferating of algae, are to be caused by supply of some nutrients, that limit algae growth. Nitrogen compounds and phosphate resulting from human activities are undoubtedly main causes of entrophication.

Against this backdrop, the Shiga Prefectural Government established in 1979, for the first time in japan, entrophication control regulations specifying limits on nitrogen compounds and phosphate in discharged water for the purpose of restoring water quality at Lake Biwa, that is a primary water source for some 13 million people living in Osaka, Kyoto and Kobe.

The regulation emphasize the promotion of sewerage construction and measures to sharply reduce the discharge of nitrogen and phosphorus. The measures address the following three areas:

1. Works discharging significant quantities of nitrogen and phosphorus and obliged to observe effluent limits attainable through the most practical engineering and investment efforts.
2. Synthetic detergents account for 18.2% of total phosphorus loading. Thus, the use and sale of synthetic detergents containing phosphate are banned.
3. It is mandatory to properly manage agricultural waters, animal wastes, garbage and other materials liable to violate effluent limits.

The measures regard sewerage system as most important for the reduction of nitrogen compounds and phosphate that run in Lake Biwa.

As shown in Fig. 1 and Table 1, regional sewerage construction plans have already been decided for areas surrounding the lake, and the sewage treatment plants are required to observe the following effluent standards for nitrogen and phosphorus concentration.

T-N: Up to 10.0 mg/ℓ

T-P: Up to 0.5 mg/ℓ

At the Konan Chubu Sewage Treatment Plant -- the only one built so far as part of the regional sewerage construction projects put into commission since March 1982 -- the nitrified liquor recycled biological nitrification-denitrification process is applied to remove nitrogen, and the coagulation process is applied to remove phosphorus. At the Kosei Sewage Treatment Plant, to be completed in October 1984 to serve the Kosei area, the nitrified liquor recycled biological nitrification-denitrification process will also be used to remove nitrogen.

There have already been many published reports on the nitrified liquor recycled biological nitrification-denitrification process, and many pilot plants have already been operated. In Japan, however, the Konan Chubu Sewage Treatment Plant is the first practical example of this process.

The Japan Sewage Works Agency tested the nitrified liquor recycled biological nitrification-denitrification process on a practical scale to provide design and maintenance data for the Konan Chubu and Kosei plants. This study reports on matters concerning operation and management of the process, design factors and other findings.

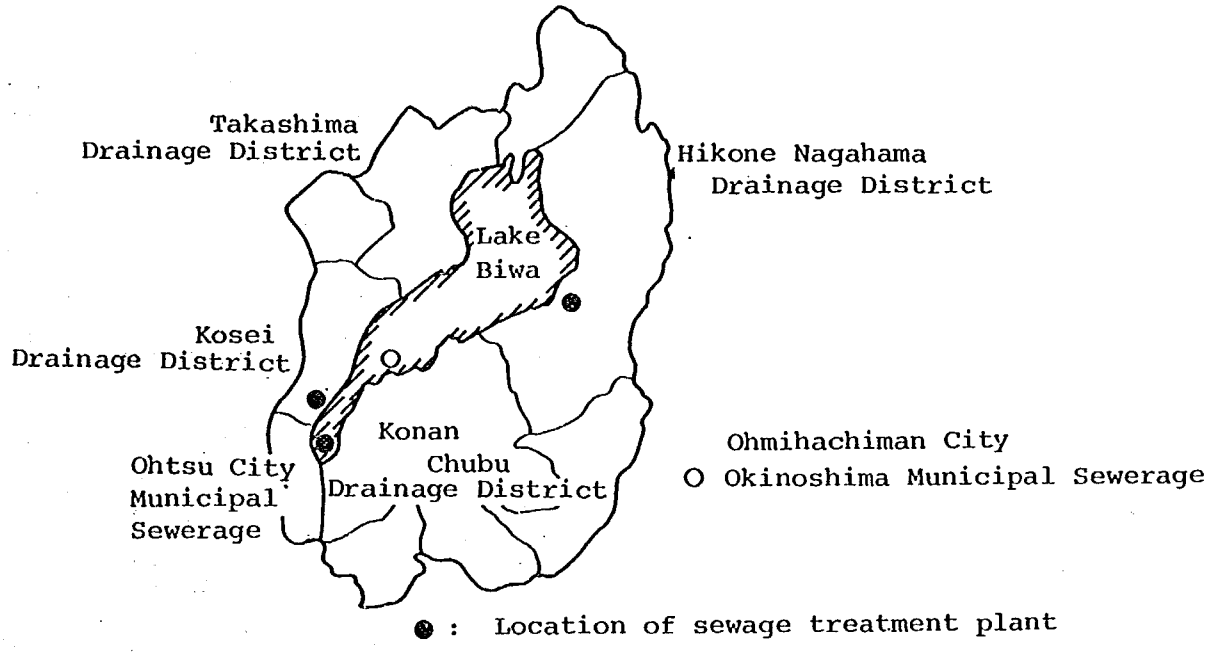


Fig. 1 Map of sewerage construction projects around Lake Biwa

Table 1 Principal particulars of sewerage construction projects around Lake Biwa

Item	Lake Biwa Basin Wide Sewerage				Ohtsu Municipal Sewerage	Okinoshima Municipal Sewerage
	Konan Chubu Drainage District	Hikone Nagahama Drainage District	Kosei Drainage District	Takashima Drainage District		
Planned drainage area	approx. 25,500 ha	approx. 17,700 ha	approx. 2,600 ha	pending	approx. 1,338 ha	approx. 8.7 ha
Planned population coverage	approx. 790,000	approx. 525,000	approx. 250,000	pending	approx. 128,000	approx. 1,000
Planned treatment capacity	approx. 1,020,000 m^3/day	approx. 505,000 m^3/day	approx. 120,000 m^3/day	pending	approx. 95,000 m^3/day	420 m^3/day
Sewerage system	Separate				Partly combined	Separate
Aggregated length of sewers	approx. 148 km	approx. 102 km	approx. 9 km	pending	approx. 283 km	approx. 3.5 km
Number of relay pump stations	6	4	2	pending	3	2
Size of wastewater treatment plant site	approx. 63.7 ha	approx. 64.1 ha	approx. 11.8 ha	pending	approx. 2.9 ha	approx. 0.2 ha
Number of affected local authorities	5 cities and 14 towns	2 cities and 17 towns	1 city and 1 town	5 towns	1 city	1 city

2. NITRIFIED LIQUOR RECYCLED BIOLOGICAL NITRIFICATION-DENITRIFICATION PROCESS

There are many alternative systems for biological nitrification-denitrification, differing in combinations of nitrification and denitrification tanks and the method of recycling. In this paper, the system in which a nitrification and denitrification tank are combined as illustrated in Fig. 2 to recycle nitrified liquor to the denitrification tank is called the nitrified liquor recycled biological nitrification-denitrification process.

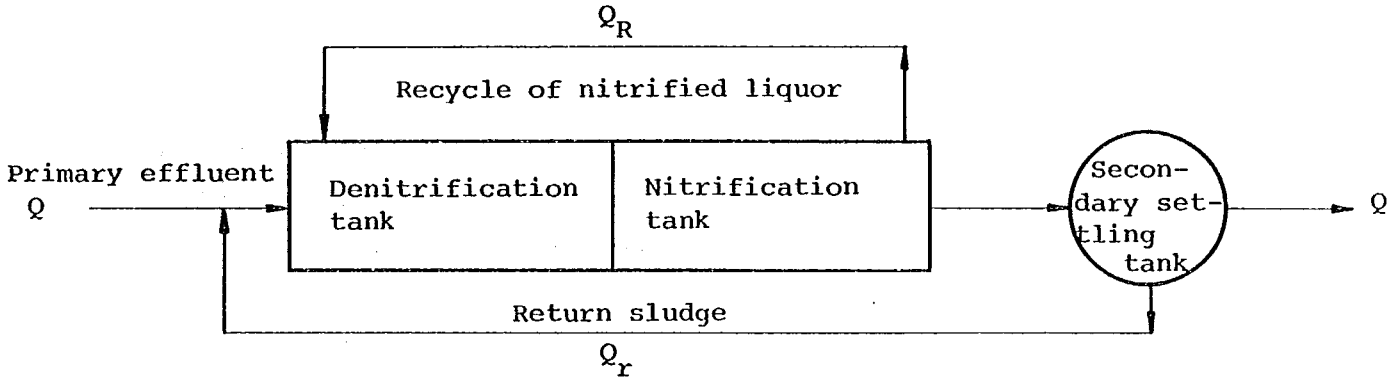


Fig. 2 Flowchart of the nitrified liquor recycled biological nitrification-denitrification process

3. EXPERIMENT FACILITIES

For this experiment of the nitrified liquor recycled biological nitrification-denitrification process, facilities of the Arakawa Sewage Treatment Center are used -- adjacent to the Research and Training Center of the Japan Sewage Works Agency.

The Arakawa Sewage Treatment Center treated about 250,000 m^3/day of sewage as of the end of March 1982. This sewage treatment plant is of a combined sewerage system, and employs the conventional activated sludge process (including deep aeration tanks). The excess sludge is thickened by gravity or centrifuge, added with lime and ferric chloride, filter-pressed for dewatering, and then finally incinerated.

Table 2 shows design factors of the experiment facilities. When operated as a conventional activated sludge process, the experiment facilities have a daily maximum design flow capacity of 17,600 m^3/day.

Table 2 Design factors of the experiment system

	Width m	Length m	Effective water depth, m	Number of tanks	Volume m^3	Remarks
Primary settling tank	4.3	50.0	4.0	2	1,720	
Aeration tank	9.0	85.0	5.0	1	3,825	Quadri-sected by baffle walls
Secondary settling tank	4.3	56.0	4.45	2	2,143	

Return sludge is pumped back to the inlet of the aeration tank. The sludge pump is capable of recycling on continuous duty the sludge equivalent of up to about 50% of daily maximum flow.

These tanks were converted into a nitrified liquor recycled biological nitrification-denitrification process as shown in Fig. 3. The aeration tank is divided into four equal parts with baffle walls. All the diffusers were removed from the first two sections. Instead, the first section (F_1) was equipped with two 11 kW agitators, and the second section with two 15 kW agitators. The agitators in the second section were designed so they could be used as diffusers.

An air supply pipe was installed toward the end of August 1981 and since September 1981 the 15 kW agitators have been used mainly as differs.

During the experiment, F_1 was used as a denitrification tank, while F_2 was used as either a denitrification or nitrification tank. The latter two sections (G_1 and G_2) were left intact, and were used as aerobic nitrification tanks using existing air diffusers.

An 11 kW submerged pump was installed near the outlet of the aeration tank to feed nitrified liquor back to the denitrification tanks. The outlets for this recycled liquor were installed at the inlets of F_1 and F_2, and one of them was chosen according to the operating condition. The return sludge pump at the secondary settling tank was also used to recycle nitrified liquor and sludge.

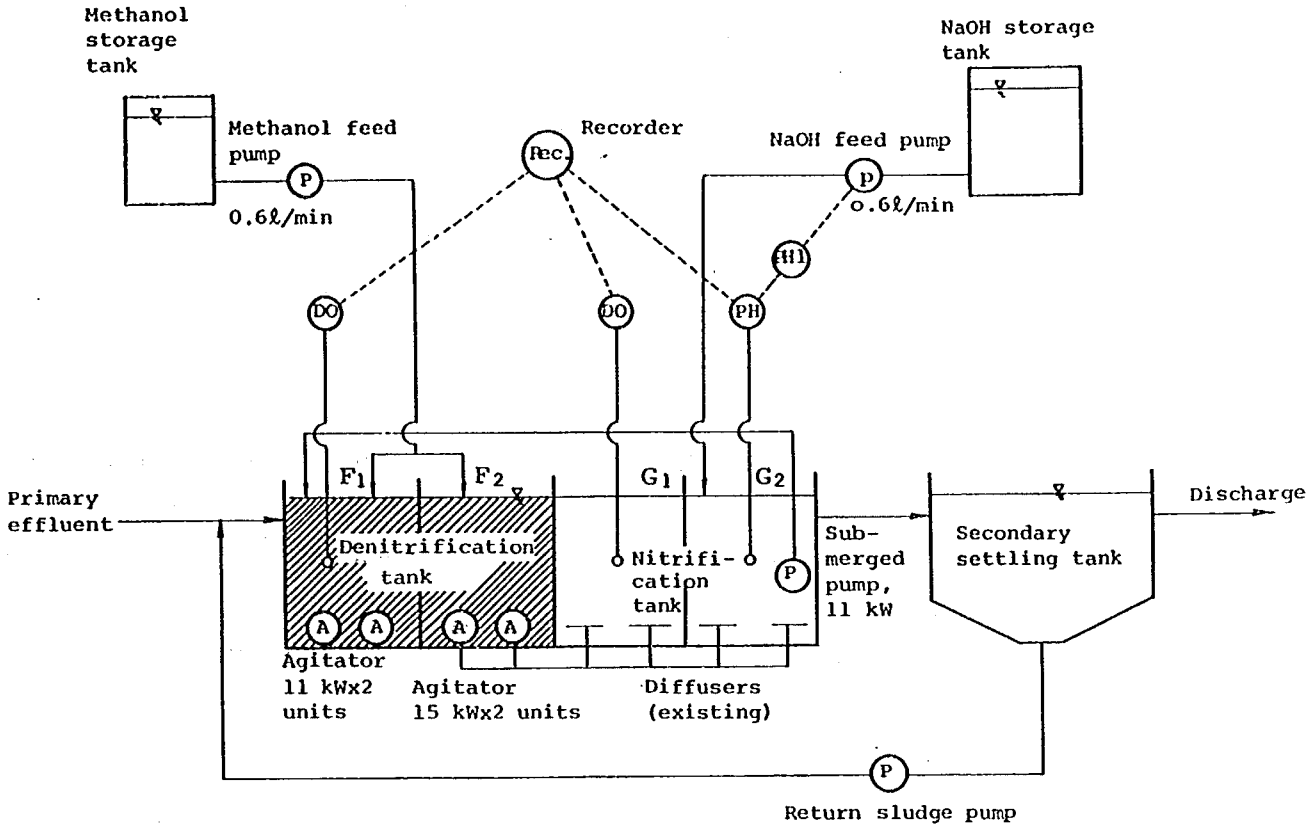


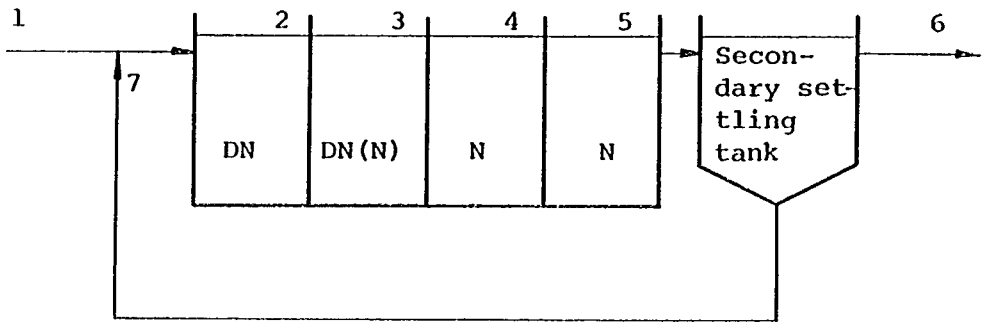
Fig. 3 Flow chart of the experiment system

In addition, a 2 m^3 tank for storing methanol (50%) and methanol injector were installed in order to make up for shortages of organic substrates. Also installed were an 3 m^3 NaOH (24%) storage tank and injector to guard against deficiency in alkalinity in the nitrification tanks.

The DO concentration in each nitrification tank and pH value in G_2 were continuously recorded.

4. WATER QUALITY MEASUREMENT

Water quality measurement was conducted three times a week. The sampling points were as illustrated in Fig. 4.



DN: Denitrification tank
N: Nitrification tank

Fig. 4 Sampling points

At sampling points No.1 (primary settling tank effluent) and No.6 (treated water), samples were taken hourly to make 24-hour composite sample by using composite samplers with cold storage. At other sampling points, grab samples were taken daily at 10:30 a.m. The activated sludge mixed liquor was sampled from the trailing end of each tank.

5. OPERATING CONDITIONS

During the experiment, the inflow, return sludge rate and nitrified liquor recycling rate were held constant unless it was necessary to check the effects of load fluctuation. Nitrified liquor was recycled by the submerged and return sludge pumps. Withdrawal of the excess sludge was usually done in night, and was extracted from the return sludge line.

The experiment period was divided into runs with respect to inflow volume and recycling ratio, as shown in Table 3. The table also shows major operation control indexes for each test run. Of the indexes, MLSS and MLRSS (return sludge MLSS) are the average values of each run. The SAT was calculated according to the following formula using accumulative values obtained during each test run, because the sludge in an aerobic state was expected to play a significant role in nitrification control:

$$\text{SAT (days)} = \frac{\text{(Volume of nitrification tank } (m^3) \times \text{(MLSS (mg/}\ell\text{))}}{\text{(Excess sludge production } (m^3/\text{day})) \times \text{(Excess sludge SS (mg/}\ell\text{)) + (Inflow (m3/day)) x Effluent SS (mg/}\ell\text{))}} \quad \text{... Eq. 1}$$

During Runs 1 = 8, the F_2 tank was anaerobic. Thereafter, F_2 was aerobic.

From the end of November 1981 till early January 1982, the experiment system was laid up for repair. Dates missing between test runs were excluded as transitional.

Table 3 Operating conditions

Run	Period	Q (m^3/d)	Q_r (m^3/d)	Q_R (m^3/d)	Q_r+Q_R (m^3/d)	R (%)	MLSS (mg/ℓ)	SAT (d)	ATDT (hrs)	Denitrification tank/ Nitrification tank
1	3/30 to 4/21	4,840	4,830	0	4,830	100	2,330	26	19.0	1:1
2	4/22 to 5/13	5,870	3,980	0	5,980	102	2,110	33	15.6	1:1
3	5/14 to 6/8	6,010	5,910	5,400	11,310	188	2,580	35	15.3	1:1
4	6/9 to 1/29	6,110	3,160	5,400	8,560	140	1,920	19	15.0	1:1
5	6/30 to 7/13	8,340	8,240	5,400	13,640	164	2,970	24	11.0	1:1
6	7/14 to 7/20	7,260	7,040	5,400	12,440	171	4,110	23	12.6	1:1
7	7/21 to 7/31	7,160	3,820	5,400	9,220	129	3,260	14	12.8	1:1
8	8/7 to 8/27	7,290	3,740	5,700	9,440	130	2,820	15	12.6	1:1
9	8/28 to 9/11	8,210	6,740	5,400	12,140	148	3,140	42	11.2	1:3
10	9/24 to 10/16	9,460	6,920	5,400	12,320	130	3,630	21	9.7	1:3
11	10/17 to 10/26	7,320	7,010	5,400	12,410	170	2,310	29	12.5	1:3
12	11/7 to 11/20	7,040	3,700	5,400	9,100	129	2,270	23	13.0	1:3
13	1/13 to 1/22	4,750	2,620	5,400	8,020	169	3,380	26	19.3	1:3
14	1/23 to 2/5	4,410	3,780	5,400	9,180	208	3,780	8	20.8	1:3
15	2/8 to 2/19	3,630	3,670	5,400	9,070	250	3,370	10	25.2	1:3
16	2/22 to 3/3	3,730	3,610	3,400	9,010	241	1,700	21	24.6	1:1
17	3/15 to 3/26	5,260	4,460	5,400	9,860	188	4,310	37	17.5	1:3
18	3/31 to 4/8	6,160	4,730	5,400	10,130	164	4,220	27	14.9	1:3

Notes: Q : Inflow volume
Q_r : Recycling volume by return sludge pump
Q_R : Recycling volume by submerged pump
R : Recycling ratio $(Q_r + Q_R)/Q$
ATDT: Aeration tank detention time on an inflow basis

6. AN OUTLINE OF TREATMENT CONDITIONS

Figs. 5.1 to 5.6 show time-dependent changes of primary settling tank effluent T-N, primary settling tank effluent soluble T-N, treated water T-N, treated water soluble T-N, primary settling tank effluent NH_4^+-N, and treated water NO_2^--N + NO_3^--N.

As these figures show, primary settling tank effluent concentration and water temperature suddenly decrease on occasion. This is because the treatment plant, of a combined sewerage system, is influenced to a degree by influxes of stormwater.

Tables 4.1 and 4.2 show the average, maximum and minimum values of the quality of primary settling tank effluents for respective experiment runs. Tables 5.1 and 5.2 show the average, maximum and minimum values of quality of treated water for respective experiment runs.

Table 6 shows the average removal rates of each water quality indexes for respective experiment runs.

6.1 AN OUTLINE OF TREATMENT CONDITIONS IN SPRING (Runs 1 ~ 4)

The test system got started at the end of March 1981 with Q at 5,000 m^3/d and recycle ratio R at 100% (recycled by the return sludge pump alone). After about a week, the nitrification-denitrification phenomena began.

From Run 3 on, the submerged pump was run with the return sludge pump. As a result, the recycling ratio was increased to 190%, and the average T-N removal rate was improved to 70.8%. From Run 3 on, NH_4^+-N removal rate was also improved, and the amount of NH_4^+-N remaining in treated water was cut to nearly zero.

In Run 4, the recycling ratio by the return sludge pump was reduced to half that in Run 3, and the recycling ratio was reduced to 140%. As a result, the average T-N removal rate dropped to 65.7%.

During Runs 1 ~ 4, SS and organic matters (BOD, COD) were removed quite satisfactorily. During Runs 3 and 4, NH_4^+-N removal rate was nearly 100%, and NH_4^+-N remaining in treated water was almost nil.

Fig 5-(1)

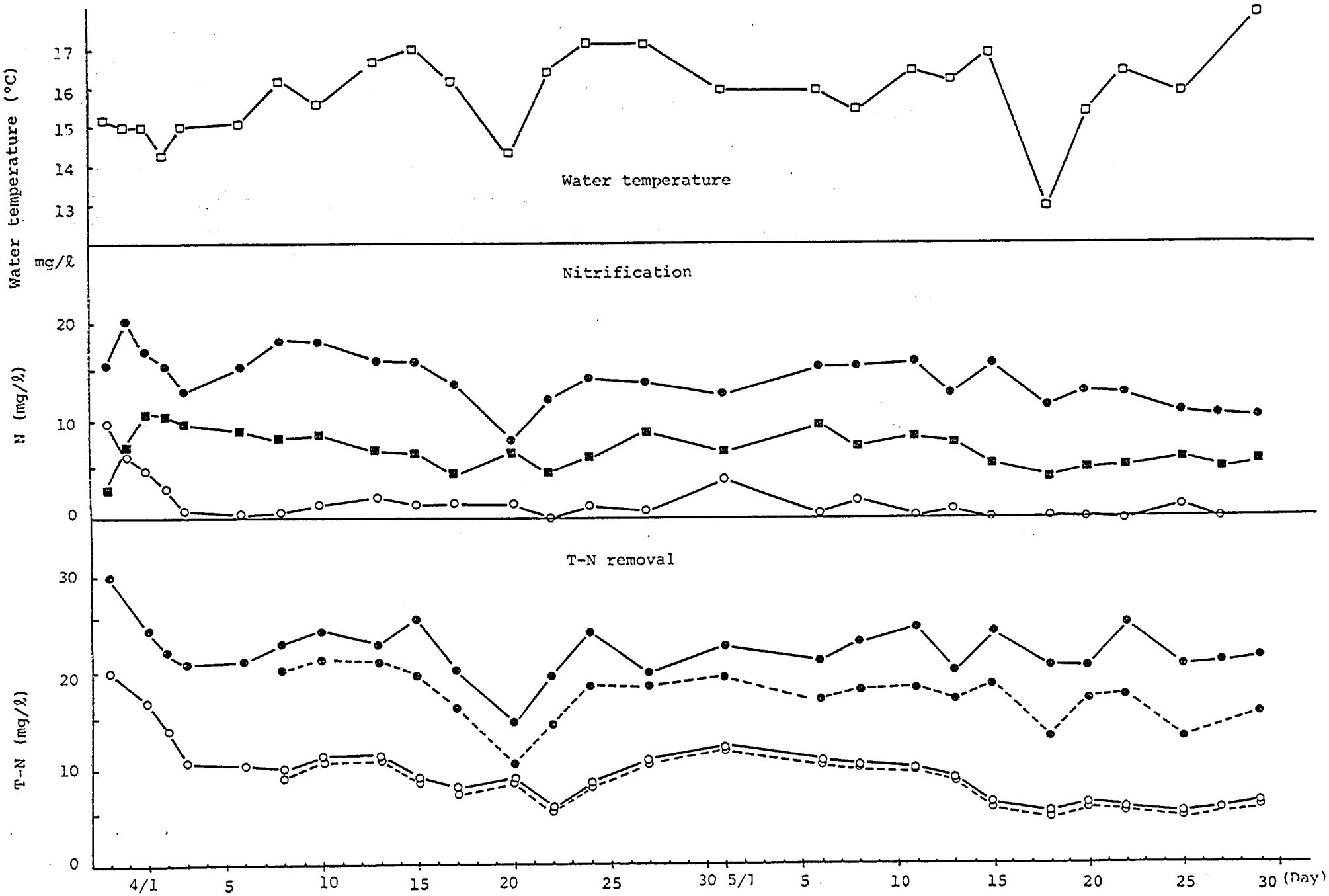

Water temperature (°C)
17
16
15
14
13
Water temperature
mg/ℓ
Nitrification
N (mg/ℓ)
20
10
0
T-N removal
T-N (mg/ℓ)
30
20
10
0
4/1
5
10
15
20
25
30
5/1
5
10
15
20
25
30
(Day)

Fig 5-(2)

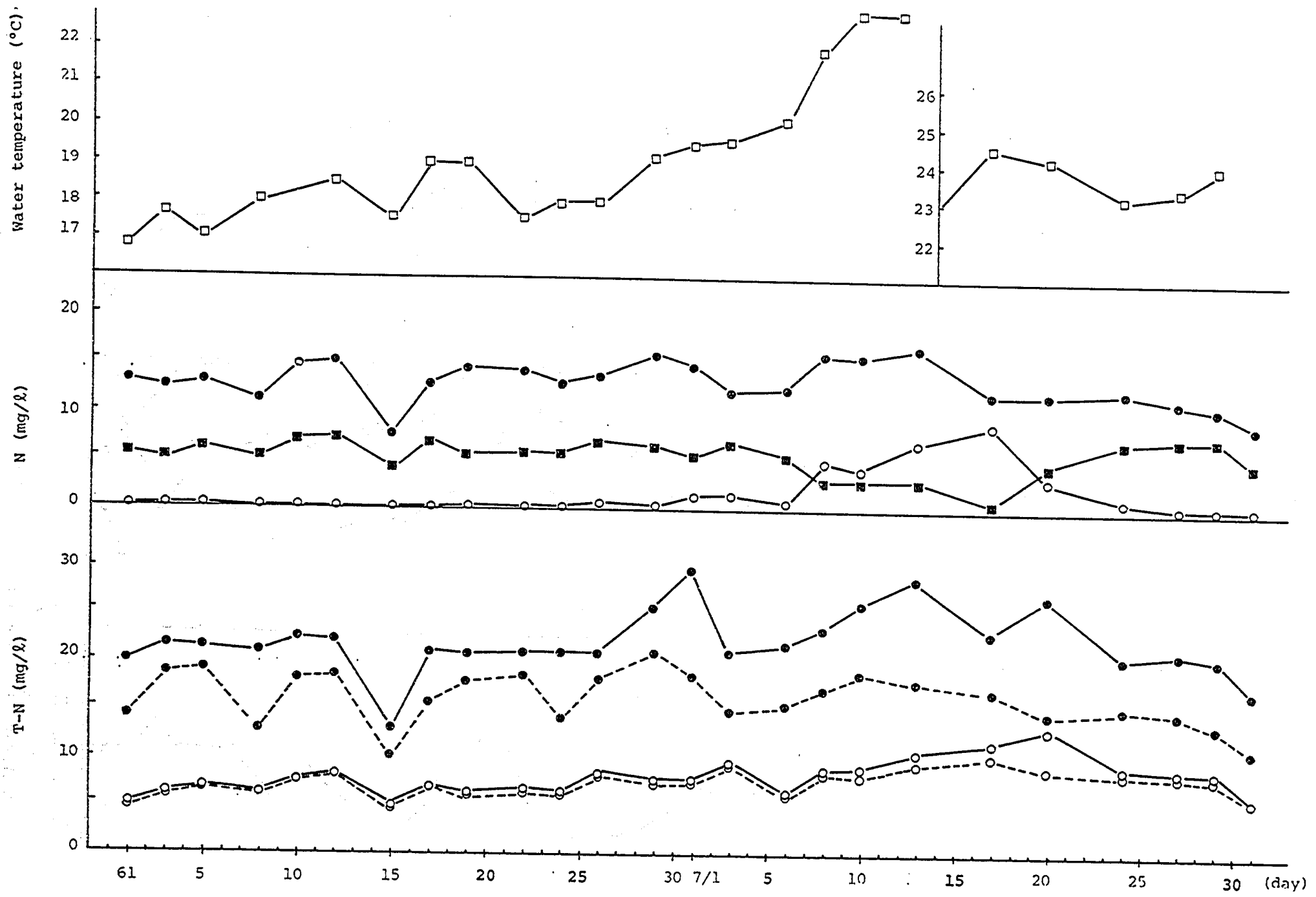

Water temperature (°C)
22
21
20
19
18
17
26
25
24
23
22
N (mg/ℓ)
20
10
0
T-N (mg/ℓ)
30
20
10
0
61
5
10
15
20
25
30
7/1
5
10
15
20
25
30
(day)

Fig. 5-(3)

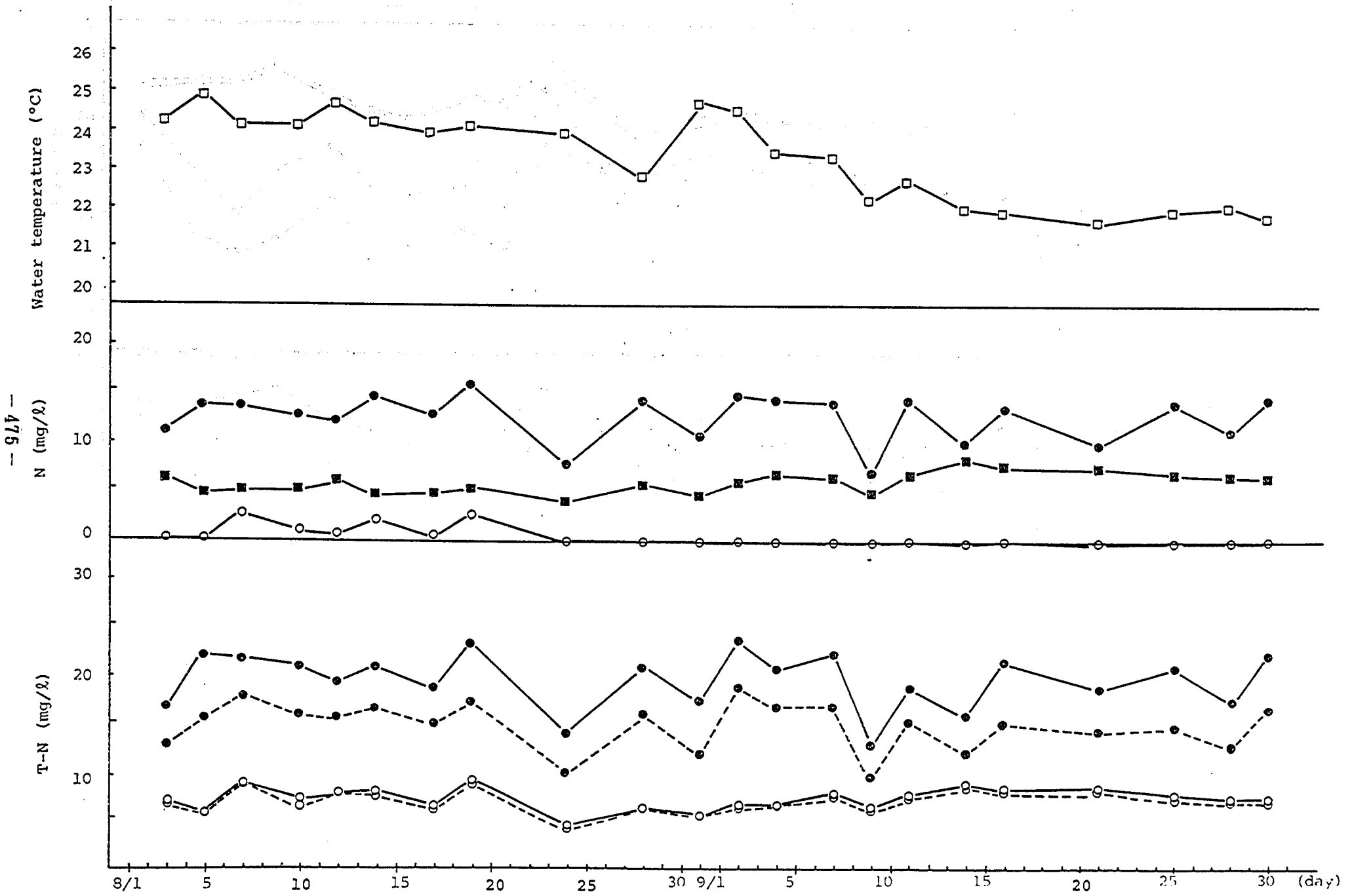

Water temperature (°C)
26
25
24
23
22
21
20
N (mg/ℓ)
20
10
0
T-N (mg/ℓ)
30
20
10
8/1
5
10
15
20
25
30
9/1
5
10
15
20
25
30
(day)

Fig 5-(4)

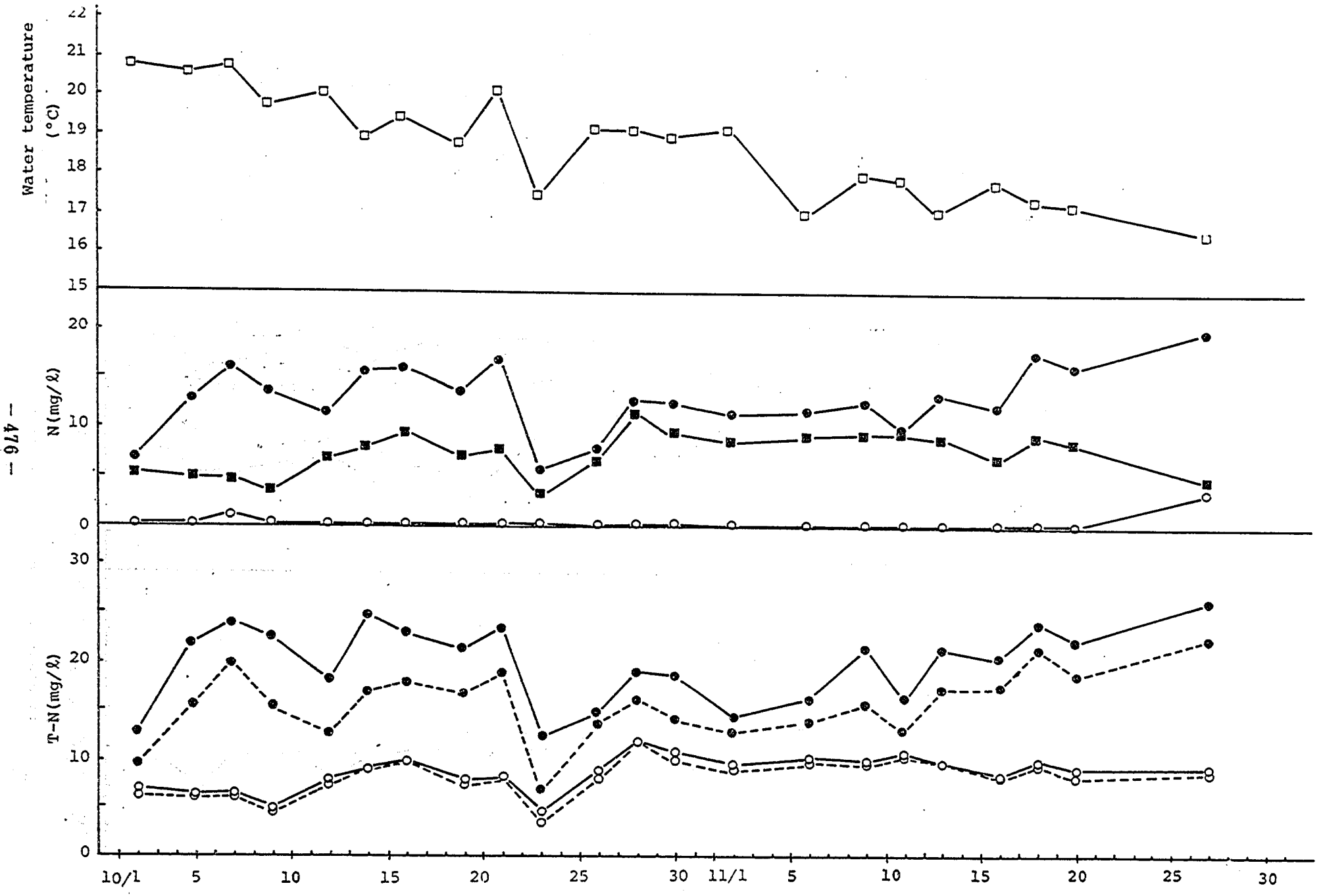

Water temperature (°C)
22
21
20
19
18
17
16
15
N(mg/ℓ)
20
10
0
T-N(mg/ℓ)
30
20
10
0
10/1
5
10
15
20
25
30
11/1
5
10
15
20
25
30

Fig. 5-(5)

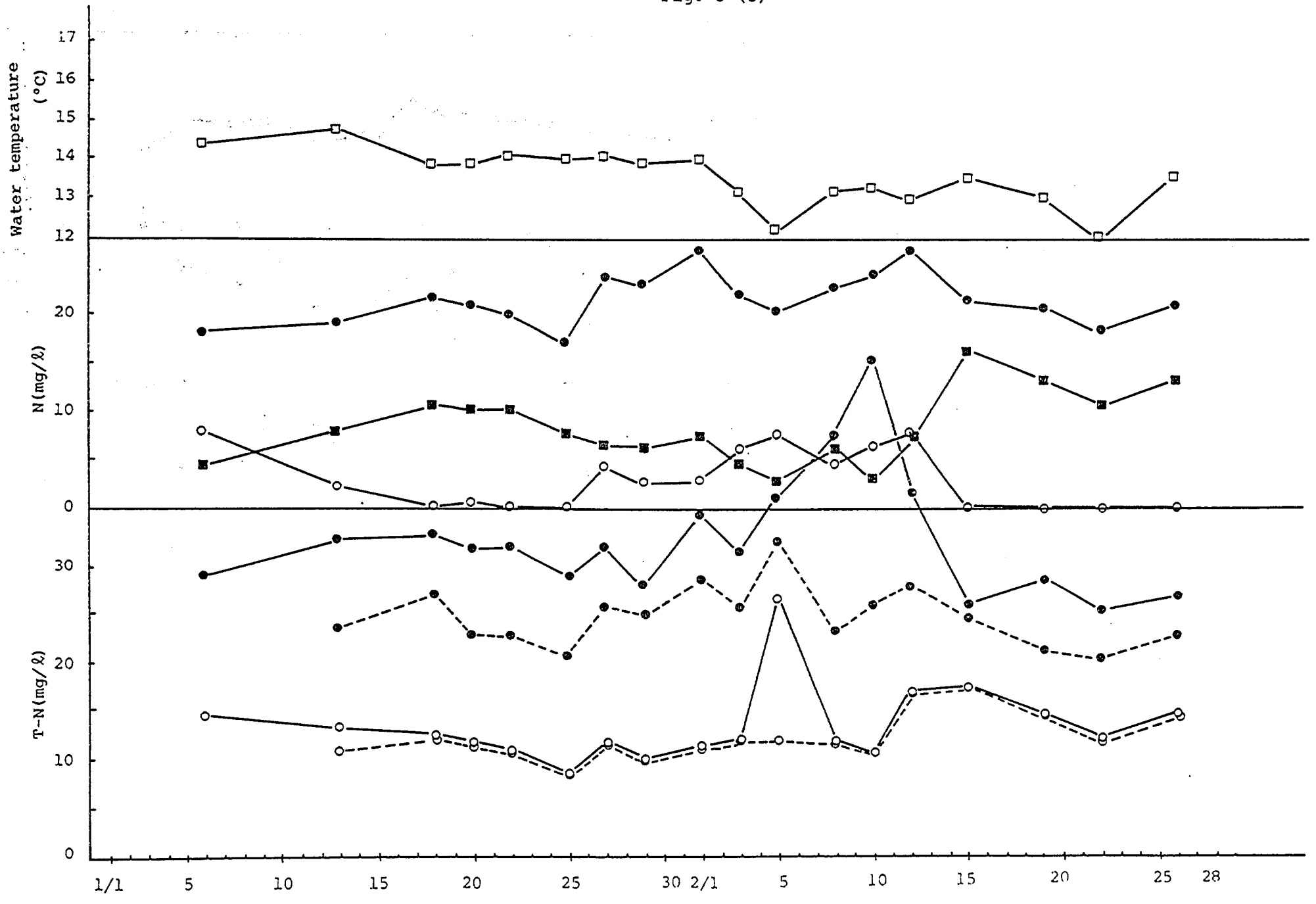

Water temperature (°C)
17
16
15
14
13
12
N(mg/ℓ)
20
10
0
T-N(mg/ℓ)
30
20
10
0
1/1
5
10
15
20
25
30
2/1
5
10
15
20
25
28

Fig. 5-(6)

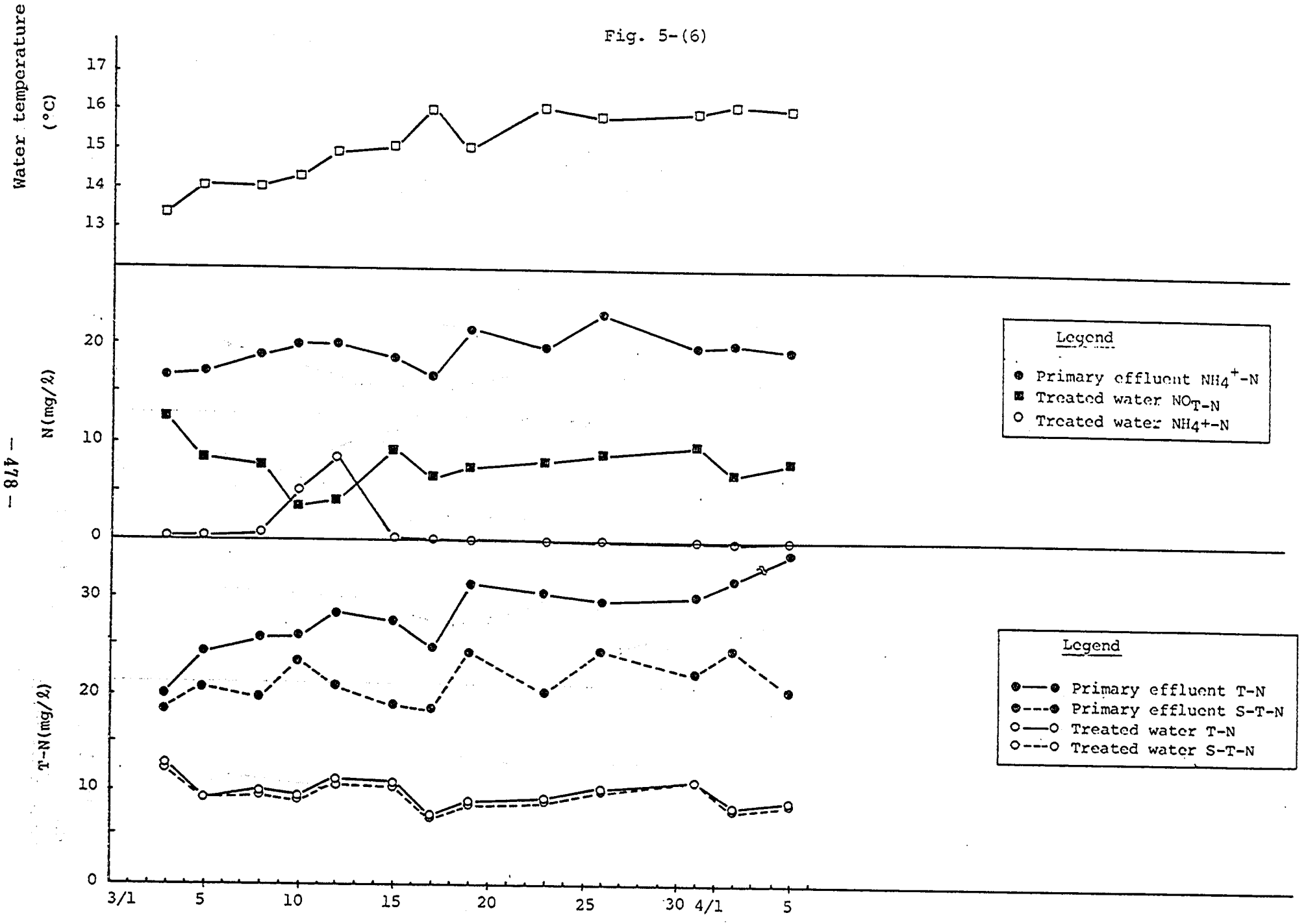

Water temperature (°C)
17
16
15
14
13
N(mg/ℓ)
20
10
0
Legend
Primary effluent NH4+-N
Treated water NOT-N
Treated water NH4+-N
T-N(mg/ℓ)
30
20
10
0
Legend
Primary effluent T-N
Primary effluent S-T-N
Treated water T-N
Treated water S-T-N
3/1
5
10
15
20
25
30
4/1
5

Table 4.1 Primary setting tank effluent characteristics (1)

	PH	Water temperature	M-alkalinity	NH_4^+ -N	NO_2^- -N	NO_3^- -N	T-N	Soluble T-N
Run-1	7.3 (7.0 - 7.6)	15.5 (14.4 - 17.0)	133.7 (76.1 - 150.7)	15.6 (8.0 - 20.8)	N.D.	0.08 (0.02 - 0.20)	22.5 (15.1 - 29.6)	17.8 (10.8 - 21.3)
Run-2	7.3 (7.0 - 7.5)	16.4 (15.5 - 17.2)	133.0 (124.1 - 146.8)	14.7 13.1 - 16.4)	N.D.	0.07 (0.03 - 0.12)	22.6 (20.2 - 24.8)	18.5 (17.5 - 19.6)
Run-3	7.4 (7.2 - 7.9)	16.6 (13.0 - 18.0)	130.9 (103.3 - 144.8)	12.8 (10.6 - 16.2)	N.D.	0.06 (0.02 - 0.15)	21.7 (19.6 - 25.4)	16.3 (13.0 - 19.3)
Run-4	7.5 (7.4 - 7.6)	18.4 (17.6 - 19.2)	130.1 (95.9 - 140.4)	13.7 (7.8 - 16.1)	N.D.	0.13 (0.02 - 0.49)	21.0 (13.2 - 25.6)	17.0 (10.2 - 21.2)
Run-5	7.4 (7.2 - 7.4)	21.2 (19.5 - 23.0)	138.2 (117.5 - 148.2)	14.9 (12.4 - 17.1)	N.D.	N.D.	25.2 (20.8 - 29.7)	17.3 (15.0 - 19.3)
Run-6	7.3 (7.1 - 7.5)	24.0 (23.0 - 24.5)	140.0 (137.2 - 142.9)	12.1 (12.1 - 12.2)	N.D.	0.10 (0.07 - 0.13)	25.3 (23.3 - 27.2)	15.8 (14.5 - 17.0)
Run-7	7.4 (7.3 - 7.6)	23.9 (23.2 - 24.6)	129.8 (106.1 - 142.1)	11.0 (9.0 - 12.5)	N.D.	N.D.	19.8 (16.7 - 21.4)	13.7 (11.2 - 15.4)
Run-8	7.4 (7.1 - 7.5)	24.2 (23.6 - 24.7)	134.2 (98.2 - 147.3)	13.0 (8.0 - 15.9)	N.D.	0.11 (0 - 0.59)	20.1 (14.4 - 23.6)	15.7 (10.3 - 18.0)
Run-9	7.4 (7.2 - 7.6)	23.5 (22.3 - 24.8)	134.9 (101.7 - 146.9)	13.0 (7.4 - 15.3)	N.D.	N.D.	19.8 (13.1 - 23.9)	15.2 (9.8 - 19.0)
Run-10	7.5 (7.1 - 7.8)	20.8 (19.0 - 22.8)	133.1 (89.4 - 147.2)	13.3 (7.2 - 16.0)	N.D.	0.16 (0 - 1.03)	20.9 (13.2 - 24.7)	15.4 (9.6 - 19.9)
Run-11	7.4 (7.2 - 7.5)	19.1 (17.5 - 20.2)	123.4 (76.7 - 150.8)	11.3 (5.9 - 17.3)	0.19 (0.10 - 0.70)	0.33 (0.04 - 0.85)	18.3 (12.6 - 23.5)	14.1 (7.0 - 18.6)
Run-12	7.4 (7.0 - 7.7)	17.6 (17.1 - 18.0)	135.0 (13.07 - 140.3)	13.7 (9.8 - 17.7)	0.19 (0.09 - 0.26)	0.38 (0.20 - 0.75)	21.0 (16.5 - 23.9)	17.3 (13.2 - 21.3)
Run-13	7.1 (6.5 - 7.4)	14.1 (13.5 - 14.8)	149.8 (130.8 - 169.5)	19.1 (7.4 - 25.4)	0.05 (0 - 0.11)	0.07 (0 - 0.16)	37.6 (13.8 - 81.8)	24.3 (23.0 - 27.1)
Run-14	7.2 (7.0 - 7.4)	13.6 (12.2 - 14.1)	157.4 (130.8 - 204.4)	22.2 (17.1 - 26.5)	0.05 (0.01 - 0.09)	0.04 (0 - 0.07)	32.3 (28.0 - 37.4)	13.6 (8.8 - 26.6)
Run-15	7.2 (7.0 - 7.5)	13.2 (13.0 - 13.5)	152.8 (131.5 - 164.0)	23.2 (20.7 - 26.5)	0.01 (0 - 0.02)	0.01 (0 - 0.02)	37.6 (26.1 - 51.5)	24.9 (21.7 - 28.2)
Run-16	7.2 (7.0 - 7.4)	13.2 (13.0 - 13.6)	142.7 (131.1 - 154.4)	18.5 (16.7 - 21.0)	0.09 (0.01 - 0.28)	0.05 (0 - 0.14)	24.1 (20.0 - 26.8)	20.8 (18.6 - 23.0)
Run-17	7.5 (7.3 - 7.8)	15.7 (15.2 - 16.2)	151.8 (136.4 - 167.8)	20.2 (17.1 - 23.7)	0.01 (0 - 0.02)	0.06 (0 - 0.10)	29.1 (25.4 - 31.7)	21.7 (19.4 - 24.6)
Run-18	6.8 (6.6 - 7.0)	16.4 (16.1 - 17.0)	164.4 (138.4 - 206.6)	21.7 (19.9 - 26.3)	N.D.	0.09 (0.07 - 0.10)	118.0 (30.5 - 170.7)	22.9 (21.2 - 24.9)

(mg/ℓ)

Table 4.2 Primary setting tank effluent characteristics (2)

	Org-N	Soluble Org-N	COD	Soluble COD	BOD	Soluble BOD	SS
Run-1	6.6 (6.0 - 9.2)	3.0 (1.9 - 4.8)	39.3 (31.0 - 45.2)	20.7 (12.7 - 25.1)	81.4 (36.4 - 104.3)	37.0 (22.0 - 46.9)	49 (19 - 77)
Run-2	7.8 (6.0 - 9.8)	3.6 (1.9 - 6.4)	41.1 (35.7 - 46.3)	20.6 (18.5 - 23.0)	85.0 (77.6 - 87.8)	42.2 (33.6 - 46.4)	50 (28 - 66)
Run-3	8.6 (6.8 - 10.2)	3.0 (1.5 - 6.1)	41.0 (33.0 - 50.5)	18.3 (15.9 - 20.7)	78.7 (34.5 - 108.5)	26.4 (13.4 - 45.6)	70 (37 - 96)
Run-4	7.1 (4.2 - 9.4)	3.1 (1.0 - 5.0)	37.3 (28.1 - 47.4)	19.2 (12.7 - 21.0)	70.6 (50.6 - 86.2)	23.5 (16.3 - 30.0)	45 (29 - 98)
Run-5	10.2 (7.5 - 14.6)	2.4 (1.0 - 3.4)	50.9 (45.5 - 59.7)	20.9 (18.4 - 23.6)	110.4 (101.4 - 122.5)	38.1 (29.8 - 42.4)	93 (70 - 110)
Run-6	13.0 (11.0 - 15.0)	3.5 (2.3 - 4.7)	56.6 (49.1 - 65.7)	23.1 (20.6 - 25.6)	104.8 (104.8 - 104.8)	57.2 (57.2 - 57.2)	121 (77 - 186)
Run-7	8.7 (7.5 - 9.7)	2.6 (2.0 - 2.9)	41.7 (29.5 - 52.3)	16.8 (13.4 - 18.6)	77.6 (61.9 - 88.5)	20.9 (14.2 - 26.2)	69 (44 - 100)
Run-8	7.0 (5.5 - 8.4)	2.6 (1.4 - 4.1)	39.2 (29.7 - 49.2)	18.8 (11.7 - 21.9)	105.1 (80.6 - 141.0)	31.9 (22.0 - 38.6)	53 (32 - 80)
Run-9	6.7 (4.8 - 8.6)	2.1 (1.0 - 3.7)	34.6 (27.4 - 40.1)	18.2 (15.4 - 22.1)	80.9 (56.8 - 95.7)	26.2 (22.6 - 32.6)	49 (23 - 88)
Run-10	7.4 (4.7 - 8.8)	1.9 (1.0 - 3.8)	35.4 (22.3 - 41.7)	18.1 (11.6 - 23.5)	81.8 (53.9 - 122.1)	22.4 (9.2 - 39.0)	68 (33 - 139)
Run-11	6.4 (5.8 - 7.5)	2.2 (02. - 4.9)	31.8 (20.9 - 42.1)	17.1 (10.7 - 20.5)	64.3 (39.2 - 89.5)	20.1 (5.7 - 34.5)	60 (37 - 79)
Run-12	6.7 (5.1 - 8.2)	3.0 (1.8 - 5.2)	39.1 (26.5 - 50.5)	21.6 (14.3 - 27.7)	82.1 (50.5 - 122.6)	32.4 (19.2 - 42.0)	52 (29 - 69)
Run-13	11.9 (10.6 - 13.5)	3.6 (1.9 - 5.1)	77.6 (45.9 - 235.9)	28.9 (24.9 - 37.2)	142.5 (97.0 - 221.6)	57.6 (30.6 - 88.3)	84 (58 - 105)
Run-14	8.0 (4.7 - 9.7)	2.5 (1.7 - 3.8)	68.9 (47.7 - 103.0)	24.5 (21.7 - 27.2)	216.9 (107.5 - 420.7)	42.2 (19.9 - 67.2)	141 (68 - 265)
Run-15	14.8 (4.4 - 27.1)	3.4 (0.9 - 12.2)	82.1 (36.8 - 143.5)	22.9 (20.2 - 28.0)	166.8 (79.6 - 339.4)	33.7 (24.0 - 51.2)	247 (66 - 470)
Run-16	5.5 (2.9 - 7.5)	2.2 (1.5 - 2.9)	45.2 (32.0 - 79.5)	21.1 (14.7 - 25.1)	41.3 (41.3 - 41.3)	20.9 (14.1 - 27.8)	42 (30 - 66)
Run-17	8.5 (6.0 - 10.8)	1.1 (0.8 - 2.2)	49.4 (40.5 - 61.5)	22.3 (18.9 - 27.5)	122.4 (103.6 - 141.3)	58.1 (26.5 - 89.8)	92 (78 - 124)
Run-18	57.5 (1.03 - 150.7)	2.7 (1.2 - 4.3)	629.0 (61.0 -)	26.9 (24.0 - 28.4)	209.7 (197.3 - 221.3)	207.3 (67.6 - 129.9)	1059 (105 - 2940)

(mg/ℓ)

Table 5.1 Treated water characteristics (1)

	PH	M-alkalinity	NH_4^+ -N	NO_2^- -N	NO_3^- -N	T-N	Soluble T-N
Run-1	7.1 (6.6 - 7.6)	68.7 (3.41 - 123.6)	2.7 (0.1 - 9.8)	1.0 (0.2 - 2.4)	6.7 (2.5 - 8.7)	11.4 (6.2 - 19.5)	9.1 (6.0 - 11.3)
Run-2	7.0 (6.6 - 7.6)	61.5 (48.4 - 74.1)	1.4 (0.1 - 4.1)	0.2 (0.1 - 0.3)	7.7 (5.0 - 9.7)	10.6 (8.9 - 12.5)	10.4 (8.9 - 12.1)
Run-3	7.4 (7.0 - 7.7)	70.6 (60.3 - 93.9)	0.2 (0.0 - 1.8)	0.1 (0.01 - 0.2)	5.2 (3.1 - 6.2)	6.4 (5.4 - 7.3)	6.2 (5.2 - 7.0)
Run-4	7.2 (6.8 - 7.4)	59.4 (50.9 - 68.7)	0.1 (0.0 - 0.7)	0.04 (0.01 - 0.1)	6.2 (4.2 - 7.4)	7.2 (4.9 - 8.6)	7.1 (4.7 - 8.5)
Run-5	7.3 (7.1 - 7.6)	85.6 (60.5 - 111.4)	3.3 (0.6 - 6.9)	0.1 (0.1 - 0.1)	4.4 (2.2 - 7.0)	8.9 (6.5 - 10.6)	8.5 (6.3 - 9.5)
Run-6	7.4 (7.1 - 7.8)	103.0 (86.4 - 119.7)	6.1 (3.3 - 9.0)	0.2 (0.1 - 0.2)	2.4 (0.3 - 4.5)	12.4 (11.6 - 13.3)	9.9 (9.4 - 10.4)
Run-7	7.3 (6.9 - 7.6)	69.0 (64.9 - 73.7)	0.3 (0.0 - 1.0)	0.1 (0.1 - 0.2)	6.8 (5.2 - 7.6)	8.2 (6.1 - 9.0)	7.9 (6.0 - 8.7)
Run-8	7.3 (7.1 - 7.7)	76.1 (54.4 - 88.1)	1.6 (0.1 - 2.9)	0.1 (0.02 - 0.2)	5.1 (4.1 - 6.0)	7.6 (4.8 - 9.5)	7.5 (4.6 - 9.4)
Run-9	7.3 (7.2 - 7.4)	68.4 (59.6 - 77.2)	0.0 (0.0 - 0.0)	0.02 (0.02 - 0.04)	6.2 (5.1 - 7.2)	7.0 (5.8 - 8.0)	6.9 (5.7 - 7.9)
Run-10	7.1 (6.7 - 7.3)	67.8 (51.7 - 81.6)	0.1 (0.0 - 1.2)	0.04 (0 - 0.1)	6.4 (3.7 - 9.3)	7.5 (5.2 - 9.9)	7.2 (4.6 - 9.8)
Run-11	7.2 (7.0 - 7.3)	59.1 (41.7 - 69.3)	0.0 (0.0 - 0.0)	0.01 (0 - 0.01)	6.4 (3.2 - 7.7)	7.5 (4.6 - 8.9)	7.0 (3.7 - 8.3)
Run-12	7.0 (6.7 - 7.2)	63.5 (59.2 - 69.3)	0.1 (0.0 - 0.1)	0.02 (0.01 - 0.05)	8.7 (6.9 - 9.9)	9.7 (8.3 - 10.9)	9.5 (8.2 - 10.7)
Run-13	6.9 (6.6 - 7.3)	58.5 (42.5 - 81.8)	1.4 (0.2 - 4.3)	0.06 (0.04 - 0.08)	9.5 (7.7 - 10.5)	11.9 (10.0 - 13.3)	11.5 (11.2 - 12.2)
Run-14	6.9 (6.8 - 7.0)	78.5 (68.1 - 85.6)	4.2 (0.1 - 7.0)	0.07 (0.06 - 0.09)	6.1 (4.6 - 7.2)	13.6 (8.8 - 26.6)	11.0 (8.7 - 12.1)
Run-15	6.9 (6.7 - 7.3)	61.2 (24.1 - 85.5)	3.9 (0.1 - 7.9)	0.09 (0.02 - 0.14)	8.4 (3.3 - 16.5)	14.5 (10.9 - 17.8)	14.5 (10.9 - 17.8)
Run-16	6.7 (6.5 - 6.9)	30.8 (26.9 - 32.9)	0.1 (0.0 - 0.2)	0.03 (0.01 - 0.04)	13.5 (10.7 - 17.2)	14.5 (12.2 - 18.3)	14.5 (12.2 - 18.3)
Run-17	7.0 (6.7 - 7.1)	57.0 (53.8 - 59.7)	0.0 (0.0 - 0.1)	0.02 (0.01 - 0.03)	8.4 (6.4 - 9.6)	9.4 (7.5 - 11.0)	9.4 (7.5 - 11.0)
Run-18	7.0 (6.7 - 7.3)	69.4 (60.7 - 89.6)	0.0 (0.0 - 0.1)	0.01 (0.01 - 0.01)	8.5 (7.4 - 10.0)	10.1 (8.5 - 11.7)	10.1 (8.5 - 11.7)

(mg/ℓ)

Table 5.2 Treated water characteristics (2)

	Org-N	Soluble Org-N	COD	Soluble COD	BOD	Soluble BOD	SS
Run-1	1.6 (0.3 - 6.5)	0.9 (0.6 - 1.1)	10.1 (8.2 - 11.7)	8.5 (7.0 - 10.0)	4.3 (2.0 - 8.4)	1.1 (0.7 - 1.9)	3.7 (2.2 - 5.7)
Run-2	1.0 (0.7 - 1.2)	0.8 (0.6 - 1.0)	9.1 (7.6 - 12.9)	7.4 (6.6 - 8.3)	3.9 (2.3 - 5.7)	1.1 (0.7 - 1.7)	3.3 (1.6 - 5.0)
Run-3	0.9 (0.6 - 1.4)	0.7 (0.5 - 1.1)	7.4 (6.0 - 8.9)	6.6 (5.5 - 8.2)	2.3 (1.6 - 2.8)	1.4 (0.7 - 2.3)	2.5 (0.9 - 5.0)
Run-4	0.8 (0.6 - 0.9)	0.8 (0.6 - 0.9)	6.9 (5.3 - 10.0)	6.3 (5.4 - 7.2)	2.1 (0.8 - 4.7)	1.3 (0.4 - 2.0)	2.1 (0.8 - 7.8)
Run-5	1.1 (0.7 - 1.4)	0.7 (0.3 - 1.0)	7.7 (7.0 - 8.6)	7.0 (6.3 - 8.0)	5.5 (3.0 - 7.3)	1.4 (1.1 - 1.7)	2.1 (1.3 - 2.7)
Run-6	3.8 (2.2 - 5.3)	1.2 (1.0 - 1.4)	8.9 (8.1 - 10.1)	7.6 (7. - 7.6)	7.5 (7.5 - 7.5)	2.6 (2.6 - 2.6)	4.1 (2.4 - 8.6)
Run-7	0.9 (0.7 - 1.1)	0.7 (0.4 - 0.8)	7.7 (7.2 - 8.3)	6.7 (6.6 - 7.0)	3.7 (3.5 - 4.0)	1.7 (1.5 - 1.9)	2.6 (1.7 - 5.0)
Run-8	0.8 (0.6 - 1.1)	0.7 (0.4 - 1.1)	7.2 (5.7 - 8.2)	6.4 (4.8 - 7.1)	6.8 (3.7 - 8.7)	3.2 (2.2 - 3.7)	2.2 (1.0 - 3.9)
Run-9	0.8 (0.7 - 0.9)	0.7 (0.5 - 0.8)	6.4 (5.9 - 7.1)	5.7 (5.4 - 6.0)	1.8 (1.1 - 2.5)	1.1 (0.5 - 2.0)	2.0 (1.5 - 2.3)
Run-10	1.0 (0.6 - 1.4)	0.7 (0.5 - 0.8)	6.8 (4.8 - 10.8)	5.5 (3.9 - 6.4)	3.6 (1.4 - 7.2)	1.6 (0.5 - 2.6)	4.3 (0.7 - 21.6)
Run-11	1.1 (0.7 - 1.4)	0.6 (0.5 - 0.8)	6.5 (6.0 - 7.6)	5.8 (4.1 - 7.2)	3.5 (1.4 - 5.6)	3.7 (0.7 - 6.7)	6.5 (2.0 - 16.7)
Run-12	0.9 (0.7 - 1.2)	0.7 (0.2 - 1.1)	8.0 (7.1 - 9.4)	7.1 (6.2 - 9.1)	3.5 (3.1 - 4.1)	0.9 (0.6 - 1.1)	5.7 (3.4 - 9.1)
Run-13	1.7 (1.1 - 3.0)	1.1 (0.9 - 1.4)	9.0 (8.0 - 11.1)	7.5 (6.9 - 8.2)	4.8 (1.9 - 10.2)	1.1 (1.0 - 1.2)	4.0 (1.6 - 13.0)
Run-14	1.0 (0.9 - 1.2)	0.8 (0.6 - 1.0)	20.8 (8.0 - 92.4)	8.1 (6.9 - 9.2)	45.4 (2.0 - 172.7)	1.8 (1.0 - 3.5)	57.8 (1.7 -)
Run-15	3.4 (0.7 - 15.5)	0.9 (0.6 - 1.0)	9.5 (8.6 - 10.3)	8.6 (8.1 - 8.9)	2.5 (2.5 - 2.6)	1.1 (0.3 - 1.7)	3.2 (1.8 - 6.0)
Run-16	0.9 (0.2 - 1.2)	0.7 (0 - 1.0)	9.3 (7.5 - 15.3)	7.7 (7.3 - 8.4)	1.4 (1.1 - 1.8)	0.6 (0.5 - 0.8)	1.8 (1.2 - 2.3)
Run-17	1.1 (1.0 - 1.3)	1.0 (0.9 - 1.0)	8.2 (7.8 - 8.6)	7.5 (7.2 - 8.0)	1.6 (1.5 - 1.6)	0.7 (0.5 - 0.9)	1.3 (0.8 - 2.2)
Run-18	1.0 (1.0 - 1.1)	1.0 (0.9 - 1.0)	9.1 (8.3 - 10.3)	8.0 (7.6 - 8.3)	2.3 (1.5 - 2.7)	0.8 (0.3 - 1.1)	3.0 (2.6 - 4.1)

(mg/ℓ)

Table 6 Average removal rates of each water quality index

	T-N	Soluble T-N	NH_4^+ -N	BOD	Soluble BOD	COD	Soluble COD	SS
Run-1	49.3	48.9	82.7	94.7	97.0	74.3	58.9	92.5
Run-2	53.1	43.8	90.5	95.4	97.4	77.9	64.1	93.4
Run-3	70.5	62.0	98.4	97.1	94.7	82.0	63.9	96.4
Run-4	65.7	57.6	99.3	97.0	94.5	81.5	67.1	95.2
Run-5	64.7	50.9	77.9	95.0	96.3	84.9	66.4	97.7
Run-6	51.0	37.3	49.6	92.8	95.5	84.3	67.3	96.6
Run-7	58.6	42.3	97.3	95.2	91.9	81.5	60.0	96.2
Run-8	62.2	52.2	87.7	93.5	90.0	81.6	65.9	95.9
Run-9	64.6	54.6	100.0	97.8	95.8	81.5	68.6	95.9
Run-10	64.1	53.2	99.2	95.6	92.9	80.8	69.5	93.7
Run-11	59.0	50.4	100.0	94.6	81.6	79.6	66.1	89.2
Run-12	53.8	45.1	99.3	95.7	97.2	79.5	67.1	89.1
Run-13	41.8	52.7	92.3	96.6	98.1	88.4	73.8	98.3
Run-14	57.9	19.1	81.1	79.1	95.7	69.8	67.1	59.1
Run-15	61.4	41.8	83.2	98.5	96.7	88.4	62.3	98.7
Run-16	39.8	30.3	99.5	96.6	97.1	79.4	63.6	95.8
Run-17	67.7	56.7	100.0	98.7	98.8	83.4	66.2	98.6
Run-18	55.9	55.9	100.0	99.7	99.3	98.6	70.2	99.8

(%)

6.2 AN OUTLINE OF TREATMENT CONDITIONS IN SUMMER (Runs 5 ~ 9)

In Run 5, inflow was increased to 8,300 m^3/day because water temperature rose, and the recycling ratio was increased to 160%. As a result, the DO concentration in the nitrification tank fell. The DO concentration did not rise even when the air supply volume increased, remaining around 0.5 mg/ℓ. Consequently nitrification got progressively worse. Finally, about 6 mg/ℓ of NH_4^+-N was detected in the G_2 tank, and T-N concentration in treated water ran in excess of 10 mg/ℓ. This tendency was noticed also in Run 6.

For Run 7, therefore, the recycling ratio of the return sludge pump was cut from 100 to 50% aiming to increase detention time in the nitrification tank and ensure sufficient nitrification time.

As a result, net detention time in the nitrification tank increased from 2.3 hours in Run 6 to 2.8 hours, and nitrification improved again. thus, T-N concentration in treated water decreased to 8.2 mg/ℓ on average.

Runs 8 conditions were identical to Run 7 except that the submerged pump recycled nitrified liquor to the F_2 tank. No substantial change in performance, including the nitrogen removal, was noticed.

Toward the end of August an air pipe was connected to the agitators in the F_2 tank to convert it to an aerobic tank (nitrification tank), changing the volumetric ratio of denitrification to nitrification tank to 1/3.

As a result, in Run 9, no NH_4^+-N was detected at the outlet of the nitrification tank even when the inflow was increased to 8,200 m^3/day, and treated water T-N concentration was 7.0 mg/ℓ on average.

6.3 AN OUTLINE OF TREATMENT CONDITIONS IN AUTUMN (Runs 10 ~ 12)

In Run 10, inflow could be increased to 9,460 m^3/day, and treatment conditions were as good as in Run 9. Aeration tank detention time was 9.7 hours, the shortest during the experiment. The average water temperature was 20.8°C. Since Run 10 came when the average water temperature begin to fall, it might be possible to reduce detention time and increase inflow volume when temperatures are highest.

In Run 11, inflow was set at 7,300 m^3/day because of declining water temperatures. But a typhoon brought downpours during the run, and for some time influent wastewater was very weak.

AS a result, the substrate level in the denitrification tank dropped sharply. On the other hand nitrification was achieved 100%. Since denitrification in the secondary settling tank was not carried out at all, the amount of NO_T-N (NO_2^--N + NO_3^--N) recycled to the denitrification tank went up. Consequently, a considerably high concentration of NO_T-N was detected in the denitrification tank. (Refer to 7.2.4.)

In Run 12, net detention time was increased, and the return sludge recycling ratio was reduced from 100 to 50% aiming to ensure sufficient denitrification time. But in the first half of the run rather high NO_T-N was detected in the denitrification tank.

The operation was suspended from the last third of November till mid-January while the sludge collectors of the settling tanks were repaired.

6.4 AN OUTLINE OF TREATMENT CONDITIONS IN WINTER (Runs 13 = 18)

Toward the end of December, when repairs were completed, an operation for acclimation was carried out. As nitrification became stable, the experiment was restarted in mid-January with inflow set at 4,800 m^3/day and the recycling ratio at 160%.

At about the same time, the filter press machine at the Arakawa plant broke down, and influent concentration started to rise under the influence of increasing return flow loading from the sludge handling process.

In Run 13, therefore, T-N concentration of primary effluent rose to 32.5 mg/ℓ on average, far above the level recorded in the spring to autumn period, and treated water T-N concentration became 12.2 mg/ℓ on average.

During Run 14, influent concentration rose further. In addition to this, on occasion primary sludge ran off to make the characteristics of primary effluent burse upward. From around that time, filamentous organisms flourished to heavily degrade sludge settlability.

As the sludge interface in the secondary settling tank therefore consequently rose, excess sludge volume was increased. As a result, the process SAT fell and nitrification went from bad to rose, sending treated water T-N concentration up.

This state continued and the T-N removal rate was low, till the first half of Run 16.

In Runs 17 and 18, methanol was injected into the F_1 tank inlet to increase by about 10 mg/ℓ the S-BOD concentration in the mixed liquor of the denitrification tank. As far as nitrogen removal is concerned, however, there was no significant difference from the cases without methanol injection.

7. REMOVAL OF NITROGEN

7.1 FORMS OF NITROGEN AT SAMPLING POINTS

Figure 6 shows an example of nitrogen forms at various sampling points.

As regards soluble nitrogen, NH_4^+-N in the nitrification tank was oxidized into NO_2^--N or NO_3^--N. A sharp decline in the concentration of soluble T-N in the F_1 tank was due to the combination of denitrification and the dilution effect of recycled liquor. Throughout the experiment period, the NO_2^--N concentration in the nitrified liquor was quite low, showing "nitrate-type nitrification". Organic nitrogen was biodegraded in the nitrification tanks (G_1 & G_2) into NH_4^+-N, therefore its concentration became low.

NH_4^+-N produced from biodegradation of organic nitrogen was oxidized into NO_3^--N by nitrifying bacteria immediately, and accordingly there was no NH_4^+-N remaining in the mixed liquor of the second nitrification tank (G_2). Some organic nitrogen remained in the mixed liquor of the second nitrification tank (G_2), as its structure was too complex to be decomposed biologically.

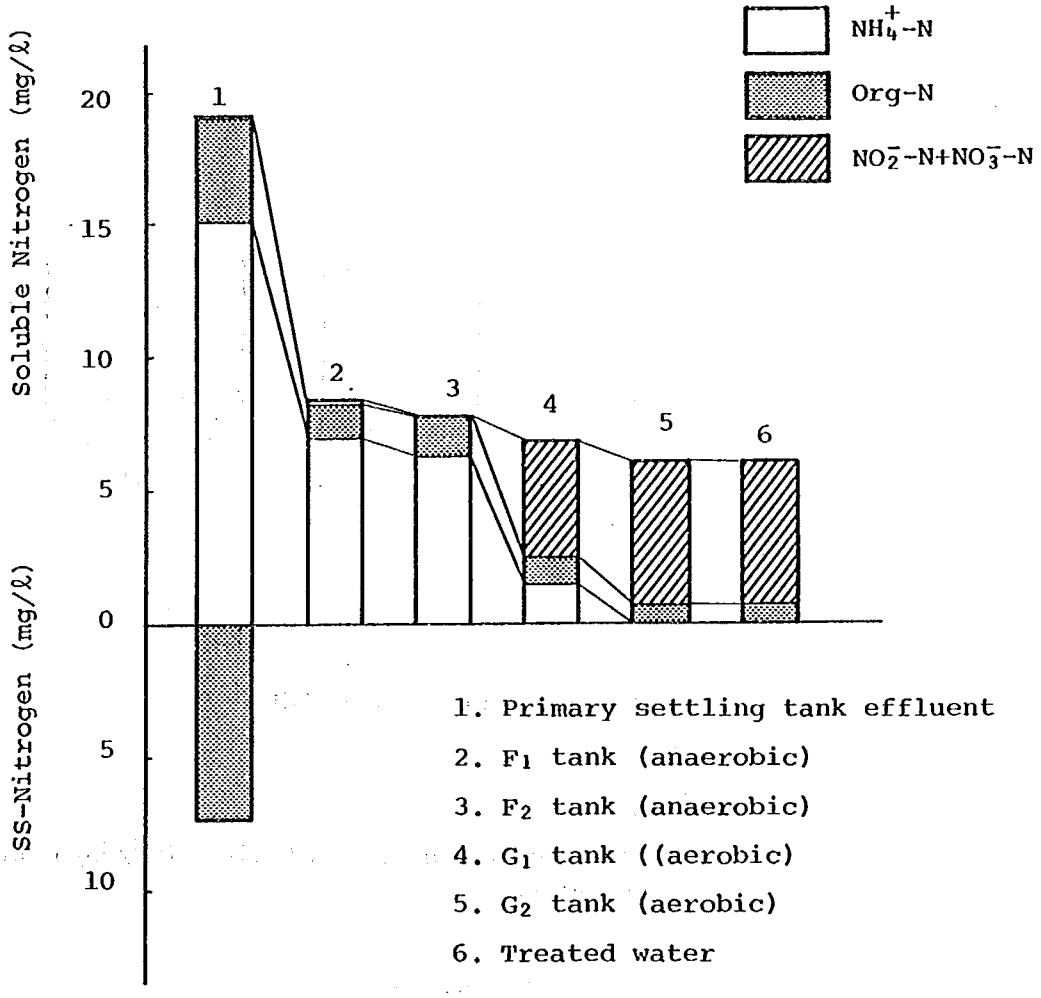


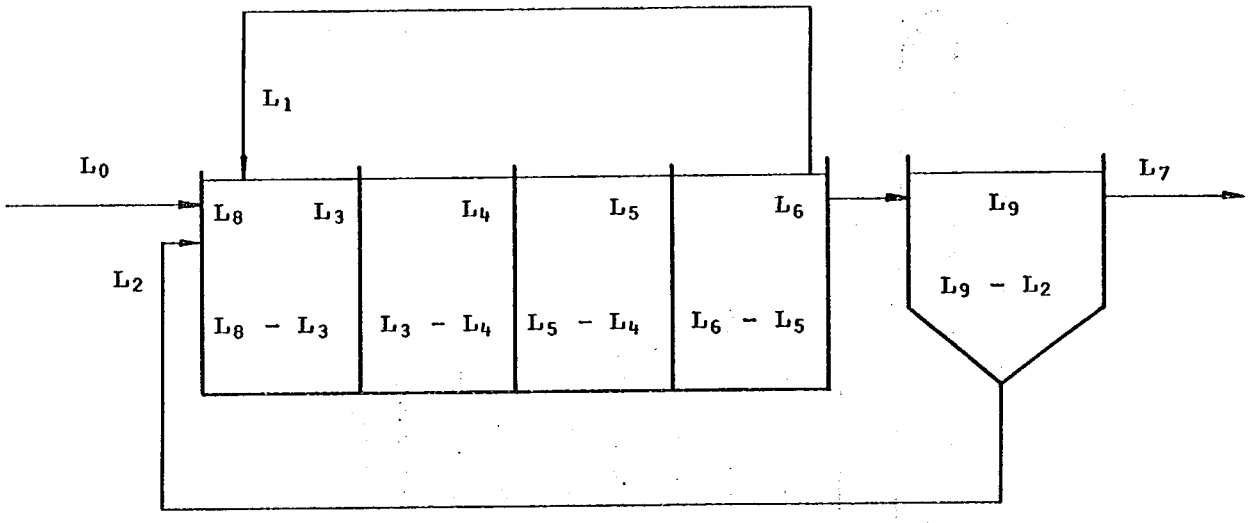
Fig. 6 Forms of nitrogen in tanks (May 22)

SS nitrogen can be considered organic nitrogen. There was no SS-nitrogen in treated water, indicating SS-nitrogen was totally removed. SS-nitrogen would have been absorbed into the activated sludge and separated from liquor. As demonstrated in Fig. 6, the ratio of SS-nitrogen removal in total nitrogen removal was considerably high.

7.2 NO_T-N BALANCE OF THE PROCESS

7.2.1 NO_T-N Balance Examples

To evaluate nitrogen removal in the nitrified liquor recycled biological nitrification-denitrification process, the NO_T-N (NO_2^--N + NO_3^--N) balance was studied. The balance calculation was made as indicated in Fig. 7.



L_{0-9}: NO_T-N loading (kg/day) at sampling point

$L_8 = L_0 + L_1 + L_2$

$L_9 = L_6 - (L_1 + L_7)$

Others were calculated as the product of flow (m^3/day) and NO_T-N concentration (mg/ℓ).

Fig. 7 NO_T-N balance

The measured NO_T-N values in the balance calculation were all of filtrated samples.

With reference to Fig. 7, the reduction in NO_T-N loading in each tank can be considered as NO_T-N having been reduced to N_2 gas through the denitrification reaction. The reduction here, however, includes that part of NO_T-N which has been taken directly into sludge for cytosynthesis. On the other hand, the increase in NO_T-N loading in the aerobic tank can be considered due to nitrification.

Figure 8 shows an example of balance during Run 2. It was found that denitrified NO_T-N amount in the secondary settling tank was greater than that in the denitrification tanks.

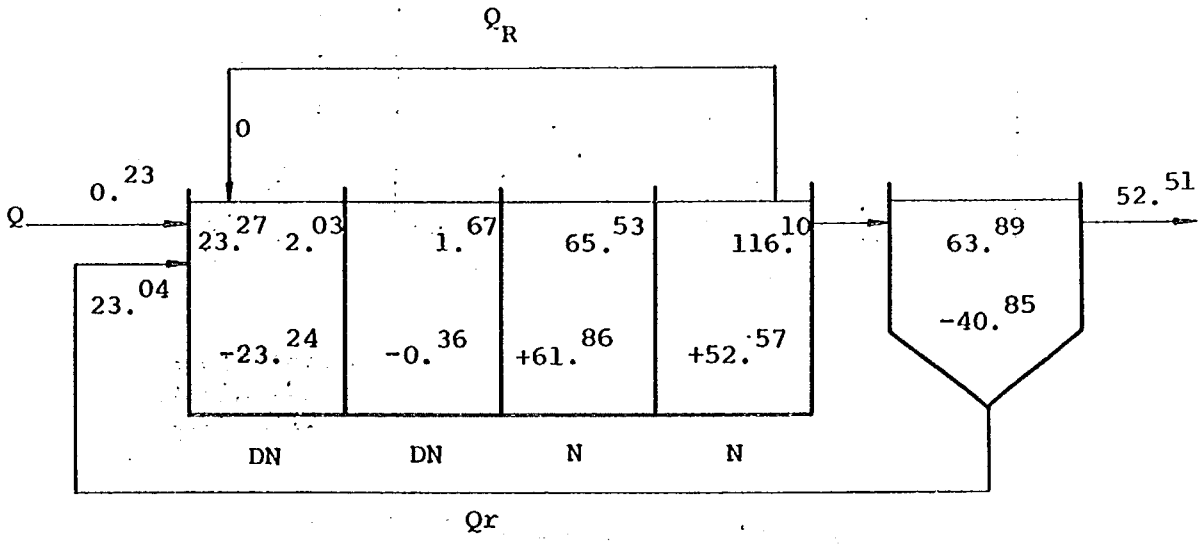


Fig. 8 NO_T-N balance during Run 2 (May 11)

The denitrification in the secondary settling tank may be attributed to endogenous respiration of activated sludge under the sludge interface of the tank. In the first denitrification tank, the bulk of recycled NO_T-N was removed, so NO_T-N loading that flows into the second denitrification tank was quite low. Considering the balance in the second denitrification tank, little denitrification would have occurred there.

Figure 9 shows an example of NO_T-N balance during Run 3. Since the submerged pump directly fed nitrified liquor back to the denitrification tank, NO_T-N loading at the inlet of that tank was heavy, and the denitrified NO_T-N quantity in the first denitrification tank became large. Accompanied with that, as NO_T-N loading from the aeration tank to the secondary settling tank decreased, the denitrified NO_T-N quantity in the secondary settling tank decreased. Even in that case, however, little denitrification occurred in the second denitrification tank.

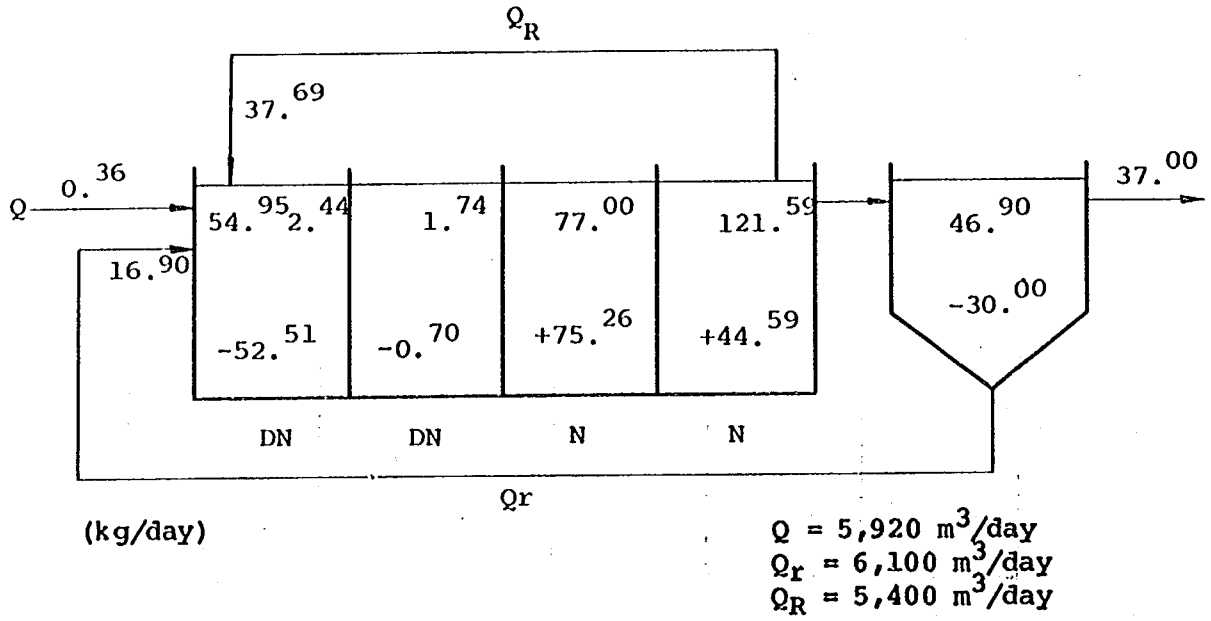


Fig. 9 NO_T-N balance during Run 3 (May 29)

Figure 10 shows an example of NO_T-N balance during Run 7. During this run, also, denitrification in the secondary settling tank was developed and, as a result, there was little or no NO_T-N in the return sludge mixed liquor; the NO_T-N quantity recycled by return sludge was extremely small. From the study of NO_T-N balance, the following was found:

(1) Denitrification takes place in the denitrification tank and secondary settling tank. The denitrified NO_T-N quantity in the secondary settling tank is considerably large.

(2) Changes in the NO_T-N quantity recycled by return sludge depend largely on changes in the denitrified NO_T-N quantity in the secondary settling tank; if the denitrified NO_T-N quantity of this tank is large, the NO_T-N quantity recycled by return sludge becomes extremely small.

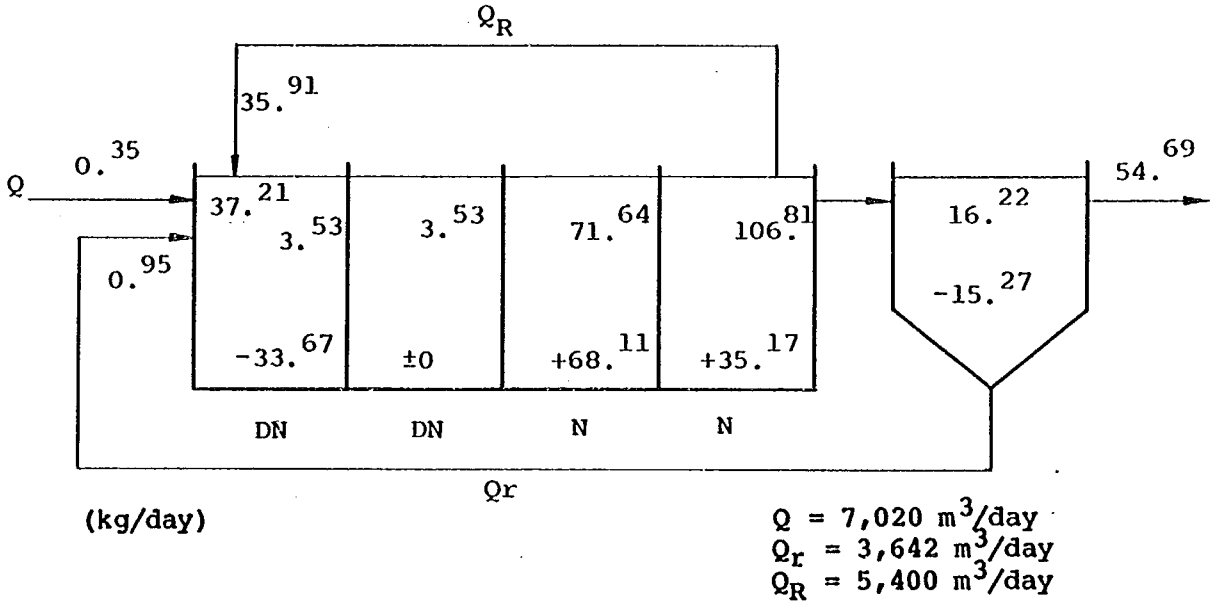


Fig. 10 NO_T-N balance during Run 7 (July 29)

7.2.2 Relationship between Removed T-N Quantity and Denitrified NO_T-N Quantity

The removal of nitrogen in this process is due not only to the inherent biological nitrification-denitrification function, but also to other various factors.

Figure 11 shows the relationship between the removed T-N quantity and total denitrified NO_T-N quantity, which is the sum of the denitrified NO_T-N in the denitrification tank and that in the secondary settling tank. The denitrified NO_T-N quantity was determined from the NO_T-N balance of each sampling day throughout the experiment period, with reduction in NO_T-N regarded as removal by denitrification.

In Fig. 11, the ratio of denitrified NO_T-N quantity to removed T-N quantity varied over a wide range depending on the sampling days. If a straight regression line passing through the origin is given to the distribution in Fig. 11, the slope of the line gives 0.65.

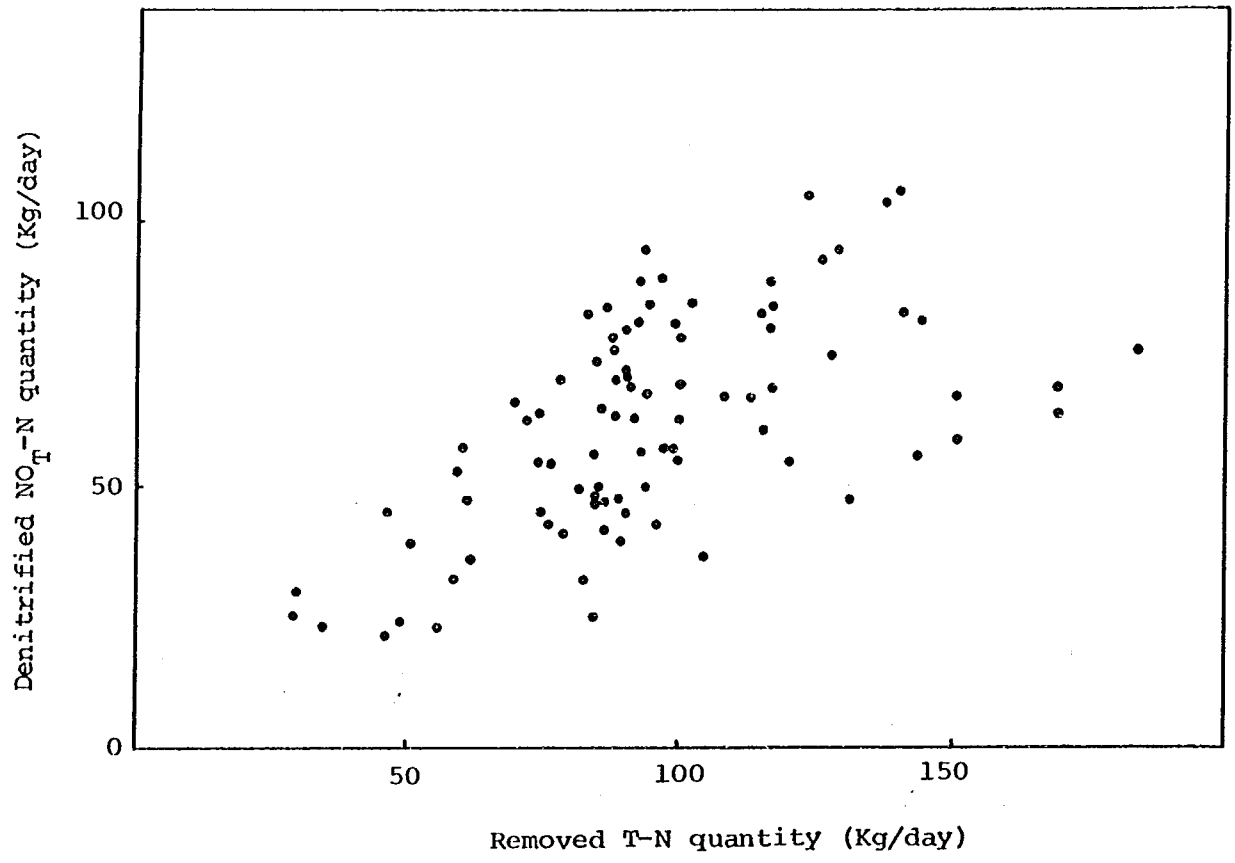


Fig. 11 Removed T-N quantity vs. denitrified NO_T-N quantity

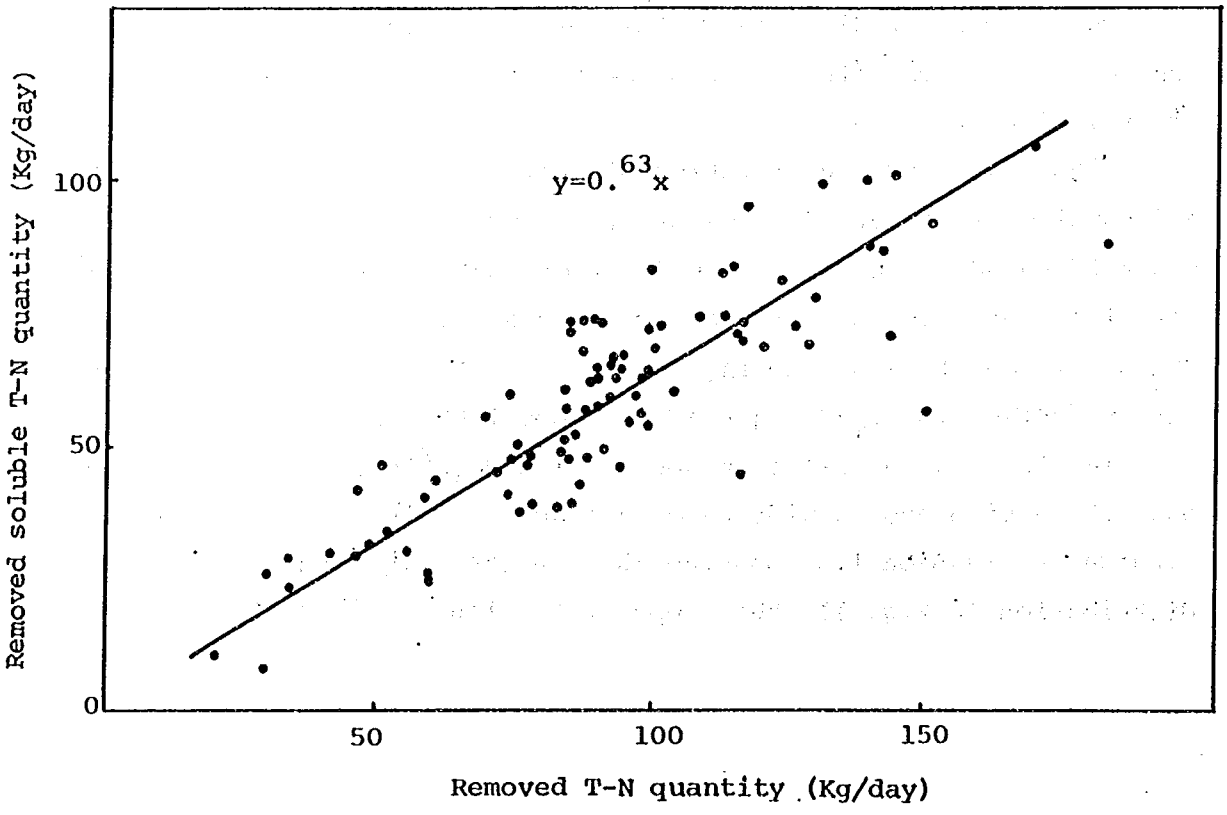


Fig. 12 Removed T-N quantity vs. removed soluble T-N quantity

Figure 12 shows the ratio of soluble T-N removal to T-N removal. Its regression line passing through the origin has a slope of 0.63; about 63% of T-N removal is accounted for by removal of soluble T-N.

Thus removed soluble T-N quantity and denitrified NO_T-N quantity can be considered to be nearly equal. It should be also considered that SS-nitrogen is removed mainly by adsorption and precipitation, but that part of which will turn into soluble nitrogen by biological decomposition and then removed by nitrification and denitrification.

7.2.3 Nitrogen Removal by Sludge Withdrawal

Table 7 shows nitrogen measurements in excess sludge by seasons.

Table 7 Nitrogen amount in excess sludge

	Spring (Runs 1 to 4)	Summer (Runs 5 to 9)	Autumn (Runs 10 to 12)	Winter (Runs 13 to 18)
Average excess sludge MLSS (mg/ℓ)	6,090	9,110	7,850	9,130
Average nitrogen in MLSS (mg/D.S.g)	79	70	67	80

Note: Nitrogen in MLSS was measured in terms of NH_4^+-N after Kjeldahl decomposition of activated sludge mixed liquor.

As shown in Table 7, N in MLSS was roughly in the range of 70 to 80 mg/D.S.g.

Figure 13 shows the relationship between removed nitrogen quantity by excess sludge as calculated according to the following formula and removed T-N quantity:

Removal of nitrogen by sludge withdrawal (kg/day)
= nitrogen in MLSS (mg/D.S.g) x MLRSS (g/ℓ) x
Excess sludge volume (m^3/day) x 10^{-3} ... Eq. 2

It was found that nitrogen removal due to excess sludge withdrawal rose extraordinarily on occasion. This is because the sludge withdrawal rate was increased in winter and bulking took place.

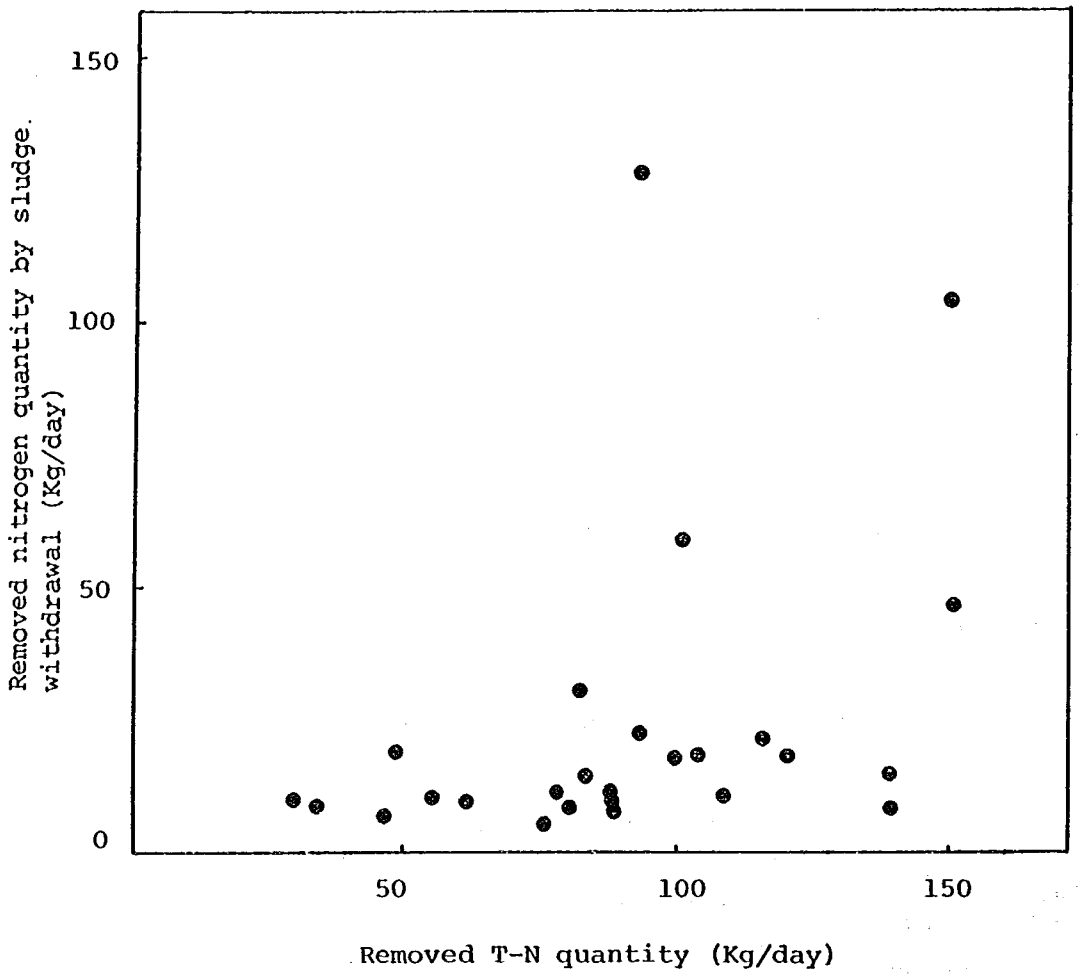


Fig. 13 Removed T-N quantity vs. removed nitrogen quantity by sludge withdrawal

If singular points are excluded, nitrogen removal by excess sludge is found to have been around 15 kg/day regardless of T-N removal.

According to Fig. 12, about 35% of T-N removal is accounted for by removal of SS-nitrogen. Thus if removed T-N quantity is 100 kg/day, 35 kg/day of T-N removal is due to removal of SS-nitrogen. Assuming that all SS-nitrogen is adsorbed into activated sludge and precipitated for removal, 15 kg/day nitrogen removal by excess sludge is too small.

This can be interpreted that a part of SS-nitrogen is decomposed biologically during the process to become soluble nitrogen (S-org-N), which then is turned into NH_4^+-N for nitrification and denitrification, and the amount of such SS-nitrogen is considered to correspond to the difference between removed T-N quantity and removed nitrogen quantity by excess sludge.

7.2.4 Denitrification in the Secondary Setting Tank

From the NO_T-N balance study, it was found that denitrification in the secondary settling tank is significant under ordinary operating conditions. Figure 14 shows the relationship between total denitrified NO_T-N quantity -- determined from NO_T-N balance during Runs 7, 8, 9 and 11 -- and that in the secondary setting tank. Figure 14 shows that the ratio of denitrified NO_T-N quantity in the secondary settling tank to total denitrified NO_T-N quantity holds almost constant at 40% regardless of the amount of return sludge.

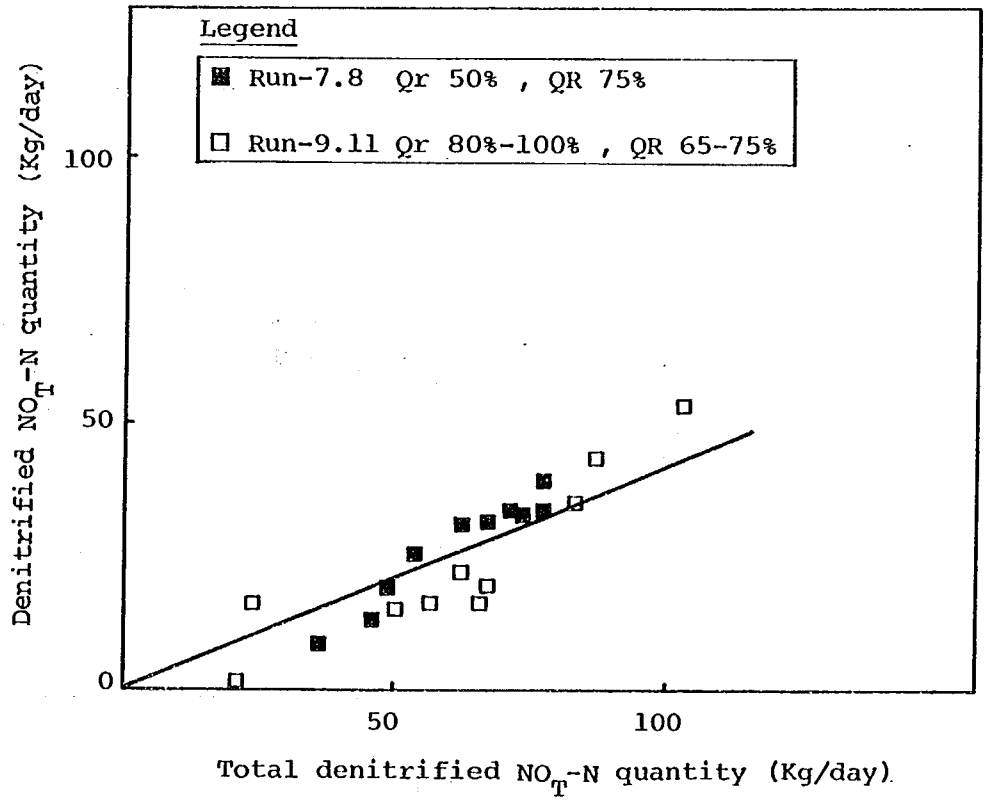


Fig. 14 Total denitrified NO_T-N quantity vs. denitrified NO_T-N quantity in the secondary settling tank

During the period (Runs 1 and 2) when recycling was done by the return sludge pump alone, the ratio was higher; some 60% of total denitrified NO_T-N quantity was accounted for by denitrification in the secondary settling tank.

7.3 RELATIONSHIP BETWEEN RECYCLING RATIO AND NITROGEN REMOVAL RATE

The recycling ratio (R) is one of the most important factors in operation of the process. Shown in Fig. 15 is the relationship between T-N removal rate and recycling ratio (R).

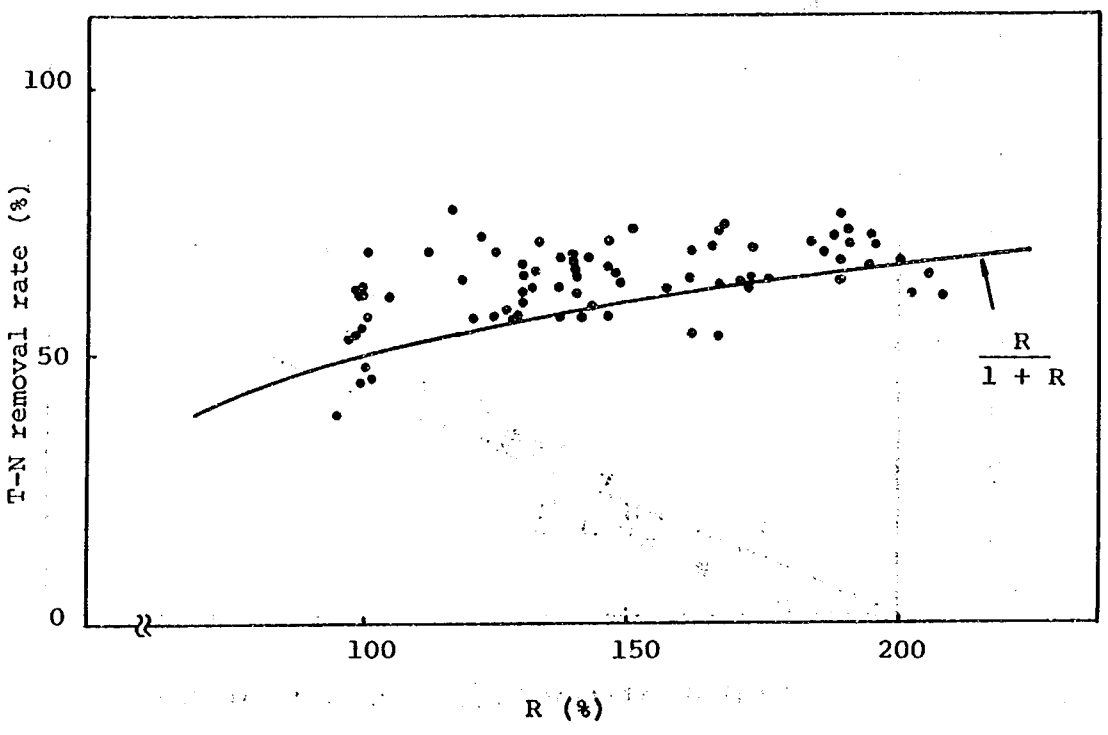


Fig. 15 T-N removal rate vs. recycling ratio

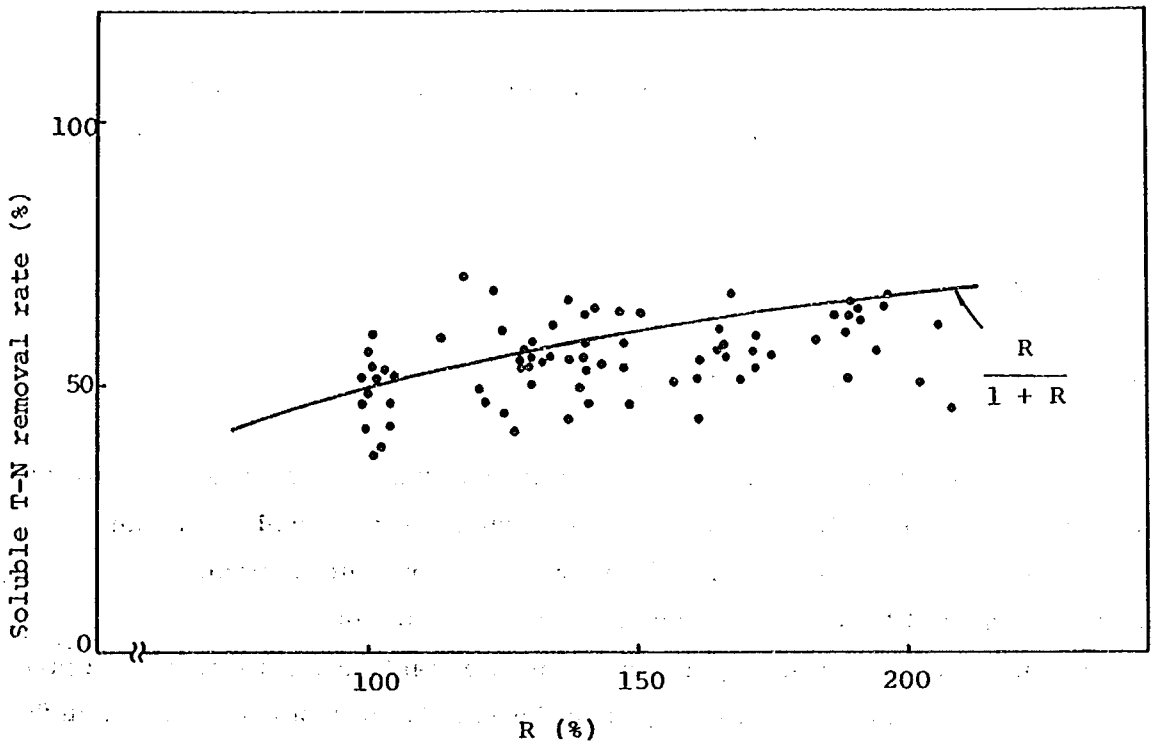


Fig. 16 Soluble T-N removal rate vs recycling ratio

In Figs. 15 and 16, the R value given is for the sum of the recycling rates for the submerged and return sludge pumps.

Figure 16 was prepared to show the relationship between soluble T-N removal rate and recycling ratio. In Figs. 15 and 16, the theoretical removal rate ($Rd = R/(1 + R)$) is shown. Assuming that most soluble T-N is removed by denitrification, the soluble T-N removal rate should follow the theoretical curve ($Rd = R/(1 + R)$).

According to Fig. 16, however, the actual soluble T-N removal rate is generally somewhat lower than the theoretical value, and remains almost unchanged at an average level of 55% even when the recycling ratio (R) is increased.

On the other hand, the T-N removal rate is generally somewhat higher than the theoretical curve ($Rd = R/(1 + R)$) at about 65% on average. When the recycling ratio is in excess of 120%, the removal rate shows no substantial change.

Taken altogether, it was inferred that although the soluble T-N removal rate is lower than the theoretical value, the overall T-N removal rate is about 5% higher than the theoretical value because SS-nitrogen removal is added to the soluble T-N removal.

Under the operating conditions in the survey, the increase in recycling ratio for example from 120% to 200% showed no define effect on the T-N removal rate.

7.4 NITRIFICATION

7.4.1 An Example of Poor Nitrification

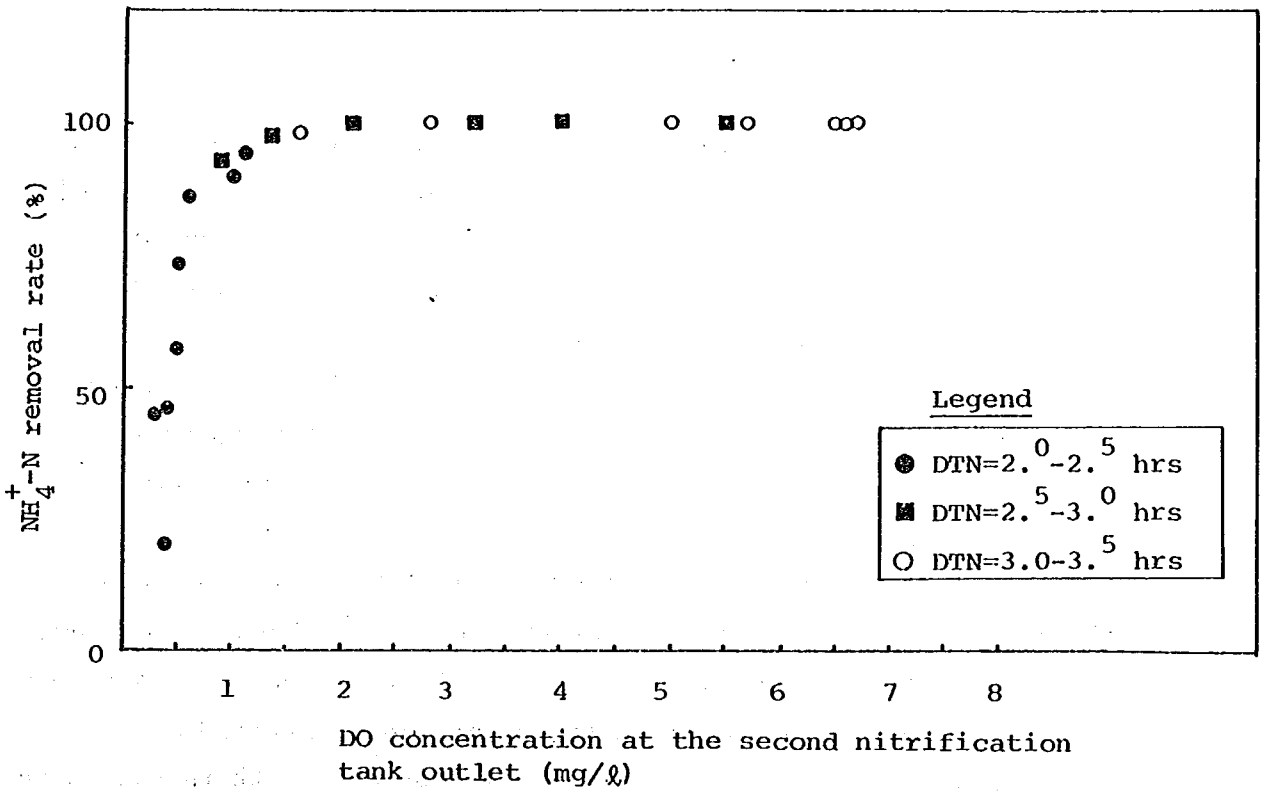
During the survey, nitrification was generally complete. In almost every test run, 100% NH_4^+-N removal rate could be obtained. In Runs 5, 6, 14 and 15, however, nitrification was poor, and a high concentration of NH_4^+-N was detected in treated water.

In runs 5 and 6, the reduction in DO concentration of mixed liquor in the nitrification tank was responsible for this trouble. As already discussed in 6.2, the DO concentration at the end of the nitrification tank did not increase even when the supply air volume was increased, and remained about 0.5 mg/ℓ.

The relationship between DO concentration in the G_2 tank and the NH_4^+-N removal rate in runs 5 and 6 was as shown in Fig. 17.

There is a highly significant relationship between DO concentration and NH_4^+-N removal rate; when DO concentration is above 2 mg/ℓ, the NH_4^+-N removal rate is 100%.

Figure 17 shows also that DO concentration is not more than 1.5 mg/ℓ when detention time in the nitrification tank (detention time for sum of inflow, recirculating liquor and return sludge) is in the range of 2.0 to 2.5 hours, and that DO concentration is not less than 1.5 mg/ℓ when detention time is in the range of 3.0 to 3.5 hours. This phenomenon can be explained by Fig. 18.



Note: The NH_4^+-N removal rate is given in terms of removal rate at the inlet and outlet of the nitrification tank. D_{TN} is the actual detention time in the nitrification tank.

Fig. 17 Nitrification tank outlet DO concentration vs. NH_4^+-N removal rate

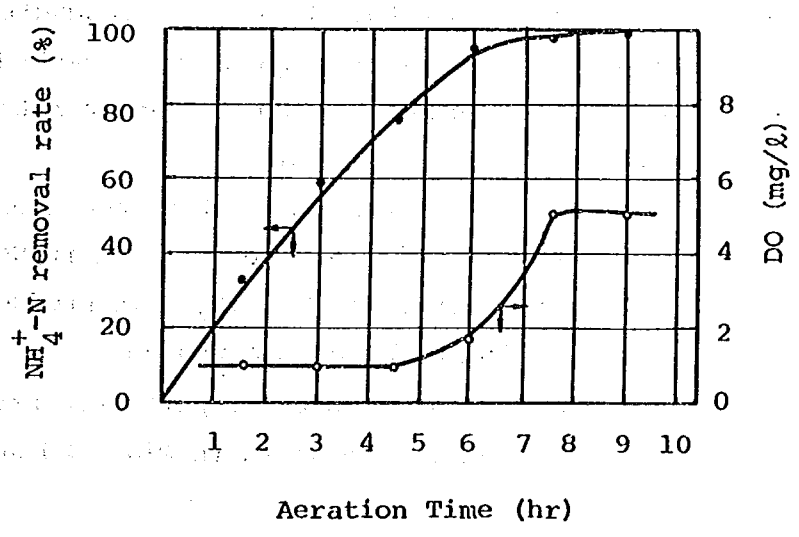


Fig. 18 Relationship between DO concentration and NH_4^+-N removal rate[2]

When nitrification is going with NH_4^+-N present in the activated sludge mixed liquor, oxygen comsunption due to nitrification continues, and DO concentration of mixed liquor does not rise. When NH_4^+-N in the liquor declines and nitrification comes near to the end, DO concentration will rise until equilibrium is attained.

In case adequate reaction time for complete nitrification is not given, DO concentration will hover at a low level as oxygen consumption by nitrification is continuing. Thus liquor will run off with un-nitrified NH_4^+-N.

This suggests that monitoring DO level at the outlet of the nitrification tank will indicate the nitrification state to some degree. If DO concentration at the outlet of the nitrification tank is more than 2.0 mg/ℓ, then NH_4^+-N in the mixed liquor has been nitrifided totally. But if the DO concentration will not rise with increases in the supply air volume, there is a strong possibility that retention time is the rate-determining factor.

7.4.2 Measurement of Nitrification Rate

(1) Measurement of nitrification rate on laboratory tests

From summer to winter, laboratory tests were conducted regularly to measure the nitrification rate. Tested sludge was sampled from the end of the F_1 tank. For measurement of the nitrification rate, the apparatus illustrated in Fig. 19 was used.

The nitrification rate was determined not in terms of the reduction in NH_4^+-N concentration, but in terms of the increase in NO_T-N concentration. The reason of this is that NH_4^+-N is produced during the measurement by degradation of organic nitrogen.

Tests were repeated at various water temperatures from summer to winter. Examples of measurements are shown in Figs. 20 and 21.

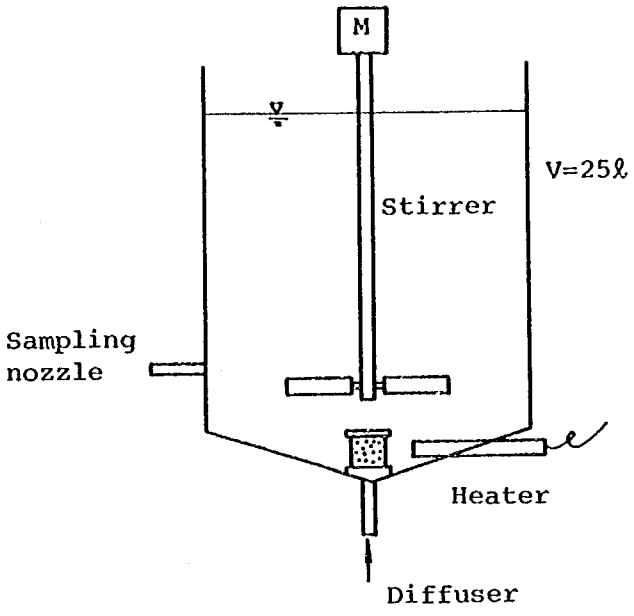


Fig. 19 Test apparatus

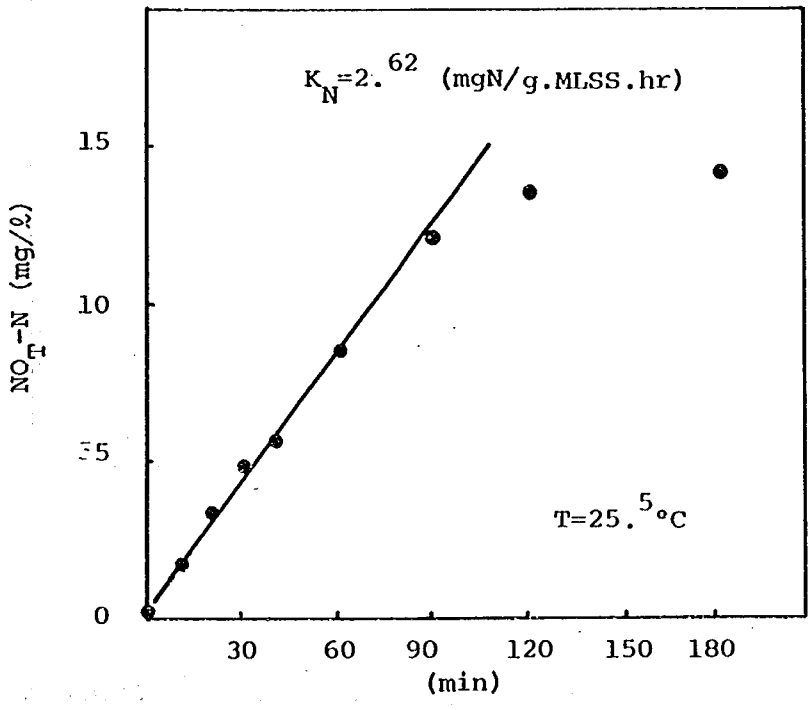


Fig. 20 Measurement of nitrification rate (August 26)

As shown, so long as alkalinity and NH_4^+-N exist, the NO_T-N concentration shows a linear increase to stand for a zero-order reaction. Assuming that the nitrification rate is proportional to MLSS concentration, the following equation can be established:

$$\frac{dC}{dt} = K_N \cdot X \qquad \text{... Eq. 3}$$

Where C: NO_T-N concentration (mg/ℓ)

X: MLSS concentration (g/ℓ)

K_N: Nitrification rate (mgN/g.MLSS.hr)

The K_N values were calculated according to this equation and plotted on a single-logarithm graph sheet vs. water temperature (see Fig. 22).

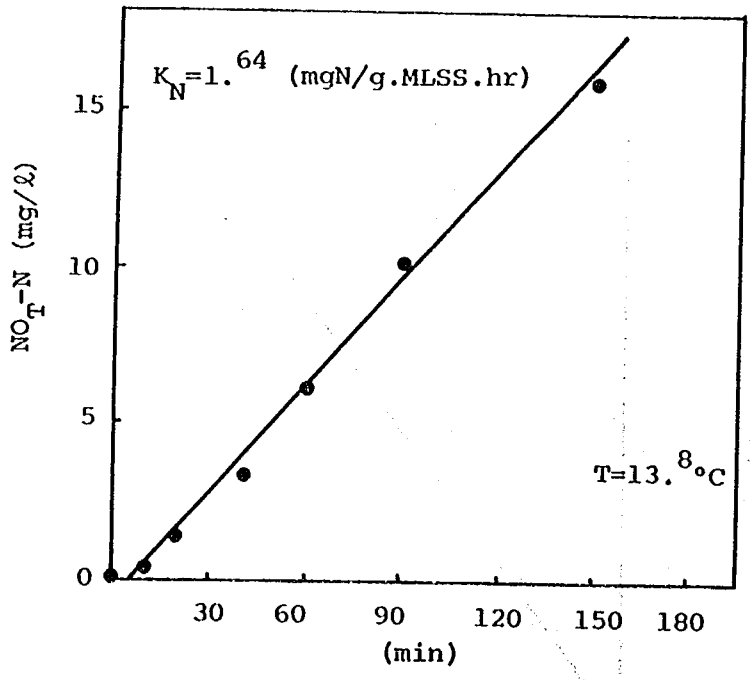


Fig. 21 Measurements of nitrification rate (January 26)

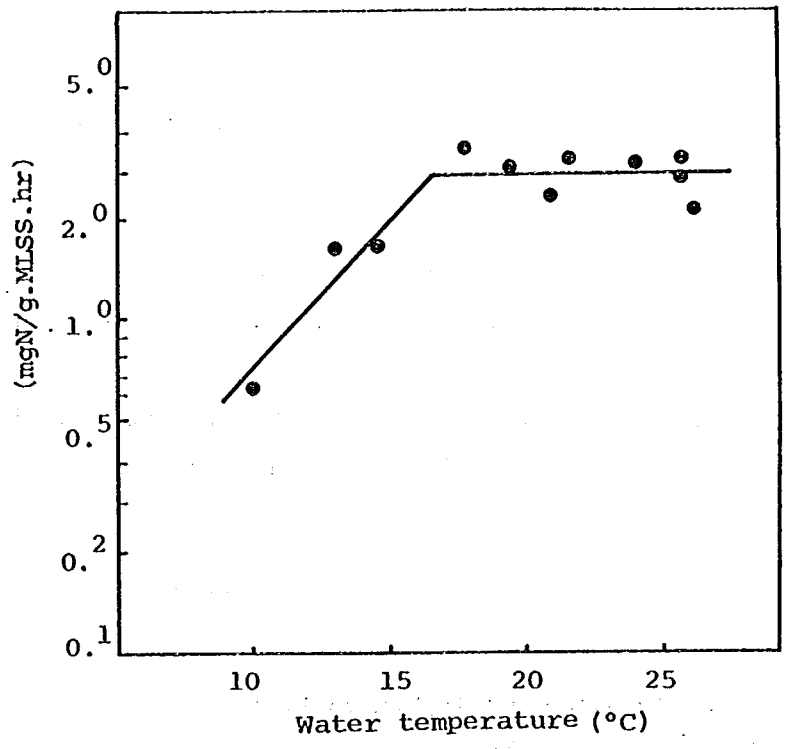


Fig. 22 Nitrification rate vs. water temperature

It was found that the nitrification rate increases with increases in water temperature as long as the temperature is below about 17°C. This relationship is represented by the following regression line:

$$K_N = 0.091 \times e^{0.0206T} \quad (T = \text{water temperature } (°C))$$

When water temperature is higher than about 17°C, $k_N = 2.66$. Thus, at water temperatures of above about 17°C, the nitrification rate remains constant.

(2) Nitrification rate determination at the experiment system

Running data of the experiment system were used for calculation of the nitrification rate. The following hypotheses were used:

1. Nitrification rate is expressed by Eq. 3.
2. The fluid in the nitrification tank assumes a plug flow.

On these assumptions, the nitrification rate in the experiment system is expressed simply by the following equation:

$$K_N = \frac{C_{N2} - C_{N1}}{D_{TN} \cdot X} \qquad \ldots \text{Eq. 4}$$

Where K_N: Nitrification rate (mgN/g.MLSS.hr)

C_{N1}: NO_T-N concentration at end of denitrification tank (mg/ℓ)

C_{N2}: NO_T-N concentration at outlet of first or second nitrification tank (mg/ℓ)

D_{TN}: Actual detention time in nitrification tank (hrs)

X: MLSS concentration (g/ℓ)

Data from the experiment system were applied to Eq. 4 for the calculation of k_N. NO_T-N concentration at the outlet of the first nitrification tank (G_1 in Runs 1 to 8 and 16, F_2 in other runs) was taken as C_{N2}, and the nitrification rate was calculated.

The result is shown in Fig. 23, using only those data which shows a residual NH_4^+-N concentration at the first nitrification tank outlet of more than 0.5 mg/ℓ.

It was found that, the nitrification rate generally is governed by water temperature when the temperature is below 20°C. And when the temperature is above 20°C, there are cases where the nitrification rate declines with increases in water temperature.

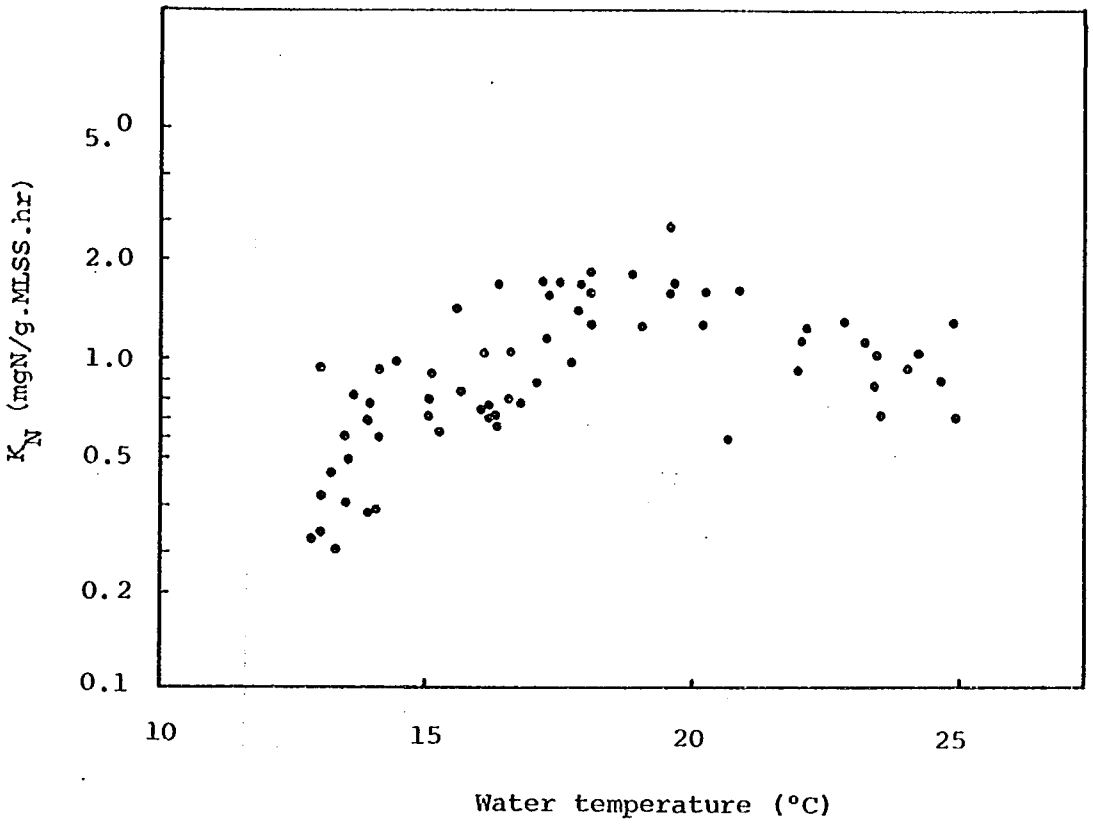


Fig. 23 Nitrification rate at the first nitrification tank

The decline in the nitrification rate at water temperatures above 20°C may show that DO concentration in the nitrification tank would have been low at high water temperatures. Figure 24 shows the relationship between the nitrification rate and DO concentration in the first nitrification tank when the water temperature was about 22°C.

It was found that when DO concentration in the tank was as low as 0.5 mg/ℓ, it became the rate-determining factor and lowered the nitrification rate. On the other hand when DO concentration was about 1.0 mg/ℓ, the nitrification rate was about 1.3 mgN/g.MLSS.hr.

In summer, biological activity of the activated sludge is high and therefore oxygen consumption increase. Even under the same air supply ratio, therefore, the mixed liquor equilibrium DO concentration in a continued nitrification state becomes lower than in winter, which is liable to result in DO concentration becoming the rate-determining factor and retarding the nitrification rate.

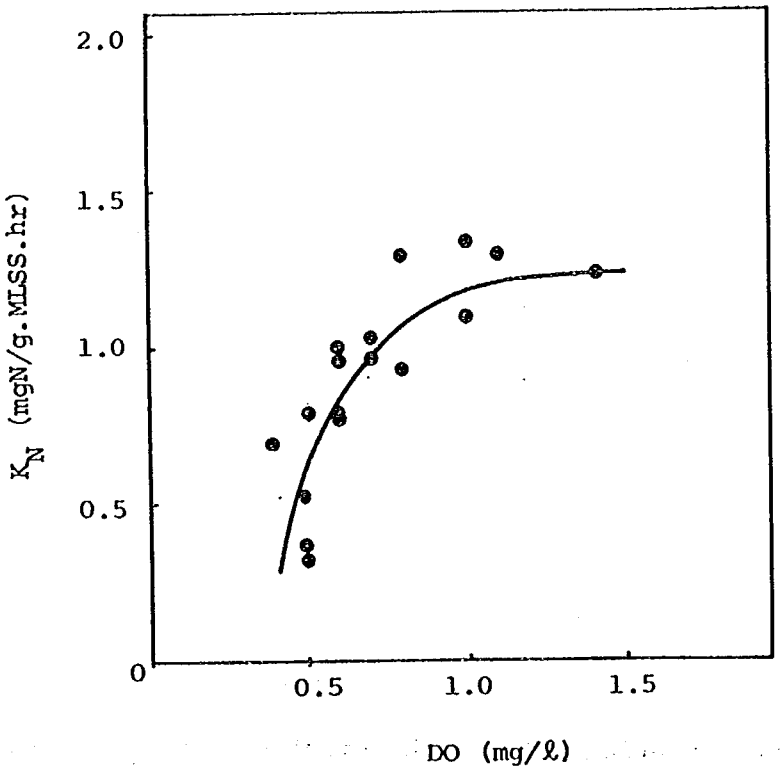


Fig. 24 Nitrification rate vs. DO concentration (first nitrification tank)

7.4.3 Nitrification in the Denitrification Tank

During most of the period (Run 1 to Run 8) when the F_2 tank was used as a denitrification tank, reduction in soluble k_j-N concentration (NH_4^+-N + soluble Org-N) was noticed in the tank. An example of soluble k_j-N balance is shown in Fig. 25.

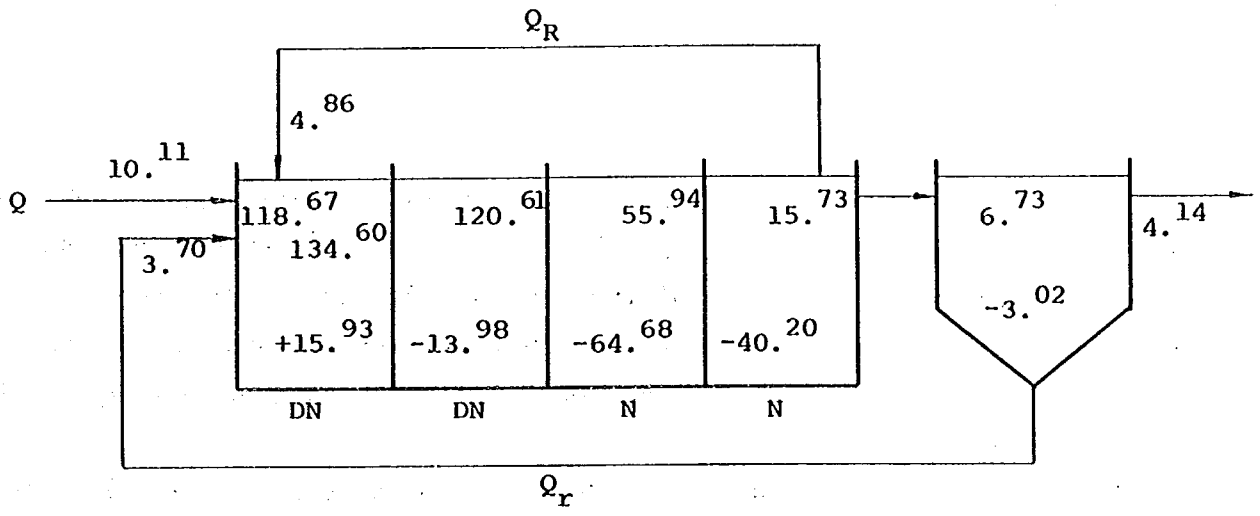


Fig. 25 S-k_j-N balance (June 3)

As shown, soluble k_j-N tended to decline in the second denitrification tank (F_2). In the first denitrification tank (F_1) soluble k_j-N increased and decreased erratically.

This may have been due to conversion of SS-Org-N to soluble k_j-N in the F_1 tank. The decrease in soluble k_j-N in F_2 tank have been caused when NH_4^+-N was nitrified into NO_2^--N or NO_3^--N and then immediately denitrified. This argument is supported by the alkalinity balance in F_2 tank (see Section 8). In this case, nitrification could have been caused by a small amount of oxygen entering F_2 tank from the boundary with the nitrification tank.

Because the NO_T-N produced by nitrification in the denitrification tank is denitrified immediately, this part of denitrification does not appear in the NO_T-N balance. For this reason, there should be figured in some amount of denitrified nitrogen due to nitrification and denitrification in the denitrification tank, in addition to the denitrified nitrogen quantity determined from the NO_T-N balance.

7.5 DENITRIFICATION

7.5.1 Denitrification Rate

The denitrification rate was measured using a testing apparatus. In the experiment facility, the capacity of the denitrification tank was large enough to denitrify all of incoming NO_T-N, and in most cases no NO_T-N was detected at the end of the denitrification tank. Unlike the nitrification rate, therefore, the denitrification rate of the experiment system could not be calculated.

The apparatus used for measurement was the same as for measurement of the nitrification rate. Tested sludge was sampled from the ends of the F_1 or G_2 tank.

Some results are shown in Figs. 26 and 27 with respect to F_1 sludge and in Figs. 28 and 29 with respect to G_2 sludge. In both cases, NO_T-N concentration shows a linear decrease. In the initial NO_T-N concentration range as used in these denitrification rate measurement, therefore, the reduction in NO_T-N concentration can be expressed as zero-order reaction:

$$\frac{dC}{dt} = K_{DN} \cdot X \qquad \ldots \text{Eq. 5}$$

Where C: NO_T-N concentration (mg/ℓ)
X: MLSS concentration (g/ℓ)
K_{DN}: Denitrification rate (mgN/g.MLSS.hr)

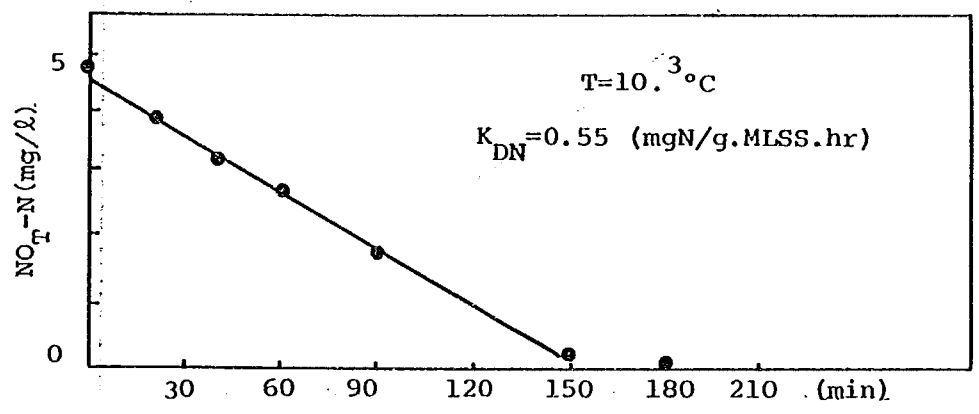


Fig. 26 Measurement of denitrification rate (F_1 sludge) (November 30)

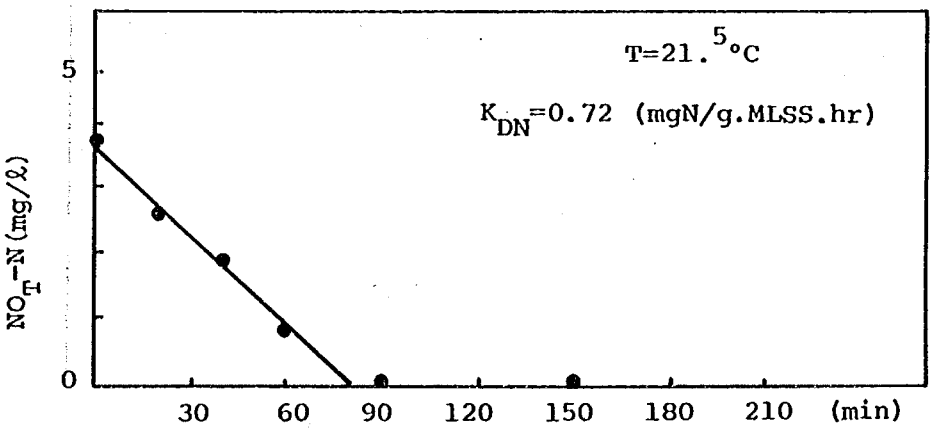


Fig. 27 Measurement of denitrification rate (F_1 sludge) (October 8)

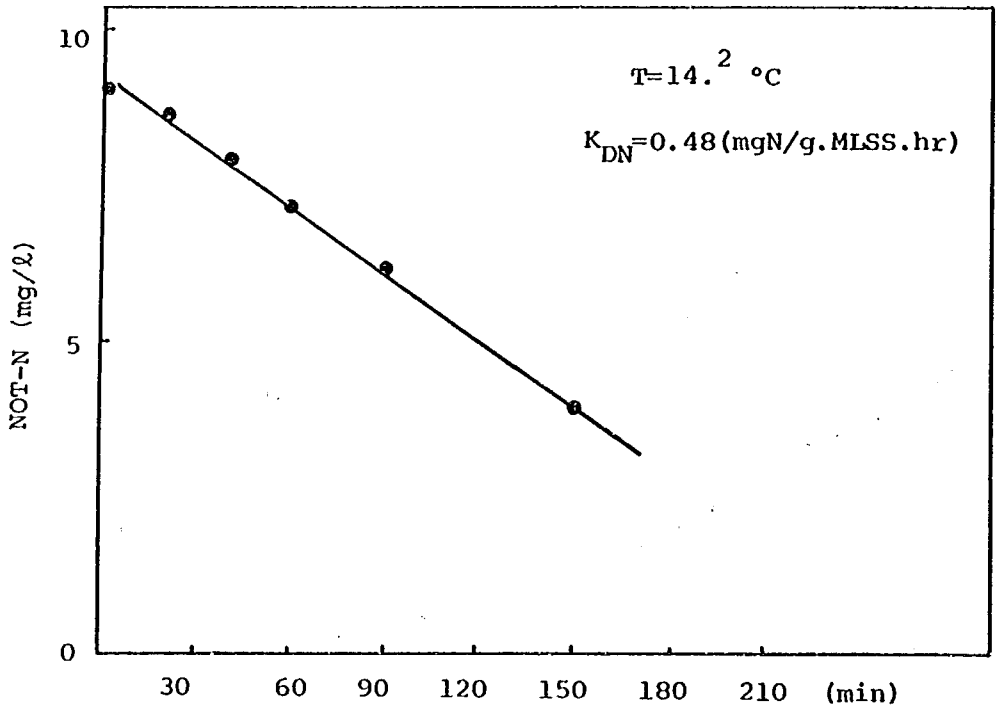


Fig. 28 Measurement of denitrification rate (G_2 sludge) (November 30)

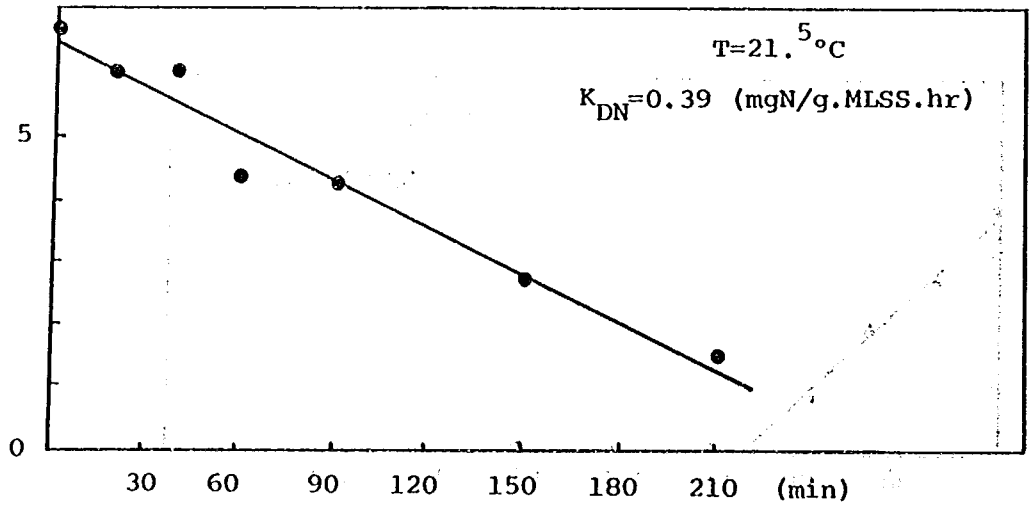


Fig. 29 Measurement of denitrification rate (G_2 sludge) (October 8)

(1) Denitrification rate of F_1 sludge

Figure 30 shows a plot of the F_1 sludge denitrification rate vs. water temperature on a single-logarithm graph sheet. The F_1 sludge denitrification rate can be expressed as a function of water temperature T:

$$K_{DN} = 0.287 \times e^{0.0687T} \quad \text{... Eq. 6}$$

Where K_{DN}: Denitrification rate (mgN/g.MLSS.hr)
T: Water temperature (°C)

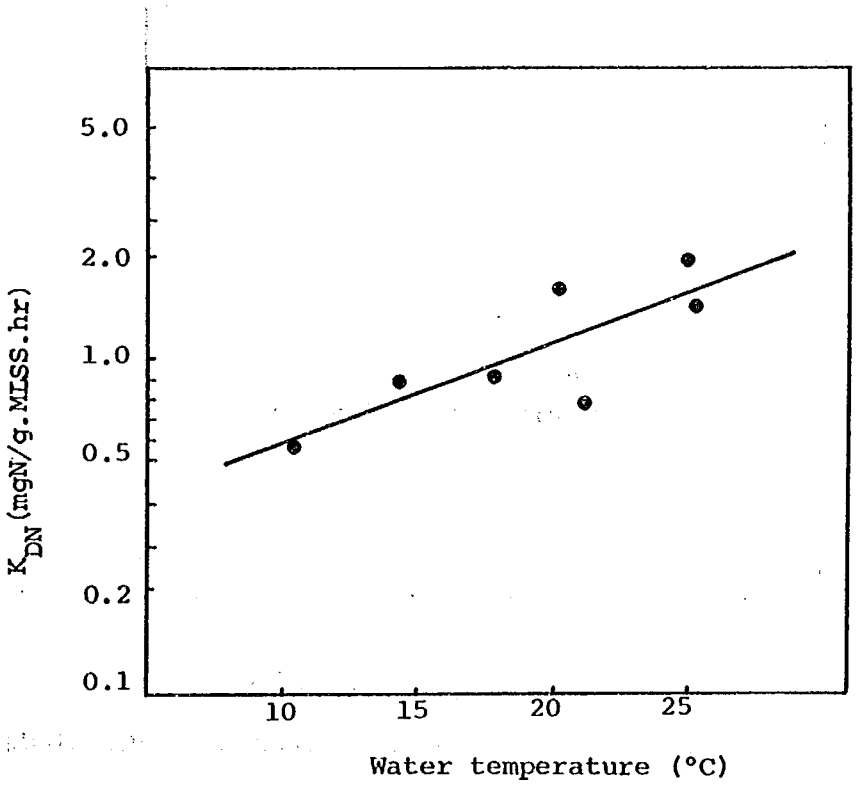


Fig. 30 Denitrification rate vs. water temperature (F_1 sludge)

According to Eq. 6, K_{DN} at a water temperature of 20°C gives 1.13 mgN/g.MLSS.hr.

(2) Denitrification rate of G_2 sludge

Figure 31 is a plot of the relationship between the denitrification rate of G_2 sludge and water temperature. It was found that the G_2 sludge denitrification rate is held almost constant at 0.37 mgN/g.MLSS.h on average regardless of water temperature.

In the case of G_2 sludge, S-BOD in the mixed liquor was 1 to 3 mg/ℓ, nearly equal to that of the treated water as it had been aerated for several hours. It is assumed that almost SS-BOD adsorbed to activated sludge may have been consumed.

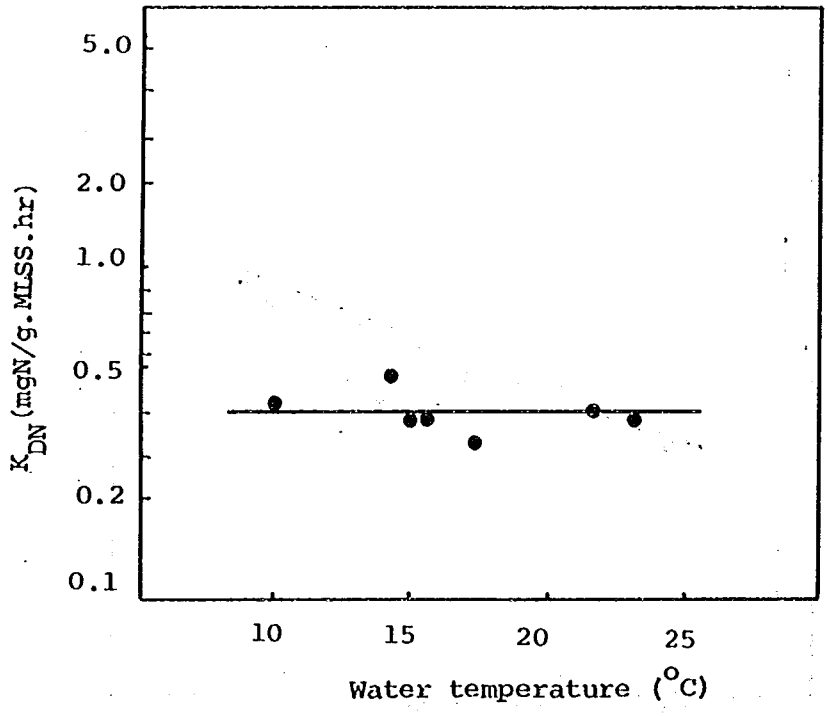


Fig. 31 Denitrification rate vs. water temperature (G_2 sludge)

Accordingly, it was inferred that in case of G_2 sludge denitrification is carried out by endogenous respiration. The denitrification rate is very small compared to that of F_1 sludge.

(3) Denitrification rate in the secondary settling tank

Denitrification in the secondary settling tank is also carried out through endogenous respiration under the sludge interface. The sludge characteristic in the secondary settling tank should be nearly the same as that in the G_2 tank. But its denitrification rate may be different from that of G_2 sludge because the MLSS concentration is increased due to thickening in the sludge hopper and because, unlike the denitrification tank, there is little possibility of contact between activated sludge and nitrate nitrogen through agitation.

The denitrification rate in the secondary settling tank was calculated by the following formula:

$$V = \frac{2,142 \text{ (Capacity of secondary settling tank)}}{L} \quad \text{... Eq. 7}$$

Where V: Sludge volume in the secondary tank (m^3)

L is the ratio of average sludge concentration in the secondary settling tank to MLSS concentration in the aeration tank -- that is, the sludge thickening ratio -- and can be defined as follows:

$$L = \frac{(MLSS + MLRSS)/2}{MLSS} \quad \text{... Eq. 8}$$

Where MLSS: Aeration tank MLSS concentration (g/ℓ)
MLRSS: Return sludge MLSS concentration (g/ℓ)

It should be noted that average sludge concentration in the secondary settling tank is taken as the mean value of the aeration tank MLSS concentration and return sludge MLSS concentration, and that the part of SS which runs off in treated water is assumed to be negligible.

Taking the denitrification rate in the secondary settling tank as k_{DNF} (mgN/g.MLSS.hr) and the numerator of Eq. 8 as A, the following formulas are established:

$$K_{DNF} \times A \times V \times 24 = NO_T\text{-}N \times 10^6 \quad \text{... Eq. 9}$$

$$V = \frac{2{,}142 \times MLSS}{A} \qquad \text{... Eq. 10}$$

$$\Delta NO_T\text{-}N = K_{DNF} \times 2{,}142 \times MLSS \times 24 \times 10^{-6} \qquad \text{... Eq. 11}$$

Where ΔNO_T-N: NO_T-N reduction in the secondary settling tank (kg/day)

In Fig. 32, the ΔNO_T-N determined from the NO_T-N balance is on the ordinate, with the 2,142 x MLSS x 24 x 10^{-6} on the abscissa. Thus the slope of the regression line in the figure gives the denitrification rate, K_{DNF}. K_{DNF} determined from Fig. 32 is 0.18 mgN/g.MLSS.hr or about half the denitrification rate of G_2 sludge.

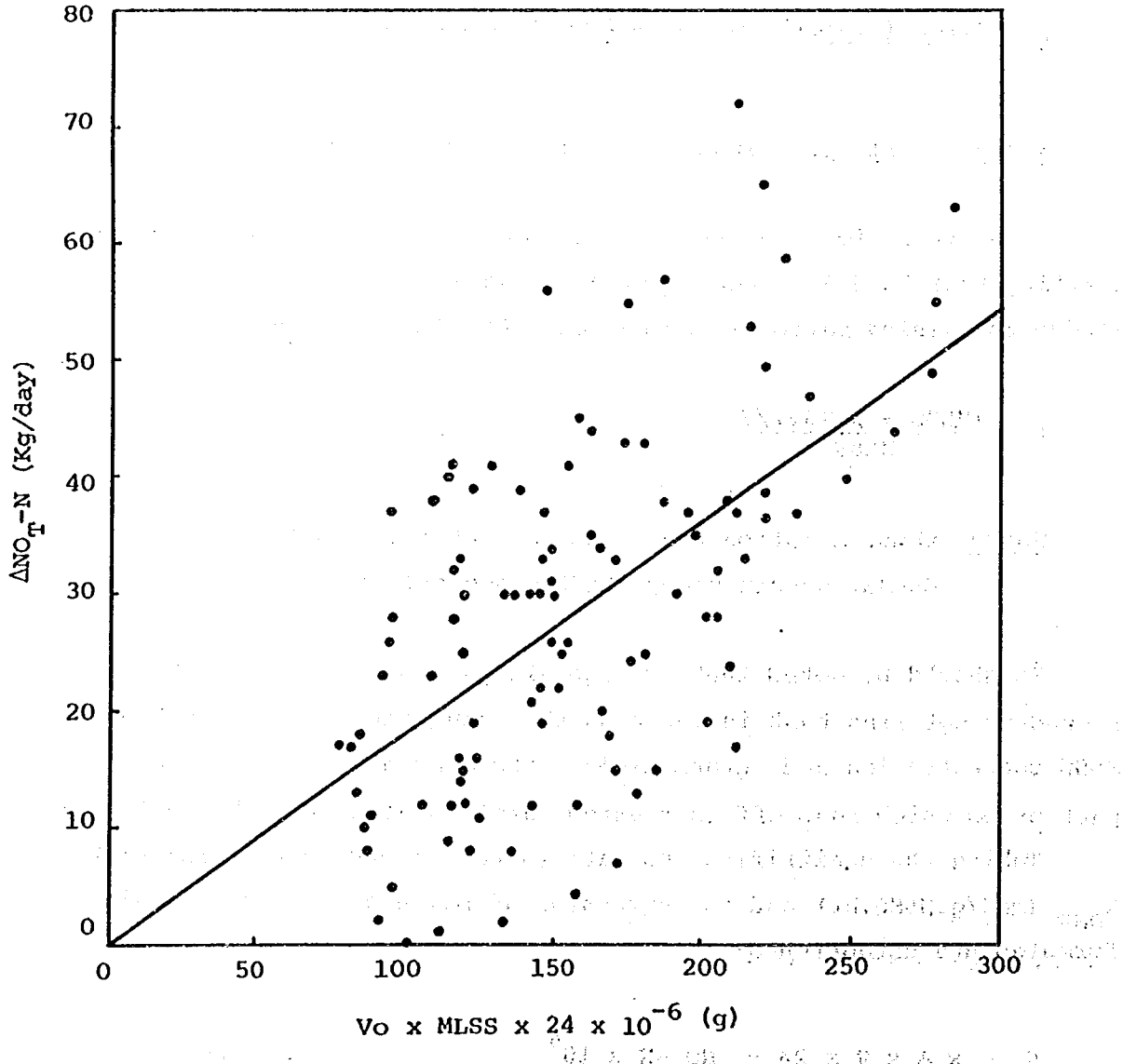


Fig. 32 Denitrification rate in the secondary settling tank

8. ALKALINITY BALANCE IN THE SECOND DENITRIFICATION TANK

It was stated in 7.4.3 that there is a possibility nitrification and denitrification occur in the denitrification tank. This was studied from the viewpoint of alkalinity balance.

In calculation of alkalinity balance, the following were hypothesized:

1. The reduction in soluble k_j-N is entirely due to nitrification and denitrification.
2. Soluble Org-N is at first totally converted into NH_4^+-N and then nitrified and denitrified.

The survey showed that alkalinity production or consumption per unit denitrification or unit nitrification generally follows the theoretical value. In the stage where soluble Org-N is transformed into NH_4^+-N, 3.57 mg of alkalinity is produced per mig of change. But this is followed by nitrification and denitrification. As a result, the whole process does not contribute to the change in alkalinity.

Accordingly, the reduction in alkalinity is expressed by the following formula:

Alkalinity reduction (mg/ℓ)

$$= NH_4^+\text{-N} \times 7.14 - (NH_4^+\text{-N} + NO_T\text{-N}) \times 3.57 \quad \ldots \text{Eq. 12}$$

Where NO_T-N: Reduction of NO_T-N in the F_2 tank. (mg/ℓ)

NH_4^+-N: Reduction of NH_4^+-N in the F_2 tank. (mg/ℓ)

The alkalinity reduction calculated according to Eq. 12 and actual measurements are compared in Fig. 33.

In Fig. 33, the regression line correlating the calculated alkalinity reduction and measured alkalinity reduction is expressed by $y = 1.017X$. This verifies hypotheses (1) and (2). Therefore soluble k_j-N reduction in the F_2 tank can be considered due to nitrification and denitrification in the tank as stated in 7.4.3.

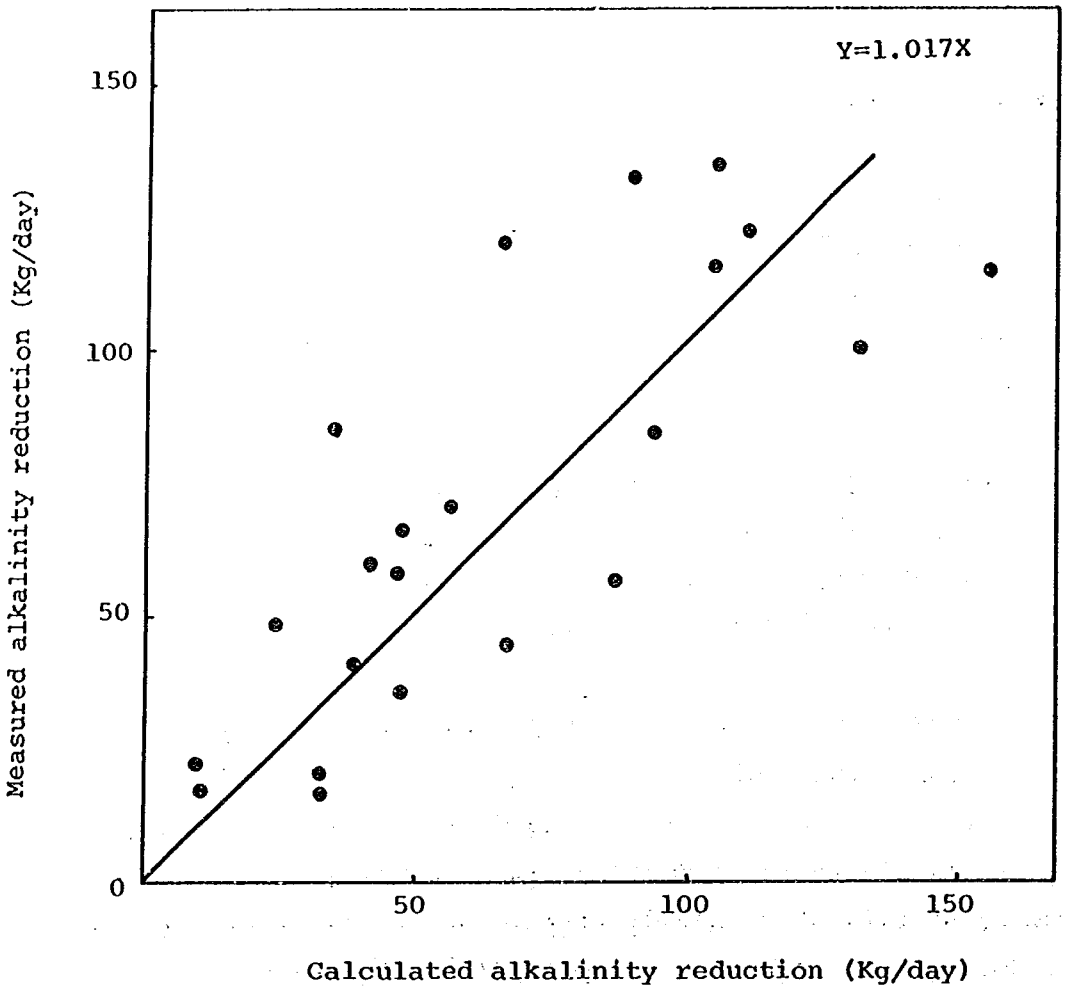


Fig. 33 Calculated vs. measured alkalinity reduction in the second denitrification tank (F_2)

9. REMOVABILITY OF ORGANIC MATTERS AND OTHER POLLUTANTS

9.1 BOD AND COD

As shown in Table 6, removal of T-BOD remained high throughout the survey. In most runs, the removal rate was around 95%, and the BOD concentration in treated water was below 5 mg/ℓ. Removal of S-BOD also went quite well, and in almost every case the S-BOD concentration in treated water runoff was below 3 mg/ℓ.

T-COD removal efficiency was also stable and high at around 80%, and T-COD concentration in treated water was usually below 10 mg/ℓ.

Runs 11 and 14 alone showed poor BOD and COD removal. In Run 11, typhoon rains affected the results. In Run 14, sludge bulking caused occasional runoff of activated sludge floc from the secondary settling tank increasing SS concentration in treated water.

Figure 34 shows an example of S-BOD and S-COD concentrations in each tank. in the denitrification tank, the dilution effect of recycled liquor, nitrate respiration and adsorption by activated sludge reduced S-BOD concentration in the mixed liquor already to less than 10 mg/ℓ, and further in the aerobic tanks. The S-COD concentration change is analogous to S-BOD.

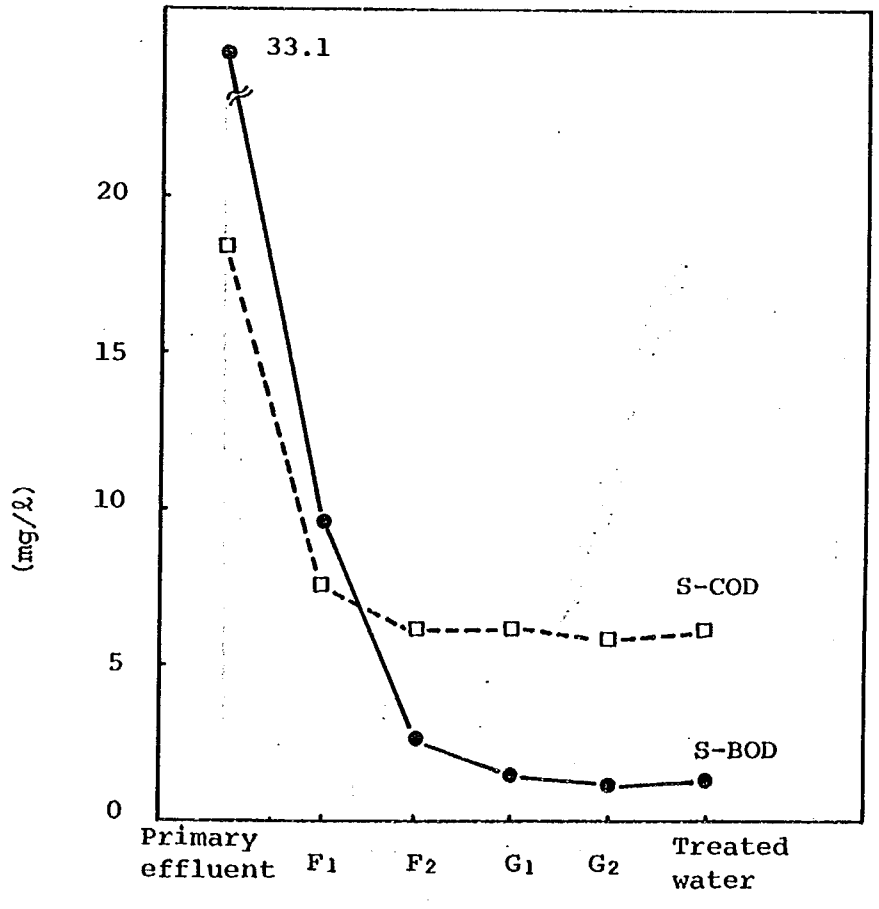


Fig. 34 An example of S-BOD and S-COD levels in each tank

9.2 PHOSPHORUS

Phosphorus removal rate fluctuated, but high removal rate was sometimes noted -- particularly when the ratio of denitrification tank volume to nitrification tank volume was equal. On these days, Orthophosphate (O-P) concentration in the denitrification tank rose while that in the nitrification tank went below 0.5 mg/ℓ and no nitrate nitrogen was detected in the denitrification tank.

When removal rate was high, nitrate nitrogen may have been reduced quickly in the denitrification tank, making the latter half of the denitrification tank perfectly anaerobic. Activated sludge may have released O-P into the mixed liquor which was taken up in the following anaerobic state.

Thus a phenomenon similar to the anearobic-oxic biological phosphorus removal process may have developed. Figure 35 shows the most typical example of this phenomenon.

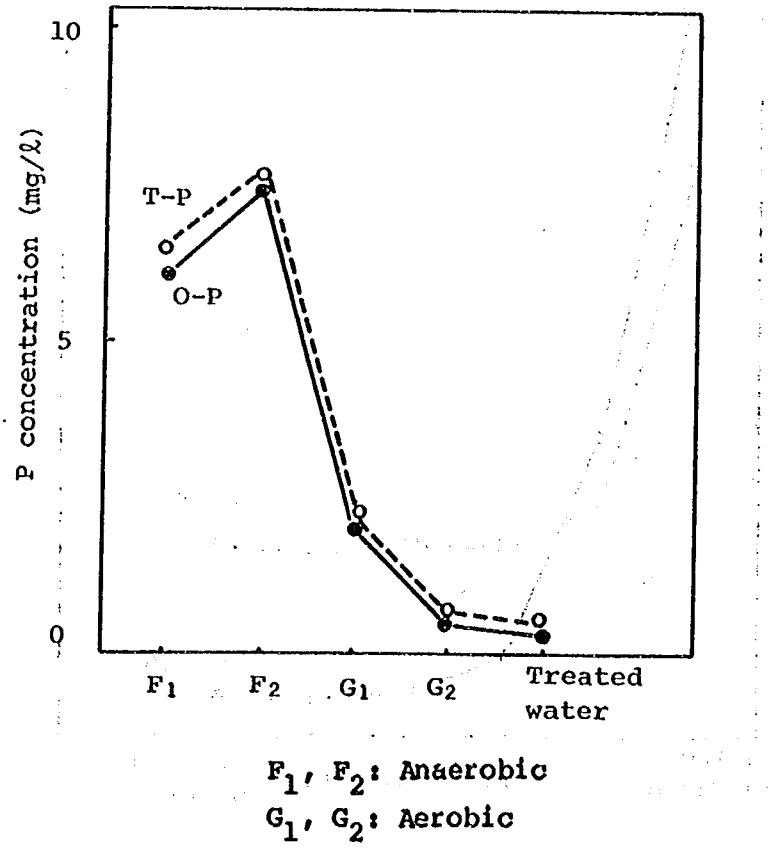


Fig. 35 P concentration change in each tank (July 6)

10. SLUDGE PRODUCTION

To study sludge production, data was used from periods when MLSS concentration in the aeration tank remained constant. BOD removal quantity, SS removal quantity and excess sludge volume and so on were calculated and are shown in Tables 8.1 and 8.2. In the calculation, SS in treated water was not included in excess sludge volume.

Table 8.1 Excess sludge production during low-temperature period (on an MLSS basis)

			(A)	(B)	(C)			
Period	Water temperature	MLSS (mg/ℓ)	BOD removal (kg/day)	SS removal (kg/day)	Excess sludge (kg/day)	$\frac{(C)}{(A)}$	$\frac{(C)}{(B)}$	$\frac{(C)}{(A)+(B)}$
May 1 to 3	16.2	2,130	479.7	298.3	117.6	-	-	-
May 29 to June 10	17.9	2,690	479.6	438.5	120.4	-	-	-
June 19 to 26	18.3	1,670	387.5	236.2	149.8	-	-	-
November 13 to 20	17.2	2,210	665.3	412.3	239.6	-	-	-
Average	17.4	2,200	486.9	344.6	144.7	0.297	0.420	0.174

Table 8.2 Excess sludge production during high-temperature period (on an MLSS basis)

			(A)	(B)	(C)			
Period	Water temperature	MLSS (mg/ℓ)	BOD removal (kg/day)	SS removal (kg/day)	Excess sludge (kg/day)	$\frac{(C)}{(A)}$	$\frac{(C)}{(B)}$	$\frac{(C)}{(A)+(B)}$
August 5 to 14	24.5	2,830	662.5	334.4	267.6	-	-	-
August 19 to September 9	23.5	3,040	754.6	471.5	230.3	-	-	-
September 11 to 30	22.2	3,470	554.6	346.9	168.1	-	-	-
Average	23.4	3,110	664.7	391.0	222.7	0.335	0.570	0.211

It was found that the ratio of sludge production to BOD removal was 0.297 during high-temperature period, and that the ratio of sludge production to SS removal was 0.420 during low-temperature period and 0.570 during high-temperature period.

The reason sludge production was higher at high temperatures than at low temperatures is unknown.

Suppose here that the following formula is established between excess sludge production and BOD removal:

$$\Delta X = Y(L_{in} - L_{out})Q - Kd \cdot X \cdot V \qquad \text{... Eq. 13}$$

Where ΔX: Excess sludge production (kg/day)

L_{in}: Influent BOD concentration (mg/ℓ)

L_{out}: Effluent BOD concentration (mg/ℓ)

Q: Inflow volume (m^3/day)

X: MLSS concentration (mg/ℓ)

V: Total aeration tank volume (m^3)

Y: Growth-yield coefficient (mass of microorganisms/mass of substrate utilized)

Kd: Microorganism-decay coefficient (day^{-1})

Note: ΔX does not include SS runoff.

By dividing Eq. 13 by X•V, the following formula is obtained:

$$\frac{\Delta X}{X \cdot V} = Y \cdot \frac{(L_{in} - L_{out}) \cdot Q}{X \cdot V} - Kd \qquad \text{... Eq. 14}$$

With Eq. 14 the relationship between (Excess sludge production/Process sludge quantity) and (Removed BOD quantity/Process sludge quantity) is expressed by a straight line. The slope of the line stands for the growth-yield coefficient (Y) and the intercept for microorganism-decay coefficient (Kd).

With respect to the experiment runs when MLSS was regarded as constant, Y and Kd were calculated according to Eq. 14. The results are shown in Figs. 36 and 37. In the figures, Y = 0.367 in the low-temperature period and 0.269 in the high-temperature period. In the low-temperature period Kd = 0.00365 (day^{-1}); in the high-temperature season, the intercept was positive and Kd could not be determined.

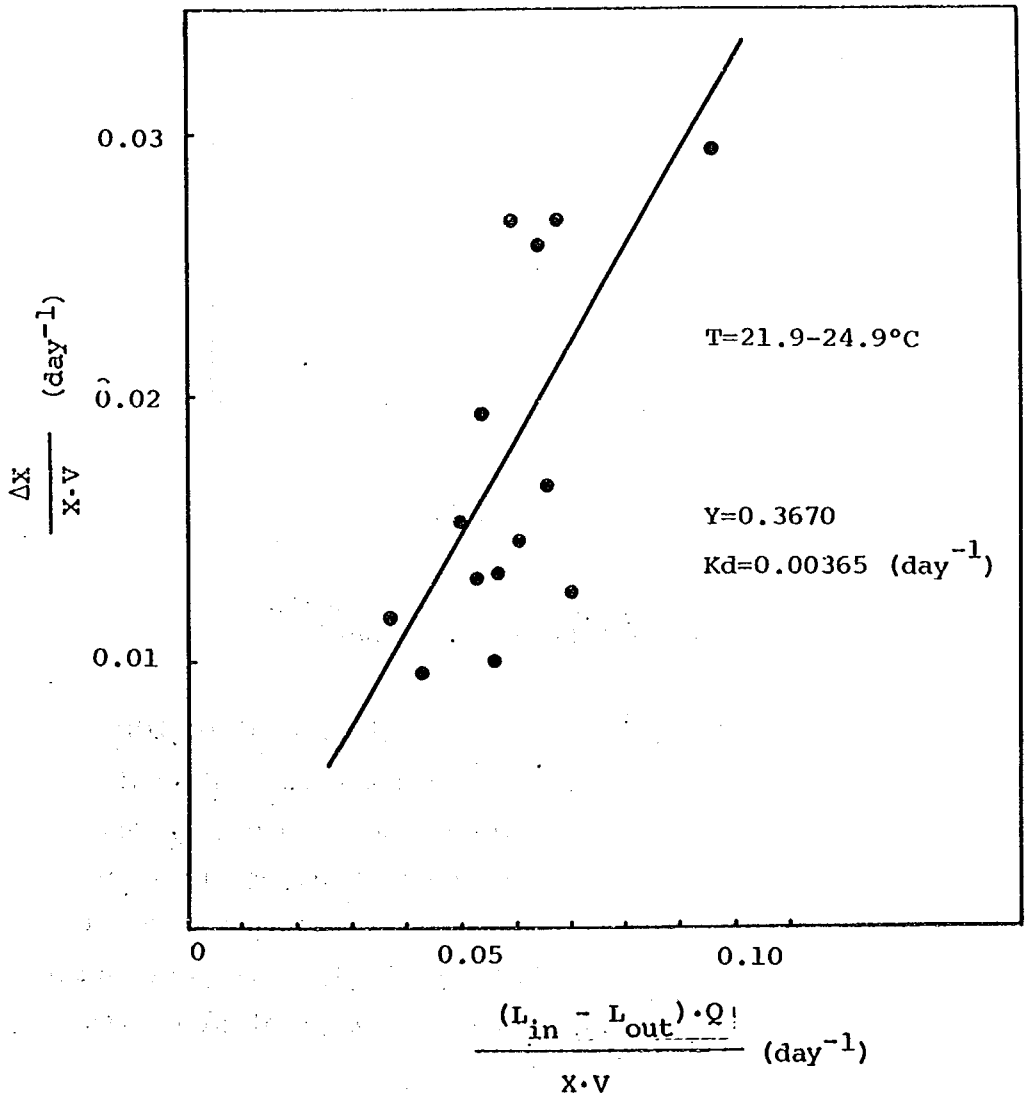


Fig. 36 Y and Kd in the low-temperature period

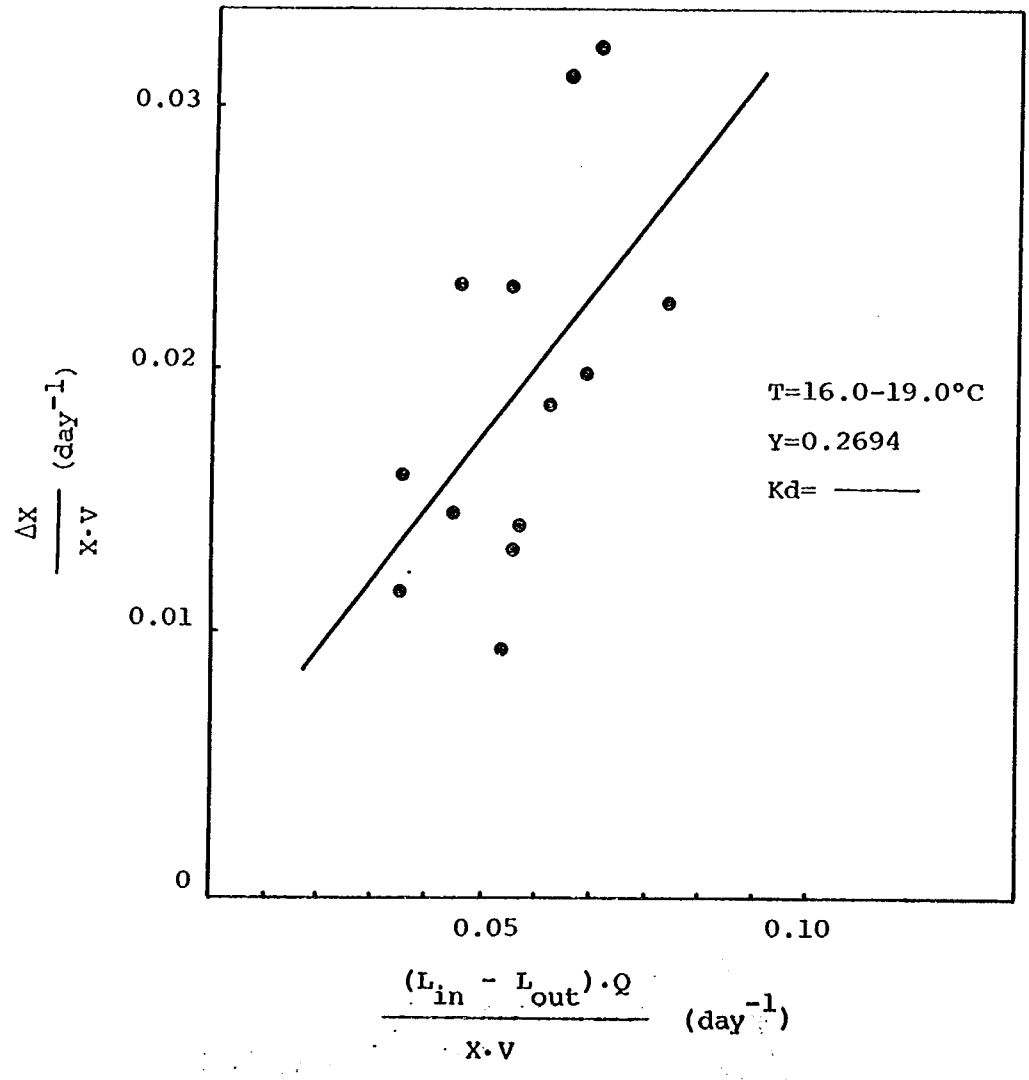


Fig. 37 Y and Kd in the high-temperature period

Figure 38 shows excess sludge production per unit amount of BOD removal as a function of SRT for comparison between the conventional activated sludge process and the nitrified liquor recycled biological nitrification-denitrification process, both tried in a pilot plant in 1978. Excess sludge production per unit amount of average BOD removal for each test run listed in Table 8.1 and 8.2 are plotted with triangular marks in Fig. 38, showing good agreement with the 1978 pilot plant test results.

The nitrified liquor recycled biological nitrification-denitrification process is usually operated with a long SRT because of the necessity of nitrification, and its excess sludge production is considered less than with the conventional activated sludge process.

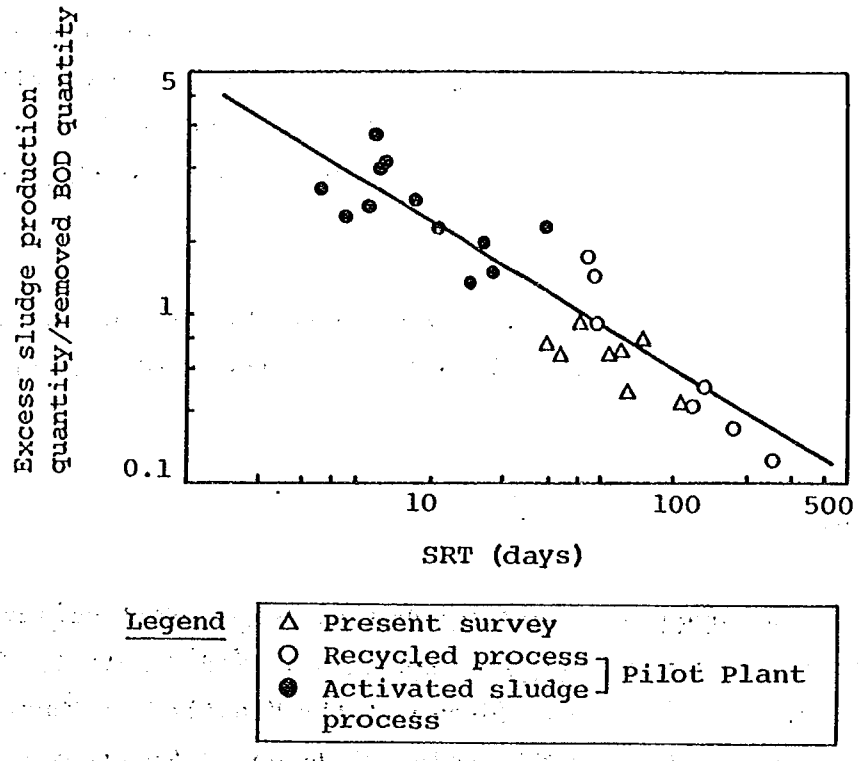


Fig. 38 Excess sludge production vs. SRT[1]

11. SUMMARY AND CONCLUSIONS

The survey findings were as follows:

1. When operated properly, the process showed a T-N removal rate of 60 to 70%, proving that it meets nitrogen effluent limit that is required to the sewage treatment plants around Lake Biwa.
2. From the study of NO_T-N balance, it was found that denitrification quantity in the secondary settling tank accounts for up to about 40% of total denitrification quantity.
3. Laboratory test results showed that the nitrification rate tends to remain constant when the water temperature exceeds about 17°C. But the experiment system data showed that the nitrification rate went down on the contrary when the water temperature exceeded 20°C. DO level in the nitrification tank may have been the cause.
4. According to laboratory tests, the denitrification rate was dependent on water temperature. But the denitrification rate in an endogenous respiration state became almost constant regardless of water temperature. Experiment system data showed that the denitrification rate in the secondary settling tank was about half that in an endogenous respiration rate.

5. The growth-yield coefficient (Y) was 0.367 in the low-temperature period and 0.269 in the high-temperature period. The nitrified liquor recycled biological nitrification-denitrification process was found to produce less excess sludge than the conventional activated sludge process.

A mathematical model will later be prepared for a general analysis of the nitrified liquor recycled biological nitrification-denitrification process.

12. REFERENCE WORKS

1. Wastewater Advanced Treatment Technology Development Survey Tertiary Treatment Pilot Plant Study for Lake Biwa; FY1979 Report, Vol. 2, Sewage Works Division, City Bureau of the Ministry of Construction; Civil Engineering Department, Shiga Prefectural Government; Japan Sewage Works Agency
2. A Study on the Total Nitrogen Removal from Sewage by Deep Reactor; H. Ishida, et al.: Environmental Technology, Vol. 10, No. 8, 1981.
3. An Analytical Study of Recycled Nitrification-Denitrification Process; K. Tanaka; June 1981.

Experiences with nitrification and denitrification processes

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1. Introduction

In West Germany more than 1ooo sewage treatment plants are nitrifying throughout the whole year and in about the same number of plants nitrification can be observed at least during some months. These plants are either activated sludge plants with aerobic stabilization or other biological plants where the design load is not yet reached. General demands as to a definite nitrification degree don't yet exist in Germany. The aim of water pollution control, however, is to keep the ammonium content in the receiving water below o,3 mg/l.

Due to the large portion of sewages in the surface waters the number of plants in West Germany for which nitrification is required by the controlling boards is steadily increasing. Large treatment plants in particular (i.e. in Munich, Hamburg, Frankfurt, Stuttgart and Nürnberg) are enlargened at present to achieve extensive nitrification.

A denitrification step for reducing the total nitrogen content has seldom been demanded until now in contrast to industrial works, which often have to limit the ammonium freight discharged into the receiving water. In most of these cases, however, attempts are made to fulfill the NH_4^+-N-standards by internal means for economical reasons.

Even if denitrification is not demanded by the controlling board it is recommended to the operators of the treatment plants to provide means for denitrification in those cases where nitrification is necessary as denitrification is followed by several operational advantages:

- reclamation of energy
- reclamation of alkalinity
- lower quantities of floated sludge in the final sedimentation tank
- less disturbances of the nitrification process and carbon degradation corresponding to the reduction of the pH-value
- prevention of the development of bulking sludge.

2. Nitrification processes

Ammonium nitrogen will be oxidized to nitrate in two steps by autotrophic microorganisms such as Nitrosomonas and Nitrobacter:

$$NH_4^+ + 1{,}5\ O_2 \longrightarrow NO_2^- + H_2O + 2\ H^+$$

$$NO_2^- + o{,}5\ O_2 \longrightarrow NO_3$$

For this reaction 4,6 g O_2 per g N are required. With the oxidation of 1 mol NH_4^+-N, 2 mol of H^+ will be set free, which react immediately according to the following equation:

$$2\ H^+ + 2\ HCO_3^- \longrightarrow 2\ CO_2 + 2\ H_2O$$

The oxidation of 4o mg/l N (mean value in municipal sewage in West Germany) leads to the decrease of 28o mg/l alkalinity in the sewage. Since the alkalinity in municipal sewage in most cases is smaller than 3oo - 4oo mg/l the pH-value may decrease significantly if most of the CO_2 remains in the system.

Because of the high gaseous exchange in trickling filters and rotating disc systems the pH-value in these systems never drops below 6 in the practice.

With a maximum dissolved oxygen content of 2 mg/l in the activated sludge system the pH-value, however, very often goes down to 6 or even 5 due to nitrification, upsetting the function of the activated sludge system. Possible remedies to this phenomena are the addition of chemicals or biological denitrification.

3. Technical solutions to achieve nitrification

3.1 Application of trickling filters

Operating one-stage trickling filters very low NH_4^+-N-contents in the final effluent can be achieved (fig. 1).

Extensive nitrification at temperatures in the filter of more than 1o ^oC is possible according West German experiences /1/ if the following design data are considered:

BOD_5-volumetric loading: o,2 kg/(m^3.d) with rock materials

BOD_5-surface loading: 2 g/(m^2.d) with regularly shaped filling elements

BOD_5 at the rotating sprinkler: 12o to 15o mg/l

In the winter season the air throughput through the trickling filter has to be limited in order to prevent an inadmissible amount of cooling. An effective outside isolation is necessary. A covering is recommendable. Feeding the filter with thin storm water over an extended period increases the ammonium content in the final effluent due to the considerable reduction of the contact time in the case of a high throughput. During continuous storm water flow recirculation of treated effluent is normally not necessary.

One-stage trickling filters with heights above 2,5 m are normally filled with rock material with grain sizes between 4o mm and 8o mm, providing a mathematical surface of 9o to 96 % and a porosity of about 5o %. Regularly formed synthetic filling elements show a larger geometric surface, that, however, is seldom completely accessible for wastewater and air. Synthetic filling materials that are generally available have not yet been used in West Germany for nitrification.

The efficiency of a trickling filter is always decisively influenced by the operational control. This is especially valid for plants with nitrification. To uniformly distribute the sewage on the filter surface is just as necessary as to keep the filter

body free from sludge. Even a hydraulic surface loading of more than o,8 m/h doesn't make a regular flushing of the trickling filter superfluous. The flushing intervals depend on local conditions and may range between 2 and 8 weeks. A strong fall of leaves can have bad consequences for the plant operation. A synthetic net covering has proved to be very useful even in winter.

3.2 Application of rotating contactors

With a one-stage rotating contactor an extensive nitrification can be achieved without problems (fig. 2). For this purpose the plant design in West Germany is based on the following data /1,2/:

BOD_5-surface loading with 3 cascades: 4 g/(m^2.d)

BOD_5-surface loading with 4 or more cascades: 5 g/(m^2.d)

The statistical values presented in fig. 3 result from investigations with BOD_5-loadings below 4,5 g/(m^2.d)/2/.

Cascade numbers of 6 to 9, that are often recommended in the U.S.A. don't provide optimal results for several reasons. Within the rotating biological contactor plant only the BOD will be eliminated in the first cascades whereas in the last cascades only nitrification takes place. By this excess sludge is only produced in the first cascades. In the case of 4 cascades of equal size the BOD_5 elimination is completed in the 2nd cascade. The BOD_5-surface loading of the first cascade thus amounts to 2o g/(m^2.d), whereas in a system consisting of 9 cascades the BOD_5-surface loading in the first cascade increases up to 45 g/(m^2.d). This high load may lead to odour nuisances. If the bottom of the tank is plain an additional aeration is necessary to prevent an oxygen deficiency. If the last cascades consist of discs with a structured surface the excess sludge is deposited there affecting nitrification.

If more than 4 cascades are considered and if a sludge deposit

on the bottom of the tank can't be excluded because of constructional factors, an intermediate clarification tank ahead of the nitrifying cascades is absolutely necessary.

The growth surface on rotating discs is not equivalent to the submerged area but must always be calculated by means of $2\pi r^2$. In the case of rotating contactors with structured surfaces only the surface that is freely accessible to wastewater and air can be considered as effective area.

The rotating biological disc contactors manufactured in West Germany show a water volume of 4 to 6 liters per m^2 growth surface.

A great advantage of the rotating biological contactor plants used for nitrification is the very low energy requirement which amounts to only about 5o Wh/kg BOD_5 removed (fig. 4). Besides, the excess sludge production of about o,5 kg/kg BOD_5 removed is considerably lower than in the activated sludge system, due to the very good oxygen supply of the whole active biomass on the fixed surfaces.

Sludge deposits on the bottom of the tank and on structured surfaces give a very adverse effect. Therefore it is necessary either to adjust the bottom of the tank to the shape of the discs or to prevent an abiding sludge deposit by means of aeration.

Because of the large water volume in the tank with its high buffering capacity the rotating biological contactor is less sensitive to cold weather than is the trickling filter. A covering is, however, indispensable just because of rainfalls. Again because of the long retention time in the tanks extended storm water flows don't affect nitrification. It was shown, that in the case of thin sewages nitrification even proceeds better at higher BOD_5-surface loadings because in this case a smaller BOD_5-concentration occurs in the contactor tank. An extensive nitrification is still possible at temperatures of about 5 °C. In the range above 5 °C nitrification is not significantly influenced by temperature.

3.3 Application of the activated sludge process

Extensive nitrification presupposes a sufficiently high concentration of specialized nitrate oxidizing organisms in the biological system. As the generation time of the autotrophic nitrifying bacteria in the activated sludge is much smaller than that of the heterotrophic organisms, converting the organic carbon, it is necessary to adjust the dimensioning sludge age to the growth rate of the nitrifying bacteria. Besides other environmental factors the growth rate is mainly influenced by the temperature of the sewage. When designing nitrification plants the following sludge ages and corresponding BOD_5-sludge loadings should be considered:

temperature °C	min. sludge age d	BOD_5-sludge-loading kg/(kg MLSS.d)
5	15	o,o8
1o	7	o,18
15	4	o,31
2o	3	o,42

Operating single stage activated sludge plants with BOD_5-sludge loadings below o,15 kg/(kg.d) or corresponding sludge ages above 1o days at temperatures of more than 1o °C usually leads to a nearly complete oxidation of the nitrogen compounds (fig. 5)/3/.

In the case of two-stage activated sludge plants nitrification takes place in the second stage /4/. With retention times of 2 hours in the first stage and 4 hours in the second stage the sludge age usually reaches such high values that an extensive nitrification can be achieved without any problems (fig. 6).

3.4 Combination of different biological systems

At present the effect of additional fixed or movable growth surfaces for microorganisms that are installed in the activated sludge tank (fig. 7) to increase the nitrification rate per unit of volume are tested /5/. By offering fixed growth surfaces it seems to be possible to provide better milieu conditions for flock-forming bacteria than for filamentous bacteria. As a consequence the sludge index goes down so that it is possible to maintain not only more heterotrophic but also more autotrophic bacteria in the system. The theoretically calculated increase of the number of nitrifying bacteria on the fixed growth surface has not been ascertained. It seems to be possible, if at all, only in plants with at least 4 cascades where nitrification exclusively occurs in the last cascade.

Fixed film reactors as the second stage in two-stage systems with the definite aim of ammonia oxidation have already been repeatedly applied. To guarantee a necessary minimum retention time in the case of trickling filters it is important to select relatively fine filter materials with a large specific surface.

The following data, concerning a trickling filter (lava cinders 4o/8o mm) are reported in /6/:

BOD_5-volumetric loading: o,17 kg/(m^3.d)

NH_4^+-N-volumetric loading: o,o5 kg/(m^3.d)

effluent BOD_5: 11 mg/l

effluent NH_4-N: 1 mg/l

If the BOD_5-removal in the first stage is too low or if the efficiency of the intermediate clarifier is insufficient, sludge deposits in the trickling filter, that are difficult to remove, may occur with fine filling materials. In the case of grain sizes below 2o mm it cannot be renounced on a backwashing device. Because of its high density quartz sand is not an optimum filling material in this sense. Burnt clay granules seem to

be better suited.

Because of economical reasons the application of fixed film reactors that must be backwashed will be advantageous only in special cases. Trickling filters with materials showing grain sizes between 3o and 6o mm appear to be more economical and to cause less operational problems. The large area required, however, is a disadvantage and a highly efficient intermediate clarifier is also necessary in the case of storm water flow.

The design of those filters is based on the following data according to /6/:

BOD_5-volumetric loading: o,2 kg/(m^3.d)

hydr. surface loading: o,8 m^3/(m^2.h)

height of the trickling filters: > 4 m

If rotating biological contactors are applied, a poor efficiency of the intermediate clarifier doesn't affect the efficiency of the second stage because the contactors cannot be clogged. Nevertheless installation of these systems in a second biological stage for nitrification should be dissuaded as the plant size must equal that of a one-stage plant to secure the necessary minimum contact time.

Investigations with the rotating biological disc filter as second stage for nitrification were characterized by the following data:

BOD_5-surface loading: 3,35 g/(m^2.d)

hydr. surface loading: 129,6 l/(m^2.d)

calculated retention time: 1,o h

NH_4-N in the effluent: 9,6 mg/l

The ammonia content in the final effluent of a two-stage plant is significantly higher than in the case of a system operated

as a one-stage plant.

On the other hand fully submerged contactors can altogether be recommended for nitrification in a second stage /7/. Here again it is, however, important to guarantee a reasonable efficiency of the intermediate clarifier to avoid sludge deposits and to provide a sufficient oxygen supply. If the CO_2 that is building in these systems is not removed, i.e. by aeration, the pH-value may drop distinctly into the acid range. Therefore addition of pure oxygen or H_2O_2 is not advantageous in general. Such systems should only be used in those cases where the alkalinity of the wastewater is high enough to provide a sufficient buffer.

To achieve a stable nitrification with fully submerged contactors, a minimum retention time of 1,5 hours in the case of storm water flow is necessary. Plants of this kind have not yet been technically tested in West Germany.

3.5 Comparison of the nitrification rate of different biological systems

A comparison of the efficiency of different biological systems as shown in table 1 is only possible if the composition of the wastewater and the length of the investigation periods are comparable.

With the one-stage rotating biological disc filter and consequently with all rotating contactors the maximum nitrification rate per m^2 growth surface, including BOD_5-removal, can be achieved. Also related to the water volume the corresponding values are quite favourable.

As the nitrification rate of constantly submerged systems is not higher than that of the conventional rotating biolocial contactors, taking the oxygen from the atmosphere, the application of the first is confined to special cases because of the higher operational costs for the oxygen supply.

Table 1

Nitrification rate of several biological systems

biological system	nitrification rate mg NH_4^+-N/(l.h)	nitrification rate mg NH_4^+-N/(m^2.h)	references
one-stage activated sludge plant	6,5	2,2	Bingel /4/
two-stage activated sludge plant	12 - 2o	4 - 7	Bingel /4/
one stage biodisc-filter	2o,8	1o4	Cheung /2/
fully submerged rotating contactor	15 - 37	4o - 1oo	Sekoulov /7/
submerged filter with burnt clay granules	15 - 22	4o - 6o	Sekoulov /7/
trickling f. as first stage (4o/8o mm)	2 - 3	2o - 3o	Wolf /6/
trickling f. as second stage (4o/8o mm)	2 - 4	2o - 4o	Wolf /6/

Considering the total costs (construction costs and operating costs) the following processes for nitrification are or will be applied in the future in West Germany:

Plant sizes up to 2o ooo equivalents:

- activated sludge system with a BOD_5-sludge loading of o,o5 kg/(kg.d)
- trickling filter with a BOD_5-volumetric loading of o,2 kg/(m^3.d)
- rotating biological disc system with a BOD_5-surface loading of 5 g/(m^2.d)

Plant sizes for more than 2o ooo equivalents:

- activated sludge system with a BOD_5-sludge loading of o,15 kg/(kg.d)
- two-stage activated sludge system
- combination of activated sludge system and trickling filter or vice versa.

4 Denitrification processes

The oxygen fixed in nitrite and nitrate can be used by many heterotrophic bacteria as a source of oxygen to break down organic carbon compounds:

$$NO_3^- + 5\ H^+ \longrightarrow 0{,}5\ N_2 + 2\ H_2O + OH^-$$

$$HCO_3^- + OH^- \longrightarrow CO_3^{2-} + H_2O$$

$$CO_3^{2-} + Ca^{2+} \longrightarrow CaCO_3$$

When denitrification proceeds in the activated sludge system, it is possible to maintain enough alkalinity in the system to protect the pH-value from dropping below 6 and to provide the heterotrophic bacteria with oxygen for respiration by pumping an adequate amount of nitrate containing effluent back to the denitrification stage.

About 2,9 kg oxygen per kg denitrified nitrate-nitrogen are set free and hence available for the oxidation of carbon.

Due to the relatively low energy recovery the decomposition of carbon with denitrification proceeds somewhat slower than is possible in the presence of dissolved oxygen, if otherwise comparable conditions are provided.

The respiration of dissolved oxygen and nitrate oxygen at any arbitrary point in the aeration tank is a function of the utilizable carbon present at this point. The respiration as well as the rate of denitrification increase with increasing contents of carbon and activated sludge. In fig. 8 the NO_3^--O-respiration is shown as a function of the BOD_5-loading.

5 Technical solutions for nitrification-denitrification

5.1 Secondary denitrification (fig. 9)

Considering the sequence of the chemical reactions it seems to be logical first to nitrify and then to denitrify afterwards. With secondary denitrification, however, oxygen is only needed for the endogeneous respiration of the activated sludge. As the corresponding oxygen demand is low denitrification proceeds very slowly, necessitating large reactors. To accelerate the oxygen consumption substrates free of nitrogen, i.e. methanol, must be added. Proper substrates, however, are costly and their feed causes an additional sludge growth with subsequent costs for an adequate sludge removal. Beyond that the oxidation of the residual substrate may require a further treatment step to meet possible overdoses that are unavoidable if an extensive denitrification is required. Due to these disadvantages the secondary denitrification of sewage is not applied in practice in West Germany.

5.2 Simultaneous denitrification (fig. 1o)

The volume of the activated sludge plant is determined by the sludge age necessary for nitrification. Fundamentally a slight denitrification can be observed with all lightly loaded activated sludge plants. To promote denitrification it is important that zones with dissolved oxygen are followed by zones without dissolved oxygen. This can be achieved by adding oxygen to the

system at one point or by intermittent oxygen input. The activated sludge, however, must not settle to the bottom of the basin. Consequently only an appropriate control of the oxygen supply is necessary to achieve simultaneous denitrification. With a complete mixing reactor simultaneous nitrification and denitrification are not possible.

Looking for advantageous types of basins the oxidation ditch or circulation basin (fig. 11) is to be mentioned first. As far as possible the influent and the return sludge should be mixed before and fed into the aeration basin behind the aerator. In circulation basins as well as in mixing basins the sewage quality at any point is practically the same as in the effluent due to complete mixing. This means, that the respiration per unit of time and hence the denitrification rate is small. Denitrification proceed significantly faster in a primary denitrification stage.

5.3 Primary denitrification (fig. 12)

The volume of the activated sludge plant is again determined by the sludge age necessary for nitrification.

Primary denitrification is characterized by the mixing of sewage, return sludge and recycled nitrified liquor in a zone that is largely free of dissolved oxygen.

An extensive nitrate removal can only be achieved with a correspondingly high internal recirculation and a high denitrification respiration. The rate of denitrification increases with increasing BOD_5 in the influent and increasing solids content in the activated sludge plant. Besides denitrification is positively influenced if there is no or only a small primary sedimentation. A retention time of o,5 h, related to the design dry weather flow, is always sufficient. The denitrification part should be anoxic. In the nitrification part the oxygen content must not exceed 2 mg/l. Otherwise the denitrification part receives a too large amount of oxygen due to the necessarily high recirculation rate. In any way it is difficult to clearly define the portion of the total aeration volume that should be provided for denitrifi-

cation.

According to the results up to now it can be stated that with an adequate supply of nitrate oxygen, biologically utilizable carbon compounds and temperatures above 1o ^{o}C the respiration in the absence of dissolved oxygen is about 75 to 8o % of that of activated sludge sufficiently supplied with dissolved oxygen. This means that the oxygen consumption with denitrification is about o,3 kg/(m^3.d). The corresponding oxygen demand can usually be covered by returning sludge and recirculation. Referring to a total oxygen demand of o,8 to o,9 kg/(m^3.d) the oxygen demand in the denitrification zone thus amounts to about one third.

If the BOD_5:N-ratio in the influent to the activated sludge plant lies in the range between 1oo:15 and 1oo:2o the zone without dissolved oxygen should occupy at most 25 to 3o % of the total volume of the aeration tanks. The denitrification zone must be enlargened with increasing nitrogen and reduced with increasing BOD_5 in the influent. A denitrification tank with vertical flow is ideal. As an alternative the lay-out providing cascades operated in series is also well suited, especially if surface aerators are used (fig. 13).

If the BOD_5:N-ratio is fluctuating distinctly a technical solution based on two separate, non-exchangeable tanks for nitrification and denitrification is not useful.

An optimum adaptation to the changing loading conditions throughout the life time of an activated sludge plant is possible by adequately operating a plug-flow basin (fig. 14), i.e. to provide anoxic zones ahead by controlling the oxygen input.

If circular tanks are prefered it is useful to place the denitrification part in the center and to separate it from the nitrification part by light division walls (fig. 15 and fig. 16). The denitrification part can be reduced to about 15 to 2o % if the sewage flow is nearly constant or if an equalizing tank is available.

In the case of a flow dependent recirculation of nitrate containing sewage to the denitrification part the mixing of the sludge in both parts of the basin must be secured independent from the

oxygen input.

Provision, however, should be made for operating the denitrification part also for nitrification without essential additional expenditure.
The return sludge flow should amount to between one and two times the design dry weather flow. The recirculation rate should be adjustable in the same order of magnitude. By this means the best possible adaptation to the changing composition of the sewage can be achieved.

If there is additional energy required for recirculation and mixing, it is generally not advisable to increase the return sludge and the recirculation ratio respectively above 1oo % because the total energy costs will significantly increase without adequate advantages. Under these operational conditions it is possible to reduce the nitrogen content (including the excess sludge production) of about 75 %. The poorer the aeration system and the more concentrated the sewage (BOD_5, nitrogen) the higher are the energy savings due to denitrification.

The surface of the denitrification basin must be kept free from installations and the floated solids must not be hindered from flowing out to the nitrification part.

To achieve a definite high denitrification rate in two-stage activated sludge plants is considerably more difficult than in single stage plants because nitrification exclusively takes place in the second stage at a very low carbon respiration rate and because in case of nitrification with short retention times (i.e. 4 h) no zones without dissolved oxygen can be provided.

The reduction of the alkalinity and possible accompanying problems with biodegradation due to low pH-values only occur in the second stage.

The undisturbed continuous operation of a two-stage activated sludge plant with extensive nitrification is only possible if the sewage contains a high natural alkalinity. Otherwise the alkalinity must be artificially be increased. The optimum solution, i.e. to operate the second stage constantly near oxygen satura-

tion, thereby guaranteeing a high CO_2-stripping, is not to be recommended because of the disproportionately high costs.

All attempts to achieve a considerable denitrification also with the activated sludge in the second stage haven't been successfull until now. Feeding the second stage with raw sewage practically leads to a single stage plant.

Consequently an increase of the alkalinity can only be achieved by denitrification in the first stage. There it proceeds in the fastest and most safe way. Besides the oxygen regained in the first stage mediately may contribute to the prevention of odour nuisances.

The flow diagram in fig. 17 is in accordance with these considerations.

According to our actual view the recirculation flow must be fully considered with the dimensioning of final sedimentation basins that are conventionally formed.

Due to the usually required recirculation rate of 1oo %, resulting in high costs, a two-stage activated sludge plant with extensive nitrification is not expedient if the alkalinity of the sewage is not extraordinary high or if it won't be possible to operate the final sedimentation in a way, that the recirculation flow needs not to be considered completely with dimensioning of the sedimentation basins.

Denitrification in a two-stage activated sludge plant without the difficulties mentioned above seems to be possible with the A-B-system of Böhnke /8/. In this system only adsorption is done in the first stage (BOD_5-reduction about 5o %), so that the second stage, where denitrification takes place, is fed with a relatively high concentration of easily biodegradable organic carbon (BOD_5 > 1oo mg/l).

5.4 Other possibilities for denitrification with the activated sludge system

The alternate denitrification (fig. 18), that is propagated in Denmark /9/, is not yet applied in West Germany. There are, however, some activated sludge plants under construction including anaerobic and anoxic stages ahead of the aerobic stage, similar to the Barnard-system /1o/ developed in South-Africa (fig. 19).

5.5 Possible nitrogen removal

Very often denitrification is taken for nitrogen elimination or the reduced nitrogen in the influent is compared only with the reduced and the oxidized nitrogen in the effluent respectively. Proceeding this way distinct differences as to the denitrification rate or the nitrogen removal may be obtained as demonstrated by the example presented in table 2.

Table 2

Denitrification and nitrogen removal rates in theory and practice

	influent mg/l N	effluent mg/l N		in excess sludge mg/l N	denitrification %		nitrogen removal %	
		theor.	pract.		theor.	pract.	theor.	pract.
N_{org}	1o	o	1	12	65	6o	95	9o
NH_4	4o	o	1					
NO_2	o	o	o					
NO_3	o	2	3					
N_{tot}	5o	2	5		72	66	96	9o

Hence it follows that an accurate nitrogen balance presupposes the determination of the nitrogen in all of its possible chemical combinations both in the influent and in the effluent. Beyond that it is necessary to determine the amount of excess sludge and the nitrogen content in it. Thus it is fundamentally impossible to calculate the denitrification capacity by means of the analytical data of random samples. To obtain reliable results an investigation period of at least one week is necessary.

To check the function of a denitrification plant on the other hand it is sufficient to measure the BOD_5 and the NH_4^+-N or NO_3^--N in the influent. Referring to a BOD_5:NH_4^+-N-ratio in the range between 4 and 6, usually meaning a good plant efficiency, the effluent quality should correspond to the following data:

$$NH_4^+\text{-N} < 1 \text{ mg/l}$$
$$NO_2^-\text{-N} < 1 \text{ mg/l}$$
$$NO_3^-\text{-N} < 3 \text{ mg/l}$$

Given optimum plant operation, the same results can be achieved with both simultaneous and primary denitrification. Whereas in case of simultaneous denitrification the oxygen must be added to the system at one point there are no limiations as to the choice of the oxygen transfer aggregate with primary denitrification. In the latter case it should be possible to vary the size of the zones without dissolved oxygen between 15 and 3o % as to the total tank volume. If thin sewages are to be treated (i.e. in case of storm water flow or considerable infiltration of ground water into the sewerage) denitrification distinctly deteriorates with both processes.

5.6 Denitrification with fixed film reactors

The number of possible solutions to accomplish denitrification in fixed film reactors is limited. Until now only fully submerged reactors like rotating biological contactors were technically examined (fig. 2o).

With the design of those systems the same aspects are to be considered as in the case of designing an activated sludge plant with a primary denitrification step.

The calculation of the total surface area can be based on a BOD_5-sludge loading of 5 g/(m^2.d). About 1o to 2o % of the total area has to be provided for the primary denitrification. Due to the recirculation of the effluent the contact time in the nitrification part is reduced. Considering a possible deterioration of the nitrification step the recirculation rate should - according to our experiences - not exceed 3oo %. Only in the case of highly concentrated sewages higher rates are useful.

5.7 Conclusions

Both the activated sludge system and the rotating biological disc system are suited for denitrification because they can easily be adapted to this purpose. At the present time fixed film reactors for denitrification ahead of an aerobic stage with the sewage as H-donator cannot be recommended for practical use.

In West Germany the following design data for biological nitrification/denitrification are proposed:

- BOD_5-sludge loading of the whole activated sludge plant (nitrification tank and denitrification tank) between o,o5 and o,15 kg/(kg MLSS.d)
- BOD_5-surface loading in the case of rotating biological discs (nitrification and denitrification part) 5 g/(m^2.d).

6 Practical experiences

6.1 General aspects

Results going back to the operation of sewage treatment plants in Germany that are designed for denitrification and supervised

by the operating staff only are not known. The results discussed below originate from different treatment plants investigated with the support of scientific institutes.

6.2 Simultaneous denitrification

Kayser /11/ has comprehensively investigated simultaneous denitrification in circulation basins with a BOD_5-sludge loading of about o,o5 kg/(kg MLSS.d) using a nitrite/nitrate-autoanalyzer for controlling the oxidized nitrogen.

The design of treatment plants with a nitrification stage should always consider a possible denitrification stage to save energy and to avoid operational problems such as scum on the surface of the final sedimentation tank or a drop of the alkalinity below 1oo mg/l $CaCO_3$.

The design data in table 3 were received by means of the nitrite/nitrate-autoanalyzer.

Table 3

Operational results of a sewage treatment plant with simultaneous denitrification (19.9.81 - 19.1o.81)

	influent	effluent
N_{org} in mg/l	13,5	2,o
NH_4-N "	26,5	7,o
NO_3-N "	o	1,2
BOD_5 "	2oo	4
COD	514	54
F/M-ratio kg/(kg MLSS.d)	o,o5	
Temperature in °C (mean)	1o	

6.3 Sewage treatment plant Biet-Tiefenbronn /12/

Fig. 15 shows the single stage activated sludge system at Biet-Tiefenbronn in Southern Germany with primary denitrification in a circulation basin.

The plant is operated with an organic loading of less than o,o5 kg BOD_5/(kg MLSS.d). There is no dissolved oxygen in the inner basin and the dissolved oxygen content in the outer basin is between 1 and 2 mg/l.

In table 4 the most important results are summarized. During the investigations the total nitrogen removal rate was not affected by the increase of the recycling ratio from 17o to 26o %. At dry weather flow the denitrification rate ranged between 4o,o and 45,5 %. Neglecting the soluble organic nitrogen in the effluent, however, great differences in the total nitrogen elimination were found. In table 5 the results at dry weather flow with and without considering the content of the soluble N_{org} in the effluent are listed. The content of N_{org} due to the suspended solids in the effluent was below 1 mg/l.

Table 5

Total nitrogen removal in % with and without considering the soluble organic nitrogen in the effluent

period	with soluble N_{org}	without soluble N_{org}
1	62,2	74,1
2	74,6	81,3
3	69,2	79,7
6	63,4	79,5
7	6o,2	79,7

Table 4

Operating data referring to primary denitrification in the single stage activated sludge plant at Biet-Tiefenbronn

		period 1	period 2	period 3	period 6	period 7
recirculation rate (including return sludge)	DWF	33o %	5oo %	48o %	26o %	38o %
	SWF	22o %	23o %	22o %	17o %	26o %

		DWF	SWF	DWF	SWF	DWF	SWF	DWF	SWF	DWF	SWF
tot. N influent (= 1oo %)	mg/l	3o,1	17,o	37,1	17,1	38,4	18,o	36,7	25,5	39,4	31,8
tot. N elimination	%	62,2	4o,1	74,6	53,8	69,2	46,6	63,4	47,1	6o,2	45,2
incorporation in the excess sludge	%	22,2	14,9	29,1	24,3	26,9	17,3	21,1	27,5	16,9	25,5
denitrification degree	%	4o,o	25,2	45,5	29,5	42,3	29,3	42,3	19,6	43,3	19,7
NH_4^+-N effluent	mg/l	o	o,4	1,4	o,6	1,9	o,7	o,8	o,3	o,7	o,7
NO_3^--N effluent	mg/l	7,5	7,3	4,7	3,5	5,4	7,3	6,4	5,5	6,7	8,9

Besides the nitrogen balance some further aspects were investigated. Fig. 21 shows the content of the soluble organic nitrogen in the effluent versus the dissolved oxygen content in the nitrification part.

Low contents of soluble organic nitrogen in the effluent were only achieved at O_2-contents in the nitrification part of at least o,5 mg/l. Increasing the oxygen content led to a decrease of the N_{org}-content.

In fig. 22 the NH_4^+-N-content in the effluent is presented as a function of the minimal oxygen content in the nitrification part. If the oxygen content constantly exceeds o,5 mg/l the NH_4^+-N-content in the effluent is constantly below 1 mg/l.

6.4 Investigations in the research sewage treatment plant at Stuttgart-Büsnau

6.4.1 Single-stage activated sludge plant with 5 cascades

Fig. 23 shows the single-stage activated sludge system with denitrification ahead of the aeration basin.

The combined influent and recirculated streams flow vertically upwards through a Dortmund-tank. Denitrification takes place as a result of the absence of dissolved oxygen. In the following aerobic stage the remaining BOD will be removed and the nitrogen will be oxidized.

The results obtained during two investigation periods are summarized in table 6.

Taking into account the amount of nitrogen that cannot be broken down and that utilized for cell growth, denitrification amounted to 66 %. With increased recirculation rate the nitrogen removal could be even higher.

Table 6

Characteristic data referring to the operation of the single-stage activated sludge system at Büsnau in the period between 3.7.8o and 31.7.8o (7.9.8o - 7.1o.8o)

Operational data			effluent mg/l		removal %	
BOD_5			15	(7)	85	(94)
$KMnO_4$-consumption			27	-	72	-
Alkalinity			151	(135)	34	(4o)
SS			4,4	(5)		
NH_4-N			2,6	(1,9)	82	(92)
tot. N			1o,1	(1o,o)	56	(62)
BOD_5 influent (mg/l)	1oo	(112)				
F/M ration	o,23	(o,2o)				
MLSS (kg/m^3)	3,5	(3,2)				
sludge index (ml/g)	85	(8o)				
sludge recirculation rate (%)	1oo	(1oo)				
temperature (^oC)	14-17	(12-16)				
retention time in the DN-tank at $Q + Q_r$ (h)	2,7	(2,7)				

6.4.2 Single stage activated sludge plant with 3 cascades

Figure 24 shows the single stage activated sludge system with two compartments for denitrification and one compartment for nitrification.

The first and second cascade are stirred without oxygen input whereas the third cascade is aerated with diffusors.

The BOD_5-sludge loadings were:

3,3	kg/(kg.d)	in the first cascade
o,47	"	in the second cascade
o,19	"	in all cascades together.

The following characteristic values were measured in the effluent:

Temperature (mean)	16 °C
pH-value	7,55 - 8,o
alkalinity	96 mg/l $CaCO_3$
BOD_5	3 mg/l
COD	34 mg/l
total nitrogen	24,2 mg/l
NH_4-N	o,5 "
NO_3-N	17,o4 "
NO_2-N	o,3 "
N_{org}	6,o

The total nitrogen removal amounted to 57,2 %.

6.4.3 Two-stage activated sludge plant

The flow sheet of the two-stage activated sludge system, investigated at Büsnau in 1979, is presented in fig. 9. Table 7 lists the results of the extended investigation period.

An increase of the recirculation ratio leads to a higher degree of denitrification. In this case, however, the intermediate and the final clarifier must also be larger. In practice this turns out very quickly to be cost effective.

Because of the high costs of construction, denitrification in a two-stage activated sludge plant is not recommended if the BOD_5-reduction in the first stage exceeds 5o %.

Table 7

Characteristic data referring to the operation of the two-stage activated sludge system at Büsnau in the period between 29.5.79 and 1.11.79

	influent	1. stage	2. stage mean	2. stage range
BOD_5 (mg/l)	199	53	6	(1-22)
COD (mg/l)	433	168	5o	(24-118)
$KMnO_4$-consumption (mg/l)	246	1o1	5o	(22-9o)
alkalinity (mg/l)	-	259	73	(48-93)
NH_4-N (mg/l)	5o	3o	o,8	(o-12)
N_{ox}-N (mg/l)	o	o	27	(16-44)
nitrogen removal		overall removal 45 %		
SS			7	(1-18)

operating data	with primary settling	sludge recirculation	nitrified effl. recirculation
		465 m^3/d	3oo m^3/d
influent	465 m^3/d = 19,4 m^3/h		
flow rate	765 m^3/d = 31,8 m^3/h		
mean retention time (h)	1/3,6		
MLSS (kg/m^3)	2,8/2,9		
sludge index (ml/g)	94/1oo		
F/M ratio (1st stage)	1,o		

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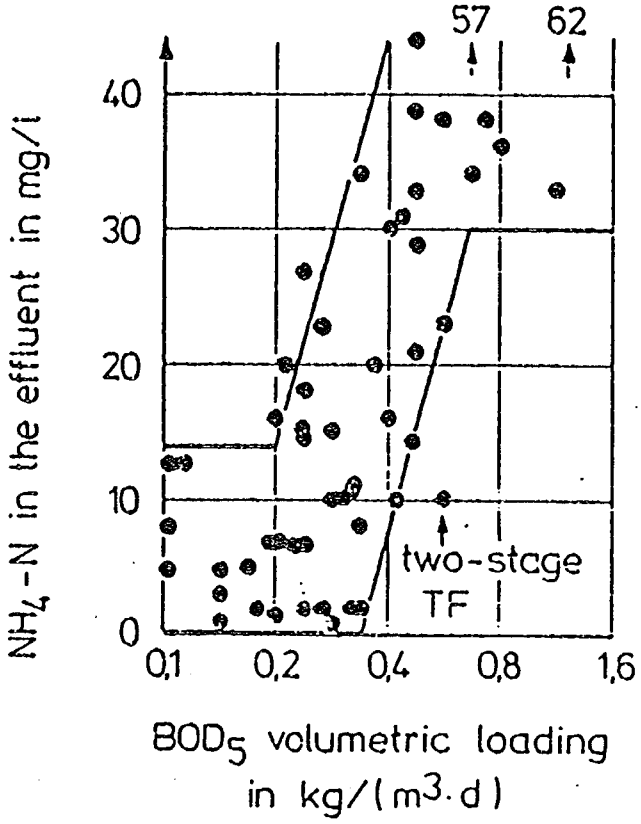


fig. 1 NH_4^+-N in the effluent of trickling filters versus BOD_5-volumetric loading

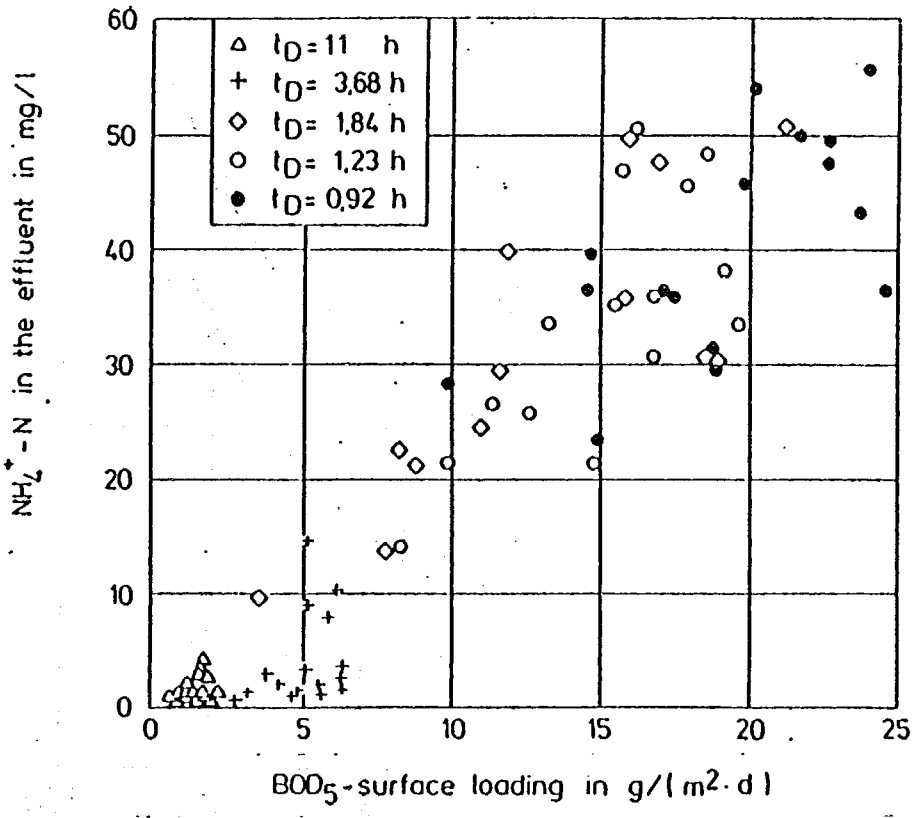


fig. 2 NH_4^+-N in the effluent of rotating biological disc filters versus BOD_5-surface loading

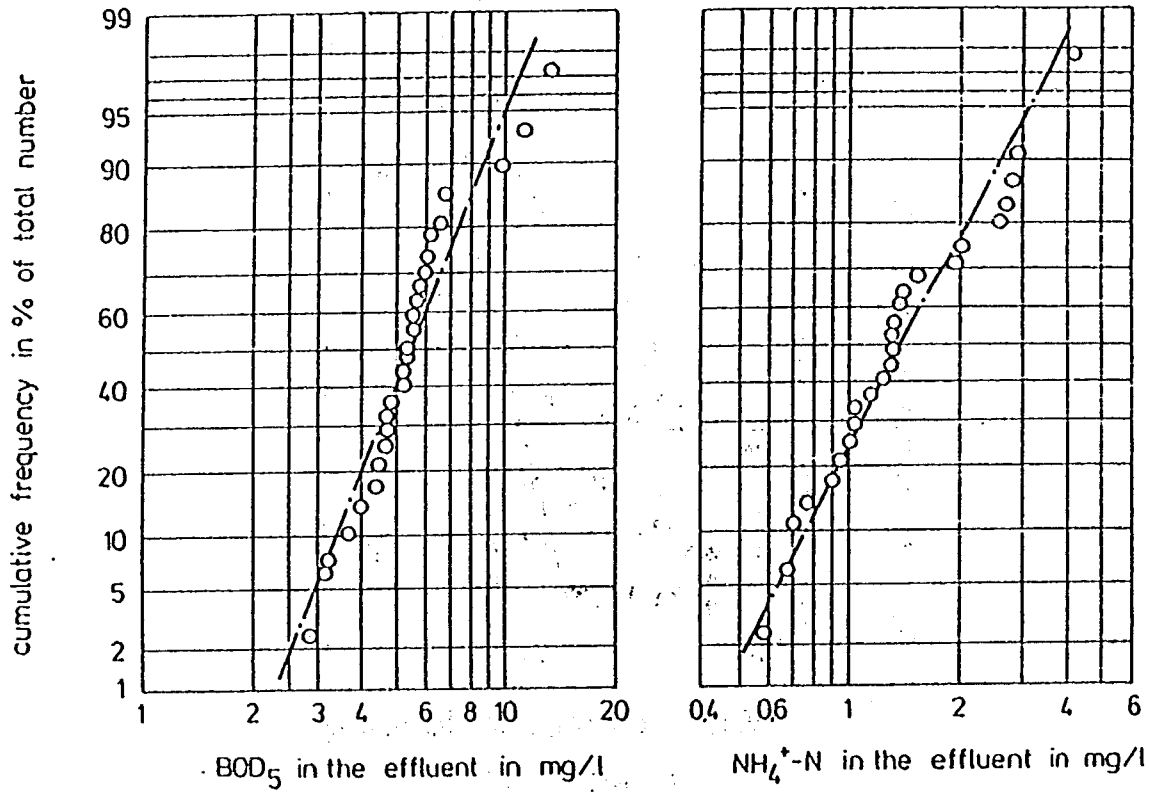


fig. 3 frequency distribution of BOD_5 and NH_4^+-N in the effluent of a rotating biological disc filter (B_A < 4,5 g/(m^2.d))

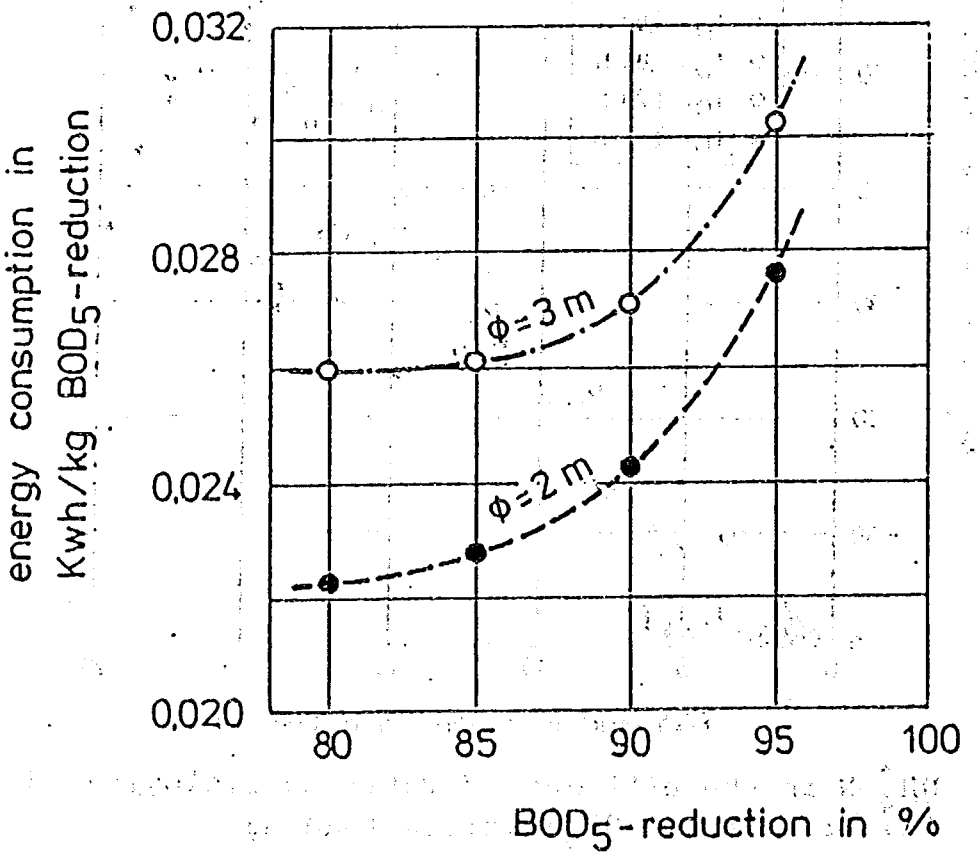


fig. 4 energy consumption of the rotating disc system

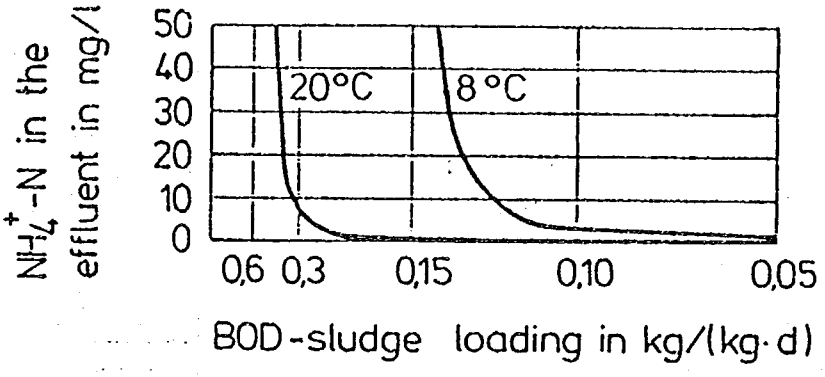


fig. 5 NH_4^+-N in the effluent of activated sludge plants versus BOD_5-sludge loading

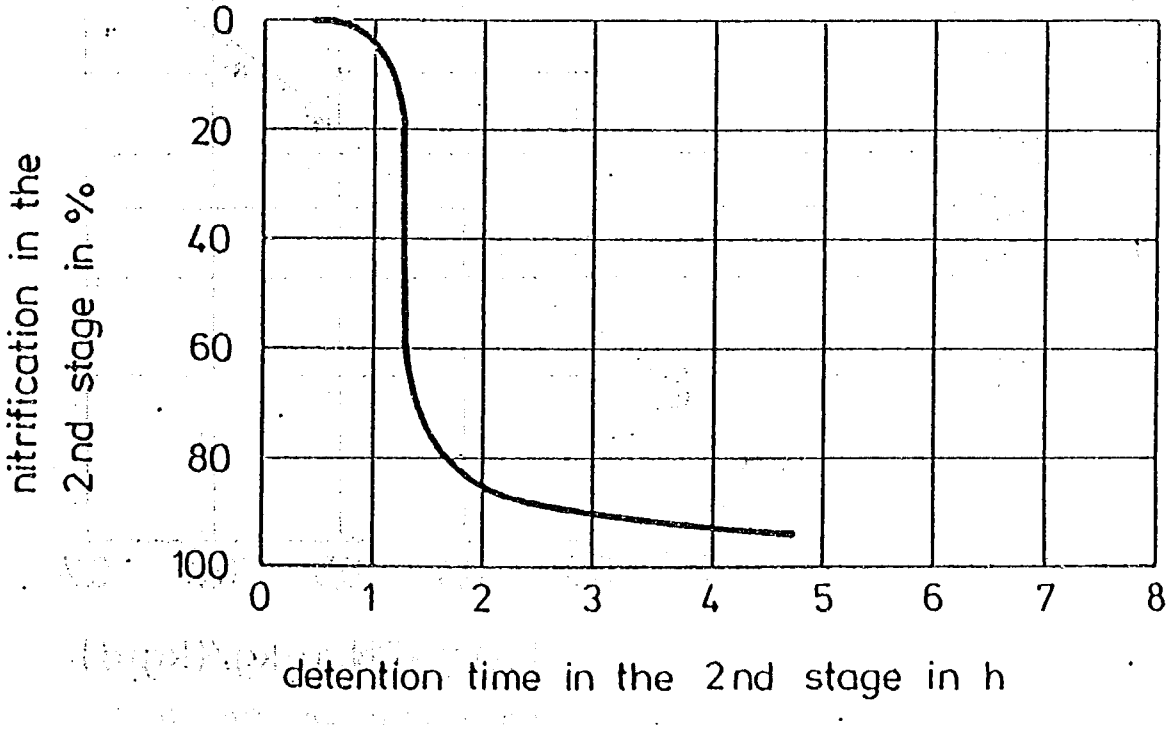


fig. 6 nitrification degree in the second stage of an activated sludge system versus retention time

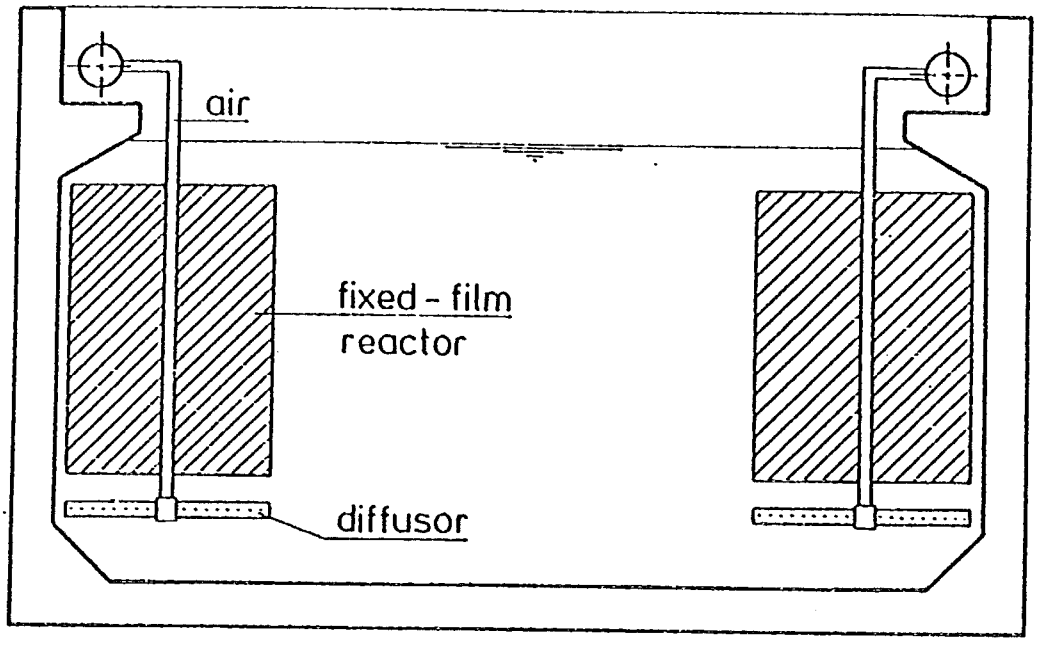
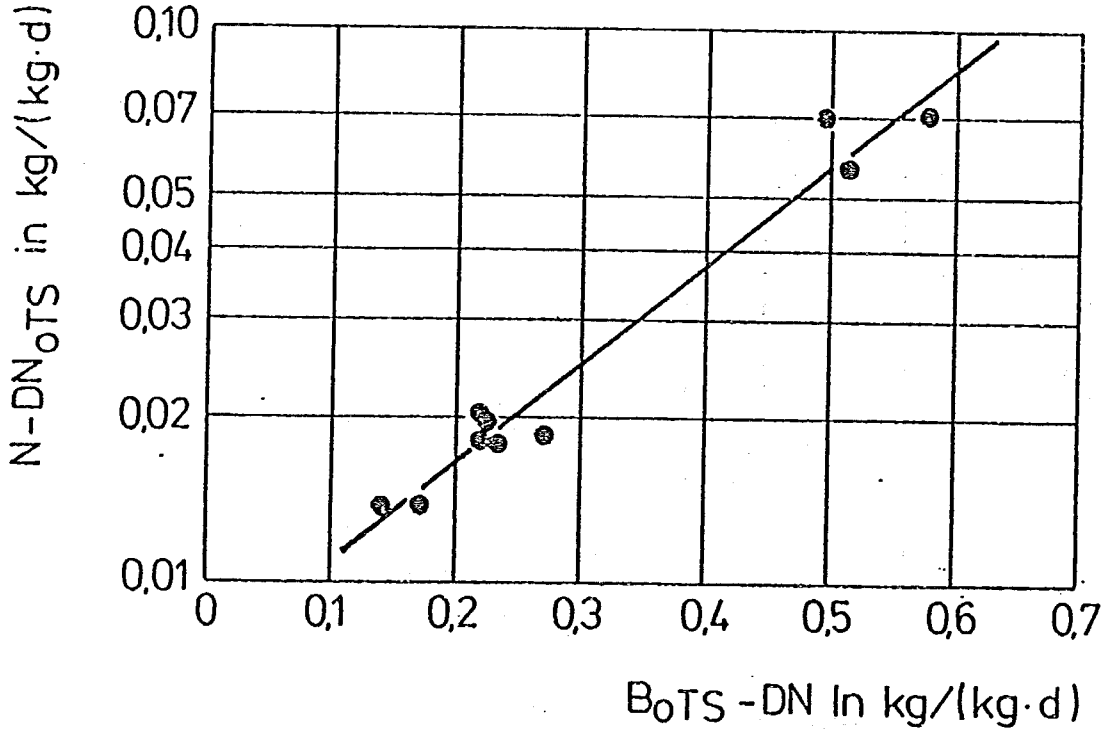


fig. 7 schematic diagram of the Bio-2-sludge system



B_{oTS} BOD_5 organic sludge load in the DN-compartment

$N\text{-}DN_{oTS}$ N-elimination in the DN-compartment

fig. 8 measured NO_3^--O-respiration versus BOD_5-loading

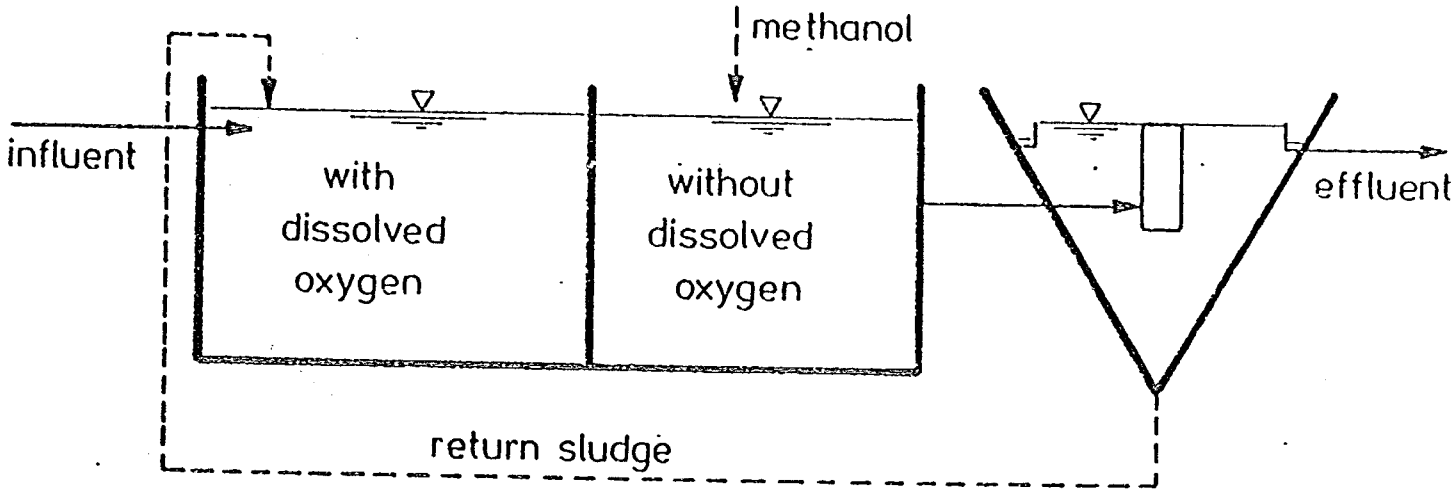


fig. 9 secondary denitrification system

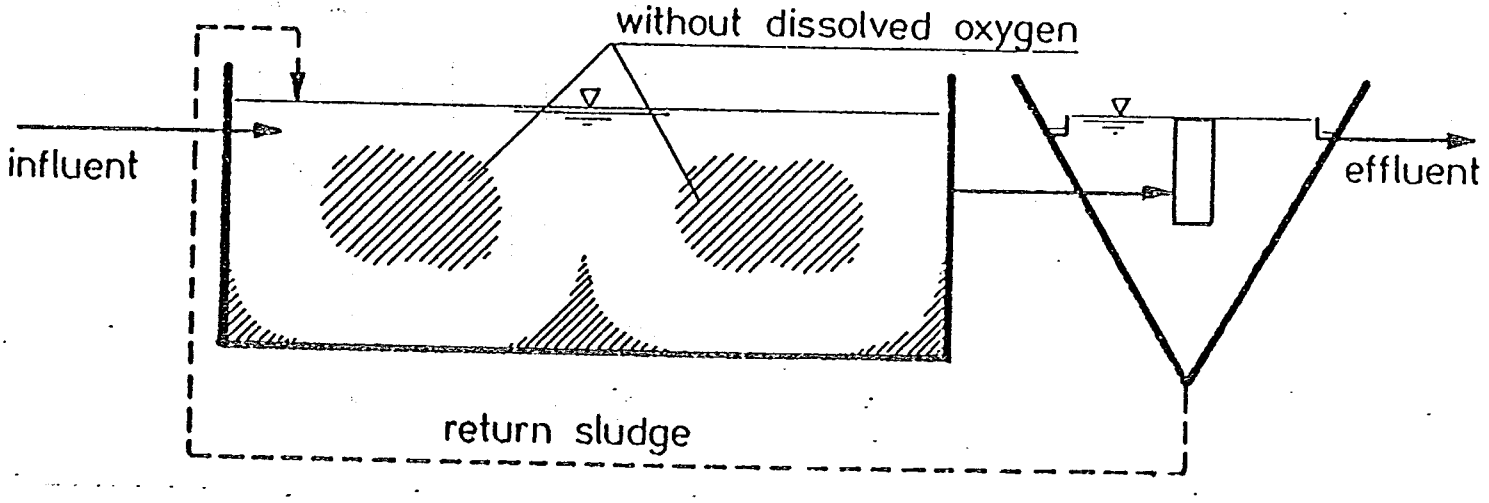


fig. 1o simultaneous denitrification system

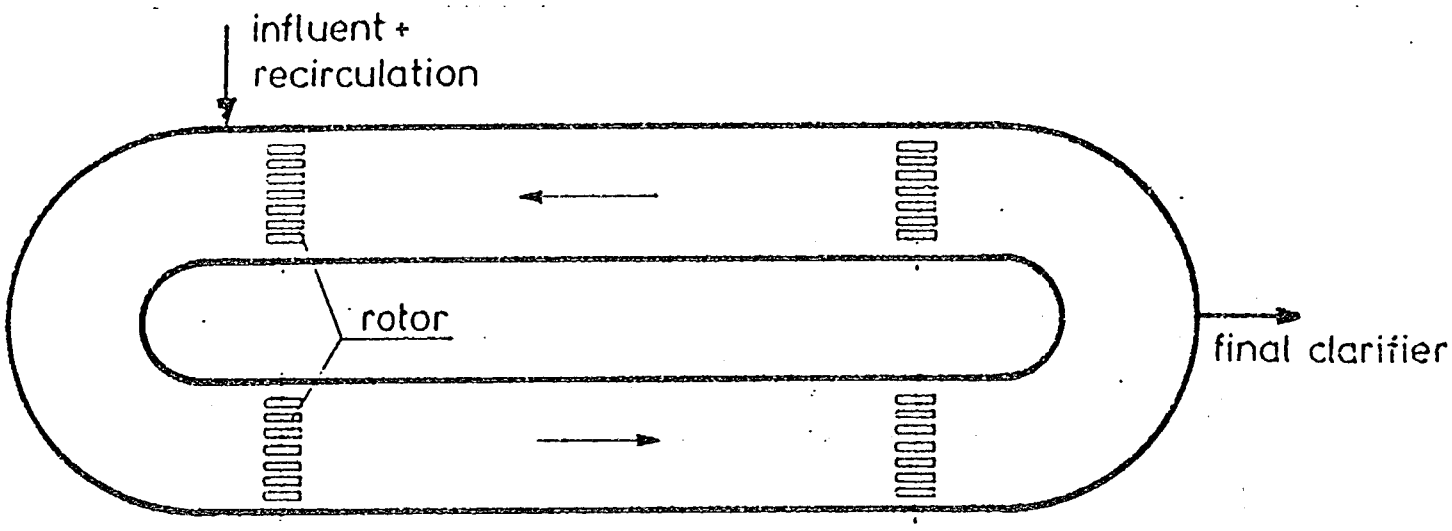


fig. 11 schematic diagram of an oxidation ditch

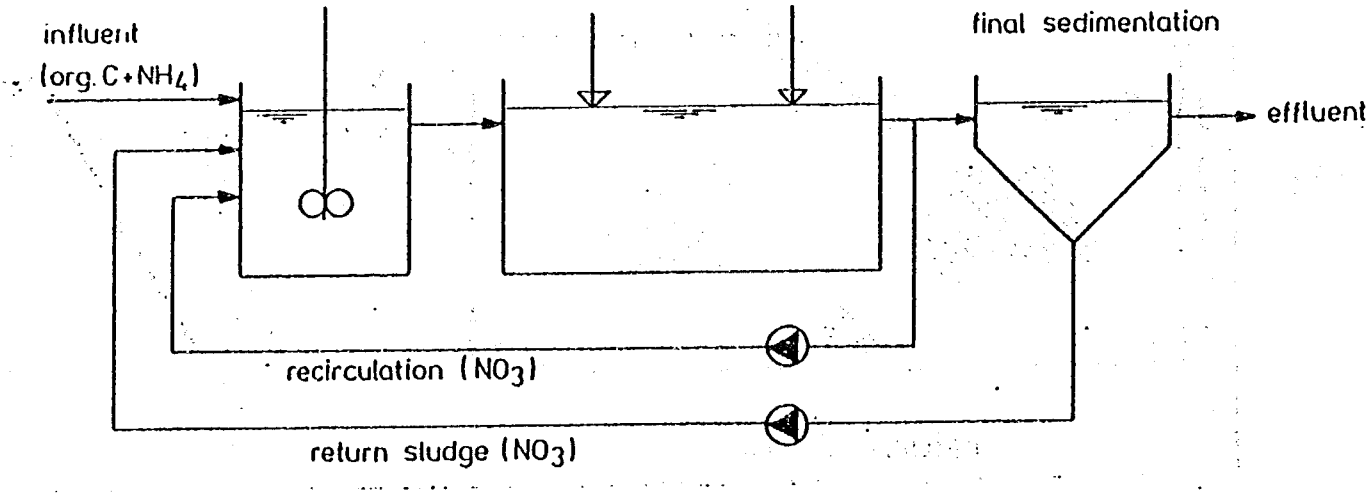


fig. 12 primary denitrification system

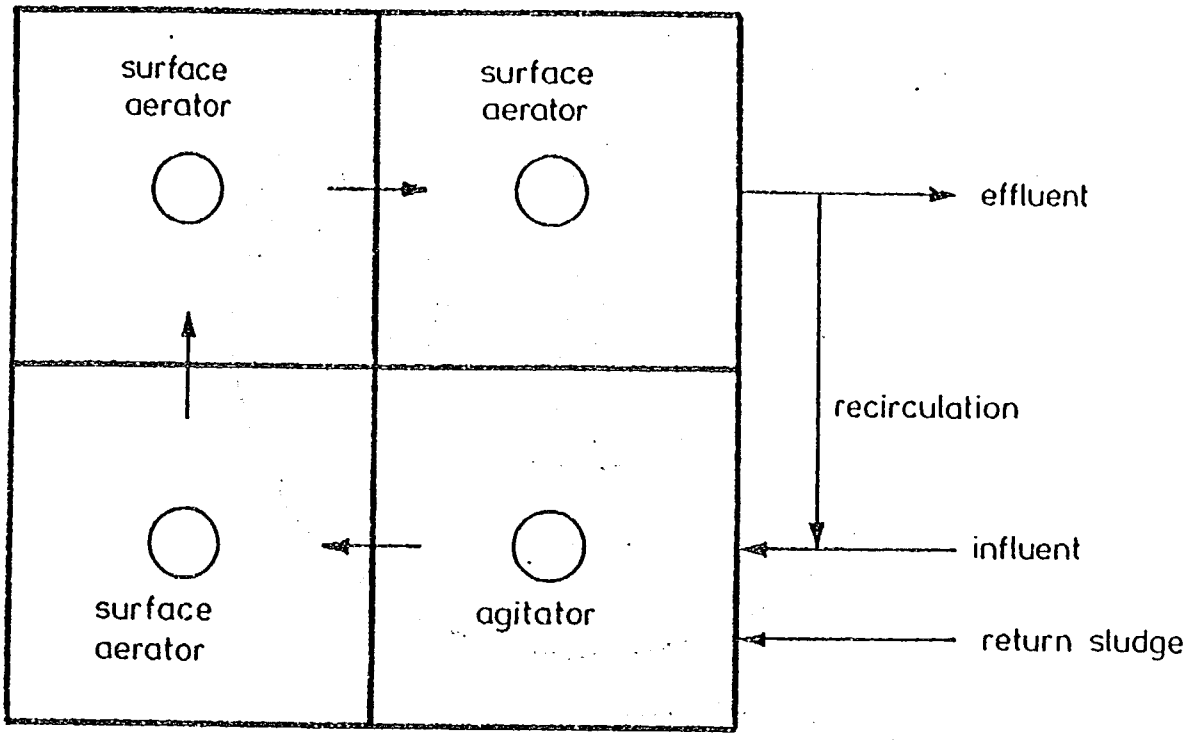


fig. 13 primary denitrification system consisting of four cascades with agitator and surface aerators

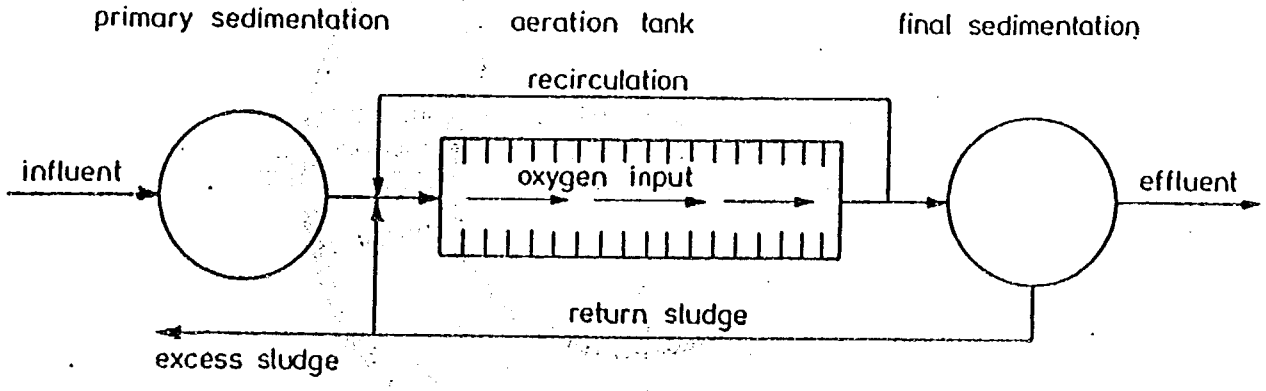
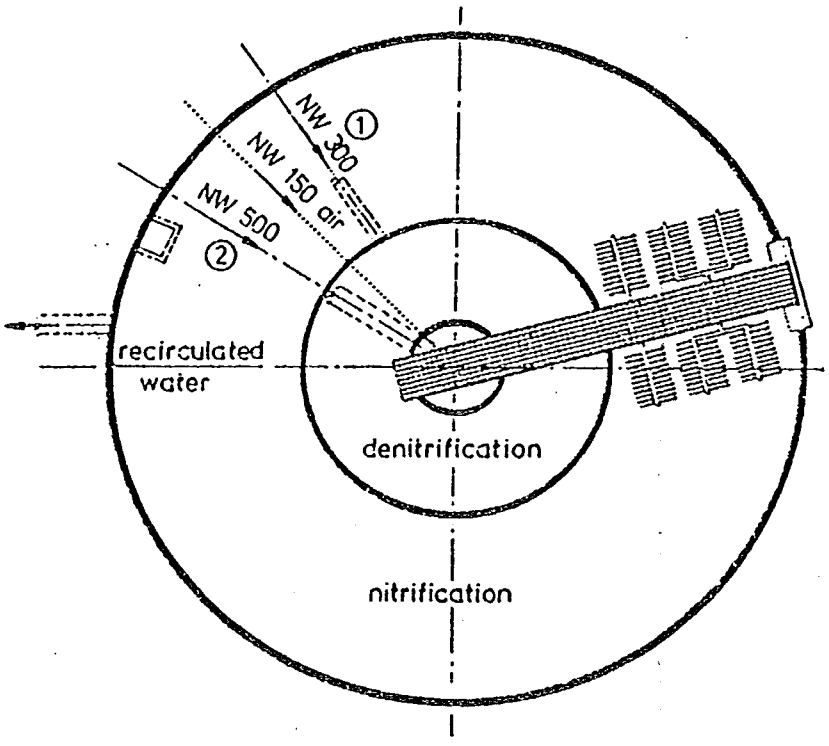


fig. 14 primary denitrification in a plug-flow basin



① influent to nitrification basin
② influent to denitrification basin including sludge return and recirculated water

fig. 15 schematic diagram of the denitrification plant Biet-Tiefenbronn (Schreiber-System)

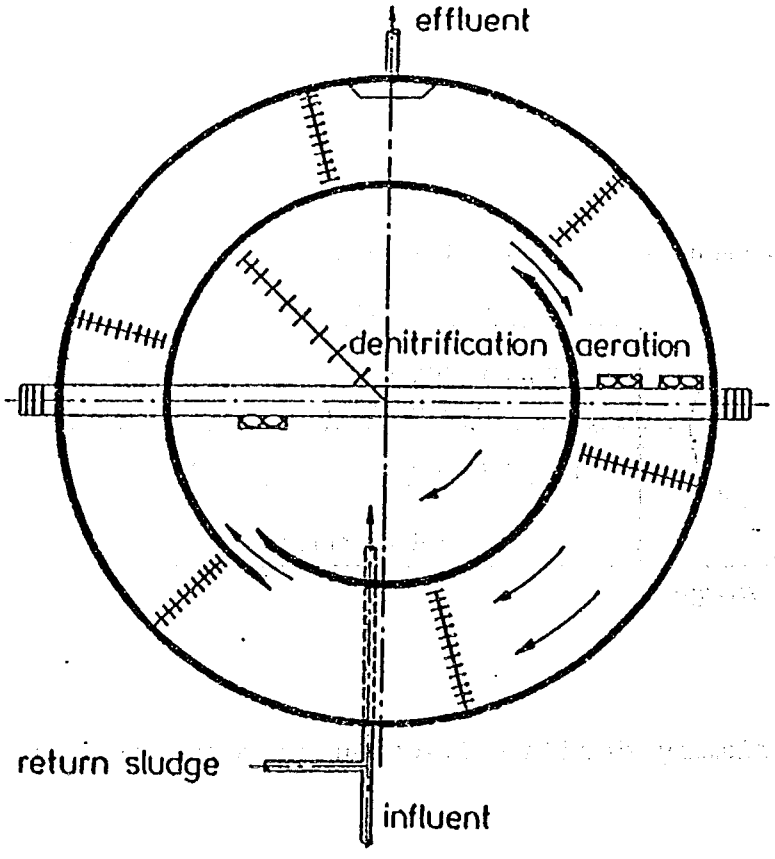


fig. 16 the Menzel-circulation system

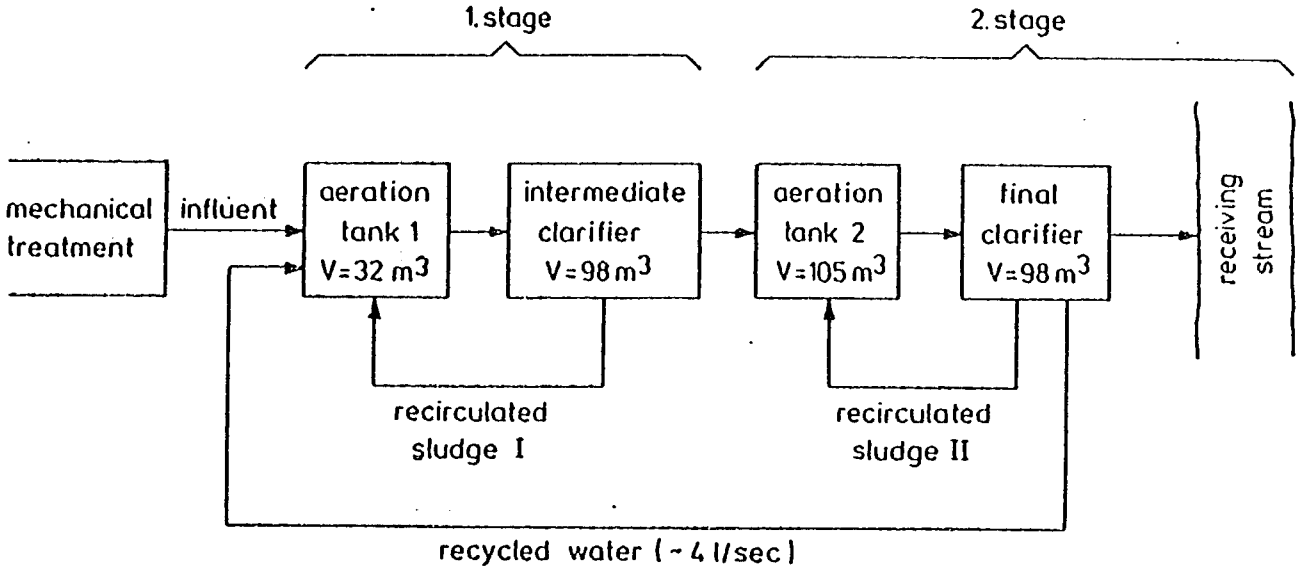


fig. 17 schematic diagram of the two-stage activated sludge system in Büsnau

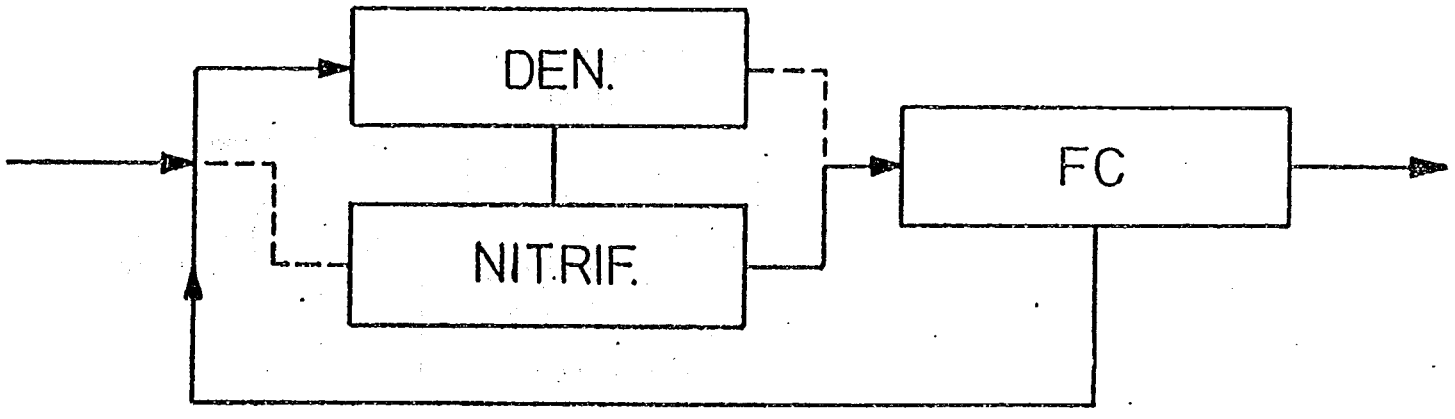


fig. 18 alternate nitrification/denitrification system

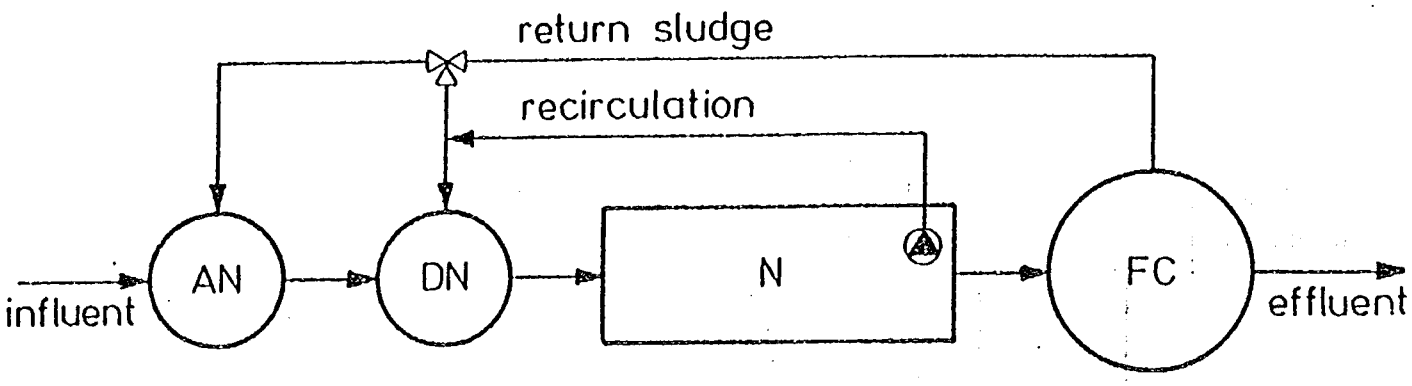


fig. 19 activated sludge plant with an anaerobic and a denitrification stage ahead of the aeration tank

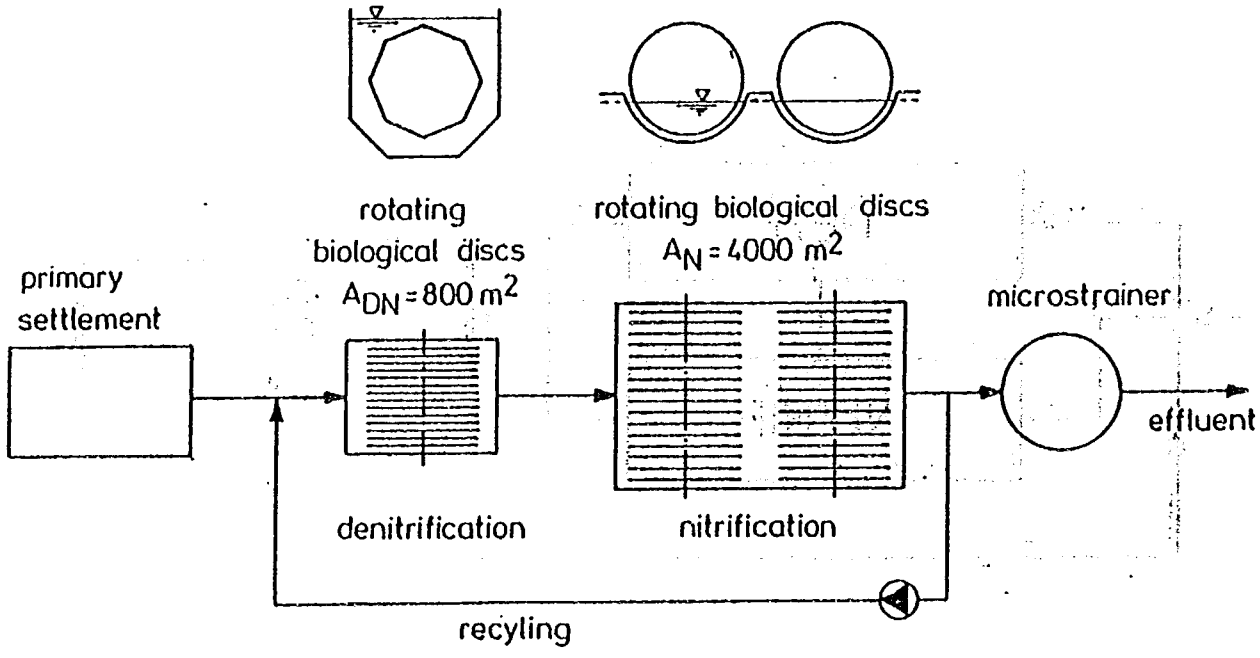


fig. 2o primary denitrification with a fully submerged rotating biological disc unit

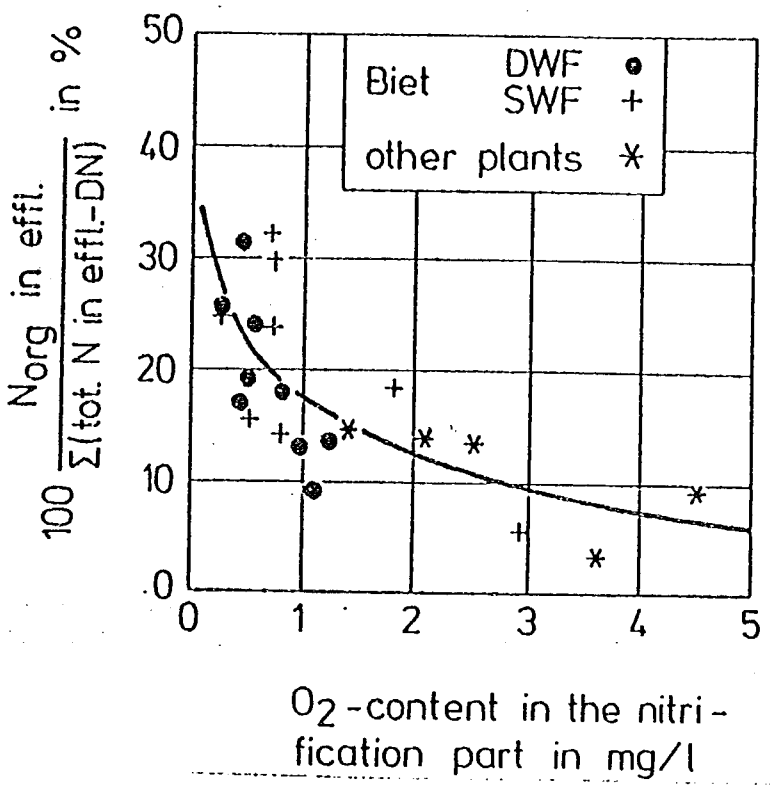


fig. 21 organic nitrogen in the effluent of the plant at Biet-Tiefenbronn versus dissolved oxygen

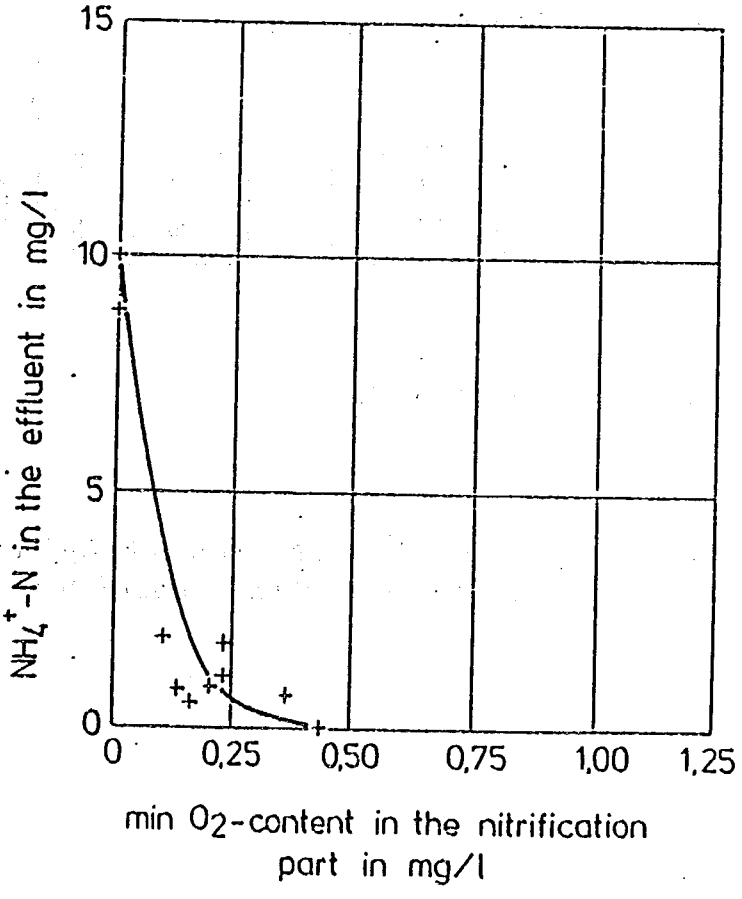


fig. 22 NH_4^+-N in the effluent versus dissolved oxygen in the nitrification part

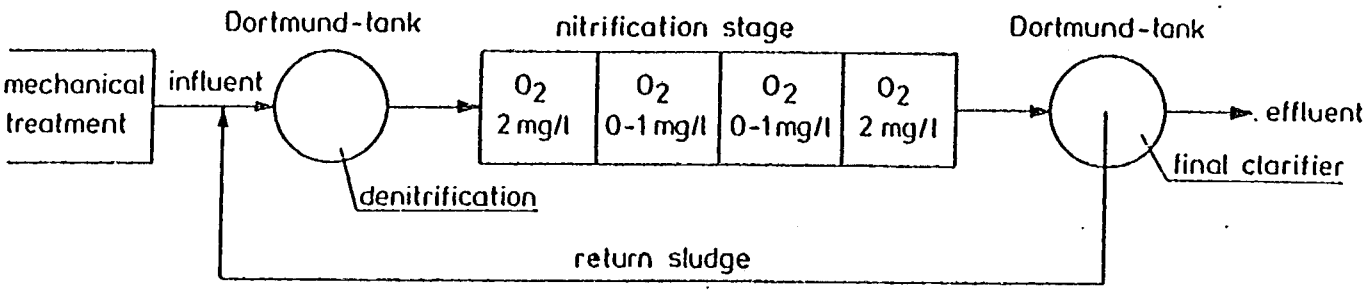


fig. 23 schematic diagram of the research sewage treatment plant at Stuttgart-Büsnau with 5 cascades

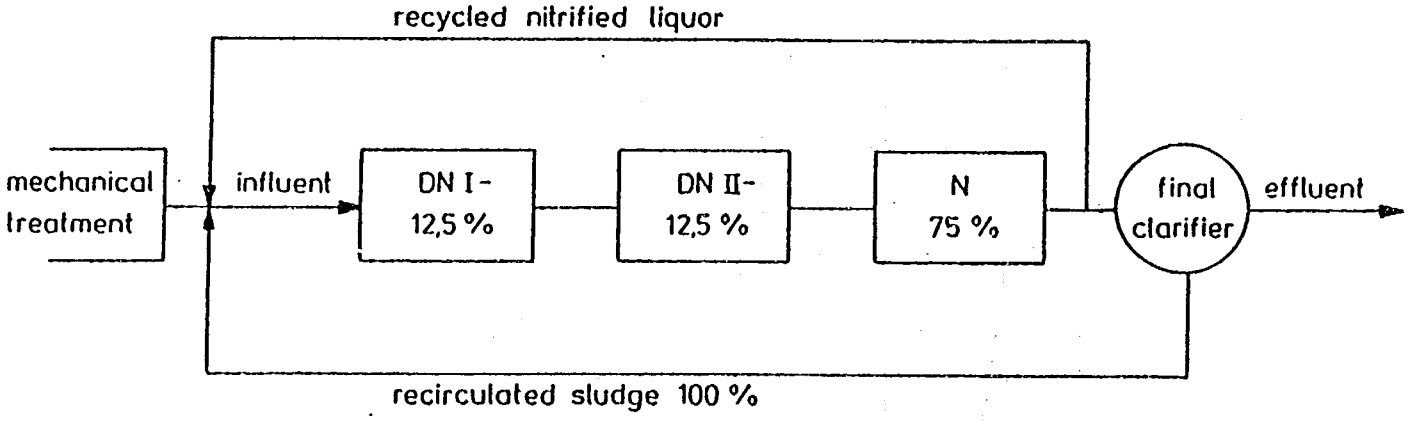


fig. 24 schematic diagram of the research sewage treatment plant at Stuttgart-Büsnau with 3 cascades

DEVELOPMENT OF CRYSTALLIZATION PROCESS FOR PHOSPHORUS REMOVAL

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1. INTRODUCTION

In Japan, the biological phosphorus removal process (anaerobic-oxic process) and the crystallization process are being earnestly researched as technology for the removal of phosphorus in wastewater as a substitute for the conventional precipitation process. Among these new processes for removal of phosphorus in wastewater, we here report on the crystallization process.

In the crystallization process, phosphorus in wastewater precipitates and grows on the surface of nuclei as calcium hydroxyapatite ($Ca_5(OH)(PO_4)_3$) of the same quality as the nuclei and is removed from the wastewater through the contact of wastewater with the nuclei. The phenomenon is called crystallization. For improving phosphorus removal by crystallization, the pH and calcium concentration in the influent flowing into the nucleus contactor must be increased. Lime is usually added to the influent for these purposes.

The characteristics of this process are as follows:

(1) It requires a rather small quantity of chemicals such as lime and also produces a rather small amount of sludge, as phosphorus is crystallized on the nuclei.

(2) As a large overflow rate is employed compared with that in the conventional precipitation process, the crystallization process requires less area and operational cost.

(3) The nuclei enlarged with crystallized phosphorus can be reused as a fertilizer or a phosphorus raw material.

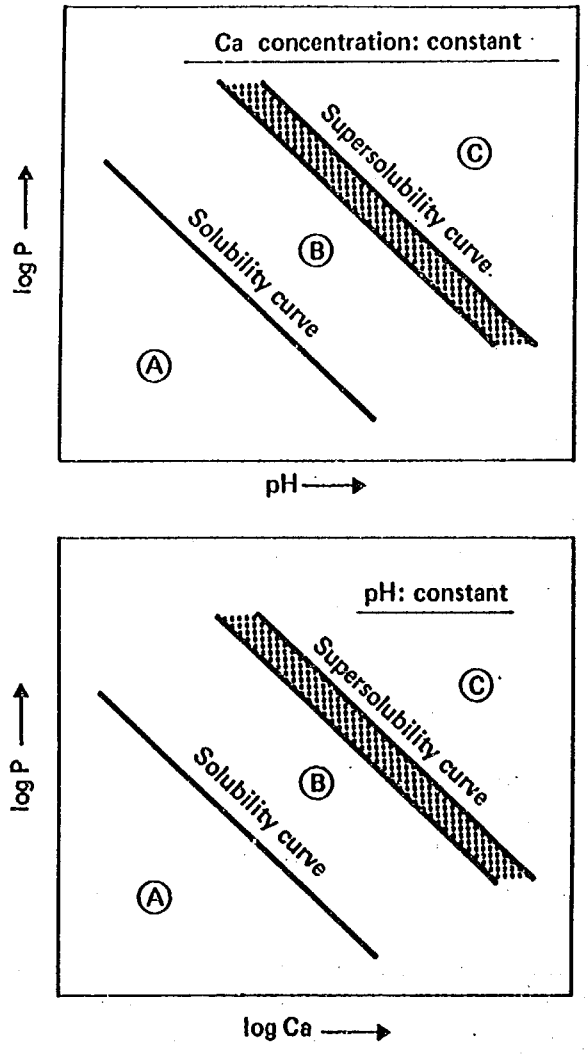
Zoltek[1] carried out fundamental experiments on the removal of phosphorus by crystallization. Since then, the process has been researched earnestly for practical use in Japan.

The crystallization process has the above-described characteristics, but there are also various problems to be solved in using it practically. Advanced Waste Treatment Section, PWRI, has been carrying out some experiments in the crystallization pilot plant installed in Toba Sewage Treatment Plant, Kyoto and researching its practical application.

2. PRINCIPLES OF CRYSTALLIZATION PROCESS

When the solute concentration in a solution is increased from a level lower than solubility, it becomes saturated. Further increasing the concentration results in no crystal production (supersaturation). However, when increasing the concentration continuously, fine crystals are precipitated. Fig. 1 shows the phenomenon with a solute of apatite which is used in the crystallization process. The zones in the figure are a stable zone, a semi-stable zone, and an unstable zone; the boundaries between the zones are a solubility curve and a supersolubility curve.

In the lime precipitation process which is one of the already developed physical-chemical phosphorus removal processes, phosphorus is precipitated in the unstable zone. As clearly seen in Fig. 1, the pH and the Ca concentration must be increased for phosphorus removal in the unstable zone, and consequently the lime dosage must be increased to about 200~400mg·$Ca(OH)_2$/liter.



Ⓐ Stable zone: Phosphorus is not precipitated.
Ⓑ Semi-stable zone: Crystallizing zone
Ⓒ Unstable zone: Precipitating zone

Fig. 1 Crystallization Process

In the crystallization process, phosphorus removal is performed in the semi-stable zone. In the semi-stable zone, no new crystals are precipitated but the existing crystals grow. In the process Ca and alkali are added to the influent, which is introduced into the semi-stable zone and allowed to contact with nuclei to precipitate phosphorus in influent as apatite on the surface of nuclei. The lime dosage is 30~90mg·$Ca(OH)_2$/liter when lime is used as dosing chemical. Generally, when supersaturation in the crystallization system is rather low, nuclei with the same or similar structure to that of the precipitating crystals are effective, while when the supersaturation is rather high, nuclei having structures of a rather wide range may be

used[2]. The crystallization process for phosphorus removal is the former case and phosphorus ore or bone black is proposed as nuclei. Fig. 2 shows an example of the effect of nuclei added[3], and indicates that the addition of nuclei promotes the removal of phosphorus.

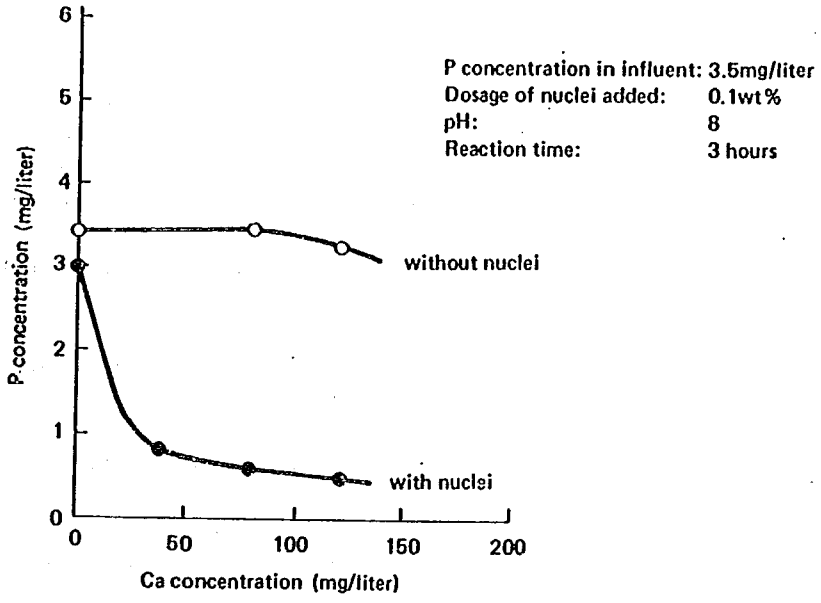


Fig. 2 Effects of Nuclei Added in the Crystallization Process

2-1. Solubility Curve

The general formula of the solubility and the supersolubility curves of calcium hydroxyapatite are as follows:

$$(Ca)^m \cdot (OH) \cdot (P)^n = 10^k \quad \cdots\cdots (1)$$

The solubility curve is obtained from the solubility product, representing the concentration by mole fraction, as seen in the following formula,

$$(Ca)^5 \cdot (OH) \cdot (PO_4)^3 = 10^{-55.6} \quad \cdots\cdots (2)$$

Soluble phosphorus becomes 0.05 mg·P/liter by putting pH=7 and Ca=20 mg/liter into formula (2) considering usual secondary effluent and the ratio of PO_4 to soluble phosphorus as pH=7. Since the soluble phosphorus in secondary effluent is usually 1~3mg·P/liter, the phosphorus in the secondary effluent is in the semi-stable zone. However, the phosphorus concentration in secondary effluent is not reduced to 0.05 mg·P/liter by contact between the nuclei and secondary effluent or lime dosed secondary effluent within the semi-stable zone. Therefore, an optimal operation condition for reducing phosphorus in effluent in the crystallization process is said to be a condition under which the concentration of phosphorus in the effluent is allowed to approach to the solubility curve as much as possible.

Table 1 shows the operational factors and their influences in the crystallization process which have been already reported.

Table 1 Operational Factors and Their Influences in the Crystallization Process

Factors	Influences
1. pH and Ca	a) It is recommended that the pH and Ca in influent approach the supersaturation curve, as long as they do not exceed the curve.
2. Carbonate concentration in influent (M-alkalinity)	a) The following demerits are observed: $CaCO_3$ produced by the reaction with lime and carbonate in influent prevents precipitation of apatite; $CaCO_3$ attaching onto the surface of nuclei reduces the activity of nuclei; $CaCO_3$ reduced the reaction rate of apatite.[1, 2] b) The surface of $CaCO_3$ is good for the formation of crystals of apatite.[3]
3. Removability of polyphosphate	a) The removability of polyphosphate is almost equivalent to that of orthophosphate.[4]
4. Flow rate	a) The flow rate is not influenced particularly.[5]
5. Contact time	a) It is mostly from 20 to 30 minutes. b) Variation of the flow rate up to about 2-fold may be acceptable.[6]
6. Nuclei	a) Larger specific surface area is better: therefore, a smaller particle size is better for removing phosphorus.
7. Water temperature	a) Theoretically, the reaction rate reduces following a decrease of water temperature according to Arrhenius's equation, but the tendency is observed only in some cases[1, 5] and not in other case.[6]
8. Ions	a) Positive influence: F^-[1,5,6,7], but not in low concentration.[5] b) Negative influence: Mg^{2+}[8], $Mg(SO_4)$[1], NH_4Cl[1], Carbonate ions [1,2,9] c) No influence: Cl^- [6], Cl_2 [7] d) SO_4^{2-}: There are two results on the ion to show no influence[6] and negative influence.[8]

2-2. Supersaturation Curve

In this process, usually phosphorus is removed in a nucleus contactor filled with nuclei. When the influent into the contactor is over the supersaturation curve, the fine crystals precipitating may inhibit crystallization, and also they may not fix on the surface of nuclei but be discharged during washing of the nucleus contactor. This phenomenon is disadvantageous to the crystallization of phosphorus. For increasing the crystallizing effect, on the contrary, the supersaturation degree of the influent must be elevated. Therefore, it is important to operate the system in the semi-stable zone near the supersaturation curve as much as possible.

The supersaturation curve cannot be directly decided as it varies depending on the reaction time, M-alkalinity of the influent and other factors. For example, the supersaturation curve obtained giving M-alkalinity of 100mg/liter and a reaction time of 1 hour[4] is represented by formula (3) with a similar form for formula (1) and the concentration represented by mole fraction:

$$(Ca)^{1.54}(OH)\,(P)^{1.54} = 10^{12.1} \quad \cdots\cdots (3)$$

The supersaturated concentration of phosphorus is obtained from formula (3). For example, when Ca=40mg·P/liter and pH=8, 8.5, 9 and 9.5, the concentration is 12.0, 5.7, 2.7, and 2.5mg·P/liter, respectively and when Ca=60mg/liter, it is 8.0, 3.8, 1.8, and 0.8mg·P/liter, respectively.

3. FLOWSHEETS OF CRYSTALLIZATION PROCESS

The flowsheets of the crystallization process in pilot plants and existing plant are shown in Fig. 3 I to IV. The figures of I and II show simple flows in which lime is added to the influent by a pump connected to the pH meter by which the lime dosage is controlled so as to adjust pH in influent at a predetermined value. pH and Ca levels are both elevated simultaneously in this method. In the contactor crystalizing phosphorus is removed as well as solid suspended in the secondary effluent and produced by lime addition. The contactors in flowsheets I and II employ fixed beds of upflow and downflow systems, respectively. As the larger the surface of nuclei is, the more crystallization is promoted, rather fine nuclei are packed in the filter. Consequently, for removing suspended solids in the contactor, as the downflow system yields abruptly a head loss increase due to clogging because of surface filtration, the upflow system will be advantageous. Nuclei packed in the contactors in the flowsheets I and II are produced from phosphorus ore and bone black, respectively. The compositions of phosphorus ore and bone black resemble each other, but the specific gravity of the former is 2.6~3.0 which is heavier than that of the latter, at about 2.5. The specific surface area of the former is about $20m^3/g$, while that of the latter is $110m^3/g$ which is much larger. Bone black has fine pores like activated carbon. If the surface area contributes to crystallization, the latter is advantageous. But there is an assertion that most pores of bone black are smaller than crystals of apatite; the effect of bone black must be further studied.

The flowsheet III is characterized by adjusting pH with caustic soda and Ca concentration with calcium chloride separately and using two contactors in series. As the influent is treated by caustic soda and calcium chloride separately, pH and Ca concentration can be independently adjusted to selected values. In addition, the influent can be prevented from mingling with impurities such as calcium carbonate. However, the system is expensive due to use of expensive chemicals. The first contactor serves to crystallize and remove suspended solids, and almost ideal crystallization is carried out as the influent is quite free from suspended solids in the second contactor, as described later.

It is known that the higher the alkalinity of the influent, the lower the phosphorus removal in the crystallization process, as mentioned above. In flowsheet IV,

the alkalinity of the influent is lowered in the carbonate removing step. Carbonate is removed as follows. Sulfuric acid is added to adjust the pH of the influent at 3 and convert carbonate of the main component of the alkalinity to carbon dioxide. Then air is blown in at an air-water ratio of 3 to strip carbon dioxide. In flowsheet IV, a filter is settled prior to the nucleus contactor for preventing the surface of nuclei from being covered with calcium carbonate in the added lime or produced by lime addition.

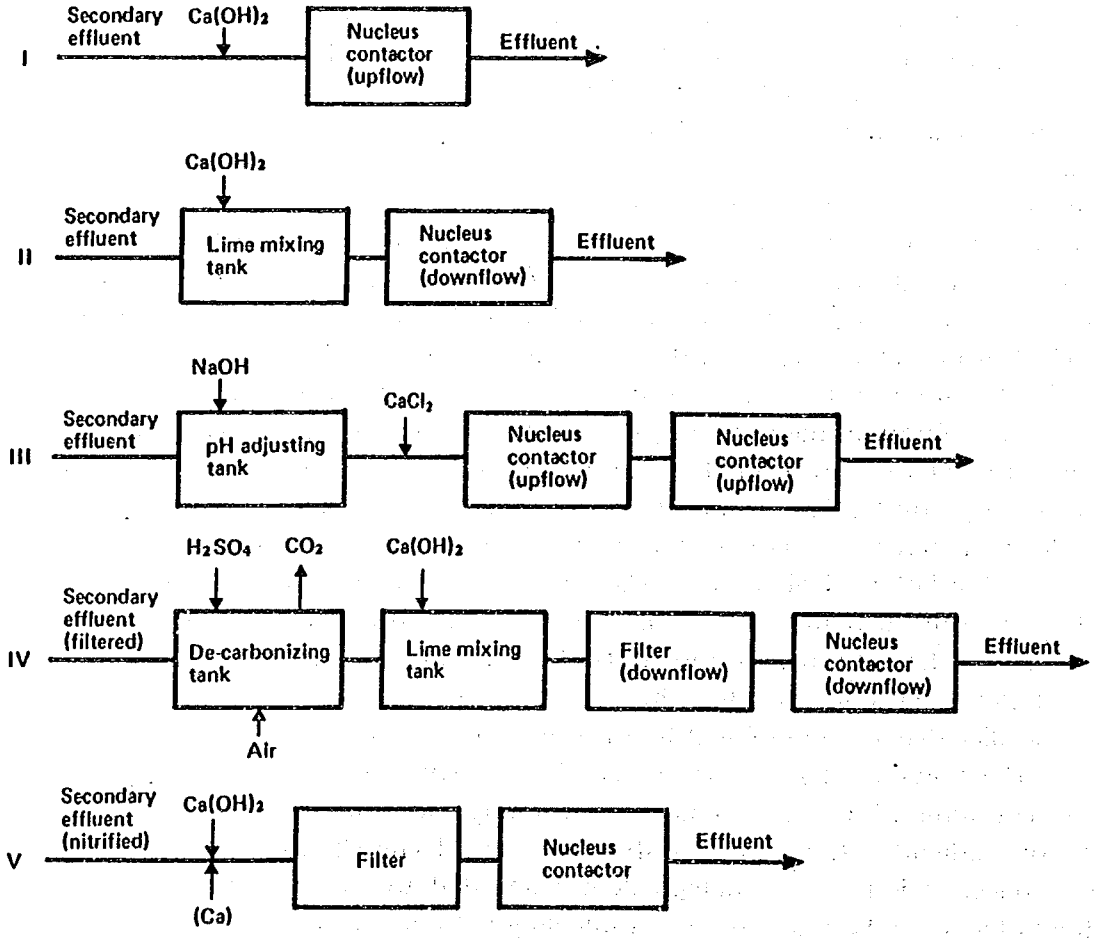
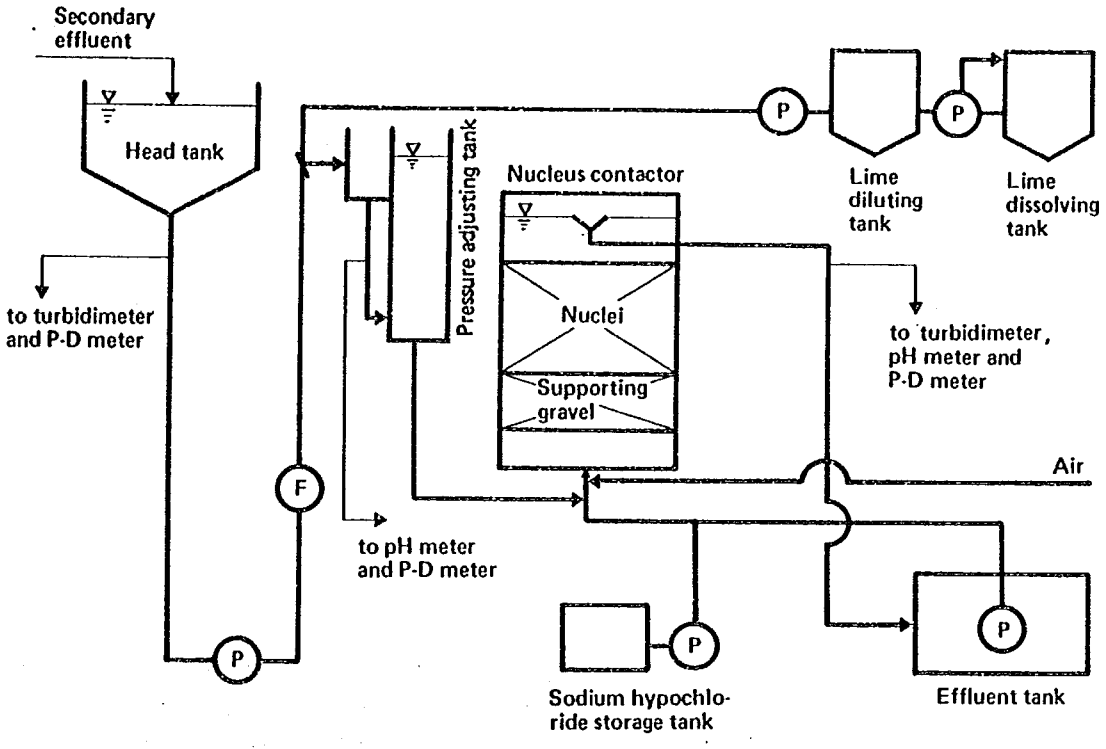


Fig. 3 Flowsheets of Crystallization Process



Influent:		secondary effluent
Contact time:		30 minutes
SV:		2
Flow volume:		89.3m^3/day
Flow rate:		74.4m/day
Set pH:		9 (pH control)
Washing interval:		1/2 days (operating for 46 hours)
Washing water:		effluent from nucleus contactor
Nuclei	Raw material:	phosphorus ore
	Effective size:	0.69mm
	Uniformity coefficient:	1.34
	Specific gravity:	3.00g/cm^3
	Bulk density:	1.15g/cm^3
	Specific surface area:	21.5m^2/g
	Height of the filled nuclei:	1550mm

Fig. 4 Flowsheet of the Crystallization Pilot Plant Installed in Toba Sewage Treatment Plant, Kyoto by PWRI and the Experimental Conditions

4. TREATMENT RESULTS OF CRYSTALLIZATION PROCESS

The average results obtained in the flowsheets described above are shown in Table 2.

Table 2 Results of the Crystallization Process

Flow-sheet	Removal of P			Alkalinity before adding lime (mg/liter)	Operating conditions of nucleus contactor					Remarks	Reference
	Influent (mg·P/liter)	Effluent (mg·P/liter)	Removal rate (%)		Nuclei	Set pH	SV	Flow rate (m^3/day)	Dosage of chemical		
I	0.99	0.37	63	109	Phosphorus ore	9.0	2	90	$Ca(OH)_2$ = 83 mg/liter	The lime dosage would be increased because of impurity of lime used.	9
II	1.31	0.26	80	40	Bone black	9.0	2	50	$Ca(OH)_2$ = 26 mg/liter	The alkalinity of the influent was characteristicly low.	6
III	2.26	0.20	91	60～300	Phosphorus ore	8.7～9.0	1	480	Adjusting pH with NaOH; adjusting Ca to 30mg/liter with $CaCl_2$	In an existing plant, SV was 1 in both contactors and SV of each filter was 2.	10
IV	1.78	0.23	84	Approx. 14	Phosphorus ore	8.75～9.25	2.5	96	$Ca(OH)_2$ = 90 mg/liter H_2SO_4 = 140 mg/liter	As pH was lowered and carbonate was removed, the alkalinity was low and the lime dosage was large.	2

From Table 2, the concentration of phosphorus in the effluent of the crystallization process may be about 0.5mg·P/liter, but considering some problems described below, the level of phosphorus in the effluent must be further discussed.

5. PROBLEMS IN CRYSTALLIZATION PROCESS

It has not been so long since the crystallization process was developed, and it still has many problems to be solved. Here are described problems which were found in the crystallization pilot plant installed in Toba Sewage Treatment Plant, Kyoto by PWRI[9]. The flowsheet of the plant is in Fig. 3-I, and the detail is shown in Fig. 4.

5.1 Coagulation of Phosphorus in Lime Dosed Influent

For keeping the condition for crystallization of phosphorus, the influent to the nucleus contactor must be kept under the supersaturation curve. However, even when the pH is kept constant after adding lime, there still remains a possibility that the lime dosed influent will be in the unstable zone beyond the supersaturation curve following increase of the phosphorus level in the influent. In the unstable zone, part of phosphorus in the influent is coagulated prior to crystallization to reduce soluble phosphorus in the influent to the contactor.

For revealing the degree of coagulation, pH of the lime dosed influent was set at 8.5, 9.0 and 9.5 and the soluble phosphorus was continuously determined in the influent (secondary effluent) and the lime dosed influent by soluble phosphorus monitors. The results are shown in Fig. 5, and summarized in Table 3. Under the conditions in this experiment, when the pH was set at 9.0 and the soluble phosphorus in the influent was about 1.5mg·P/liter or more, the lime dosed influent entered the unstable zone and phosphorus began to coagulate.

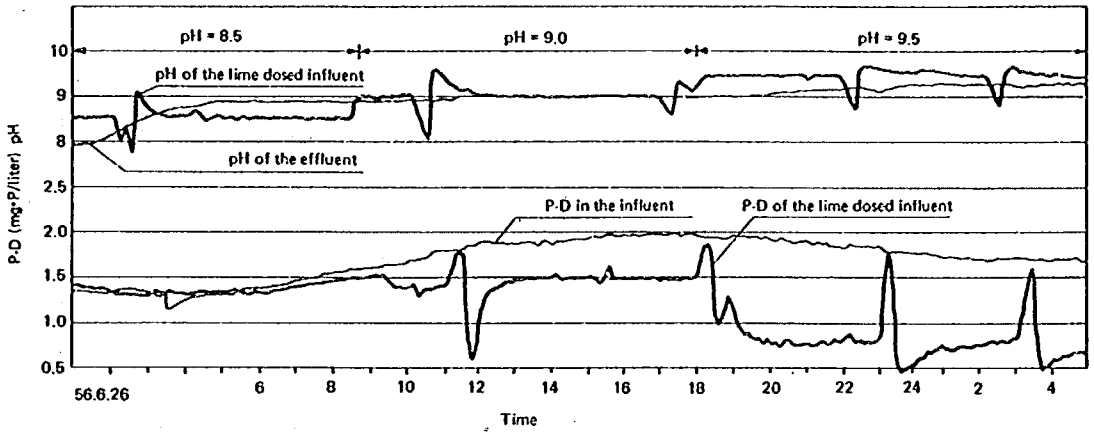


Fig. 5 Decrease of Soluble Phosphorus due to Its Coagulation after Adding Chemicals

Table 3 Decrease of Soluble Phosphorus due to Coagulation

pH	Soluble phosphorus (mg·P/liter)	
	Influent	Lime dosed influent
8.5	1.45	1.38
9.0	1.88	1.42
9.5	1.80	0.78

The influence of the coagulation of phosphorus in lime dosed influent is summarized as follows:

(1) Head loss increase: As flocs produced by the coagulation are caught in the nucleus layer in the contactor, the run length of nucleus contactor is reduced.

(2) Increase of suspended solids and phosphorus in the washing waste produced by washing the contactor: Flocs of phosphorus coagulated and precipitated are non-crystalline; they are not converted to apatite in the contactor but mostly discharged out of the system as washing waste[7]. Therefore, one of the characteristics of this process that little sludge is produced is eliminated.

(3) Reduction of the phosphorus removing capacity of nuclei: As flocs produced by coagulation cover the surface of nuclei, the capacity of nuclei to remove phosphorus is reduced. In the midst of the pilot plant experiment, the soluble phosphorus in the influent (secondary effluent) varied greatly with time, and frequently exceeded the level of 5mg·P/liter which is the full range of the soluble phosphorus monitor. Fig. 6 shows the results of column experiments in a laboratory using nuclei taken from the contactor before and after great variation of the phosphorus level in the influent. As seen in the figure, the phosphorus removing activity of nuclei when the phosphorus level varied greatly became worse than that before the variation, and the difference of the soluble phosphorus was about 0.3mg·P/liter between both effluents.

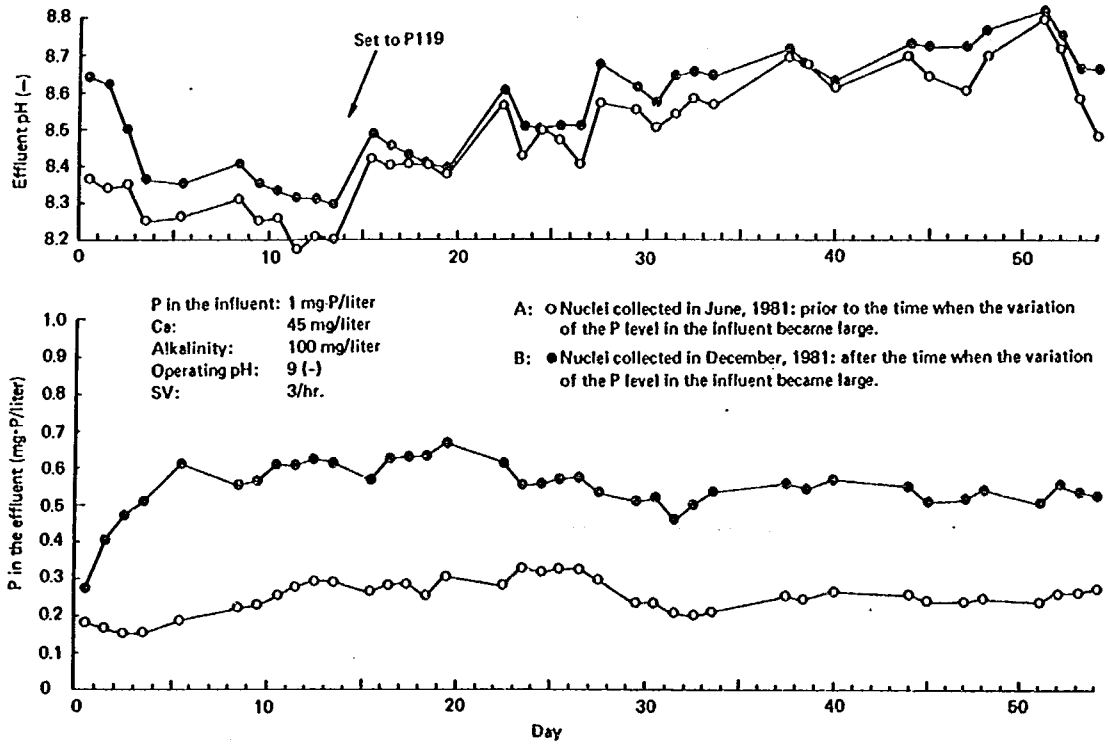


Fig. 6 Results of Column Experiments Filled with Nuclei Collected from the Pilot Plant

5.2 Re-dissolution of Phosphorus in Washing Waste

It seems general that the washing waste is returned to a primary settling tank. When phosphorus in the influent to the contactor is coagulated by adding lime as described in 5.1 and the washing waste is mixed with almost neutral influent to the primary settling tank, it is possible that the insoluble phosphorus in the washing waste may re-dissolve. Washing waste containing coagulated phosphorus was mixed with filtered primary effluent with ratios of 5:100, 10:100, and 30:100, and the solubility of phosphorus in the mixture was studied. The mixture was kept in a brown bottle, stirred continuously for 5 days with a stirrer, and then stirred for 1 hour before collecting a sample. The results are shown in Table 4.

As seen in the table, the soluble Ca level increased with time, but the soluble phosphorus level did not increase so greatly. It indicates that the washing waste containing coagulated phosphorus can be returned to the primary settling tank almost without the fear of re-dissolution of phosphorus.

5.3 Flowing-out of Nuclei

As described previously, one of the characteristics of this process is that since phosphorus in the wastewater is removed by the crystallization on the surface of nuclei, the enlarged nuclei may be reused. In this pilot plant study, however, a reverse phenomenon, that is, reduction of the height of the nuclei layer was ob-

Table 4 Solubility of Phosphorus in the Washing Waste

Unit: mg/liter, except pH

Sample No.	Time elapsed	RUN 1 (5/100) *					RUN 2 (10/100)					RUN 3 (30/100)				
		Water temperature	pH	P·D, ortho	P·D	Ca·D	Water temperature	pH	P·D, ortho	P·D	Ca·D	Water temperature	pH	P·D, ortho	P·D	Ca·D
1	0	18.1	7.8	1.37	1.51	10.7	17.6	7.9	1.41	1.56	11.6	18.9	8.1	1.52	1.58	18.5
2	1 hr	20.1	7.9	〃	1.49	12.6	19.7	8.0	1.42	1.58	15.1	20.8	8.2	1.55	1.66	15.8
3	3 hrs	22.5	8.0	1.36	〃	12.1	22.2	8.0	〃	1.56	13.7	23.9	8.2	1.54	1.47	18.5
4	5 〃	23.9	8.0	1.35	〃	11.7	23.9	8.1	〃	1.58	14.2	25.9	8.2	〃	1.58	12.6
5	19 〃	25.0	7.6	1.27	1.38	14.0	25.2	7.7	1.33	1.51	17.0	27.0	7.8	1.42	1.56	18.6
6	26 〃	25.4	7.5	1.24	1.34	14.5	25.8	7.6	1.34	〃	16.8	27.2	7.8	1.40	1.49	18.7
7	45 〃	25.2	7.4	1.25	1.28	14.3	25.7	7.4	1.32	1.49	17.6	28.0	7.5	1.47	1.55	25.5
8	5 days	25.0	7.0	1.30	1.35	14.1	25.2	7.0	1.44	1.48	18.9	27.4	6.6	1.71	1.68	37.0
9	11 〃	23.3	6.4	1.38	—	—	23.6	6.5	1.52	1.57	21.5	24.6	6.6	1.78	〃	36.4
10	19 〃	23.2	6.2	1.42	1.42	17.9	23.3	6.4	1.54	1.54	26.4	23.2	7.0	1.85	1.85	36.1
11	30 〃	21.5	6.2	〃	1.44	19.1	22.5	6.7	1.56	1.56	22.7	21.6	7.2	〃	〃	40.5

* Mixing ratio of washing waste/primary effluent (filtered)

** The No. 5C or GF/B pass was taken as a dissolved component.

	P, ortho	P	Ca
Primary effluent (filtered)	1.28	1.51	16.2
Washing waste	—	6.96	109.7
Washing waste (filtered)	1.48	1.58	68.1

served. About 4 and 9 months after beginning the experiment, the height was reduced by 10cm and 12cm, respectively. The reduction corresponded to 6.5% and 7.7% against the height of the primary nucleus layer, respectively. The reduction of the nuclei per unit of the effluent was about 5.6g/m^3 which cost about ¥1.7/m^3 providing that the price of nuclei was ¥0.3/g.

The increase of the bulk density of nuclei in the contactor may be considered as a cause of the reduction in the height of the nucleus layer, but it is more likely caused by scrubbing off of the surface of nuclei by mutual abrasion of nuclei on washing. At the end of washing, fine nuclei were always found on the surface of the funnel tube for effluent in the upper part of the contactor. Since the contactor was expected to remove not only phosphorus but suspended solids and thus a grid[11] was set 10cm under the upper surface of the nuclei layer and also the area of the cross section of the contactor was as small as 1.2m^2 in this experiment, the pressure on washing to the nuclei would be larger than that in the practical contactor which would promote the decrease of nuclei.

Considering the above, it is necessary to develop nuclei with high anti-abrasion, to study washing methods, and to develop a contactor to prevent flowing-out of fine nuclei.

5.4 Decrease of Purity of Lime in Lime Solution

In the experiment, pH of the influent to the contactor is adjusted to 9 with an emulsion prepared by dissolving lime in water. The lime emulsion is stirred to keep the concentration of lime constant. It was observed in laboratory experiments that about 0.03% carbon dioxide in the air solves into the lime emulsion to form calcium carbonate resulting in decrease of the lime component in the lime solution. To prevent this, it was proposed to dry feed lime or carbon deoxide free air to purge the upper air in the lime tanks. In the pilot plant the latter was employed. Air which had passed through a potassium hydroxide solution to remove carbonate was introduced into the upper parts of the lime dissolving tank and the lime diluting tank. Then, the lime dosage necessary to adjust the pH of the secondary effluent at 9 was reduced by about 20%.

As partial conversion of the lime to calcium carbonate not only increases the lime dosage and the sludge production but also may inhibit the crystallization of phosphorus, as seen in Table 1. The conversion to calcium carbonate must be prevented, and some countermeasures are required in performing this process.

6. DISCUSSION OF FLOWSHEET OF CRYSTALLIZATION PROCESS

Considering various problems such as those described in 5, the 1982 experiment is being performed with flowsheet V in Fig. 3 in Toba Advanced Waste Treatment Pilot Plant.

6.1 Change of Influent

The lower the alkalinity of the influent, the lower the phosphorus concentration in the effluent is expected to be in the crystallization process. In flowsheet IV, thus, pH is lowered and carbonate is removed. However, this technique leads to increase of the dosage of chemicals. In addition, dissolution of the nuclei in the contactor is a possibility if the alkali addition to pH lowered influent for carbonate

removal fails and the influent flows into the contactor. On the other hand, the alkalinity of the effluent in the nitrification-denitrification process for removing nitrogen is usually considerably lower than that of the secondary effluent. The nitrification-denitrification effluent is used as an influent in the crystallization process considering a combination of nitrogen removal and phosphorus removal from the wastewater.

6.2 Filter

Coagulation of phosphorus accompanying lime addition as previously described cannot be avoided as long as the phosphorus level in the influent varies with time, and it may adversedly affect crystallization. A filter for removing flocs of impurities such as calcium phosphate, calcium carbonate and organic suspended solids is installed prior to the nucleus contactor for improvement of the crystallizing conditions.

6.3 Nucleus Contactor

As a filter is installed prior to the nuclei filter, the removal of suspended solids can be neglected in the nucleus contactor. Consequently the interval of washing the contactor may be prolonged and it may be washed enough even with a weaker method than the conventional one. It will greatly reduce the amount of nuclei flowing out. In addition, it may be possible to use smaller nuclei to increase the specific surface area.

A technique to regenerate nuclei of which the phosphorus removing capacity is lowered is to be researched.

The crystallization process is one of the few domestic techniques for wastewater treatment in Japan. However, this process has not been used practically for very long and has many problems to be solved. The Advanced Waste Treatment Section is further researching the practical application of this process.

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Investigation of algal born organic substances and their effect on water treatment

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1. Introduction

The production of drinking water from eutrophic surface waters containing high densities of planktonic organisms is often hindered, due to disturbances encountered particularly in autumn. Water works which use an iron or aluminium salt flocculation for the main treatment of such water have experienced the disturbances as:

- an increase in flocculant residues in the filtrate
- an inhibition of turbidity elimination
- an increase in chlorine consuming processes which impeds the disinfection of the water
- a regermination of the water in the distribution system.

Relevant studies carried out by the Wahnbach Reservoir Association several years ago showed clearly that a relation exists between the disturbances of the flocculation during the water treatment and the extensive algal developments which occurred yearly until 1974 in the reservoir (1, 2).

2. Algal born organic substances as the cause of flocculation disturbances.

Organic substances of biogenic origin are considered to be the cause of the flocculation disturbances. They are released from the plankton cells into the water by:

- active or passive release
- autolysis
- breakdown of the cells by microbial exoenzymes.

In this connection, the planktonic algae can be regarded as the most important primary producers in salt and fresh water biotopes (3, 4).

Fogg (5) has given a survey on the nature of the extracellular products from algae and of their ecological effects. Hellebust (6) presented a review which included tabulated data on extracellular carbohydrates, acids and amines, and Harris (7) has summarized work on carbohydrate and glycolate metabolism.

The organic substances released consist of low molecular weight excrements and of high molecular weight compounds, which are released into the water for example during the "unlimited swelling" of mucilagenous outer cell wall layers of many completely different types of algae. It can also be assumed that mainly polysaccharides go into solution during the hydrolytic dissolution of the cell wall (e.g. following spore release by the coccal green algae). As well as the algal metabolic products, substances arising from the breakdown of cells may also be of importance. These substances include not only those formed during the breakdown of dead cells, but also the excretory products of zooplankton which feed on phytoplankton (8 - 11).

The destruction of dead algal cells leads to the release of cell contents, particularly from algae which do not have a cellulose cell wall (12, 13). These substances are utilized by bacteria present in the water, resulting in part to a complete mineralization of the substances and in part to the production of high molecular weight organic compounds e. g. humic substances, which are difficult to degradate biologically.

The planctonic algae are the basic food source of zooplankton. The ingested algal substance is metabolized by the zooplankton and the remainder is excreted in the form of faeces. The faeces also contain some high molecular weight soluble substances which cannot be degenerated further.

In this paper, we want to report on organic substances of algal origin which influence the flocculation process. The amount of each class of substance in the total amount of excreted substance is given in percent in Table 1 (10). It can be seen that glycolate comprises maximally 34 % of the total excreted substances. The most important group of excreted compounds is that containing the sugars. The amino acids have a proportionally lesser importance.

Table 1: Percentage radioactivity (2 hour incubation with 14-CO_2) found in individual classes of substances in the soluble excretion (2 hour exposure of phytoplankton to sunlight (taken from 10))

Date	11.7.75	14.8.75	29.10.75
glycolate	20 %	34 %	12 %
amino acids	15 %	18 %	62 %
sugars	58 %	33 %	
lipids	7 %	15 %	

Mono and oligosaccharides such as glucose, galactose, maltotriose and maltotetrose are the main components of the excretions. The proportion of the single amino acids relative to the total amount of excreted substances is small. However, a wide spectrum of different amino acids could be found to be present (10).

Among the organic compounds which arise in surface waters containing large amounts of algae, the macromolecular carbohydrates of algal origin, some of which are biologically difficult to metabolize, comprise the substantial part. For example, Weinmann (9) measured a total carbohydrate content of 1.1 mg/l in the eutrophic Lake Pluss during summer. Stabel (14) found the total carbohydrate in the moderatively productive Lake Schöh to be up to 0.24 mg/l. Bolze and Soeder (11), using synchronous cultures of Scenedesmus acutus, showed in agreement with Gocke (8) and Weinmann (9), that peptides and carbohydrates were the main constituents of the dissolved organic compounds. The peptides comprised only 20 % of the total DOC in the filtrate, the carbohydrates however, 50 %. These two classes of compounds also comprised more than 95 % of compounds having a molecular weight > 1500; the carbohydrates predominating with about 60 %.

Macromolecular organic compounds accumulate in surface waters with an extensive bioproduction. Some of these compounds are resistant to biological degradation (15).

These compounds include polysaccharides, uronic acids and certain mono- and oligosaccharides as well as organic nitrogen compounds of the amino acid and polypeptide category. However, amino sugars such as glucosamine and galactosamine also occur. Humic acid and fulvic acid, as well as similar compounds lacking an acid group, which are formed as endproducts of humification processes, are also detectable. Whereas some macromolecular substances are present as colloids, a large part of these compounds are present in true solution.

According to studies carried out by Watt (16), glycolate and polysaccharides are the substantial excretions of the freshwater plankton, whereas Ohle (15) considers the mucopolysaccharides and amino compounds as well as mucoproteins to be dominant. The concentration ranges for dissolved substances were given by Ohle to be dependant on the trophic state of a lake as follows:

Oligoproductive lake:	0.5 - 3 g/m^3 C
meso-to euproductive lowland lake (primary production 1 - 4 $g/m^2.d$ C):	5 -10 g/m^3 C
polyproductive lake (primary production 4 - 6 $g/m^2.d$ C):	> 10 g/m^3 C

Before operation of the phosphate elimination plant at the inflow of the Wahnbach Reservoir with the aim of producing an oligotrophic reservoir (17 - 20), 1 - 2 g/m^3 C were measured in the water of the reservoir, which was eutrophic at that time (primary production 1.6 - 2 $g/m^2.d$ C).

In studies on the mesotrophic lake Schöh in Holstein, Stabel (14) reported that in October the largest amount of macromolecular polysaccharides was found in the upper epilimnion and that a maximum of 13.7 % of the dissolved organic carbon was bound in carbohydrates. In contrast, the amount of carbohydrate found in the dissolved organic substances in spring was very small.

Glucose occurred most commonly as a polysaccharide unit in the

water of the Lake Schöh, comprising more than 33 % of the total carbohydrate content, followed by galactose with 11 - 20 % and xylose and rhamnose with 12 - 17 %. Even fructose occurred relatively often with 8 - 16 %. Fucose was found less often with 4 - 6 % and arabinose and merythritol were the least encountered substances comprising 1 - 2 % of the total carbohydrates. The proportion of galactose decreased with increasing water depth, whereas the proportions of glucose, xylose and fructose increased towards the lake bottom. However, these monosaccharides could only be found in traces in the free form.

Seventy percent of the carbohydrates found by Stabel (14) had a molecular weight $\geq$ about 160.000. These compounds were mainly breakdown products resulting from cell metabolism and cell decomposition, particularly from blue-green and green algae. These substances are relatively resistant to microbial degradation and accumulate therefore in the water. They are found in the Hypolimnion of eutrophic lakes after longer periods of extensive bioproduction.

Particularly the cell wall substances, which are released into the water during cell degradation, occur in increasing concentrations when the extent of the bioproduction increases. According to Hinssen (21) such substances of the peripheral cell wall are acid macromolecular polysaccharides with functional groups, e.g. sulphonic acid esters and uronic acid esters as well as glucoproteins. Cellulose also occurs in which the glucose molecules are linked by 1,4 - β-glucosidic bonds. The proportion of these microfibrillar substances in the cell wall of the plankton is subject to large variation and can reach values of 41 %. Regarding these facts, it is therefore not surprising, that acid polysaccharides are found in water and that, as found by Stabel (14), glucose molecules constitute the largest part of the total carbohydrate frame. Lochtmann (22) found in membrane filtered culture suspensions of the green alga Dictyosphaerium, an alga, which occurred in the Wahnbach reservoir, as an average 1 mg/l hexoses, 1.3 mg/l pentoses and 0.5 mg/l uronic acids. Using different analytical methods he found 10 - 22 mg/l total sugar and 3.4 - 6.6 mg/l total uronic acids

in the culture filtrate of the green alga Coelastrum microporum depending upon the growth phase in which the algae were harvested.

Following the collapse of a larger algal biomass in September 1975, Lochtmann found a total sugar content of 0.13 mg/l and a total uronic acid content of 0.27 mg/l in the raw water of the eutrophic Wahnbach reservoir, taken 10 m above the reservoir floor for drinking water production. At that time disturbances in the flocculation process with aluminium salts occurred.

3. Flocculation disturbances due to organic compounds of algal origin

It has been already reported elsewhere on the correlation between increased bioproduction in the Wahnbach reservoir and disturbances to the aluminium salt flocculation process during the production of drinking water (1). Following mass developments of the blue-green alga Oscillatoria rubescens in 1968 - 1970, disturbances in the flocculation process with alum occurred for the first time in 1969. Occurring in the autumn months, these disturbances were characterized by an increased residual aluminium content of 0.04 - 0.05 mg/l in the drinking water instead of the normal 0.01 - 0.02 mg/l. The values could not be normalized with higher aluminium sulphate dosage.

Many years of experience have shown that higher residual aluminium contents in the filtrate (> 0.02 mg/l) indicate that the flocculation process with aluminium salts is not functioning properly. The aluminium sulphate dosage can be regulated excellently according to the amount of residual aluminium in the filtrate or drinking water (23). However, as soon as such organic substances of algal origin are present in the water it is no longer possible, even with a slight increase in the dosage rate of aluminium sulphate, to restrict the aluminium content in the filtrate to the desired value of 0.01 - 0.02 mg/l Al.

With the aid of direct studies, we were able to recognize a correlation between the appearance of these algal born organic substances in the reservoir water following extensive algal growth and the flocculation disturbance in each autumn. The disturbance is effected

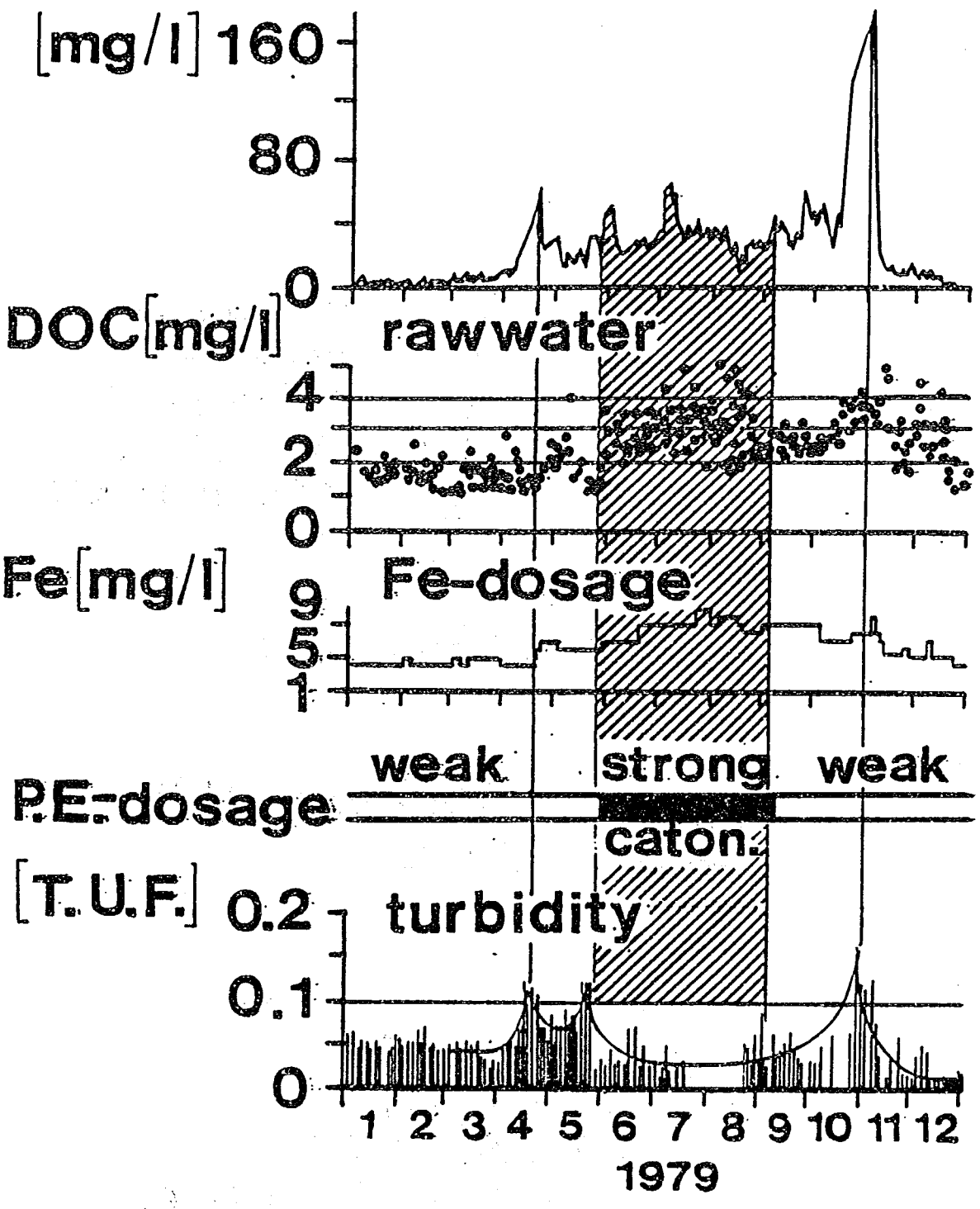


Fig. 1 The association of the occurrence of increased chlorophyll concentrations with increased DOC as an indication of algal organic substances and an interference with flocculation (practical results from the PEP 1979).

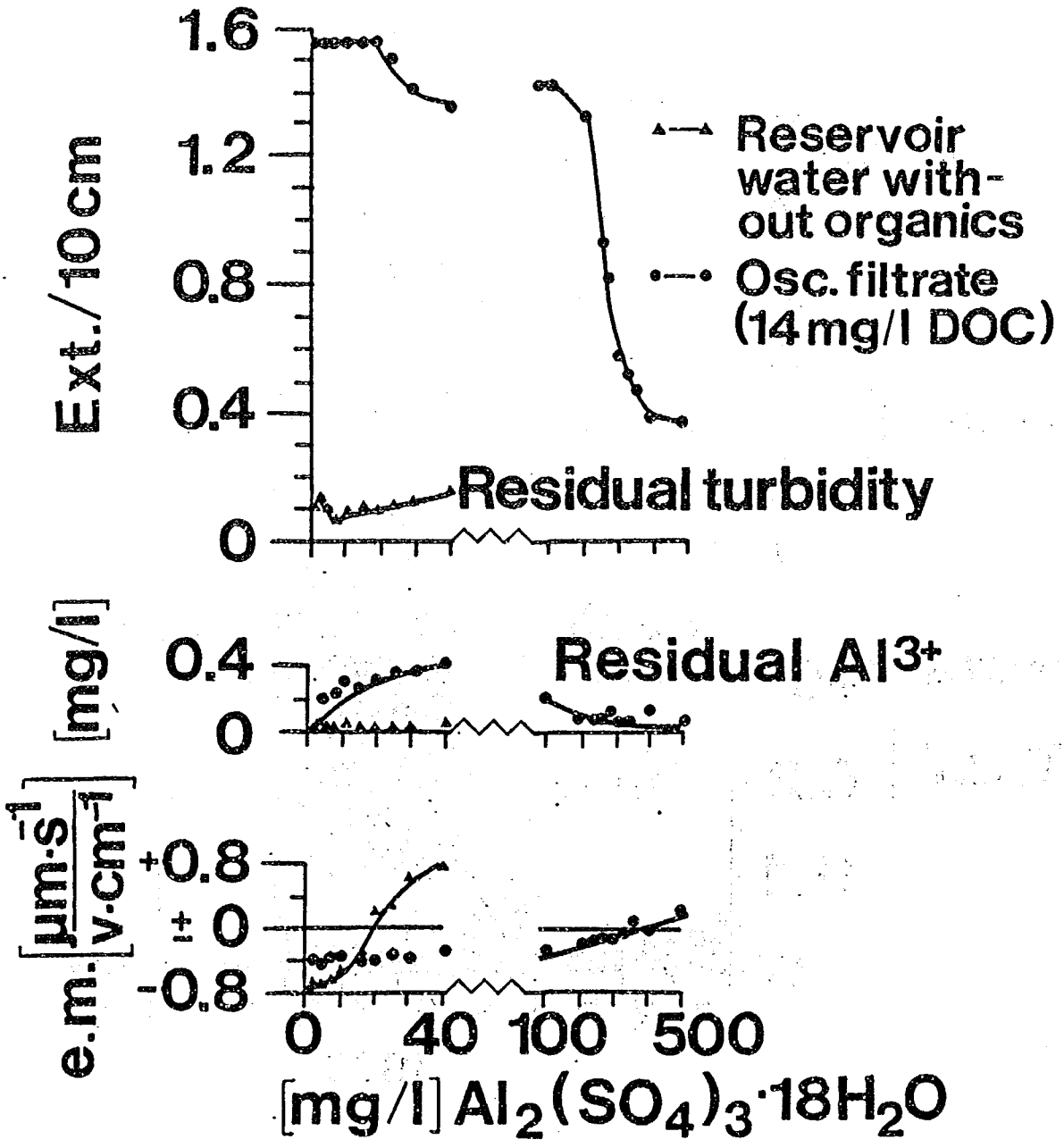


Fig. 2 Interference with flocculation caused by the filtrate of an Oscillatoria suspension from the Wahnbach reservoir.

already by very low concentration of these substances (~0.1 mg/l). An increase in the content of DOC of the raw water, which can be proved to be due to an extensive development of algae, can indicate that such substances are present. However, it cannot be overlooked, that DOC is a cumulative parameter in which the total amount of dissolved organic compounds in water is included and from which only a small proportion is able to interfere with flocculation.

Values for the chlorophyll- and DOC-content in the pre-reservoir water 1979 have been plotted in Fig. 1. Phosphate is eliminated from the Wahnbach following its storage in the pre-reservoir by flocculation with iron salts and a weakly, or when necessary a strongly cationic flocculation aid, followed by direct filtration with multimedia filter (17, 18).

The impairment to the flocculation process caused by algal born organic substances contributing to the DOC, led to an increase in the turbidity values in the filtrate from the multimedia filters (see for example the middle of April and May and the middle of October). These disturbances, which occurred due to the mass development of algae in the pre-reservoir, mainly in summer, could only be countered by increasing the dosage of ferric salts from an average of 5 mg/l Fe to 9 - 10 mg/l and by using a strongly cationic flocculation aid instead of a weakly cationic one. Without this technical modification, new turbidity breakthroughs occurred in the second half of October due to an interference of the flocculation process at the time of a mass development of algae with high chlorophyll concentrations. The highest turbidity values were measured in the filtrate during Oct./Nov. 1979.

The influence of algal born organic substances on the flocculation process has been studied in numerous experiments with algae in the laboratories of the Wahnbach Reservoir Association (1, 2). It has been found, that pectin (partially methylesterified poly-D-galacturonic acid) is a suitable model substance for studying the effect of substances with flocculation inhibiting properties.

Among the many experiments which have been carried out in our laboratories, a particular experiment will be described here to give a

better understanding of the related events. In this experiment, the interference caused by a filtrate of an Oscillatoria suspension on the flocculation of a bentonite suspension using aluminium sulphate as flocculant was studied. The Oscillatoria rubescens suspension was collected from the Wahnbach reservoir during a mass development of the blue-green algae as aggregates at the surface of the water. The algal filaments were removed from the water by filtration and the filtrate was found to contain 14 mg/l DOC. 10 mg/l bentonite were added to the filtrate to give a turbidity and flocculation studies were carried out, the results of which are given in Fig. 2. It can be seen that a neutralisation of the negatively charged particles was first possible with the addition of 300 mg/l aluminium sulphate and a reversal of the charge was only achieved with 500 mg/l aluminium sulphate.

If the course of the electrophoretic mobility of the bentonite suspension in unpolluted reservoir water is drawn for comparison, it can be seen that the bentonite particles can be neutralized already with the addition of 20 mg/l aluminium sulphate and positively recharged with a maximum of 40 mg/l aluminium sulphate in solution.

A reduction in the residual turbidity was also first achieved in the algal filtrate with the addition of 300 mg/l aluminium sulphate, whereas the bentonite suspension in reservoir water devoid of algal substances needed less than 10 mg/l aluminium sulphate to achieve a minimum residual turbidity.

The complexing influence of the algal organic substances from the Oscillatoria suspension on the aluminium ions is very apparent in these experiments. The addition of 40 mg/l aluminium sulphate to the bentonite suspension in the Oscillatoria filtrate resulted in an increase in the residual aluminium content in the membrane filtrate (0.15 µm) to 0.4 mg/l Al^{3+}.

The properties of the Oscillatoria filtrate were very similar to those of a reservoir water in which high concentrations of algae are present and which contain an increased concentration of biogenically formed organic substances. However, the effects are, of course, not so conspicuous as those obtained with the algal filtrate containing 14 mg/l DOC.

4. Mechanisms involved in the flocculation interference.

It can be concluded from our studies and from corresponding data in the literature that flocculation is disturbed by

acid polysaccharides and
algal born hydroxy carbonic acids as well by
acidic macromolecules carrying negative charges.

The compounds in the first group are capable of forming complexes with the trivalent metal ions used as flocculants when they have particular structural configurations (position of the carboxy groups relative to the hydroxy groups).

Hydroxymonocarboxylic acids and particularly hydroxydicarbonic acids have the property of being able to form complexes with cations. The distance between the reactive centres and the possibility of forming an unstressed ringsystem with the cation play a decisive role during complexation. Extensive studies have been made in this field. Stumm and Morgan (24) calculated the extent of the formation of complexes by different organic complexing agents with cations in productive lakes under natural conditions. They showed that in a Ca^{++}-free solution within a pH range of 4.5 - 7.5 trivalent iron ions are bound in complexes with two and three dented ligands (e.g. oxalic acid and citric acid respectively). Citric acid as a three dented ligand forms more stable complexes than oxalic acid.
A similar situation exists with the two dented tartaric acid used by us as an example of a hydroxydicarbonic acid, and Al^{3+}- or Fe^{3}- ions as flocculants.

In the presence of 10^{-3} mol/l calcium ions (40 mg/l) such complex formation no longer takes place when the pH is higher than 7, because the organic complexing agents are preferentially bound to alkali earths. It follows, that flocculation disturbances should not occur to such an extent in harder waters as they do in the soft reservoir waters (Ca^{++}-concentration 15 - 20 mg/l).

Sugar acids have the property of being able to form complexes with different ions such as Ca^{++}, Mg^{++}, Al^{3+} and Fe^{3+}. For this reason it was

suggested that they should be used as a substitute for phosphate in detergents (25). Fialkov and Peryshkina (26) have described soluble gluconic acid metal complexes which were formed for example by dissolving a freshly precipitated aluminium oxide hydrate in a 30 % gluconic acid solution at pH 3. Following addition of methanol, a precipitate could be obtained with the formular $Al_2C_{18}H_{34}O_{23}$.

A comparison of the results from studies of the Metal-III-gluconic acid system (Metal = Fe, Al, Cr) at different pH values showed that the stability of these complexes decreased in the following succession:

iron-III > Al > chromium III
(Pecsok and Sandera (27)).

It is also known that higher iron concentrations can occur in waters containing humic compounds. The iron is bound by the humic substances giving a soluble complex in which the iron can occur both in the divalent and trivalent form (29).

It can be assumed that the metal ions, consequently also the iron ions, are bound via OH groups of the humic acid because Ziechmann (28) found that when the OH groups were methylated, which inhibited a proton exchange with metal ions, a definite decrease in the cation exchange capacity was the result. According to Zeichmann the metals were not only bound ionically like salts, but also by chelate formation, where the carbonyl groups function as electron donators. Similar structural features occur also in the acid polysaccharides which have been found in algal excretions.

In the presence of these algal born organic complex-forming substances, a competition occurs between the OH^- ions and the anionic macromolecules for the trivalent cations during the hydrolysis of Fe^{3+} and Al^{3+}. Although these reactions occur simultaneously, the formation of the polynuclear metal-III-hydroxycomplexes, necessary for the destabilization, is repressed depending upon the strength of the complexes being formed and upon the pH value.
As a result, the positively charged polynuclear hydroxy complexes, necessary for the particle destabilization and the actual flocculation process, are either not formed or formed only in insufficient amounts.

A part of the metal ions added, therefore, does not take part in the flocculation process and appears, bound in complexes, in soluble or colloidal form in the filtrate.

Several of these organic compounds react like anionic polyelectrolytes. They are capable of attatching themselbes to the active centres of the particles to be flocked by chemical reactions over covalent bonds and hydrogen bonds and thereby increasing the already negative charge of the particles (30, 24). Moreover they also react with the positively charged polynuclear metal hydroxy complexes which then become negatively charged and incapable of taking part in flocculation. On the whole, this leads to a stabilization of the colloids and to an inhibition of both the charge neutralization and the destabilization of the particles in water to be flocked.
This necessitates the addition of higher amounts of flocculant in order to neutralize the negatively charged colloids and particles. The use of cationic flocculation aids can be of advantage (2).

Some of these acid macromolecular organic compounds form insoluble substances with the trivalent metal ions added as flocculants and with lime water dependent upon pH. Precipitates are formed which can be removed by filtration (30, 31).
Using this reaction, it may be possible to remove some of the organic substances from the water, but higher amounts of flocculant have, therefore, to be added.

According to our earlier studies and using data from relevant publications the following mechanism for the interference with flocculation by algal born organic substances can be inferred as a guideline:

4.1 The formation of complexes with the trivalent metal ions used as flocculants (Al^{3+} and Fe^{3+}).

The disturbances to flocculation are caused mainly by algal born organic substances which have carboxy groups in the vicinity to hydroxy groups, such as hydroxy carbonic acid, acid polysaccharides, humic and fulvic acids. The hydroxy carbonic acids compete with the

OH^--ions for the trivalent cations, whereby, depending upon the strength of the complexes being formed and on the pH value (availability of OH^--ions), the one or the other complex formation dominates. Because of the competitive complex forming reactions, a proportion of the trivalent metal ions added are withdrawn from the formation of positively charged hydroxy complexes. Because, however, the positively charged hydroxy complexes of trivalent aluminium or iron serve to effect the destabilization process, the process of neutralization the electrical charge becomes inhibited or even prevented. At the same time, increased iron and aluminium concentrations occur in the filtrate as a result of the formation of dissolved and microparticulate iron- and aluminium complexes. These complexes have colloid properties due to the macromolecular structure of the biogenically formed complexing agent. They pass through the filter of the flocculation plant, but are retained on the whole by membrane filters with a pore size of 0.45 µm.

4.2 Expansion of the negative charge density.

Some of the biogenically formed interfering substances have a high negative charge density and cause an increase in the negative charge potential in the aquatic medium. In order to neutralize the negatively charged colloids and particles (isoelectric point), therefore, the addition of positively charged carriers must be considerably increased. This can either be achieved by adding cationic polyelectrolytes or by considerably increasing the amount of iron and aluminium ions to be added with the aim of forming larger amounts of positively charged hydroxy complexes in water.

4.3 Function of protective colloids.

Some of the biogenically formed acidic organic substances behave like anionic polyelectrolytes. They are capable of attatching themselves to the active centres of the particles to be flocked by chemical reactions involving covalent and hydrogen bonds which results in an

increase in the charge of the already negatively charged particle. Moreover, they also react with the polynuclear iron- and aluminium hydroxy complexes and cause a neutralization and negative recharge of the complexes. The result is, that the hydrolytically formed hydroxy complexes needed for the destabilization are effectively lost. On the whole, this reaction leads to a stabilization of the colloids and to an inhibition of the charge neutralization and destabilization of the colloids and particulate substances. Negatively charged colloids are even formed in addition.

4.4 Formation of insoluble macromolecular metal-III-organic complexes.

Macromolecular organic compounds are capable of forming insoluble compounds with aluminium and trivalent iron ions. For example, it is known that humic acid can form a sedimentable insoluble aluminium humate in the pH range of 5 - 7.
Although this is a favourable phenomenum from the view of drinking water works, it does, however, demand an increased use of flocculants containing aluminium and iron ions needed for particle destabilization.

5. Studies on the flocculation interference caused by algal born organic substances.

The Institute for Civil Engineering of the Technical University, Aachen, the Wahnbach Reservoir Association and the Institute for Biotechnology 3 in the JÜlich Nuclear Research Centre have launched an extensive research programme on the elucidation of the nature of the disturbances to flocculation caused by algal born organic substances present in the water. The research programme is financed by the Federal Ministry for Research and Technology of the German Federal Republic, and is a continuation of studies made by the Wahnbach Reservoir Association, in which it was first shown that the disturbances to flocculation were caused by organic substances of algal origin (1, 2).

The research programme comprises:

- Experimental proof of the presumed correlation between the presence of certain algal born organic substances in the raw water and the occurrence of the interference to flocculation.
- Identification of the organic compounds causing the disturbance at least to the level of the class of compound. Particular value is to be laid on the molecule size and the number, type and reactivity of the functional groups in the classes of compounds in order to be able to make conclusions on the inhibital properties of these types of organic compounds.
- The development of suitable technological preventative measures to deal with flocculation disturbances.

These experiments are carried out under the condition that:

- a standardized flocculation test can be used
- sufficient amounts of relevant biogenically formed substances can be isolated.

For these purposes, the flocculation test apparatus developed by the Wahnbach Reservoir Association (34, 35) and the plant developed by the Institute for Algal Research in Dortmund for mass cultivation of algae were used (36, 37).

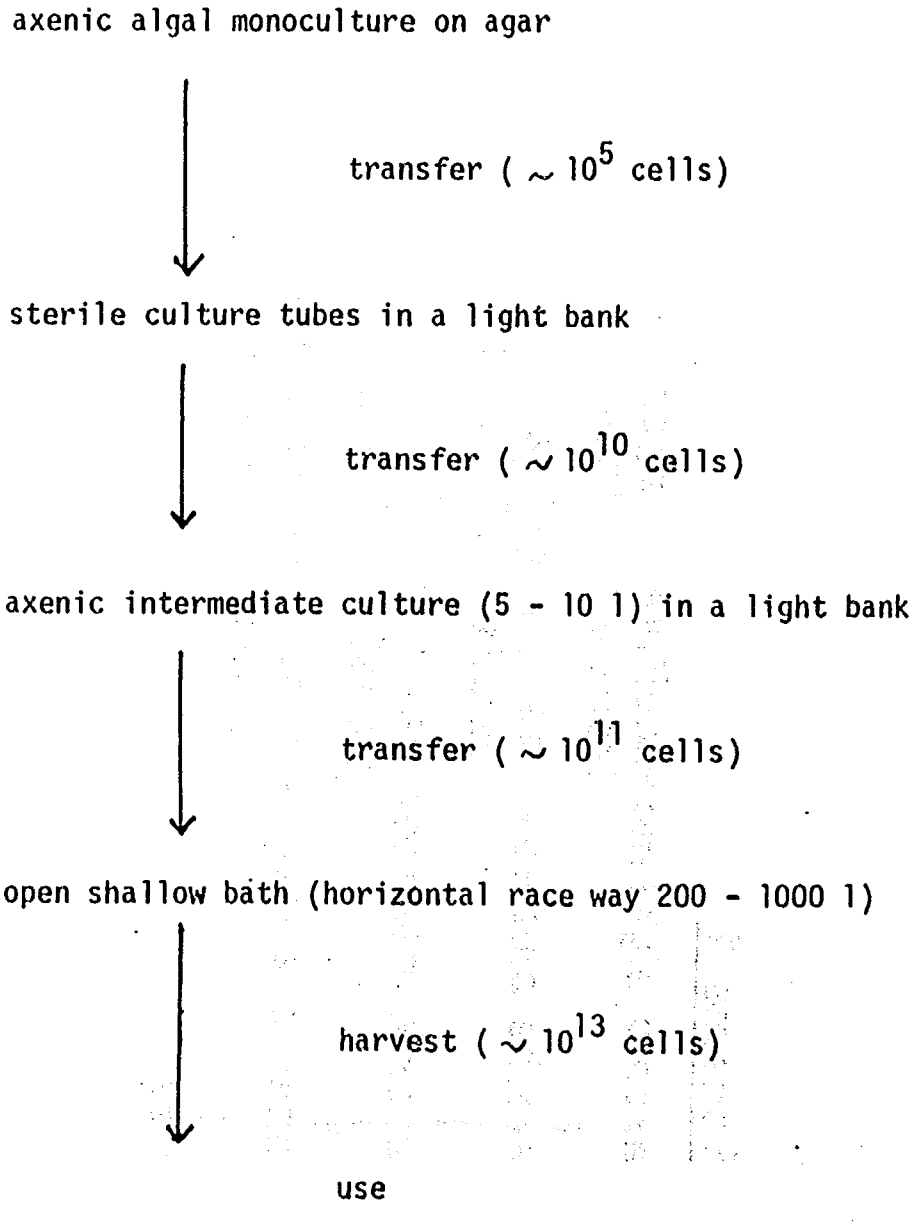
5.1 The mass cultivation of algae.

In order to produce sufficient amounts of the substances needed, it is necessary to cultivate the algae on a large scale. Several steps are needed during the cultivation because the large scale cultures cannot be run axenically. However, high inocculation densities with axenic pre-cultures of a particular algal strain create good competitive conditions which suppress the growth of other foreign alga accidentally carried into the culture. In this way it is possible to cultivate suitable planktonic algae which occur in mesotrophic or eutrophic lakes and reservoirs, on a large scale, under non-sterile conditions and with a purity of 98 % over longer periods of time (36), see scheme 1).

Scheme 1

axenic algal monoculture on agar
↓ transfer (~10^5 cells)
sterile culture tubes in a light bank
↓ transfer (~10^{10} cells)
axenic intermediate culture
↓ transfer (~10^{11} cells)
open shallow bath (horizontale race way)
↓ harvest (~10^{13} cells)
use

Scheme 1


axenic algal monoculture on agar
transfer (~ 10^5 cells)
sterile culture tubes in a light bank
transfer (~ 10^10 cells)
axenic intermediate culture (5 - 10 l) in a light bank
transfer (~ 10^11 cells)
open shallow bath (horizontal race way 200 - 1000 l)
harvest (~ 10^13 cells)
use

The mass cultivation is mainly dependent on the following conditions (38):

nutrient medium
illumination
turbulence
temperature
pH-value

The conditions are adjusted to suit the cultivation of a particular algal species and can be varied within certain limits. The preparative cultivation of the algal monocultures is carried out in 250 and 1000 l culture units, the design of which was based on the units developed by Stengel (36). These horizontal troughs, which are used for the "clean cultivation technique" with chemically defined cultivation medium, can be used either as batch cultures or run continuously as chemostate. Whereas during a batch culture the nutrient concentration decreases with increasing culture density, in a chemostatically run culture, the nutrient solution is continually added to achieve a steady state in which the culture density and nutrient concentration are held in narrow limits. The culture units are illustrated in Fig. 3. The algae are kept in suspension to ensure that they receive sufficient light and nutrients. The necessary turbulence and mixing is carried out by a paddle wheel (10 - 20 rpm). A homogenous current through the trough is effected by conductor plates in the curves. Mercury vapour lamps (Philips HPLR) are used to illuminate the cultures at a light density of between 10 and 50 K Lux.

The temperature suitable for algal growth is regulated using a thermostat with a temperature variation of $\pm$ 0.1 C. Cooling is effected using a cooling medium at 4° C. The usable temperature range lies between 10° C and 30° C.

The input of CO_2 into the culture is not only necessary to supply the algae with a carbon source, but also serves to regulate the pH value, which can be held at values between 6 and 11 $\pm$ 0.25.

Generally, the chemostat culture is run as a batch culture until a certain algal density is achieved. Thereafter the nutrient medium is

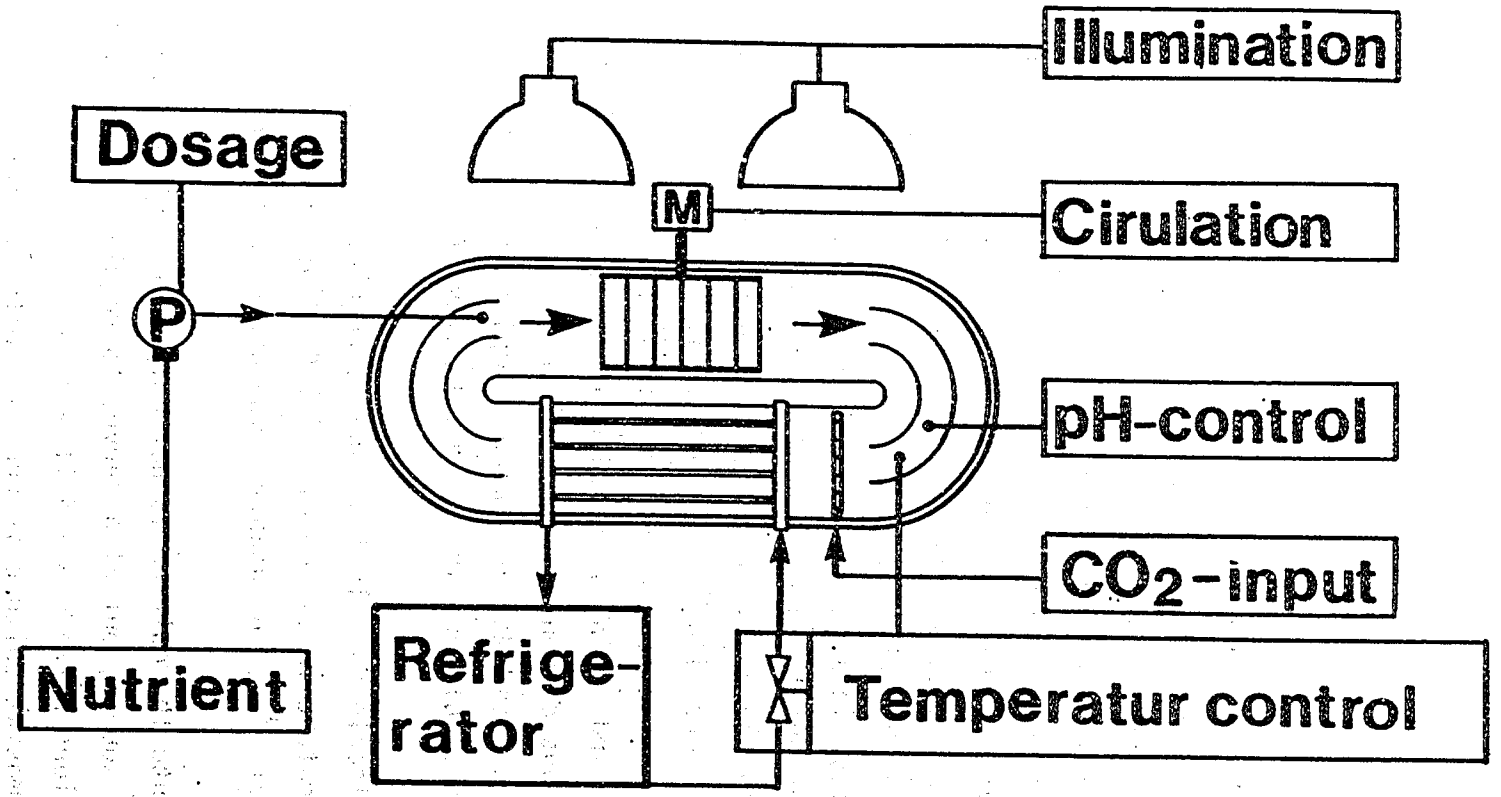


Fig. 3 Algal mass culture plant according to Stengel (36).

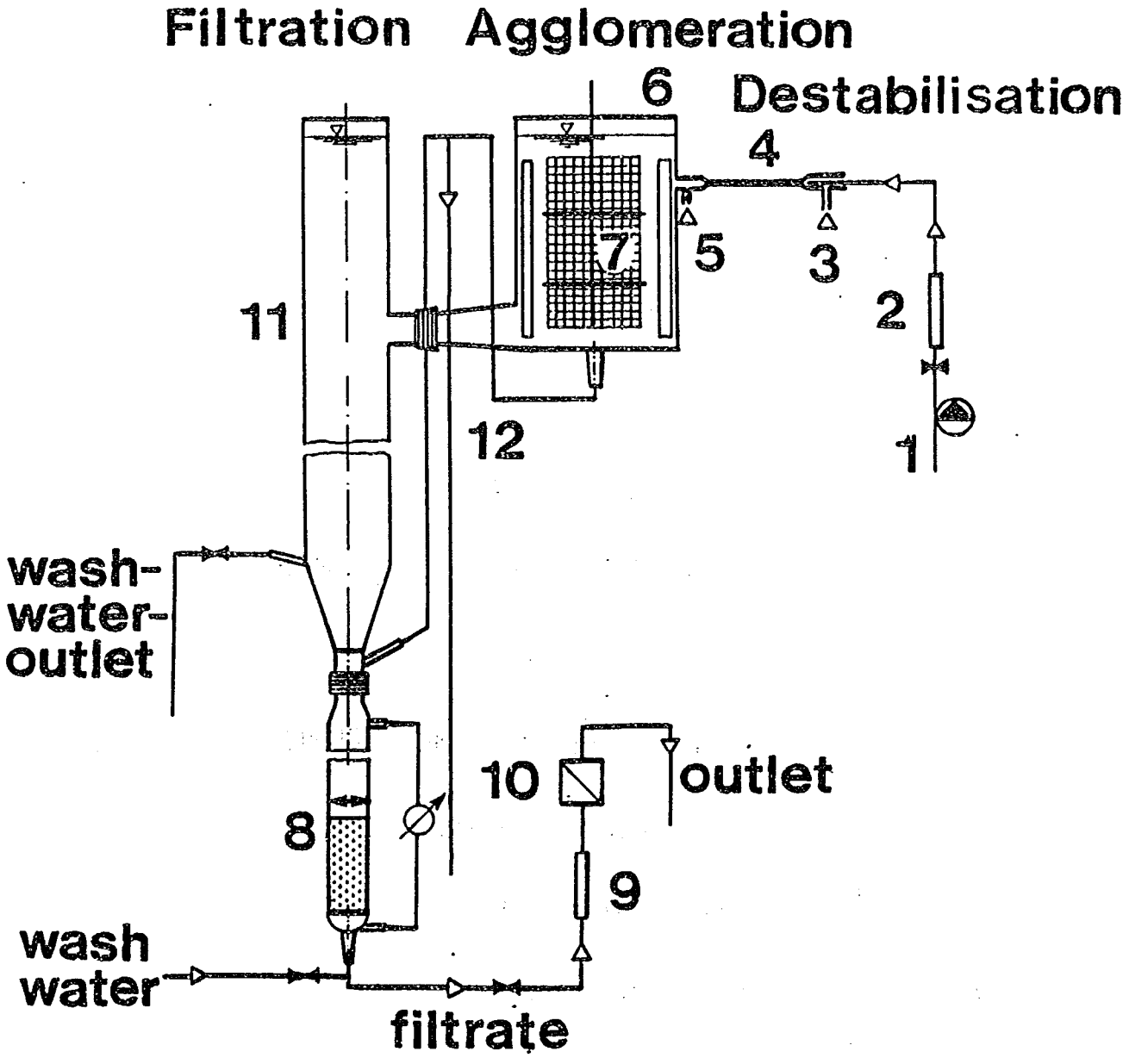
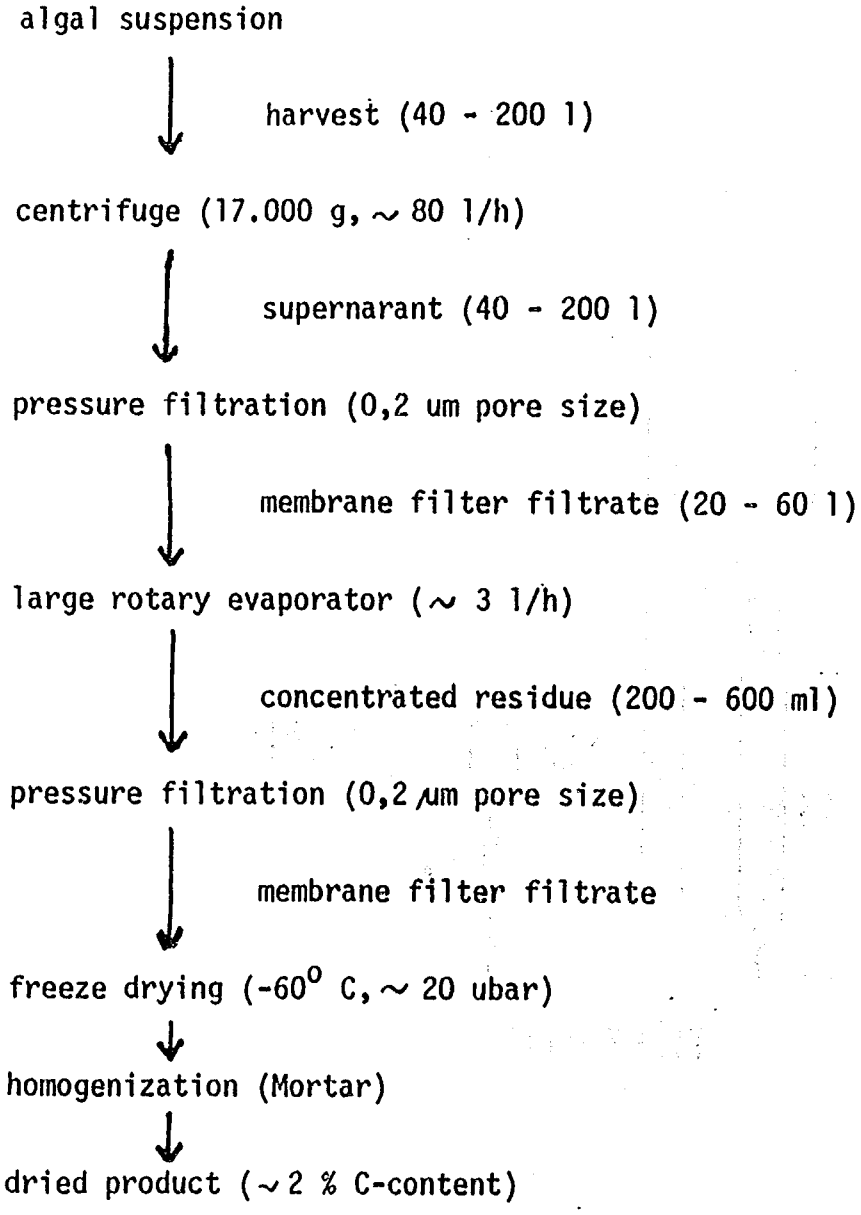


Fig. 4 Apparatus used for flocculation tests by the Wahnbach Reservoir Association (40).

Scheme 2


algal suspension
harvest (40 - 200 l)
centrifuge (17.000 g, ~ 80 l/h)
supernarant (40 - 200 l)
pressure filtration (0,2 um pore size)
membrane filter filtrate (20 - 60 l)
large rotary evaporator (~ 3 l/h)
concentrated residue (200 - 600 ml)
pressure filtration (0,2 µm pore size)
membrane filter filtrate
freeze drying (-60° C, ~ 20 ubar)
homogenization (Mortar)
dried product (~ 2 % C-content)

added at a continual rate and corresponding amounts of medium are removed from the culture at constant intervals. Losses due to evaporation are also taken into account. After about 3 to 5 days the culture stays at a constant algal density and a steady state is achieved.

The composition of the nutrient medium used was similar to that of the water of the Wahnbach reservoir (39).
It was important for the success of the experiments that the trace elements necessary for algal growth were not rendered soluble in the nutrient medium through the use of complexing agents. The addition of such complexing agents would have led to a distortion of the results obtained from the effect of the biogenically produced organic compounds under study on the flocculation process.

Different green algae, blue-green algae and diatoms were grown in these mass cultures and were harvested in both the logarithmic and steady state growth phases.
The excreted organic substances were prepared from the algal culture according to scheme 2 and used for the flocculation tests.

5.2 The flocculation test apparatus.

5.2.1 Construction

The flocculation test apparatus (34) (see Fig. 4) consists of three parts relative to the processes which take place during flocculation.

1. Destabilization
In the destabilization step, the negative ly charged colloids and particles become neutralized.
This is effected by the addition of flocculants (iron-III or Al-III salts) and by the formation of positively charged hydroxy complexes which are immediately formed following hydrolysis. The turbulent water current brings the particles in a suitable condition for destabilization. This event, which occurs in a fraction of a second, takes place in the mixing capillary (3 mm Ø) in front of the flocculation vessel (35).

2. Agglomeration

In the following agglomeration step, which occurs in the flocculation vessel, a specific energy application with a G-value of 92 s^{-1} continued over a period of 240 seconds (GT = 22,000) causes an accumulation of the destabilized particles (micro-flocks) into larger units, which are later removed by filtration. The flocculation vessel has a capacity of 5 l and is equipped with stators. The energy is applied by means of a variable speed grid stirrer.

3. Filtration

The removal of the flocks, which contain the turbidity substances, is carried out in the filtration step. The filter container has the following dimensions and characteristics:

depth of filter bed	10	cm
filter sand, seived, particle size	0.8	mm
filter velocity	15.7	m/h
water head	60	cm
volume of water head	615	cm^3
retention time in volume of water head	37	s
filter diameter	4.5	cm

5.2.2 Determination of the filtrate quality.

The filtrate quality is determined by continual measurements of the residual turbidity (Formazine units, 90^0 scattered light) and the residual iron content (o-phentanthroline method with an autoanalyzer (40)). Initially, only iron-III-salts were used as flocculants.

5.2.3. Characterization of the filtration progress.

In order to characterize the filtering property of the flocks formed and to monitor the filtration process, the development of pressure loss during filtration is measured (cm water column).

The parameter pressure loss, residual turbidity (minimum value) and residual iron content (Fe_{tot}) as well as the filter running time were used to characterize the filtration progress. This is done numerically with a value obtaining by dividing the filter running time

by the residual turbidity (minimum value). The dead time at the beginning of the flocculation test was substracted from the total filter running time in order to use the actual filter running time in the calculation.

Fig. 5 shows an example of the turbidity change in a flocculation test with D-galactaric acid corresponding to 0,1 mg/l DOC. 3, 5 and 7.5 mg/l Fe^{3+} were used for flocculation in this test. The minimum values of the residual turbidity are plotted as horizontal lines in Fig. 5.

$$KZ = \frac{t_L - t_d}{T\ min} \qquad (1)$$

KZ = characterization number

t_L = filter running time until breakthrough

t_d = dead time at the beginning of the filter run

T_{min} = minimum value for residual turbidity in the filtrate

In addition the filtrability number according to Ives (41, 42) is calculated in which pressure loss occurring after a passage of 2.5 l and the residual iron content or residual turbidity in the 2.5 l of filtrate are included in the calculation (see the horizontal line in the 3 diagrams in Fig. 6).

$$F = \frac{H}{v.t} \cdot \frac{C}{C_o} \qquad (2)$$

F = filtrability number according to Ives (41)

H = pressure loss (m) after a passage of 2.5 l through the flocculation apparatus

v = filtration velocity ($m.s^{-1}$)

t = filter running time (s)

C = residual concentration of Fe_{tot} (mg/l) or turbidity (T.U.F.) in the 2.5 l filtrate collected

C_o = initial concentration of Fe_{tot} (mg/l) or turbidity (T.U.F.) before filtration

5.2.4 Flocculation test runs with model water and a quartz suspension without organic additives (basis data).

In order to determine the basis data of the flocculation test runs a model water was used which was similar in composition to the water of the Wahnbach reservoir (Table 2).

Table 2: Composition of the model water used for flocculation experiments in the flocculation test apparatus

Na_2SO_4	0.26 m Mol/l	36.8 mg/l
$CaCl_2.2H_2O$	0.24 m Mol/l	35.0 mg/l
$KHCO_3$	2.7 m Mol/l	267.0 mg/l
quartz powder (Sicron F 100, particle size 2 - 12 µm)		25 mg/l

The model water also contained 25 mg/l quartz powder of grain size 2 - 12 µm to obtain turbidity which corresponded to 10 T.U.F. Typical results from flocculation tests using this model water are given in Fig. 6.

The reproducibility of the results from the flocculation test apparatus was tested using distilled water with 50 mg/l Sicron F 100 at a pH of 6 and a temperature of 20 $\pm$ 1° C. Flocculation was realized with 5 mg/l Fe and the flocculation test was repeated 15 times within a period of 2 months. The results are given in Table 3. It can be seen that the standard deviation from the mean value of the residual turbidity in the filtrate was 0.39 $\pm$ 0.047 T.U.F. The individual values lay between 0.31 and 0.44 T.U.F. It must be considered that the absolute values of the residual turbidity represented here with a mean of 0.4 T.U.F. are very low.

The mean residual iron content was 0.15 mg/l Fe with a very low standard deviation of 0.0177 mg/l Fe. These values lay at the limit of detection of the analytical method used (o-phenanthroline method with an autoanalyzer (40)).

Table 3: Flocculation test with a model water to ascertain the repeatability of the method

model water = dist. water + 50 mg/l Sicron F 600
flocculation with 5 mg/l Fe^{III} at pH 6, G.t 21000, G = 80 sec^{-1}
filtration speed = 16 m/h
filter = 11 cm layer, quartz sand 0.8 - 0.9 mm grain size

experiment No.	residual turbidity in filtrate (T.U.F.)		deviation from the mean value (%)	residual iron content in filtrate (mg/l Fe)		deviation from the mean value (%)	filter running-time (min)		deviation from the mean value (%)
1	0.44	+	13.1	0.20	+	33.3	18.5	+	3.9
2	0.42	+	8.0	0.18	+	20.0	18.5	+	3.9
3	0.42	+	8.0	0.16	+	6.7	18.0	+	1.1
4	0.40	+	2.8	0.15	±	0	16.5	-	7.3
5	0.42	+	8.0	0.14	-	6.7	17.5	-	1.7
6	0.40	+	2.8	0.14	-	6.7	16.5	-	7.3
7	0.35	-	10.0	0.14	-	6.7	19.0	+	6.7
8	0.38	-	2.3	0.16	+	6.7	18.0	+	1.1
9	0.31	-	20.3	0.14	-	6.7	20.0	+	12.4
10	0.33	-	15.1	0.14	-	6.7	18.0	+	1.1
11	0.34	-	12.6	0.14	-	6.7	19.5	+	9.6
12	0.39	±	0	0.15	±	0	17.5	-	1.7
13	0.33	-	15.1	0.14	-	6.7	16.0	-	10.1
14	0.44	+	13.1	0.15	±	0	17.0	-	4.5
15	0.46	+	18.2	0.14	-	6.7	17.0	-	4.5
mean value $\bar{X}$	0.39			0.15			17.8		
standard deviation S	± 0.047 (± 12 %)			± 0.0177 (± 12 %)			± 1.14 (± 6 %)		

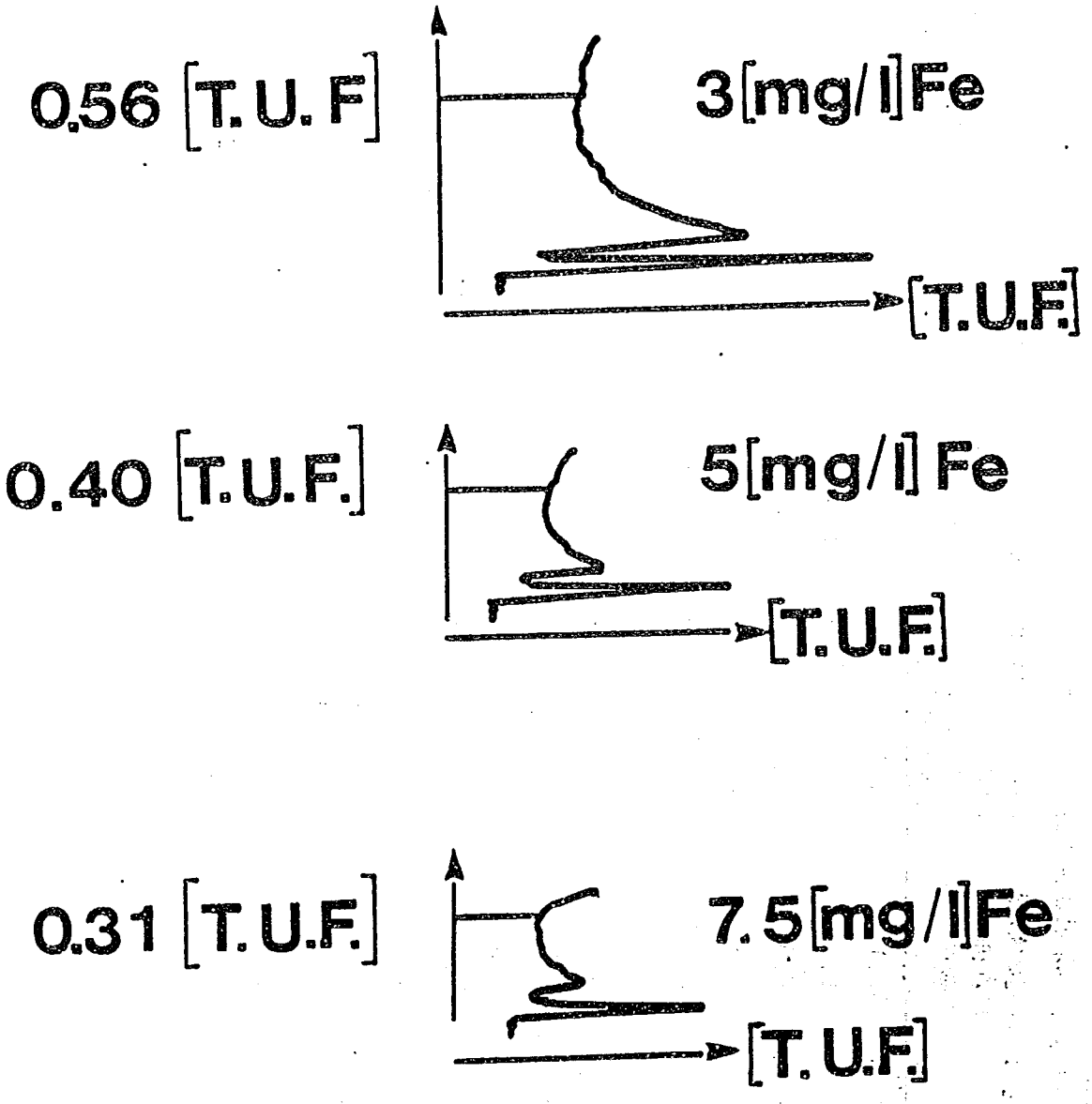


Fig. 5 Example of flocculation tests using D-galactaric acid to determine the minimal residual turbidity in the filtrate.

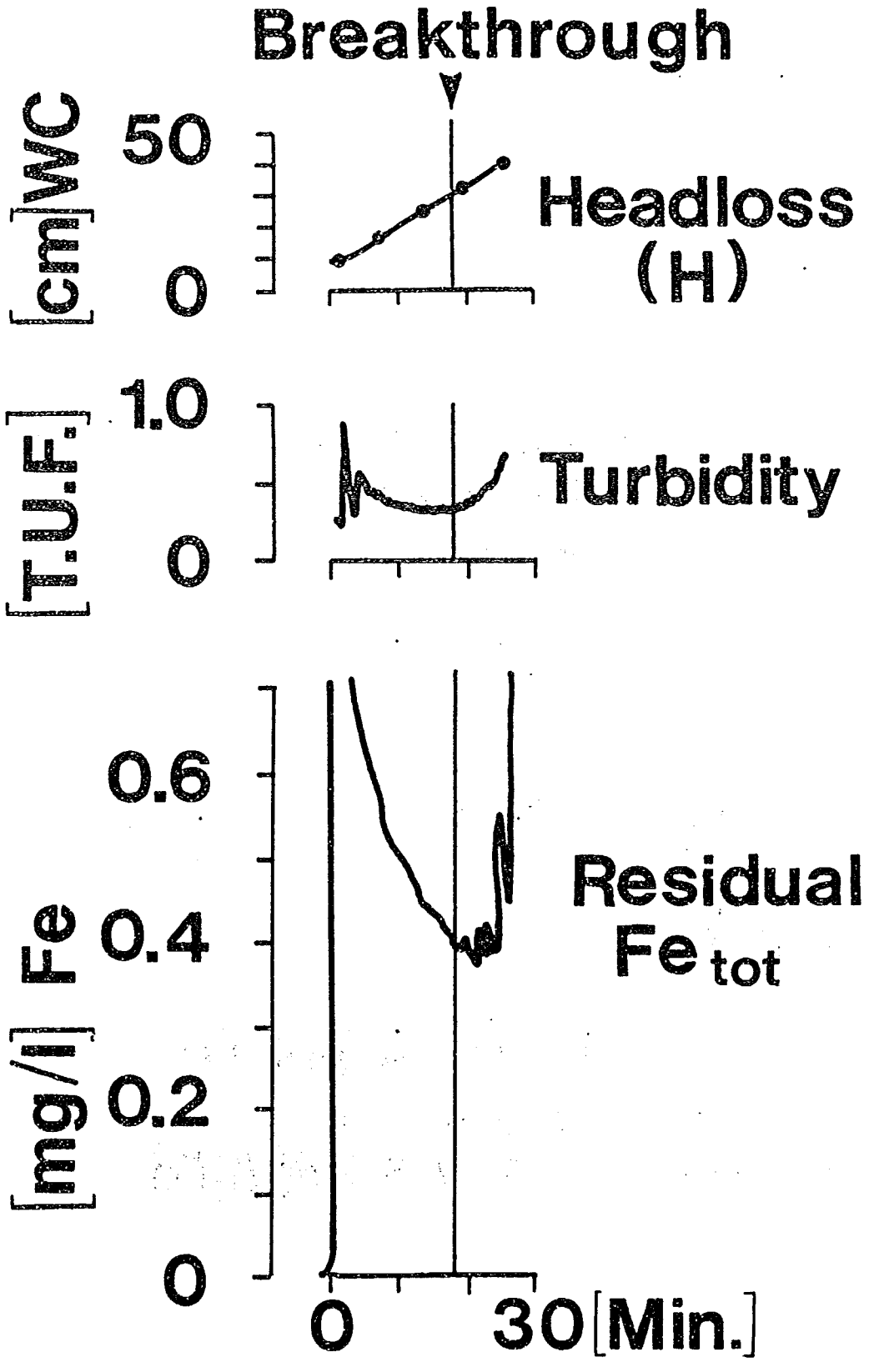


Fig. 6 Course of the curves for head loss, residual turbidity and Fe_{tot} in the filtrate from flocculation tests using a model water containing no organic substances.

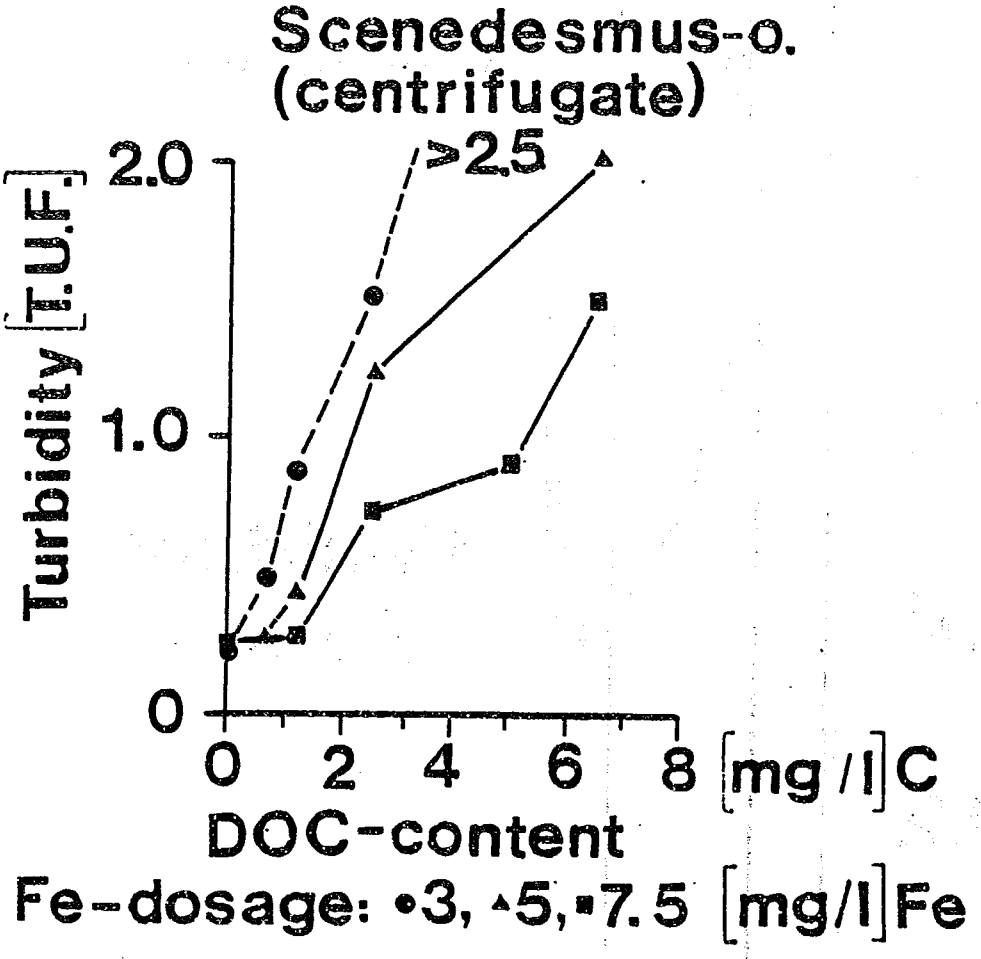


Fig. 7 Influence of increasing DOC-concentration of minimum residual turbidity for 3 different Fe^{3+} dosages.

If one disregards the values obtained from experiments 1 and 2, the deviation of the individual values from the mean for the residual iron in the filtrate was not more than ± 6.7 %.

The filters were run until iron oxide hydroxide flocks broke through the filter. The running times correspondingly ranged from 16 to 20 minutes with an average of 18 minutes. The standard deviation from the mean value amounted to ± 1.1 minute.

These deviations are so low that results obtained with the flocculation test apparatus regarding the influence of substances which alter the flocculation can be considered to be reliable.

6. Results with algal excretions and model substances.

In the following, a selection of the first results from our experimental programme will be presented. Using the apparatus described in 5.1 a culture of Scenedesmus obliquus was grown and harvested in the logarythmic phase. Flocculation tests were carried out with the centrifuge from this culture. The different concentrations of algal born organic substances were prepared by dilution with model water (Table 2) using DOC-content as the decisive measure.

The relationship between the amount of algal born substances and the minimum turbidity value in the filtrate following the addition of 3, 5 and 7.5 mg/l Fe as flocculant is shown in Fig. 7.

Because the values for the residual turbidity and the residual iron in the filtrate parallelled each other, only the values for the turbidity have been plotted.

It can be seen from Fig. 7 that using only 3 mg/l Fe as flocculant, the minimum values of residual turbidity increase linearly with increasing DOC concentration. Already with 4 mg/l DOC the turbidity in the filtrate exceeds the range of 2.5 T.U.F. resulting from a breakthrough of the flocks.

A similar tendency can be seen when flocculation is carried out with 5 mg/l Fe although a level, even though small, can be recognized in the range of 0 - 0.5 mg/l DOC in the filtrate occurs.

In Fig. 8 the filter running times until breakthrough are plotted versus DOC content. Using 3 mg/l Fe, the filter running time decreases rapidly with increasing DOC-concentration. The addition of 5 mg/l DOC organic substance caused an immediate filter breakthrough.

When the Fe dosage was increased to 5 mg/l resp. 7.5 mg/l Fe, it appeared that the addition of 0.5 mg/l and 1.5 mg/l DOC of the organic substance promoted the flocculation process. The filter running times achieved are about twice as long as those obtained with the zero value tests (0 mg/l C). This effect must be further studied but will not be further commented upon here.

However, with >1.5 mg/l DOC the interference to flocculation in the presence of 5 and 7.5 mg/l Fe also was clearly seen as a decrease in the filter running time. With the addition of 7.5 mg/l Fe though, no filter breakthrough occurred even in the presence of 6.6 mg/l DOC when a breakthrough is considered to be a turbidity >2 T.U.F.

These results emphasize the interference caused to flocculation by the algal born organic substances released from the Scenedesmus algal cells during the logarithmic growth phase. The flocculation disturbances could be controlled to a certain extent by increasing the amount of iron-III-salts.

The same trend could be seen in similar curves, when D-galactaric acid was used as a model substance, even when the concentrations which caused an interference of flocculation were about ten times lower.

In Fig. 9 the minimum residual turbidity is plotted against DOC content. Also in this case, the minimum residual turbidity in the filtrate increases - depending upon the amount of flocculant used - to different extents. The turbidity level at DOC concentrations <0.05 mg/l C occurs because the lowest iron-III-salt dosage of 3 mg/l Fe is too high for such a small DOC concentration, and no immediate increase in the residual turbidity takes place. For DOC-concentrations >0.05 mg/l, the dosage of only 3 mg/l Fe was too small and a rapid increase in the minimum turbidity in the filtrate (Fig. 9) following filter breakthrough occurred with little more than

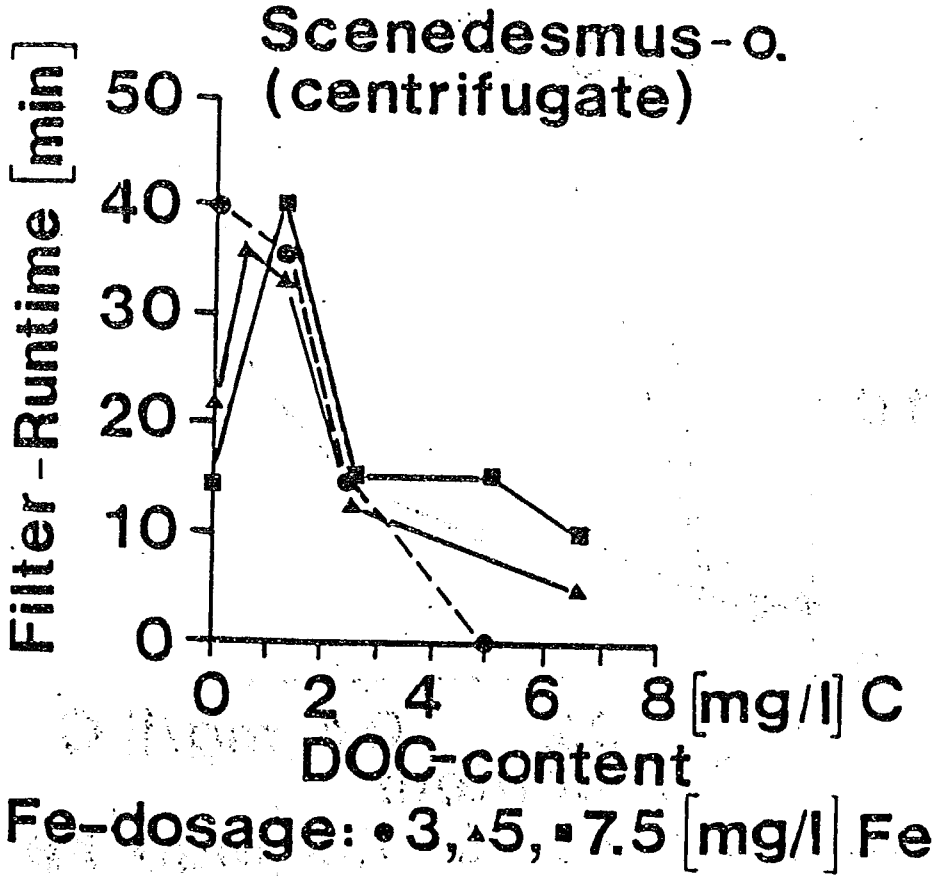


Fig. 3 Influence of increasing DOC-concentration of filter running times in flocculation tests for 3 different Fe^{3+} dosages.

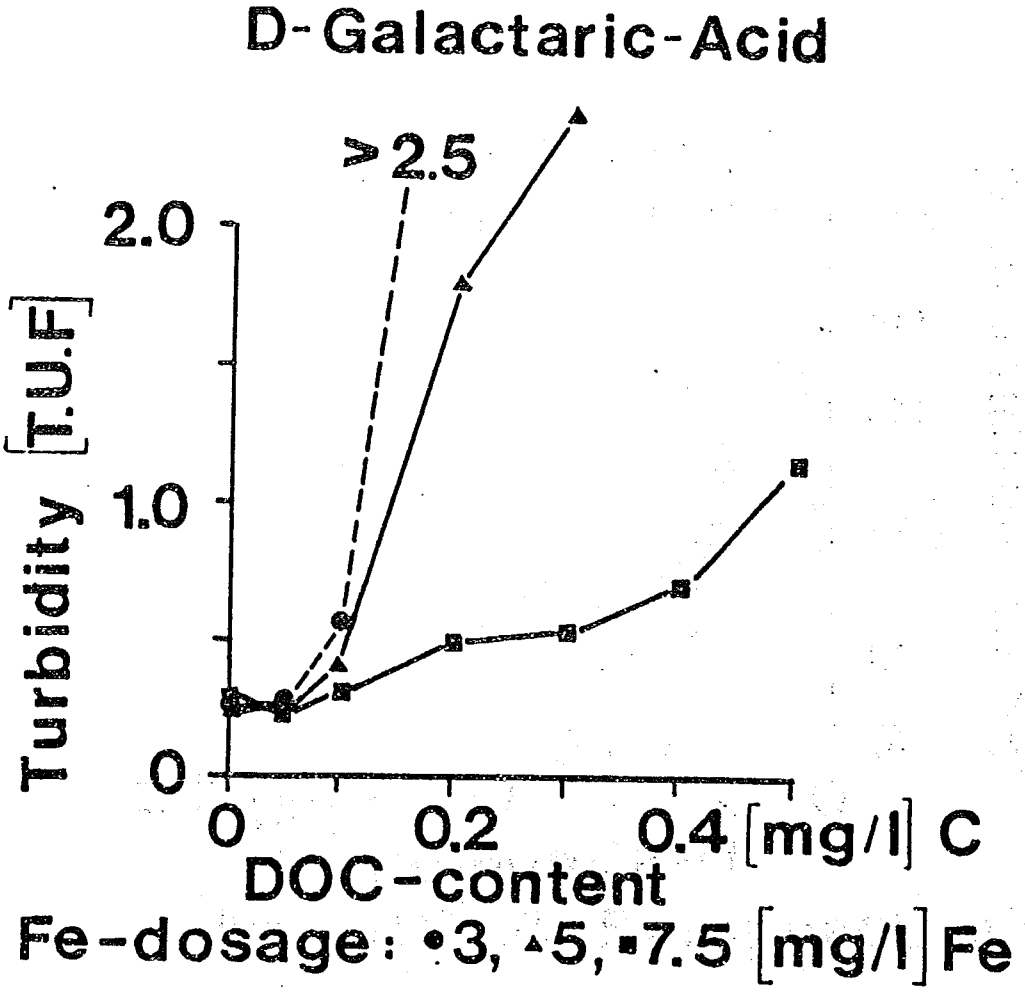


Fig. 9 Influence of increasing DOC-concentration of minimum residual turbidity for 3 different Fe^{3+} dosages.

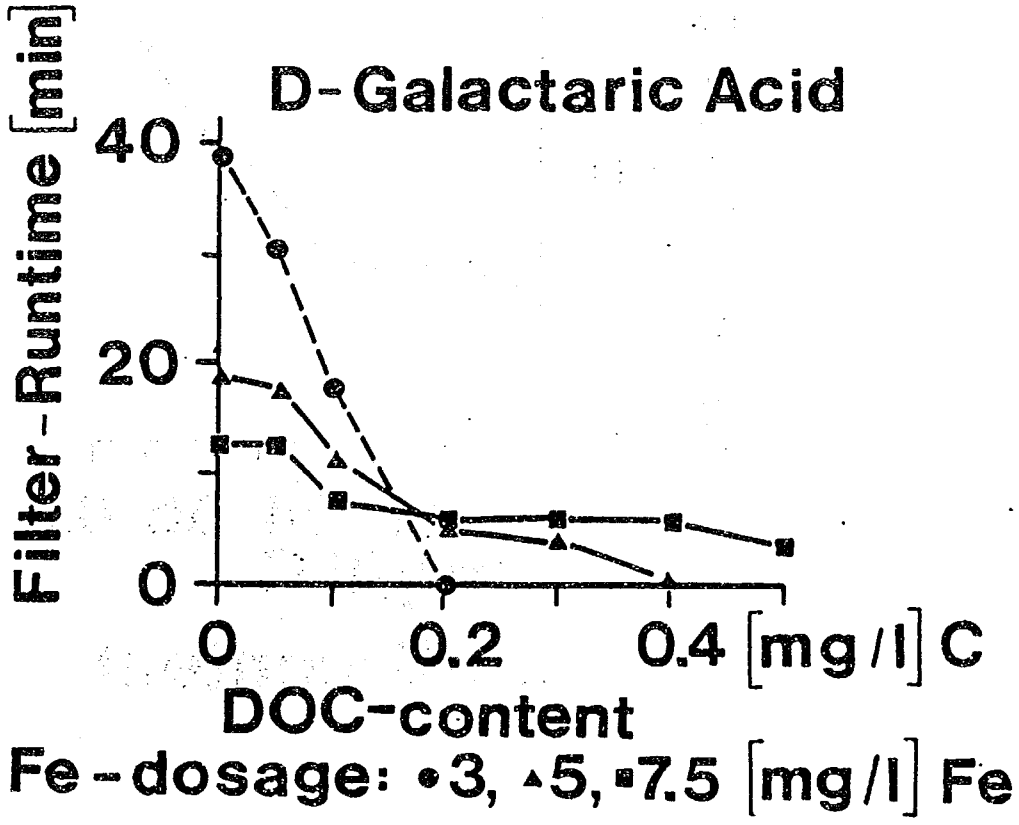


Fig. 10 Influence of increasing DOC.concentration of filter running times in flocculation tests for 3 different Fe^{3+} dosages.

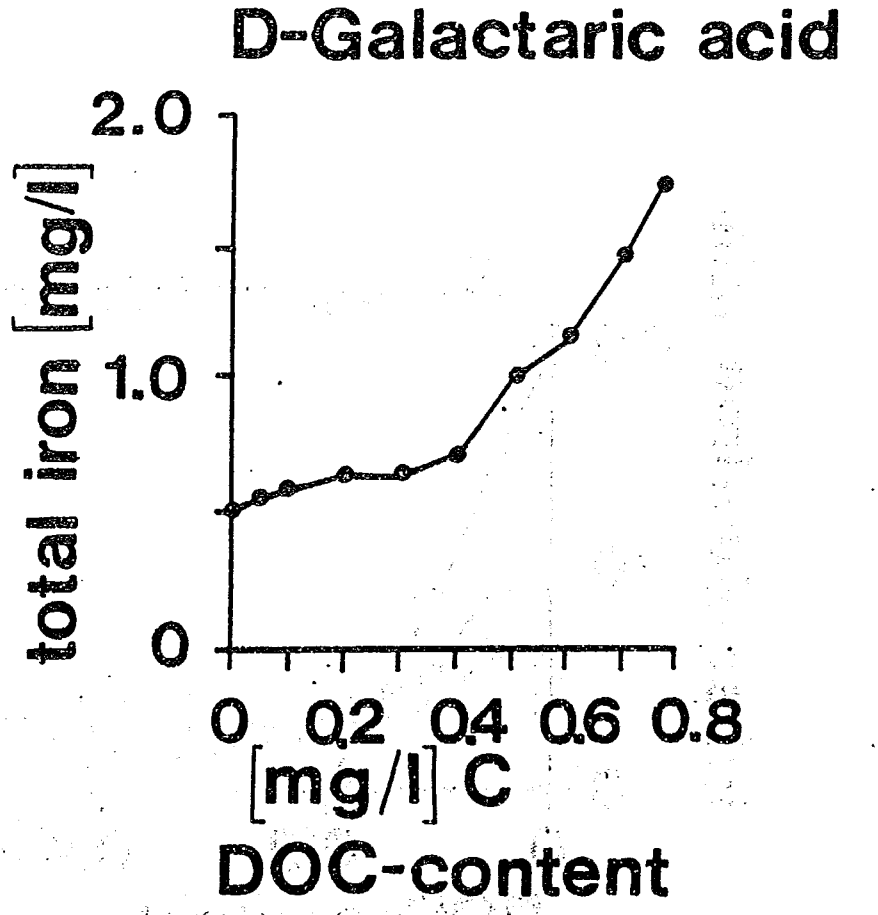


Fig. 11 The dependency of the residual iron content in the filtrate from the content of D-galactaric acid as DOC.

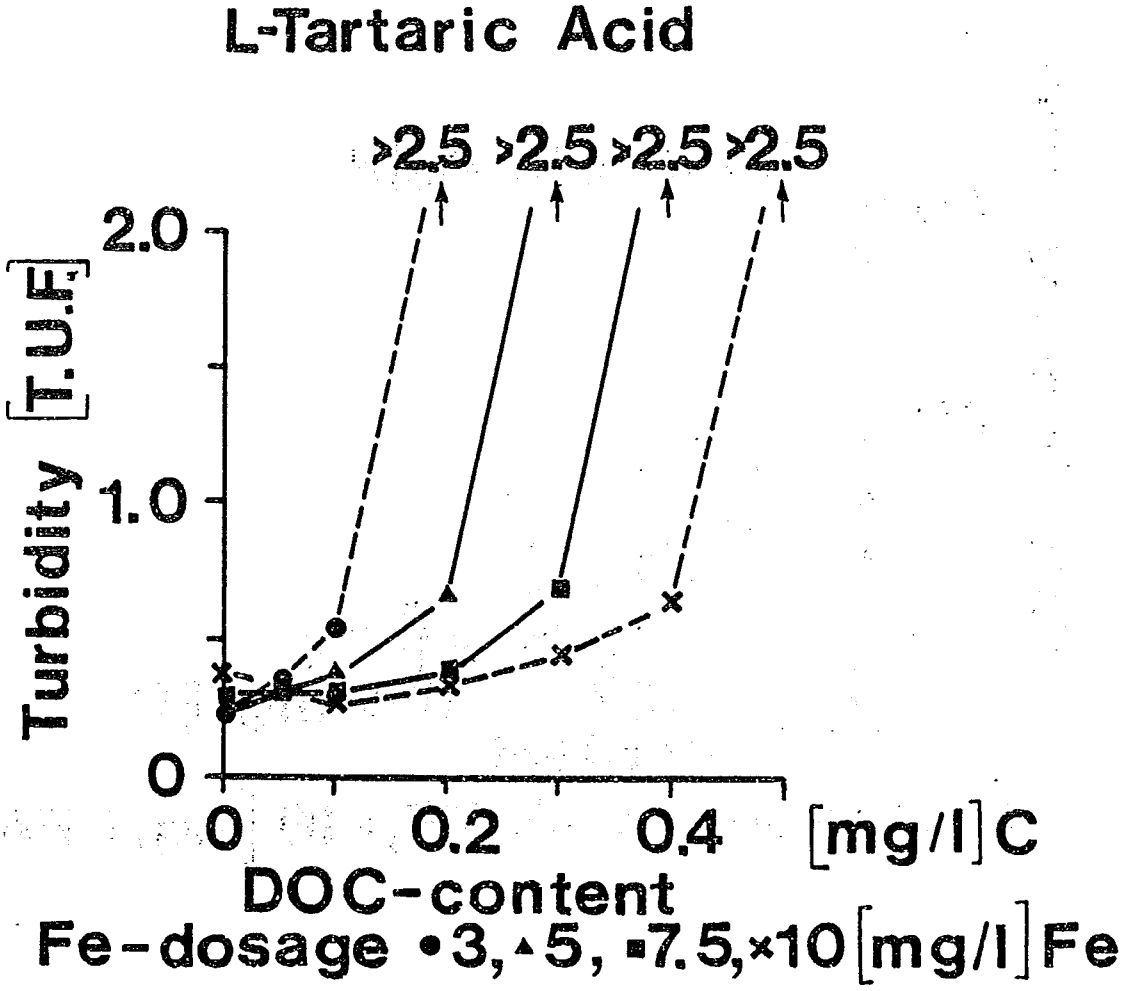


Fig. 12 Influence of increasing DOC-concentration of filter running times in flocculation tests for 4 different Fe^{3+} dosages.

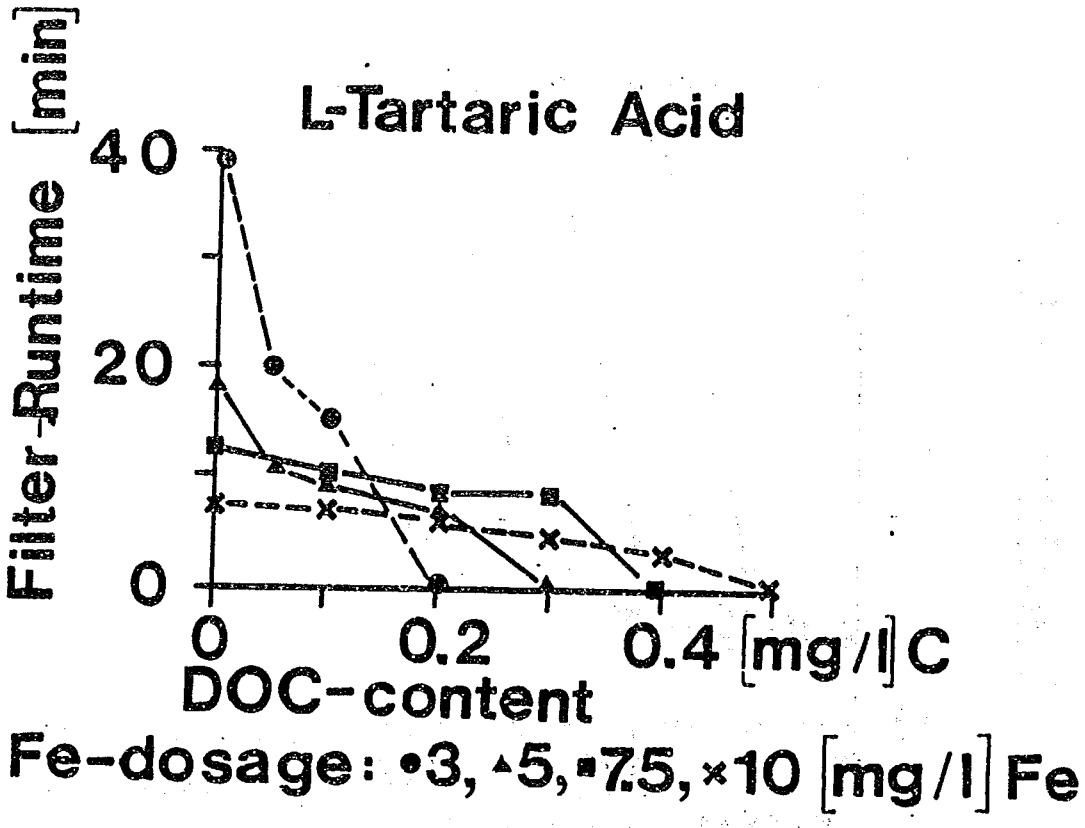


Fig. 13 Influence of increasing DOC-concentration of filter running times in flocculation tests for 4 different Fe^{3+} dosages.

0.1 mg/l C added as D-galactaric acid. Increasing the dosage of iron-III-salt to 5 mg/l reduced this effect distinctly and prevented an immediate breakthrough of the particles up to a concentration of 0.4 mg/l DOC. The effect can be seen also from Fig. 10 in which the filter running time is plotted against DOC content.

It can be seen from the data presented in Fig. 11, in which the total iron concentration in the filtrate is plotted against the DOC-concentration in D-galactaric acid, that D-galactaric acid form a complex with the Fe^{3+}-ions added. Starting with the values for residual iron in the filtrate from tests without additional organic substance it can be seen that with increasing concentration of D-galactaric acid the residual iron content of the filtrate also increases.

This tendency becomes even more apparent when tartaric acid is used as a model substance (see Figs. 12 and 13). In these studies a dosage of 10 mg/l Fe^{3+} as flocculant was also tested.

The complex forming ability of tartaric acid is so strong, that the formation of positively charged polynuclear iron hydroxy complexes is largely prevented and a destabilization of the particulate substances even with 10 mg/l Fe does not occur in the presence of more than 0.4 mg/l DOC. A filter breakthrough is the immediate result (Fig. 13).

Due to the similarity between the results from tests with model substances and from tests with algal born organic substances, it can be concluded that the organic substances released from the Scenedesmus culture have complexating properties similar to D-galactaric and tartaric acids. It can also be concluded that the algal substances inhibit the destabilization of the negatively charged colloids and particles according to the mechanisms described in 4.1. When more iron is present in solution as can be bound complex by the algal born organic substances, the surplus iron ions form positively charged polyhydroxy-complexes by the normal hydrolytic process, which can then effect the destabilization of the negatively charged colloids and particles in the aqueous phase. The residual iron content in the filtrate increases only in that extent as the iron ions added have become bound

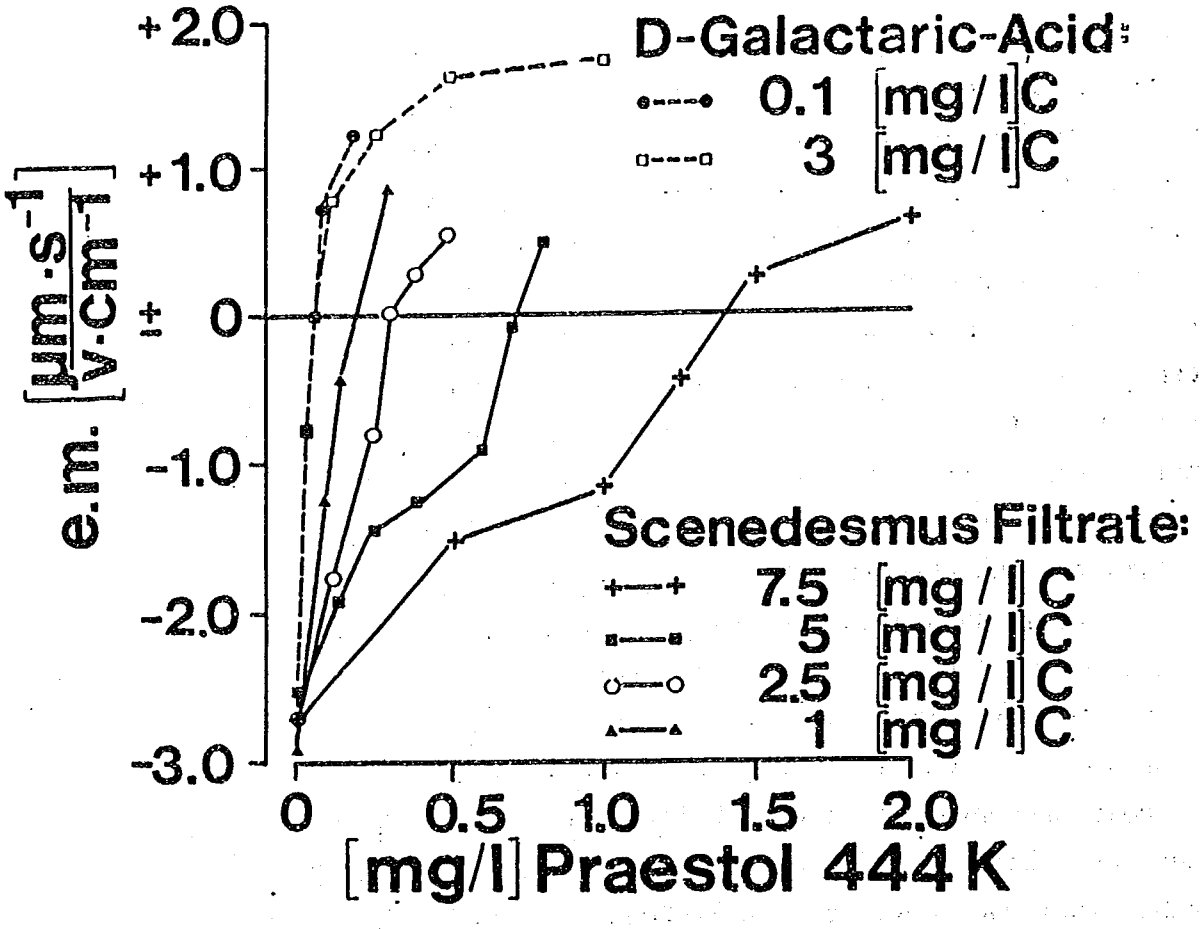


Fig. 14 The influence of organic substances of algal origin (Scenedesmus o.) and of D-galactaric acid on the negative charge potential of a quartz powder suspension.

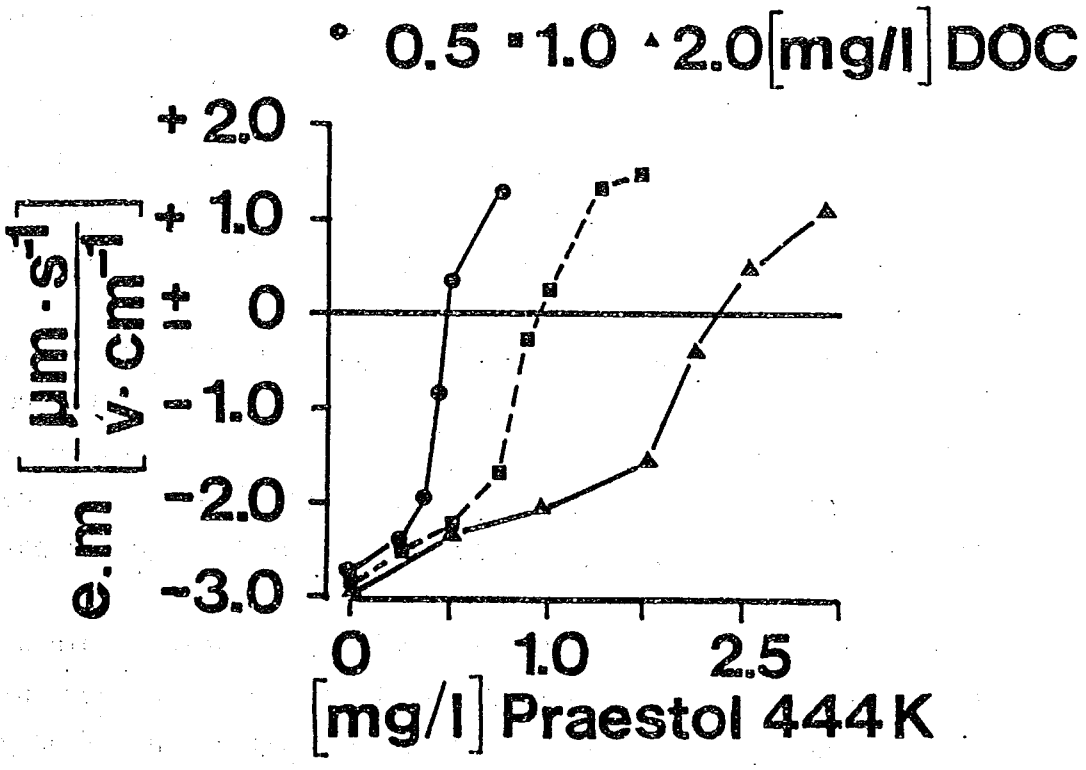


Fig. 15 The influence of different lignin sulphonic acid concentrations on the negative charge potential of a quartz powder suspension.

in complexes with the interfering organic substances.

Moreover, the algal born organic substances are able to raise the negative charge potential of an aqueous medium considerably without altering the absolute value of the electrophoretic mobility (see 4.2) (43,44). This is shown clearly in Fig. 14 in which the charge neutralization of Scenedesmus culture centrifugate with the strong cationic polyelectrolyte Praestol 444 K is plotted, and compared with charge neutralization of 2 different concentrations of D-galactaric acid. x)

It can be seen from Fig. 14, that increasing concentrations of organic substance prepared from the Scenedesmus culture centrifugate consumed increasing amounts of Praestol 444 K in order to arrive the isoelectric point.

This algal born organic substance behaves in the same way as lignin sulphonic acid regarding the elevation of the negative charge potential (Fig. 15). Lignin sulphonic acid has a higher negative charge density due to its sulphonic acid groups. This compound also causes a concentration dependent shift of the isoelectric point of a quartz powder suspension.

In contrast, in the presence of D-galactaric acid within a concentration range of 0.1 mg/l C to 3 mg/l C the quartz powder suspension can be discharged with practically the same small amount of only 0.1 mg/l Praestol 444 K (Fig. 14).

x)
For these experiments, different concentrations of the organic substances were added to 1.8 l of a Sikron F 100 (grain size 2 - 12 µm) suspension (25 mg/l) in model water (Table 2) in a beaker fitted with stators. Following the addition of Praestol 444 K, the suspension was stirred for 1 minute at 370 - 380 rpm (G-value = 1000 s^{-1}). The electrophoretic mobility was measured using a zeta meter (Rank Bro., G. B.) at 100 V tension.

It can be seen, that D-galactaric acid, which can interfere the flocculation due to its ability to form complexes (see Figs. 9 - 11), has no considerable influence on the negative charge potential of the aqueous medium in contrast to the organic substances excreted by Scenedesmus cells during the logarithmic growth phase.

Due to the fact that these substances are not only able to form complexes but can also increase the overall electrical charge of particles in aqueous suspension, it must be concluded, that the substances excreted from the algal cells consists of a mixture of different organic compounds which have both the one and the other property. These substances will be characterized not only by hydroxy dicarbonic acids, e.g. D-galactaric acid or tartaric acid, but also by organics with functional groups, which give the molecule a strong negative charge. The latter are able to form covalent bonds with colloids, turbidity-forming substances as well as with the polynuclear hydroxy complexes of iron. This is the case with humic acids and fulvic acids, as well as with cell wall products shown to carry sulphonic acid groups (21). Lignin sulphonic acid was used as a model substance representing this class of compounds.

Further studies will be carried out on the structure of these compounds.

Summary

Investigations of the Wahnbach Reservoir Association have shown that the treatment of surface waters containing large amounts of plankton, seasonal will be impaired. Disturbances of the flocculation process occurred which were characterized by

- an increase in the residual amount of flocculant in the filtrate
- a decrease in turbidity elimination
- an impairment to the disinfection process.

Organic substances of algal origin, which are released from the algal cells actively or passively by autolysis and breakdown of the cell, are responsible for the flocculation disturbances. These substances are mainly polysaccharides which often have a molecular weight $\geq$ 160,000 and other macromolecular organic compounds which are not degraded biologically and which, therefore, accumulate in the water.

The larger part of these compounds are present in the dissolved form although some can occur as colloids.

Amongst the cell wall substances which are released into the water following cell breakdown, the peripheral cell wall substance play the most important role. Thex can be described as acidic macromolecular polysaccharides with reactive groups e.g. carbonic acids and sulphonic acids. The concentrations of these substances in eutrophic lakes and reservoirs can lie within the range of 0.1 - 0.5 mg/l.

The consequences resulting from the interference of the flocculation by algal born organic substances are described using the treatment process of the eutrophic pre-reservoir water of the Wahnbach reservoir as an example.

The reaction mechanisms involved in the flocculation disturbance are discussed from results obtained from studies by the Wahnbach Reservoir Association and from an evaluation of the literature. These conclusions are given in short in the following:

1. Complexes are formed between the flocculant i.e. trivalent metal ions (Al^{3+} and Fe^{3+}-ions) and the algal born organic substances which

carry carboxy groups vicinal to hydroxy groups, e.g. hydroxycarbonic acids, acid polysaccharides, humic and fulvic acids. The structure of the substances and the positions of the reactive groups relative to each other play a substantial role.

These organic complex forming substances compete with the OH^--ions for the metal-III-ions which results in an inhibition of the formation of positively charged polynuclear hydroxy complexes of the trivalent metal ions. These positively charged complexes are necessary for the destabilization of the negatively charged colloids and particles. This competition impairs or even prevents the destabilization of the negatively charged colloids and particles with the result that a substantial part of the flocculation process is disturbed. Moreover, these organic substances form dissolved and micro-particular complexes with the Al^{3+}- and Fe^{3+}-ions which can pass through the sand filters.

2. Some of the biogenically formed interfering substances cause an increase in the negative charge potential in the aquatic medium. Because of this, increasing addition of compounds carrying positive charges is necessary to neutralize higher negative charge density on the colloids and particles.

3. Certain types of these acid substances bring about a protective colloidal effect by attatching themselves to the active centres of the colloids and particles via covalent bonds and hydrogen bonds. This causes a considerable increase in the already negatively charged colloids and particles. Moreover, they are able to combine by sorption with the hydrolytically formed, positively charged polynuclear iron- and aluminium-hydroxy-complexes and cause charge reduction or even a reversal of the charge. This leads to a decrease in the availability of positive charges and correspondingly to a disturbance of the destabilization process.

4. Lastly, some of these macromolecular organic compounds can form insoluble compounds with aluminium- and trivalent iron-ions. This is

actually desirable but necessitates an increased application of flocculant.

These relation ships are being studied experimentally in a combined effort of the Institute of Civil Engineering, Technical University Aachen, together with the Wahnbach Reservoir Association and the Institute for Biotechnology 3 at the Nuclear Research Station, JÜlich. The research project has the purpose of elucidating the cause of the flocculation interference and the type of disturbances caused by algal born substances in the water. The project is being financed by the Federal Ministry for Research and Technology, F.R.G.

Using a plant for large scale cultivation of algae developed by the Department of Algal Research in Dortmund the necessarily large amounts of algal born organic substances are produced. Diatoms, green algae and blue-green algae have been cultivated and harvested both in the logarithmic growth phase and from steady state cultures.

The different algal substances are tested for their influence on the flocculation process in a flocculation test apparatus developed by the Wahnbach Reservoir Association. The reproducibility of the tests was confirmed. The apparatus and the test procedure are described.

The initial results obtained with the centrifugate of a mass culture of Scenedesmus oblique is described. It could be shown that even a few mg/l of the dissolved algal born organic substances (expressed as mg/l DOC) could cause an interference with flocculation.

Flocculation of a turbid model water containing quartz powder (grain size 2 - 12 µm) with 3 mg/l Fe, as an example, the residual turbidity found in the filtrate increased linearly with increasing DOC concentration in the model water. Already with 4 mg/l DOC, the turbidity exceeds the detection range due to flocks breaking through the filter.

Flocculation test experiments with galactaric acid and tartaric acid as model substances showed the same tendency as the tests with algal born organic substances from the Scenedesmus culture. The similar course of the flocculation test experiments allows the conclusion to be made that the organic substances released from the Scenedesmus culture not only

have the property of being able to form complexes, but they also interfere with the destabilization of the negatively charged colloids and particles in the aqueous medium in the same way as the hydroxy dicarbonic acids used as model substances.

Moreover, algal substances originating from the Scenedesmus culture are also capable of considerably increasing the negative charge potential of a quartz powder suspension. This could be determined by measuring the electrophoretic mobility of the quartz particles after addition of different amounts of a strongly cationic polyelectrolyte (Praestol 444 K) using a Zetameter. Increasing amounts of Praestol 444 K were needed to discharge the particles (isoelectric point) in the presence of increasing concentrations of Scenedesmus culture centrifugate. In contrast, only small amounts of Praestol 444 K (ca. 0.1 mg/l) were needed to neutralize the negatively charged particles when galactaric acid was added at concentrations between 0.1 mg/l C and 3 mg/l C. This shows that galactary acid exerts no influence on the negative charge potential of the suspension although it can cause a distinct disturbance to flocculation due to its considerable ability to be able to form complexes with Me-III-ions. It is, therefore, clear that the algal born organic substances as well as having complex forming properties are also capable of increasing the negative charge potential of colloids and suspended particles.

Further studies are being carried out in order to determine the structure of these organic substances which cause flocculation disturbances.

Fig. 1 The association of the occurrence of increased chlorophyll concentrations with increased DOC as an indication of algal organic substances and an interference with flocculation (practical results from the PEP 1979).

Fig. 2 Interference with flocculation caused by the filtrate of an Oscillatoria suspension from the Wahnbach reservoir.

Fig. 3 Algal mass culture plant according to Stengel (36).

Fig. 4 Apparatus used for flocculation tests by the Wahnbach Reservoir Association (40).

Fig. 5 Example of flocculation tests using D-galactaric acid to determine the minimal residual turbidity in the filtrate.

Fig. 6 Course of the curves for head loss, residual turbidity and Fe_{tot} in the filtrate from flocculation tests using a model water containing no organic substances.

Fig. 7 Influence of increasing DOC-concentration of minimum residual turbidity for 3 different Fe^{3+} dosages.

Fig. 8 Influence of increasing DOC-concentration of filter running times in flocculation tests for 3 different Fe^{3+} dosages.

Fig. 9 Influence of increasing DOC-concentration of minimum residual turbidity for 3 different Fe^{3+} dosages.

Fig. 10 Influence of increasing DOC.concentration of filter running times in flocculation tests for 3 different Fe^{3+} dosages.

Fig. 11 The dependency of the residual iron content in the filtrate from the content of D-galactaric acid as DOC.

Fig. 12 Influence of increasing DOC-concentration of filter running times in flocculation tests for 4 different Fe^{3+} dosages.

Fig. 13 Influence of increasing DOC-concentration of filter running times in flocculation tests for 4 different Fe^{3+} dosages.

Fig. 14 The influence of organic substances of algal origin (Scenedesmus o.) and of D-galactaric acid on the negative charge potential of a quartz powder suspension.

Fig. 15 The influence of different lignin sulphonic acid concentrations on the negative charge potential of a quartz powder suspension.

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SESSION 6

SMALL-SCALE MUNICIPAL WASTEWATER TREATMENT FACILITIES

Sewage Treatment in Ponds–German Experiences
K. Bücksteeg 663
Study of Rotating Biological Contactor Process at Wastewater Treatment Plants for Small Communities
Atsushi Ohshima 641
Waste Water Treatment in Small Communities by Combined Processes
B. Boehnke 699

SEWAGE TREATMENT IN PONDS -GERMAN EXPERIENCES

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Sewage Treatment in Ponds - German Experiences

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2. Kinds of Pond Systems
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7. Ponds Combined with Rotating Disc Filters
8. Summary

1. Introduction

In Germany there are both highly industrialized regions with large towns and extended rural areas with small communities. Most of these communities built central water supply systems after World War II. The villages are growing, since novadays people like to live permanently or seasonally in rural areas. In many villages industries based on agricultural products are existing, especially dairies, breweries, slaughter houses and canneries. For these reasons controlled sewage disposal is a necessity.

Because of large distances between rural villages it is too expencive in most cases to connect them with long sewer lines for constructing large sewage treatment plants. Hence, individual small treatment units are necessary, quite often located at receiving streams with a very poor flow. These

treatment plants have to be well adapted to the special situation in rural areas.

Ponds fulfill the requirements in an excellent way; they have many advantages:

- high treatment efficiency with great process stability;
- extremely high equalization capacity to hydraulic and/or organic shock loads;
- no problems with treatment of stormwater when combined sewer systems are applied;
- simple plant construction;
- no or only little machinery;
- easy plant operation and little maintenance;
- low construction and operation costs.

2. Kinds of Pond Systems

Three kinds of systems are applied in Germany:

- non-aerated pond systems (~ 135o * plants)

  in use for serving less than about 1ooo i + ie

- aerated pond systems (~5o* plants)

  applied for the sewage of about 1ooo to 1o ooo i+ie, especially when also food industries are connected or as an individual solution for those industries;

- ponds combined with trickling filters or rotating disc filters (~1oo* plants)

  solution for treating preferably domestic sewage of about 1ooo to 4ooo i.

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*) Number of plants in Bavaria; Bavaria has about 11 Mio inhabitants (≙ 18 % of the population of the Federal Republic of Germany) and covers about 7o ooo km² (≙ 28 %); large parts of Bavaria are in agricultural use.

3. German Effluent Standards

A special act of the German water law contains varying quality standards for sewage treatment plant effluents depending on plant seizes (tab. 1).

Table 1

Effluent quality standards according to the German water law

Group	Influent load BOD_5 (kg/d)	Corresponding plant seize (i+ie)	Effluent standards SS (ml/l)	BOD_5 (mg/l)	COD (mg/l)
I	6o	1ooo	o,3	3o/45	12o/18o
II	6o-6oo	1ooo-1o ooo	o,3	25/35	11o/16o
III	6oo	1o ooo	o,3	2o/3o	1oo/14o
				*) **)	*) **)

*) for 24-h-samples
**) for 2-h-samples

The average of 5 samples from the last 3 years must not exceed the effluent standards of table 1. The act also allows to set more stringent effluent values in an individual case, when it is necessary from the point of flow or quality conditions in the receiving water.

Non-aerated ponds are plants of group I in general;aerated ponds and ponds combined with trickling filters or rotating disc filters mainly belong to group II.

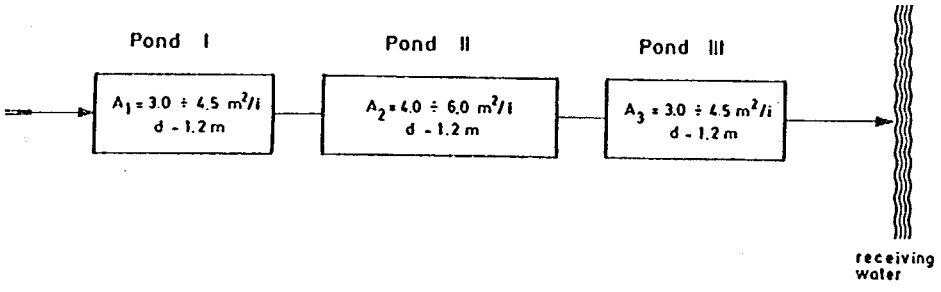
4. Non-aerated Ponds

Two different non-aerated pond systems are in use in Germany:

- Pond system "A" (applied in the northern part of Germany)

specific surface area totally required	1o - 15 m²/i
total area distributed among 3 ponds in series with a ratio of area	3 : 4 : 3
depth of ponds	about 1.2 m

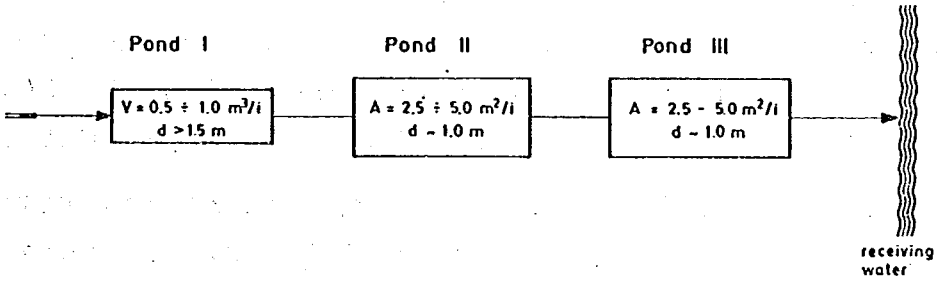
Fig. 1 A
Non aerated pond system "A"



- Pond system "B" (applied in the southern part of Germany, especially in Bavaria)

sedimentation pond as the first unit, specific volume required	o.5 - 1.o m³/i
depth	about 1.5 m
two ponds as the secound and third unit, specific surface area totally required	5 - 1o m²/i
depth	about 1.o m

Fig. 1B
Non aerated pond system "B"



It should be kept in mind that detention times are 6o - 1oo days in pond systems "A" resp. 3o - 7o days in pond systems "B" for dry weather flow rates (~ 15o - 2oo 1/i.d).

The following discussions refer to pond systems "B" as they are in operation in Bavaria. Effluent samples of more than 6o plants were evaluated. Figures 2,3,4,5 show the

results for COD and BOD_5 depending on specific pond surface areas. The curves in the diagrams represent the 9o %-probability of the results.

Conclusions:

- Below a specific pond aerea of 1.5 m²/i the results differ considerably; in that case sedimentation and facultative aerobic/anaerobic processes for biodegradation go on. Between 1.5 and 5.o m²/i an high reduction of the organic load occurs. From 5 to 1o m²/i there still is little improvement of the effluent values. Above 1o m²/i effluent values do not decrease.
- Because of the very long detention times and the extremely great egalization capacities of non-aerated pond systems random effluent samples show more or less equal results compared with 2-hour or 24-hour samples. Hence, in the specific case of non-aerated pond systems the results of random samples can be compared with the 24-h-standards of table 1 for group II: BOD_5 = 45 mg/l, COD = 18o mg/l.

  Fig. 2,3,4,5 show that the results of filtrated as well as non-filtrated samples fulfill the effluent standards with a great margin of safety, when pond systems are designed and operated with 5 to 1o m²/i.
- In figure 2 effluent results of shock loads of liquid manure and silage wastes are indicated. Influent values for COD of 2ooo mg/l are reduced to less than 2oo mg/l in non-aerated pond systems with about 1o m²/i. In cases where manure or silage wastes may enter occasionally the sewer pipes, ponds should be designed with a specific area of 1o m²/i. (This figure is suggested for ponds in moderate climates by many authors in the international literature.)

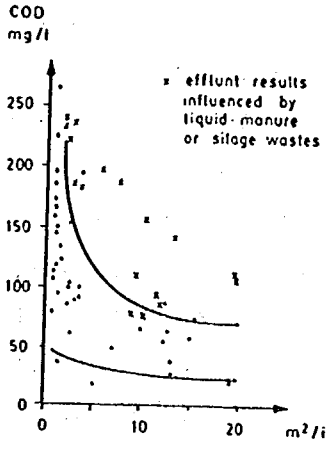


Fig. 2
COD-effluent results of non-aerated ponds depending on the specific pond area (averages from non-filtrated samples)

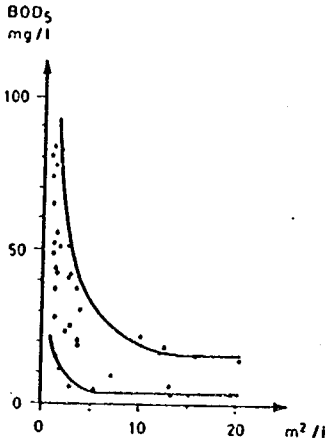


Fig. 3
BOD_5-effluent results of non-aerated ponds depending on the specific pond area (averages from non-filtrated samples)

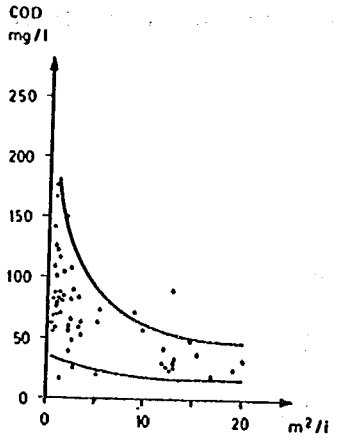


Fig. 4
COD-effluent results of non-aerated ponds depending on the specific pond area (averages from filtrated samples)

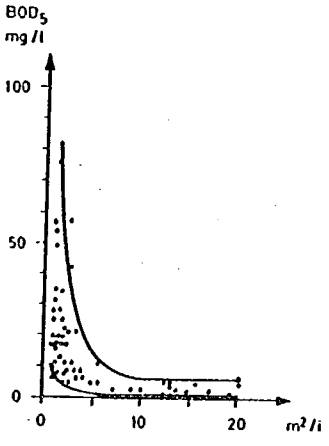


Fig 5
BOD_5-effluent results of non-aerated ponds depending on the specific pond area (averages from filtrated samples)

- A special evaluation of the effluent values for storm flow conditions from combined sewer networks show, that hydraulic as well as organic shock loads are equalized very efficiently by the large area and volume of those pond systems. Run off from roads and places of rural villages may have COD-concentrations of more than 1ooo mg/l and BOD_5-concentrations of more than 5oo mg/l for a short time after beginning of rainstorms. It was found, that pond systems with a specific area ≥ 5 m²/i and with some storage volume and with suitable inlet and outlet constructions can be loaded with 4o DWF (dry weather flow) during rainy periods Under these conditions COD and BOD_5 effluent results are of the same order for dry weather flow as well as for storm flow. Pond systems "A" can be loaded even with higher flow rates. Treating storm flow rates of that magnitude without any problem is a foundamental advantage of pond systems compared to technical sewage treatment plants, which are able to treat only 2 - 4 DWF under suitable conditions.

- Fig. 6 shows the reduction of ammonia in non-aerated pond systems. Specific surface areas up to 15 m²/i give considerable NH_4-reductions. The average efficiency for nitrification during one year amounts to about .o.5 g/m².d. Nitrification and denitrification take place simultanously and

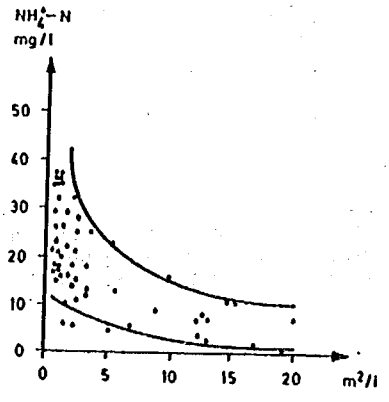


Fig. 6
NH_4-N-effluent results (averages) of non-aerated ponds depending on the specific pond area

successively. Pond effluents have rather low nitrate concentrations, since the nitrogen escapes from the system into the atmosphere.

- Figures 7,8,9 give some information on the efficiency of non-aerated pond systems with specific areas > 5 m²/i under winter conditions. The histograms for COD, BOD_5 and NH_4 show very similar distributions of effluent concentration frequencies in the summer (water temperature: average 17 °C, max. 25 °C, min. 1o °C) as well as in the winter (water temperature: average 3.5 °C, max. 17 °C, min. o.5 °C) and in periodes with an ice layer on pond surfaces. Figures show that the relative frequency of minimum effluent concentrations sligthly decreases and that the relative frequency of the maximum effluent values slightly increases under winter conditions. But, these tendencies are insignificant in comparison to the overall effluent values as well as to the effluent standards of table 1.

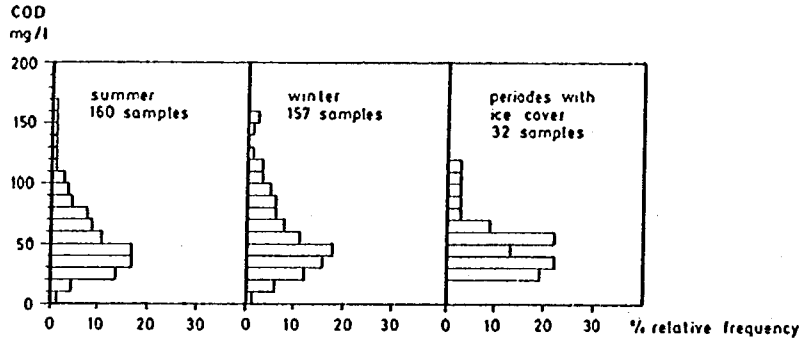


Fig. 7

Relative frequency of COD-effluent results of non-aerated ponds >5 m²/i for different seasons (non-filtrated samples)

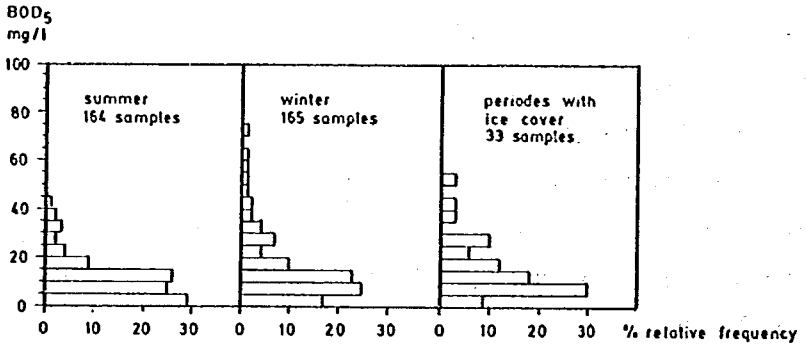


Fig. 8

Relative frequency of BOD_5-effluent results of non-aerated ponds >5 m²/i for different seasons (non-filtrated samples)

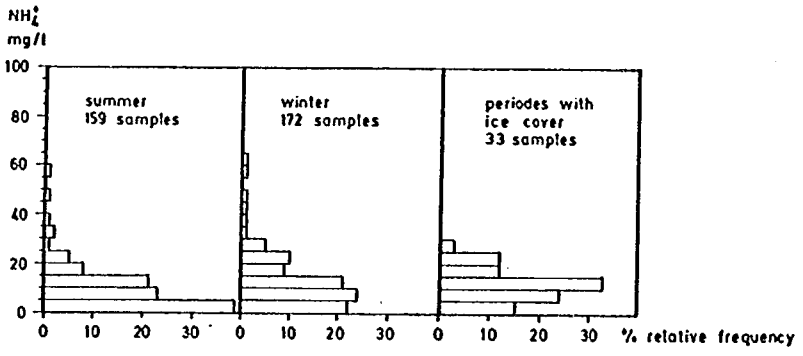


Fig. 9

Relative frequency of NH_4^+-effluent results of non-aerated ponds >5 m²/i for different seasons (non-filtrated samples)

Summarizing it can be said, that German effluent quality standards can be fulfilled by non-aerated ponds with a specific surface area > 5 m²/i under dry weather flow- as well as under storm flow- and under winter conditions; With a specific surface area of about 1o m²/i even unfavourable situations (for example short time shock loads caused by liquid manure or silage wastes) can be solved without great difficulties. Sewage treatment in non-aerated ponds should not be considered as an old fashioned solution. In fact, it has proved to be a very favourable solution for solving sewage treatment problems in rural communities.

Odour nuisance is prevented by removing the sludge settled in the ponds before its surface has grown up near to or above the minimum water level. The sludge surface should be covered by a water layer of at least 3o cm.

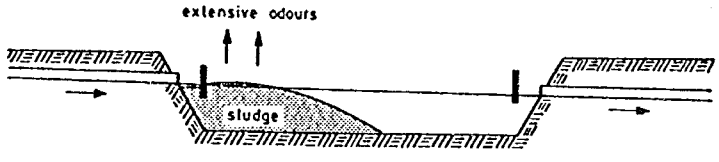


Fig. 10 a
Cross section through a not well operated pond

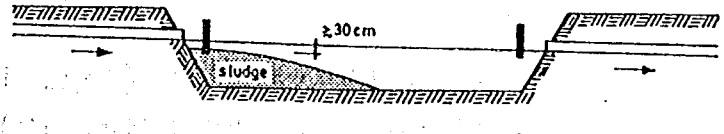


Fig. 10 b
Sludge should be removed from the pond

The greatest disadvantage of non-aerated pond systems is the large area needed. For that reason we were looking for solutions, which include the advantages of non-aerated pond systems and prevent the disadvantages. Alternatives are: aerated pond systems or ponds combined with trickling filters or rotating disc filters.

5. Aerated Ponds

Aerated ponds are designed with a specific volume of 2 - 3 m³/ie; in general the total volume is distributed among two ponds in series. Behind the aerated ponds there is a sedimentation pond with a specific volume of 0.5 - 1.0 m³/ie.

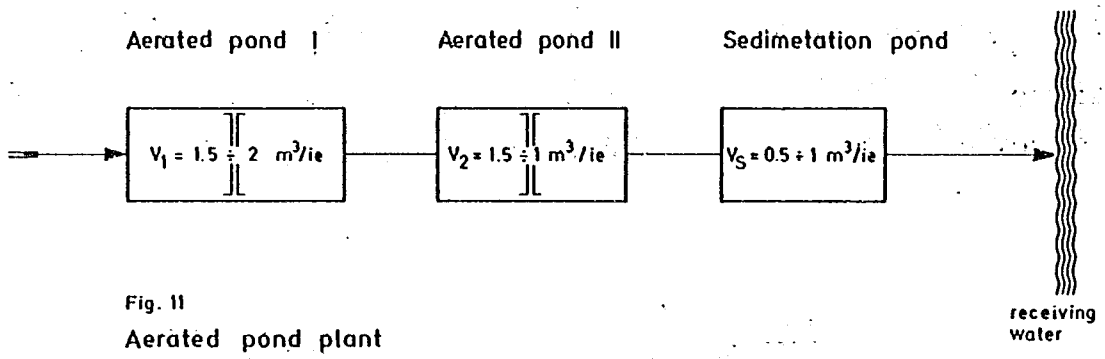


Fig. 11
Aerated pond plant

The necessery oxigen input amounts to 1.0 - 1.5 kg O_2/kg BOD_5. When aeration devices are well adapted to the size and the shape of the ponds an energy capacity of 1 Watt per m³ pond volume is sufficient for simultaneous turbulence performance. In Germany four different types of special devices for pond aeration are in use:

- Fixed air bubble aeration "Universal" (fig. 11)
- Moving air bubble aeration "Biolak" (fig. 12)
- Submerged aerator units (fig.13)

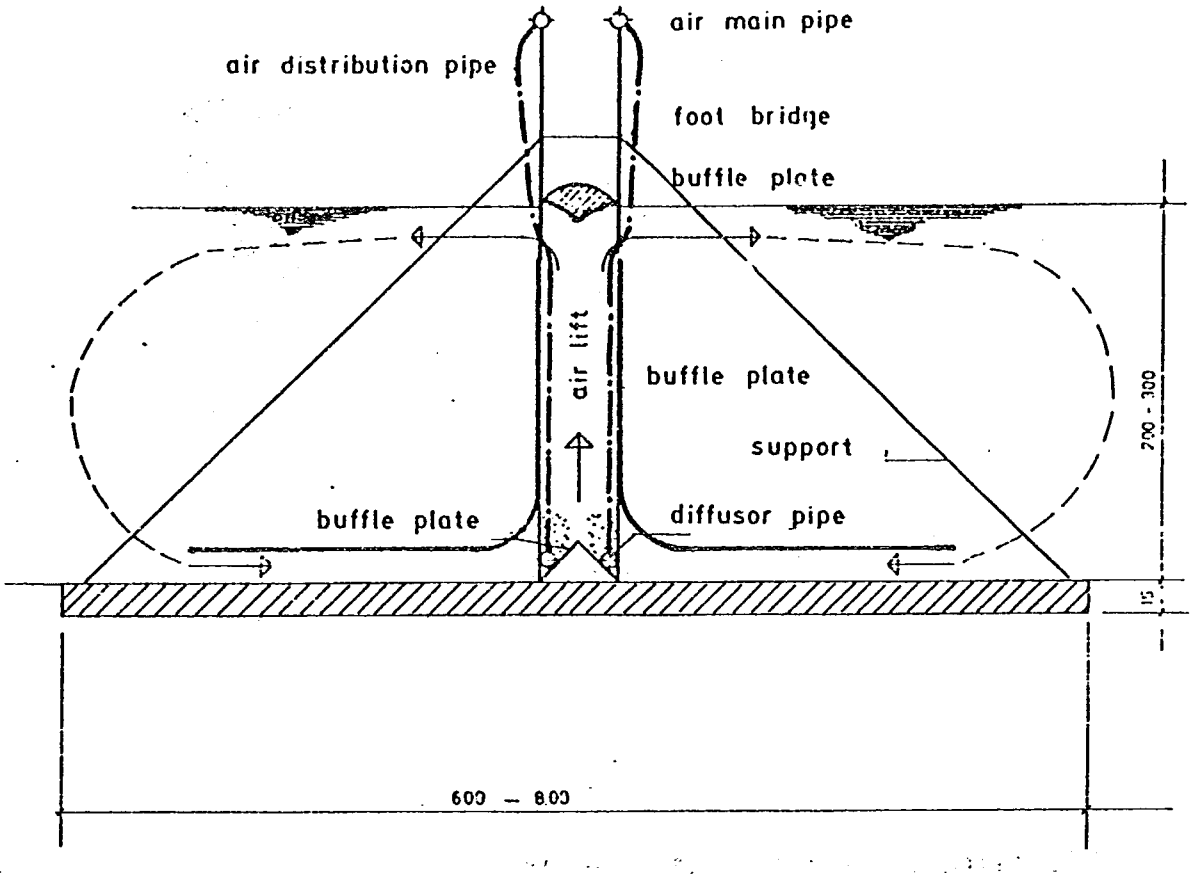


Fig. 11

Fixed air bubble aeration "Universal" (cross section)

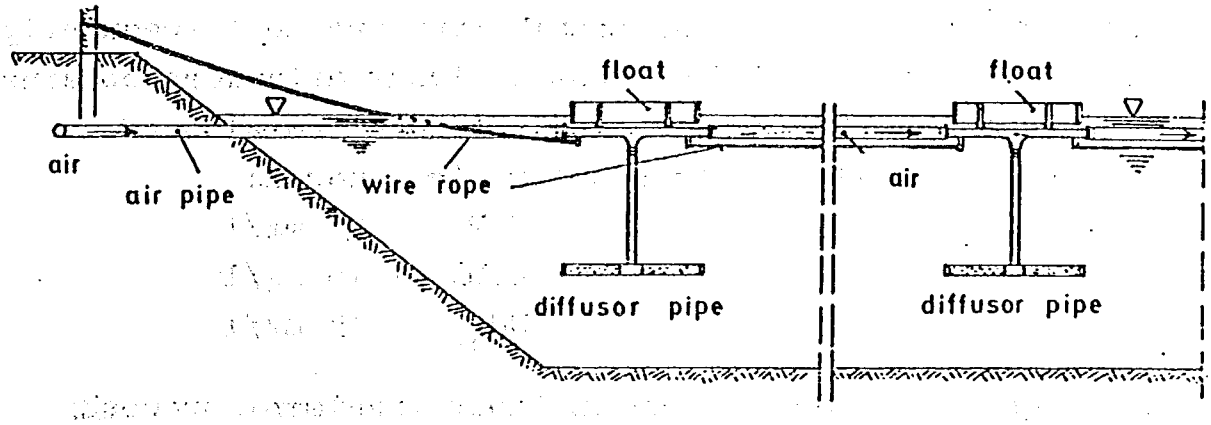


Fig. 12

Moving air bubble aeration "Biolak"

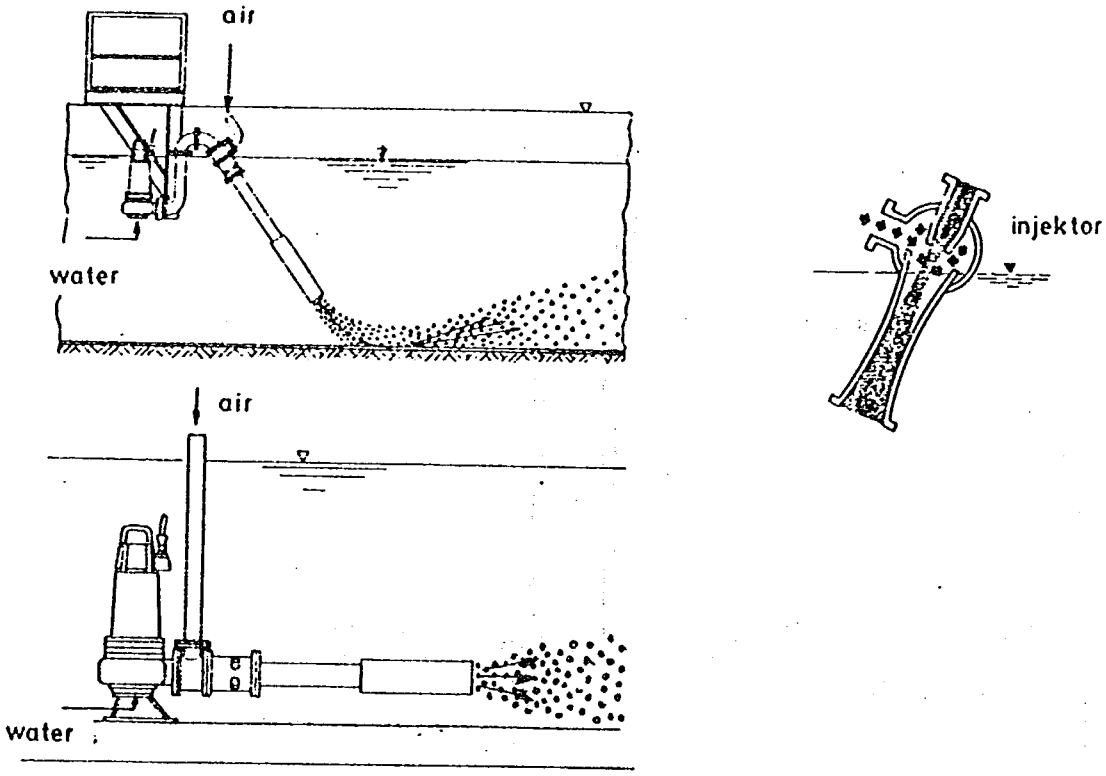


Fig. 13
Submerged aerator units (type EMU)

Aerated ponds of the "Universal" type are most frequently in use in Germany. Hence, most effluent data were obtained from that specific pond system. Effluent results of "Universal" ponds in Bavaria show in general:

COD	< 90 mg/l
BOD_5	< 15 mg/l
NH_4	< 15 mg/l

All plants are very stable in their treatment process.

BÖHNKE found very similar effluent results of "Universal" ponds which were figured out statistically (see table 2).

Table 2

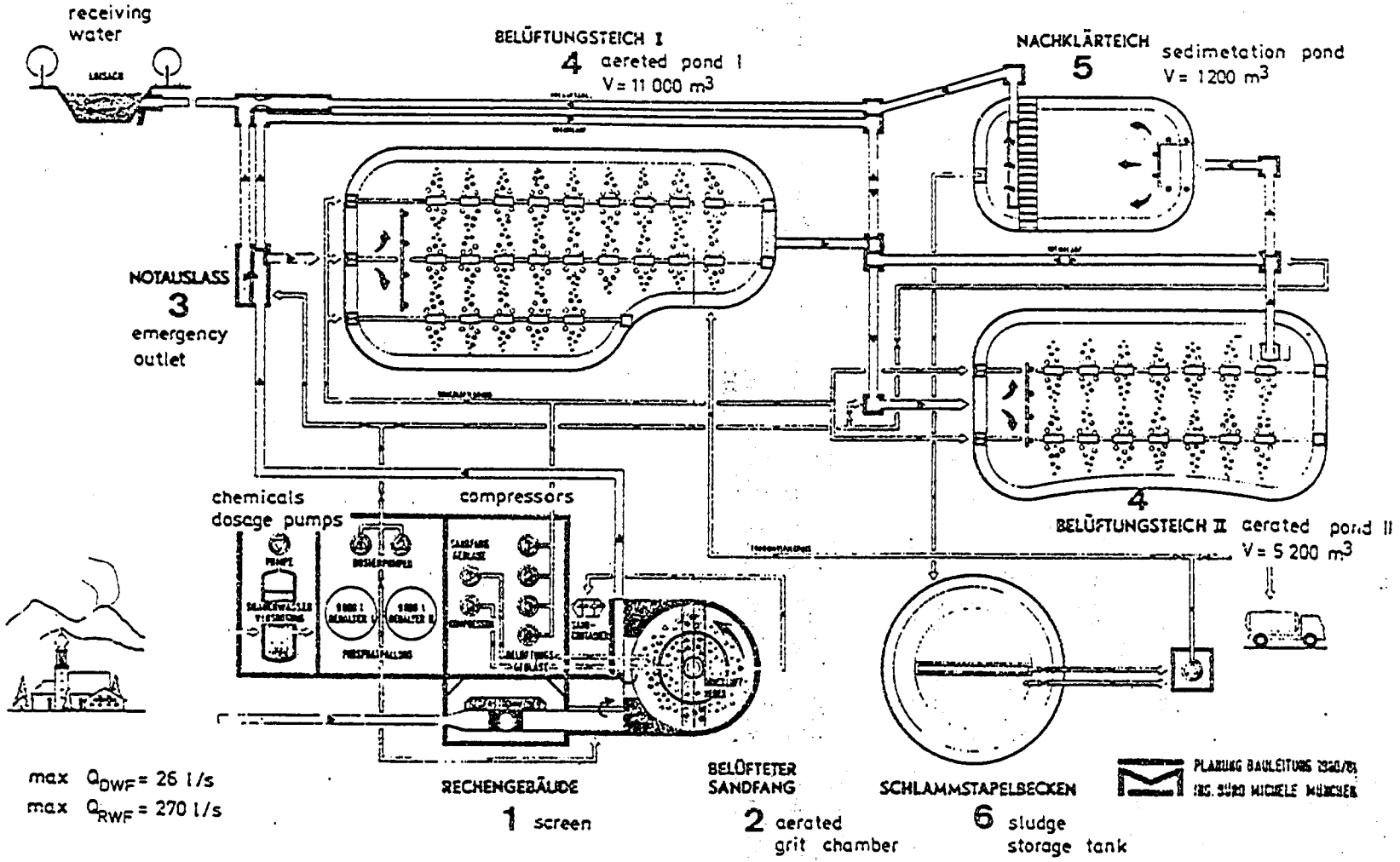
Effluent results of 25 aerated "Universal" ponds (after BÖHNKE)

Effluent	Time of sampling	Average $\bar{x}$ (mg/l)	Deviation σ from average (mg/l)	Deviation rate ν (%)	Number of samples
BOD_5	over the year	9,3	5,8	62,2	84
	winter time (Nov.-March)	9,4	5,6	59,6	44
COD	over the year	64,3	28,6	44,4	49
	winter time (Nov.-March)	62,3	25,9	42,o	27

Chemical treatment can also be integrated in aerated pond systems. Figure 14 shows a plant of the "Biolak" type combined with chemical treatment.

Fig. 14

TREATMENT PLANT OF THE COMMUNITY OHLSTADT 5000 i+ie



6. Ponds combined with Trickling Filters

Plants of this type are in use mainly in Bavaria; their flow-scheme is shown in figure 15.

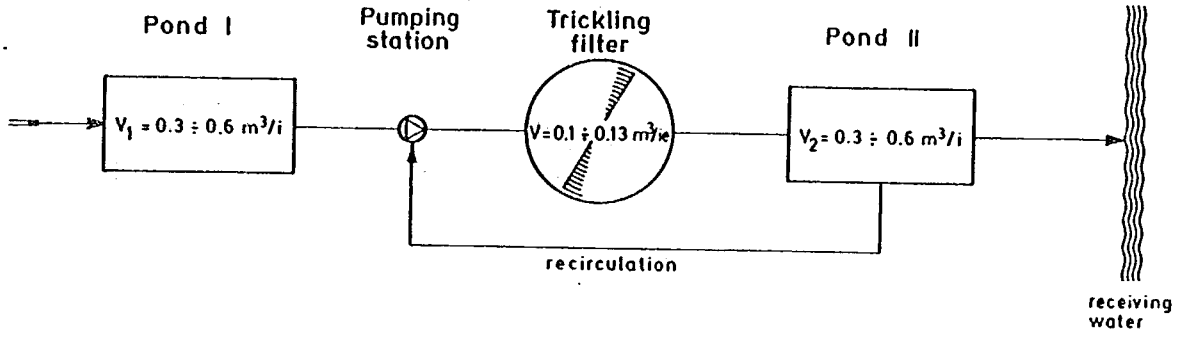


Fig. 15
Ponds combined with a trickling filter

The sedimentation pond is designed with a specific volume of o.3 - o.6 m³/ie. In general the final sedimentation pond has about the same size as the primary pond. The volume of trickling filter is determined by a specific load of 3oo - 4oo g BOD_5/m³.d; that corresponds to about 7.5 - 1o ie/m³ respectively o.13 - o.1 m³/ie.

Effluent results from 15 plants (loaded with < 8o % of the design load):

COD	< 1oo mg/l
BOD_5	< 25 mg/l
NH_4	< 2o mg/l

7. Ponds combined with Rotating Disc Filters

The flow scheme of this type of plant is shown in figure 16. Sometimes a filter unit is placed between the rotating disc filter unit and the final pond in order to separate the solids from the biodisc effluent.

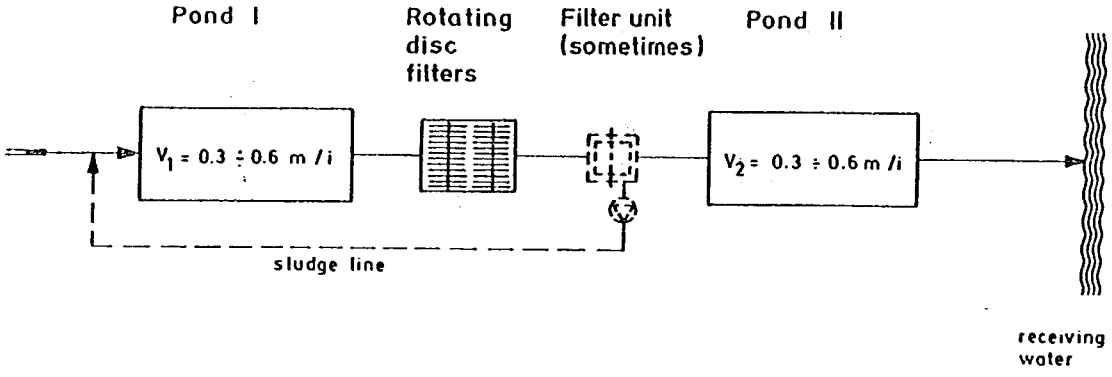


Fig. 16
Ponds combined with rotating disc filters

The sedimentation pond is designed with a specific volume of o.3 - o.6 m³/ie. In general the final pond has the same size as the primary pond. The area of the rotating discs is determined by a specific load of about 1o g BOD_5/m².d; that corresponds to about 4 m²/ie.

Effluent results from 1o plants (loaded with <65 % of the design load):

COD	< 7o mg/l
BOD_5	< 15 mg/l
NH_4	< 15 mg/l

The specific energy consumption (kWh/ie.year) of this type of plant may become extremely high because the house has to be heated where the bio-discs are placed.

8. Summary

Non-aerated pond systems as well as aerated ponds and ponds combined with trickling filters or rotating disc filters designed according to the above given figures are able to fulfill the German effluent quality standards with a great margin of safety. In most cases actual effluent results are considerably below these standards. All kind of pond systems show some ammonia oxidation and a partial denitrification.

In table 3 all kinds of pond systems are compared with each other.

Table 3

Comparison of pond systems

		Non-aerated pond systems	Aerated ponds	Ponds combined with trickling filters or rotating disc filters
Consumption of area	[m²/ie]	8-16	2-4	1.5-3
Energy consumption	[kWh/ie.y]	0	12-24	6-18
Time required for maintenance	[h/week]	2-3	3-6	4-8
Construction costs	[DM/ie]			
for plant seize	500 ie	400-600	-	-
	1000 ie	300-500	400-600	400-600
	5000 ie	-	200-300	200-300
Annual costs	[DM/ie.y]			
for plant seize	500 ie	25-35	-	-
	1000 ie	20-30	30-40	30-40
	5000 ie	-	14-20	14-20

STUDY OF ROTATING BIOLOGICAL CONTACTOR PROCESS AT WASTEWATER TREATMENT PLANTS FOR SMALL COMMUNITIES

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Kazuhiro Tanaka, Section Chief
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Japan Sewage Works Agency

1. Introduction

It has not been long since the Rotating Biological Contactor process (RBC process) was introduced into Japan. In April 1979, the Ryutsu-Center Treatment Plant in Yamagata City became the first plant incorporating the RBC process in the public sewage systems in Japan. By 1981, 12 sewage treatment plants for small communities using the RBC process started to work.

Field investigations were conducted at three of these plants to study the RBC process performance. Figs. 1-1 to 1-3 show schematic flow diagrams for the three plants. Design data for the plants is outlined in Table 1-1.

As shown in Figs. 1-1 to 1-3, the typical sewage treatment plant in Japan using the RBC process for secondary treatment is composed of grit chambers, primary settling tanks, RBC units, final settling tanks and chlorination tanks. The RBC process is usually used as four stage system and, unlike the activated sludge process, does not need sludge recycling.

To determine the daily average performance, among other things, composite samples of RBC influent and final settling tank effluent were monitored.

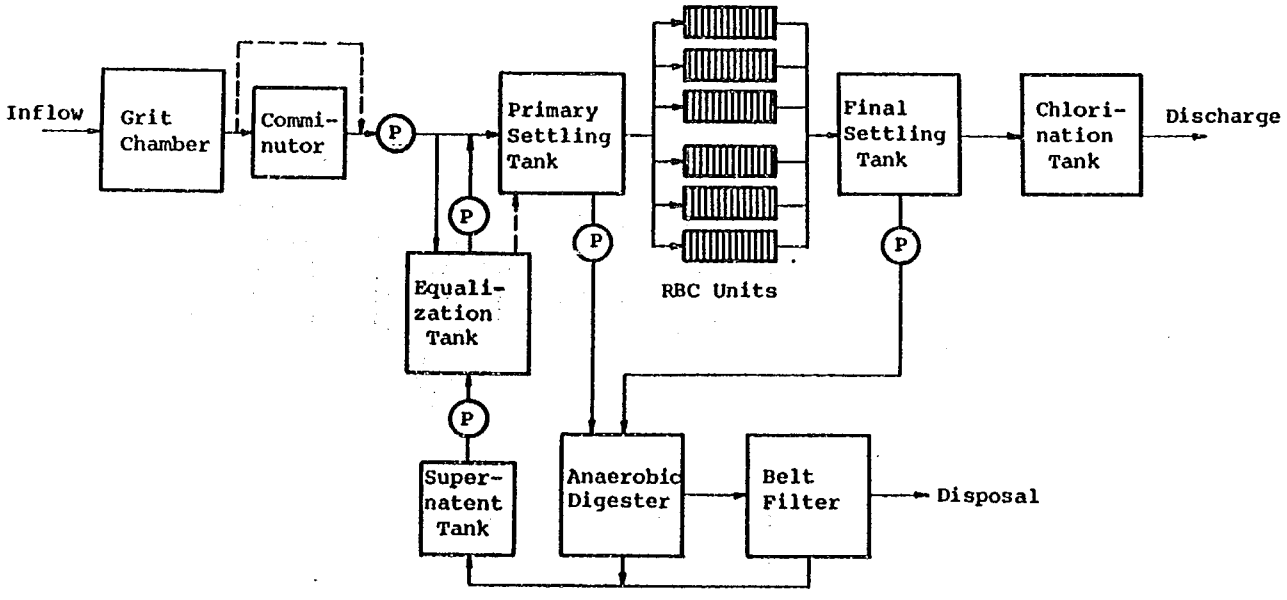


Fig. 1-1 Schematic Flow Diagram of "A" Treatment Plant

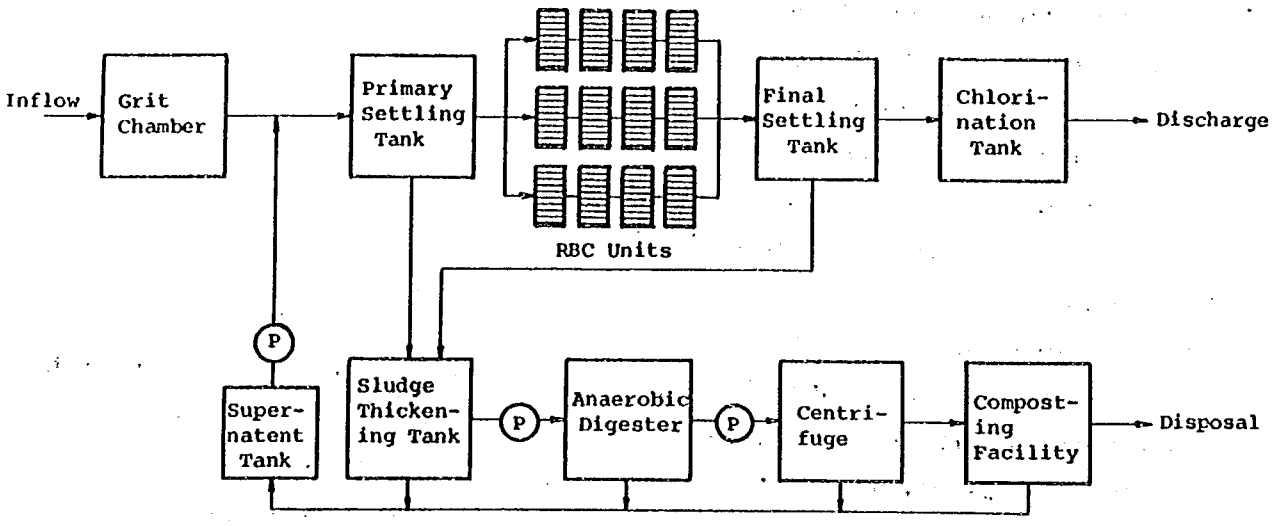


Fig. 1-2 Schematic Flow Diagram of "B" Treatment Plant

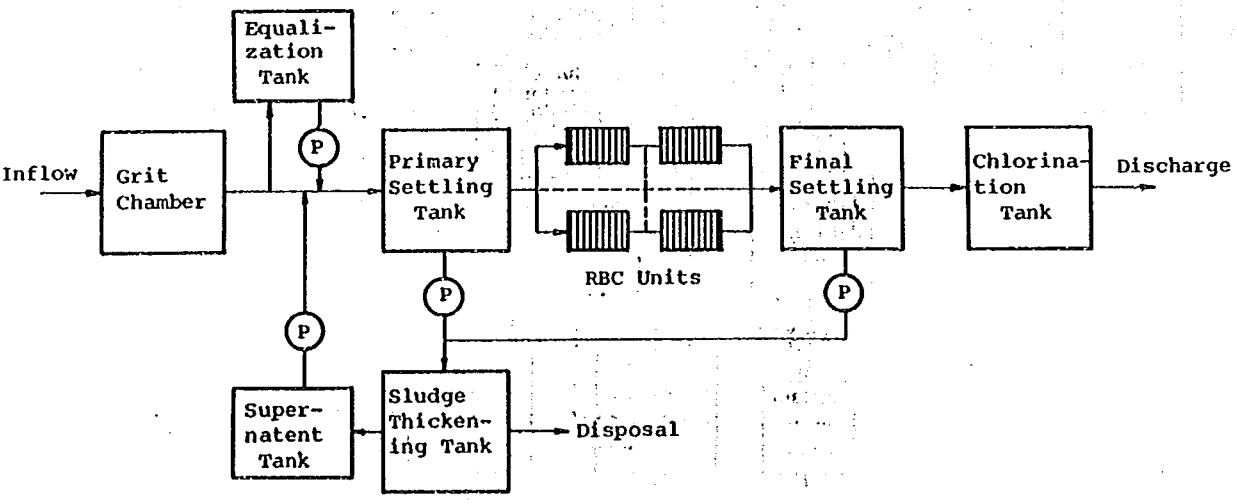


Fig. 1-3 Schematic Flow Diagram of "C" Treatment Plant

Table 1-1 Outline of Design Data

		A	B	C
Maximum Daily Design Flow (m^3/d)		1,000	6,700	1,150
Design Quality (mg/ℓ)	Influent BOD	140	180	190
	Influent SS	130	200	170
	Effluent BOD	15	18	20
	Effluent SS	23	30	20
Primary Settling Tank	Number of Tank	1	2	2
	Overflow Rate ($m^3/m^2.d$)	35	34	25
	Settling Time (hr.)	1.5	2.1	2.4
Rotating Biological Contactor	Hydraulic Loading ($\ell/m^2.d$)	42	57	72
	BOD Loading ($g/m^2.d$)	4.1	7.1	9.6
	Detention Time (hr.)	3.6	2.1	2.2
	Volume to Surface Ratio (ℓ/m^2)	6.2	4.9	6.4
	Flow Paths	Parallel	Perpendicular	Parallel
	Number of Unit	6	3	2
	Number of Stage in Each Unit	4	4	4(2)
	Number of Shaft	1 x 6 = 6	4 x 3 = 12	2 x 2 = 4
	Rotational Velocity (RPM)	Variable 0.40 - 1.59	Variable 0.51 - 1.76	Variable 1.0,1.5,1.8
	Media Diameter (m)	3.6	3.6	3.2
	Spacing between the Sheets of Media (mm)	20	30	20
	Thickeness of Media Sheet (mm)	2.0	1.4	1.0
	Total Effective Surface Area of Media (m^2)	3,983 x 6 = 23,900	9,850 x 12 = 118,200	4,000 x 4 = 16,000
Final Settling Tank	Number of Tank	2	2	2
	Overflow Rate ($m^3/m^2.d$)	25	24	20
	Settling Time (hr.)	1.9	2.9	3.0

2. OPERATING CONDITIONS DURING THE INVESTIGATION

Monthly average data on RBC inflow, hydraulic loading*, BOD loading** and final settling tank overflow rate for each plant are shown in Tables 2-1 to 2-3.

As shown in Fig. 2-1, DO in the RBC reactor revealed wide variations in each stage. This is because DO in the reactor reflects influent DO, reaeration by media rotation and biomass oxygen uptake. Insufficient reaeration or heavy loading will result in dissolved oxygen limitations in the initial stages of the RBC systems and reduce substrate removal rates.

Then, the data gathered when the reactors were kept in aerobic state and rotational velocity of media was in the range of 1.0 to 1.6 RPM were used for study.

* Hydraulic loading: inflow per unit effective surface area of media. The hydraulic loading is defined by following equation:

$$H = 1{,}000 \frac{Q}{A}$$

Where H = hydraulic loading (ℓ/m^2.day)
Q = inflow (m^3/m^2.day)
A = total effective surface area of media (m^2)

Detention time is in inverse proportion to hydraulic loading. Accordingly, if the volume-to-surface ratio of the reactor is constant, the relationship between process performance and hydraulic loading parallels that between process performance and detention time.

** BOD loading: BOD loaded per unit effective surface area of media per unit time. BOD loading is defined by following equation:

$$L = \frac{CQ}{A} = \frac{1}{1{,}000} CH$$

Where L = BOD loading (g/m^2 day)
C = RBC influent BOD concentration (mg/ℓ)

Soluble BOD loading can be determined by substituting RBC influent soluble BOD concentration into C in the above equation.

Table 2-1 Operating Conditions during Investigation Period in "A" Treatment Plant

		Inflow (m^3/d)			Hydraulic Loading (/m^2.d)			BOD Loading (g/m^2.d)			Overflow Rate of Final Settling Tank (m^3/m^2.d)			Number of Units in Operation	
		avg.	Max.	Min.	avg.	Max.	Min.	avg.	Max.	Min.	avg.	Max.	Min.	RBC	Final settling tank
1979	Aug. 19 to Sep. 14	512	914	336	64.3	114.7	42.2	2.5	3.7	1.5	12.6	22.6	8.3	2	2
	Sep. 15 to Oct. 28	531	1,014	339	44.4	84.9	28.4	1.8	3.0	1.2	13.1	25.0	8.4	3	2
	Oct. 29 to Nov. 24	572	883	357	47.9	73.9	29.9	2.3	4.1	0.5	14.1	21.8	8.8	3	2
	Nov. 25 to Dec. 23	578	835	407	48.4	69.9	34.1	2.9	5.1	0.8	14.3	20.6	10.0	3	2
1980	Dec. 24 to Feb. 2	504	802	311	42.2	67.1	26.0	1.3	1.8	0.6	12.4	19.8	7.7	3	2
	Feb. 3 to Feb. 29	562	732	446	70.5	91.9	56.0	2.0	3.1	1.0	13.9	18.1	11.0	2	2

Table 2-2 Operating Conditions during Investigation Period in "B" Treatment Plant

		Inflow (m^3/d)			Hydraulic Loading (/m^2.d)			BOD Loading (g/m^2.d)			Overflow Rate of Final Settling Tank (m^3/m^2.d)			Number of Units in Operation	
		avg.	Max.	Min.	avg.	Max.	Min.	avg.	Max.	Min.	avg.	Max.	Min.	RBC	Final settling tank
1980	Apr. 10 to 27	1880	3050	1390	23.9	38.5	17.6	-	-	-	7.2	11.1	5.1	2	2
	Apr. 28 to May 31	1570	2140	1260	39.9	54.3	29.5	2.9	5.7	1.5	11.5	15.5	9.6	1	1
	Jun. 1 to 30	1680	1940	1450	42.6	49.2	36.7	3.1	4.3	2.0	12.3	14.2	10.6	1	1
	Jul. 1 to 31	1920	2210	1600	48.8	56.2	40.7	4.2	5.6	3.1	14.0	16.1	11.7	1	1
	Aug. 1 to 31	2040	2800	1640	51.9	71.1	41.6	4.7	6.1	2.7	14.9	20.4	12.0	1	1
	Sep. 1 to 30	1660	2420	1360	42.2	61.4	34.5	3.9	6.0	2.7	12.1	17.7	9.9	1	1
	Oct. 1 to 31	1690	2120	1440	42.8	53.7	36.5	3.3	5.3	2.0	12.3	15.5	10.5	1	1
	Nov. 1 to 30	1660	2100	1410	42.2	49.6	34.3	2.8	3.3	2.1	12.1	15.3	10.3	1	1
	Dec. 1 to 31	2200	3390	1390	55.9	86.0	35.3	-	-	-	14.9	22.0	10.1	1	1
1981	Jan. 1 to 31	4310	6220	3440	54.7	78.9	43.7	7.2	12.7	4.6	15.5	22.4	10.9	2	2
	Feb. 1 to 28	3740	4250	3360	47.5	53.9	42.6	4.7	6.1	3.8	13.6	15.5	12.3	2	2
	Mar. 1 to 31	3500	4540	2730	43.1	57.6	34.7	4.9	8.2	3.0	12.8	16.6	10.0	2	2

Table 2-3 Operating Conditions during Investigation Period in "C" Treatment Plant

		Inflow (m^3/d)			Hydraulic Loading ($\ell/m^2.d$)			BOD Loading ($g/m^2.d$)			Overflow Rate of Final Settling Tank ($m^3/m^2.d$)			Number of Units in Operation	
		avg.	Max.	Min.	avg.	Max.	Min.	avg.	Max.	Min.	avg.	Max.	Min.	RBC	Final settling tank
1980	May 1 to 31	390	500	270	24.3	31.3	16.6	3.0	3.9	1.7	6.7	8.6	4.6	2	2
	Jun. 1 to 30	480	710	320	29.7	44.1	19.9	3.3	4.3	2.7	8.2	12.2	5.5	2	2
	Jul. 1 to Aug. 4	620	870	440	38.9	54.4	27.6	-	-	-	11.0	14.9	7.5	2	2
	Aug. 5 to 31	510	880	350	63.2	111.0	43.6	4.4	5.9	3.2	8.6	15.3	6.0	1	2
	Sep. 1 to 30	600	780	480	74.9	98.0	60.3	10.6	14.7	8.7	10.3	13.4	8.2	1	2
	Oct. 1 to 31	550	660	420	69.0	82.1	52.6	8.9	10.9	5.9	9.4	11.3	7.2	1	2
1981	Feb. 25 to Mar. 4	740	970	630	92.9	121.0	79.3	4.6	5.2	3.1	12.7	16.6	10.8	1	2

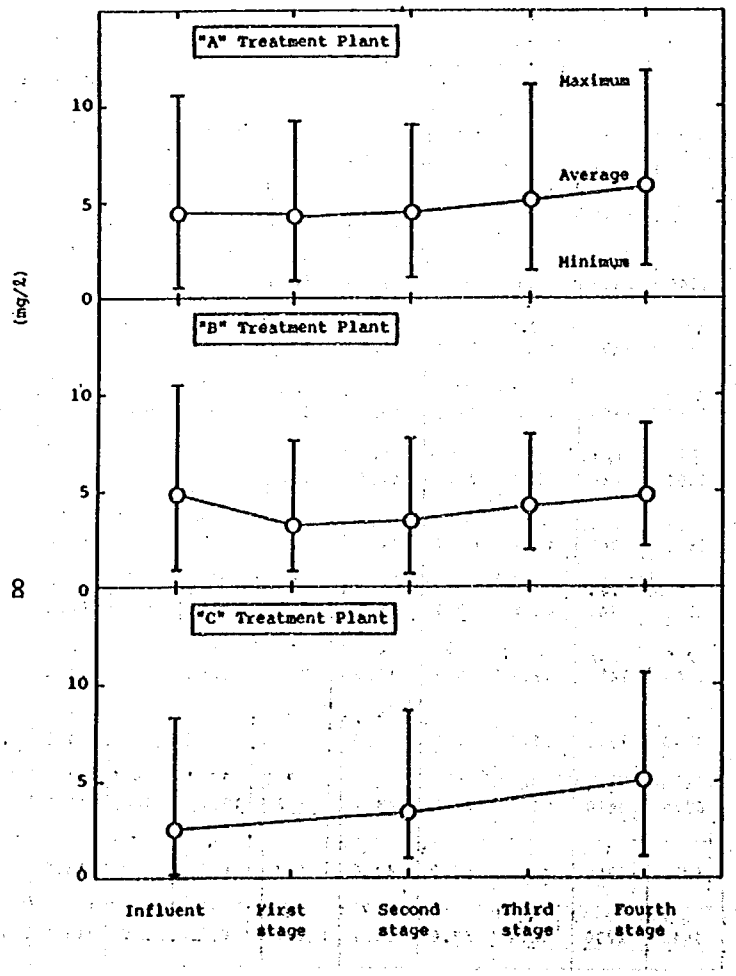


Fig. 2-1 Interstage DO Profiles

3. PROCESS PERFORMANCE

3.1 BOD REMOVAL

3.1.1 Effect of Loadings on BOD Removal

BOD loading is one design parameter associated with RBC process performance. As shown in Fig. 3-1, BOD loading effects effluent BOD as the latter tends to decline with decreases in BOD loading. Since the data cover a relatively wide range at each specific level of BOD loading, some of them exceed a daily average BOD concentration of 20 mg/ℓ, the secondary effluent standard in Japan.

Hydraulic loading is another design parameter. Fig. 3-2 shows the same data as Fig. 3-1, but gives them in temrs of the relationship between hydraulic loading and effluent BOD. When RBC influent BOD is up to 100 mg/ℓ, the effect of hydraulic loading on effluent BOD is not clear if hydraulic loading is up to 120 ℓ/m^2 day. When RBC influent BOD is in the range of 100 to 200 mg/ℓ, effluent BOD tends to increase with increases in hydraulic loading. The data cover a wide range at each specific level of hydraulic loading, and there are some which exceed 20 mg/ℓ in effluent BOD even when hydraulic loading is low.

The effect of BOD loading on BOD removal efficiency is given in Fig. 3-3, and the effect of hydraulic loading on BOD removal efficiency is shown in Fig. 3-4. It was found that BOD removal efficiency is 75 to 90% when BOD loading is in the range of 5 to 10 g/m^2.day. When RBC influent BOD is 100 to 200 mg/ℓ, BOD removal efficiency is 75 to 95% at a hydraulic loading of 50 ℓ/m^2.day and 60 to 75% at a hydraulic loading of 100 ℓ/m^2.day.

The effect of water temperature on the relationship between the loadings and process performance is not clear.

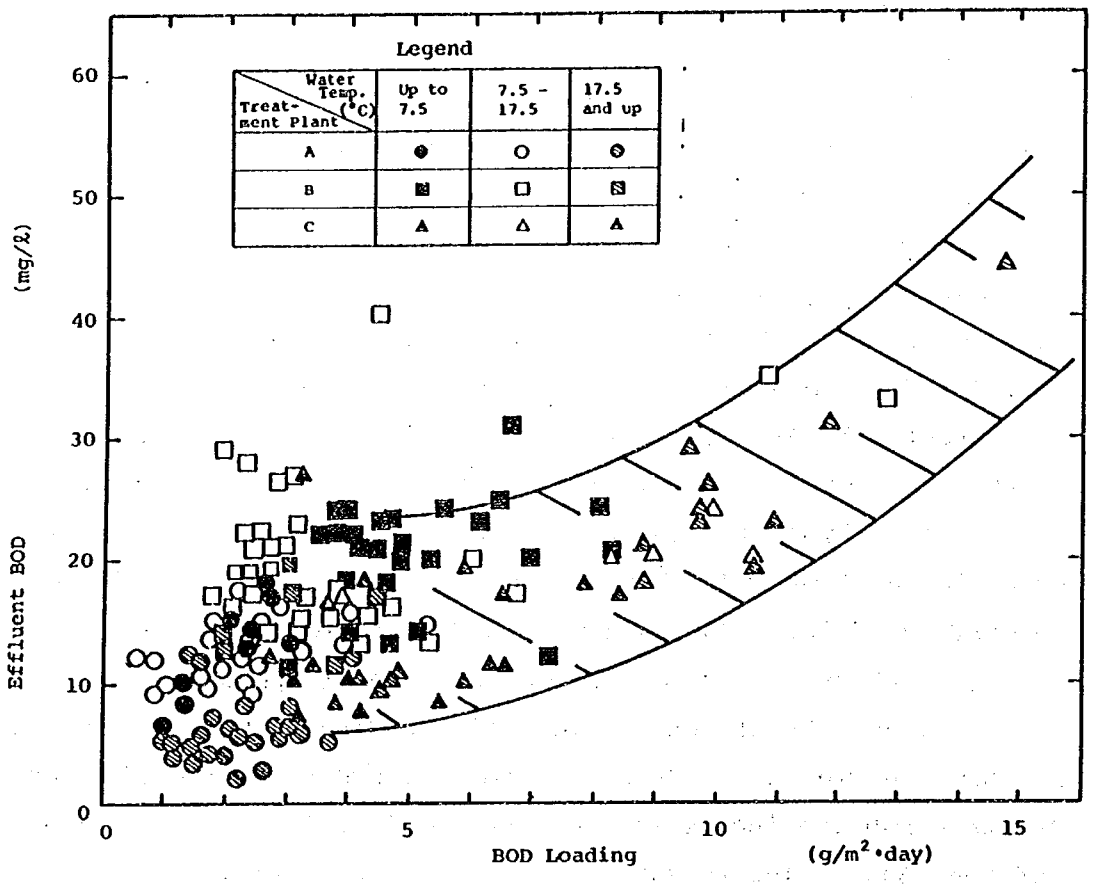
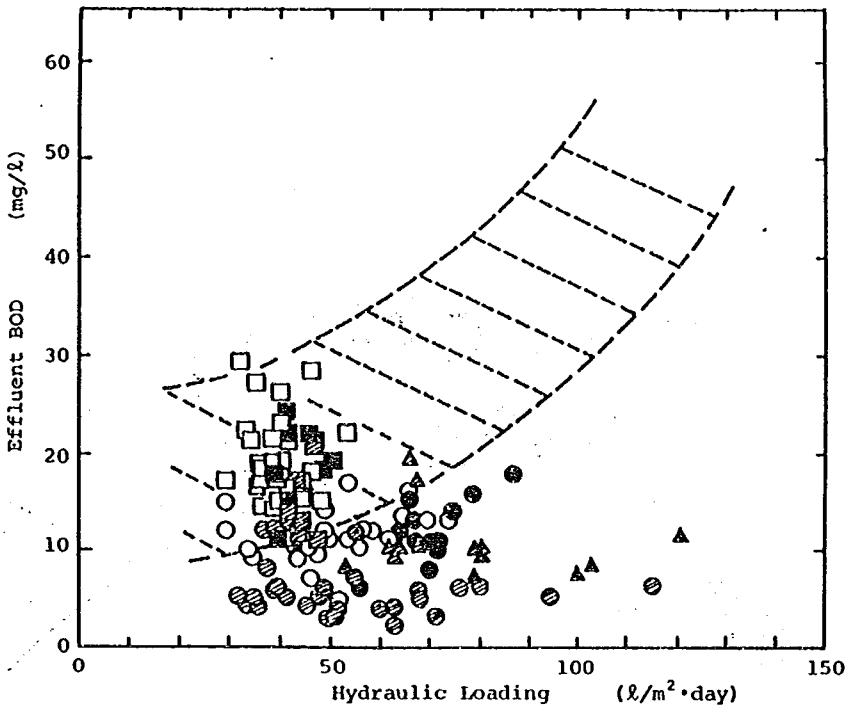
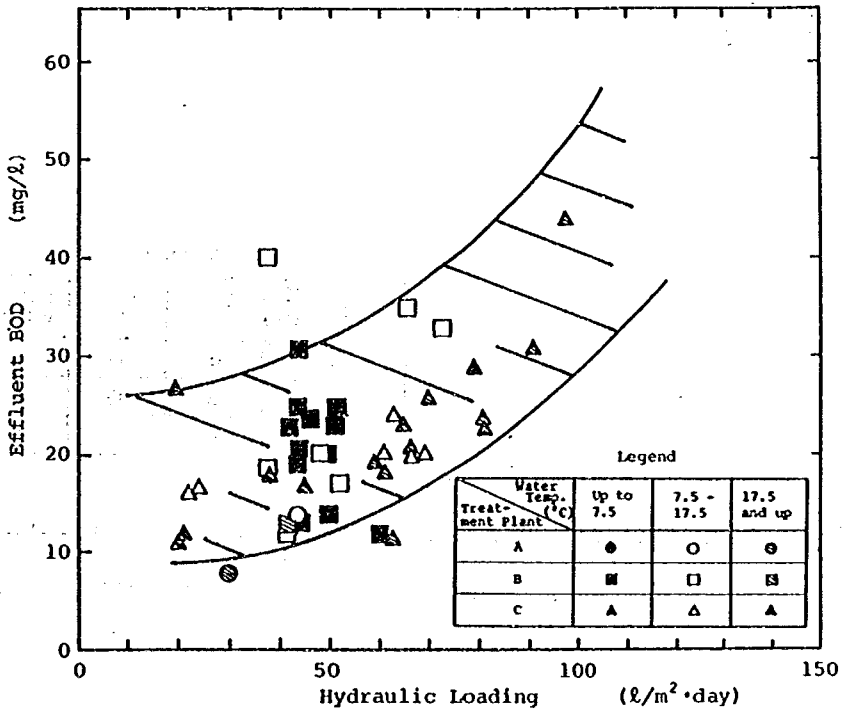


Fig. 3-1 Effect of BOD Loading on Effluent BOD



(1) RBC Influent BOD Concentration, up to 100 mg/ℓ



(2) RBC Influent BOD Concentration, 100 mg/ℓ to 200 mg/ℓ

Fig. 3-2 Effect of Hydraulic Loading on Effluent BOD

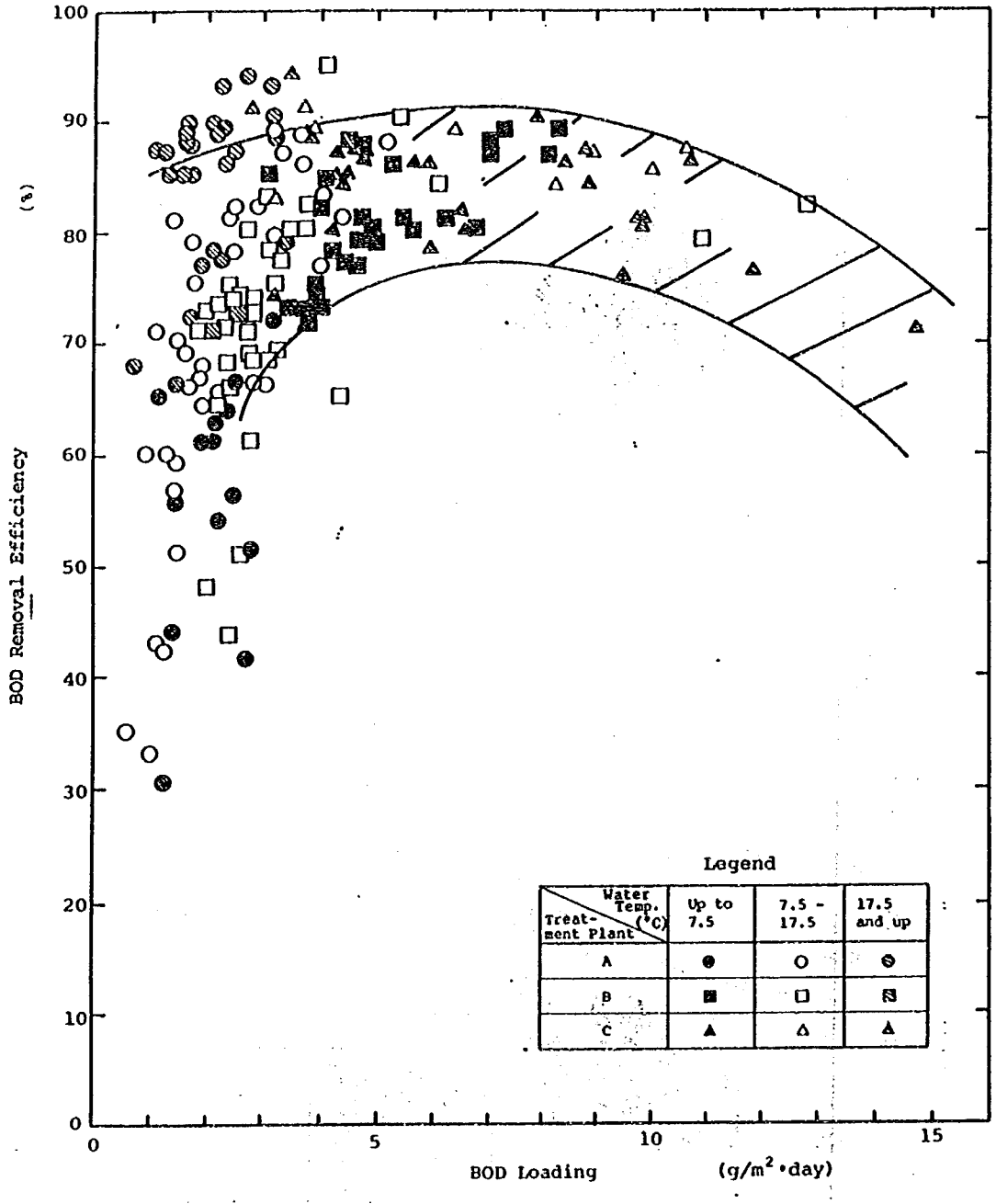
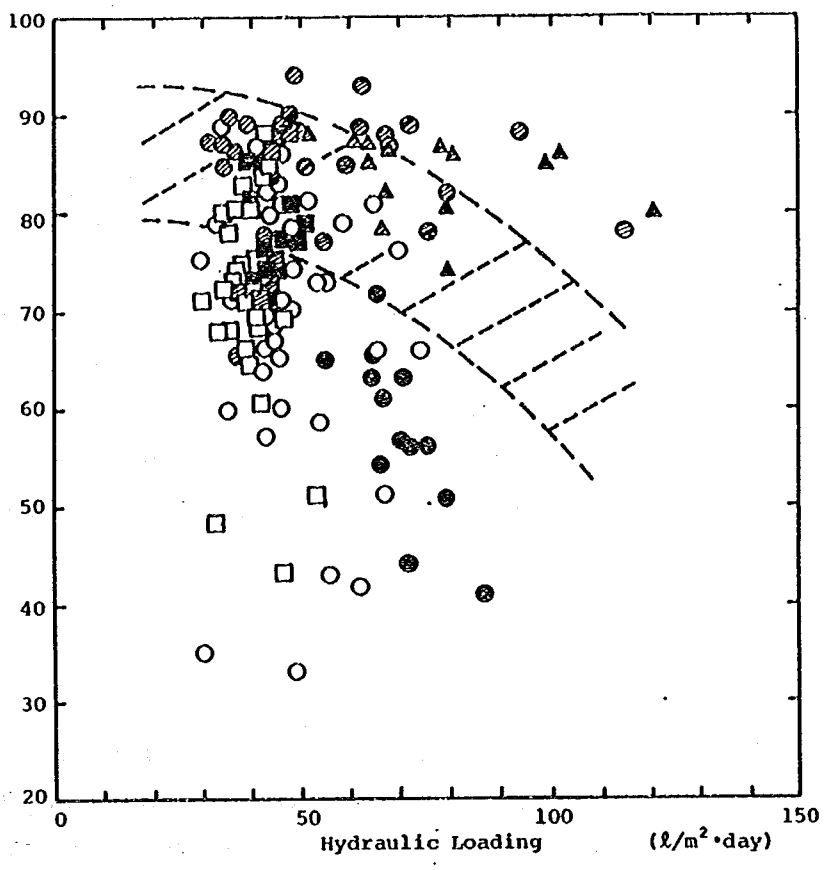
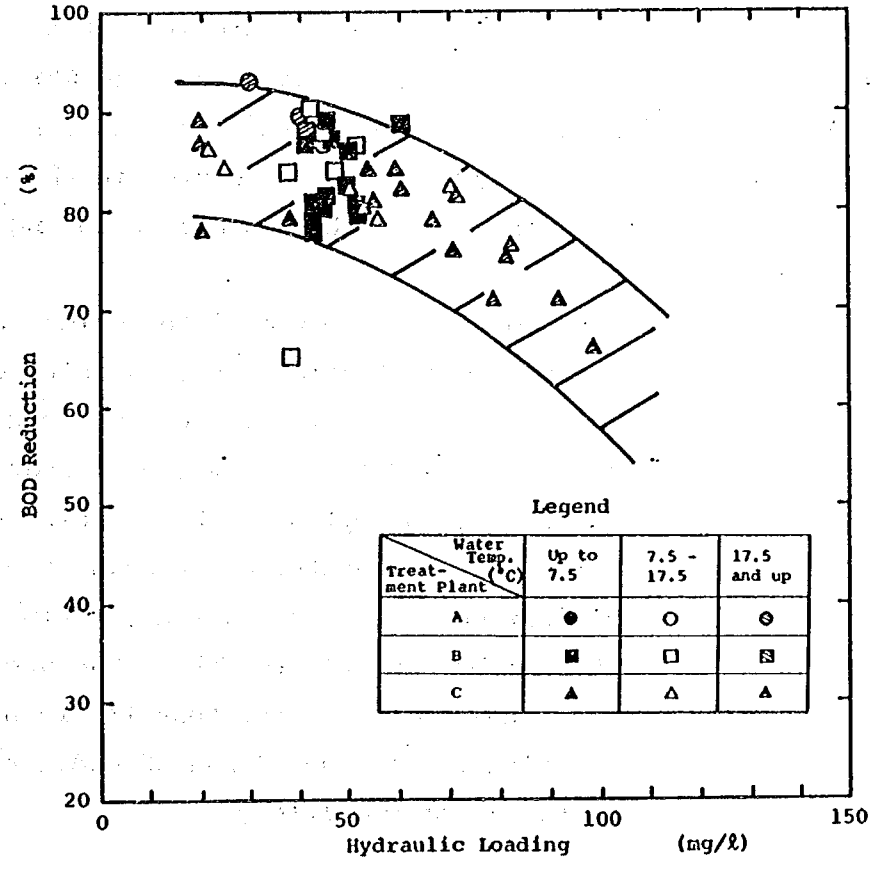


Fig. 3-3 Effect of BOD Loading on BOD Removal Efficiency



(1) RBC Influent BOD Concentration, up to 100 mg/ℓ



(2) RBC Influent BOD, 100 mg/ℓ to 200 mg/ℓ

Fig. 3-4 Effect of Hydraulic Loading on BOD Removal Efficiency

3.1.2 Composition of Effluent BOD

Effluent BOD could be divided into 3 parts; i.e. soluble BOD (S-BOD), insoluble BOD (SS-BOD) and oxygen demand by nitrification (N-BOD). In addition to measurement of effluent BOD according to the ordinary method, BOD with nitrification inhibited by allylthiourea (ATU) -- ATU-BOD -- was also measured, and N-BOD was calculated as the difference between ordinary BOD and ATU-BOD.

The ratios of N-BOD, SS-BOD and S-BOD to effluent BOD are shown in Fig. 3-5. The largest fraction of effluent BOD is accounted for by SS-BOD at 60 to 70%.

3.1.3 N-BOD

In wastewater treatment by the RBC process, nitrification efficiency (Kjeldahl nitrogen reduction) increases with decreases in BOD loading, as shown in Fig. 3-6. When BOD loading is 2 to 3 $g/m^2.day$, nitrification efficiency rises to 80%. In this way, nitrification can easily occur in the RBC process.

When this process is applied to secondary treatement of domestic wastewater, nitrification efficiency achievable is 40 to 60% because BOD loading is usually in the range of 5 to 10 $g/m^2.day$. As a result, nitrifier and un-nitrified nitrogen compounds coexist in the effluent, and further nitrification progresses in the BOD bottle during incubation for the BOD test.

The N-BOD undergoes the combined effect of interactions between BOD load, effluent suspended solids concentration and un-nitrified nitrogen compounds concentration, dilution for BOD test, etc. The study results show that N-BOD is 10 mg/ℓ maximum and about 3 mg/ℓ on average.

Inhibited BOD has not yet been accepted in the Japanese effluent standards. Therefore, if the RBC is operated under such loading conditions that make nitrification take place halfway, effluent BOD will be increased by N-BOD. It will be possible to make N-BOD negligibly low either by increasing RBC loadings to the extent that little nitrification will take place or by increasing the surfance area of media to the extent that nitrification will occur almost completely.

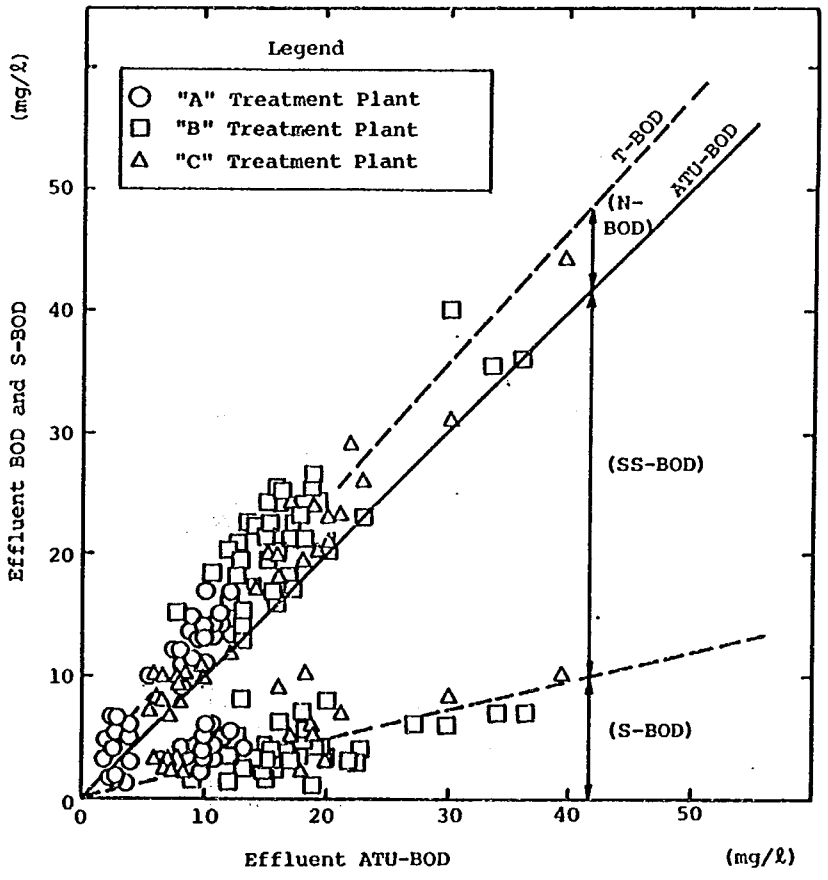


Fig. 3-5 Composition of Effluent BOD

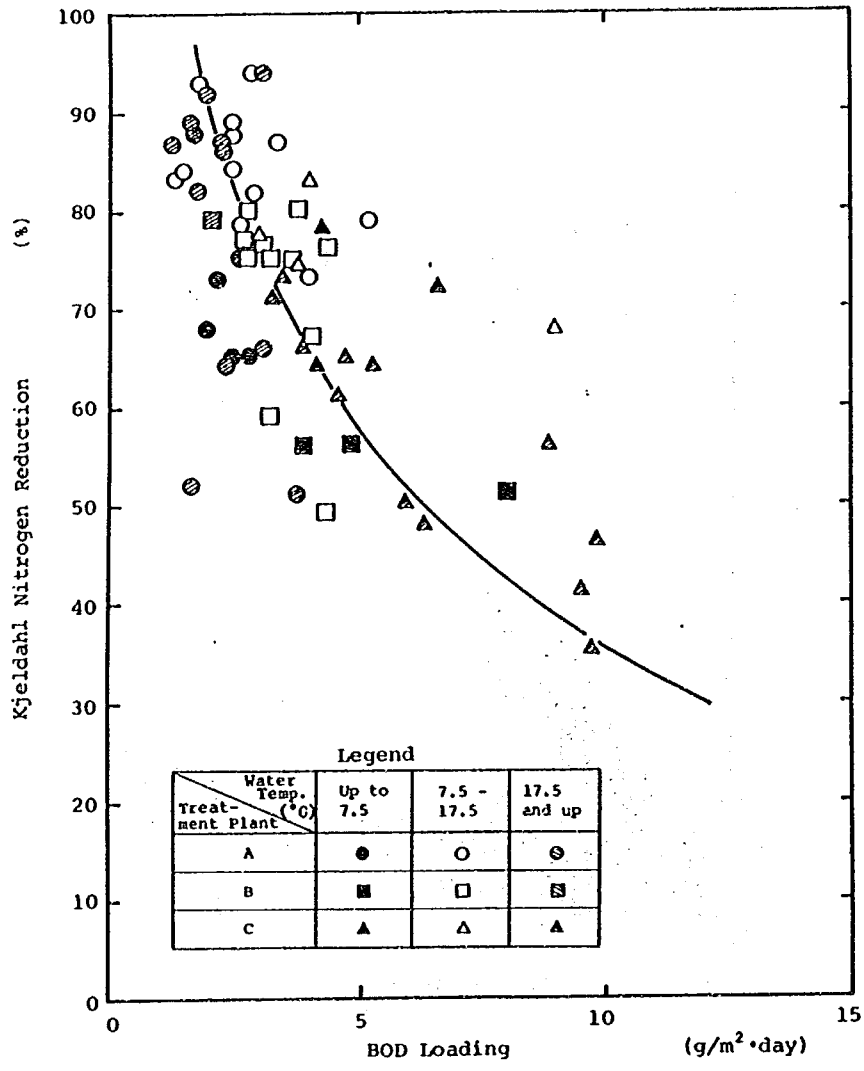


Fig. 3-6 Effect of BOD Loading on Nitrification

3.1.4 S-BOD

The effect of soluble BOD loading (S-BOD loading) on effluent S-BOD is shown in Fig. 3-7, and the effect of S-BOD loading on S-BOD removal efficiency is given in Fig. 3-8. In every investigatd plant, the S-BOD/BOD ratio in RBC influent was nearly 0.5. This means that an S-BOD loading of 5 g/m^2.day is equivalent to a BOD loading of 10 g/m^2.day. Even under such a comparatively high loading condition, effluent S-BOD is held down to about 5 mg/ℓ with an S-BOD removal efficiency up to about 90%.

Generally speaking, the efficiency of biologicl treatment is influenced by water temperature. So far as Figs. 3-7 and 3-8 go, however, the effects of water temperature on the process performance are not clear. In the multistage process, the decrease in water temperature may reduce substrate removal in the initial stages, but subsequent stages are capable of increasing their respective substrate removal rates in response to higher influent substrate concentrations. Because of this shifting in substrate removal, overall process performance is not influenced by water temperature under specific loading conditions.

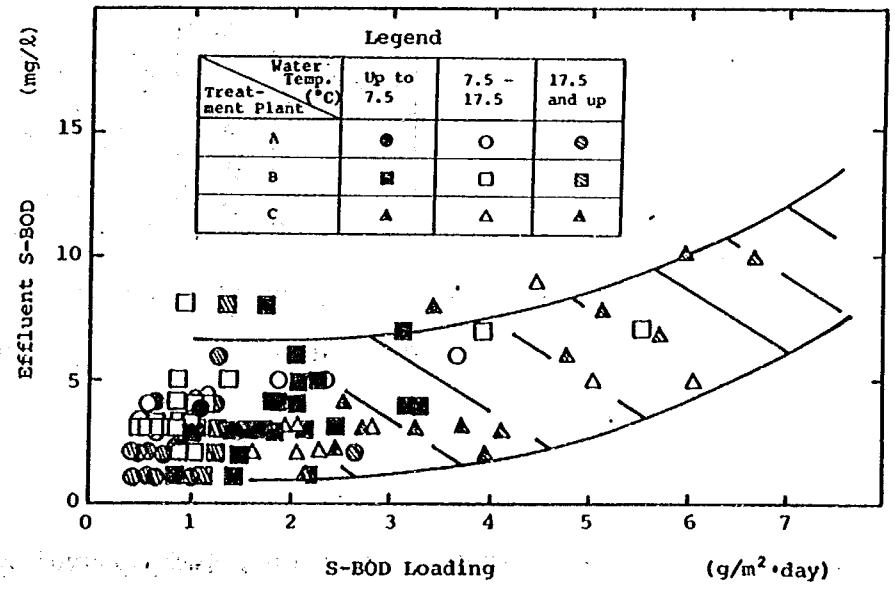


Fig. 3-7 Effect of S-BOD Loading on Effluent S-BOD

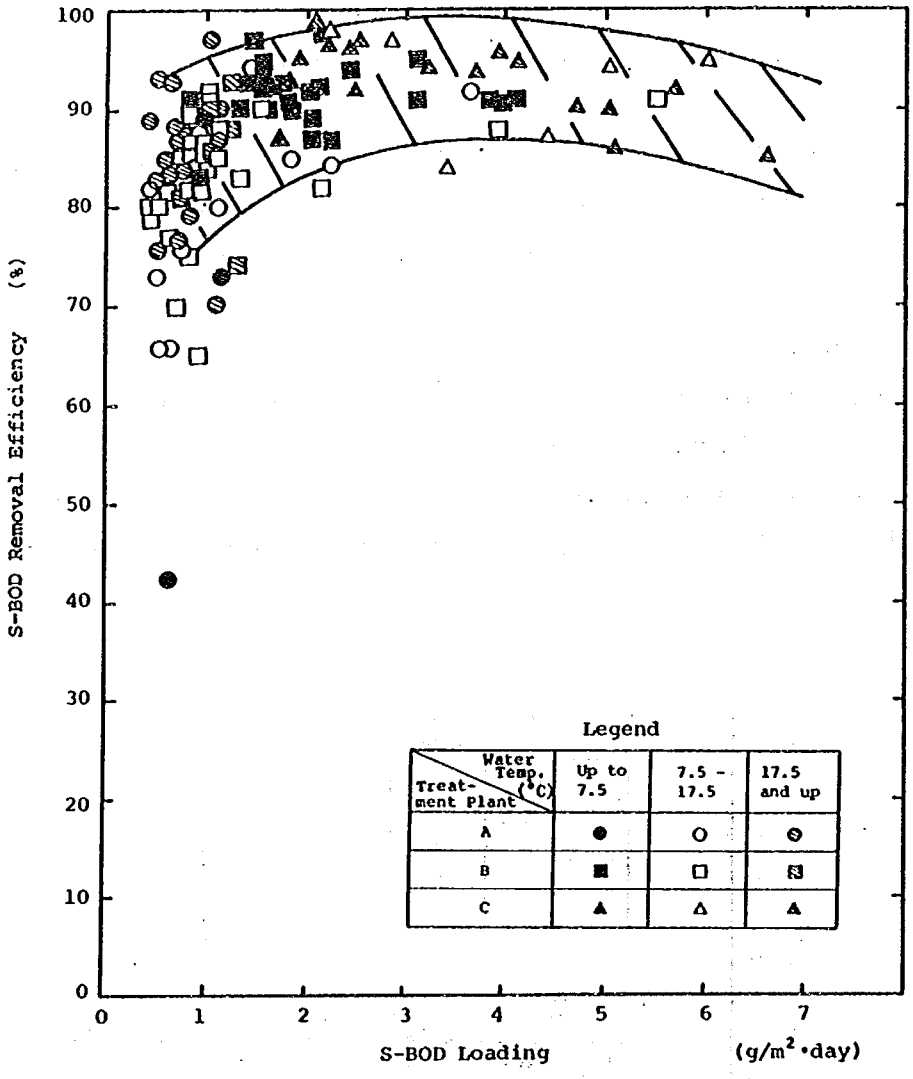


Fig. 3-8 Effect of S-BOD Loading on S-BOD Removal Efficiency

3.1.5 SS-BOD

SS-BOD represents insoluble organic carbonaceous BOD. It was calculated as the difference between ATU-BOD and S-BOD. There is a linear relationship between insoluble matter -- suspended solids -- and SS-BOD in the effluent as shown in Fig. 3-9. The mean value of SS-BOD/SS ratio was 0.75.

As will be discussed later, SS removal in the final settling tanks during the investigation period was carried out nearly to maximum level, and the greater part of suspended solids in the effluent was composed of fine particles which are hard to settle down. In other words, SS-BOD in the effluent is governed by fine suspended solids. As explained earlier, SS-BOD acount for 60 to 70% of effluent BOD. Therefore, BOD removal is largely governed by suspended solids.

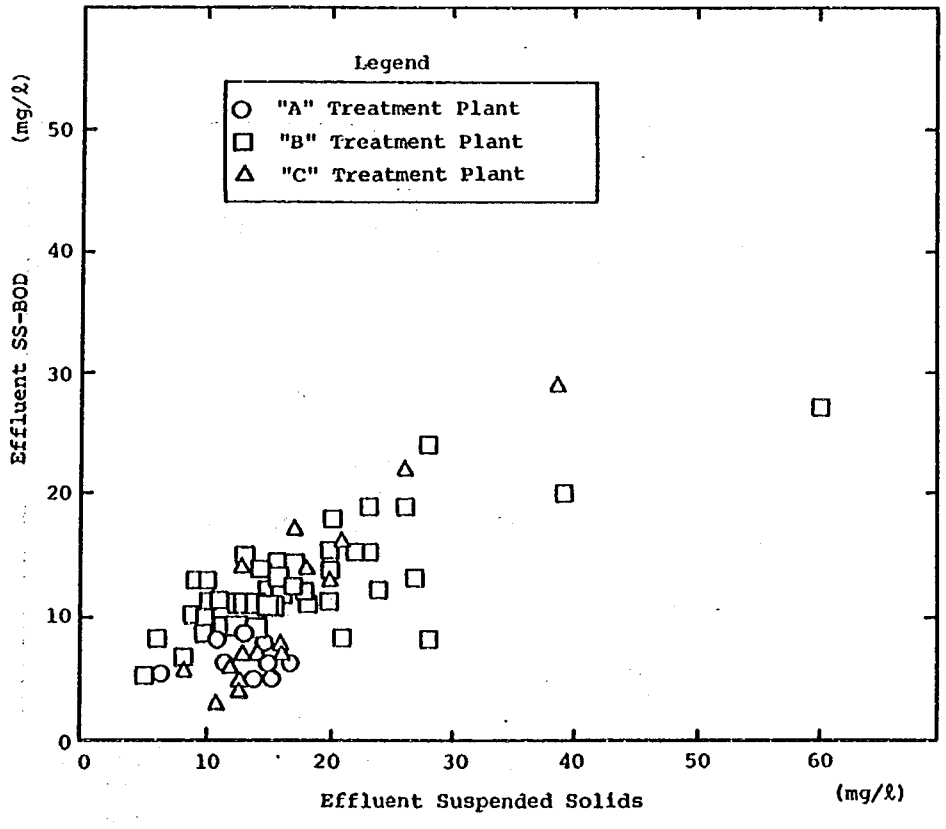


Fig. 3-9 Relationship between Suspended Solids and SS-BOD in Effluent

3.2 SUSPENDED SOLIDS REMOVAL

3.2.1 Suspended Solids in RBC Effluent

The amount of biomass flowing out of the RBC reactors in the form of suspended solids is affected by the loading conditions of RBC. If BOD loading increases, the amount of suspended solids flowing out of the reactors also increases.

In the RBC process, treatment of wastewater is carried out by the biomass attached to the media, thus, the sludge recycling from final settling tanks is not necessary. The majority of suspended solids flowing out of the reactors is the biomass sloughing off the media. As the biomass sloughing increases with increases in BOD loading, the MLSS in the reactors and suspended solids flowing out of the reactors tends to rise (see Fig. 3-10)

The concentration of MLSS in RBC reactors is much lower than that in the activated sludge process; it is in the range of 50 to 100 mg/ℓ at a BOD loading of 10 g/m^2.day.

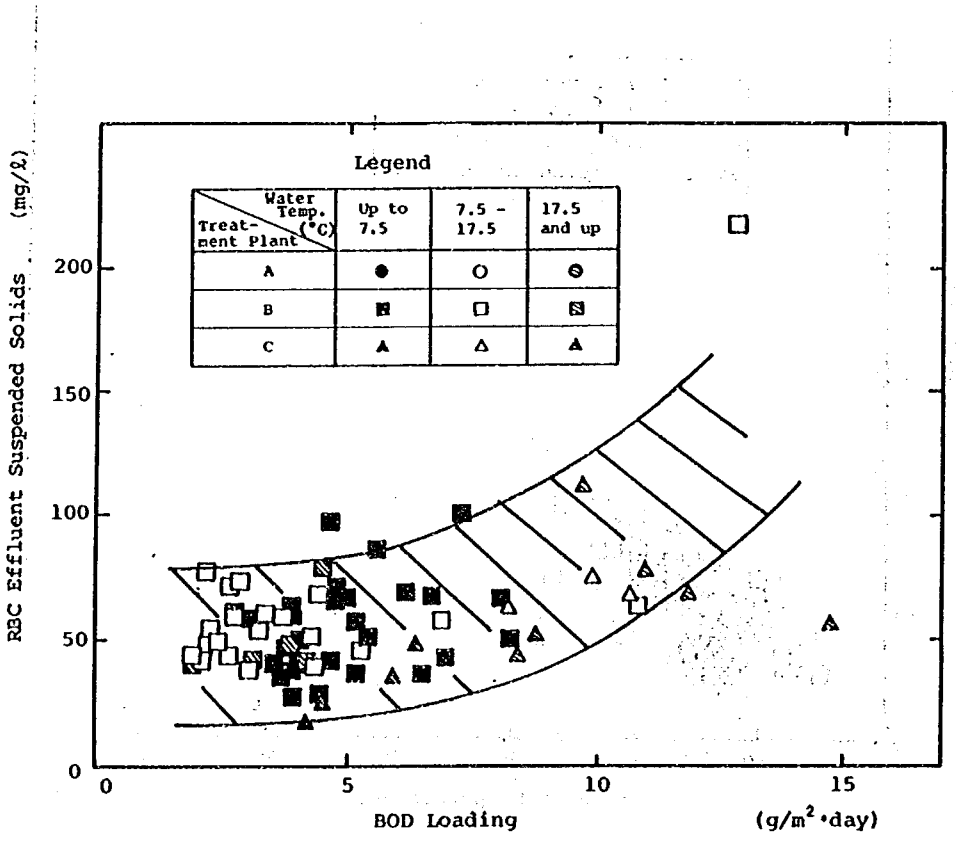


Fig. 3-10 Effect of BOD Loading on Suspended Solids in RBC Effluent

3.2.2 Settlability of Suspended Solids from RBC Reactors

RBC effluent was settled quiescently in a settling column measuring 40 cm in diameter and 2 m in height. Fig. 3-11 shows the time-dependent changes of the suspended solids concentration at points 30 cm and 150 cm below the water surface. After starting the settling test, the concentration of suspended solids began to decrease, but the decrement of the concentration decreased with time, and little decrease of suspended solids concentration could be observed in 30 min after the start of settling.

Suspended solids flowing out of the RBC reactors show wide varietions in both shape and size, and range from filaments of about 10 mm length to fine particles. Their concentration is low, and they do not form floc. Hence, most suspended solids settle down independent of each other in about 30 min but fine suspended solids remain afloat in the supernatant liquor.

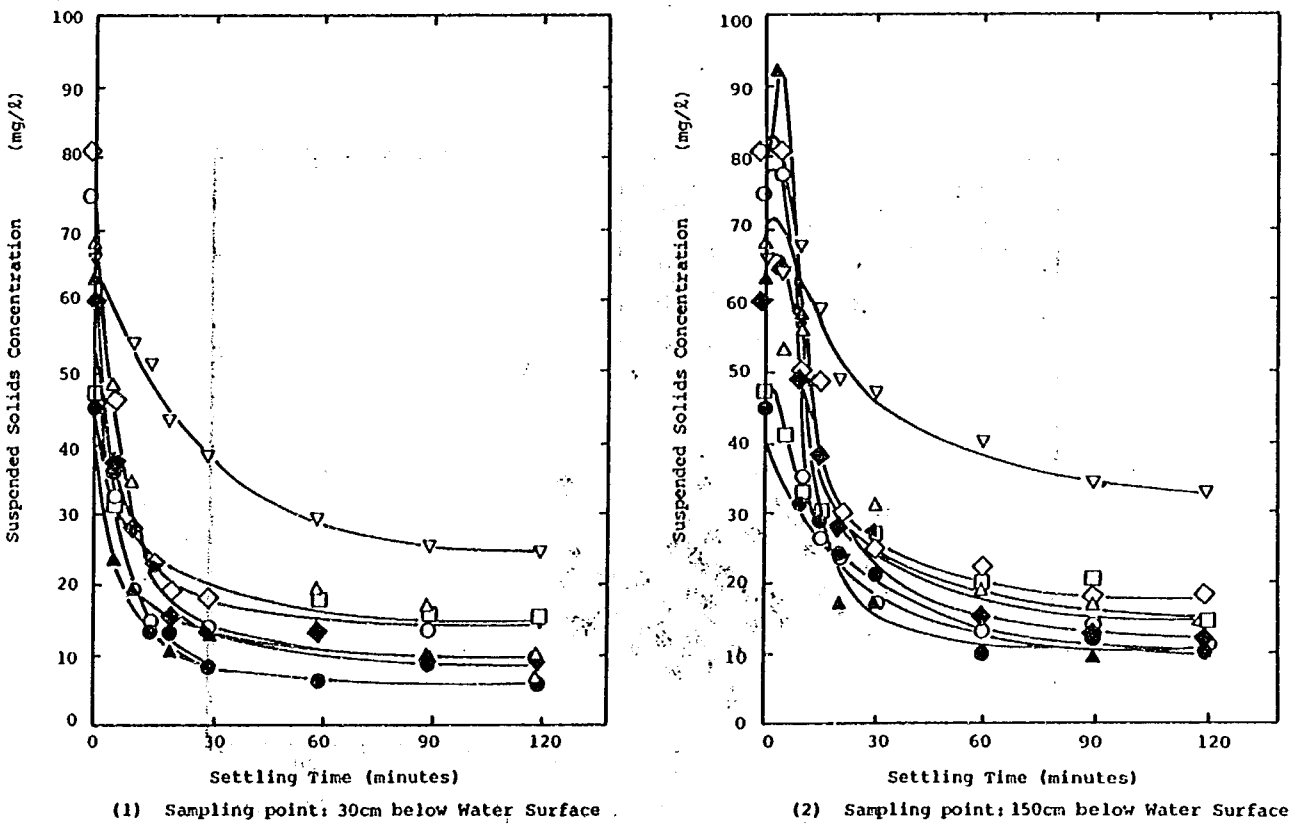


Fig. 3-11 Results of Quiescent Settling Test

3.2.3 Suspended Solids Removal in Final Settling Tanks

The concentration of suspended solids in RBC effluent is 50 to 100 mg/ℓ even when BOD loading is 10 g/m^2.day, and the amount of suspended solids which should be removed in the final settling tanks is quite small compared to the activated sludge process.

During the investigation period, the overflow rate of the final settling tank was below 25 m^3/m^2.day, and mostly in the range of 10 to 15 m^3/m^2.day. As shown in Fig. 3-12, effluent suspended solids concentration was not influenced by the overflow rate of the final settling tank. This is because when the overflow rate is around 25 m^3/m^2.day, the settlable suspended solids, that can settle down in 30 min of quiescent settling, are removed almost totally leaving only unsettlable fine suspended solids in the effluent.

The increase in suspended solids in the influent of the final settling tank increases the fine suspended solids in it, thus increasing the amount of suspended solids flowing out with the effluent (see Fig. 3-13)

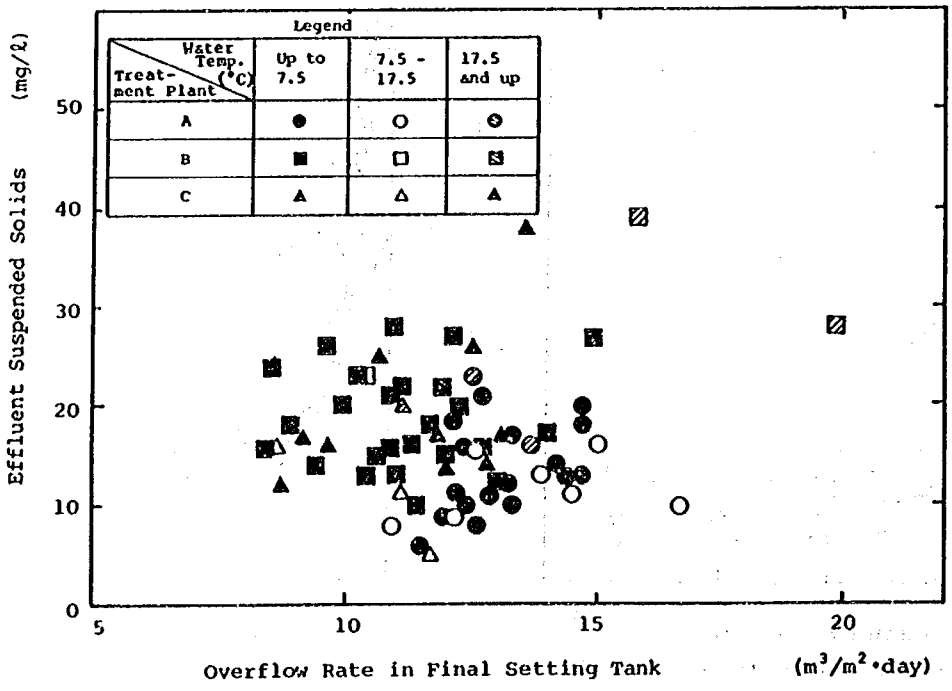


Fig. 3-12 Relationship between overflow Rate and Effluent Suspended Solids

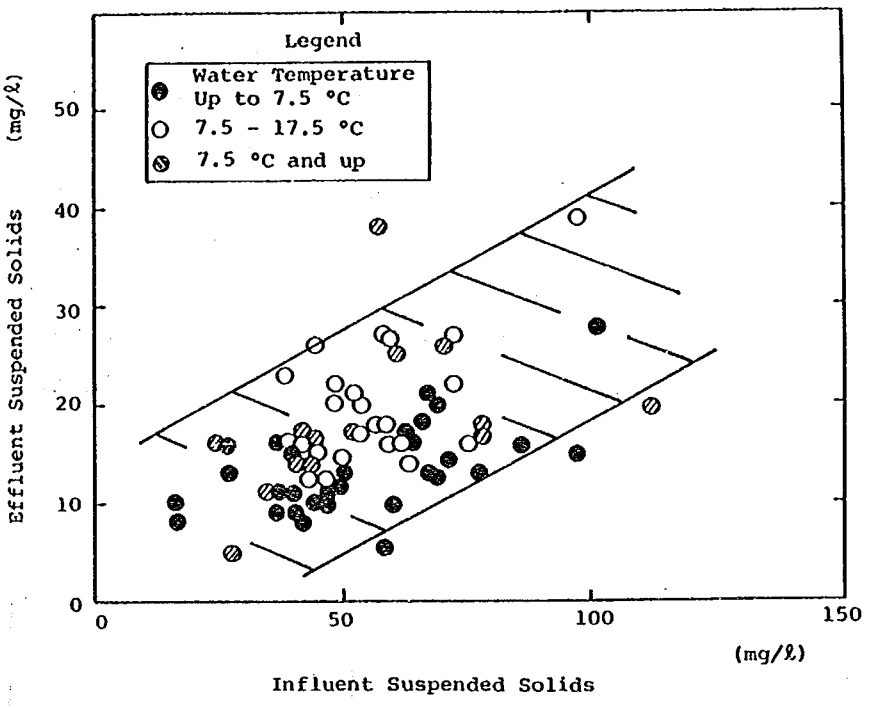


Fig. 3-13 Relationship of Suspended Solids Concentration between Influent and Effluent of Final Settling Tank

3.2.4 Suspended Solids in Effluent

The increase in BOD loading tends to increase the concentration of suspended solids in the influent of the final settling tank, increasing BOD in the effluent. In Japan, the concentration of suspended solids at the outfall is restricted below 70 mg/ℓ as daily average. When the overflow rate of the final settling tank was held below 25 m^3/m^2.day, and BOD loading was less than 15 g/m^2.day, as shown in Fig. 3-14, the suspended solids concentration of the effluent was below this value.

Concentration of suspended solids in the effluent is affected not only by the suspended solids concentration in the influent of final settling tanks, but also by the operating conditions of the final settling tanks, settlability of suspended solids sloughing off the media, etc.

Accordingly, the increase in suspended solids concentration in the effluent due to increases in BOD loading is not so conspicuous, and the suspended solids data cover a wide range at each specific level of BOD loading. Even at a low level of BOD loading, the suspended solids concentration in effluent can assume a high value.

As effluent BOD is largely influenced by suspended solids, BOD removal performance is not controlled only by the operating conditions of RBC; the relationship between hydraulic loading or BOD loading and BOD removal is unstable. As the result, insufficient BOD removal performance may occur even when the RBC is operated under lower loadings.

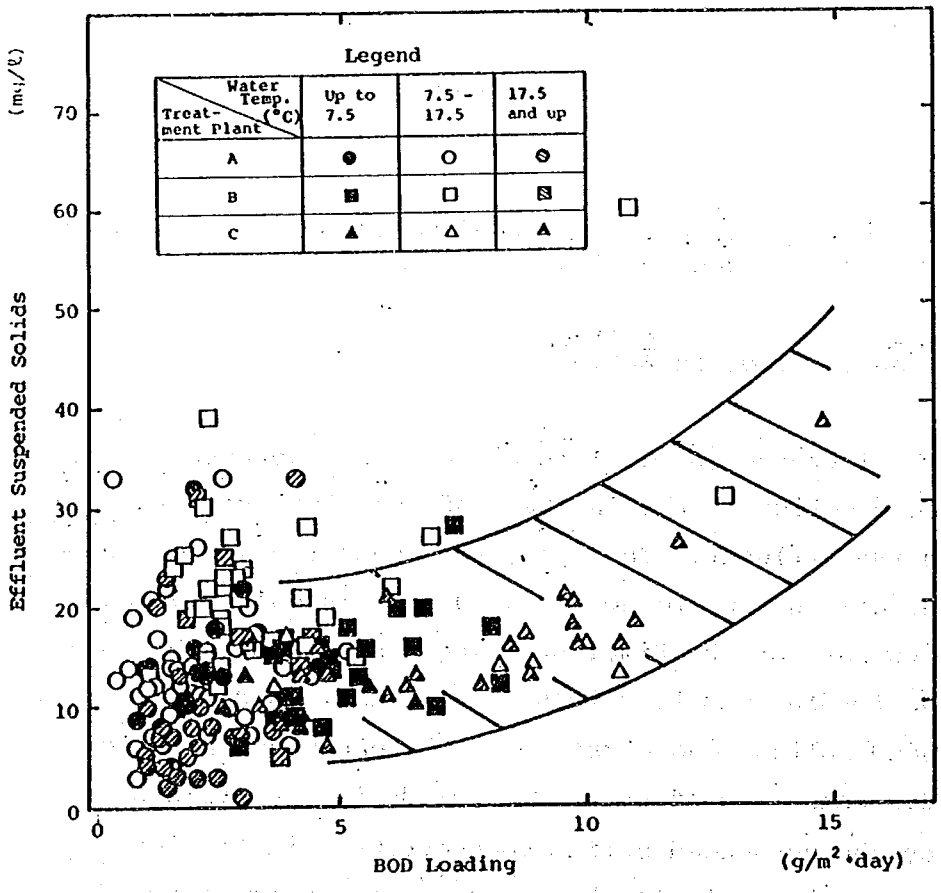


Fig. 3-14 Effect of BOD Loading on Effluent Suspended Solids

4. UPGRADING OF EFFLUENT BOD

4.1 UPGRADING METHODS

As stated earlier, effluent BOD consists of N-BOD, SS-BOD S-BOD. Of these, S-BOD is removed satisfactorily, and N-BOD will be reduced to a negligibly small amount if nitrification is inhibited or promoted sufficiently by optimizing the design or operation of the process.

On the other hand, it will be possible to improve and stabilize RBC process perfomance if the suspended solids removal efficiency of the final settling tank is improved by coagulation, or if fine suspended solids are removed through a screen or strainer to maintain SS-BOD in the effluent at a low level.

During the survey, the coagulation, microstrainer and wedge-shaped wire screen were tried in the field to find a way to improve suspended solids removal efficiency. The results were encouraging, proving that these means can improve and stabilize BOD removal in the RBC process.

4.2 FIELD TESTS

4.2.1 Coagulation Test

In this test, poly-aluminum chloride (PAC) was used as a coagulant with sodium hydroxide as a coagulant aid. The results of the jar test are shown in Fig. 4-1; it was found that PAC takes effect when introduced at a rate of 100 to 150 mg/ℓ.

In the field test, PAC was introduced downstream of the RBC units at three levels: 100, 150 and 200 mg/ℓ. Composite samples of effluent from the final settling tanks were taken and compared to those obtained without PAC. Suspended solids in the effluent remained almost constant regardless of whether PAC was added or not (see Fig. 4-2). But the injection of PAC was accompanied by the formation of inorganic floc, reducing the SS-BOD/SS ratio in the supernatant liquor (see Fig. 4-3). The effect of PAC addition on SS-BOD is evident (see Fig. 4-4) When the N-BOD level is neglibibly low, the coagulation can reduce effluent BOD well below 20 mg/ℓ.

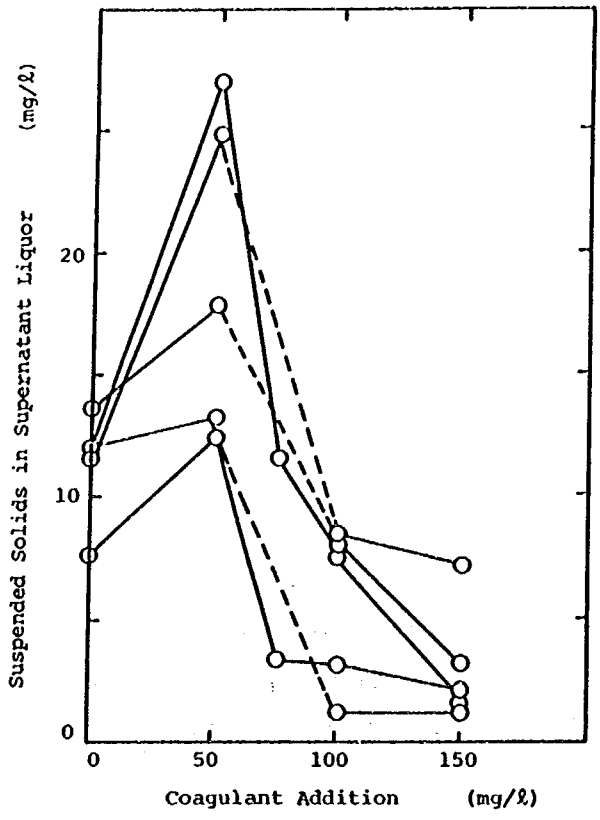


Fig. 4-1 Effect of Coagulant Addition on Suspended Solids in Supernatant Liquor (Jar Test)

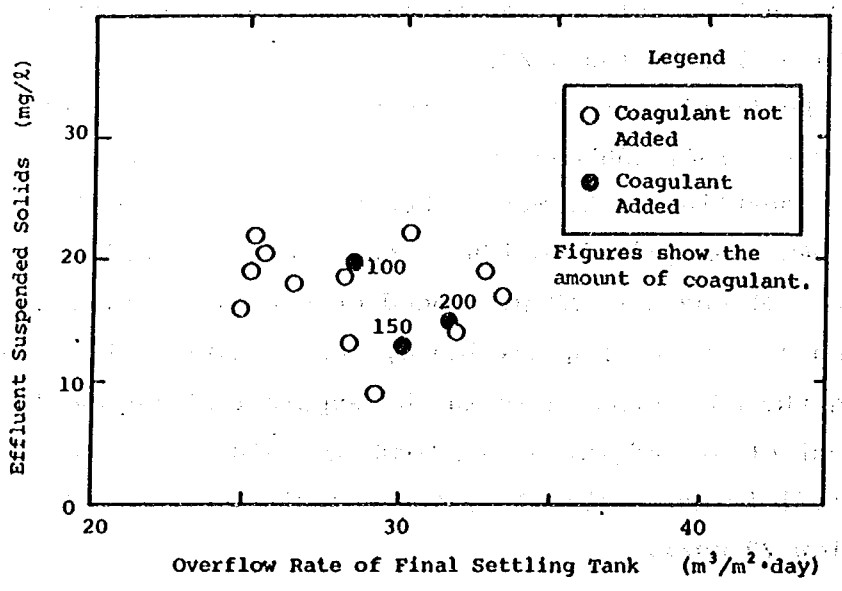


Fig. 4-2 Effect of Coagulant Addition on Suspended Solids in Effluent (Full Scall Test)

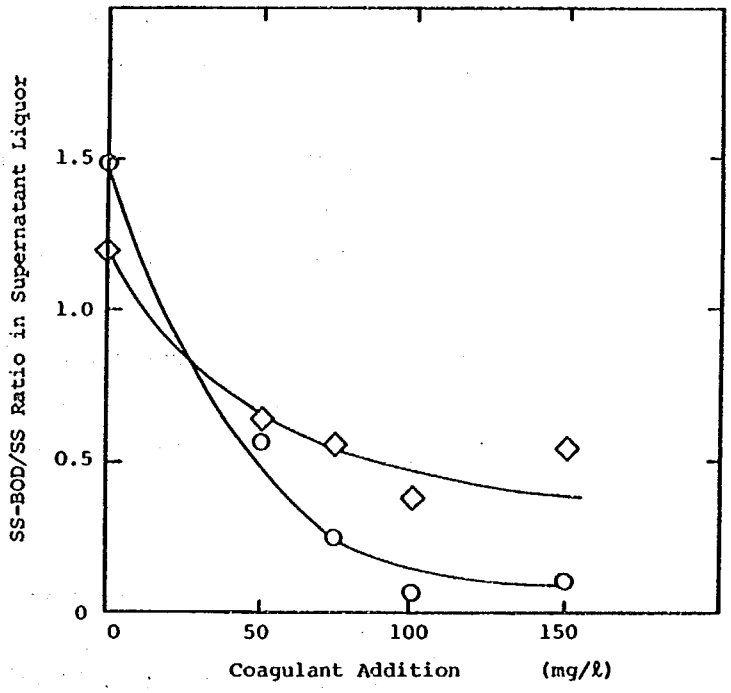


Fig. 4-3 Relationship between Coagulant Addition and SS-BOD/SS Ratio (Jar Test)

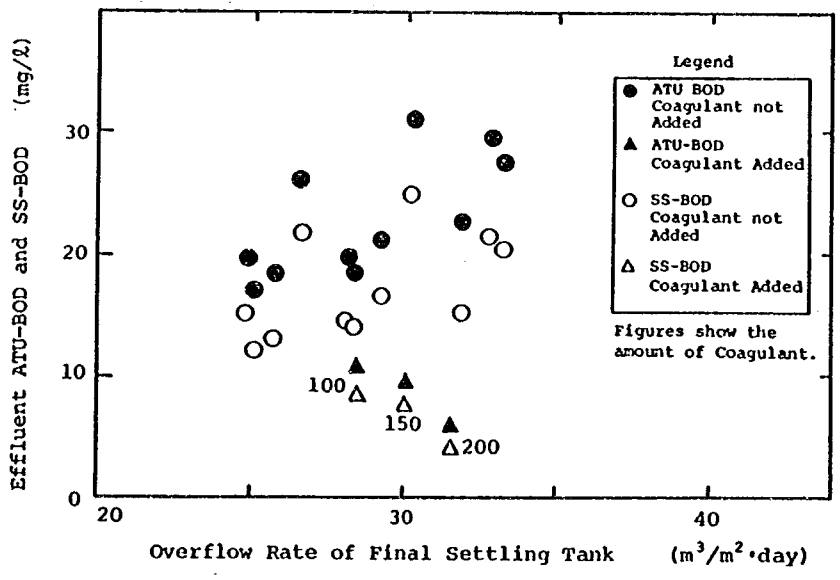


Fig. 4-4 Effect of Coagulant Addition on Effluent BOD

4.2.2 Microstrainer Test

A microstrainer is a fine strainer supported on a drum. Influent is passed through the microstrainer from the inside out to remove suspended solids. Suspended solids trapped in the strainer are scrubbed out as the drum rotates.

The microstrainer specified in Table 4-1 with three types of strainer panels were tested on effluent of the RBC and final settling tank. Grab samples of influent and effluent of the microstrainer were monitered.

It was found from the relationship between suspended solids and ATU-BOD of the microstrainer effluent (see Fig. 4-5) that, if the concentration of suspended solids is held below 15 mg/ℓ, BOD in the effluent can be held below 20 mg/ℓ.

Table 4.1 Microstrainer Specifications

Dram Dimension	Diameter 1200 mm x Length 600 mm
Total Straining Area	2.3 m^3
Submerged straining area	1.0 m^3
Mesh Size of Straining Panel	20μ, 30μ, 50μ
Rotational Velocity	1 - 4 RPM

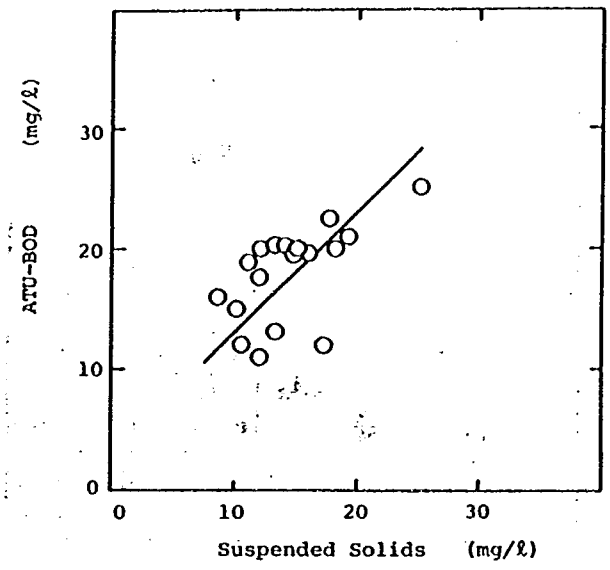


Fig. 4-5 Relationship between Suspended Solids and ATU-BOD in Effluent of Microstrainer

As shown in Fig. 4-6, the suspended solids concentration in microstrainer effluent varied according to strainer mesh size and suspended solids concentration in the influent. If a microstrainer panel of 30 μ to 50 μ is used, the suspended solids concentration in the effluent can be held below 15 mg/ℓ. If a microstrainer panel of 20 μ to 30 is used, it is possible to reduce the suspended solids concentration in the effluent to less than 15 mg/ℓ without the final settling tank.

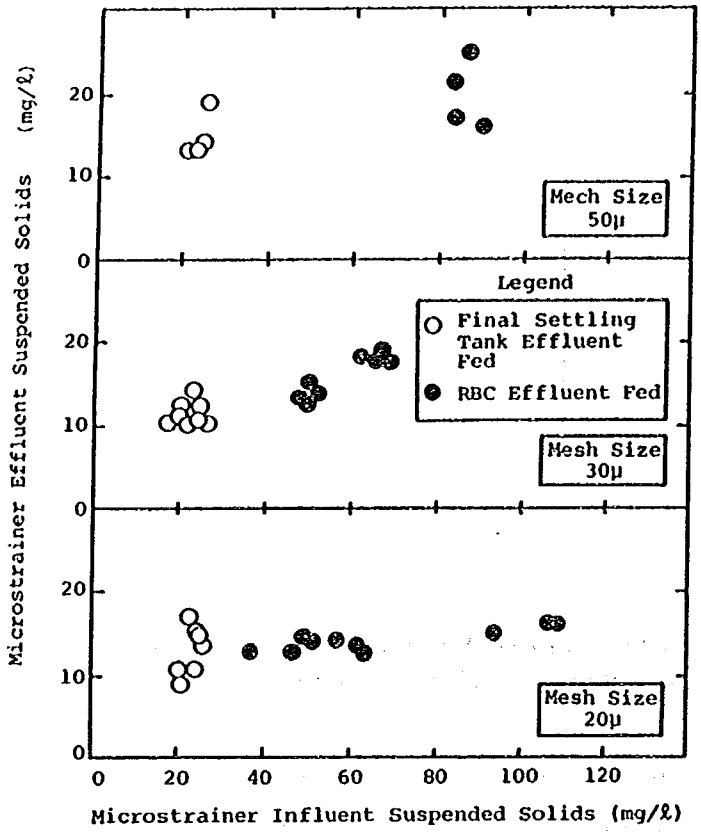


Fig. 4-6 Relationship of Suspended Solids between Influent and Effluent of Microstrainer

4.2.3 Wedge-Shaped Wire Screen Test

The wedge-shaped wire screen consists of a number of stainless steel wires, each having a wedge-shaped cross section. These wires were set parallel to each other, and were titled about 60° against the horizontal plane (see Fig. 4-7). Influent poured over the screen passes through the slits between the wires. The solids which can not pass through the slits slide down along the screen and are removed. The screen is brushed from underneath intermittently to prevent clogging.

As shown in Table 4.2, two types of apparatus with four slit sizes were tested on effluents of the RBC and the final settling tank. Grab samples of influent and effluent of the screens were monitered. Of the screens used, the 30 μ-I type and the 30 μ-II type had the attack angle adjusted. The 30 μ-I type has less solids removal efficiency than the 30 μ-II type, though the former has more flow capacity than the latter.

The relationship between suspended solids and ATU-BOD in effluent from the screen was much the same as that of the microstrainer (see Fig. 4-8). It is possible to reduce ATU-BOD below 20 mg/ℓ if the concentration of suspended solids is held below 15 mg/ℓ.

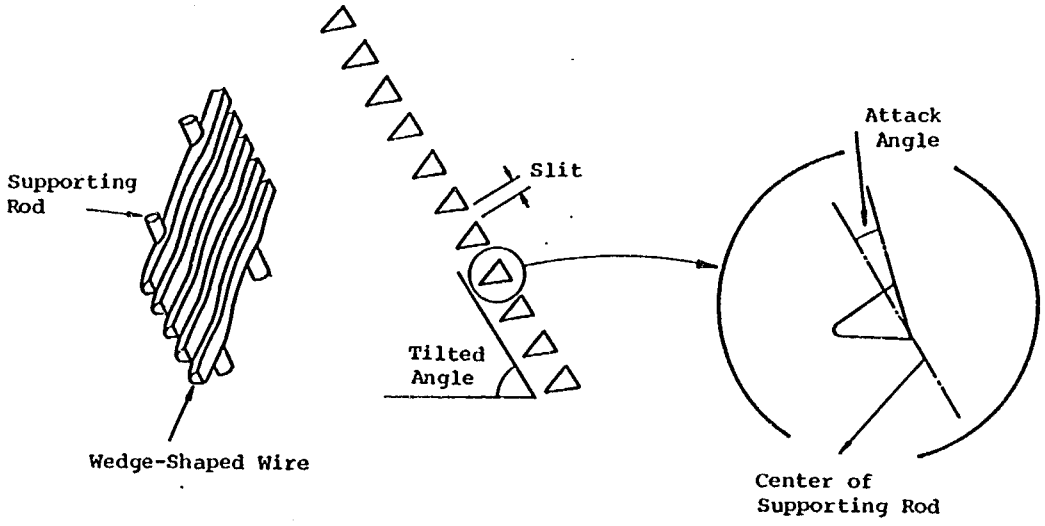


Fig. 4-7 Sketch of Wedge-Shaped Wire Screen

As shown in Fig. 4-9, the suspended solids concentration in the effluent varied according to the slit opening of the screen and suspended solids concentration in the influent.

For satisfactory treatment of the effluent of final settling tanks, the 50 μ or 30 μ-I type screen is suitable. If the 30 μ-II type screen is used, the suspended solids concentration can be reduced to less than 15 mg/ℓ even when RBC effluent is directly screened.

Table 4.2 Wedge-Shaped Wire Screen Specifications

	Type A		Type B	
Width of Screen	900 mm		330 mm	
Length of Screen	400 mm		600 mm	
Slit Size	50 μ	150 μ	30 μ-I	30 μ-II
Effective Screen Area	0.378 m^2	0.240 m^2	0.198 m^2	0.198 m^2
Tilted Angle	60°		60°	

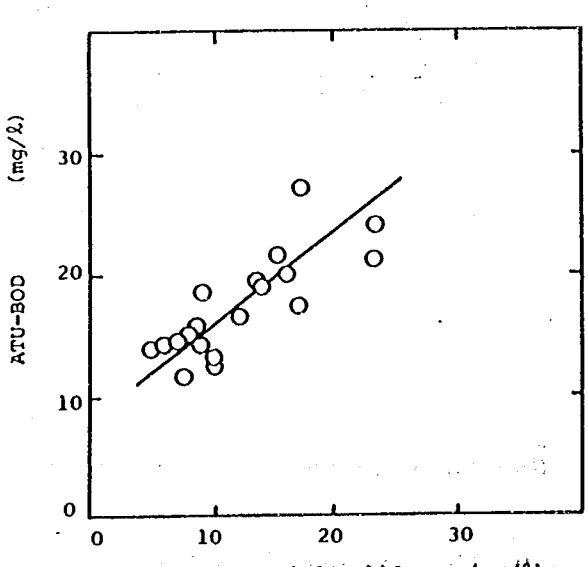


Fig. 4-8 Relationship between Suspended Solids and ATU-BOD in Effluent of Wedge-Shaped Wire Screen

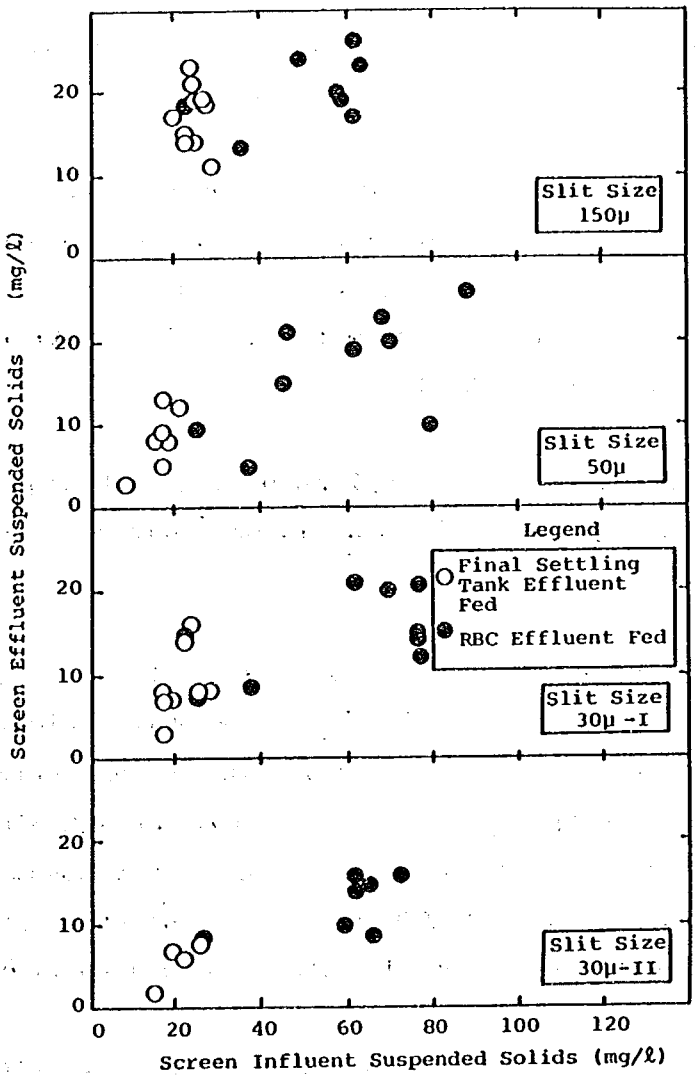


Fig. 4-9 Relationship of SS between Influent and Effluent of Wedge-Shaped Wire Screen

5. SUMMARY

Findings from field investigations of wastewater treatment plants using the RBC process are as follows:

(1) When the effluent BOD is classified into N-BOD, SS-BOD and S-BOD, the percentage of SS-BOD is largest at 60 to 70%.

(2) Nitrification can easily occur in the RBC process. Nitrification efficiency has close relation with loading conditions of the RBC. When the RBC is operated at a load that makes nitrification occur incompletely, effluent BOD will rise since nitrirication can progress in the BOD bottle.

(3) S-BOD removal can be accomplished to a high degree even when BOD and S-BOD loadings are as high as 10 g/m^2.day and 5 g/m^2.day, respectively.

(4) SS-BOD is governed by fine suspended solids, thus the effluent BOD is directly related to the solids removal efficiency of the final settling tank. For this reason, BOD removal performance is not determined by the operating conditions of RBC alone. As the relationship between BOD loading and BOD removal or between hydraulic loading and BOD removal is unstable, BOD removal may be unsatisfactory even when the RBC is operated under lower loadings.

(5) The suspended solids concentration in RBC effluent is comparatively low, in the range of 50 to 100 mg/ℓ when BOD loading is 10 g/m^2.day. Suspended solids in RBC effluent are not uniformi in shape and size and do not form floc. By simple gravitational settling, therefore, the majority of larger suspended solids can easily be settled down, but those fine suspended solids which are hard to settle down remain afloat in the supernatant liquor.

(6) In a simple gravitational settling system using a final settling tank, effluent BOD may become higher when fine suspended solids flow out of the final settling tank. By removing the fine suspended solids chemically or physically using coagulants or screens, BOD removal performance of the RBC process can be highly improved and stabilized.

The RBC process performance is largely affected by the solids separation method. In this process, wastewater treatment is carried out by the biomass attached to the media. Accordingly, there is no need to recycle sludge to the reactors, and the MLSS concentration in the RBC reactors is by far less than that in the aeration tanks of the activated sludge process. The amount of solids separated in the form of sludge is quite small. Therefore, solids separation in the RBC process can be achieved in a way differnet from the activated sludge process, etc.

Only the coagulation, microstrainer and wedge-shaped wire screen were tried for solids separation in this study. Other methods of separating fine suspended solids will be also available. These methods will upgrade the effluent quality of the RBC process. In addition, it is expected that these solids separation methods would reduce the size of the final settling tanks or eliminate them, the economic viability, operation and maintenance aspects of the RBC process will be tremendously improved.

WASTE WATER TREATMENT FOR SMALL COMMUNITIES BY COMBINED PROCESSES

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Waste Water Treatment in Small Communities by Combined Processes

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Small communities have very often been connected to group treatment plants in 5000/10000 inh./inh. equiv. range in the past in Germany, even if relatively long trunk sewers were necessary. The reasons for this were better maintainance of the treatment plants, lower specific clarifier costs and better clarification efficiency. This trend however, does not exist since the last 2-3 years, since investigations of existing plants and development of new treatment systems have shown that the smallest and small plants have very good clarification efficiencies with high process stability and are much cheaper in comparison to conventional techniques.

In the Federal Republic of Germany almost all the medium-sized and large treatment plants have been constructed for 50000 inh./inh. equiv. With regard to the existing laws and minimum requirements, some of these plants will have to be enlarged, the main task however lies in the construction and expansion of the numerous small treatment plants up to a size of approx. 30000/50000 inh./inh. equiv.

The following aims and demands are in this occasion to be strived or must be attained:

1.) Reduction of the high construction costs = less investment outlay

2.) Less electricity consumption per inh./inh. equiv.

3.) High clarification efficiency and process stability

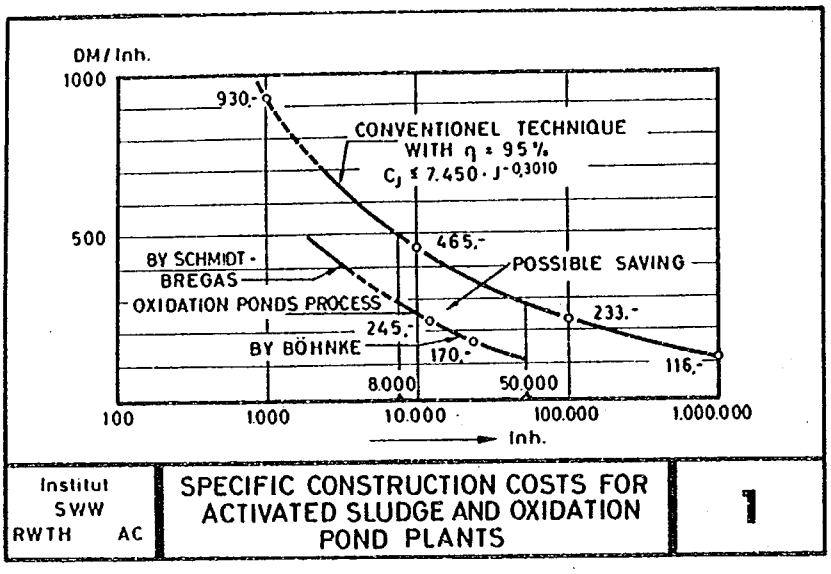
4.) Increased degradation of hard degradable organic substances

5.) Development of systems that allow the integration of existing trickling filter or activated sludge plants

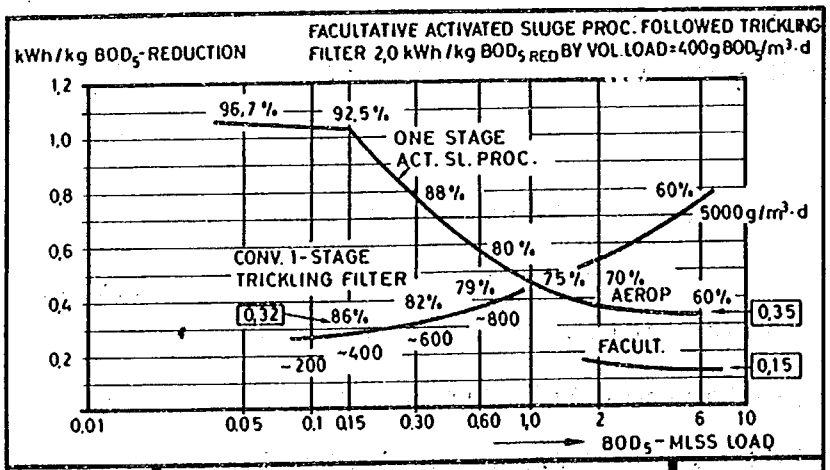
6.) Construction of treatment plants that are unsensitive to qualitative and quantitative waste water peaks

7.) Construction of multiple stage systems

In order to delve further into this topic it is necessary to review briefly the basic questions of specific costs, specific energy needs, clarification efficiency, process stability and area required from the present viewpoint.



Institut SWW RWTH AC	SPECIFIC CONSTRUCTION COSTS FOR ACTIVATED SLUDGE AND OXIDATION POND PLANTS	1

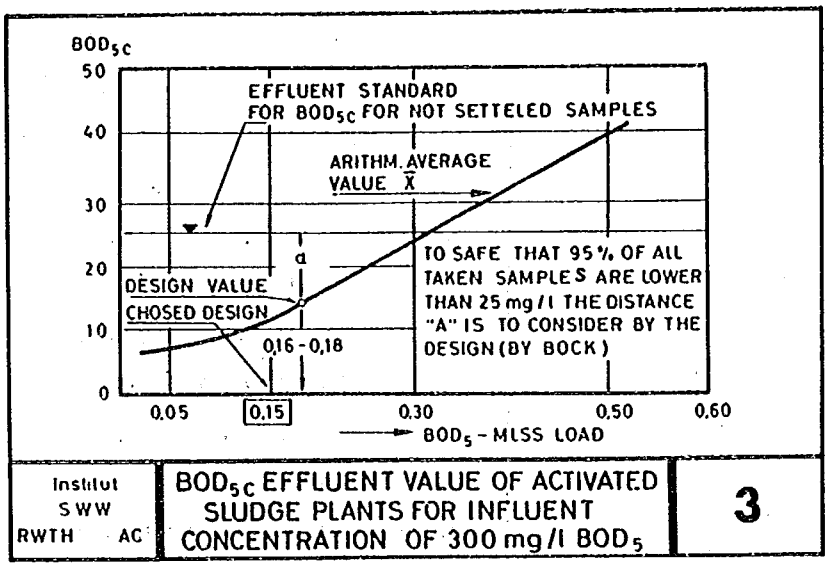


Institut SWW RWTH AC	SPECIF. ENERGY DEMAND OF ACTIV. SLUDGE PROC. AND TRICKLING FILTER	2

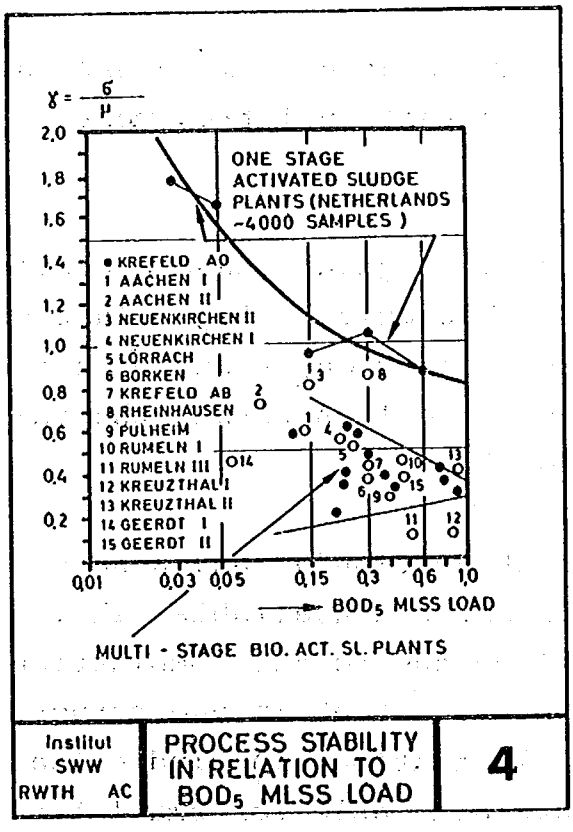
Conventional technologie requires intensive maintainence and is too expensive for smaller inflow values. In contrast to this are oxidation ponds, for examples, very much cheaper, equely efficient, have a higher process stability, require however 2-3 times the area (fig. 1) [1].

For single-stage activated and trickling filter plants, the adjacent functions for the specific power requirements per kg BOD_5-reduction are valid, assuming 200 l/inh. and 60 g BOD_5/inh.·d [2]. depending on the BOD_5 MLSS and volume load. This leads to the conclusion that in order to save energy, clarification plants must be run as 2 or 3 biological stage systems, whereby the first stage should be run as a stage with a very high BOD_5-MLSS load, e.g. 5 kg $\frac{BOD_5}{kg MLSS \cdot d}$. A 2-stage activated sludge plant with BOD_5-MLSS load= 5 in the first stage and BOD_5-MLSS load = 0,3 BOD_5/kg MLSS·d in the

second stage gives the same results as a single stage nitrification plant with a BOD_5-MLSS load of 0,15 [3]. Savings in area, costs and energy are coupled to this.

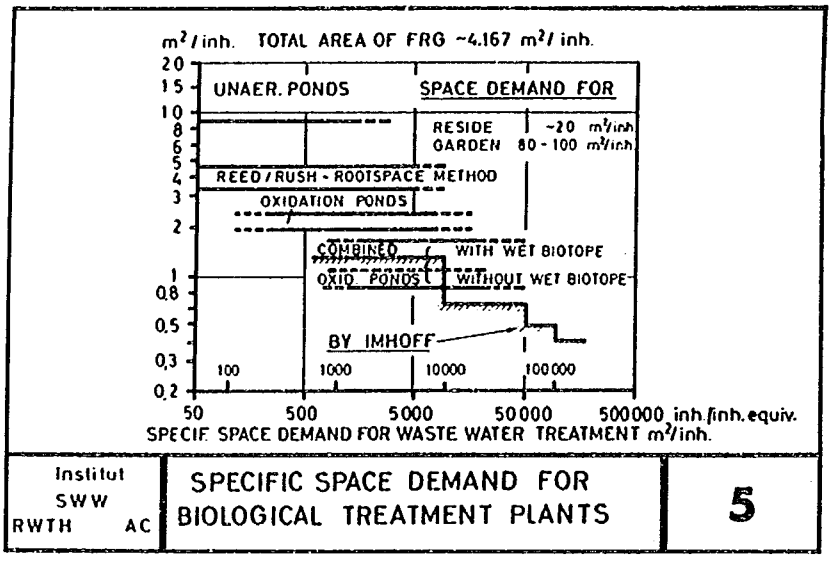


The adjacent effluent values as a function of the sludge load result from thousands of effluent samples primarily from dutch, swedish, swiss and some german plants (24 h mixed samples) (fig. 3). In order to comply with the legally specified (WHG, AAG) minimum requirements, for W = 200 l/inh..d and inh. equiv. = 60 g BOD_5/inh..d a sludge load of BOD_5-MLSS = 0,15 has to be observed. Although a BOD_5 of 25 mg/l is specified as the limiting value for unsedimented samples; this value may not be exceeded by an average of 5 samples, so that according to the theory of probability a distance "a" is necessary [4,5].



The relative deviation from the average clarification efficiency σ is presented in fig. 4 as γ (%) depending on the BOD_5-MLSS load. It is astonishing but quite understandable that the process stability (expressed as relative spread γ) becomes smaller with decreasing sludge load. The distinctly higher process stability of multiple stage plants is clearly discernible. Oxidation ponds exhibit for BOD_5-MLSS

load = 0,05 in the last aerated stage a deviation of γ = 0,6. Reliably operated treatment plants should therefor have biological multiplestage functions.



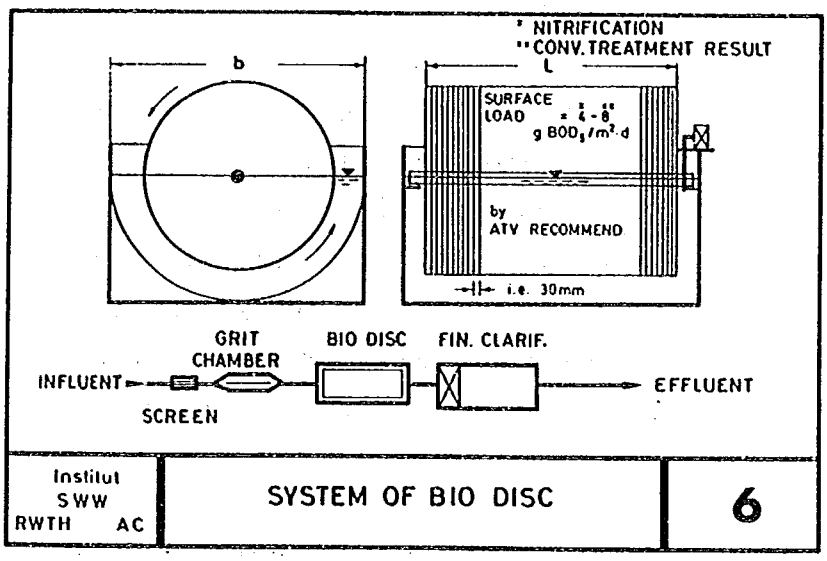
The specific area requirements for clarification processes of large and small volumes are given in fig. 5. Unaerated and areated ponds, of correctly dimensioned for 10 - 20 m^2/inh. or 2,0 - 2,2 m^2/inh., show good effluent values. However, for unaerated ponds, which are suited to very small settlements in rural areas, a certain amount of inconvenience due to odours is unavoidable. Depending on the weather, oxygen deficiency can also occur in aerobic ponds.

Reed-, rush- and root space processes require about 4 m^2 area/inh. They are not suited as substitute for conventional treatment plants [1].

Combined aerated ponds have in the first biological stage a conventionally highly or extremely high loaded biological stage. This combination, which will be discussed further later on, requires a specific area of 0,7 - 1,2 m^2/inh. and therefore lies within the values given by Imhoff (Stadtentwässerung) [6].

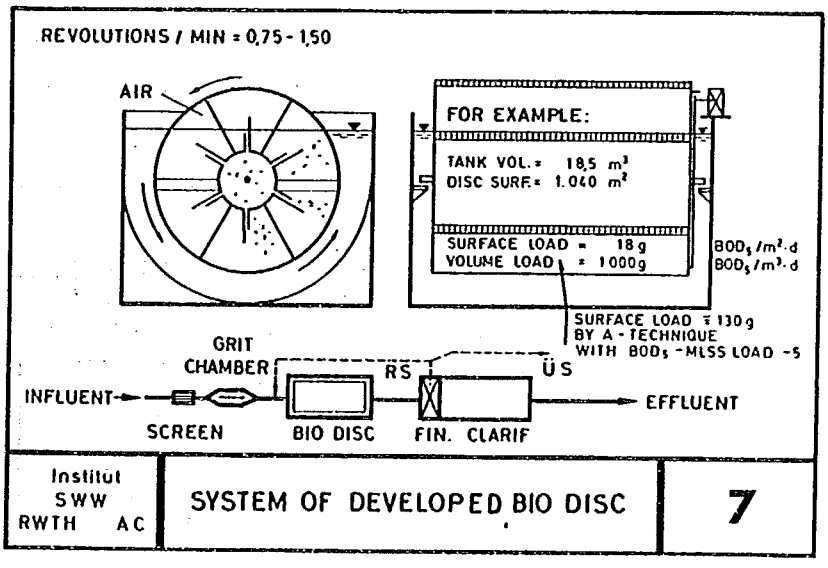
For smaller treatment capacity upto about 3000/5000 inh. a number of different combined clarification systems are available. They will not be dealt with further here, since due to the combination of a biological stage and final sedimentation tank in one structure, they are not adaptable to large fluctuations in sludge load. From the varions activated sludge plants available for small treatment plants the biodisc stands out. This development by Hartmann and

Pöpel has meanwhile been adopted all over the world due to its small specific power requirements by the descompose of organ. matter (kWh/kg·BOD_5). The surface load is 4 - 8 g·BOD_5/m^2 d.



The biodisc from Stähler is a development of this system. The covering of the cylinder surface which is provided with longitudinal slits, leads to further forced aeration with only very little extra energy requirements.

The system ZR6 shown in fig. 7 takes along 3,9 m^3 air at one revolution per minute (i.e. 5,645 m^3 daily). This air which escapes partly through the internal area but mostly through the opposite submerged side, aerates the disc biomass and freely floating activated sludge flocs in the tank. This additional aeration allows a considerably higher surface load of about 18 g BOD_5/m^2 d and permits the expansion of a trickling filter system into a trickling filter-activated sludge system with activated sludge recycle. According to investigations by Bischofsberger and Seyfried with the Stählermatic plant effluent values of 15 mg BOD_5/l at volume of 1 kg/m^3 d have been achieved with percentage clarification efficiencies of 95 % in longterm experiments. With each unit of 3,2 m diameter, 2,55 m breadth and I_B = 25 m^3 contents, waste water of 500 inh. can be biologically clarified (BOD_5 = 20 mg/l)

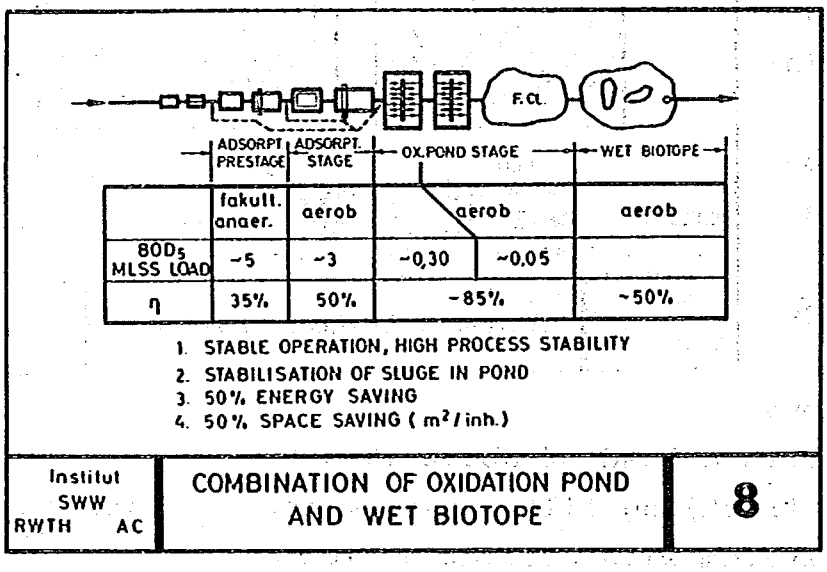


according to the minimum requirements.

By use in an A-stage with a BOD_5-MLSS load of 5 volume loads of 7,5 kg $BOD_5/m^3 \cdot d$ are possible by aerobic operation.

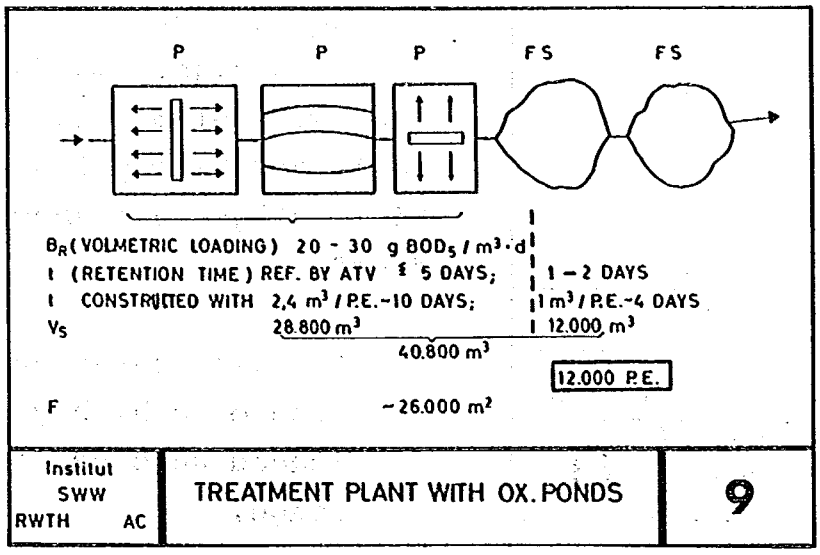
For smaller communities with a size of 8000/10000 inh./inh. equiv., oxidation pond systems are well suited for the clarification of communal or similar waste water. As Bucksteg displayed,about 3 m^3/inh. treatment volume and about 2,2 m^2/inh. pond area are necessary, so that for systems larger than 10000 inh./inh. equiv. the necessary land requirements cannot generally be met. In the Federal Republic of Germany about 300 aerated basins are presently in operation.

As shown by the effluent values presented by Bucksteeg, stable clarification efficiences are achieved in winter as well as in summer. On an average, values of about 10 mg/l can be expected for the BOD_5-effluent and about 65 mg/l for the COD effluent.



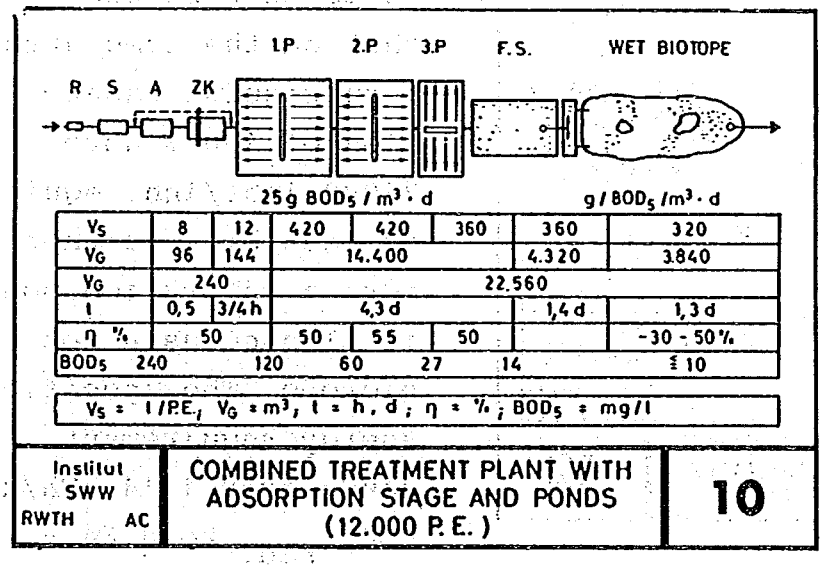
It is therefore a pity, that an area of 2,2 m^2/inh. is necessary. For plants larger than 10000 inh./inh. equiv., the area requirements cannot generally be satisfied.

Efforts are being made to show methods of retaining the advantages of the oxidation pond for a maximum capacity of upto 40 000 inh./inh. equiv..In particular, the high clarification efficiency, the high process stability as well as the economic viability are to be preserved whereby sludge digestion should not be required.



This is possible through the connection of a highly loaded biological stage to the aereated ponds. The sludge produced in the preliminary biological stage is stored in the ponds.

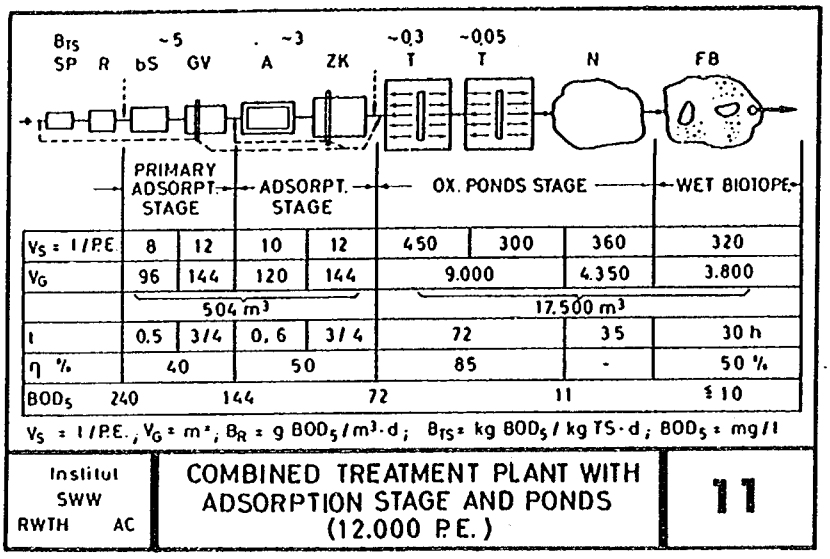
The working principle of a combined oxidation pond plant is shown in fig. 8. The highly loaded activated stage can comprise of one or two A-stages. As shown by experiments still in progress, an aerated grit chamber can simultaneously be used as an adsorption stage. These will be reported subsequently. The succesion of differing biocoenosis lead to very stable clarification efficiencies. As an example, such a system for a community with a size of 12000 inh./ inh. equiv. is presented. The dimensioning values for a conventional oxidation pond plant are shown in fig. 9. A total volume of 40800 m^3 and an area of 26000 m^2 is necessary.



Such savings in volume are only possible, if

a) on the one hand most of the dissolved organic load is converted to sedimentable material in a preliminary biological stage and

b) on the other this increase in sludge content is stored and stabilised in the following smaller ponds.



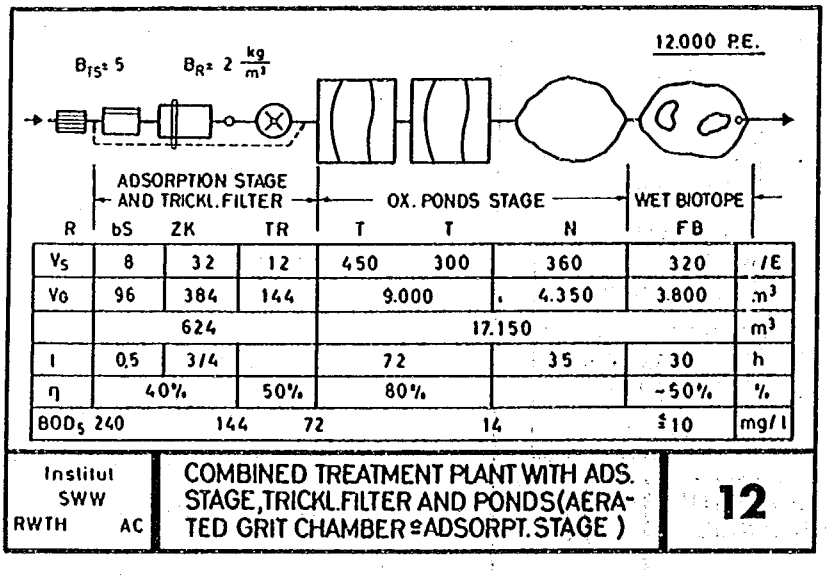
3 possibilities of such a process system are shown in figs. 10; 11 and 12. Preliminarily connected highly loaded biol. stages are shown "red".

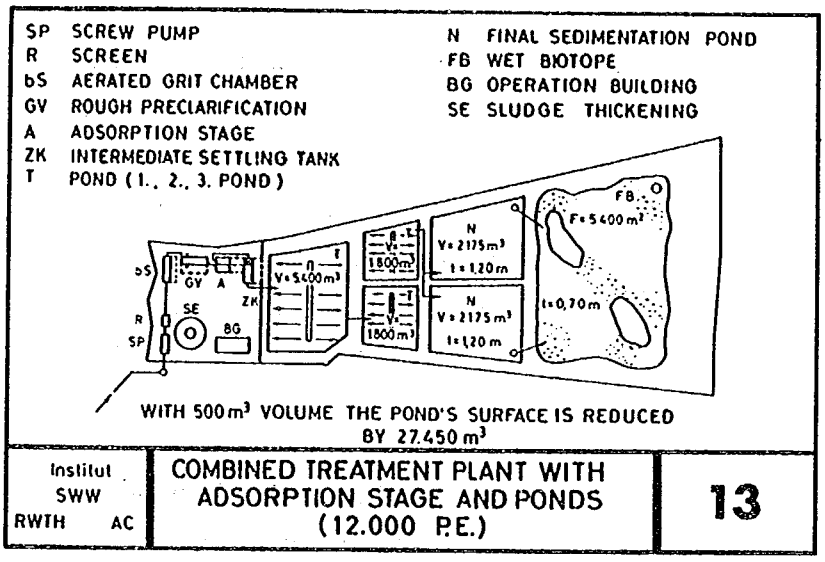
1.) Preliminaryconnection of a highly loaded adsorption stage,

2.) connection of an aerated grit chamber as a preliminary adsorption stage as well as of a highly loaded adsorption stage and

3.) connection of an aerated grit chamber as the A-stage followed by a highly loaded trickling filter.

The second solution (fig. 11) requires an additional volume (500 m³)for the preliminary highly loaded stage, however 26650 m^3 pond volume can thus be saved. Fig. 13 presents a solution for a community with very limited area. It is to be stressed that on the same area a subsequent expansion of 100 % upto 24000 inh./inh. equiv. is possible. Furthermore a maturation pond is planned as a wet biotope. The specific energy requirement sinks from 1,14 kWh/kg BOD_5 red. to 0,64 - 0,85 depending on the system chosen. The

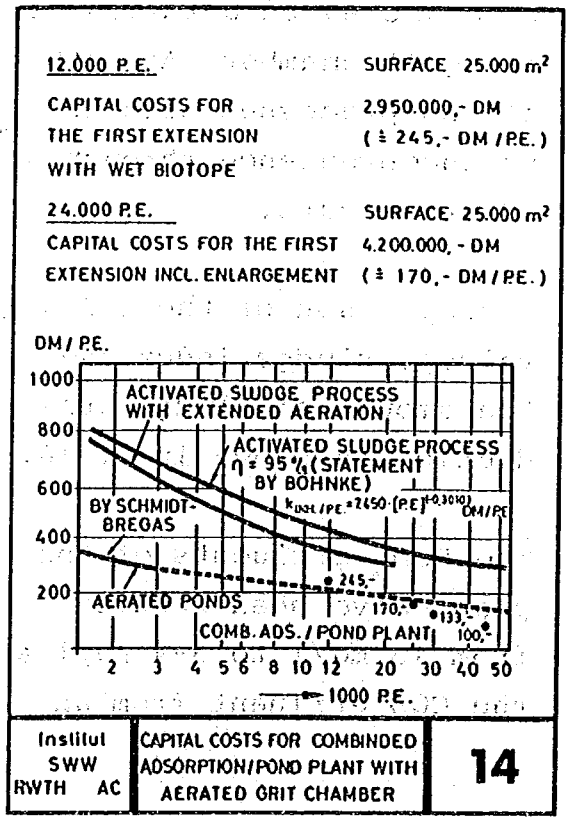




stabilised sludge stored in the aerated ponds is removed once a year, thickened in the sludge dencifiers and dewatered with the decanting centrifuges present.

The maintainance requirements of such a combined oxidation pond plant are naturaly a bit larger than those of an oxidation pond plant, however not larger than a corresponding conventinal biological treatment plant with digester. The maintainance requirements are of absolutely no consequence when the achievable clarification efficiency and process stability are considered.

The costs of the solution presented in fig. 13 are 245 DM/inh., whereby due to the demands stipulated by the authorities significants amounts have to be spent on sludge removal and sludge dewatering.

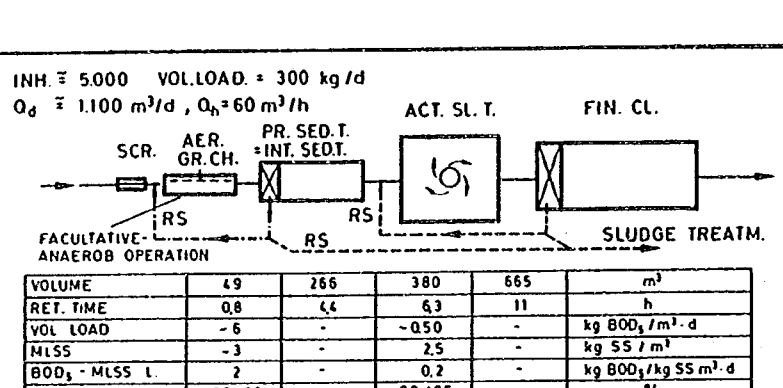


As further examples for the expansion of existing treatment plants the two following short descriptions are included in this paper: the expansion of the Wittlage treatment plant from 11000 inh. to 30000 inh/inh. equiv. and the expansion of the Rossbach treatment plant from 6500 inh. to 30000 inh./inh. equiv. The Rossbach treatment plant presents an interesting solution due to the strong seasonal fluctuation in the load between 2000 inh. and 30000 inh./inh. equiv..

Finaly the newest experimental results are presented for the equi-

valent use of aereated grit chambers as a preliminary adsorption stage or adsorption stage. In particular during these experiments, which are being carried out in a 1 : 1 scale at the Gruiten (BRW) treatment plant, one must take care to guarantee the problem-free operation of the grit chamber and the improvement in the clarification efficiency. These experiments, which have been funded by the Minister foe Research and Technology (PTWT - Karlsruhe), have to date produced the following results:

The dimensioning and load of the single stage activated sludge plant is given in fig. 15. In comparison to the previous operation, only the sludge circulation drawn in red has been added.



VOLUME	49	266	380	665	m³
RET. TIME	0,8	4,4	6,3	11	h
VOL LOAD	~6	-	~0,50	-	kg BOD_5/m³·d
MLSS	~3	-	2,5	-	kg SS/m³
BOD_5 - MLSS	2	-	0,2	-	kg BOD_5/kg SS m³·d
η_B / η_{CSB}	35/38	-	98/85	-	%
BOD_5	255	165		3-4	mg/l
COD_{CR}	544	336		50	mg/l

Institut SWW RWTH AC	INVESTIGATION AT GRUITEN S.T.P. AER. GRIT CHAMBER=A-STAGE	15

POSITION	EXISTING PLANT ONE STAGE ACT SLUDGE PROC.		EXPERIMENTAL PERFORMANCE AER.GRIT CHAMBER+A-STAGE	
—	n	AVERAGE VALUES (NOT SETTLED)	n	AVERAGE VALUES (NOT SETTLED)
BOD_{5C} - EFFLUENT	6	15 mg/l	5	3-4 mg/l
COD_{CR} - EFFLUENT	6	80 mg/l	5	50 mg/l
SLUDGE INDEX ACT. SL. TANK	6	220 g/cm³	5	125 g/cm³
EXCESS SLUDGE IN [m³/d]	22	11 m³/d	29	14
WITH α_{SS} [%]	4	65 %	29	73 %

n = NUMBER OF SAMPLES

Institut SWW RWTH AC	COMPARISON OF PRESENT EXPERIMENT RESULTS AT S.T.P. GRUITEN	16

For the preliminary BOD_5-MLSS load = sludge load ratio [kg · MLSS · m^3 · d] =2 a facultative anaerobic biocoenosis dominates the grit chamber with a higher COD reduction (38%) than BOD_5 reduction. The efficiency is achieved with increased aeration of the grit chamber. As with all AB-processes here too no inconvenience through odours occurs.

A comparison of the effluent values, sludge index and the amount of sludge production is shown in fi.16.

The BOD_5-effluents of the plant have been reduced from 15 mg/l to 3-4 mg/l and the COD effluent from an

average of 80 mg/l to 50 mg/l.

The sludge index of the old single stage plant was very light at 220 cm^3/g (average). During experimental operation it improved to 125 cm^3/g (average). The sludge production itself increased by $(\frac{14}{11} - 1) \cdot 100 = 27$ %, however the organic content increased by $(\frac{14}{11} \cdot \frac{0,73}{0,65} - 1) \cdot 100$) 43 %. A similar increase is to be expected in the specific biogas production.

It must be noted that apparantly aerated grit chambers can take over a double function. The simultaneous operation of the grit chamber as an adsorption stage allows the utilization of the advantages of the AB-process with the lowest possible extra volume requirement.

The retention time during dry weather is normally t = 15 min. For t = 30 min double the volume would be necessary. Such a change in the process technology could either lead to a significant or the total volume requirement could be greatly reduced. The latter would undoubtedly lead to a reduction in the specific construction costs (DM/inh.)

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APPENDIX

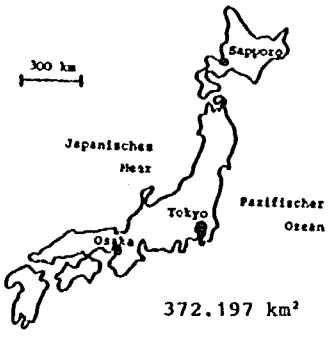
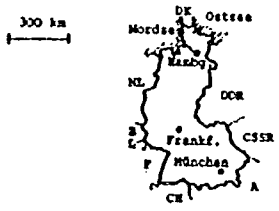
Comparison of Rudiments
K. Hibbeln 715

COMPARISON OF RUDIMENTS

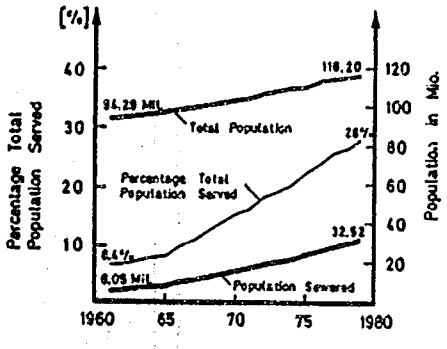
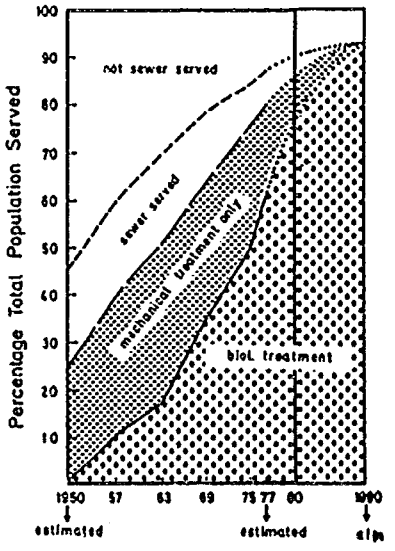
by K. Hibbeln

Inst. für Siedlungswasserwirtschaft

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	JAPAN	FR GERMANY
Geology, Geography, Climate		
Latitude	45 - 24 degrees N latitude, the four main island between 45 and 31 degrees N. Tokyo lies at about the same level as Cairo, however has a cold climate in winter due to cold air masses from siberia. 	55 - 47 degrees N latitude. Climate determined by atlantic ocean, continental influence clearly noticable east of Berlin. The whole of FRG lies farther north than Japan 
Area	372.197 km²	248.533 km²
inhabitable area	Only 15 - 20 % total japanese area is flat. The rest consists of steep volcanic mountains that are difficult to inhabit.	Except for the Alps and some parts of secondary chain of mountains generally inhabitable. In Germany the secondary chain of mountains are not densely inhabited, there exist however, cultivation techniques (e.g. milk), in comparison to Japan.
average yearly temperature	in Tokyo 14,7° C	in Frankfurt 10,2° C
lowest monthly temperature	in Tokyo 3,7° C (January)	in Frankfurt 0,8° C (January)
highest monthly temperature	in Tokyo 26,4° C (August)	in Frankfurt 19,4° C (July)
average yearly rainfall	about 1600 mm, from which 500 mm falls within a few days during taifun season (water economical not useable)	803 mm (average fro 1851 - 1930)
Snowfall	Seldom on the densely populated oceanic side between Tokyo and Osaka. However only 250 km away on the asiatic mainland side, often up to 1m even at sea-level. Several meters in the north at higher altitudes.	primarily dependent on heigth above sea-level

	JAPAN	FR GERMANY	
average yearly evaporation	about 500 mm	438 mm 318 mm through plants, only 10 mm through evaporation of surface waters.	
largest river	Shinanogawa	Rhein	Donau
length	376 km	865 km in FRG 1320 km spring-mouth	647 km in FRG 2850 km spring-mouth
		at frontier-crossing:	at frontier-crossing:
mean flow	472 m³/s (1951-1978)	2220 m³/s (1931-1974)	629 m³/s (1901-1976)
minimum flow	84 m³/s	590 m³/s (1947)	165 m³/s (1909)
maximum flood	5869 m³/s	12200 m³/s (1925)	447 m³/s (1845)
Population			
Inhabitants	about 116 mio	about 61 mio	
population increase	1,1 %	<0,1 %	
Inhabitants per km² (average)	311 (It must however be considered that only a small area is inhabitable)	245	
actual situation	66 mio japanese live in a propulation density of 8069 people/km² Tokyo (city) 8,3 mio inhabitants Population density 14.200/km² Osaka 2,7 mio inhabitants population density 14.580/km²	5,4 mio inhabitants of the Ruhr area live in a population density of 1220 people/km² Munich (most densely populated german city) 1,3 mio inh. Population density 4189/km²	
Behaviour of inhabitants with regard to water consumption			
Private water consumption	1975 175 l/(inh.·d) 1985 236 l/(inh.·d) (projected)	130 - 140 l/(inh.·d)	
Bathing habits	The tourist to Japan, who takes his information only from the travel guide, may think that this important difference for water supply and waste water treatment lies in the famous daily japanese bath. This is however not the case, since the japanese wash themselves with relatively little water before taking a bath which only serves as relaxing function. This bath measuring about 60 · 50 · 50 cm = 150 l for private families is therefore scarcely dirty and is renewed all few days only. A separate circulating heating system exists. The main difference lies in both the following differences.	The average value for bath and shower consumption is 20 - 40 l/(inh.·d) whereby 40 - 80 l/shower and 200 - 250 l/bath.	

	JAPAN	FR GERMANY
laundry	not heated, not automated tub system. Water is twirled with the help of electric motors. Sometimes reheated bath water is also used for washing. The method requires a relatively high water level. Cleaning takes place primarily through water movement. The real difference in consumption lies in the following rinsing process: In comparison to the multiple batch method prevalent in Germany, the japanese housewife rinses in a continnous water flow. Furthermore, the system definitely does not allow 4 kg laundry per washing process.	Mostly fully automatic drum washing machines with little water consumption in the normal washing process. The combination of laundry rubbing and water movement achieves a better cleaning effect as compared to water movement only. This is followed 4 - 5 times by a separate rinsing process 20 - 40 l/(inh.·d) 100 - 180 l/4 kg laundry
dish washing	Here is another main reason for the high japanese water consumption. The japanese housewife generally washes in flowing water. Since japanese food contains very little fat, slighthy warm water is enough for washing. Automatic dish washers are unknown in private families.	Often only one washing process according to the batch method. In some cases rinsing with flowing water. In the meanwhile fully automatic dish washers are widespread. The relatively high fat content of german food requires the use of fat dissolvers and hot water. Consumption for manual washing 4 - 6 l/(inh.·d); for dish washers about twice as much.
actual specific waste water yield (including groundwater and small scale industry)	very much higher than in Germany about 300 - 400 l/(inh. equiv.·d)	about 200 l/(inh. equiv.·d) in NRW (1976) 210 l/(inh.equiv.·d) in Bavaria (1978) 188 l/(inh.equiv.·d)
normal BOD_5 influent concentration for communal waste water	100 - 150 mg/l	200 - 400 mg/l
normal P-concentration for communal waste water	about 4 - 5 mg/l	about 15 mg/l
Population connected to sewer and treatment plants	 From the 28% of the total population connected to the sewerage system, 69% live in the 10 largest cities with over 1 mio. inhabitants	
	biol. treated (1982): appr.40 mio. inh.	biol. treated (1982): appr. 42 mio.inh.

	JAPAN	FR GERMANY
existing communal waste water treatment plants	about 450 (1982) normally very large. A comparison only according to numbers is misleading. Whereas Bavaria with 2650 plants has only 20 with a size of over 100.000 inh. equiv. Japan has about 50% of the plants in this scale. Furthermore 1186 night soil treatment plants	The exact figure for the whole of the Federal Republic is not possible. An assesment is possible from the figures for NRW and Bavaria: NRW: 1976 in operation, construction or planing 1120 plants Bavaria: 1978 in operation 2650 plants
Summary of effluent standards	Different effluent standards for communities and industries. Furthermore differing standards exist for discharge into inland waters and coastal waters and for industries extra standards for discharge into the public sewer system. Measurement of COD with potassium permanganate as oxidising agent (COD_{Mn}) Measurement of BOD_5 without hindering the Nitrogen oxidation (BOD_5-T) The conditions imposed on a national level are supplemented by ordinances from the local communal administrations (prefectures).	"Federal Water Act" with minimum demands for the average of the last 5 inspections for direct discharger. Different minimum demands for communities and the various industuries. For communities, a graduated average concentration is permissible according to the amount of discharged loading. The following are registered: settelable matter, BOD_5-C and COD_{cr}. Furthermore the "Waste Water Charges Act" is valid nationaly, which aims at better waste water clarification through economic incentives and leads to a limitation of the highest individual concentration. The waste water charge is to be paid for settleable matter, COD_{cr}, Mercury, Cadmium and the fish poisons. In addition to the laws valid for the whole country, each state has additional rules stipulated in the water laws of the states. Europ. Comm. and international guidelines are to be taken into consideration in the national laws.

National effluent standards for communities by discharge into rivers (24 h mixed sample)

Treatment method	pH	BOD_5-T mg/l	SS mg/l	number of coli per ml
activated sludge proc. low load. trickl. filter	5,8 - 8,6	20	70	≤3000
high load. trockl. filter	5,8 - 8,6	60	120	≤3000
mech. treatm. only	5,8 - 8,6	120	150	≤3000

For direct discharge into coastal water are other values valid

Minimum requirements for communal discharger

Magnitude	settleable matter ml/l	COD_{Cr} [1] mg/l	BOD_5-C [1] mg/l
for <60 kg BOD_5/d			
spot sample	0,3	-	-
2 h mixed sample	-	180	45
24 h mixed sample	-	120	30
for 60 - 600 kg BOD_5/d			
spot sample	0,3	-	-
2 h mixed sample	-	160	35
24 h mixed sample	-	110	25
for >600 kg BOD_5/d			
spot sample	0,3	-	-
2 h mixed sample	-	140	30
24 h mixed sample	-	100	20

1) from settled sample

	JAPAN	FR GERMANY
		Examples for industrial discharge (a total of about 50 different administrative regulations)

National effluents standards for industry

Parameter	allowed values
pH	5,8 - 8,6 Riverdischarge 5,0 - 9,0 into coastal waters & seas
BOD_5 (river discharge) COD_{Mn} (coastal waters- & sea discharge)	160 mg/l (daily peak) 120 mg/l (day average)
SS	200 mg/l (daily peak) 150 mg/l (day average)
number of coli-groups	⩽3000 (day average)
and other single matter limits	

For industrial influents into the public sewer system, the communities can demand a preclarification upto 600 mg BOD_5.

minimum requirements for fruits and vegetables processing factories

	settleable matter ml/l	COD_{Cr} [1] mg/l	BOD_{5-C} [1] mg/l
spot sample	0,3	-	-
2 h mixed sample	-	250	60
24 h mixed sample	-	200	45

minimum requirements for paint matters producing factories

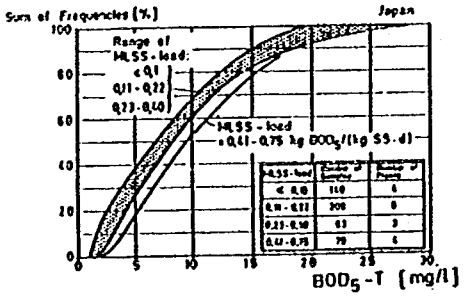
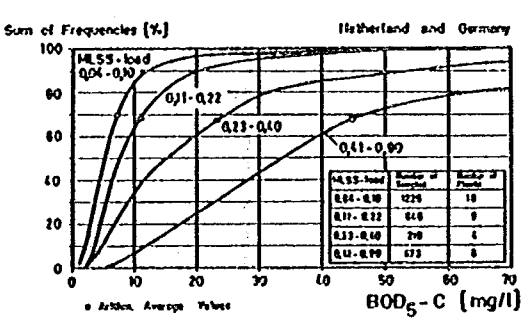
	settleable matter ml/l	COD_{Cr} [1] mg/l	BOD_{5-C} [1] mg/l
spot sample	0,3	-	-
2 h mixed sample	-	155	30
24 h mixed sample	-	115	20

[1] from settled sample

The industrial discharges into the public sewer networks is regulated by local regulations.

normal planing data for activated sludge plants

detention time primary sed.tank	1,5 h for sep. sewer syst. 3 h for comb. sewer syst. } average of peak day	0,5 - 1,5 h } for daily peak load
surface load primary sed.tank	25 m/d 50 m/d } average of peak day	generally 1,3 - 4,0 m/h } for daily peak load
detention time aeration tank	for MLSS-BOD_5load = 0,3 6 - 8 h recommended for MLSS ~ 1 - 2 g/l	for MLSS-BOD_5load = 0,3 about 5 h in practice for MLSS ~ 3 - 3,5 g/l
detention time final clarifier	2,5 h	generally 2,5 - 5 h. This is not a dimensioning value. It results from the necessary surface and basin depth. The basin depth is principaly determined by the necessary storage of activated sludge in the final clarifier during rainy weather.
surface load final clarifier	20 - 30 m/peak day	variable, dependent on the comparitive sludge volume of the activated sludge, generally, 0,4 - 1,6 m/h for daily peak hours.

	JAPAN	FR GERMANY
The most important methods of analysis BOD_5	generally as BOD_5-T i.e., without N-inhibition, T = Total	generally as BOD_5-C now statutory control with N-inhibition. (with 0,5 mg/l Allythiourea) C = carbon oxidation.
	Experience with effluent concentrations in both countries differs widely. The primary reasons are the different influent concentrations and the different methods of analysis.	
		
COD	In case without indices, always COD_{Mn} oxidation with potassiumpermangenate for 30 min. at 100° C (As a value not to be compared with $KMnO_4$-consumption!)	At present always as COD_{cr} oxidation with the help of dichromate with mercury shield in order to prevent chloride oxidation.
others	For the Nitrogen, Phoshorous, TOC - and other analyses, both countries use the internationaly accepted methods and apparatus.	
Investment for treatment plants	during the current 5-year-plan 3.000,-- DM/Inhabitant	1980/81 for 100 000 inh. + inh. equiv. plants about 200 - 250,-- DM/inhabitant + inh. equiv. on an average per "actual" inhabitant 400 - 500,-- DM/inhabitant
National investments for canals and clarification plants	1971-1975 26,1 billion DM 1976-1980 68,4 billion DM 1981-1985 118,0 billion DM (planned) (The amount forseen for 1982 has already been reduced to 91%)	in the public sector 1971-1981 for clarification plants 14,0 billion DM for sewer 25,0 billion DM 39,0 billion DM whereby during 1975-1981 27,0 billion DM Average investments in the private industrial sector during the past years about 800 million DM/annum

of general interest

	JAPAN	FR GERMANY
Life expectancy		
women	78,9	74,8
men	73,5	68,3
pension age		
for men	60 years	63-65 years
for women	55 years	58-60 years
education (high school)	> 90 %	< 20 %
homicides per 100 000 inhabitants	1,9	4,7
robberies and similar offenses per 100 000 inhabitants	1,9	32,9
Budhists	80 %	
Shintoists	80 %	
Protestants	380.000 } about 1 %	49,0 %
Catholics	740.000 }	44,6 %
private savings as a percentage of the income (1978)	20,4 %	13,9 %

Remark: This value is typical for a misleading comparison. Whereas the Federal Republic of Germany generaly has adequate old-age support, this is still in the process of being developed in Japan. The pensions for government employees are also rather meager. Furthermore, japanese stop working 5 years earlier and on a statistical average live five years longer. Savings are therefore very essential. Basic social security exists however through the family, i.e., for the older generation through the children.

export surplus in 1981 (US dollars)	+ 8,6 billion	+ 12,2 billion